Geotechnical Investigation

Proposed Residential Development

5993 & 6115 Flewellyn Road & 6030 & 6070 Fernbank Road,
Ottawa, Ontario

Prepared for Caivan (Stittsville South) Inc. & Caivan (Stittsville West) Ltd.

Report PG5570-2, Revision 5 dated July 9, 2025

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Appendices

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 5993 & 6115 Flewellyn Road and 6030 & 6070 Fernbank Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

☐ Determine the subsoil and groundwater conditions at this site by means of test holes.

☐ Provide geotechnical recommendations pertaining to 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 the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

The following report should be read in conjunction with the latest revision of the following Paterson Group reports:

☐ Hydrogeological Study and Water Budget Assessment Report PH4681-1.
☐ Hydrogeological Existing Conditions Report PH4625-1.

2.0 Proposed Development

Based on available drawings, it is understood that the proposed development will consist of a series of low-rise single and townhouse style residential dwellings with associated driveways, local roadways and landscaped areas. Storm water management facilities are to be located within the southern portion of the site. It is understood that the development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out between December 14, 2021 and January 10, 2022. At that time, a total of thirty-eight (38) boreholes were advanced to a maximum depth of 10.2 m below the existing ground surface. A supplemental field program was carried out by Paterson at the subject site from September 28 to 30, 2022 and consisted of advancing 7 boreholes and 1 hand auger hole to maximum depths of 9.1 and 0.7 m, respectively. The test holes were distributed in a manner to provide general coverage of the subject site taking into consideration site features.

A previous geotechnical investigation was also completed by Paterson between November 20 and December 10, 2020 for 6070 & 6115 Flewellyn Road. At that time, 18 test pits were excavated to a maximum depth of 3.4 m below ground surface using a hydraulic shovel excavator. The test hole locations are shown on Drawing PG5570-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of drilling to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Rock core samples were recovered from boreholes BH1-22 to BH5-22, BH1-21, BH2-21, BH3-21, BH22A-21, BH24-21, BH33-21 and BH34-21 drilled during the investigations using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

Soil samples from the test pits from the previous investigation were recovered from the side walls of the open excavation and all soil samples were initially classified on site. All samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the grab samples were recovered from the test pits are shown as "G" on the Soil Profile and Test Data sheets in Appendix 1.

Subsurface conditions observed in the test pits were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test pits locations.

Groundwater

Monitoring wells were installed in all boreholes during the September 2022 investigation and outfitted with data loggers to permit monitoring of the groundwater level subsequent to the completion of sampling program. Additionally, data loggers were outfitted in the monitoring wells installed at boreholes BH1-21 to BH3-21, BH22A-21, BH24-21 and BH33-21.

The remaining boreholes were fitted with flexible piezometers to allow groundwater level monitoring. Further, the depth at which groundwater infiltration was encountered through the sidewalls of the test pits were recorded prior to the completion of excavation as noted in the field. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision handheld GPS and referenced to a geodetic datum. Reference should be made to Drawing PG5570-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. 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.

A total of 12 grain size distribution tests were completed on selected soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution Results sheets presented in Appendix 1.

3.4 Analytical Testing

Four (4) soil samples were 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 Subsection 6.7.

3.5 Permeameter Testing

In-situ permeameter testing was conducted using a Pask (Constant Head Well) Permeameter to confirm infiltration rates of the surficial soils at the subject site. At each location, two (2) 83 mm holes, located approximately 1.5 m away each other, were excavated using a Riverside/Bucket auger to approximate depths ranging from 0.3 to 0.6 m below the existing ground surface. All soils from the auger flights were visually inspected and initially classified on-site. The permeameter reservoir was filled with water and inverted into the hole, ensuring that it was relatively vertical and rested on the bottom of the hole. As the water infiltrated into the soil, the water level of the reservoir was monitored at various time intervals until the rate of fall reached equilibrium, known as *“quasi steady state”* flow rate. Quasi steady state flow can be considered to have been obtained after measuring 3 to 5 consecutive rate of fall readings with identical values. The values for the steady state rate of fall were recorded for each location. The results of testing are further discussed in Subsection 4.2.

3.6 Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) testing was conducted at each monitoring well location with the exception of borehole BH1A-22. The testing was completed to assist in confirming anticