

Geotechnical Investigation

Proposed Residential Development

4795 Bank Street & 3301 Blais Road
Ottawa, Ontario

Prepared for Caivan Communities

Report PG7301-1 Revision 3 dated July 9, 2026

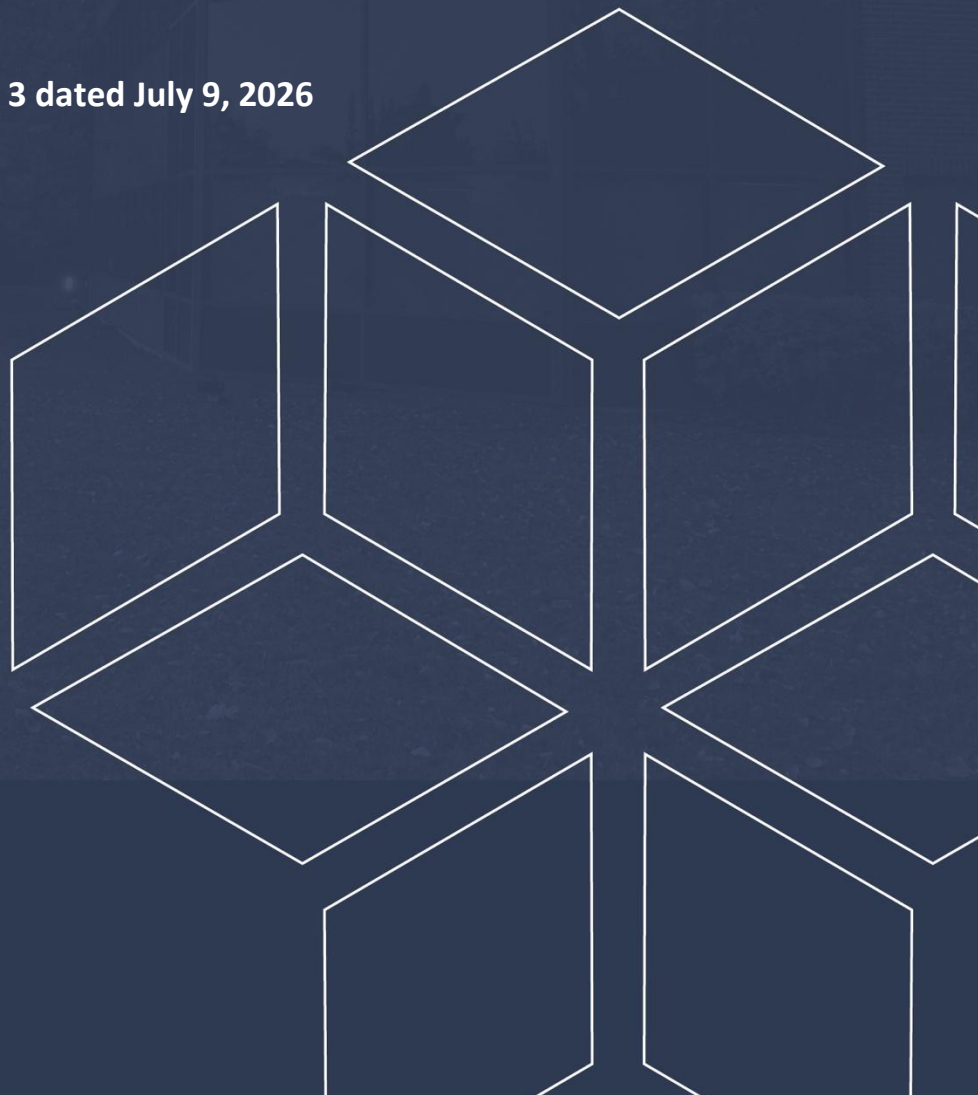


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1.0 Introduction

Paterson Group (Paterson) was commissioned by Caivan Communities to conduct a geotechnical investigation for the proposed residential development to be located at 4795 Bank Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 for the general site location). **The geotechnical investigation also included 3301 Blais Road, however, it is understood that only the development of the 4795 Bank Street property is being proposed at the time of report writing.**

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.
- ☐ Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

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

Investigating for the presence or potential presence of contamination on the subject property was not part of the scope of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the late concept plans, it is expected that the proposed development at 4795 Bank Street will consist of a series of single and townhouse-style residential dwellings, as well as stacked townhomes with basements or of slab-on-grade construction. Access lanes, at-grade parking areas, associated driveways, local roadways and landscaped areas are also anticipated as part of the development. No development is currently proposed at 3301 Blais Road.

It is also expected that the proposed development is to be municipally serviced. A stormwater management pond is proposed within the northeast limits of the 4795 Bank Street property.

It is anticipated that the existing structures located at the site will be demolished to accommodate the construction of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A supplemental geotechnical investigation was carried out to support the preliminary design of the proposed stormwater management pond to be located at the subject site. The supplemental investigation was completed on March 2 and 3, 2026 and consisted of advancing a total of 6 boreholes to a maximum depth of 10.3 m below the existing ground surface.

The field program for the original geotechnical investigation was carried out between September 25 and October 4, 2024 and consisted of advancing a total of 20 boreholes and 50 probe holes to maximum depths of 6.1 m and 14.9 m below the existing ground surface, respectively. A previous geotechnical investigation was completed by Paterson at the subject site on November 10, 2023. At that time, a total of three (3) test pits were advanced to a maximum depth of 5.1 m below the existing grade within the northeast portion of the site.

The borehole and probe hole locations for the current geotechnical investigation were distributed in a manner to provide general coverage of the proposed development, taking into consideration existing site features and underground services. The approximate locations of the test holes are presented on Drawing PG7301-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance, auger drill rig operated by a two-person crew. The probe holes were advanced using a track mounted, pneumatic drill rig operated by a two-person crew.

The test pits were excavated using a hydraulic shovel and backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedure consisted of augering/excavating to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In-Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained from the boreholes advanced within the footprint of the stormwater management pond using 47.6 mm inside diameter coring equipment.

All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification.

The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at boreholes BH 9-24, BH 15A-24 and BH 16-24 and via the probe hole program. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Monitoring wells were installed in boreholes BH 1-26 through BH 6-26, BH 2-24, BH 4-24, BH 6-24, BH 9-24, BH 10-24, BH 11-24, BH 12-24, BH 13-24, BH 15-24 and BH 17-24 during the geotechnical investigations to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Flexible piezometers were installed in the remaining boreholes.

All monitoring wells were equipped with a submersible datalogger (TD-Diver, VanEssen Instruments) to accurately monitor fluctuations in the groundwater levels for a specified period of time. The datalogger can measure the equivalent hydrostatic pressure of the water above the sensor diaphragm to calculate the total water depth. The dataloggers were programmed to continuously measure and record groundwater levels at a fixed rate of one (1) reading every 24 hours.

In addition to the continuous datalogger measurements, manual water level measurements were periodically taken throughout the groundwater monitoring program using an electronic water level meter. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The current test hole locations, and ground surface elevation at each test hole location, were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG7301-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Moisture content testing was completed on recovered soil samples from all boreholes. Additionally, 11 grain size distribution and hydrometer analysis and three (3) Atterberg Limits tests were completed on selected soil samples. The results are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

3.5 Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) testing was conducted at each monitoring well location installed within the footprint of the proposed stormwater management pond. The testing was completed to assist in confirming anticipated groundwater flow rates within the subsoils and within the proposed pond block. The test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden and bedrock aquifers.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy 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.

The 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. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin. The semi-log drawdown vs. time plots for rising and falling head at each borehole location are presented in Appendix 1.

The results of testing and hydrogeological recommendations are further discussed in Subsection 4.2.

4.0 Observations

4.1 Surface Conditions

4795 Bank Street

The majority of the subject site consists of agricultural lands with sparse areas of mature trees which generally follow the property boundary and/or subdivide the fields. Residential and agricultural buildings are located within the central and southeast limits of the property. The western portion of the property is bisected by the Findlay Creek.

The property is bordered by an existing residential development and associated stormwater managing system to the north and east respectively, by Blais Road to the south and by Bank Street to the west. The ground surface across the property slopes gently downward from west to east at approximate geodetic elevations of 94 m to 88 m.

3301 Blais Road

The property consists of agricultural lands with mature trees located along the perimeter of the property. The property is bordered by Findlay Creek and further by the aforementioned a storm water management facility to the north and west, by vacant lands to the east and by Blais Road to the south. The ground surface across the site property slopes downward from west to east at approximate geodetic elevations of 90 m to 88 m.

4.2 Subsurface Profile

Overburden

4795 Bank Street

Generally, the subsurface profile at the 4975 Bank Street property consists of topsoil underlain by a layer of silty sand overlying a deposit of sandy silt to clayey silt and/or glacial till. A fill layer, consisting of brown silty sand and/or silty clay to with gravel, cobbles, asphalt and crushed stone was encountered at boreholes BH 2-24 and BH 4-24 within the southern portion of the property and extended to approximate depths of 1.5 m and 0.7 m, respectively.

A layer of loose brown silty sand was encountered below the topsoil within the northern portion of the property and was noted to extend to a maximum depth of 1.0 m below the existing ground surface.

A compact brown sandy silt was observed below the topsoil and/or silty sand at the test pit locations, within the eastern limits of the property.

An approximate 1.0 to 5.3 m thick deposit of compact to loose brown sandy silt to clayey silt was encountered below the topsoil and/or silty sand at test holes BH 1-26, BH 2-26, BH 4-26, BH 5-26, BH 11-24, BH 12-24, TP 2-23 and TP 3-23, becoming grey in colour by approximate depths of 2.0 to 2.3 m.

A glacial till deposit was encountered underlying the above-noted soils at all test hole locations and was generally observed to consist of a loose to very dense, brown silty sand/or silty clay with gravel, cobbles and boulders. The glacial till deposit was observed to transition from brown to grey in colour at approximate depths ranging from 1.5 to 3.9 m below ground surface. Practical refusal to augering on inferred large boulders was encountered at several borehole locations at approximate depths ranging from 1.0 to 5.3 m. Large boulders were cored through at approximate depths 4.9 and 6.3 m at borehole BH 2-26.

3301 Blais Road

Generally, the subsurface profile at the 3301 Blais Street property consists of topsoil underlain by a deposit of silty sand to sandy silt underlain by glacial till. Fill material, intermixed with topsoil layers, was encountered at borehole BH 18-24 within the northeast corner of the property. The fill and topsoil layers were observed to extend to an approximate depth of 1.5 m below the existing ground surface.

A deposit of loose to dense brown silty sand to sandy silt was generally encountered below the topsoil across the property, extending to approximate depths of 0.7 to 4.1 m below the existing ground surface.

An approximate 3.6 m thick layer of stiff to firm grey clayey silt to silty clay was encountered below the topsoil and sandy silt at boreholes BH 15-24.

A glacial till deposit was encountered underlying the above-noted soils at all test hole locations and was generally observed to consist of a loose to very dense, silty sand with gravel, cobbles and boulders. The glacial till deposit was observed to transition from brown to grey in colour at approximate depths 1.5 to 3.1 m below ground surface at boreholes BH 17-24 and BH 18-24, respectively. Practical refusal to augering on inferred large boulders was encountered at depths ranging from 3.9 to 5.4 m at boreholes BH 17-24 to BH 20-24.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Grain Size Distribution and Hydrometer Testing

Eleven (11) grain size and hydrometer tests were completed to further classify selected soil samples. The results are summarized in Table 1 below and are presented in Appendix 1.

Table 1 – Summary of Grain Size Distribution Analysis					
Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-26	SS3	0.3	1.8	82.0	16.0
BH 1-26	SS5	0.0	0.4	87.6	12.0
BH 2-26	SS3	0.0	1.6	85.4	13.0
BH 2-26	SS8	13.3	35.1	47.6	4.0
BH 4-26	SS3	3.2	3.2	78.1	15.5
BH 4-26	SS5	1.1	1.3	85.0	12.5
BH 11-24	SS2	0.0	1.7	88.2	10.2
BH 12-24	SS3	0.0	1.1	90.9	8.0
BH 12-24	SS5	0.0	4.5	84.9	10.6
BH 12-24	SS6	0.0	3.6	86.6	9.8
BH 15-24	SS5	0.0	1.2	85.5	13.3

Atterberg Limit Tests

A total of three (3) samples were submitted for Atterberg Limits testing. The test results indicate that the samples are classified as Inorganic Silts of Slight Plasticity (ML). These classifications are in accordance with the Unified Soil Classification System. The results are summarized in Table 2 on the following page.

Table 2 – Summary of Atterberg Limits Results						
Test hole	Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 12-24	SS5	3.0 – 3.6	23	20	3	ML
BH 12-24	SS6	3.8 – 4.4	21	19	2	ML
BH 15-24	SS5	3.0 – 3.6	23	19	4	ML
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; ML: Inorganic Silts of Slight Plasticity; CL: Inorganic Clays of Low Plasticity						

Bedrock

The bedrock was cored during the supplemental investigation at borehole BH 1-26, BH 2-26, and BH 5-26 within the proposed pond block and was observed to consist of poor to fair quality limestone bedrock, becoming excellent in quality at depth. The bedrock was cored to depths ranging from 6.5 to 10.3 m below the existing ground surface at the aforementioned borehole locations.

Practical refusal on the inferred bedrock surface was encountered at approximate depths ranging from 3.0 to 14.9 m below the existing ground surface during the supplemental probe hole program. The inferred bedrock surface was generally observed to shallow from east to west across the subject site. Reference should be made to Drawings PG7301-2 – Bedrock Contour Plan which illustrates the interpolated elevation of the inferred bedrock surface across the subject site.

Based on available geological mapping, the bedrock in the area of the subject site generally consists of shale of the Carlsbad Formation, with the exception of the southeast corner of the site which is understood to consist of sandstone of the Nepean Formation.

Hydraulic Conductivity Values

Hydraulic conductivity (slug) testing values were recorded at each monitoring well installed at the proposed pond block during the supplemental investigation. The results are presented in Table 3 below.

Table 3 – Summary hydraulic conductivity values					
Test Hole ID	Ground Surface Elevation (m)	Depth to Bottom of Screen (m)	K (m/sec)	Test Type	Soil Type / Bedrock
BH 1-26	89.47	8.71	1.7×10^{-4}	Falling Head	Bedrock
			1.2×10^{-4}	Rising Head	
BH 2-26	88.86	10.11	1.3×10^{-5}	Falling Head	Bedrock
			1.6×10^{-5}	Rising Head	
BH 3-26	89.19	6.10	9.0×10^{-6}	Falling Head	Inferred Glacial Till
			5.4×10^{-6}	Rising Head	
BH 4-26	89.18	4.95	6.8×10^{-7}	Falling Head	Clayey Silt
BH 5-26	89.25	6.50	5.7×10^{-3}	Falling Head	Bedrock
			2.3×10^{-3}	Rising Head	
BH 6-26	89.81	5.21	2.6×10^{-5}	Falling Head	Inferred Glacial Till
			9.3×10^{-6}	Rising Head	

Slug testing completed at the monitoring wells screened in the overburden (BH 3-26, BH 4-26, and BH 6-26) identified hydraulic conductivity values ranging from approximately 6.8×10^{-7} to 2.6×10^{-5} m/sec. These values are generally consistent with similar material Paterson has encountered on other sites and typical published values for silt and glacial till, which typically range from 1×10^{-5} to 1×10^{-9} m/sec and is dependent on the ratio of sand and/or clay to silt within the material.

Slug testing completed at the monitoring wells screened in the bedrock (BH 1-26, BH 2-26, and BH 5-26,) identified hydraulic conductivity values ranging from approximately 1.3×10^{-5} to 5.7×10^{-3} m/sec. These values are generally consistent with similar material Paterson has encountered on other sites and typical published values for limestone bedrock, which typically range from 1×10^{-5} to 1×10^{-10} m/sec and is dependent on the quality of the bedrock at a given location.

4.3 Groundwater

Groundwater levels were measured within the installed monitoring wells and piezometers on March 20 and May 21, 2026 and on October 9, 2024, at the time of the original geotechnical investigation. Groundwater infiltration into the open test pits was also observed and recorded. A long-term groundwater monitoring program for the overall site was initiated in the fall of 2024 and continued through the capture of the spring high water levels in 2026. A groundwater monitoring program was also initiated within the proposed pond block, beginning in March of 2026. The measured groundwater levels are summarized in Table 4 below and the results of the monitoring programs are included in Appendix 2.

Table 4 – Summary of Groundwater Levels				
Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-26	89.47	1.51	87.96	March 20, 2026
		1.80	87.67	May 21, 2026
BH 2-26	88.86	1.12	87.74	March 20, 2026
		1.40	87.46	May 21, 2026
BH 3-26	89.19	1.00	88.19	March 20, 2026
		1.39	87.80	May 21, 2026
BH 4-26	89.18	1.08	88.10	March 20, 2026
		1.52	87.66	May 21, 2026
BH 5-26	89.25	1.36	87.89	March 20, 2026
BH 5-26	89.25	1.65	87.60	May 21, 2026

Table 4 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 6-26	89.81	1.96	87.85	March 20, 2026
		1.99	87.82	May 21, 2026
BH 1-24	93.47	Dry	-	October 9, 2024
BH 2-24*	93.13	2.90	90.23	October 9, 2024
		3.50	89.63	October 22, 2025
		2.23	90.90	May 21, 2026
BH 3-24	91.48	Dry	-	October 9, 2024
BH 4-24*	91.80	Dry	-	October 9, 2024
		Dry	-	October 22, 2025
		2.68	89.12	May 21, 2026
BH 5-24	91.56	Dry	-	October 9, 2024
BH 6-24*	92.27	Dry	-	October 9, 2024
BH 7-24	91.61	Dry	-	October 9, 2024
BH 8-24	93.69	Dry	-	October 9, 2024
BH 9-24*	92.90	3.20	89.70	October 9, 2024
		2.31	90.59	May 21, 2026
BH 10-24*	91.91	5.76	86.15	October 9, 2024
BH 11-24*	94.30	Dry	-	October 9, 2024
		1.42	92.88	October 22, 2025
		1.15	93.15	May 21, 2026
BH 12-24*	89.53	2.05	87.48	October 9, 2024
		2.02	87.51	October 22, 2025
		1.77	87.76	May 21, 2026
BH 13-24*	89.31	1.93	87.38	October 9, 2024
		2.01	87.30	October 22, 2025
		1.42	87.89	May 21, 2026
BH 14-24	90.17	Dry	-	October 9, 2024
BH 15-24*	89.29	2.17	87.12	October 9, 2024
		2.11	87.18	October 22, 2025
		1.75	87.54	May 21, 2026
BH 16-24	88.88	2.05	86.83	October 9, 2024

Table 4 – Summary of Groundwater Levels				
Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 17-24*	89.28	2.07	87.21	October 9, 2024
		2.31	86.97	October 22, 2025
		1.46	87.82	May 21, 2026
BH 18-24	88.30	1.31	86.99	October 9, 2024
TP 1-23	88.83	3.50	85.33	October 4, 2023
TP 2-23	89.05	1.70	87.35	
TP 3-23	89.07	1.60	87.47	
Note: The ground surface elevation at each test hole location was surveyed using a GPS referenced to a geodetic datum. *- Denotes Monitoring Well				

Average groundwater levels were found to range from approximately **2.0 to 3.0 m** below the ground surface at the time of the geotechnical investigation. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential development. It is recommended that the proposed residential buildings be founded on conventional spread footings bearing on undisturbed compact sandy silt to silty sand, stiff clayey silt, compact to very dense glacial till or clean surface sounded bedrock.

Depending on the proposed site grading, it is expected that some bedrock removal may be required for service installation. Further, it is anticipated that cobbles and large boulders will be encountered frequently throughout servicing trench and building excavation. All contractors should be prepared for bedrock and oversized boulder removal throughout the site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic or deleterious materials, should be stripped from under the proposed building and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

Bedrock/Boulder Removal

Bedrock and/or boulder removal may be required at the subject site and can be accomplished by hoe ramming where the bedrock is weathered, and/or where only small quantities need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site activities.

The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations. The blasting operations must be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

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

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. This material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the proposed building should be compacted to a minimum 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where this fill material is open-graded, a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. Where excavated bedrock is used as engineered fill under a building footprint, it should be capped with a minimum 300 mm of OPSS Granular A or Granular B Type II prior to the construction of footings and floor slabs. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. This can be assessed at the time of construction.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, compact sandy silt to silty sand and/or compact clayey silt can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

Footings placed on an undisturbed, compact to very dense glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

If the silty sand, sandy silt, clayey silt or glacial till bearing medium is encountered in a loose state of compactness at the time of construction, the bearing surface should be proof-rolled using a suitable compaction material making several passes. The proof-rolling should be completed under dry conditions and above freezing temperatures and approved by the Paterson at the time of construction.

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in-situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings supported directly on clean, surface-sounded bedrock, or on lean concrete which is placed directly over clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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

Footings supported directly on clean, surface sounded bedrock and design for the bearing resistance values provided above will be subject to negligible post-construction total and differential settlements.

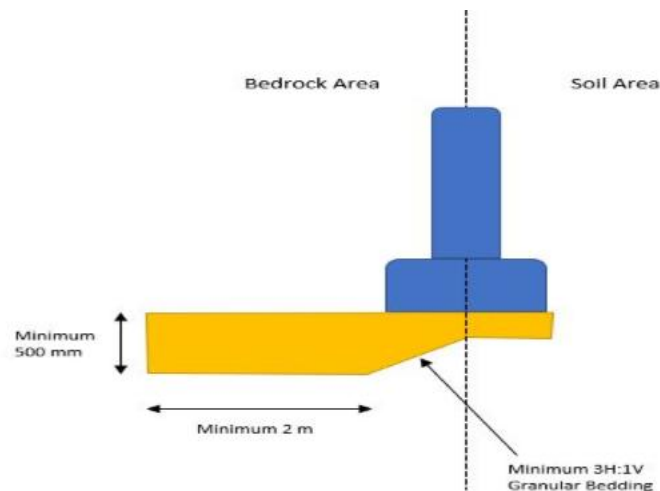
Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in-situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A soil bearing medium or a heavily fractured, weathered bedrock will require a lateral support zone of 1H:1V (or flatter).

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended at the soil/bedrock and bedrock/soil transitions that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material, see below. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.



Bedrock/Soil Transition Treatment

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_c**. If a higher seismic site class is required for the proposed buildings which are founded on, or within 3 m of, the bedrock surface, a site-specific shear wave velocity test should be completed in accordance with the Ontario Building Code (OBC) 2024 to accurately determine the applicable seismic site classification for foundation design of the proposed building.

Based on Paterson's review of the in-situ soil characteristics, the soils underlying the subject site are not considered susceptible to liquefaction.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the native soil and/or approved fill is considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

For structures with slab-on-grade construction, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in a maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

For structures with basement slabs, it is recommended that the upper 300 mm of sub-floor fill consists of 19 mm clear crush stone.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Pavement Design

For preliminary design purposes, the following pavement structures, presented in Tables 4 and 5, are recommended for car parking areas and access lanes.

Table 4 – Recommended Pavement Structure – Driveways / Car only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course – Superpave 12.5 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
300	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either fill, in situ soils or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 5 – Recommended Asphalt Pavement Structure – Local roadways	
Thickness (mm)	Material Description
40	Wear Course – Superpave 12.5 Asphaltic Concrete
50	Binder Course – Superpave 19.0 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
450	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either fill, in situ soils or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for each proposed structure which has below-grade space. The system should consist of a 100 or 150 mm diameter, geotextile-wrapped, perforated and corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone, and placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer, or to the sump pit where sump pumps are proposed at the residential dwellings.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geo-composite board, such as Delta Drain 6000, connected to the perimeter foundation drainage system.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

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

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. For the proposed development, it is anticipated that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes in the overburdened soils, above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. A flatter slope is required for excavation below groundwater level, such as 3H:1V. The subsurface soil at this site is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the department of public works and services, infrastructure services branch of the City of Ottawa.

A minimum 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipe. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the site generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavation should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (GU – General Use cement) would be appropriate for this site. The chloride content and pH of the sample indicate that they are not a significant factor in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a low to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Based on the results of the investigation, a silty clay deposit was generally not encountered between the anticipated underside of footing and 3.5 m below the underside of footing. As such, tree planting setbacks are not required at the subject site.

6.9 Slope Stability Assessment

Summary of Assessment

A slope stability analysis was completed at the subject site to determine the applicable geotechnical Limit of Hazard Lands setback from the slopes of the Findlay Creek which runs through the western portion of the 4795 Bank Street property, as well as from the slopes located along the southern and eastern limits of the property, which abut the existing stormwater management pond. The analysis was completed for both the existing conditions, as well as for the proposed conditions.

Based on the current field investigation and available topographic information three (3) slope cross-sections were analyzed for slope stability using SLIDE, a computer program for two-dimensional slope stability analysis. The cross-section locations are presented on Drawing PG7301-1-Test Hole Location Plan in Appendix 2.

Slope Stability Analysis

The slope stability analysis was modelled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several methods

including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces favoring failure.

Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.171 g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

Three (3) cross-sections (Section A through C) were studied as the worst-case scenario. It should be noted that details of the slope height and slope angle under existing and proposed conditions at the cross-section locations are presented in Figures 2 through 5, based on the available topographic data, as well as available preliminary grading information.

The effective and total strength soil parameters used for the static analyses were chosen based on the subsoil information recovered during the site investigation. The effective strength soil parameters used for the analyses are presented in Table 6 below:

Table 6 - Total Strength Soil and Material Parameters – Static Analysis			
Soil Layer	Unit Weight (kN/m³)	Friction Angle (degrees)	Cohesion (kPa)
Topsoil	16	33	5
Engineered Fill	19	35	2
Brown Silty Clay	17	33	7
Silty Sand	19	35	1
Brown Clayey Silt	17	33	7
Grey Clayey Silt	16	33	10
Glacial Till	20	33	2
Bedrock	22	-	-
Concrete	22	-	-

The total strength soil parameters used for seismic analysis were also chosen based on the subsoil information recovered within the boreholes. The strength parameters used for seismic analysis presented in Table 7 below.

Table 7 - Total Strength Soil and Material Parameters – Seismic Analysis			
Soil Layer	Unit Weight (kN/m³)	Friction Angle (degrees)	Cohesion (kPa)
Topsoil	16	33	5
Engineered Fill	19	35	2
Brown Silty Clay	17	0	80
Silty Sand	19	35	1
Brown Clayey Silt	17	0	80
Grey Clayey Silt	16	0	60
Glacial Till	20	33	2
Bedrock	22	-	-
Concrete	22	-	-

Sections A & B – Existing Stormwater Management System

Stable Slope Allowance

The results of the static analysis at Section A under existing and proposed conditions are shown in Figures 2A and 2C, respectively, while the results of the static analysis at Section B under existing and proposed conditions are shown in Figures 3A and 3C. The factor of safety was found to be greater than 1.5 at slope Sections A and B under existing and proposed conditions. Further, the results of the seismic analyses at Sections A and B under existing and proposed conditions are shown in Figures 2B, 2D, 3B and 3D. The factor of safety was found to be greater than 1.1 at all slope sections. As such, the slope is considered to be stable under both existing and proposed conditions and a stable slope allowance setback from top of slope is not required.

Toe Erosion Allowance

The toe of slope at Sections A and B is noted to consist of the bottom of an engineered slope abutting the existing stormwater management pond. The engineered stormwater management pond is not considered to be an active watercourse. As such a toe erosion setback is not considered to be required at Sections A-A and B-B.

Erosion Access Allowance

As the slope is considered to be stable under seismic and static conditions, under existing and proposed conditions, and the SWMP located along the toe of slope is not considered to be an active watercourse, an erosion access allowance is not required.

It is recommended that the existing vegetation on the slope faces not be removed, as it plays a vital role in maintaining slope stability and mitigating erosion.

Limit of Hazard Lands

Based on these observations and the results of the slope stability analysis under existing and proposed conditions, a Limit of Hazard Lands setback from the observed top of the slope is not required along the slope at the southern and eastern limits of the 4795 Bank Street property which slope down towards the adjacent existing SWMP.

Slope Section C-C – Findlay Creek

Existing Conditions

Stable Slope Allowance

The results of the static analysis at Section C under existing conditions along the southern bank of Findlay Creek are shown in Figure 4A. The factor of safety was found to be less than 1.5 under existing conditions. Further, the results of the seismic analyses under existing conditions are shown in Figure 4B. The factor of safety was found to be less than 1.1.

As such, a **stable slope allowance of 2.5** was applied to the overall Limit of Hazard Lands setbacks at Sections C under existing conditions, which extends to the required factors of safety of 1.5 and 1.1 under static and seismic analysis conditions, respectively.

The results of the static slope stability analysis for Section C under existing conditions along the eastern bank of Findlay Creek are presented in Figure 5A. The factor of safety was found to be greater than 1.5 under existing conditions. The results of the seismic slope stability analysis under existing conditions are presented in Figure 5B, with the factor of safety found to be greater than 1.1.

Based on the results of the analyses, the slope along the eastern bank of Findlay Creek is considered stable under existing conditions. Therefore, a stable slope allowance setback from the top of slope is not required along the eastern bank of the creek.

Toe Erosion Allowance

The toe of slope at Section C directly abuts the Findlay Creek. Erosion of the toe of slope is considered to be a notable factor in assessing slope stability. The banks abutting the watercourse were observed to be affected by minor signs of erosion. Signs of active erosion were mostly noted in the lower portion of the slopes and consisted of occasional small patches of loss vegetation or exposed root systems along the face of slope.

Based on the above-note observations, and in accordance with the City of Ottawa's Slope Stability Guidelines for Development Applications (2004) and the Ministry of Natural Resource's Technical Guide – River and Stream Systems: Erosion Hazard Limit (2002), it is considered that a toe erosion allowance of 6.0 m is appropriate for the corridor walls confining the creek. The toe erosion allowance should be applied from the top of stable slope.

Erosion Access Allowance

Based on our review and current guidelines, a 6.0 m erosion access allowance is recommended from the top of the stable slopes to allow for future maintenance of the slope.

Proposed Conditions

Based on discussion with the design team, it is understood that creek improvements designed by others are proposed to address the potential for erosion. The creek improvements are understood to consist of reshaping the creek such that floodplain and meander belt are fully contained. Further, it is understood that the creek will be reshaped with slopes less than 2 m high. As such, **under proposed conditions the slopes of the Findlay Creek will be considered stable, such that a Limit of Hazard Lands is not required.** Paterson should complete a review of relevant drawings, in advance of detailed design.

Limit of Hazard Lands

Based on these observations and the results of the slope stability analysis under existing and proposed conditions, Paterson has established a Limit of Hazard Lands setback from the observed top of the slope. The Limit of Hazard Lands is determined by a combination of stable slope allowance, toe erosion allowance, and erosion access allowance.

As a result, **under existing conditions**, the required Limit of Hazard Lands designation extends approximately **14.5 m** from the top of slope along the south side of Findlay Creek and approximately **12 m** from the top of slope along the eastern side of the creek. This Limit of Hazard Land setback is shown on the attached Drawing PG7301-1 - Test Hole Location Plan.

As noted above, a Limit of Hazard Lands Setback is not considered to be required along Findlay Creek under proposed conditions, taking into consideration the creek improvements designed by others.

6.9 Stormwater Management Pond Design Recommendations

While details of the proposed pond were not available at the time of reporting, it is understood that the proposed stormwater management pond (SWMP) will be located within the northeast portion of the 4795 Bank Street property. A supplemental field investigation was completed and a groundwater monitoring program has been initiated to support the detailed design of the SWMP.

It is assumed that the pond will include inlet and outlet structures with concrete headwalls and that inlet structures will be connected to the stormwater drainage infrastructure.

It is expected that the side slopes for the proposed pond will tie into the proposed ground surface and will be provided with a geosynthetic clay liner, due to the subsurface soil conditions encountered within the pond block.

Summary of Supplemental Investigation – SWMP Block

As outlined in Section 3.1 above, Paterson completed a supplemental field investigation within the proposed pond block to provide design recommendations from a geotechnical and hydrogeological perspective. The supplemental investigation was completed on March 2 and 3, 2026 and consisted of advancing a total of 6 boreholes to a maximum depth of 10.3 m below the existing ground surface.

Monitoring wells were installed within the supplemental boreholes and had hydraulic conductivity testing completed. Additionally, the monitoring wells were outfitted with data loggers to allow for completion of a water level monitoring program. The test hole locations are shown on Drawing PG7301 – Test Hole Location Plan in Appendix 2.

Subsurface Soil Profile – SWMP Block

Generally, the subsurface soil profile encountered at the proposed pond consists of topsoil underlain by a deposit of sandy silt to clayey silt underlain by glacial till.

A compact to loose brown silty sand to sandy silt was encountered underlying the topsoil at boreholes BH 1-26 and BH 3-26 within the pond block and extended to approximate depths of 4.1 and 1.4, respectively. A deposit of loose to compact clayey silt was observed underlying the topsoil and/or sandy silt at boreholes BH 2-26, BH 4-26 and BH 6-26 and extended to depths ranging approximately between 2.8 and 5.3 m below ground surface. The glacial till was generally observed to consist of loose to very dense silty sand to sandy silt with gravel, cobbles and boulders, and variable amounts of clay. Large boulders were cored through at approximate depths 4.9 and 6.3 m at borehole BH 2-26.

Bedrock was cored at boreholes BH 1-26, BH 2-26, and BH 5-26 to approximate depths ranging from 6.5 and 10.3 m below the existing ground surface. The bedrock at these locates was observed to consist of a poor to excellent quality limestone bedrock, based on the RQD values.

Groundwater Level Monitoring – SWMP Block

In addition to manual water level measurements, a groundwater monitoring program was carried out to provide an overview of the variations of the monitoring well water levels based on seasonal fluctuations. The monitoring wells installed at boreholes at BH 1-26 through BH 6-26 were equipped with a submersible datalogger to accurately monitor fluctuations in the water levels. The datalogger can measure the equivalent hydrostatic pressure of the water above the sensor diaphragm to calculate the total water depth.

Groundwater level monitoring was initiated at the monitoring wells installed within the pond block in March 2026. While the program is ongoing, the available data is considered adequate to support preliminary design of the proposed pond. The data obtained from the 2026 spring freshet will be used to support design of the proposed SWMP.

Further, groundwater level monitoring across the subject site was initiated in October 2024 and is currently ongoing. The dataloggers were programmed to continuously measure and record groundwater levels at a fixed rate of one (1) reading every 24 hours. The monitoring data was also compared with Environment and Natural Resources Canada precipitation data from the Ottawa International Airport over the same time period. Monitoring Well Water Elevation Plots are presented in Appendix 2.

Hydraulic Conductivity Testing

The results of the hydraulic conductivity (slug) testing conducted at the monitoring wells installed within the proposed pond block are discussed in Section 4.2, and are presented in Table 3 above.

In summary, slug testing completed at the monitoring wells screened in the overburden identified hydraulic conductivity values ranging from approximately 6.8×10^{-7} to 2.6×10^{-5} m/sec and slug testing completed at the monitoring wells screened in the bedrock identified hydraulic conductivity values ranging from approximately 1.3×10^{-5} to 5.7×10^{-3} m/sec.

Design and Construction Recommendations

From a geotechnical perspective, the proposed block is suitable for construction of a SWMP. The following sections detail our preliminary design and construction recommendations for the proposed pond. **A supplemental review will be required once details of the pond are available and a formal review of the proposed pond will be completed under a separate memorandum.**

Groundwater Infiltration During Construction

Dependent on the design of the proposed pond, and based on the findings of our geotechnical investigations, the excavation may extend through layers of compact to loose silty sand to sandy silt, compact to loose clayey silt, loose to very dense glacial till and fair to excellent quality limestone bedrock. It is anticipated that excavation will advance below the current groundwater table to reach the bottom of the pond elevation.

The proposed SWMP will be located in an area where water infiltration will be required to be managed during the construction phase. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium and to undertake all proposed works in the dry.

Based on the results of the hydraulic conductivity testing, it is likely that an Environmental Activity and Sector Registry (EASR) will be required to manage groundwater inflows during construction. A minimum of two to four weeks should be allotted for the completion of the EASR registration and Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg.63/16. Paterson personnel should review the excavation at the time of construction to assess the soil type at the bottom of the excavation and the water infiltration observations, once exposed.

Long-Term Groundwater Control

It is expected that the base and portions of the pond sidewalls will be located below the groundwater table. As such, preliminary design considerations such as the use of use of subdrains installed around the perimeter of the pond, the installation of a geosynthetic liner over the pond bottom, subdrains beneath the liner and a ballast to resist uplift pressure should be taken in consideration during design of the pond, to ensure functionality of the system as a pond during periods when local groundwater levels may rise above the permanent pool elevation.

Perimeter and Liner Subdrains

Portions of the pond which extend below the groundwater table may be subject to uplift pressure. To reduce the potential uplift pressure acting against a pond liner, a series of subdrains may be placed around the perimeter of the pond. The subdrains are recommended to be placed at an elevation coinciding with the permanent pool elevation, to maintain a static groundwater table elevation at the pond location. The subdrain system should be connected to, and gravity drain towards, the pond outlet structures.

The subdrain system should be suitably sized to prevent groundwater migration into the pond footprint and can consist of lines of HDPE perforated pipes fitted with a geosock liner and surrounded on all sides by 300 mm of 19 mm clear crushed stone. The clear stone should be wrapped in a non-woven geotextile.

Subdrains beneath the pond liner, located within the engineered fill subbase, can be connected to the perimeter subdrain to allow for the dissipation of additional pressure and are intended to function in a passive manner. Drain sizing and spacing is to be determined at detailed design.

Geosynthetic Liner

Due to the permeable soils and the bedrock expected to be encountered along the sidewalls and at the base of the proposed pond, a geosynthetic liner is recommended across the pond footprint to act as a hydraulic barrier.

For preliminary design purposes, it is recommended that the geosynthetic liner be placed above an approximate 300 mm thickness of 19 mm clear crushed stone or reworked native soil which is placed directly over the approved pond subgrade or bedrock surface. The liner should be placed at an elevation which accounts for the overlying ballast thickness, if applicable. The liner should extend up the side slopes of the pond, to approximately 0.3 m above the permanent pond elevation. The geomembrane liner is recommended to be installed as per the manufacturer's recommendations and installation guidelines to provide a relatively watertight liner system.

Ballast

Dependent on the proposed pond design, a ballast may be required to resist uplift pressure acting against the pond liner. Where required, the ballast should consist of blast rock placed above the geosynthetic liner. To maintain storage volumes within the proposed pond, the top of ballast should match the proposed pond bottom elevation. The top of ballast should be surfaced in accordance with applicable City of Ottawa standards for accessibility and maintenance.

Excavation of Base and Pond Sidewalls

The long-term performance of the subject pond will depend on the stability of its excavation side slopes. The pond sidewalls, within overburden soils and poor quality rock, should be maintained at 3H:1V or flatter. In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors. A minimum 1 m horizontal ledge should remain between the overburden soils and sound bedrock surface.

Fill Placement

Fill used for grading, where required, can be undertaken by re-using site-generated silty sand to sandy silt, clayey silt, and/or glacial till. The site-generated material should be in a workable, relatively dry state (i.e., not saturated, wet, grey soils) and able to be placed, compacted and shaped using a suitably sized vibratory roller. All fill should be free of organics, debris, and cobbles or fractured rock greater than 200 mm in diameter.

The placement of the fill should be reviewed by Paterson at the time of placement. Compaction of the material may be achieved by making several passes using a suitably sized roller and is recommended to be undertaken during dry and above-freezing conditions. The fill should not be frozen at the time of placement.

Fill used to raise the grade surrounding the pond in areas where settlement of the ground is not a major concern (i.e. beyond areas of settlement sensitive structures

and surfaces, such as hardscaping), could be placed in suitably-sized lifts and compacted by the tracks of the earthworks equipment making several passes. The placement and preparation of the above-noted fill should be reviewed and approved at the time of construction by Paterson personnel.

Bearing Capacity for Concrete Structures

Structures for the proposed SWMP placed over an undisturbed, compact silty sand to sandy silt and/or compact clayey silt, can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

Structures for the proposed SWMP placed over an undisturbed, compact to very dense glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

Where loose or disturbed native soil is encountered at subgrade level, a proof-rolling program should be implemented, consisting of compacting the loose material with several passes of a vibratory drum roller under dry conditions and above freezing temperatures, and under the observation of Paterson.

Structures for the proposed SWMP placed over a clean surface-sounded limestone bedrock can be designed using a bearing resistance value at ultimate limit states (ULS) of **1000 kPa**.

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

Footings designed using the bearing resistance values at SLS provided above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. Footings placed on clean, surface sounded bedrock will be subjected to negligible settlements.

Field Inspections

All fieldwork, including service bedding/spring-line/cover placement, roadway subgrade fill placement and roadway subbase, base and asphalt layer placement, should be reviewed and approved by Paterson at the time of construction. Paterson should be notified upon the commencement of work and establish a schedule for periodic inspections to be completed by field personnel throughout construction.

It should be noted that if winter construction is anticipated, the contractor should take every precaution to prevent frost from migrating below settlement sensitive structures. Reference should be made to the aforementioned geotechnical report for further detail regarding winter construction recommendations.

7.0 Recommendations

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

- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Caivan Communities, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.



Mrunmayi Anvekar, M.Eng.



Kevin Pickard, P.Eng.

Report Distribution:

- ☐ Caivan Communities (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

ELEVATION: 89.47

FILE NO. : PG7301

HOLE NO. : BH 1-26



PAGE: 1 / 1

COORD. SYS.: MTM ZONE 9

EASTING: 376514.79

NORTHING: 5020324.14

ELEVATION: 88.86

PROJECT: Geotechnical Investigation

FILE NO. : PG7301

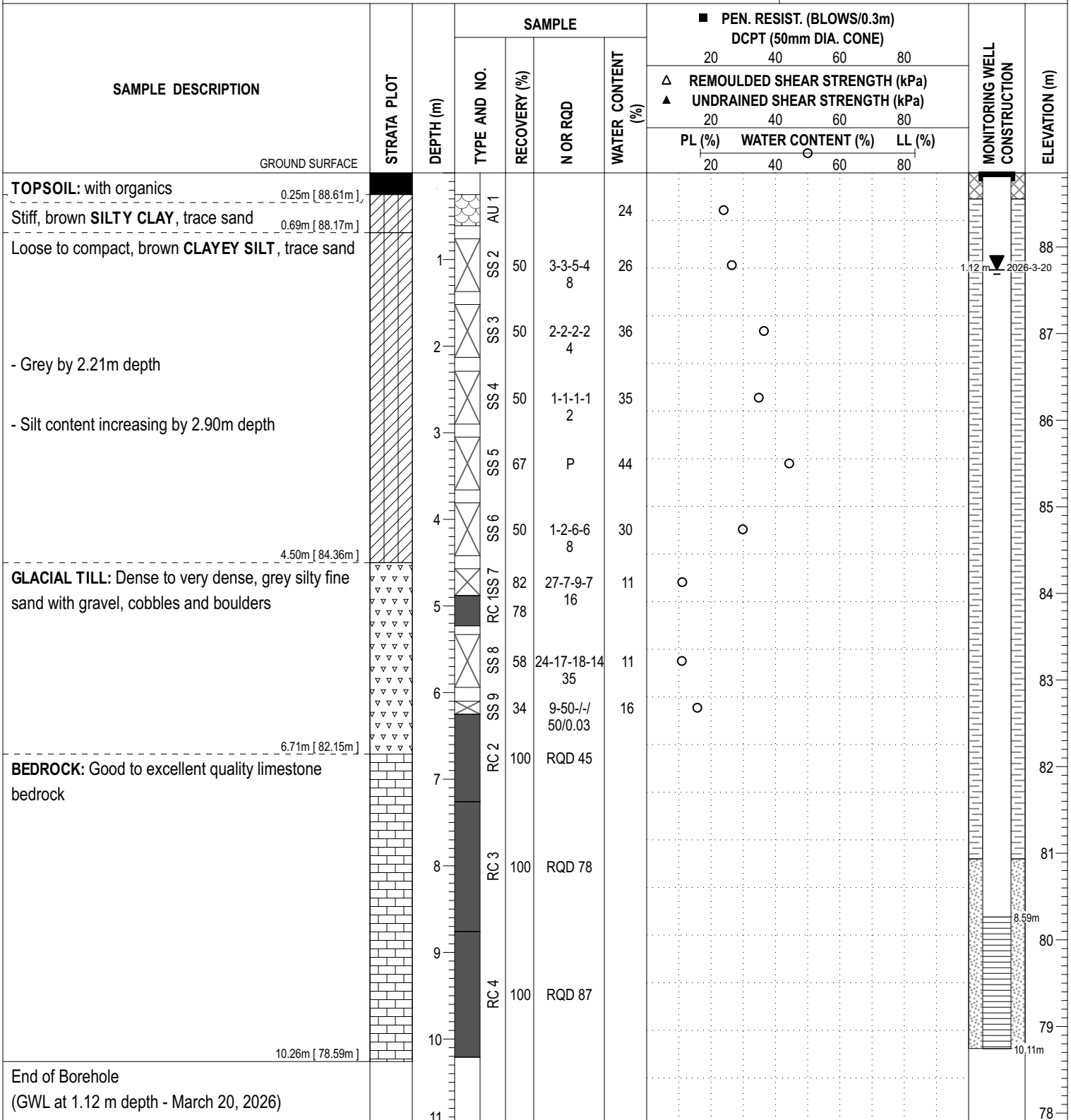
ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada)

Geoid: HT2-2010

DATE: March 2, 2026

HOLE NO. : BH 2-26



DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

COORD. SYS.: MTM ZONE 9	EASTING: 376510.67	NORTHING: 5020331.96	ELEVATION: 89.19
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PROJECT: Geotechnical Investigation	FILE NO. : PG7301
--	--------------------------

ADVANCED BY: Track Mounted Drill Rig	HOLE NO. : BH 3-26
---	---------------------------

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010	DATE: March 3, 2026	
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SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH (kPa)					
							▲ UNDRAINED SHEAR STRENGTH (kPa)					
							20	40	60	80		
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									

GROUND SURFACE

For soil profile refer to BH 2-26

6.10m [83.09m]

End of Borehole

(GWL at 1.00 m depth - March 20, 2026)

1

2

3

4

5

6

7

8

9

10

11

1.00 m

2026-3-20

4.58m

6.10m

89

88

87

86

85

84

83

82

81

80

79

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COORD. SYS.: MTM ZONE 9 **EASTING:** 376547.38 **NORTHING:** 5020340.33 **ELEVATION:** 89.18

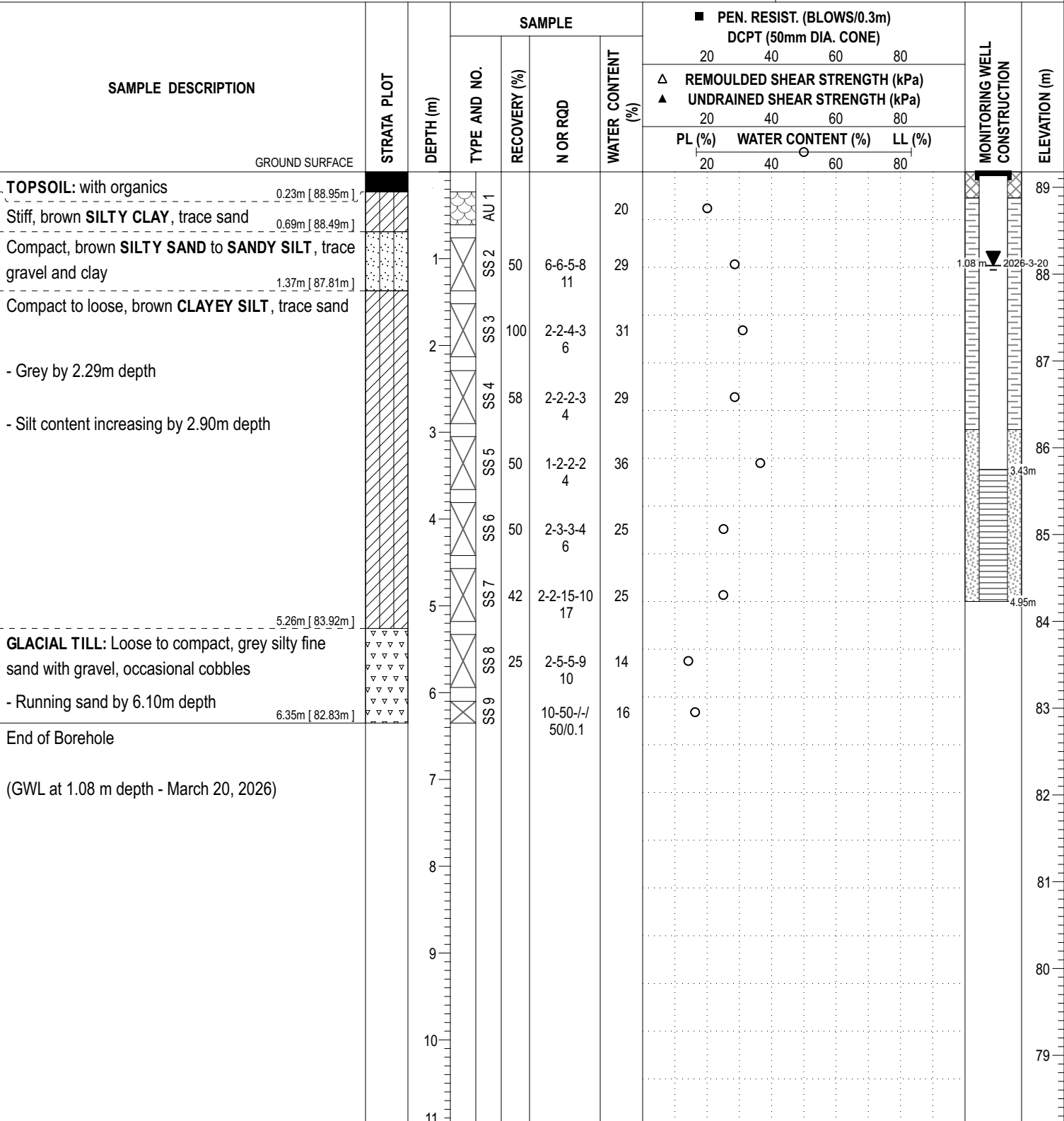
PROJECT: Geotechnical Investigation

FILE NO. : PG7301

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010

DATE: March 3, 2026

HOLE NO. : BH 4-26


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COORD. SYS.: MTM ZONE 9

EASTING: 376492.29

NORTHING: 5020369.66

ELEVATION: 89.25

PROJECT: Geotechnical Investigation

FILE NO. : PG7301

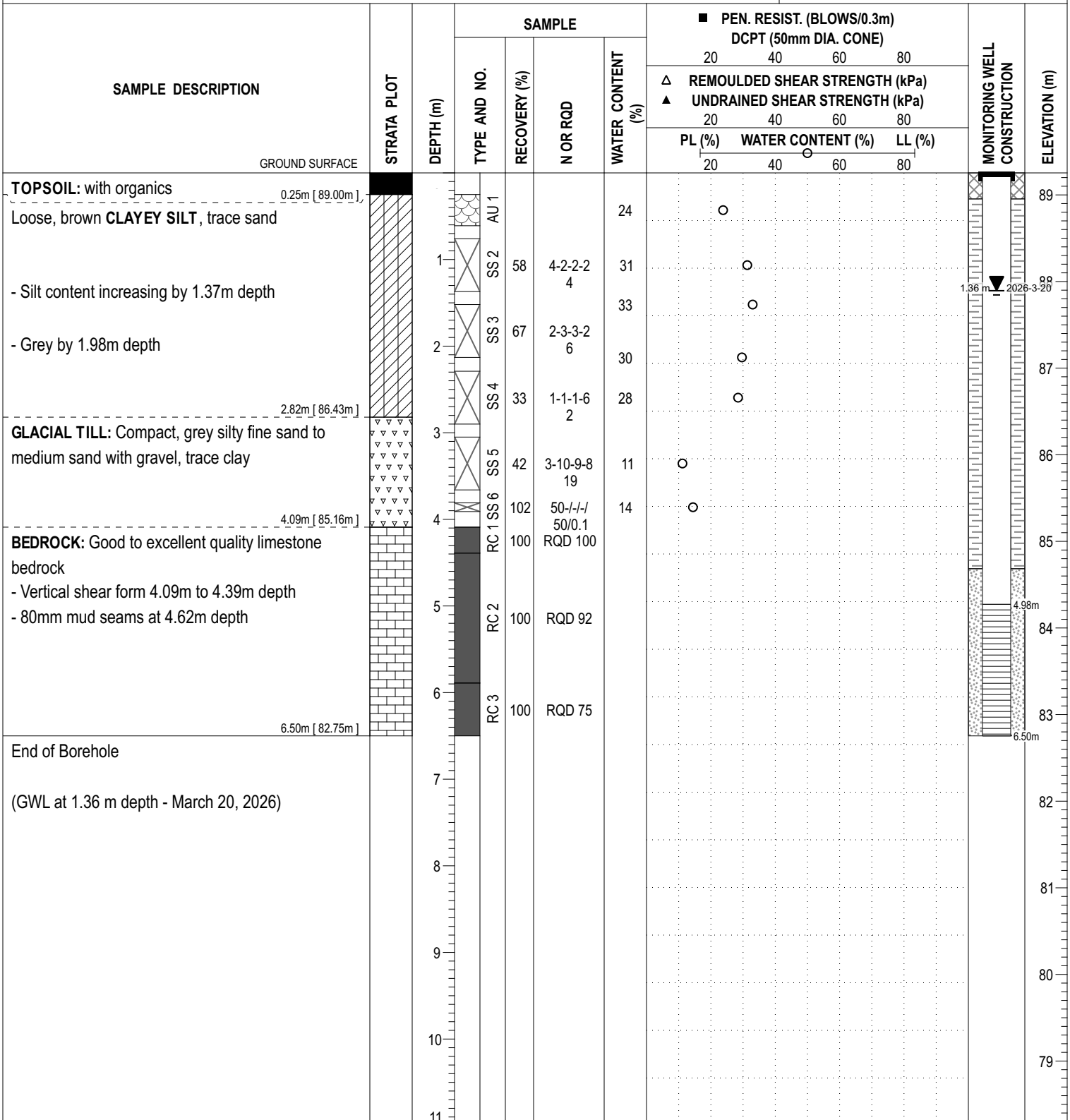
ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada)

Geoid: HT2-2010

DATE: March 3, 2026

HOLE NO. : BH 5-26



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ELEVATION: 89.81

FILE NO. : PG7301

DATE: March 3, 2026

HOLE NO.: BH 6-26

PAGE: 1 / 1

COORD. SYS.: MTM ZONE 9 EASTING: 376166.29 NORTHING: 5019808.33 ELEVATION: 93.47

PROJECT: Geotechnical Investigation FILE NO.: **PG7301**

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 25, 2024 HOLE NO.: **BH 1-24**

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80					
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20 40 60 80					
PL (%) WATER CONTENT (%) LL (%)				20 40 60 80								
GROUND SURFACE												
TOPSOIL and organics		0										
GLACIAL TILL: Loose brown silty sand with gravel, cobbles and boulders			AU 1								93	
- Very dense by 0.80 m depth		1	SS 2	74	6-23-50-/ 73/0.25	9.5						
End of Borehole											92	
Practical refusal to augering at 1.17 m depth		2										
(BH Dry - October 09, 2024)		3									91	
		4									90	
		5									89	
		6									88	
		7									87	
		8									86	
		9									85	
		10									84	

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ELEVATION: 93.13

HOLE NO. : BH 2-24

COORD. SYS.: MTM ZONE 9	EASTING: 376312.22	NORTHING: 5019885.37	ELEVATION: 91.48
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PROJECT: Geotechnical Investigation BORINGS BY: CME-55 Low Clearance Drill REMARKS:	FILE NO. : PG7301 HOLE NO. : BH 3-24
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SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)			PIEZOMETER CONSTRUCTION	ELEVATION (m)	
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60			80
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									

GROUND SURFACE											
TOPSOIL and organics 0.15m [91.33m]		0		AU 1							
GLACIAL TILL: Loose brown silty sand, trace clay, occasional gravel, cobbles and boulders - Very dense by 0.8 m depth 1.02m [90.46m]		1		SS 2	86	16-50-/-/ 50/0.3	10.19	○			91
End of Borehole											90
Practical refusal to augering at 1.02 m depth		2									
(BH Dry - October 09, 2024)											89
		3									88
		4									87
		5									86
		6									85
		7									84
		8									83
		9									82
		10									

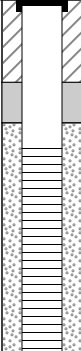
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COORD. SYS.: MTM ZONE 9 **EASTING:** 376195.40 **NORTHING:** 5019922.75 **ELEVATION:** 91.80

PROJECT: Geotechnical Investigation **FILE NO. :** PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: **DATE:** September 25, 2024 **HOLE NO. :** BH 4-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80					
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20 40 60 80					
PL (%) WATER CONTENT (%) LL (%)				20 40 60 80								
GROUND SURFACE												
FILL: Loose, brown silty sand, trace crushed stone		0	AU 1									
0.69m [91.11m]												
GLACIAL TILL: Dense, brown silty sand, with gravel, trace cobbles and boulders		1	SS 2	71	13-22-23-16 45	6.4	○				91	
- Grey by 1.5 m depth												
2.62m [89.18m]												
End of Borehole												

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COORD. SYS.: MTM ZONE 9	EASTING: 376185.77	NORTHING: 5020018.09	ELEVATION: 91.56
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PROJECT: Geotechnical Investigation BORINGS BY: CME-55 Low Clearance Drill REMARKS:	FILE NO. : PG7301 HOLE NO. : BH 5-24
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SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							PL (%)	WATER CONTENT (%)		LL (%)		
20	40	60	80									

GROUND SURFACE												
TOPSOIL and organics 0.18m [91.38m]		0		AU 1								
GLACIAL TILL: Loose, brown silty sand, trace gravel, some cobbles, boulders												
- Compact by 1.10 m depth		1		SS 2	75	7-12-18-21 30	8.59	○				
							5.51	○				
1.83m [89.73m]				SS 3	84	9-18-50-/ 68/0.18	4.64	○				
End of Borehole		2										
Practical refusal to augering at 1.83 m depth												91
(BH Dry - October 09, 2024)												90
					</							

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COORD. SYS.: MTM ZONE 9	EASTING: 376093.56	NORTHING: 502000.31	ELEVATION: 92.27
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PROJECT: Geotechnical Investigation	FILE NO. : PG7301
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BORINGS BY: CME-55 Low Clearance Drill	HOLE NO. : BH 6-24
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REMARKS:	DATE: September 25, 2024
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SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20	40	60	80		
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									
GROUND SURFACE												
TOPSOIL and organics		0	AU 1									92
Loose, brown SILTY SAND												
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders		1	SS 2	92	15-29-37-30 66	8.91	○					91
			SS 3	78	15-46-50-/ 96/0.2	8.82	○					
End of Borehole		2										90
Practical refusal to augering at 1.88 m depth												
(BH Dry - October 09, 2024)		3										89
		4										88
		5										87
		6										86
		7										85
		8										84
		9										83
		10										

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COORD. SYS.: MTM ZONE 9	EASTING: 376044.89	NORTHING: 5020116.72	ELEVATION: 91.61
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PROJECT: Geotechnical Investigation BORINGS BY: CME-55 Low Clearance Drill REMARKS:	FILE NO. : PG7301 HOLE NO. : BH 7-24
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SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)			PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80				
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)				
							▲ PEAK SHEAR STRENGTH, Cu (kPa)				
							PL (%)	WATER CONTENT (%)	LL (%)		
20 40 60 80	20 40 60 80	20 40 60 80									
GROUND SURFACE											
TOPSOIL and organics		0									
0.30m [91.31m]			AU 1							91	
Loose, brown SILTY SAND											
0.76m [90.85m]											
GLACIAL TILL: Compact, brown silty sand, occasional gravel, cobbles and boulders		1	SS 2	54	3-5-11-7 16	12.03	○				
- Grey by 1.5 m depth											
2.13m [89.48m]		2	SS 3	25	2-1-5-18 6	9.17	○			90	
End of Borehole											
Practical refusal to augering at 2.13 m depth										89	
(BH Dry - October 09, 2024)										88	
										87	
										86	
										85	
										84	
										83	
										82	

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COORD. SYS.: MTM ZONE 9 **EASTING:** 376132.74 **NORTHING:** 5020181.66 **ELEVATION:** 93.69

PROJECT: Geotechnical Investigation **FILE NO. :** PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: **DATE:** September 26, 2024 **HOLE NO. :** BH 8-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20	40	60	80		
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									
GROUND SURFACE												
TOPSOIL and organics		0										
Loose, brown SILTY SAND, trace gravel			AU 1									
												93
GLACIAL TILL: Very dense, brown silty clay, with gravel, occasional cobbles and boulders		1	SS 2	96	17-25-48-37 73	9.38	○					
- Grey by 1.5 m depth												
			SS 3	92	16-21-26-24 47	7.14	○					92
			SS 4	79	9-30-35-33 65	7.51	○					91
			SS 5	42	15-37-29-10 66	7.74	○					90
			SS 6	0	50-/-/-/ 50/0.03							90
End of Borehole		4										
Practical refusal to augering at 3.84 m depth												89
(BH Dry - October 09, 2024)		5										88
												87
												86
												85
												84

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COORD. SYS.: MTM ZONE 9

EASTING: 376290.59

NORTHING: 5020275.07

ELEVATION: 92.90

PROJECT: Geotechnical Investigation

FILE NO. : PG7301

BORINGS BY: CME-55 Low Clearance Drill

DATE: September 26, 2024

HOLE NO.: BH 9-24

REMARKS:

[illegible]

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PAGE: 1 / 1

COORD. SYS.: MTM ZONE 9

EASTING: 376347.34

NORTHING: 5020162.69

ELEVATION: 91.91

PROJECT: Geotechnical Investigation

FILE NO. : PG7301

BORINGS BY: CME-55 Low Clearance Drill

DATE: September 26, 2024

HOLE NO. : BH10-24

REMARKS:

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)			MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80				
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)				
							▲ PEAK SHEAR STRENGTH, Cu (kPa)				
							PL (%)	WATER CONTENT (%)	LL (%)		
GROUND SURFACE		20	40	60	80						
TOPSOIL and organics 0.23m [91.68m]		0	AU 1								
Firm, brown SILTY CLAY, trace sand 0.69m [91.22m]											
GLACIAL TILL: Very dense, brown silty clay, with gravel, cobbles and boulders		1	SS 2	79	10-17-28-44 45	9.09	○			91	
- Grey by 1.8 m depth		2	SS 3	100	18-23-24-28 47	9.87 7.12	○ ○			90	
		3	SS 4	75	4-4-9-15 13	9.94	○			89	
		4	SS 5	71	3-12-15-30 27	12.2	○			88	
		5	SS 6	100	15-24-35-48 59	6.99	○			87	
		6	SS 7	91	38-50-/- 50/0.13	6.8	○			86	
6.10m [85.81m]		6	SS 8	100	25-50-/- 50/0.08	5.73	○			86	
End of Borehole											
(GWL at 5.76 m depth - October 9, 2024)		7								85	
		8								84	
		9								83	
		10								82	

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PAGE: 1 / 1

COORD. SYS.: MTM ZONE 9 EASTING: 376217.33 NORTHING: 5020152.31 ELEVATION: 94.30

PROJECT: Geotechnical Investigation FILE NO.: **PG7301**

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 26, 2024 HOLE NO.: **BH11-24**

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20	40	60	80		
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									

GROUND SURFACE												
TOPSOIL and organics		0										94
0.25m [94.05m]												
Loose, brown SILTY SAND												
0.69m [93.61m]												
Very stiff, brown CLAYEY SILT		1										
1.68m [92.62m]												
GLACIAL TILL: Very dense, brown silty sand, trace clay, with gravel, occasional cobbles and boulders		2										
- Grey by 2.3 m depth												
2.62m [91.68m]												
End of Borehole		3										
Practical refusal to augering at 2.62 m depth		4										91
(BH Dry - October 09, 2024)		5										90
		6										89
		7										88
		8										87
		9										86
		10										85

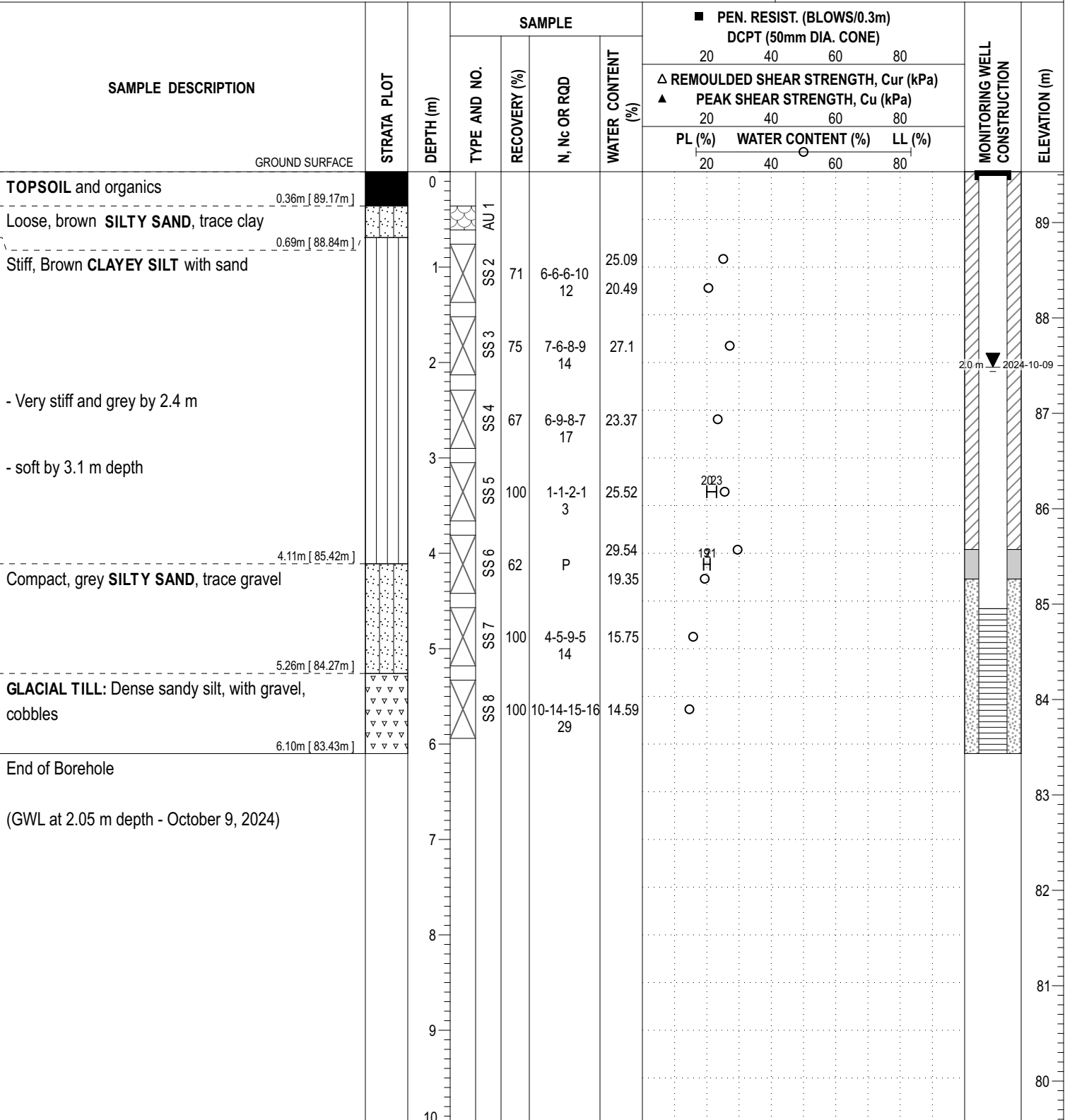
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COORD. SYS.: MTM ZONE 9 EASTING: 376432.01 NORTHING: 5020344.88 ELEVATION: 89.53

PROJECT: Geotechnical Investigation FILE NO. : **PG7301**

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 26, 2024 HOLE NO. : **BH12-24**



COORD. SYS.: MTM ZONE 9 EASTING: 376509.59 NORTHING: 5020188.33 ELEVATION: 89.31

PROJECT: Geotechnical Investigation FILE NO. : PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 27, 2024 HOLE NO. : BH13-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80					
							Δ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20 40 60 80					
PL (%)	WATER CONTENT (%)		LL (%)									
20 40 60 80												

GROUND SURFACE												
TOPSOIL and organics		0	AU 1									89
Loose, brown SILTY SAND, trace clay												
GLACIAL TILL: Compact, brown silty sand, trace clay with gravel, occasional cobbles		1	SS 2	79	7-6-5-7 11	11.11	○					88

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COORD. SYS.: MTM ZONE 9 **EASTING:** 376413.92 **NORTHING:** 5020249.09 **ELEVATION:** 90.17

PROJECT: Geotechnical Investigation **FILE NO. :** PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: **DATE:** September 27, 2024 **HOLE NO. :** BH14-24

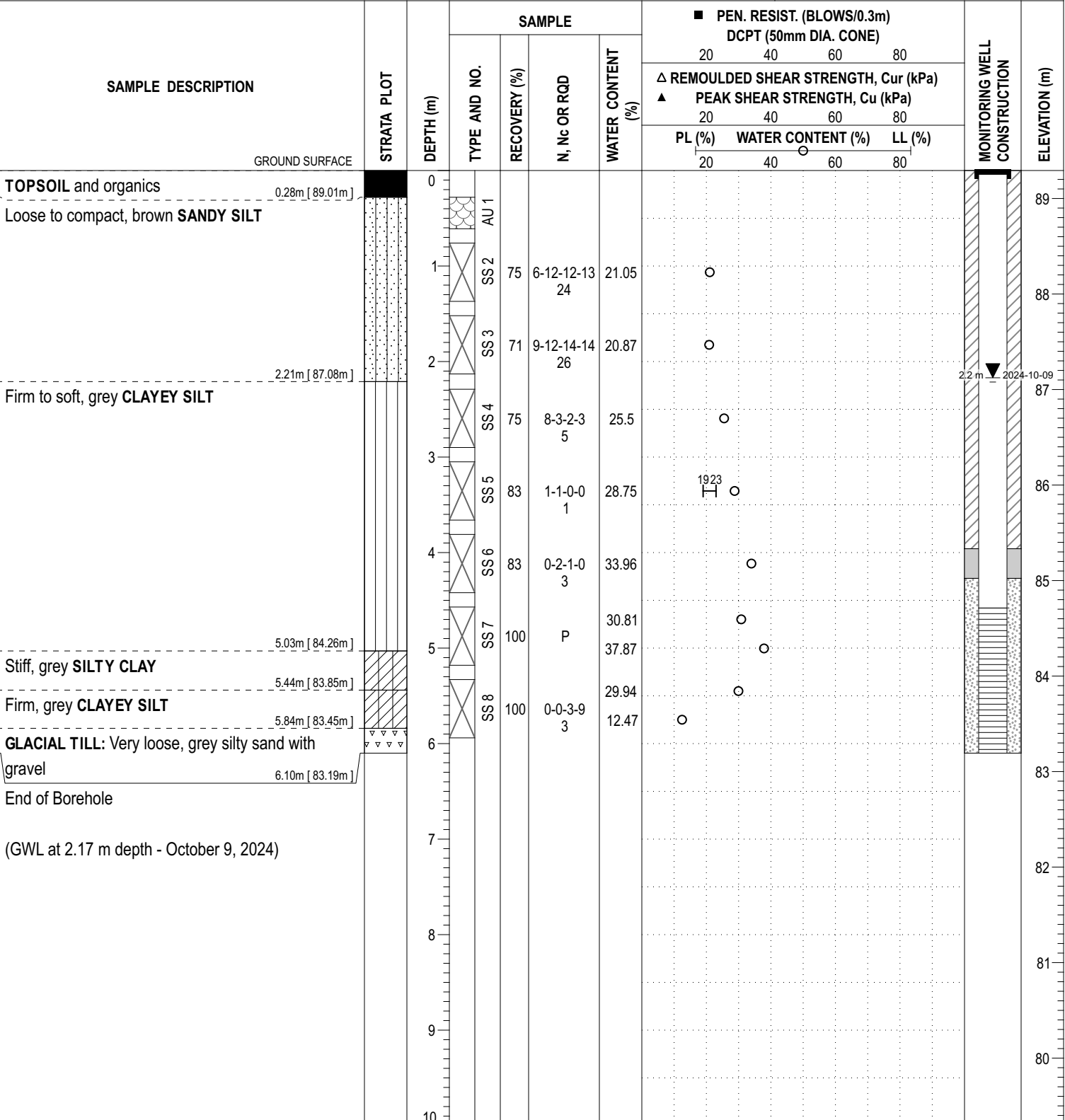
SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80					
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20 40 60 80					
PL (%) WATER CONTENT (%) LL (%)				20 40 60 80								
GROUND SURFACE												
TOPSOIL and organics		0									90	
Loose, brown SILTY SAND , trace clay			AU 1									
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders		1	SS 2	67	5-24-24-18 48	8.75	○				89	
- Grey by 1.5 m depth												
		2	SS 3	79	18-30-36-50 66	6.93	○				88	
		3	SS 4	100	34-50-/-/ 50/0.13	6.56	○				87	
		4	SS 5	100	39-50-/-/ 50/0.13	6.43	○				86	
		5	SS 6	100	16-50-/-/ 50/0.1	6.13	○				85	
		6	SS 7	100	23-50-/-/ 50/0.1	5.57	○				84	

COORD. SYS.: MTM ZONE 9 EASTING: 376738.02 NORTHING: 5020086.71 ELEVATION: 89.29

PROJECT: Geotechnical Investigation FILE NO.: **PG7301**

BORINGS BY: CME-55 Low Clearance Drill

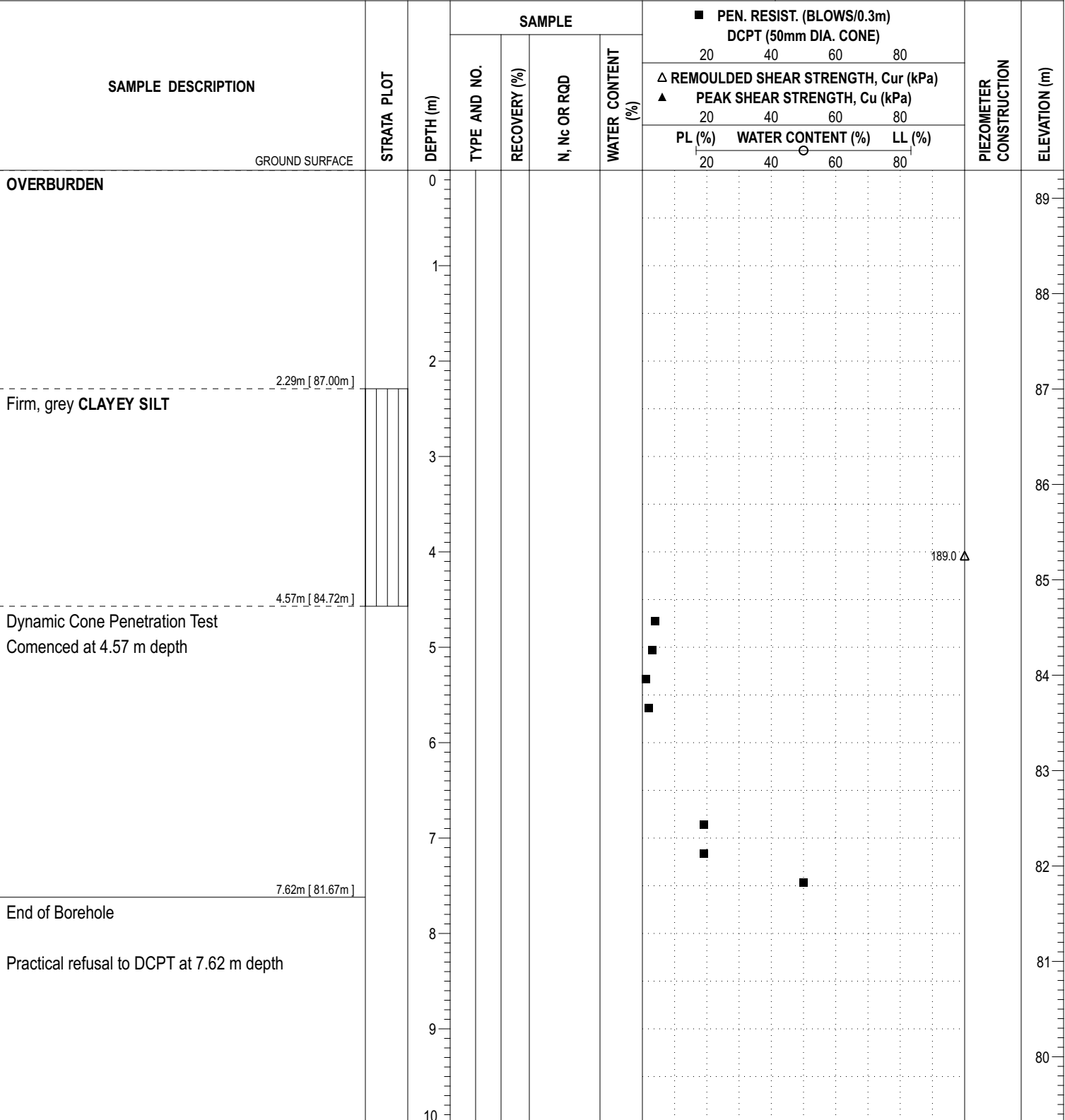
REMARKS: DATE: September 27, 2024 HOLE NO.: **BH15-24**



COORD. SYS.: MTM ZONE 9 **EASTING:** 376738.02 **NORTHING:** 5020086.71 **ELEVATION:** 89.29

PROJECT: Geotechnical Investigation **FILE NO. :** PG7301

BORINGS BY: CME-55 Low Clearance Drill

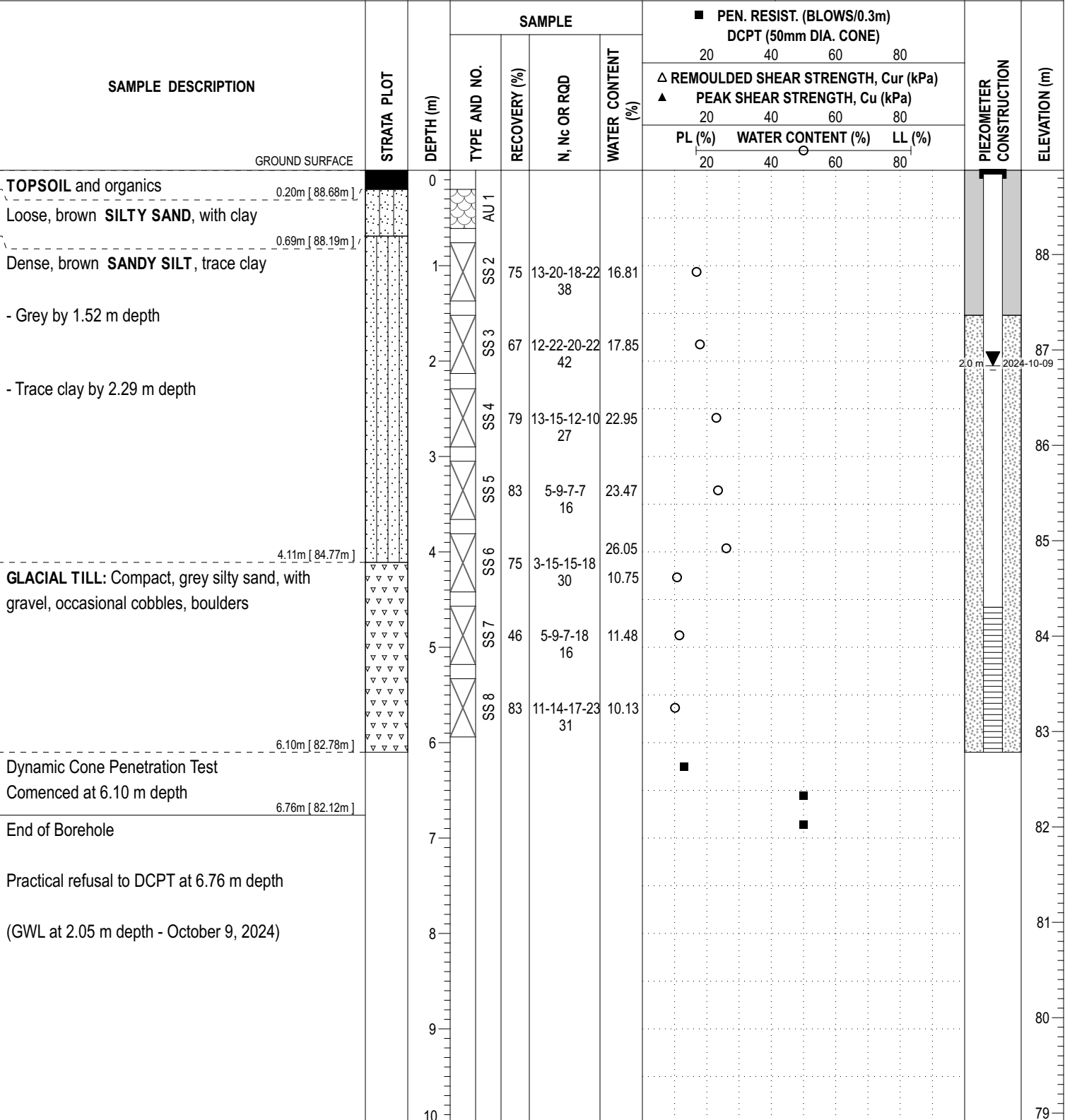
REMARKS: **DATE:** September 30, 2024 **HOLE NO. :** BH15A-24


COORD. SYS.: MTM ZONE 9 EASTING: 376652.26 NORTHING: 5020184.99 ELEVATION: 88.88

PROJECT: Geotechnical Investigation FILE NO.: PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 30, 2024 HOLE NO.: BH16-24



COORD. SYS.: MTM ZONE 9 **EASTING:** 376837.58 **NORTHING:** 5020217.72 **ELEVATION:** 89.28

PROJECT: Geotechnical Investigation **FILE NO. :** PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: **DATE:** September 30, 2024 **HOLE NO. :** BH17-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				MONITORING WELL CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20	40	60	80		
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									
GROUND SURFACE												
TOPSOIL and organics		0	AU 1								89	
Loose, brown SANDY SILT, trace clay												
		1	SS 2	87	10-19-20-29 39	9.28	○				88	
GLACIAL TILL: Dense brown silty sand with gravel, cobbles, boulders, trace clay												
- Grey by 1.5 m depth		2	SS 3	58	21-23-20-12 43	9.29	○				87	
		3	SS 4	67	9-21-12-14 33	7.67	○				86	
- Very dense below 3.1m depth												
		4	SS 5	100	23-50-/-/ 50/0.13	7.18	○				85	
		4	SS 6	100	19-50-/-/ 50/0.08	7.89	○				84	
End of Borehole												
Practical refusal to augering at 4.14m depth		5									83	
(GWL at 2.07 m depth - October 9, 2024)												
		6									82	
		7									81	
		8									80	
		9										
		10										

COORD. SYS.: MTM ZONE 9 EASTING: 376938.22 NORTHING: 5020281.76 ELEVATION: 88.30

PROJECT: Geotechnical Investigation FILE NO.: PG7301

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 30, 2024 HOLE NO.: BH18-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							20	40	60	80		
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									
GROUND SURFACE												
TOPSOIL and organics		0	AU 1								88	
FILL: Firm, brown silty clay												
TOPSOIL		1	SS 2	75	4-4-4-3	18.61						
FILL: Firm clayey silt					8	29.23						
TOPSOIL and organics			SS 3	8	2-2-2-2	17.43						
GLACIAL TILL: Compact, brown silty sand with gravel, occasional cobbles		2			4							
			SS 4	46	8-8-15-16	14.27						
					23							
		3	SS 5	46	5-7-10-8	11.77						
- Grey by 3.1 m depth					17							
			SS 6	87	35-15-18-18	13.4						
					33							
			SS 7	100	50-/-/-	10.33						
					50/0.15							
End of Borehole		4.72m [83.58m]										
Practical refusal to augering at 4.72 m depth		5									83	
(GWL at 1.31 m depth - October 9, 2024)		6									82	
		7									81	
		8									80	
		9									79	
		10										

COORD. SYS.: MTM ZONE 9	EASTING: 376684.09	NORTHING: 5020118.93	ELEVATION: 90.08
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PROJECT: Geotechnical Investigation BORINGS BY: CME-55 Low Clearance Drill REMARKS:	FILE NO. : PG7301 HOLE NO. : BH19-24
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DATE: September 30, 2024

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20	40	60	80		
							Δ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
							PL (%)	WATER CONTENT (%)		LL (%)		
					20	40	60	80				
GROUND SURFACE		0									90	
OVERBURDEN		1									89	
		2									88	
		3									87	
		4									86	
		5									85	
		6									84	
		7									83	
		8									82	
		9									81	
		10									80	
		11									79	
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	218									-128		
	219									-129		
	220											

COORD. SYS.: MTM ZONE 9 EASTING: 376740.96 NORTHING: 502042.35 ELEVATION: 88.78

PROJECT: Geotechnical Investigation FILE NO. : **PG7301**

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: DATE: September 30, 2024 HOLE NO. : **BH20-24**

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	20 40 60 80					
							Δ REMOULDED SHEAR STRENGTH, Cur (kPa)					
							▲ PEAK SHEAR STRENGTH, Cu (kPa)					
						20	40	60	80			
						PL (%)	WATER CONTENT (%)		LL (%)			
						20	40	60	80			
OVERBURDEN		0										
												88
		1										
												87
		2										
												86
		3										
												85
		4										
												84
	5											
											83	
	6											
											82	
	7											
											81	
	8											
											80	
	9											
											79	
	10											
GROUND SURFACE												
5.44m [83.34m]												
End of Borehole												
Practical refusal to augering at 5.44m depth												



4795 Bank St. & 3301 Blais Rd., Ottawa, Ontario

DATE: November 10, 2023

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4795 Bank St. & 3301 Blais Rd., Ottawa, Ontario

REMARKS: DATE: November 10, 2023

RSLog / Geotechnical Test Pit - Geodetic / paterson-group / admin / November 28, 2023 08:55 AM

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4795 Bank St. & 3301 Blais Rd., Ottawa, Ontario

DATE: November 10, 2023

The diagram illustrates a vertical cross-section of a test pit. The left side shows the soil profile with layers G1 through G7, their elevations, and descriptions. The right side shows a grid with a vertical axis labeled 0, 1, 2, 3 and a horizontal axis labeled 12.6, 30.6, 28.9, 24.1, 13.9, 30.7. A north arrow is located on the right side of the grid.

Layer	Soil Description	Thickness (m)	Elevation (m)
G1	TOPSOIL	0.3	EL 88.77
G2	Compact, brown SANDY SILT with thin interbedded layers of silty sand - trace clay by 0.7m depth	1.6	EL 87.47
G3			
G4			
G5	Stiff, grey CLAYEY SILT with thin interbedded layers of silty clay	3.2	EL 85.87
G6			
G7	GLACIAL TILL: Dense, grey clayey silt to sandy silt with gravel, occasional boulders	3.3	EL 85.77

Ground Surface: EL 89.07 m
End of Test Pit: EL 85.77 m

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SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

Cc and Cu are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$

Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

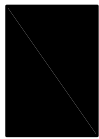
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

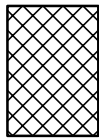
STRATA PLOT



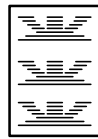
Topsoil



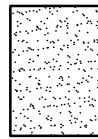
Asphalt



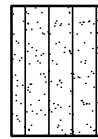
Fill



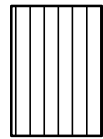
Peat



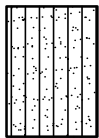
Sand



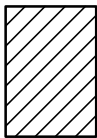
Silty Sand



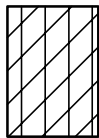
Silt



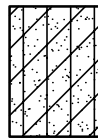
Sandy Silt



Clay



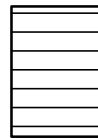
Silty Clay



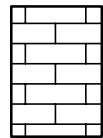
Clayey Silty Sand



Glacial Till



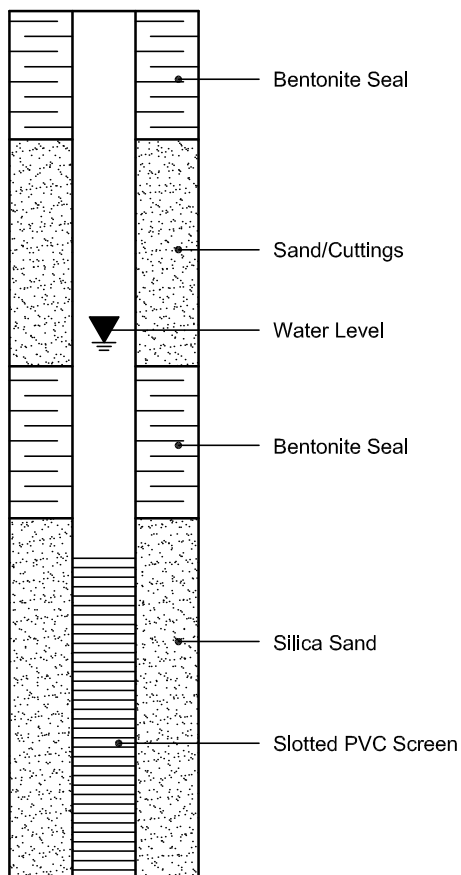
Shale



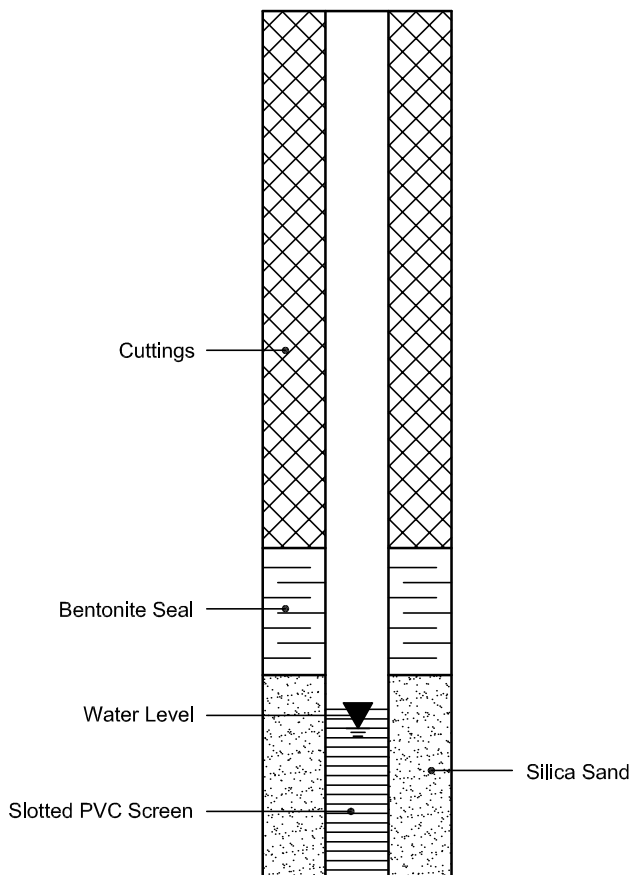
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION

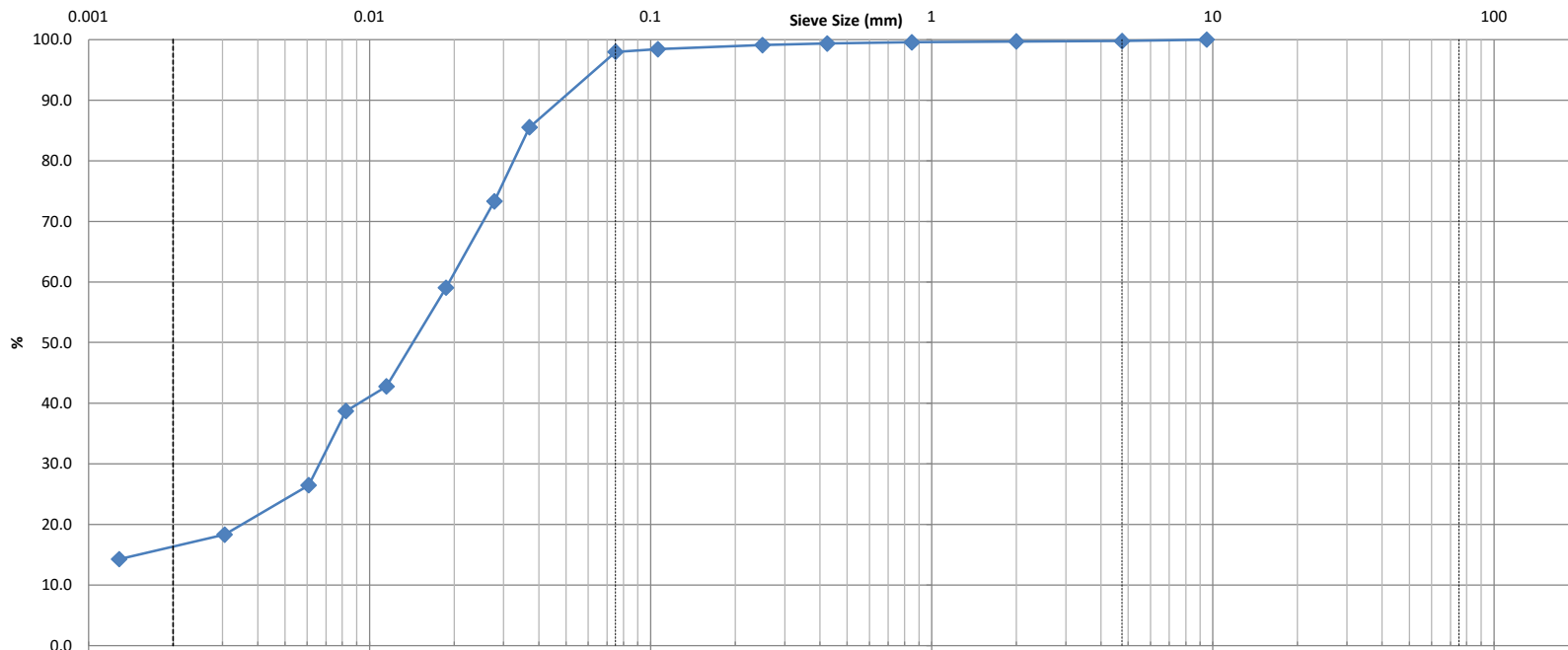


PIEZOMETER CONSTRUCTION



**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	-	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH1-26 SS3	LAB NO:	65709
PROJECT:	4795 Bank Street			DATE RECEIVED:	6-Apr-26
				DATE TESTED:	7-Apr-26
DATE SAMPLED:	6-Apr-26			DATE REPORTED:	15-Apr-26
SAMPLED BY:	A.E.			TESTED BY:	C.M



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

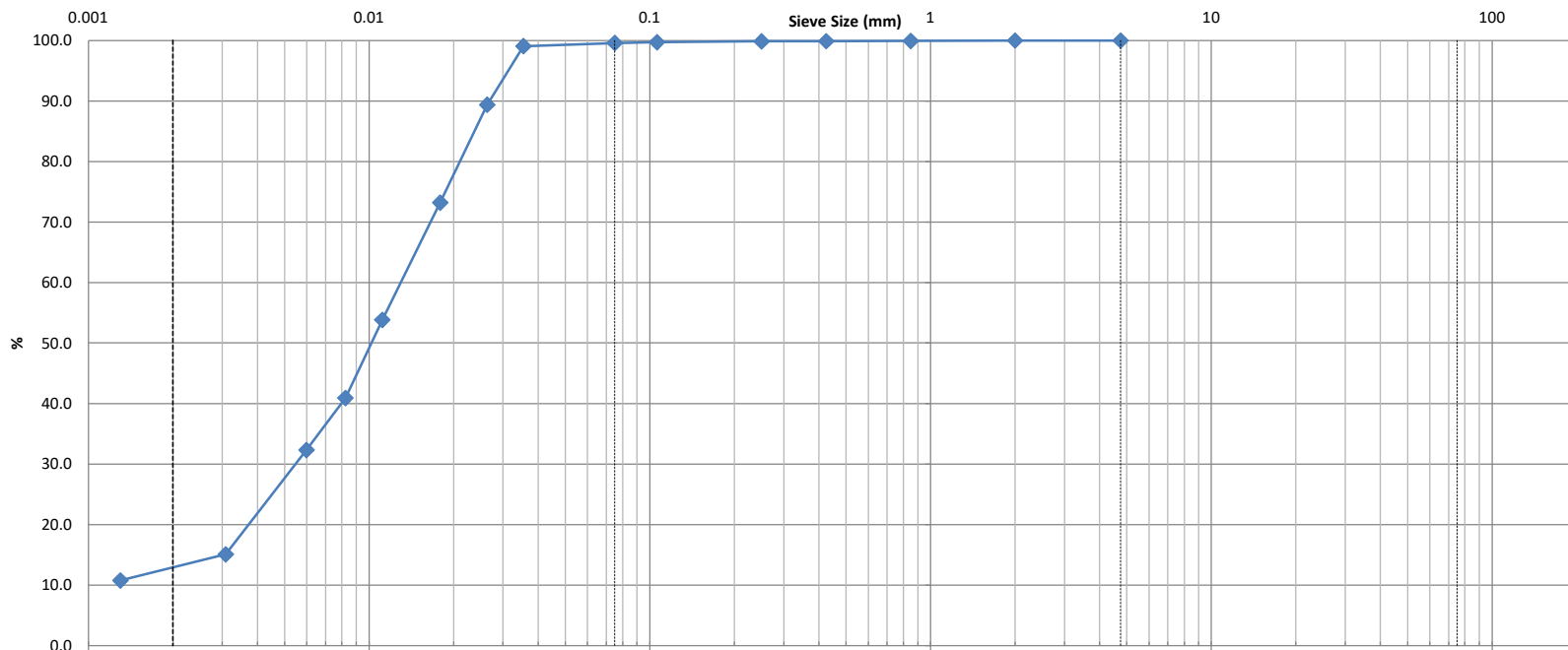
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						21.3%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.3	1.8		82.0		16.0	

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	-	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH1-26 SS5	LAB NO:	65710
PROJECT:	4795 Bank Street			DATE RECEIVED:	6-Apr-26
				DATE TESTED:	7-Apr-26
DATE SAMPLED:	6-Apr-26			DATE REPORTED:	15-Apr-26
SAMPLED BY:	A.E.			TESTED BY:	C.M



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						31.0%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	0.4		87.6		12.0	

Comments:

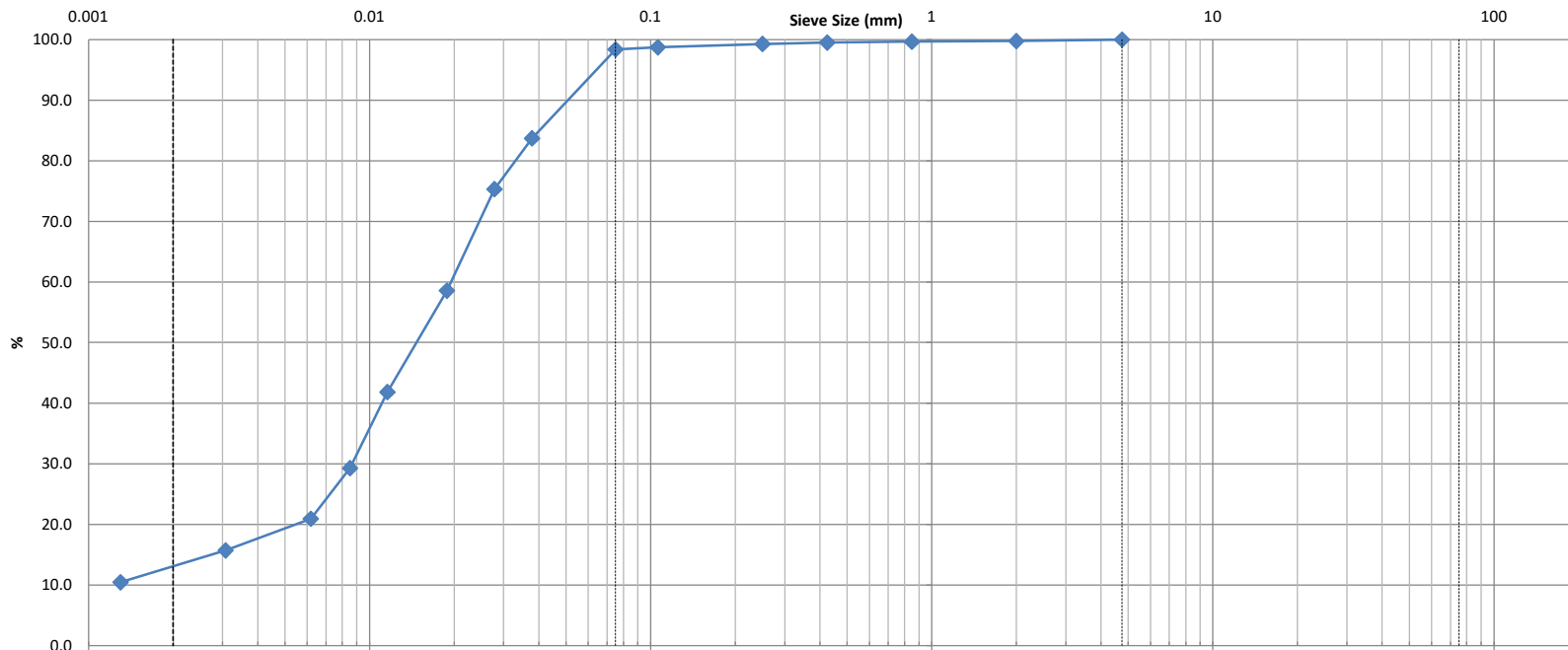
REVIEWED BY:

Curtis Beadow

Joe Forsyth, P. Eng.

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	-	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH2-26 SS3	LAB NO:	65711
PROJECT:	4795 Bank Street			DATE RECEIVED:	6-Apr-26
				DATE TESTED:	7-Apr-26
DATE SAMPLED:	6-Apr-26			DATE REPORTED:	15-Apr-26
SAMPLED BY:	A.E.			TESTED BY:	C.M



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

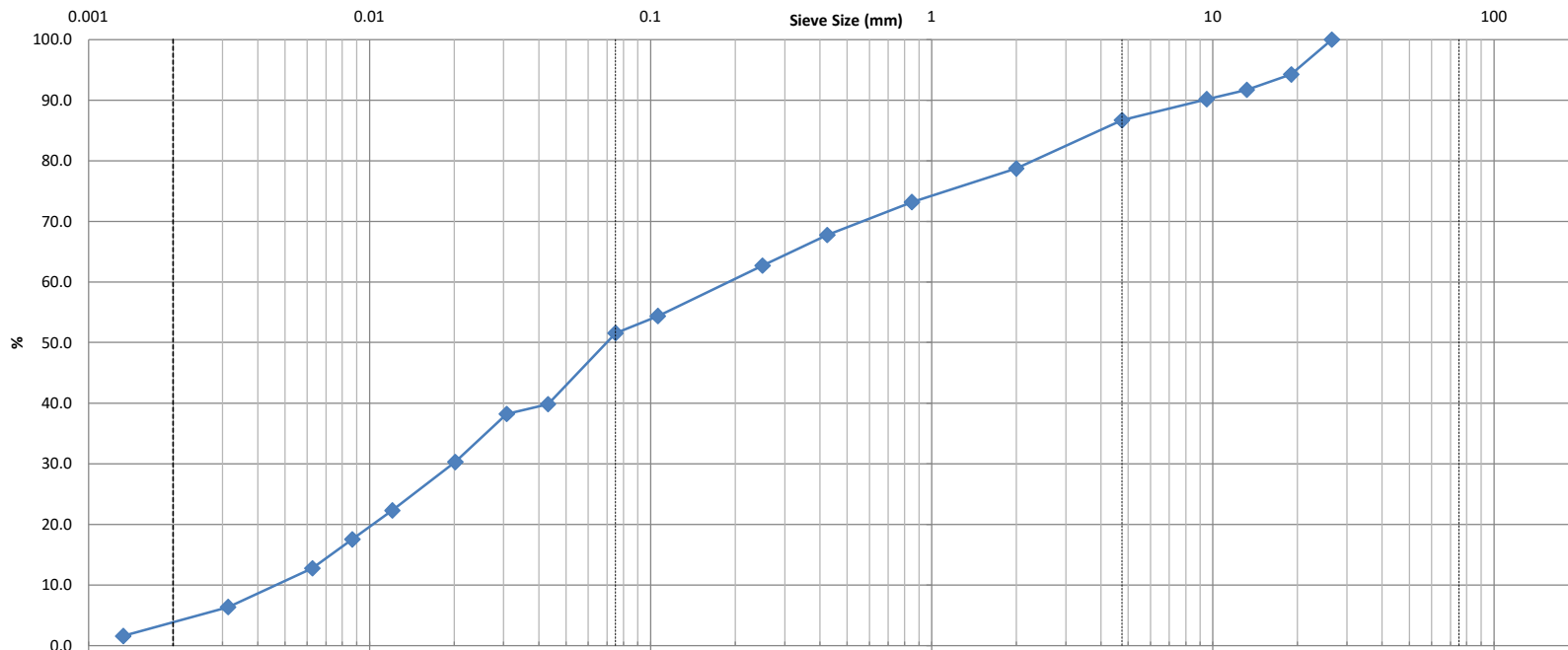
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						29.9%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	1.6		85.4		13.0	

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	-	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH2-26 SS8	LAB NO:	65712
PROJECT:	4795 Bank Street			DATE RECEIVED:	6-Apr-26
				DATE TESTED:	7-Apr-26
DATE SAMPLED:	6-Apr-26			DATE REPORTED:	15-Apr-26
SAMPLED BY:	A.E.			TESTED BY:	C.M



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

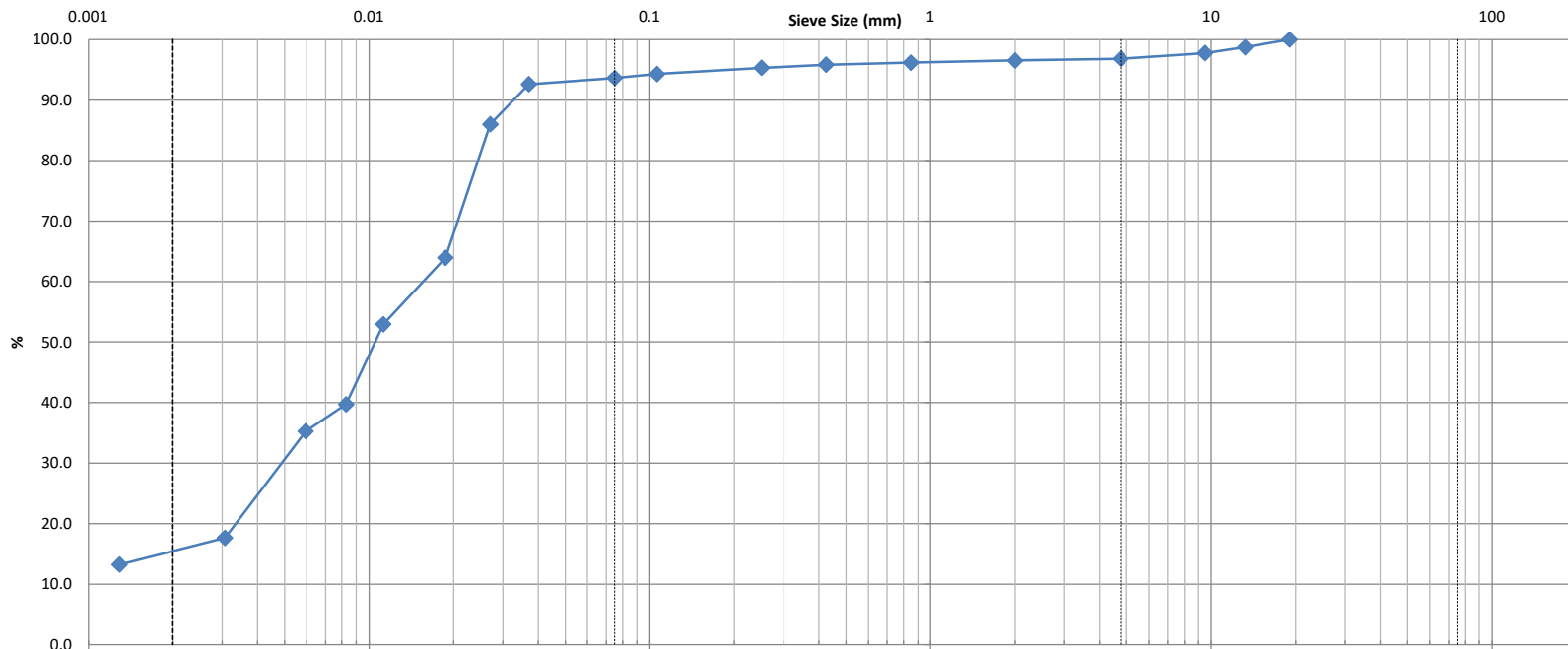
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	10.0%					
					13.3	Sand (%)	35.1	Silt (%)	47.6	Clay (%)	4.0

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	-	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH4-26 SS3	LAB NO:	65713
PROJECT:	4795 Bank Street			DATE RECEIVED:	6-Apr-26
				DATE TESTED:	7-Apr-26
DATE SAMPLED:	6-Apr-26			DATE REPORTED:	16-Apr-26
SAMPLED BY:	A.E.			TESTED BY:	C.M



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

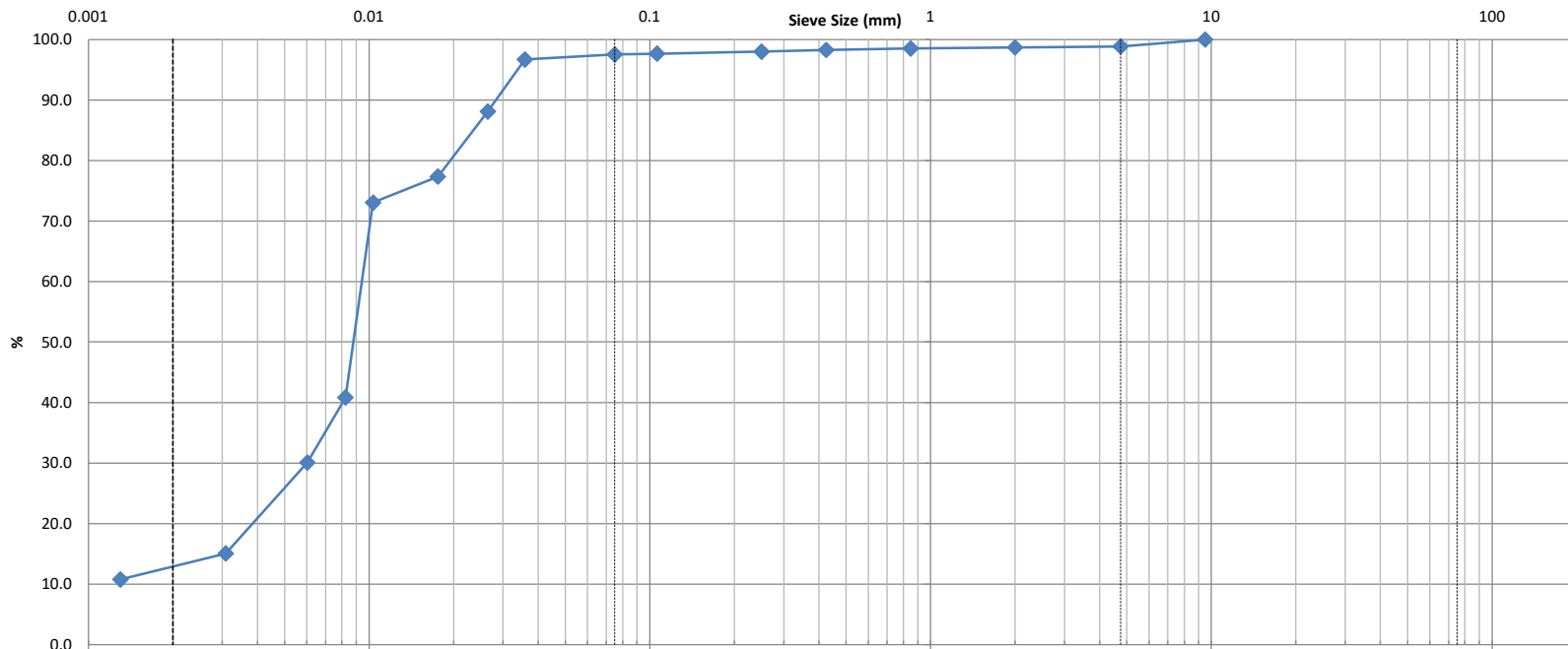
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						29.2%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					3.2	3.2		78.1		15.5	

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	-	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH4-26 SS5	LAB NO:	65714
PROJECT:	4795 Bank Street			DATE RECEIVED:	6-Apr-26
				DATE TESTED:	7-Apr-26
DATE SAMPLED:	6-Apr-26			DATE REPORTED:	16-Apr-26
SAMPLED BY:	A.E.			TESTED BY:	C.M



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

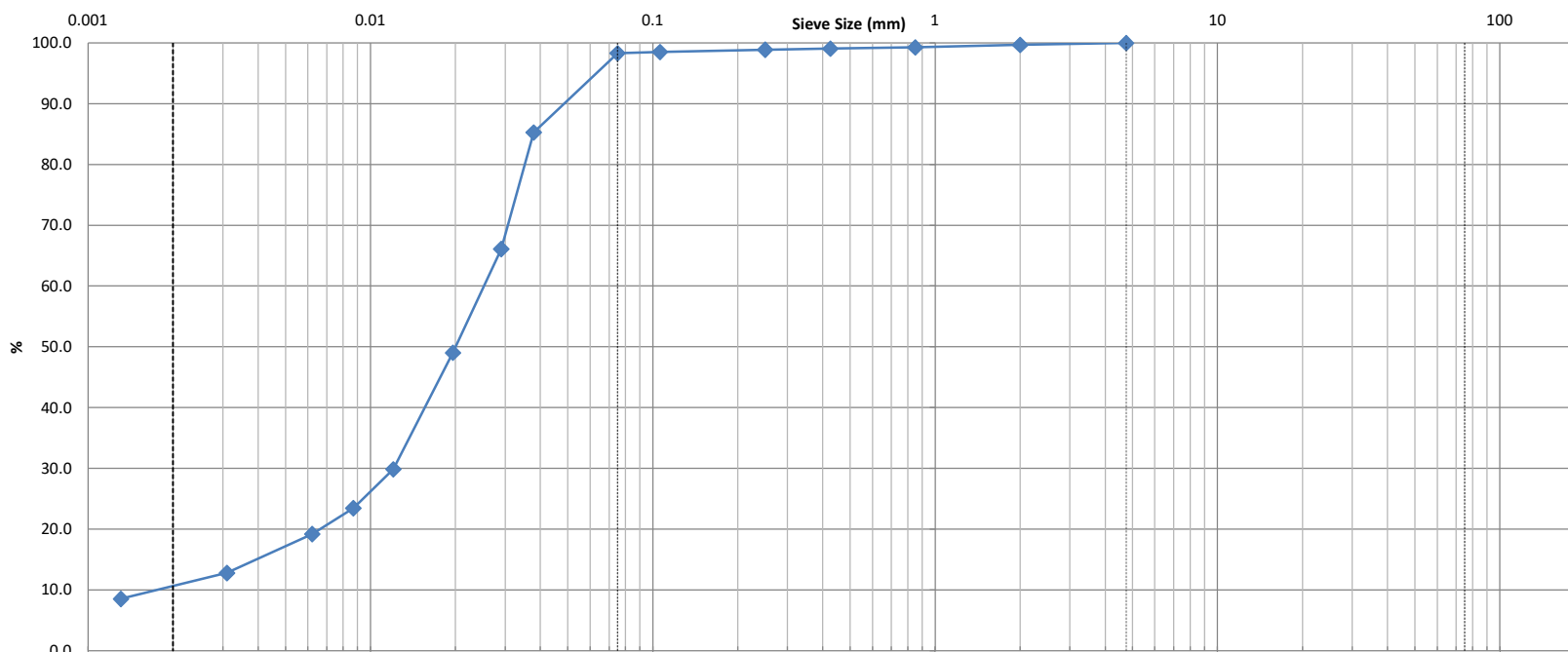
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						28.6%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					1.1	1.3		85.0		12.5	

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	2'6" - 4'6"	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH11-24 SS2	LAB NO:	57070
PROJECT:	4795 Bank Street, Ottawa, ON			DATE RECEIVED:	2-Oct-24
				DATE TESTED:	3-Oct-24
DATE SAMPLED:	26/27-Sep-24			DATE REPORTED:	17-Oct-24
SAMPLED BY:	Max R.			TESTED BY:	D.K



Clay	Silt	Fine Sand	Medium Sand	Coarse Sand	Fine Gravel	Coarse Gravel	Cobble
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Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	1.7	88.2	10.2		

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

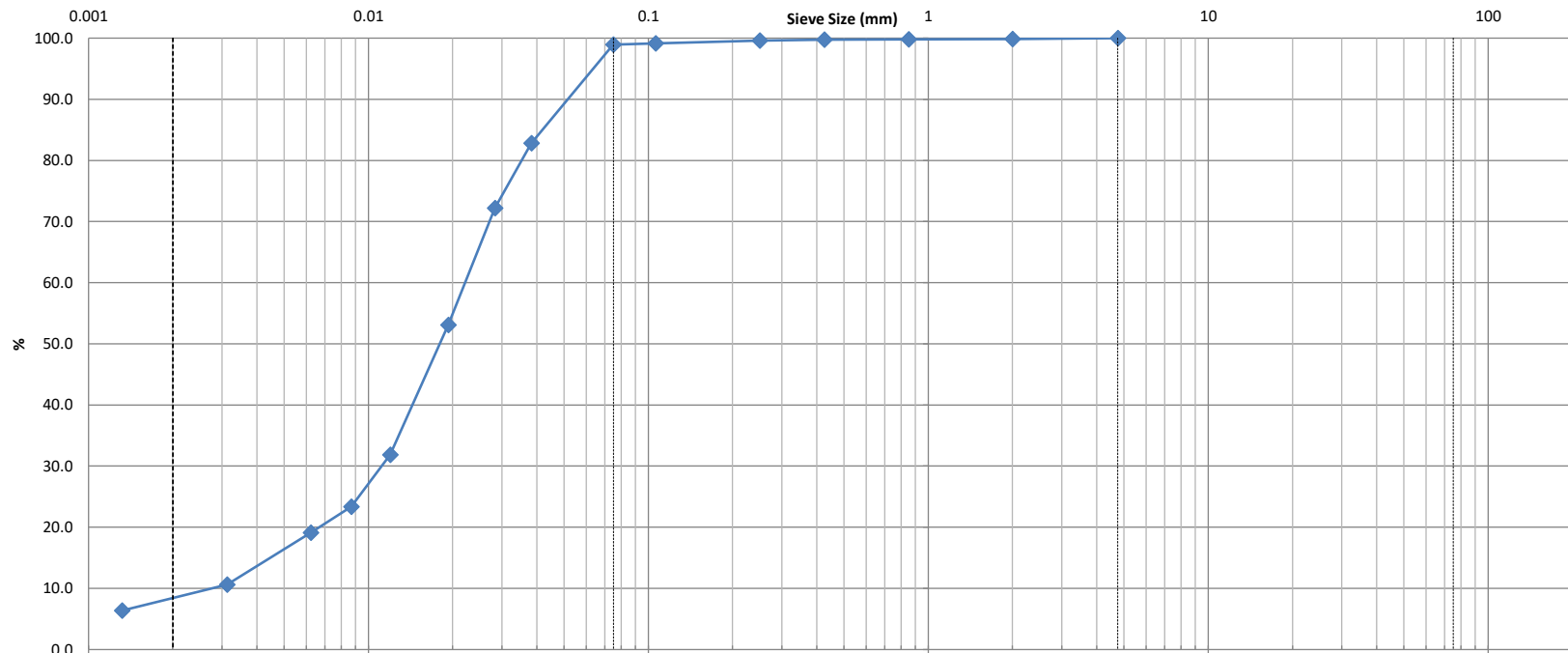


SIEVE ANALYSIS

C136

ASTM

CLIENT:	Caivan	DEPTH:	5' - 7'	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH12-24 SS3	LAB NO:	57071
PROJECT:	4795 Bank Street, Ottawa, ON			DATE RECEIVED:	2-Oct-24
				DATE TESTED:	3-Oct-24
DATE SAMPLED:	26/27-Sep-24			DATE REPORTED:	17-Oct-24
SAMPLED BY:	Max R.			TESTED BY:	D.K



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	1.1	90.9	8.0		

Comments:

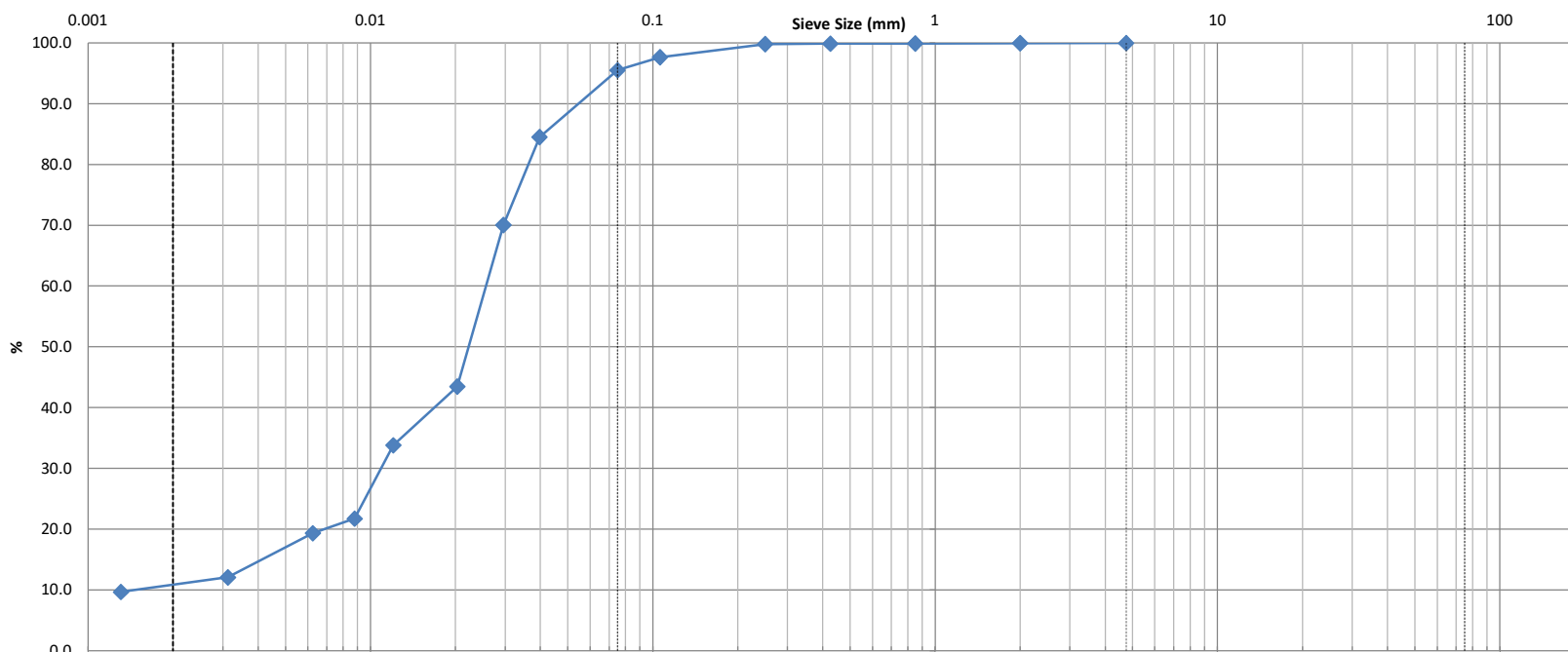
REVIEWED BY:

Curtis Beadow

Joe Forsyth, P. Eng.

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	10' - 12'	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH12-24 SS5	LAB NO:	57072
PROJECT:	4795 Bank Street, Ottawa, ON			DATE RECEIVED:	2-Oct-24
				DATE TESTED:	3-Oct-24
DATE SAMPLED:	26/27-Sep-24			DATE REPORTED:	18-Oct-24
SAMPLED BY:	Max R.			TESTED BY:	D.K



Clay	Silt			Sand			Gravel		Cobble
				Fine	Medium	Coarse	Fine	Coarse	

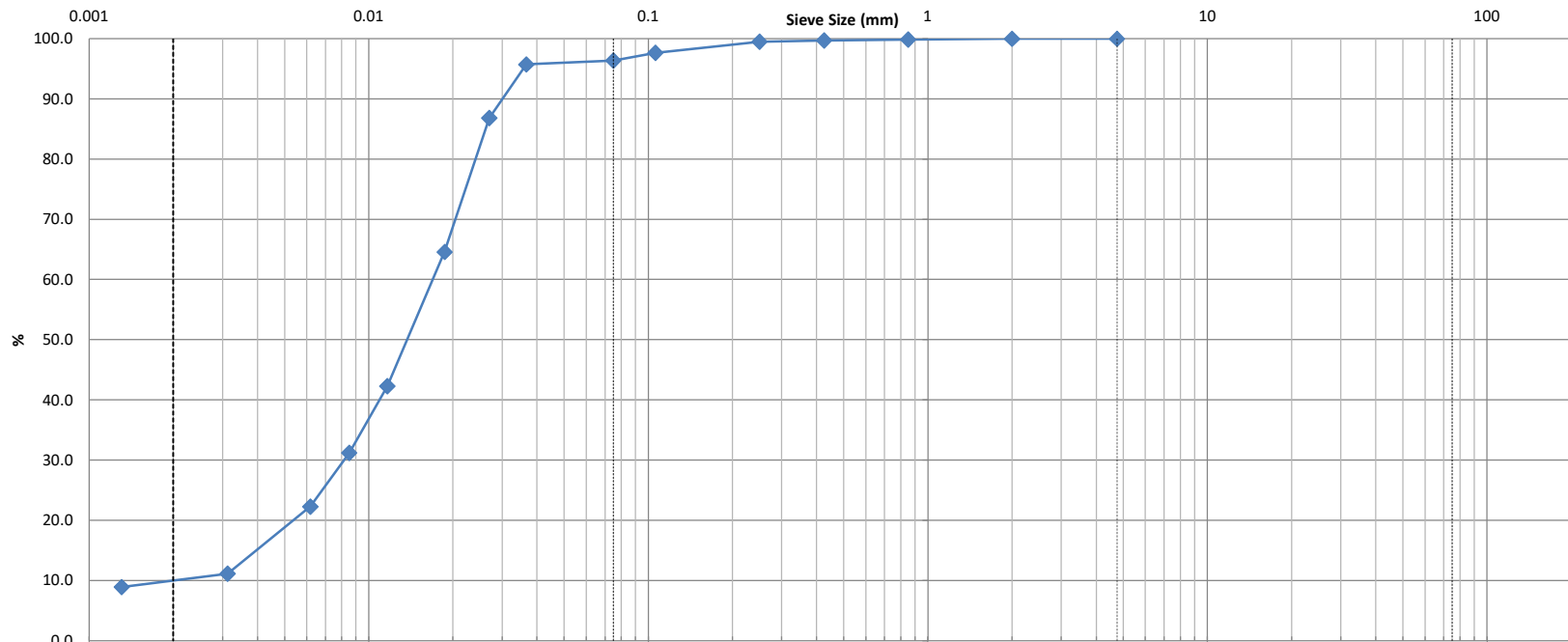
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	31.5%					
					0.0	4.5				84.9	10.6

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	12'6" - 14'6"	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH12-24 SS6	LAB NO:	57073
PROJECT:	4795 Bank Street, Ottawa, ON			DATE RECEIVED:	2-Oct-24
				DATE TESTED:	3-Oct-24
DATE SAMPLED:	26/27-Sep-24			DATE REPORTED:	18-Oct-24
SAMPLED BY:	Max R.			TESTED BY:	D.K



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

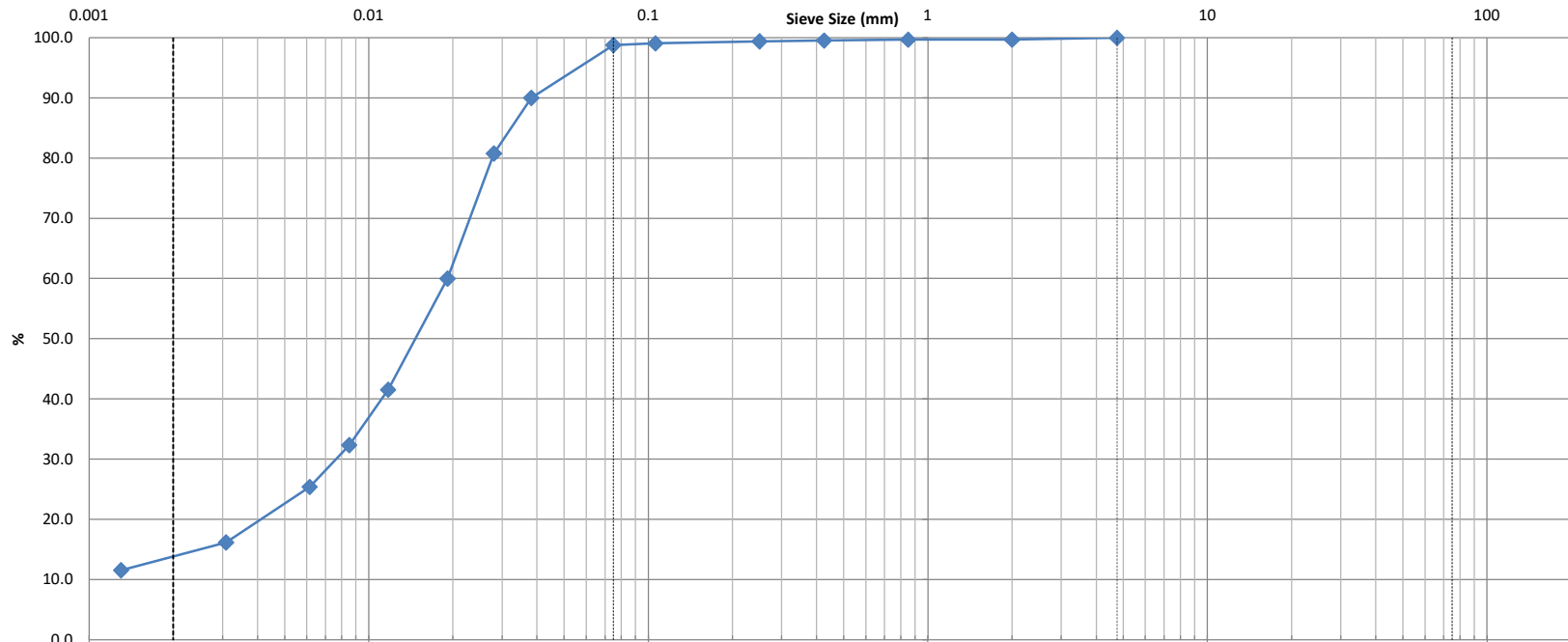
Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	3.6	86.6	9.8		

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

**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Caivan	DEPTH:	10' - 12'	FILE NO:	PG7301
CONTRACT NO.:		BH OR TP No.:	BH15-24 SS5	LAB NO:	57074
PROJECT:	4795 Bank Street, Ottawa, ON			DATE RECEIVED:	2-Oct-24
				DATE TESTED:	3-Oct-24
DATE SAMPLED:	26/27-Sep-24			DATE REPORTED:	17-Oct-24
SAMPLED BY:	Max R.			TESTED BY:	D.K

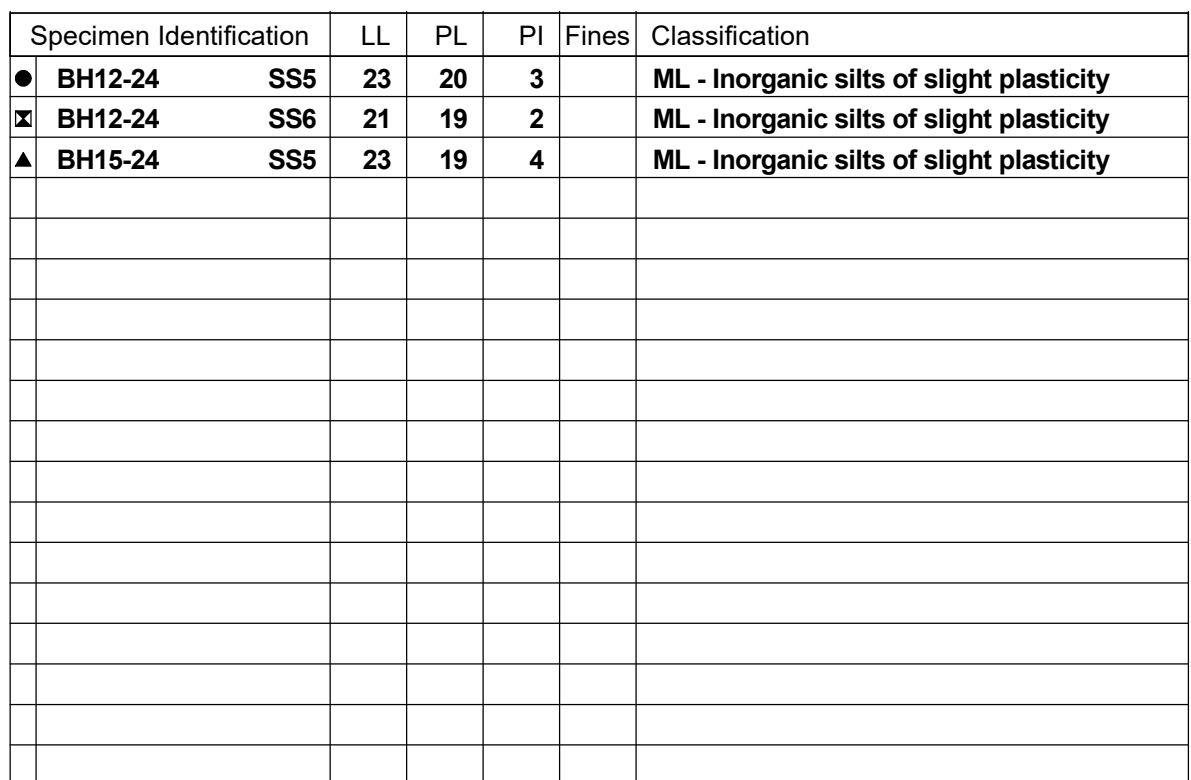


Clay	Silt				Sand			Gravel		Cobble
					Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						30.7%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	1.2		85.5		13.3	

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



Certificate of Analysis

Report Date: 08-Oct-2024

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 3-Oct-2024

Client PO: 61459

Project Description: PG7301

Client ID:	BH16-24 SS4	-	-	-	
Sample Date:	30-Sep-24 09:00	-	-	-	-
Sample ID:	2440489-01	-	-	-	
Matrix:	Soil	-	-	-	
MDL/Units					

Physical Characteristics

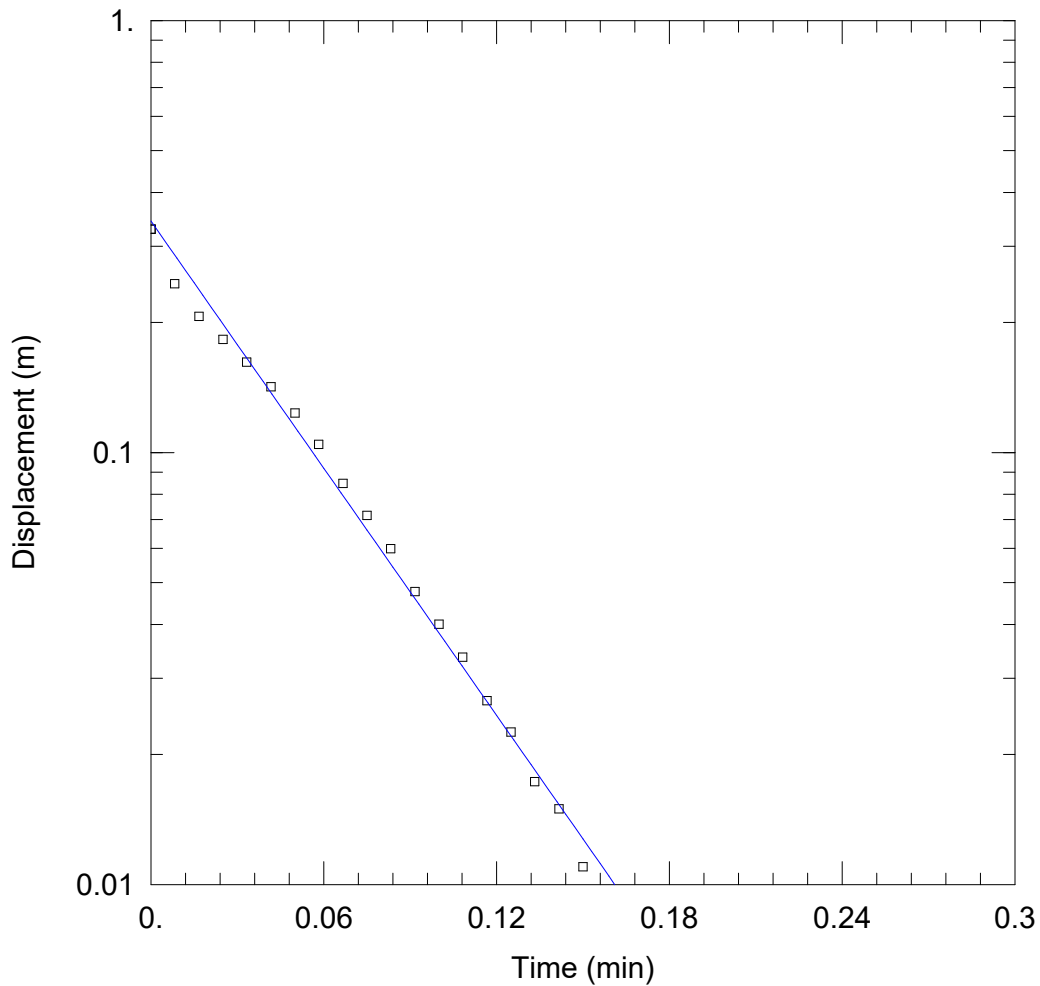
% Solids	0.1 % by Wt.	82.0	-	-	-	-
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General Inorganics

pH	0.05 pH Units	7.43	-	-	-	-
Resistivity	0.1 Ohm.m	51.4	-	-	-	-

Anions

Chloride	10 ug/g	<10	-	-	-	-
Sulphate	10 ug/g	68	-	-	-	-



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH1-26
 Test Date: May 26, 2026

WELL DATA (BH1-26)

Initial Displacement: 0.329 m
 Total Well Penetration Depth: 6.98 m
 Casing Radius: 0.01588 m

Static Water Column Height: 6.98 m
 Screen Length: 1.524 m
 Well Radius: 0.0381 m

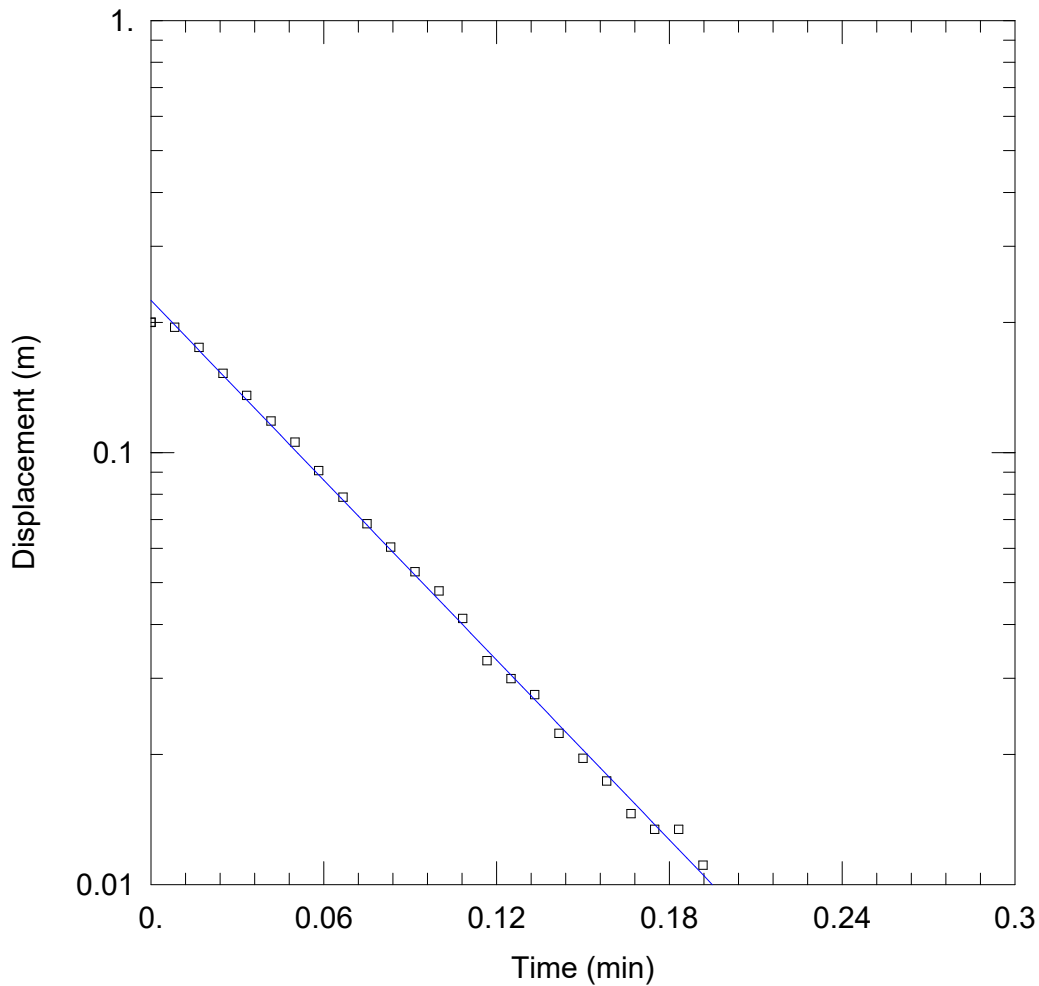
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

K = 0.0001674 m/sec

y0 = 0.3433 m



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH1-26
 Test Date: May 26, 2026

WELL DATA (BH1-26)

Initial Displacement: 0.2 m
 Total Well Penetration Depth: 6.98 m
 Casing Radius: 0.01588 m

Static Water Column Height: 6.98 m
 Screen Length: 1.524 m
 Well Radius: 0.0381 m

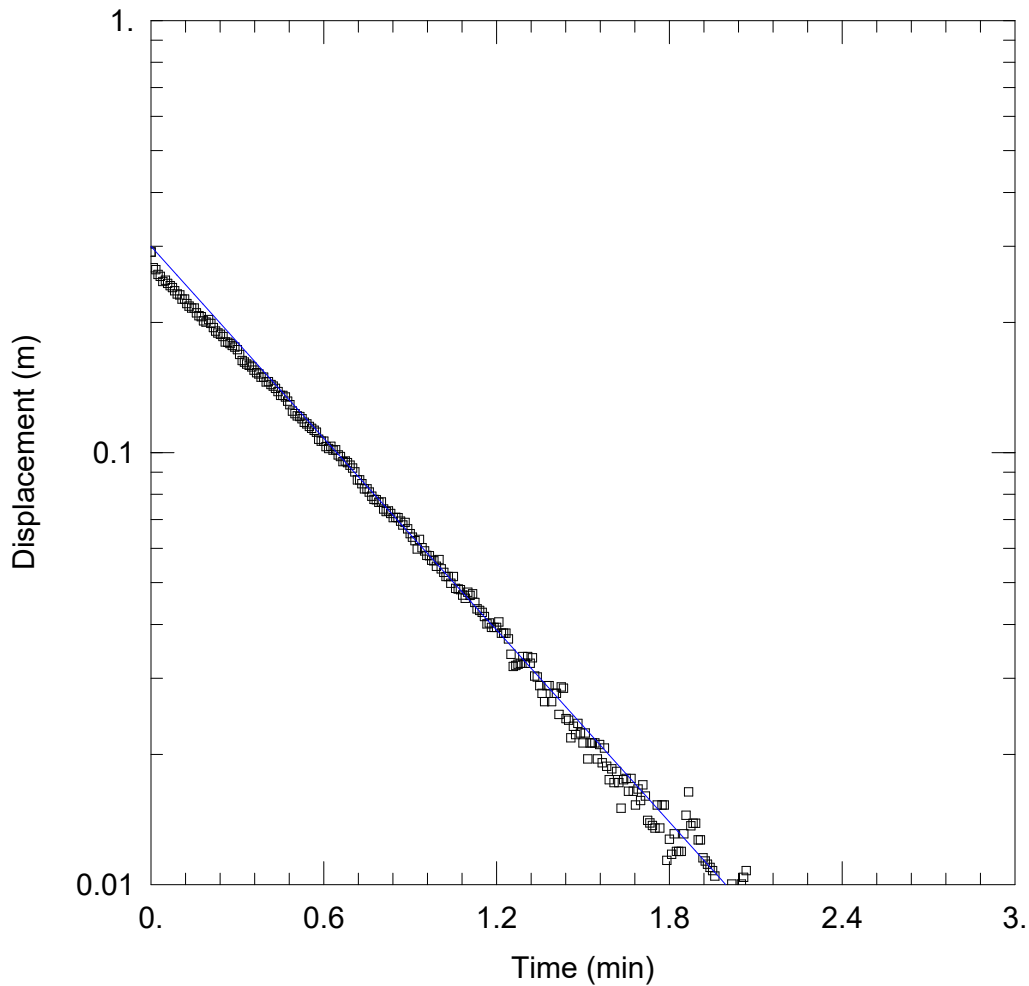
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

K = 0.0001218 m/sec

y0 = 0.225 m



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH2-26
 Test Date: May 26, 2026

WELL DATA (BH2-26)

Initial Displacement: 0.291 m
 Total Well Penetration Depth: 8.73 m
 Casing Radius: 0.01588 m

Static Water Column Height: 8.73 m
 Screen Length: 1.524 m
 Well Radius: 0.0381 m

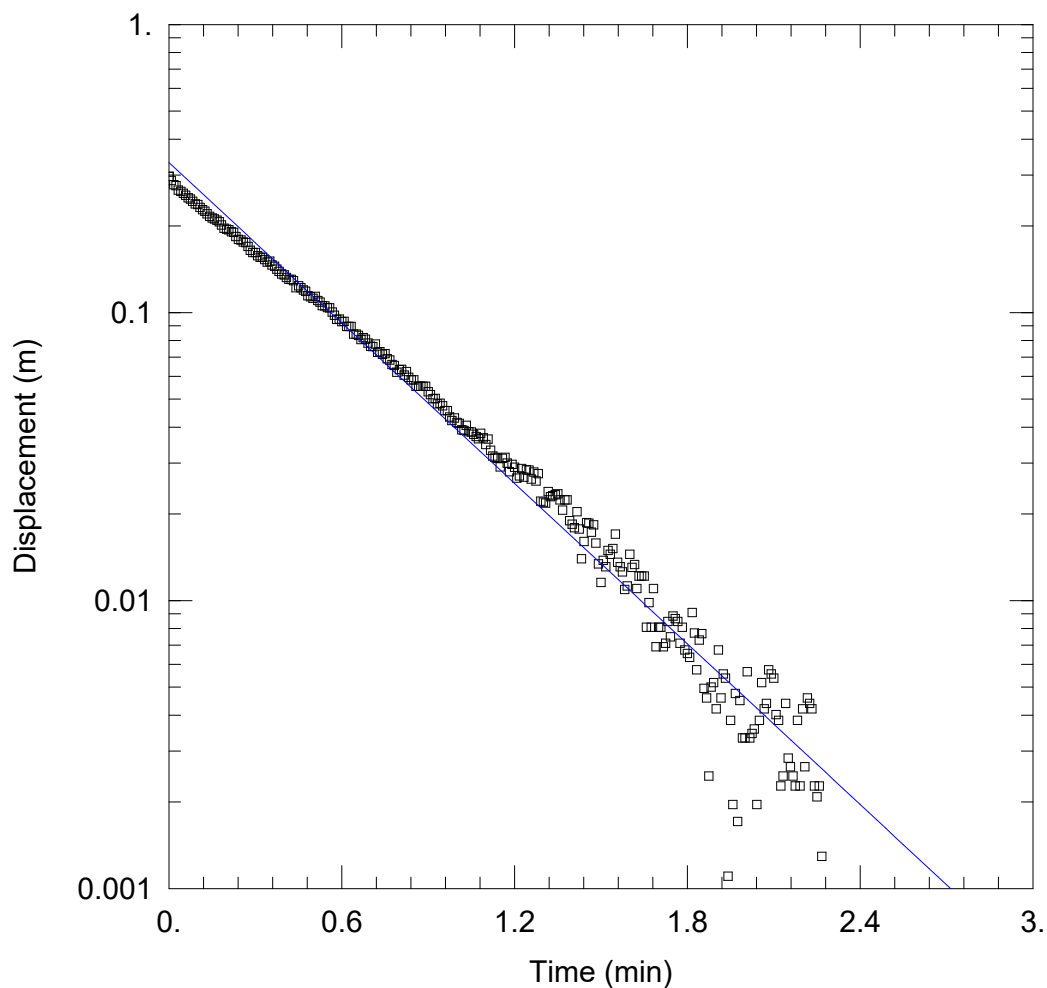
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 1.299E-5$ m/sec

$y_0 = 0.2999$ m



WELL TEST ANALYSIS

Data Set: P:\...\PG7301- BH2-26 Rising Head 2 of 2.aqt

Date: 05/27/26

Time: 15:23:39

PROJECT INFORMATION

Company: Paterson Group

Client: Caivan Communities

Project: PG7301

Location: 4795 Bank Street

Test Well: BH2-26

Test Date: May 26, 2026

AQUIFER DATA

Saturated Thickness: 8.74 m

Anisotropy Ratio (K_z/K_r): 0.1

WELL DATA (BH2-26)

Initial Displacement: 0.2973 m

Static Water Column Height: 8.74 m

Total Well Penetration Depth: 8.74 m

Screen Length: 1.524 m

Casing Radius: 0.01588 m

Well Radius: 0.0381 m

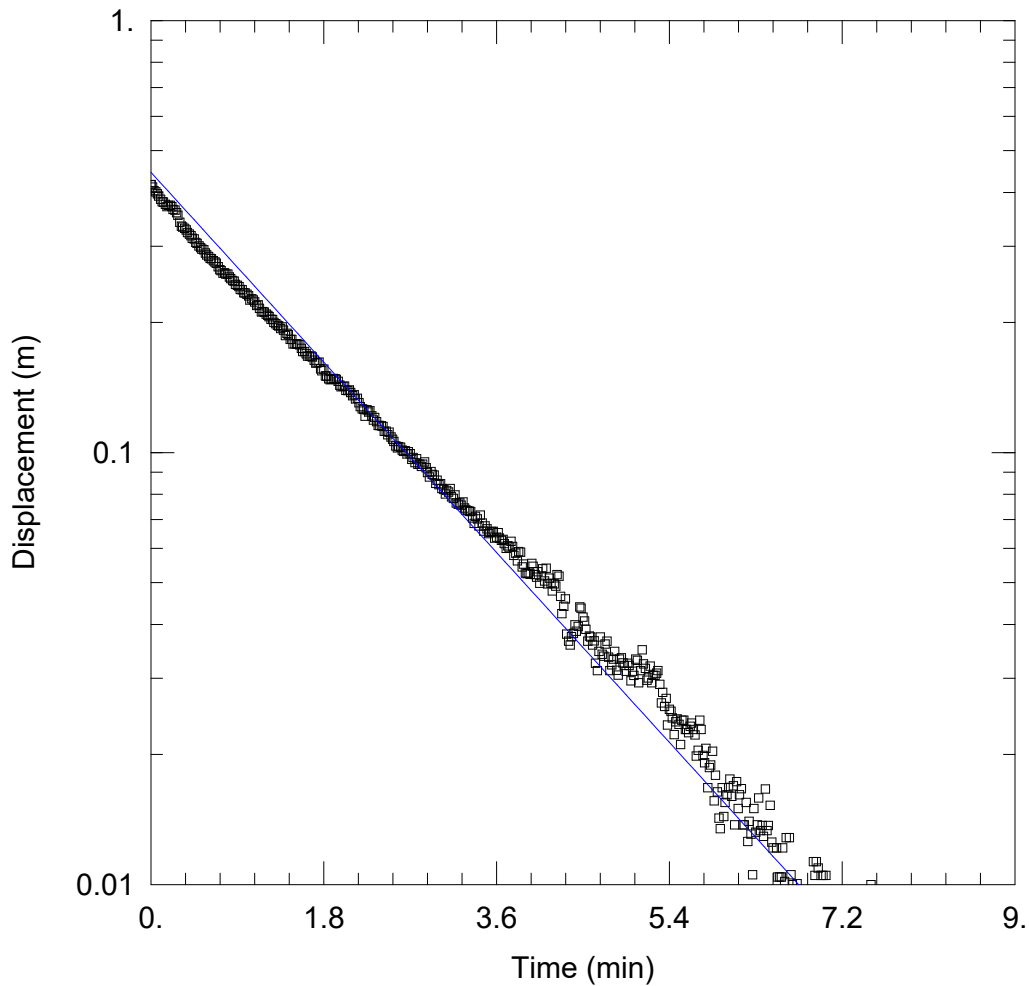
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 1.63E-5$ m/sec

$y_0 = 0.3317$ m



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH3-26
 Test Date: May 26, 2026

WELL DATA (BH3-26)

Initial Displacement: 0.417 m
 Total Well Penetration Depth: 4.9 m
 Casing Radius: 0.0254 m

Static Water Column Height: 4.9 m
 Screen Length: 1.524 m
 Well Radius: 0.1048 m

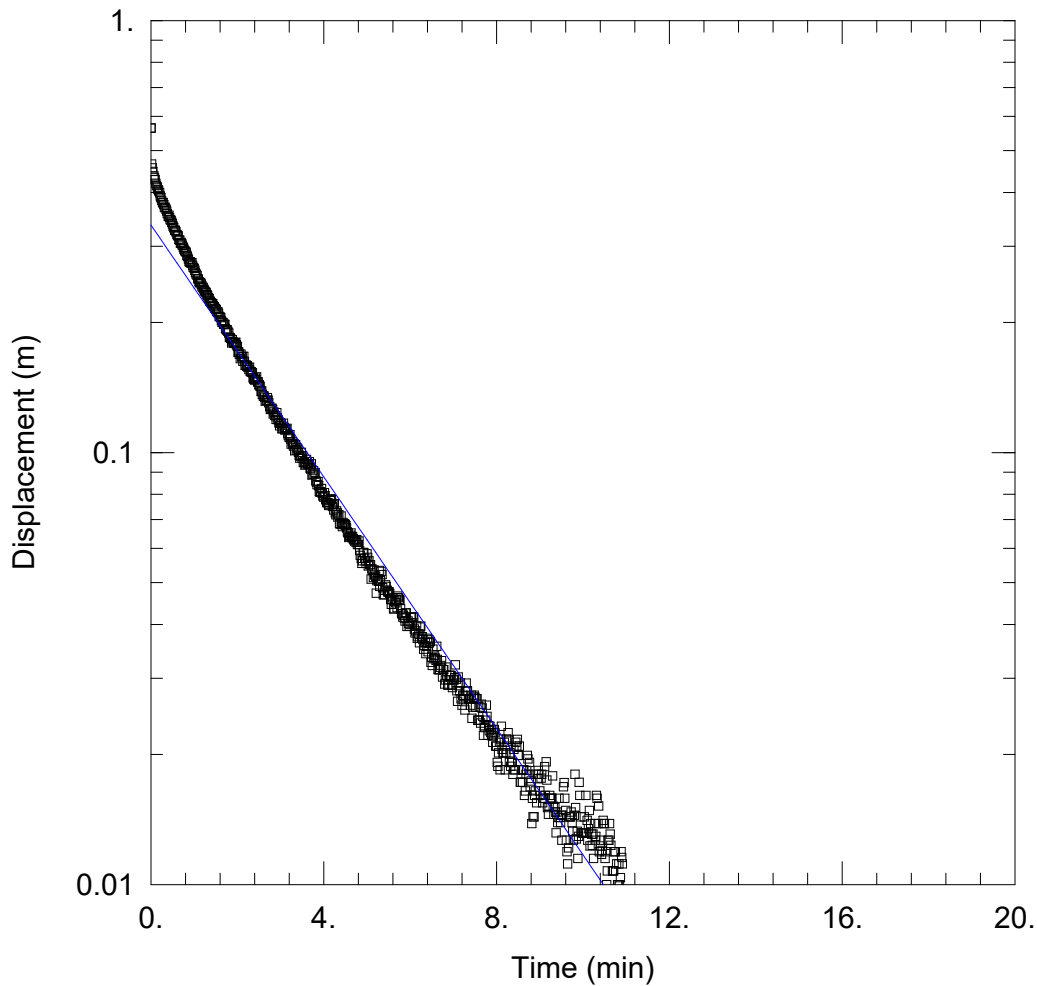
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

K = 8.971E-6 m/sec

y0 = 0.4452 m



WELL TEST ANALYSIS

PROJECT INFORMATION

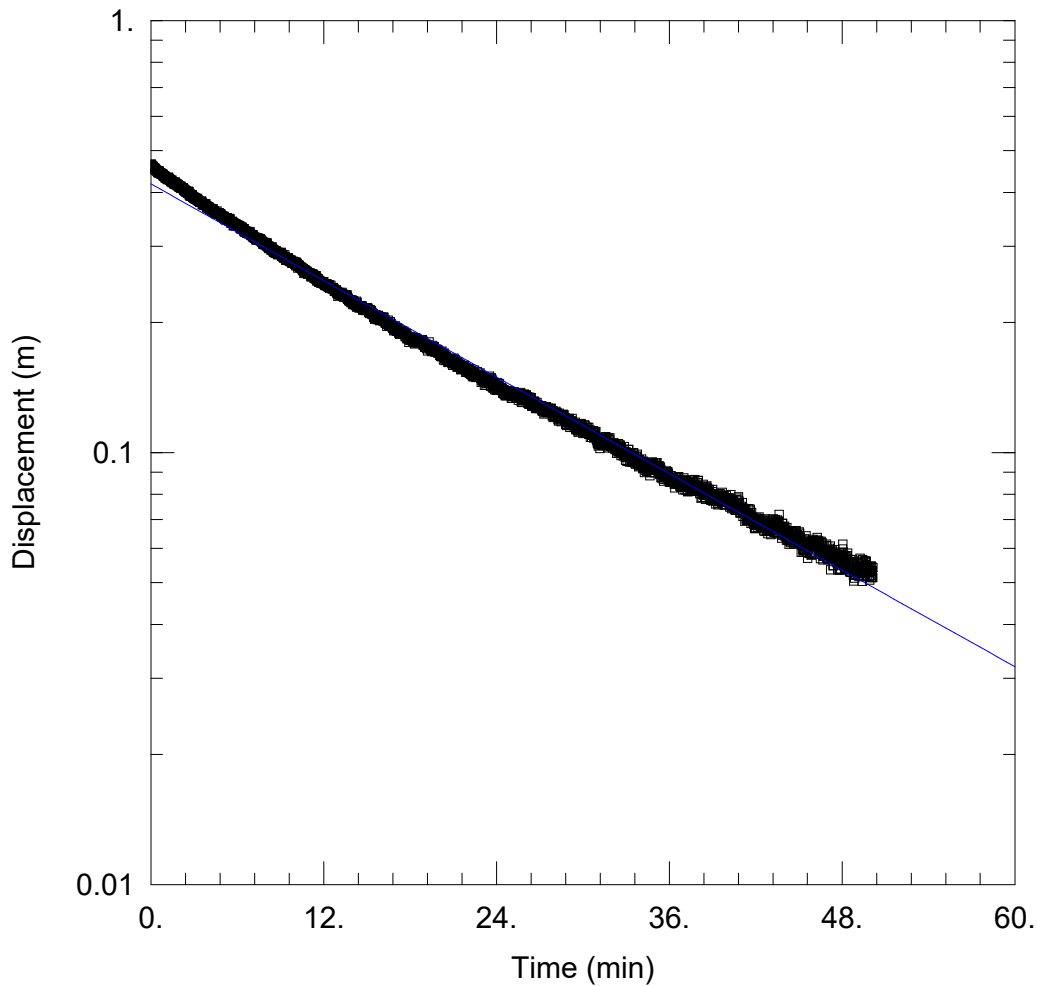
Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH3-26
 Test Date: May 26, 2026

WELL DATA (BH3-26)

Initial Displacement: <u>0.564 m</u>	Static Water Column Height: <u>4.92 m</u>
Total Well Penetration Depth: <u>4.92 m</u>	Screen Length: <u>1.524 m</u>
Casing Radius: <u>0.0254 m</u>	Well Radius: <u>0.1048 m</u>

SOLUTION

Aquifer Model: <u>Unconfined</u>	Solution Method: <u>Hvorslev</u>
K = <u>5.35E-6 m/sec</u>	y0 = <u>0.3359 m</u>



WELL TEST ANALYSIS

PROJECT INFORMATION

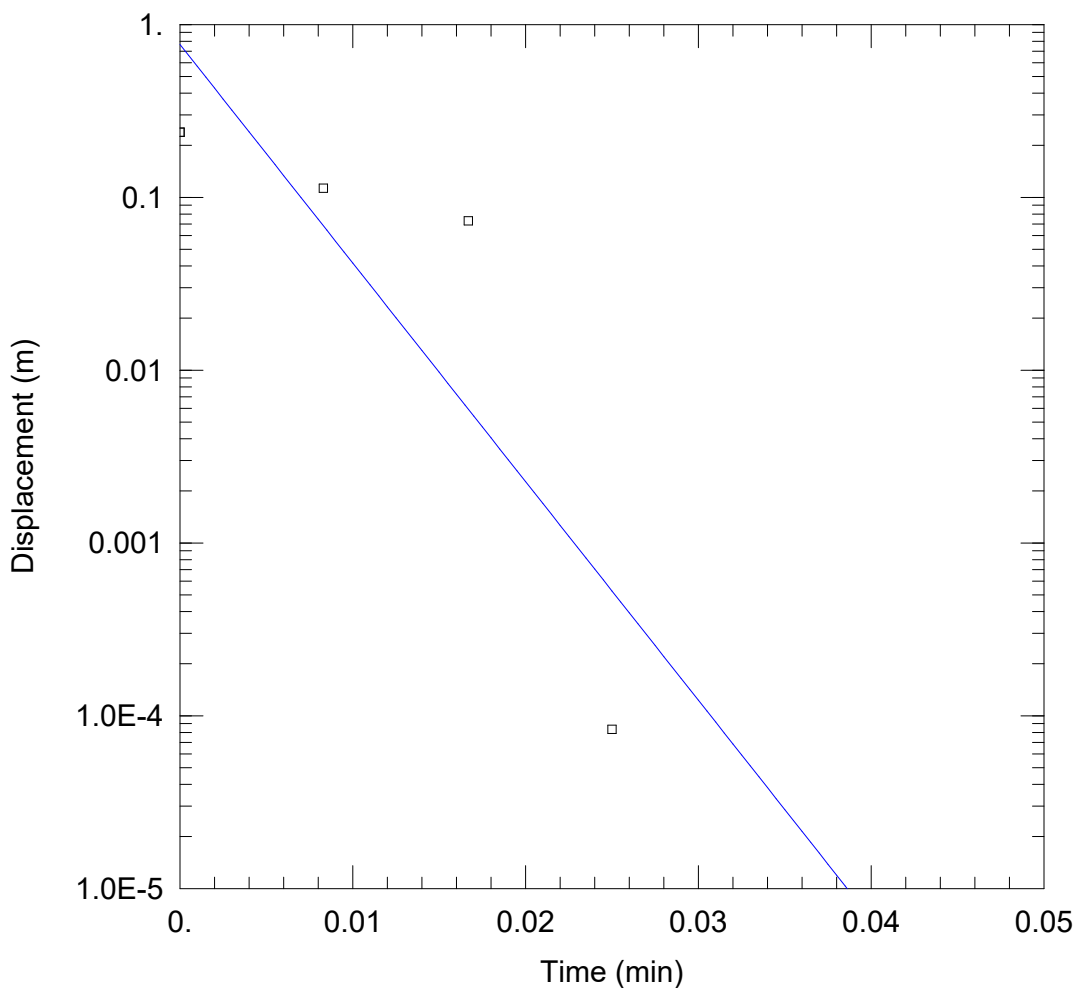
Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH4-26
 Test Date: May 26, 2026

WELL DATA (BH4-26)

Initial Displacement: <u>0.465 m</u>	Static Water Column Height: <u>3.49 m</u>
Total Well Penetration Depth: <u>3.49 m</u>	Screen Length: <u>1.524 m</u>
Casing Radius: <u>0.0254 m</u>	Well Radius: <u>0.1048 m</u>

SOLUTION

Aquifer Model: <u>Unconfined</u>	Solution Method: <u>Hvorslev</u>
K = <u>6.84E-7 m/sec</u>	y0 = <u>0.4184 m</u>



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH5-26
 Test Date: May 26, 2026

WELL DATA (BH5-26)

Initial Displacement: 0.2381 m
 Total Well Penetration Depth: 5. m
 Casing Radius: 0.0254 m

Static Water Column Height: 5. m
 Screen Length: 1.524 m
 Well Radius: 0.0381 m

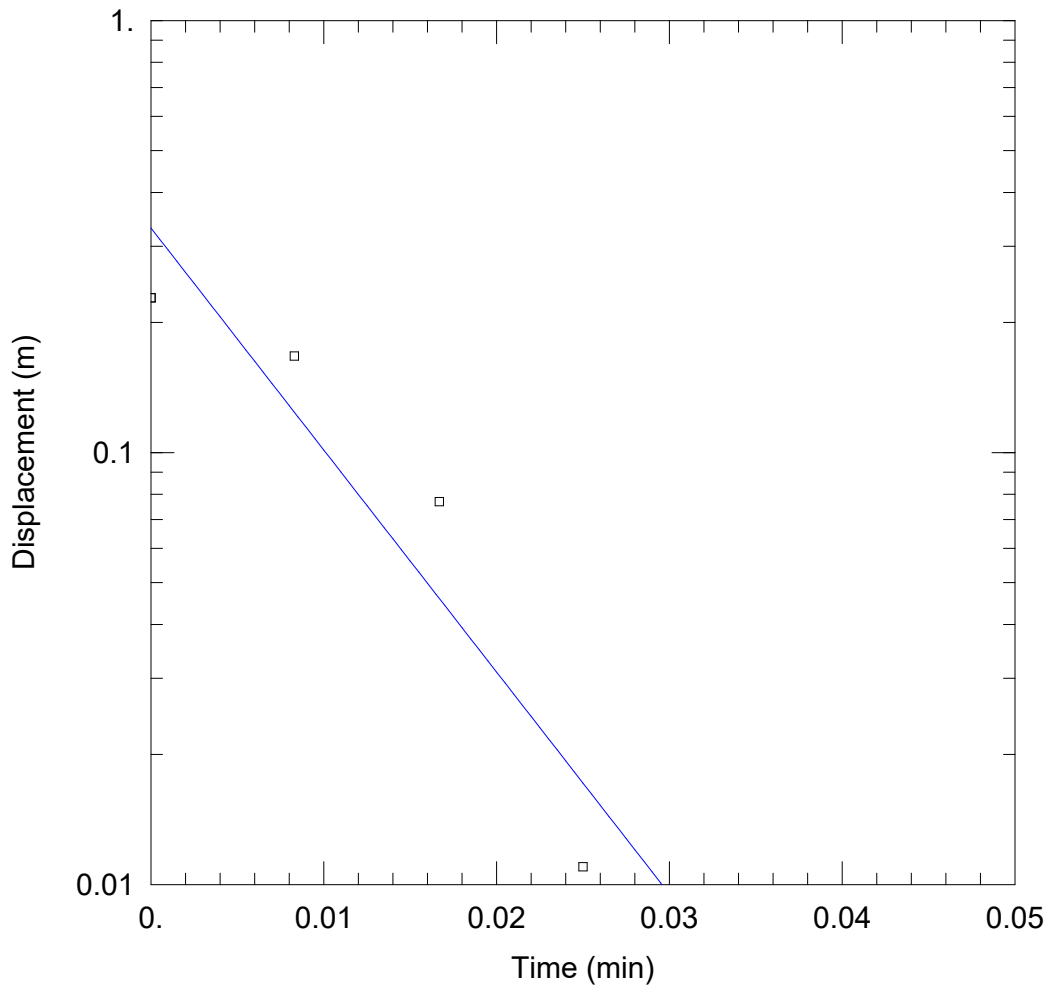
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

K = 0.005686 m/sec

y0 = 0.7668 m



WELL TEST ANALYSIS

Data Set: P:\...\PG7301- BH5-26 Rising Head 2 of 2.aqt

Date: 05/27/26

Time: 16:18:16

PROJECT INFORMATION

Company: Paterson Group

Client: Caivan Communities

Project: PG7301

Location: 4795 Bank Street

Test Well: BH5-26

Test Date: May 26, 2026

AQUIFER DATA

Saturated Thickness: 5. m

Anisotropy Ratio (K_z/K_r): 0.1

WELL DATA (BH5-26)

Initial Displacement: 0.228 m

Static Water Column Height: 5. m

Total Well Penetration Depth: 5. m

Screen Length: 1.524 m

Casing Radius: 0.0254 m

Well Radius: 0.0381 m

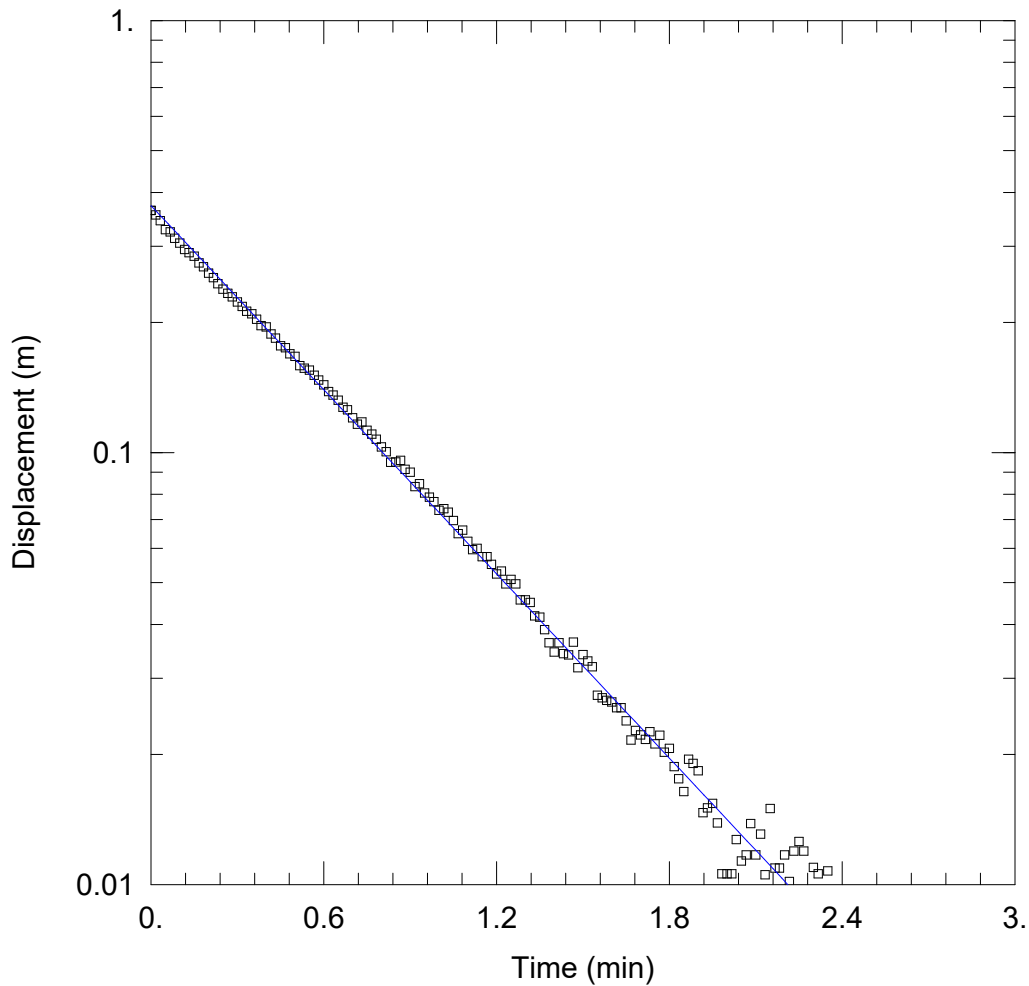
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 0.00231$ m/sec

$y_0 = 0.3309$ m



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH6-26
 Test Date: May 26, 2026

WELL DATA (BH6-26)

Initial Displacement: 0.3633 m
 Total Well Penetration Depth: 3.23 m
 Casing Radius: 0.0254 m

Static Water Column Height: 3.23 m
 Screen Length: 1.524 m
 Well Radius: 0.1048 m

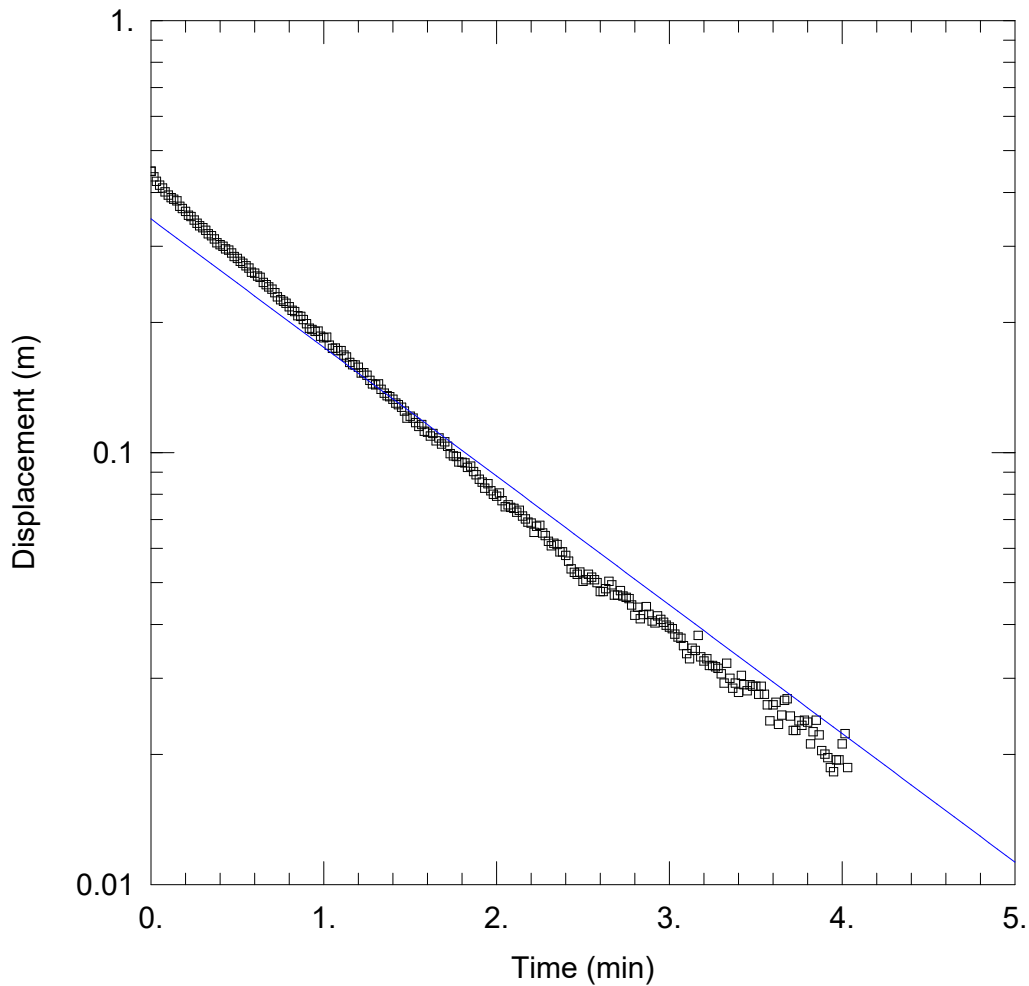
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 2.61E-5$ m/sec

$y_0 = 0.3729$ m



WELL TEST ANALYSIS

PROJECT INFORMATION

Company: Paterson Group
 Client: Caivan Communities
 Project: PG7301
 Location: 4795 Bank Street
 Test Well: BH6-26
 Test Date: May 26, 2026

WELL DATA (BH6-26)

Initial Displacement: 0.448 m
 Total Well Penetration Depth: 3.22 m
 Casing Radius: 0.0254 m

Static Water Column Height: 3.22 m
 Screen Length: 1.524 m
 Well Radius: 0.1048 m

SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

K = 9.259E-6 m/sec

y0 = 0.3474 m

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 to 5 – SLOPE STABILITY SECTIONS

MONITORING WELL WATER ELEVATION PLOTS

DRAWING PG7301-1 – TEST HOLE LOCATION PLAN

DRAWING PG7301-2 – BEDROCK CONTOUR PLAN

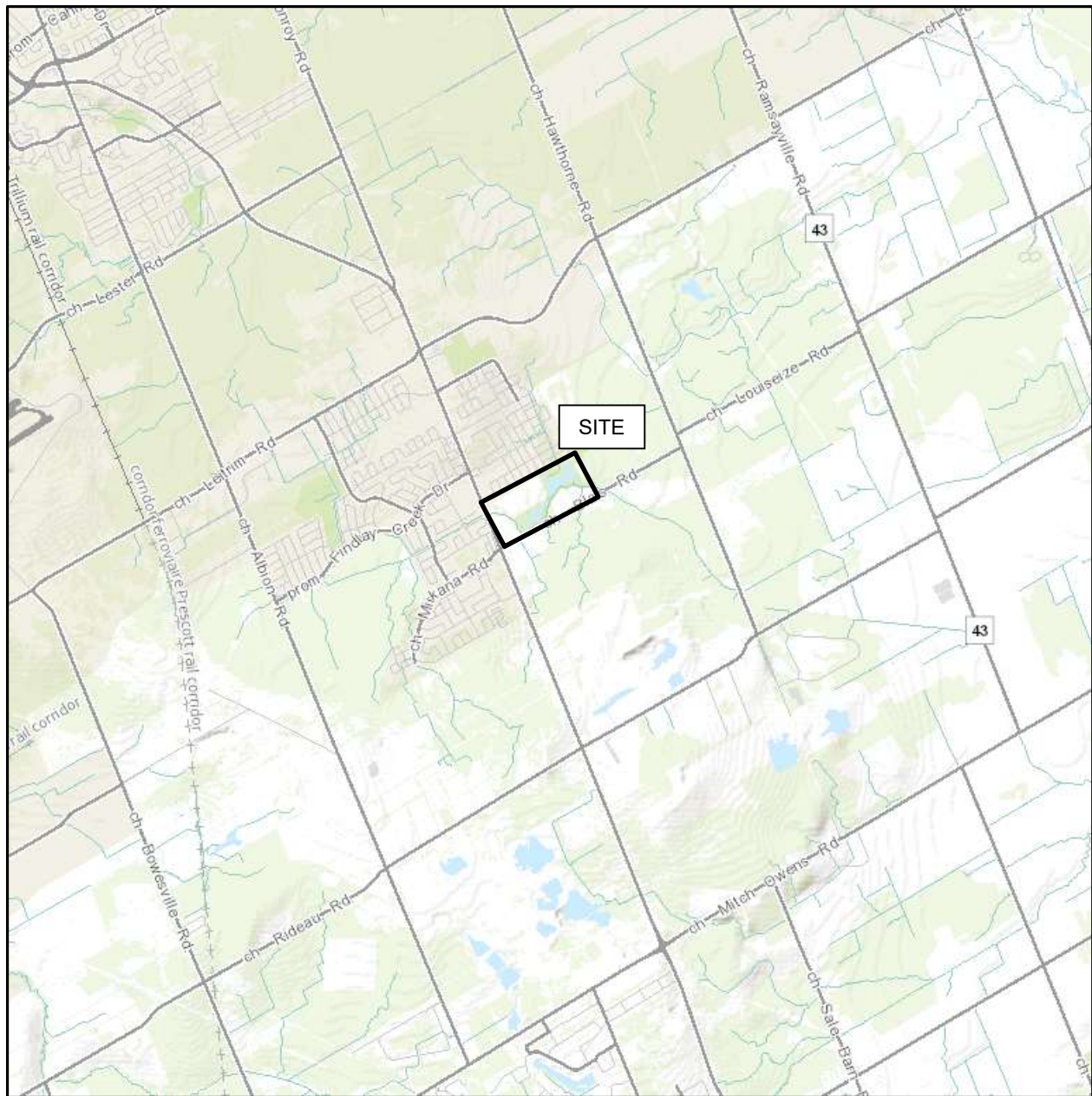
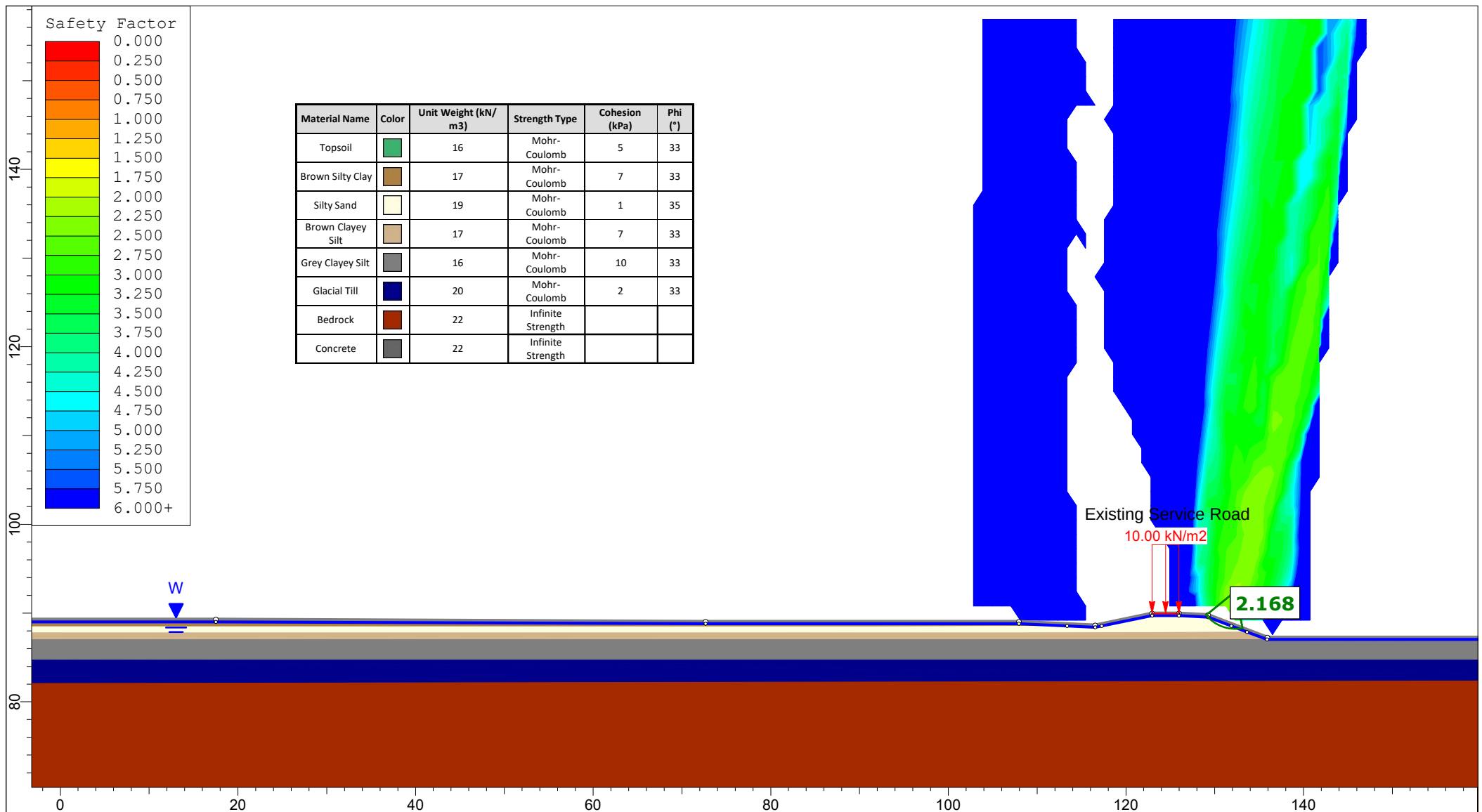
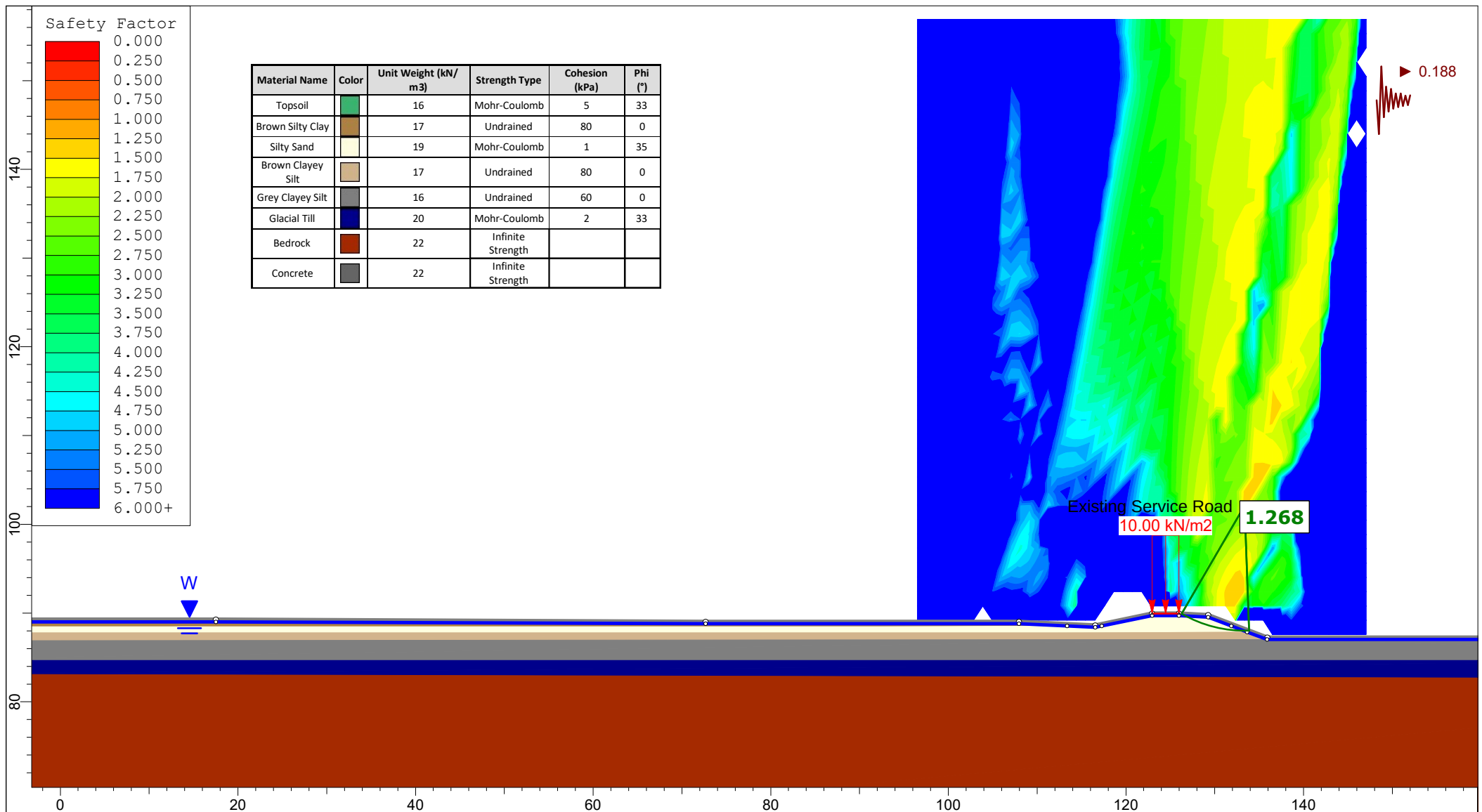


FIGURE 1

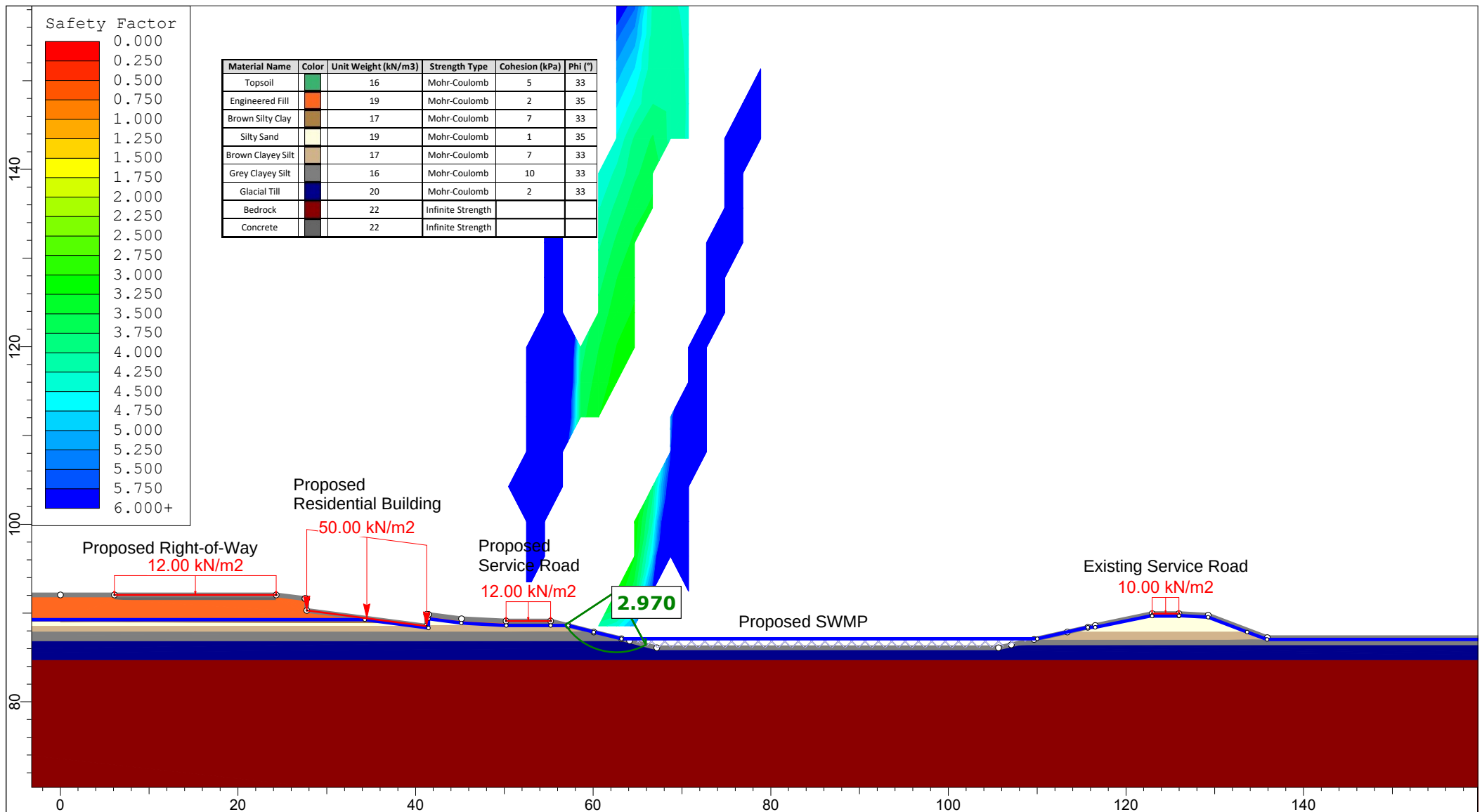
KEY PLAN



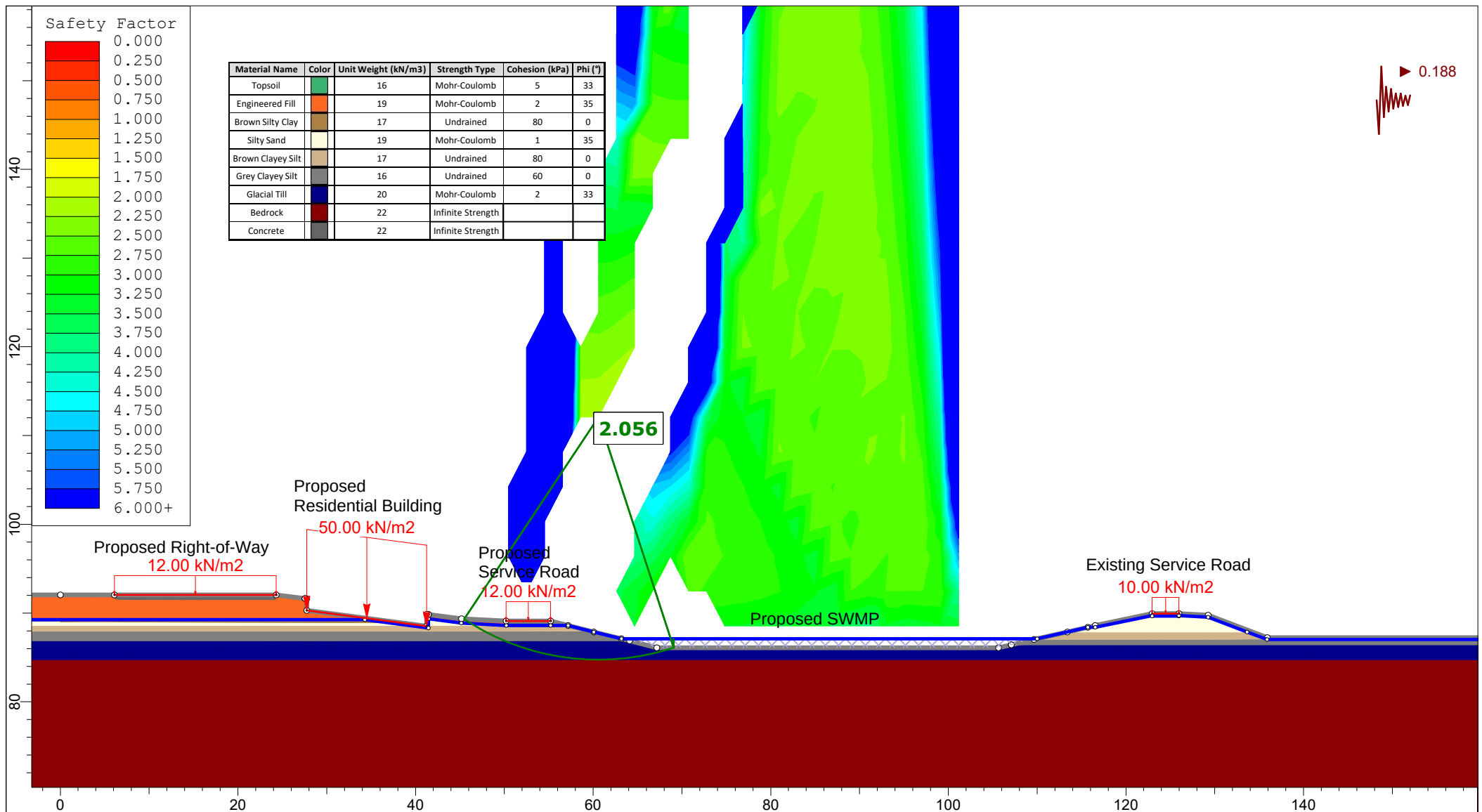
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	Drawing No.: Figure 2A - Section A-A - Existing Conditions - Static Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26



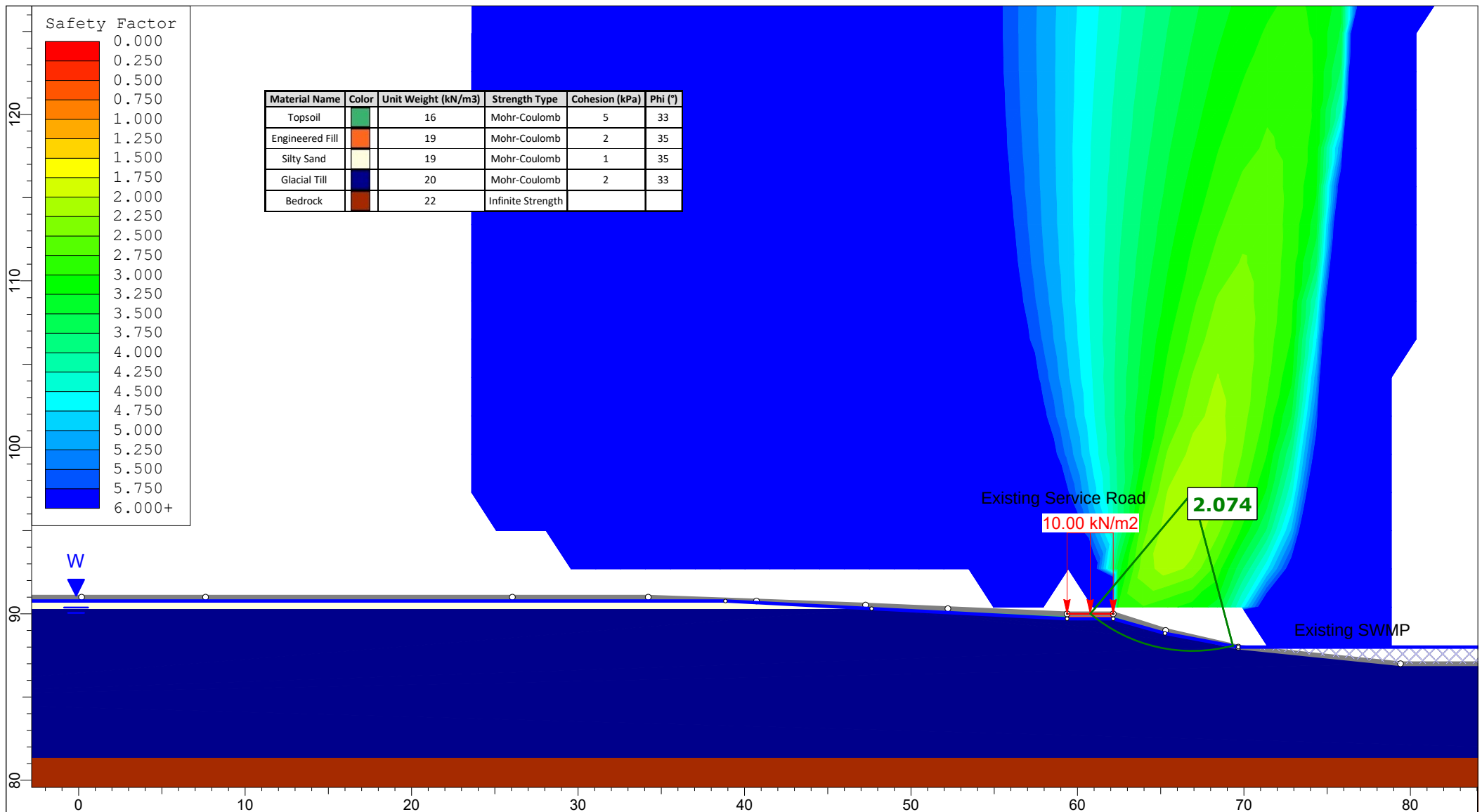
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	Drawing No.: Figure 2B - Section A-A - Existing Conditions - Seismic Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26



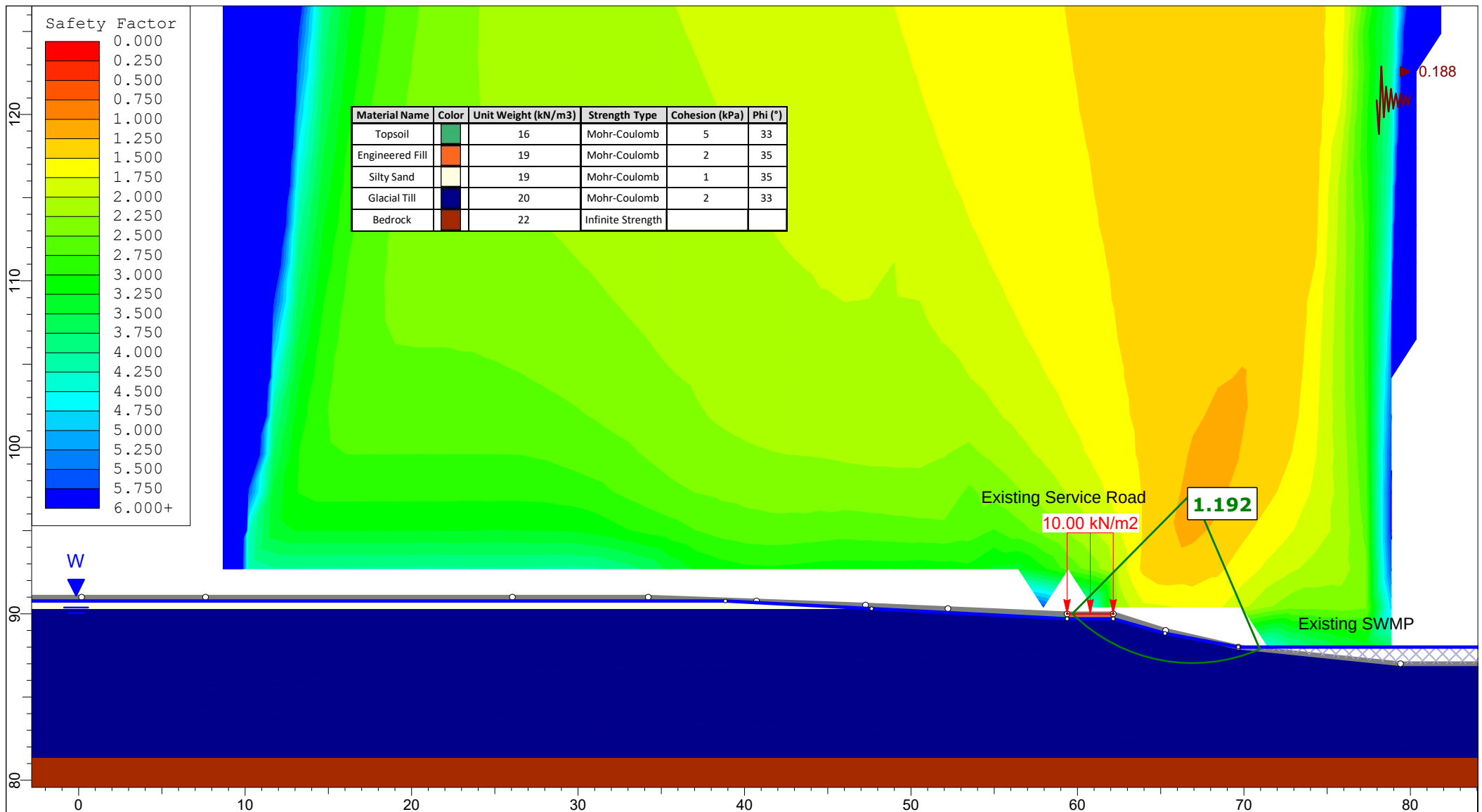
	Project No.: PG7301 - 4795 Bank Street, Ottawa, ON		
	Drawing No.: Figure 2C - Section A-A - Proposed Conditions - Static Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26



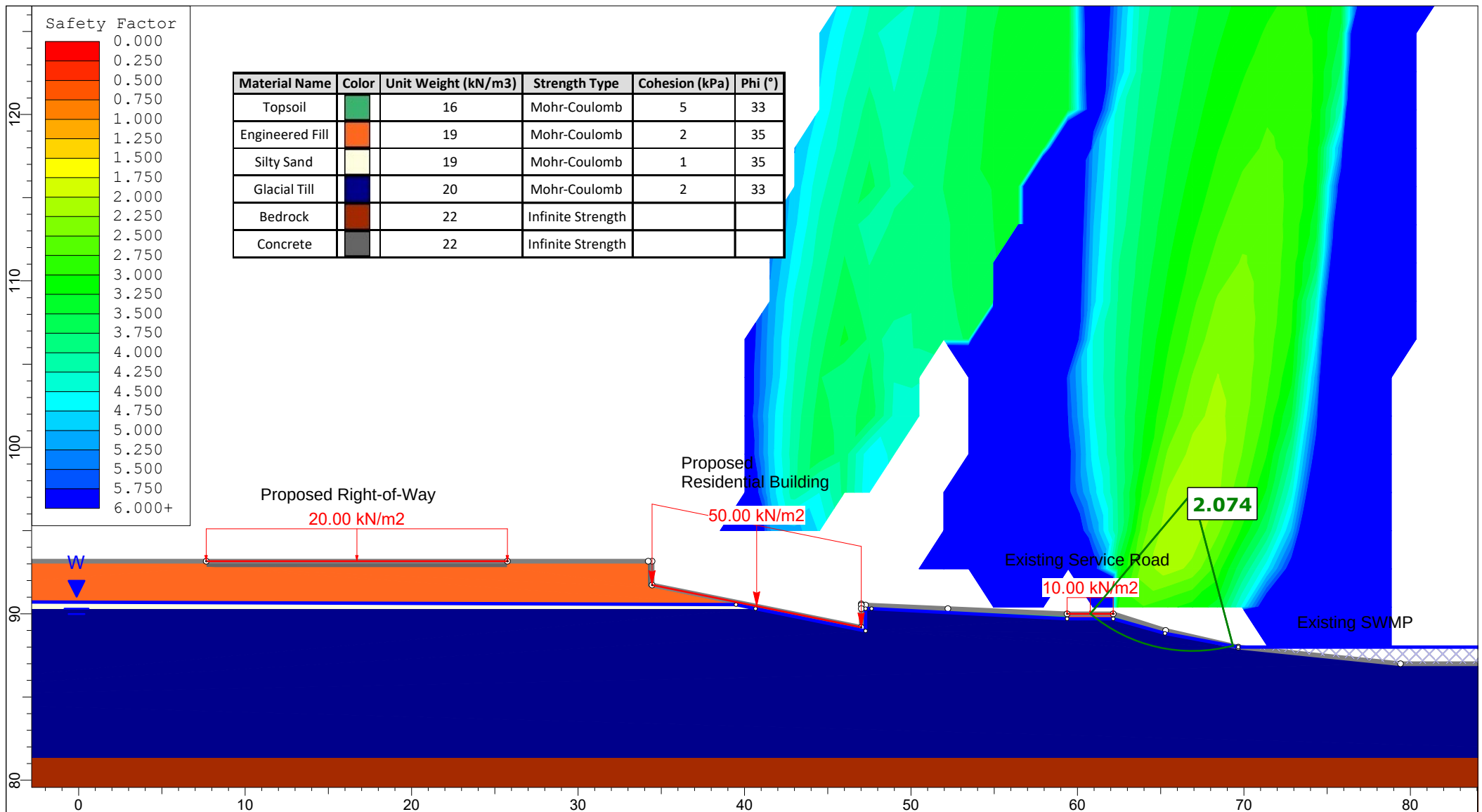
	Project No.: PG7301 - 4795 Bank Street, Ottawa, ON		
	Drawing No.: Figure 2D - Section A-A - Proposed Conditions - Seismic Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26



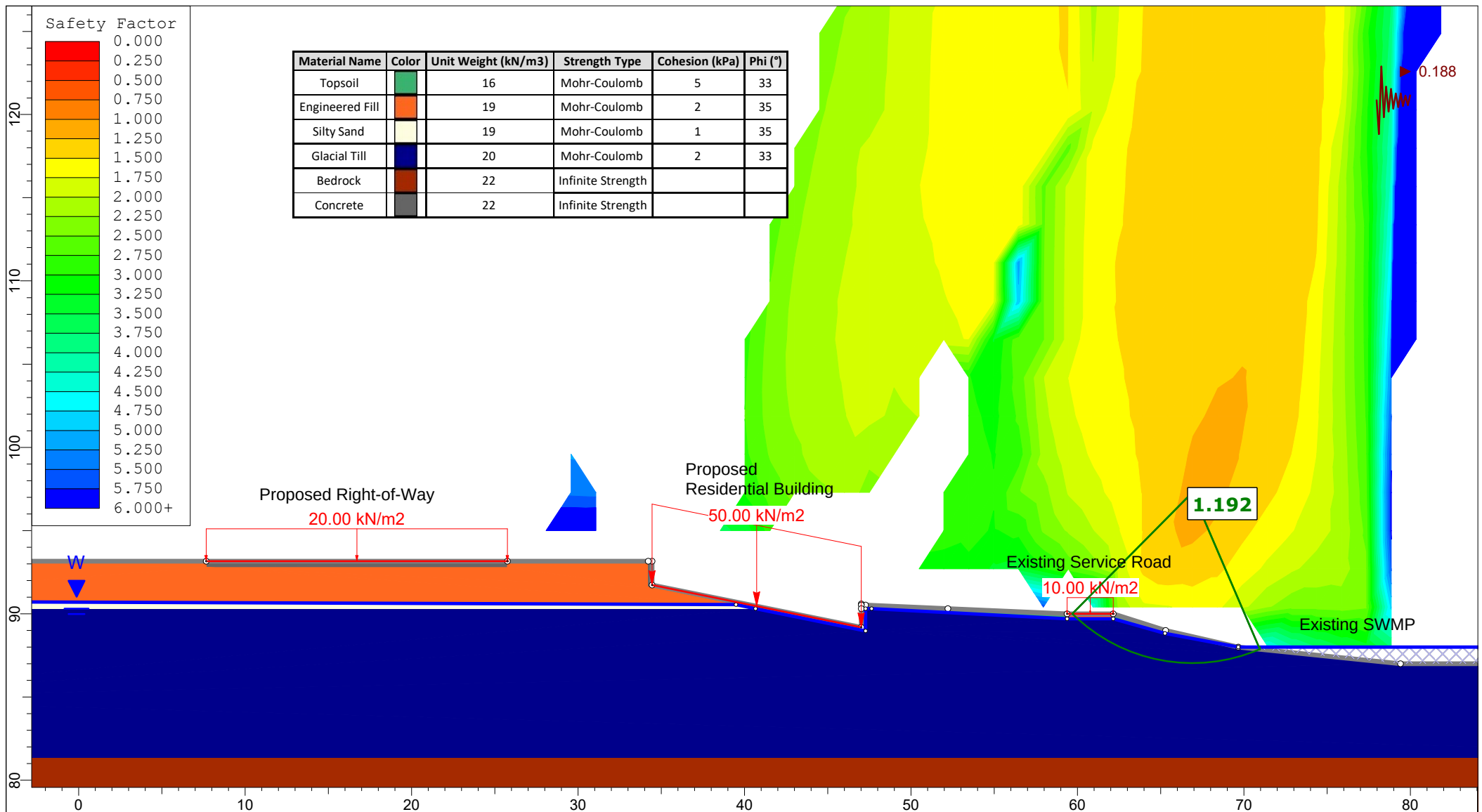
	Project No.: PG7301 - 4795 Bank Street, Ottawa, ON		
	Drawing No.: Figure 3A - Section B-B - Existing Conditions - Static Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26



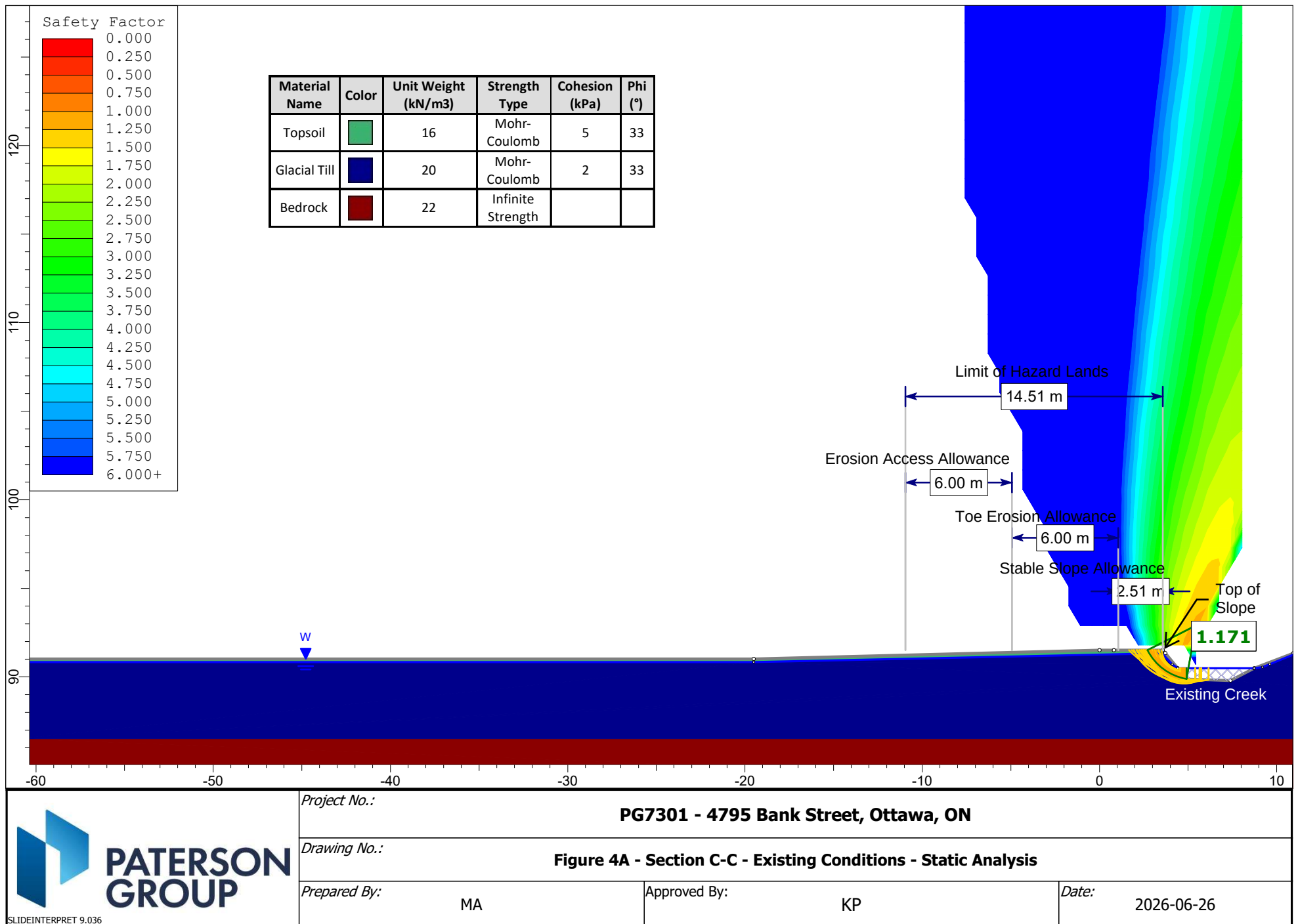
	Project No.: PG7301 - 4795 Bank Street, Ottawa, ON		
	Drawing No.: Figure 3B - Section B-B - Existing Conditions - Seismic Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26

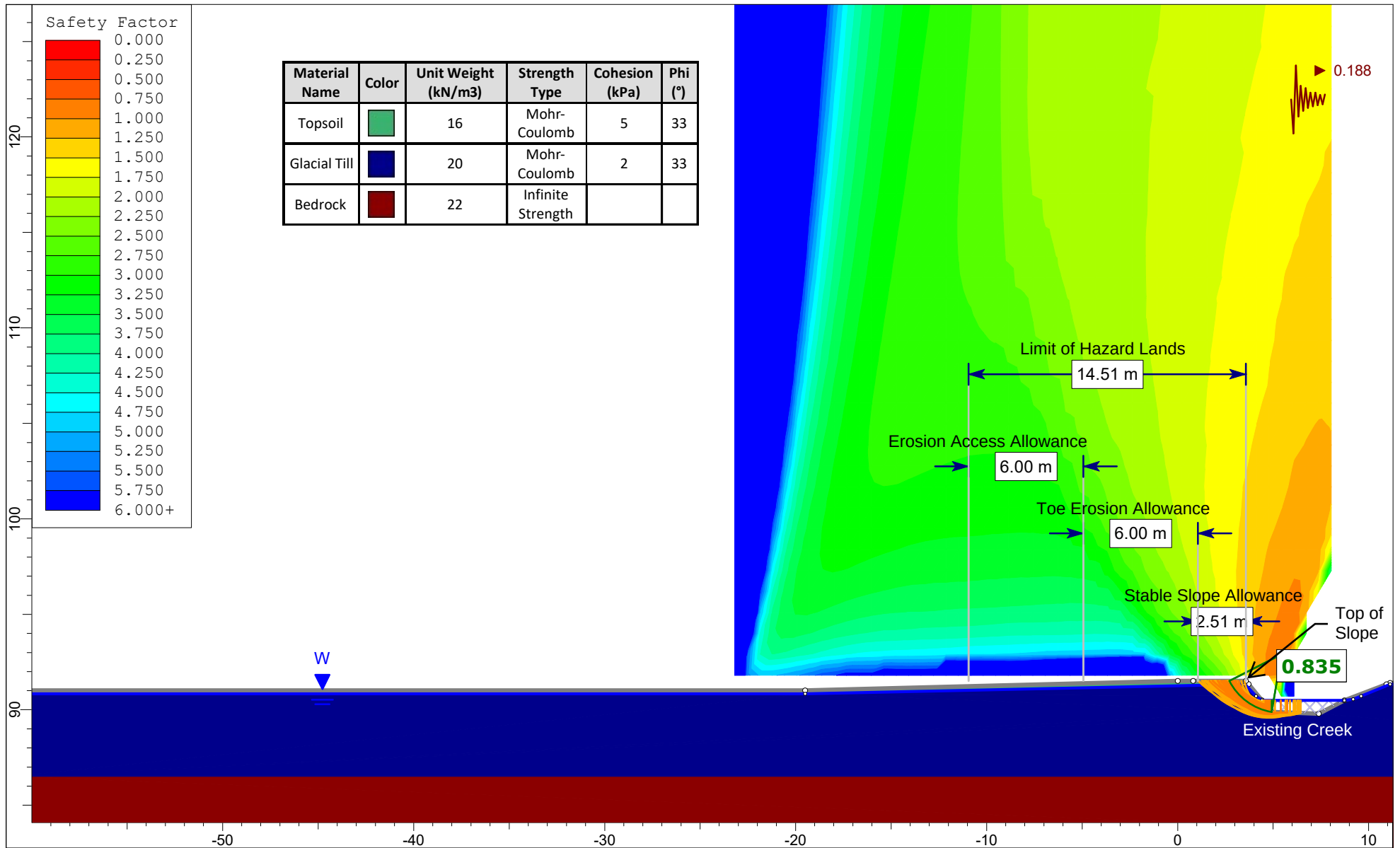


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	Drawing No.: Figure 3C - Section B-B - Proposed Conditions - Static Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26

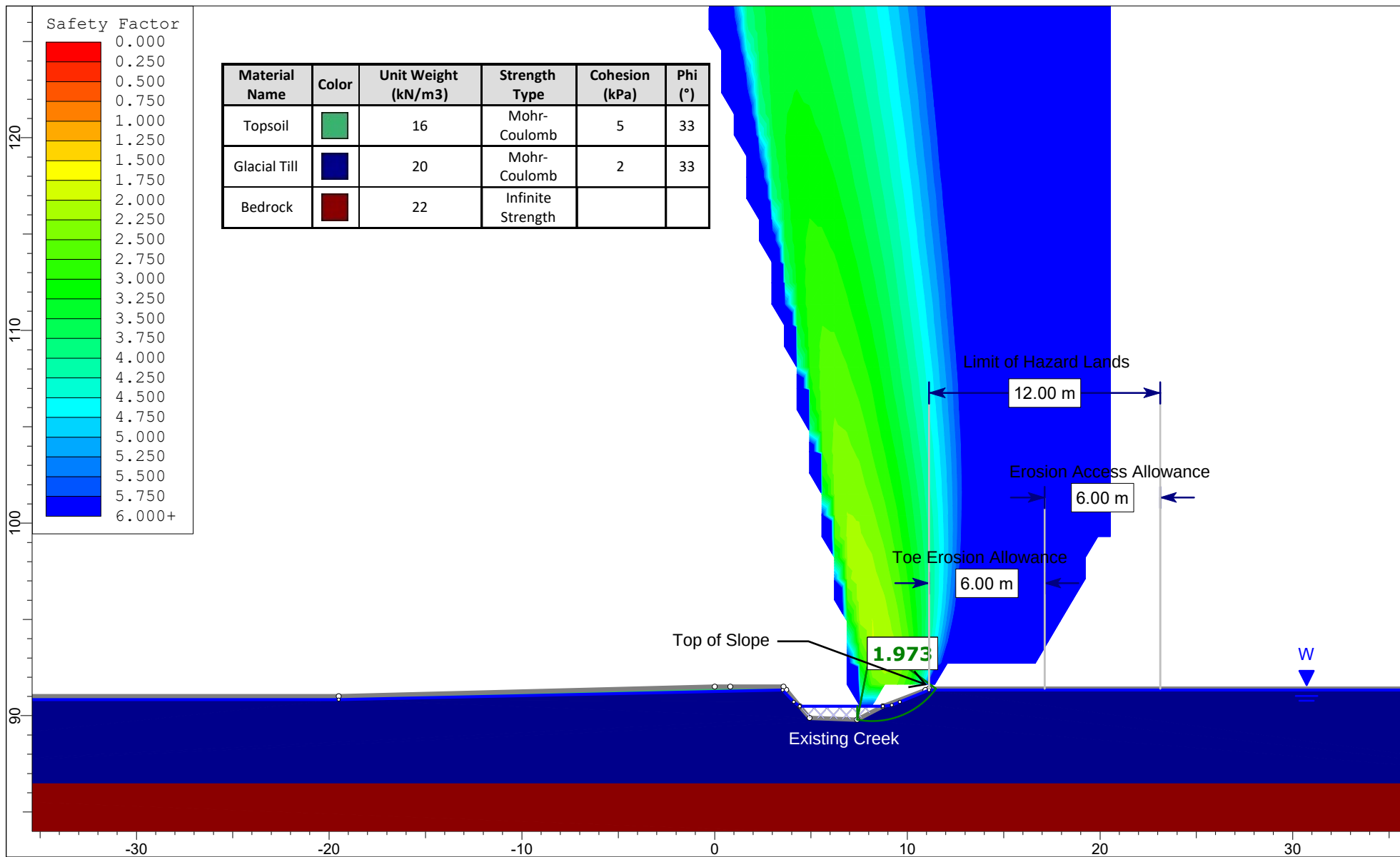


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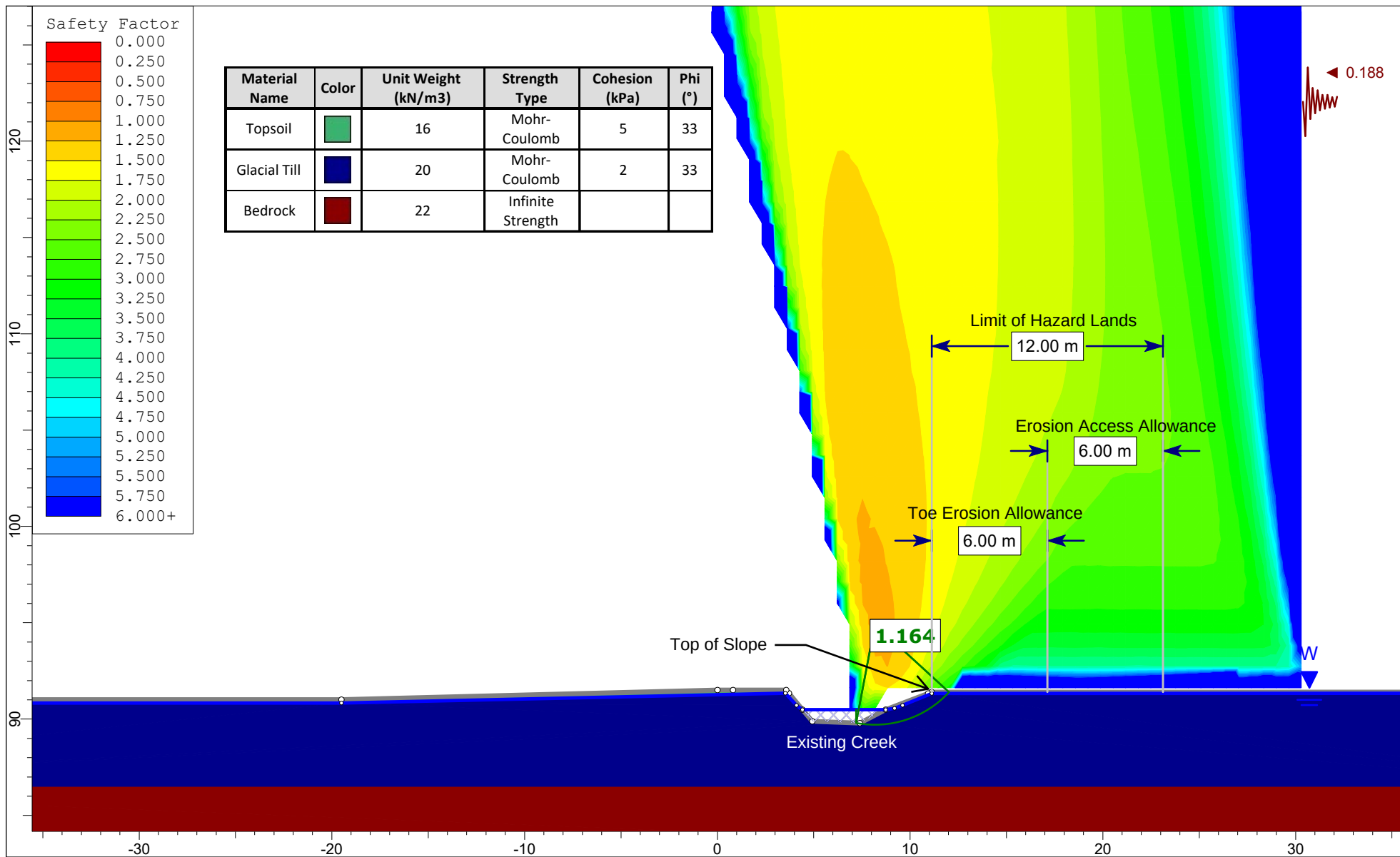




	Project No.: PG7301 - 4795 Bank Street, Ottawa, ON		
	Drawing No.: Figure 4B - Section C-C - Existing Conditions - Seismic Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26

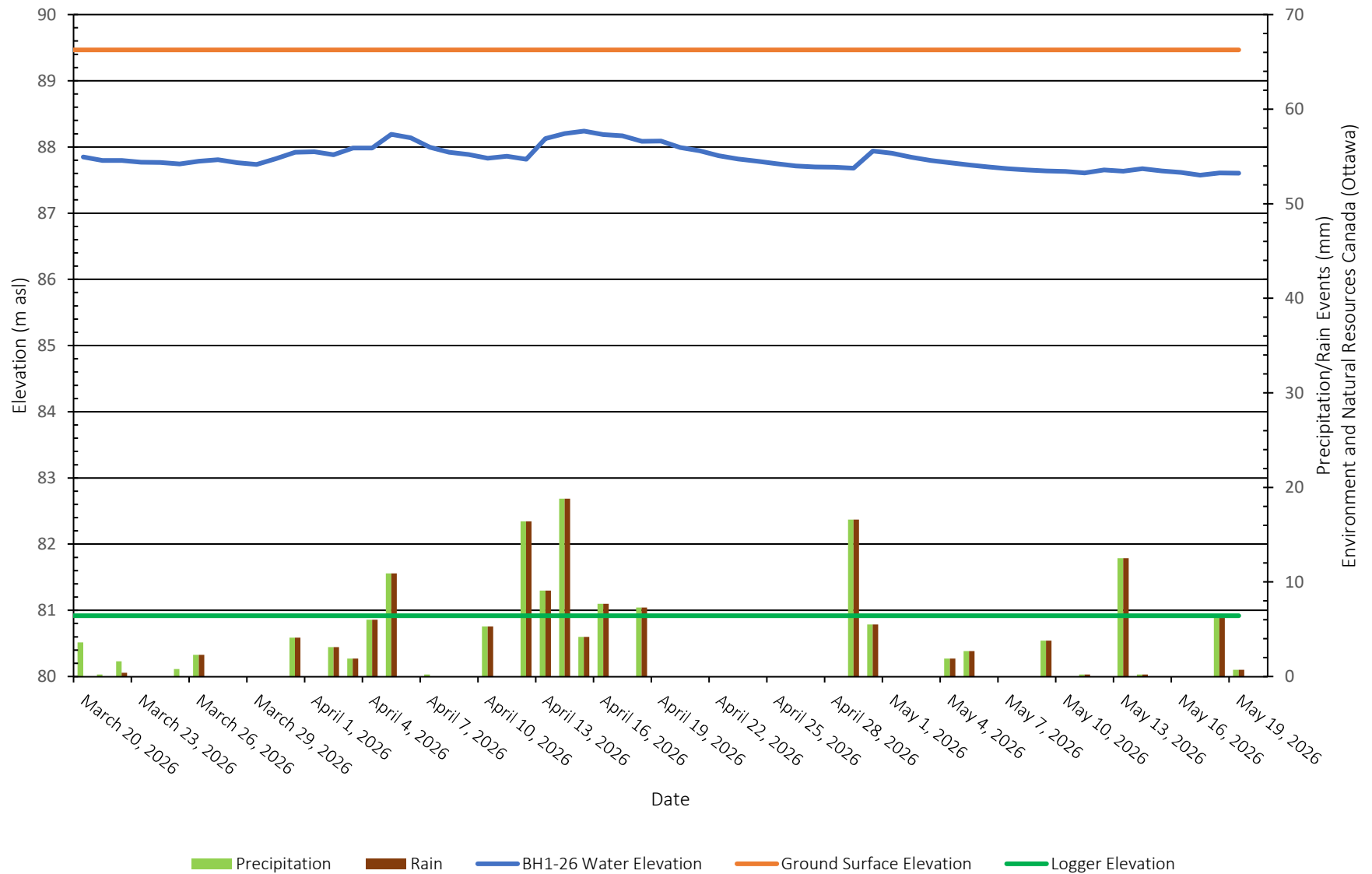


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	Drawing No.: Figure 5A - Section C-C - Existing Conditions - Static Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26

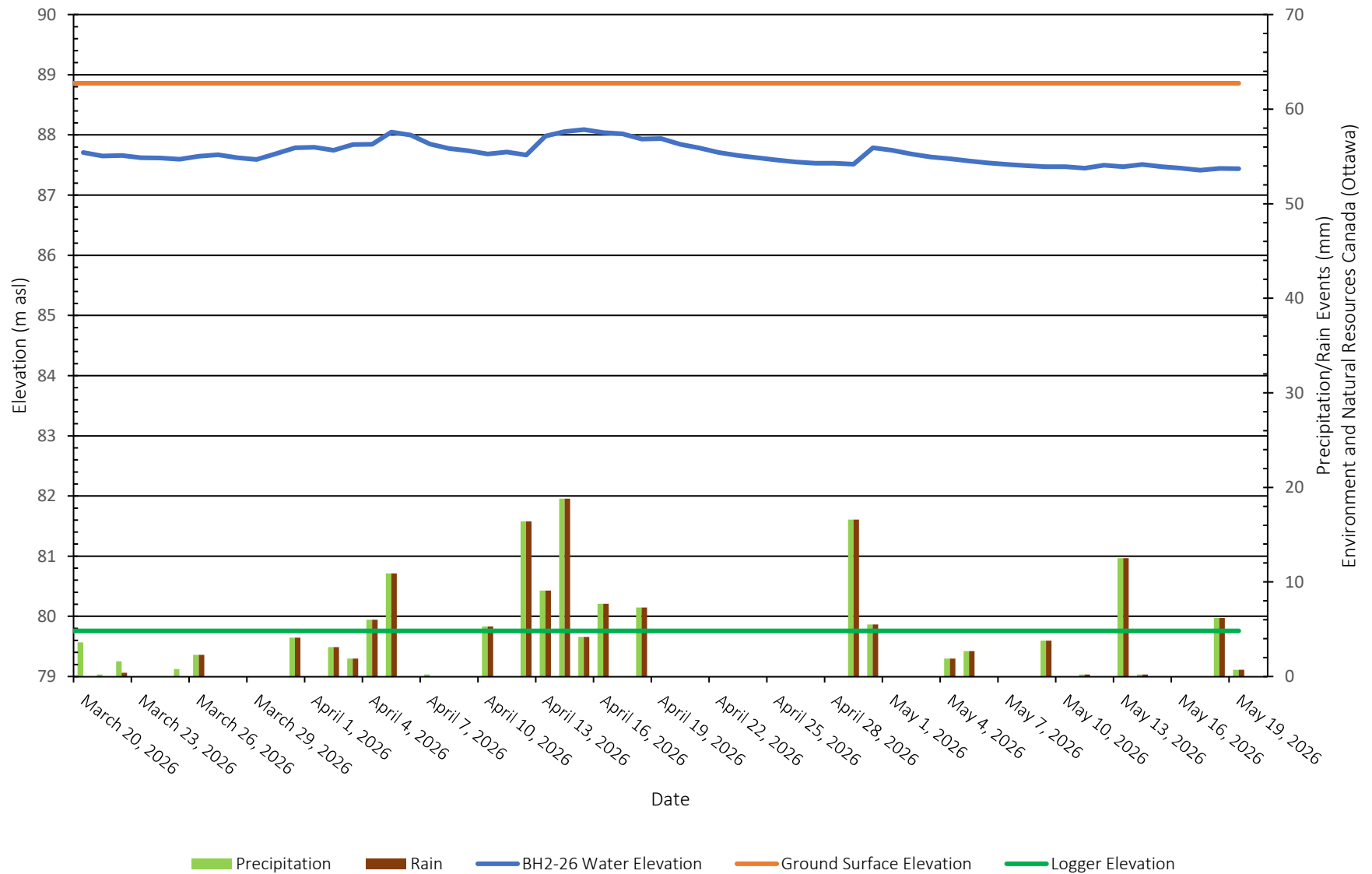


	Project No.: PG7301 - 4795 Bank Street, Ottawa, ON		
	Drawing No.: Figure 5B - Section C-C - Existing Conditions - Seismic Analysis		
	Prepared By: MA	Approved By: KP	Date: 2026-06-26

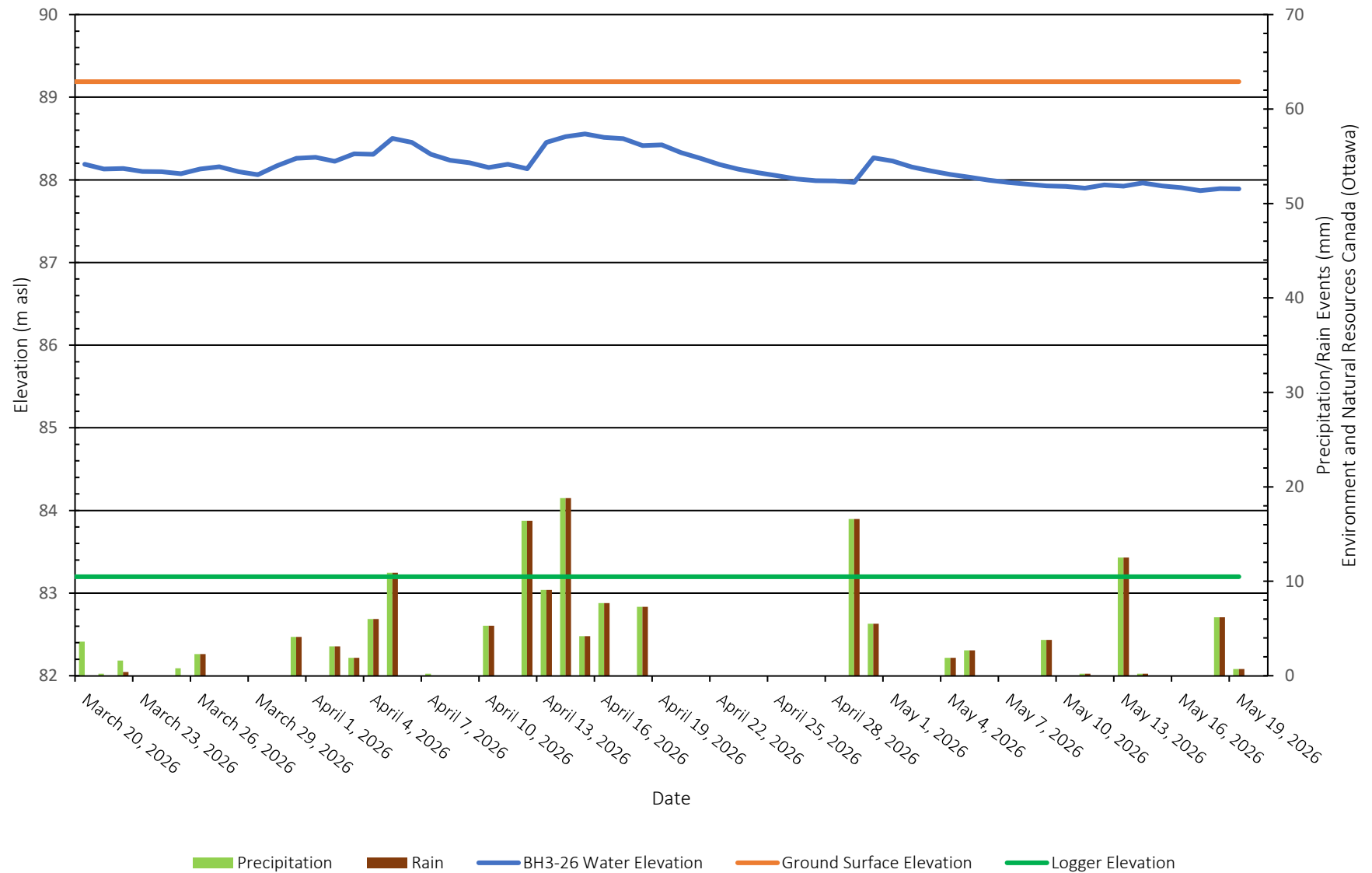
BH1-26 - Monitoring Well Water Elevations



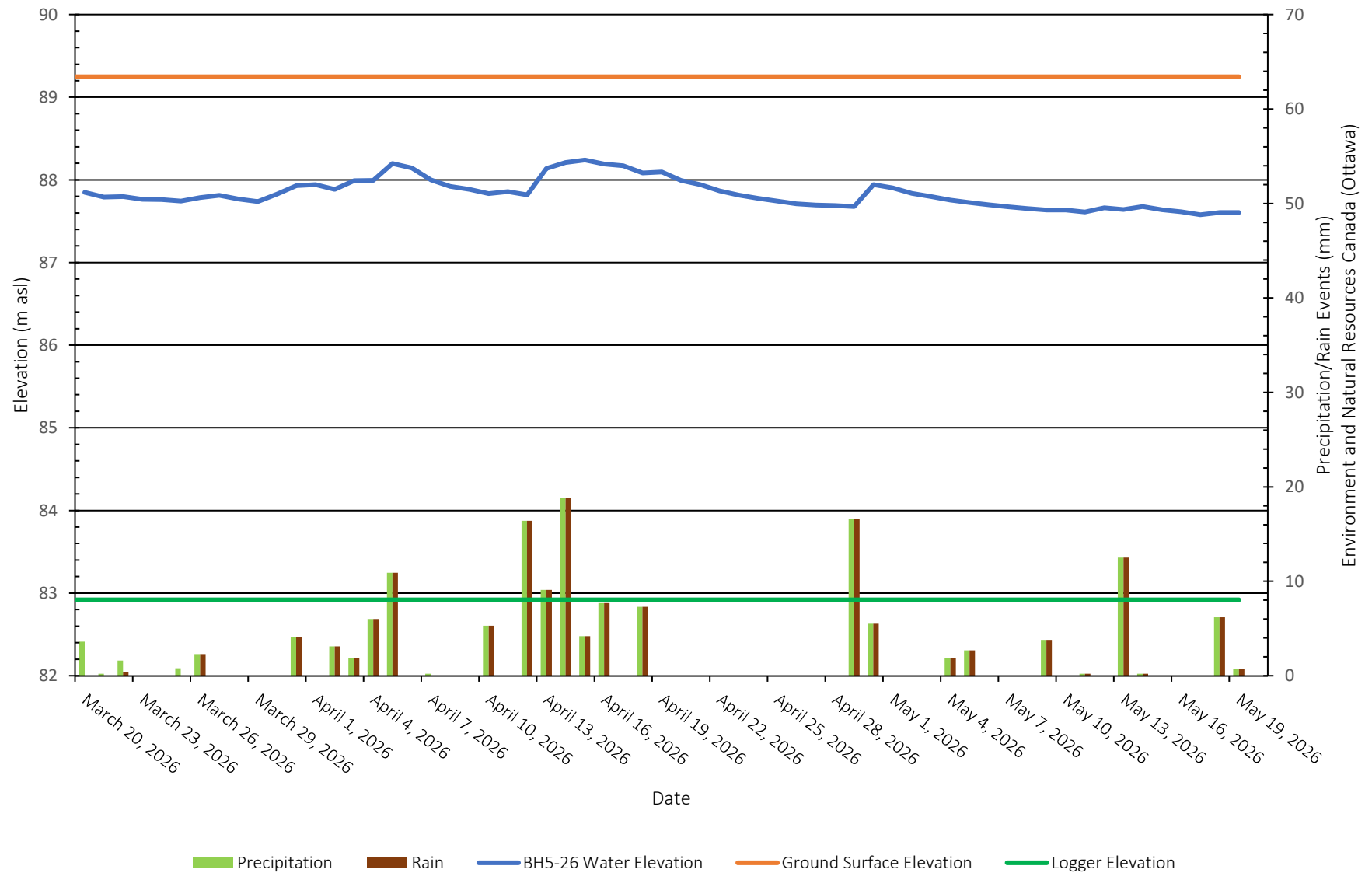
BH2-26 - Monitoring Well Water Elevations



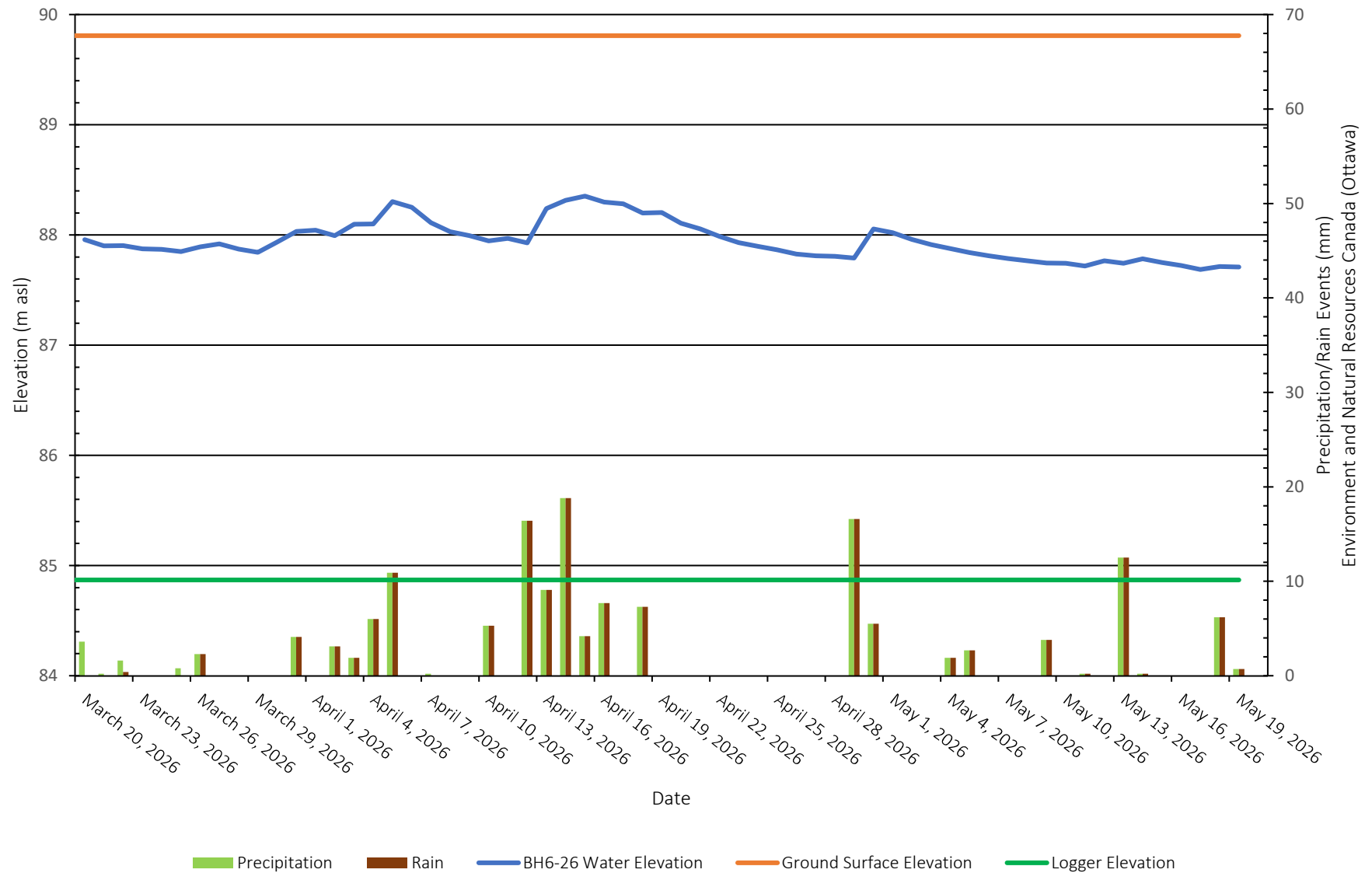
BH3-26 - Monitoring Well Water Elevations



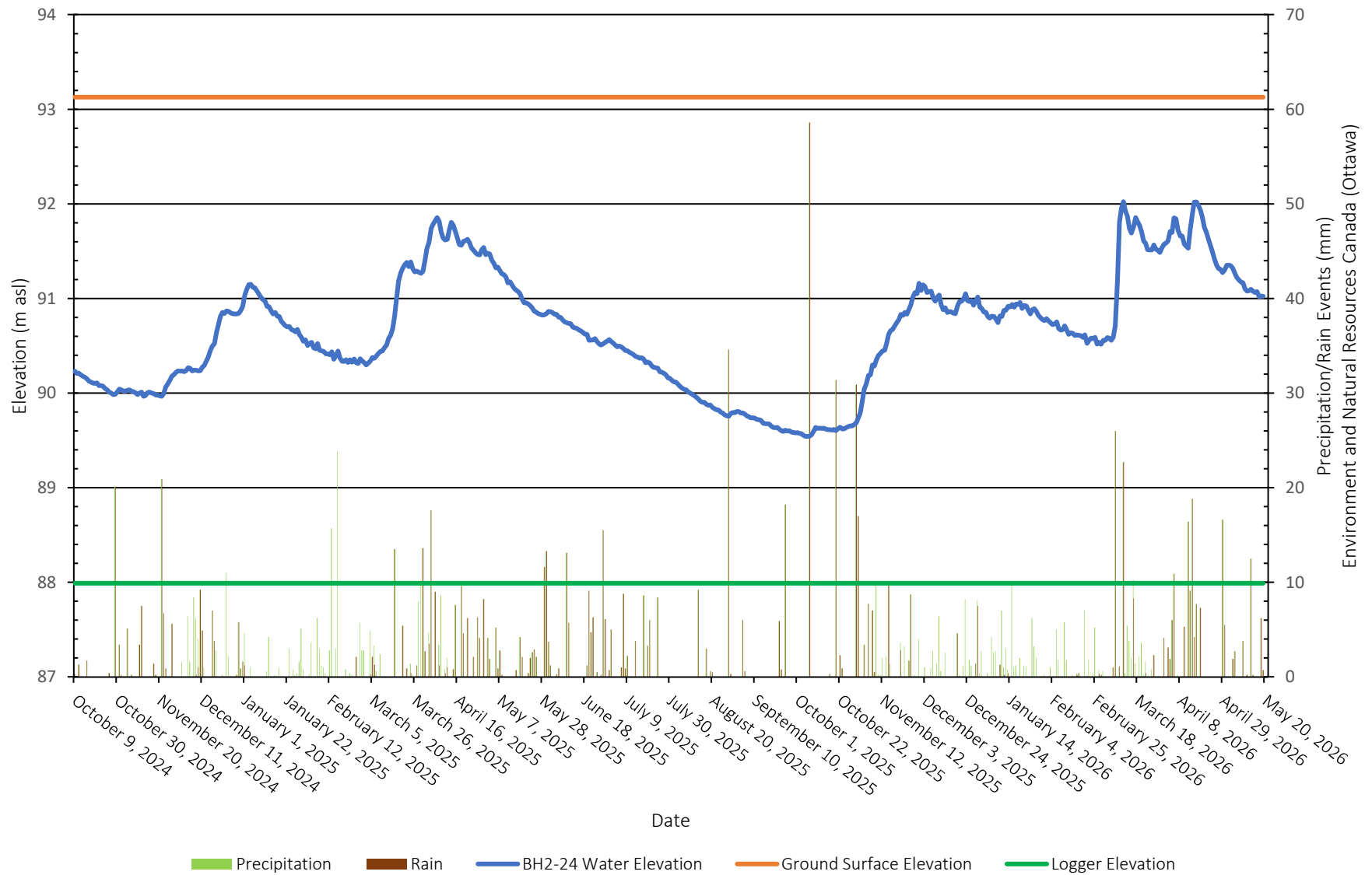
BH5-26 - Monitoring Well Water Elevations



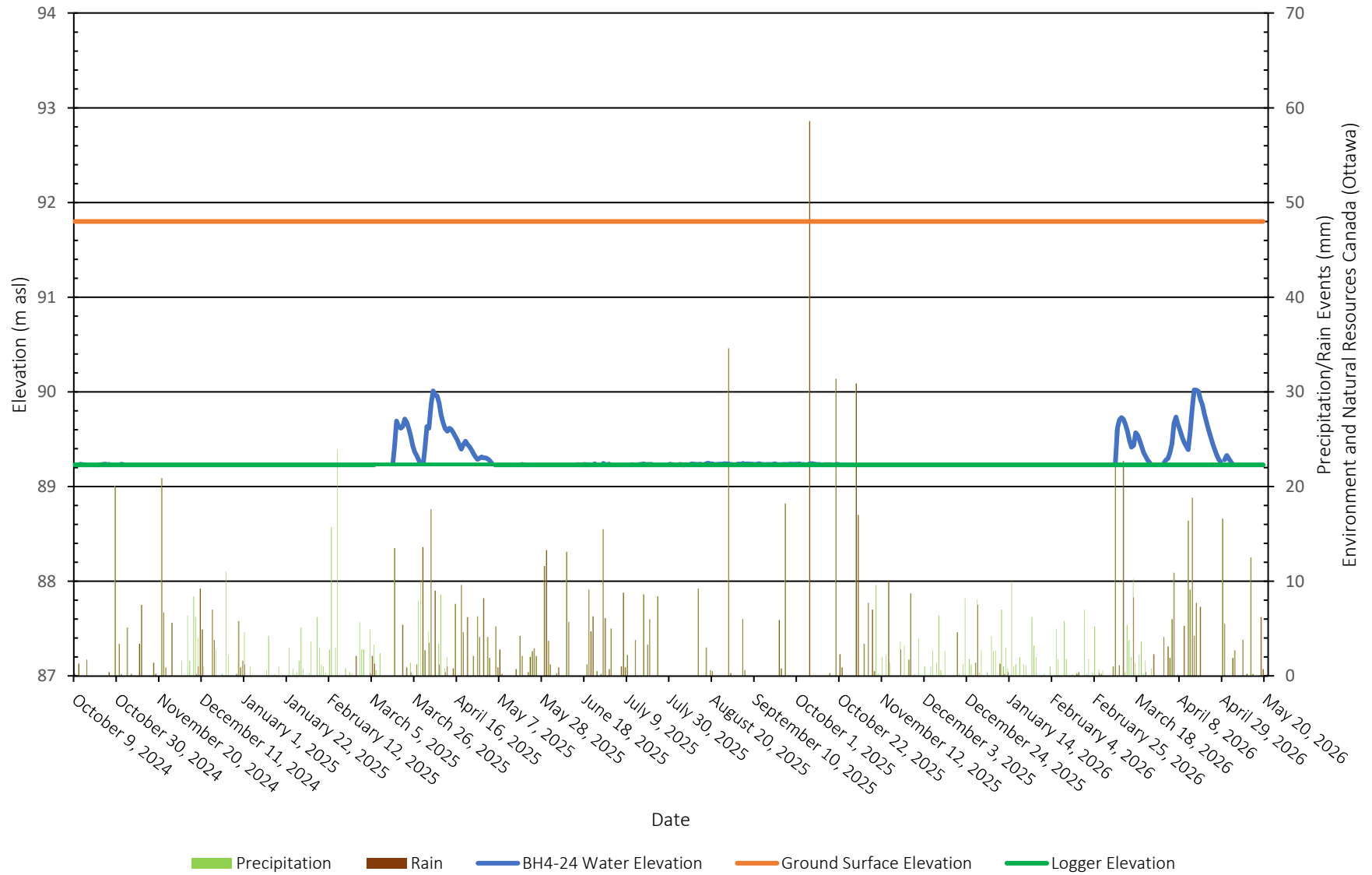
BH6-26 - Monitoring Well Water Elevations



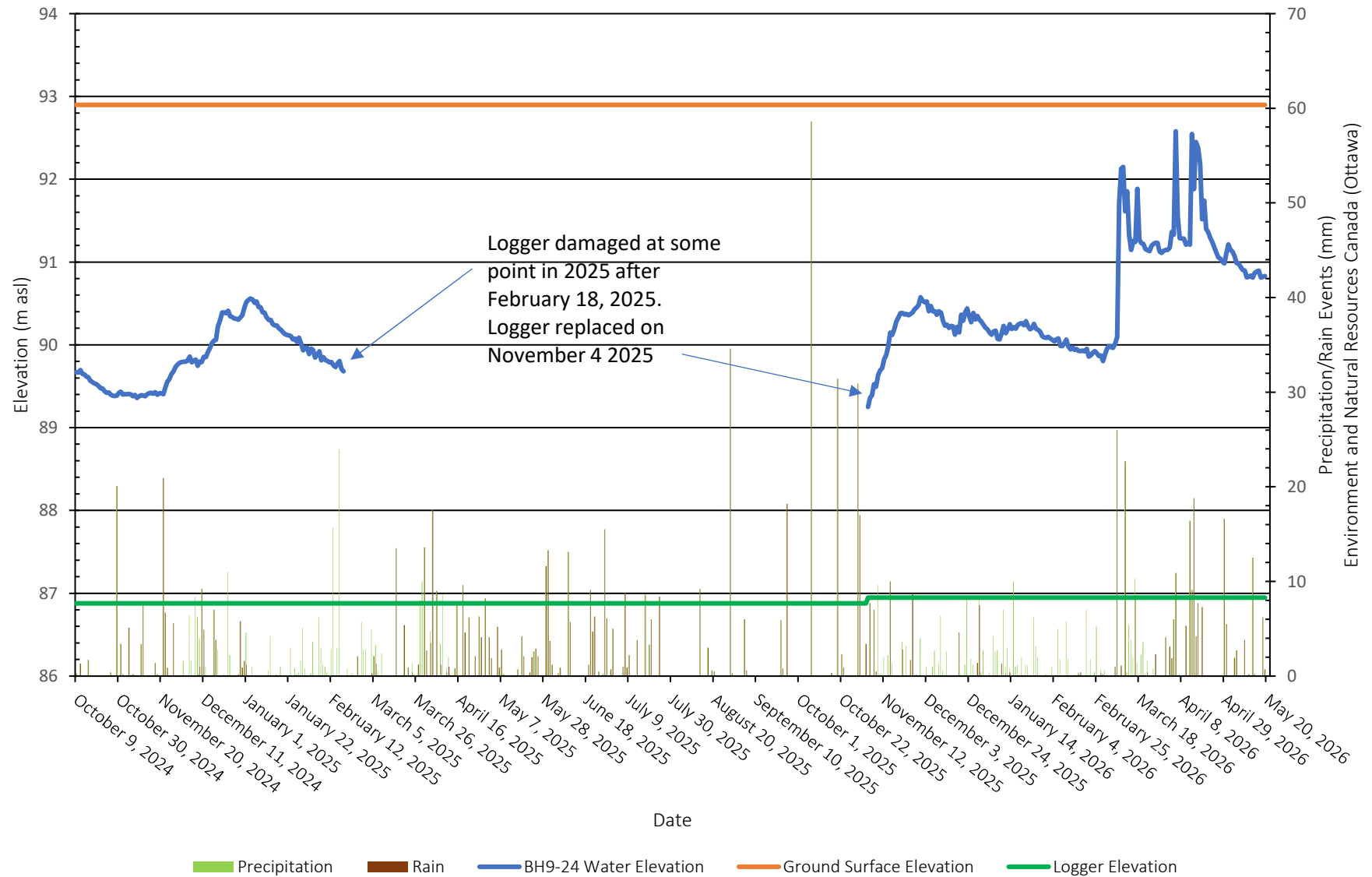
BH2-24 - Monitoring Well Water Elevations



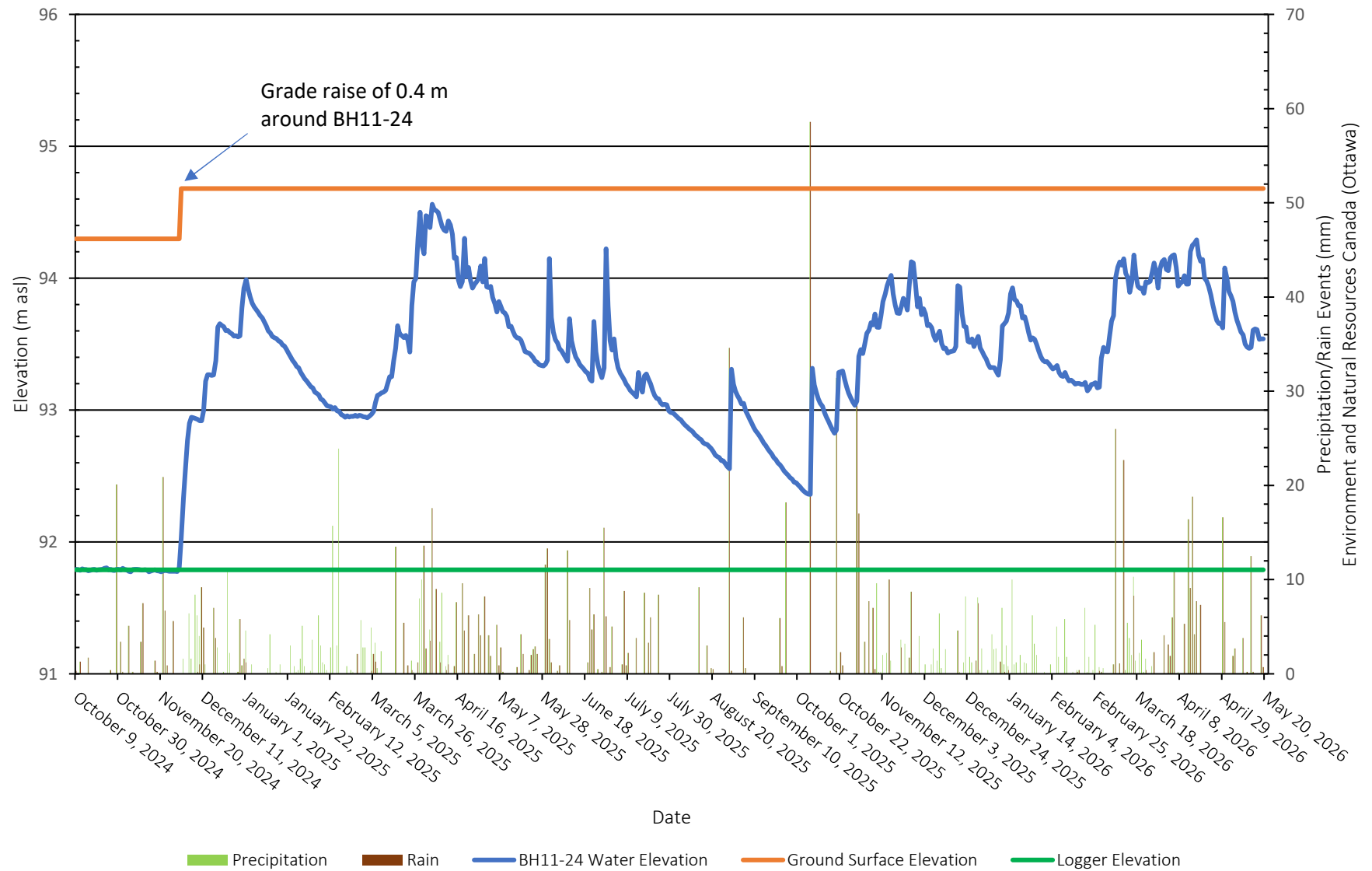
BH4-24 - Monitoring Well Water Elevations



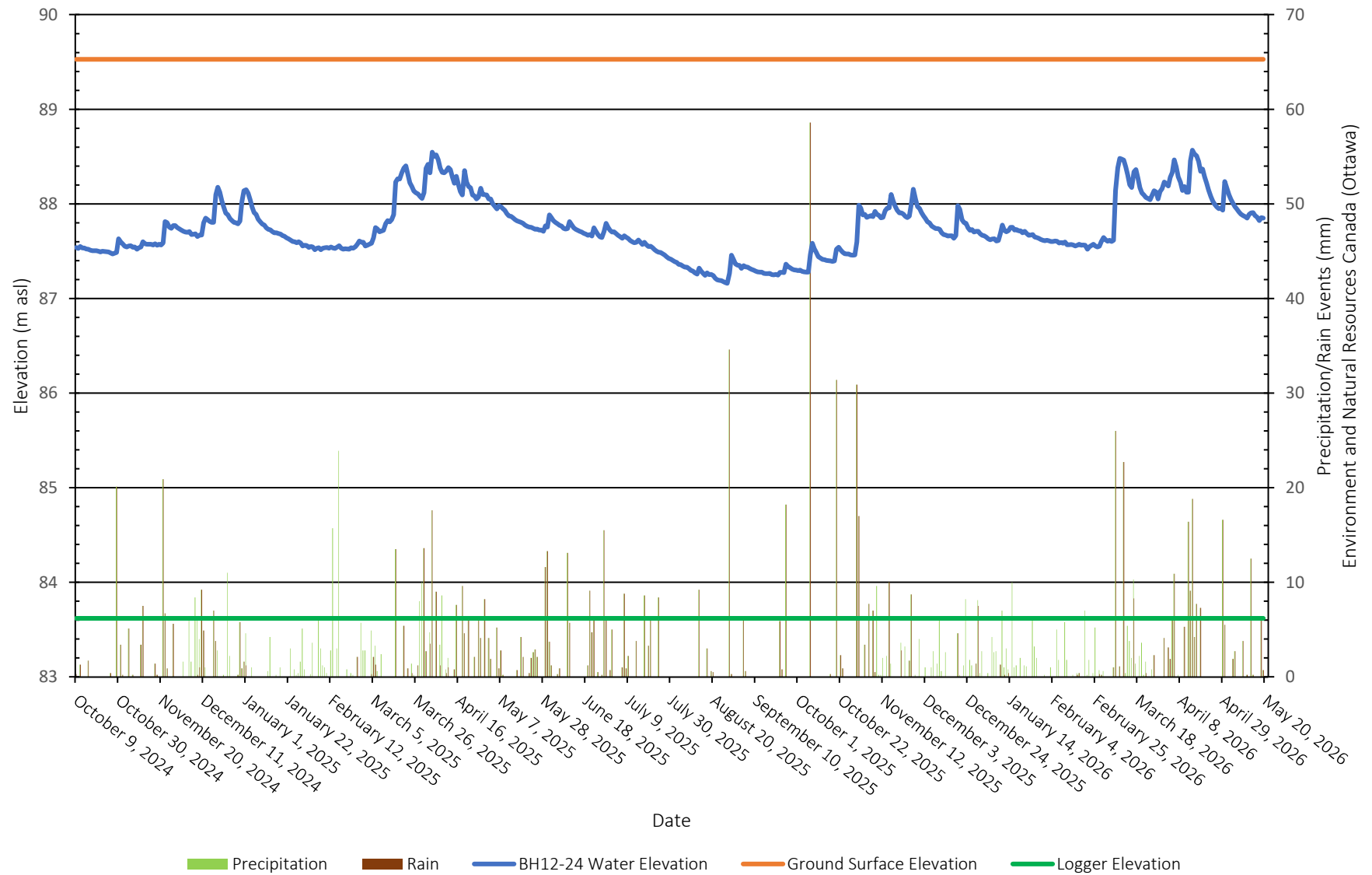
BH9-24 - Monitoring Well Water Elevations



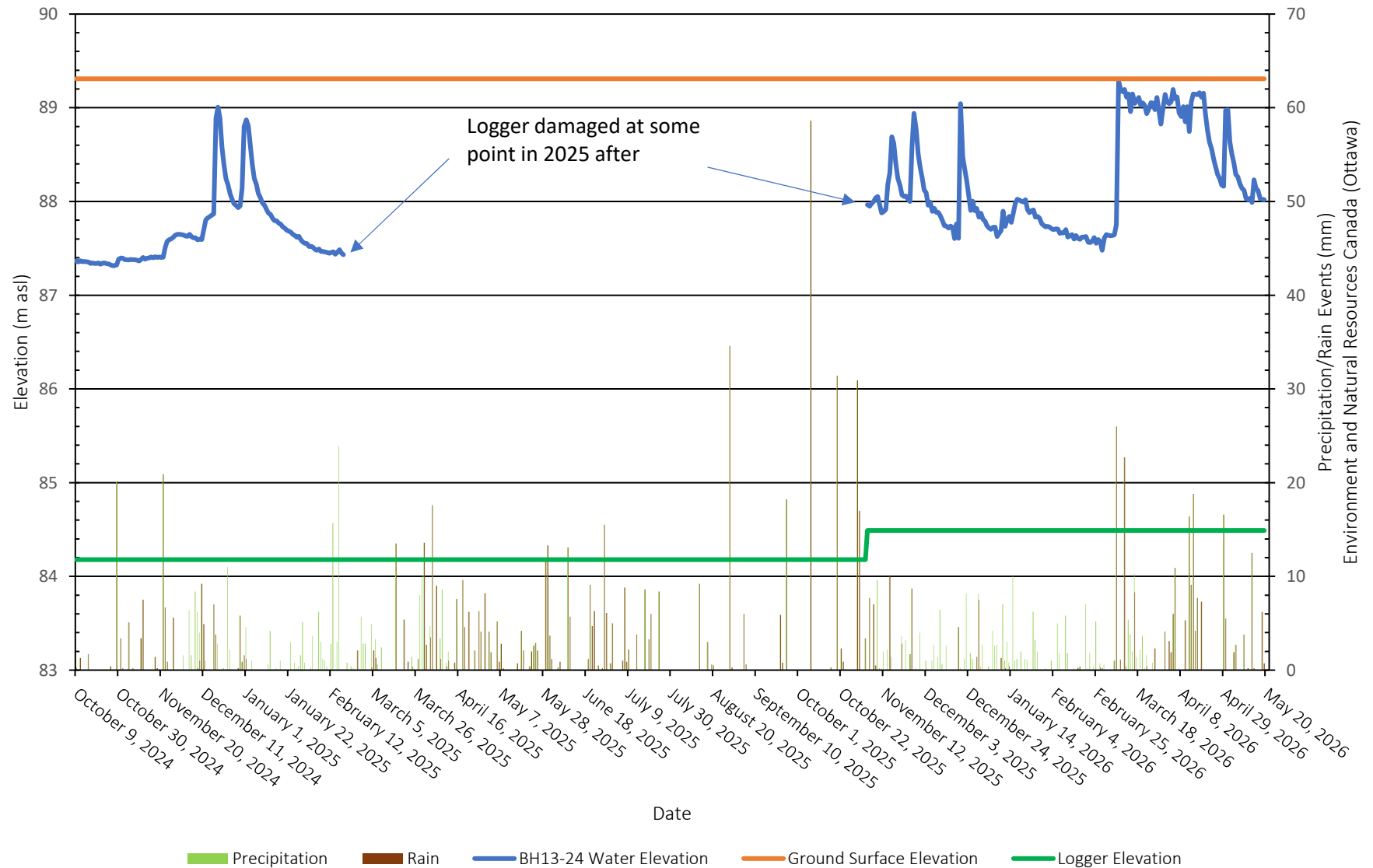
BH11-24 - Monitoring Well Water Elevations



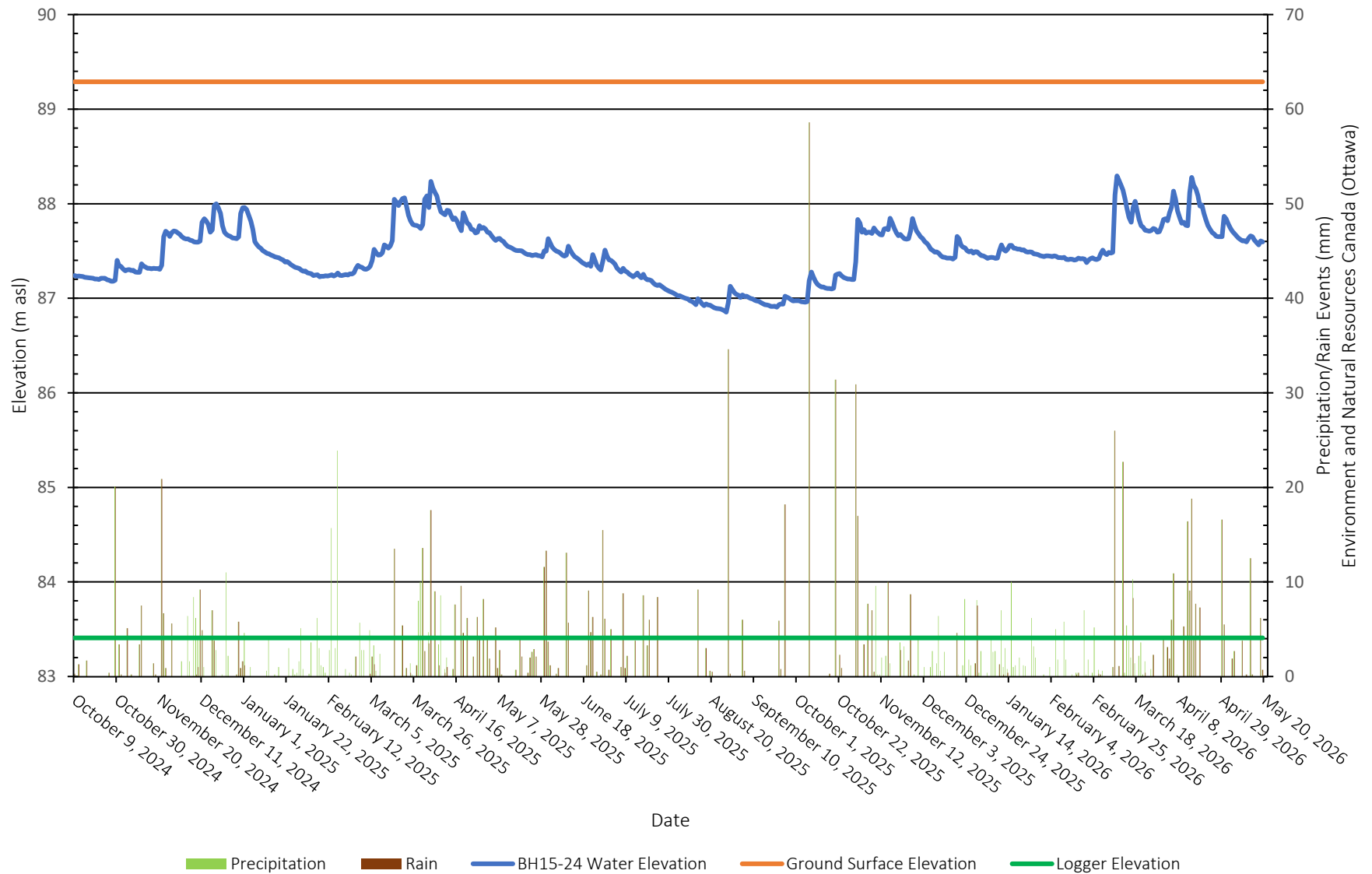
BH12-24 - Monitoring Well Water Elevations



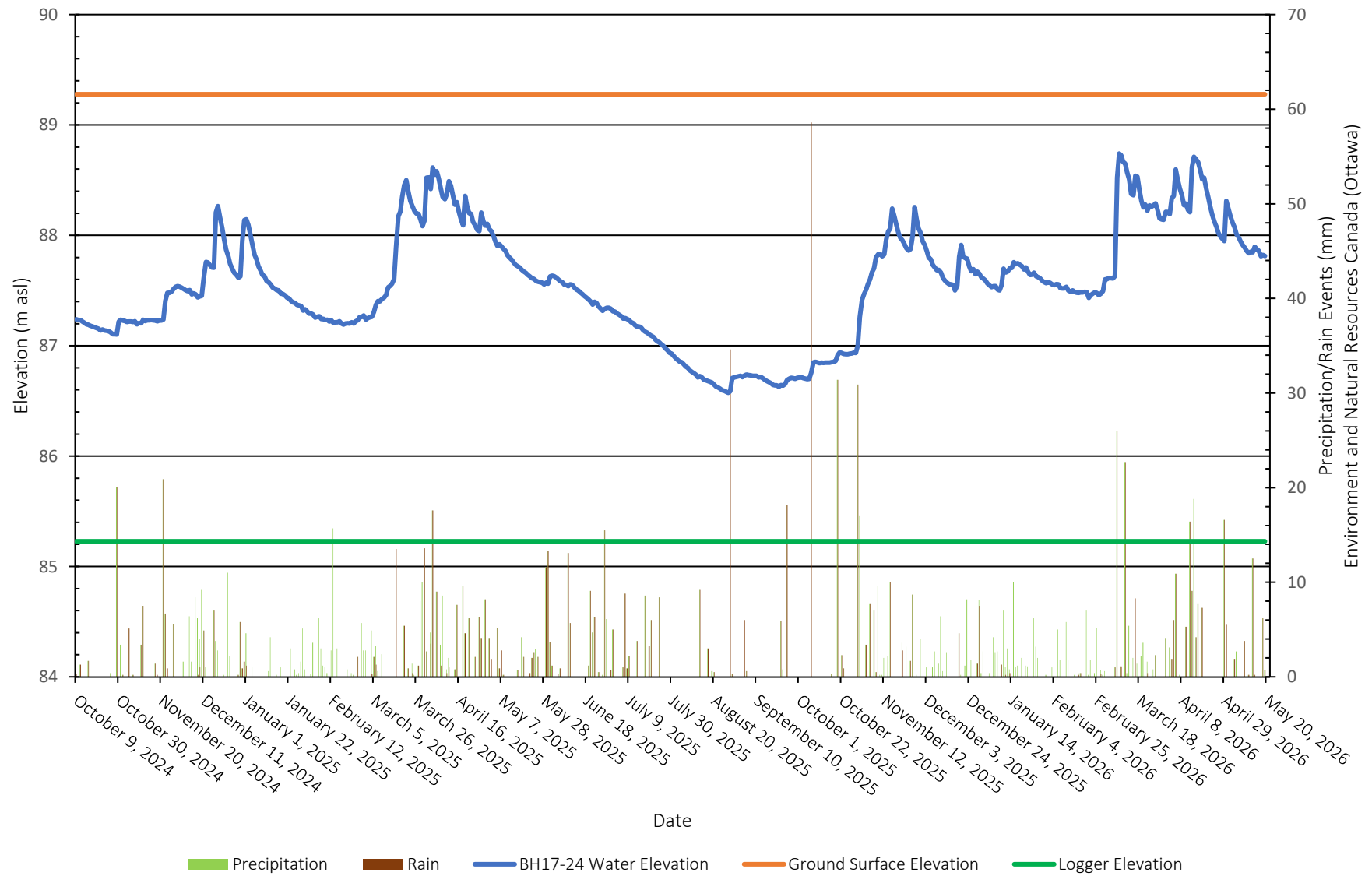
BH13-24 - Monitoring Well Water Elevations

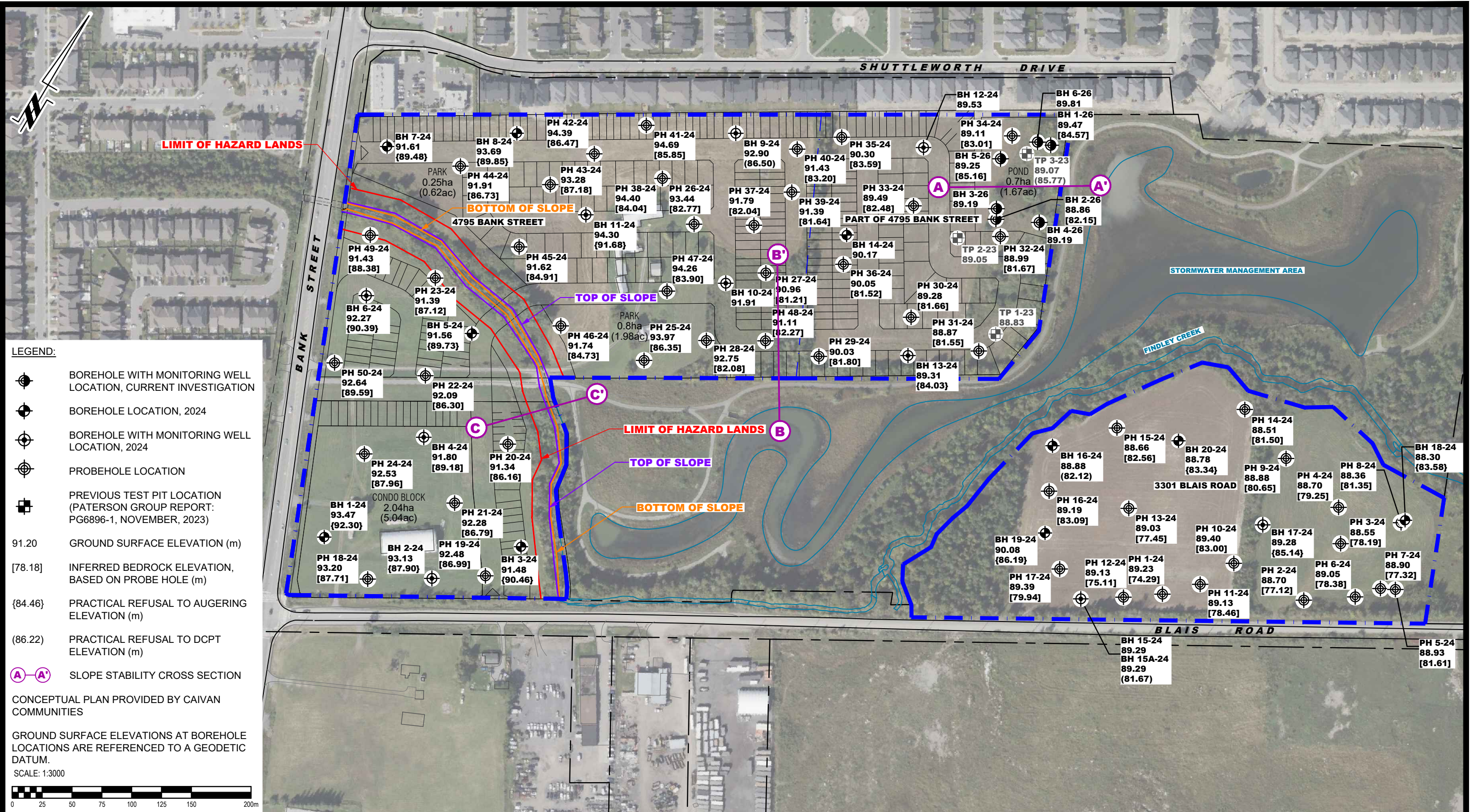


BH15-24 - Monitoring Well Water Elevations



BH17-24 - Monitoring Well Water Elevations







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

2	ADDED CONCEPTUAL PLAN AND SLOPE STABILITY CROSS-SECTIONS	07/03/2026	KP
1	ADDED 2026 BOREHOLE LOCATION BH 1-26 TO BH 6-26	27/05/2026	KP
NO.	REVISIONS	DD/MM/YYYY	INITIAL

CAIVAN COMMUNITIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4795 BANK STREET & 3301 BLAIS ROAD

ONTARIO

OTTAWA,
Title:

TEST HOLE LOCATION PLAN

Scale:	1:3000	Date:	10/2024
Drawn by:	ZS	Report No.:	PG7301-1
Checked by:	MA	Dwg. No.:	PG7301-1
Approved by:	KP	Revision No.:	2



LEGEND:

- BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
- BOREHOLE LOCATION, 2024
- BOREHOLE WITH MONITORING WELL LOCATION, 2024
- PROBEHOLE LOCATION
- PREVIOUS TEST PIT LOCATION (PATERSON GROUP REPORT: PG6896-1, NOVEMBER, 2023)

91.20 GROUND SURFACE ELEVATION (m)

[78.18] INFERRED BEDROCK ELEVATION, BASED ON PROBE HOLE (m)

{84.46} PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

(86.22) PRACTICAL REFUSAL TO DCPT ELEVATION (m)

—85.00— BEDROCK CONTOURS (m)

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

SCALE: 1:3000

0 25 50 75 100 125 150 200m

PATERSON GROUP 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381 <table><tr><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td></tr><tr><td>1</td><td>ADDED 2026 BOREHOLE LOCATION BH 1-26 TO BH 6-26 AND UPDATED BEDROCK CONTOURS</td><td>27/05/2026</td><td>KP</td></tr><tr><td>NO.</td><td>REVISIONS</td><td>DD/MM/YYYY</td><td>INITIAL</td></tr></table>													1	ADDED 2026 BOREHOLE LOCATION BH 1-26 TO BH 6-26 AND UPDATED BEDROCK CONTOURS	27/05/2026	KP	NO.	REVISIONS	DD/MM/YYYY	INITIAL	CAIVAN COMMUNITIES GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL DEVELOPMENT 4795 BANK STREET & 3301 BLAIS ROAD OTTAWA, ONTARIO Title: BEDROCK CONTOUR PLAN	Scale: 1:3000 Drawn by: ZS Checked by: MA Approved by: KP	Date: 12/2024 Report No.: PG7301-1 Dwg. No.: PG7301-2 Revision No.: 1
1	ADDED 2026 BOREHOLE LOCATION BH 1-26 TO BH 6-26 AND UPDATED BEDROCK CONTOURS	27/05/2026	KP																				
NO.	REVISIONS	DD/MM/YYYY	INITIAL																				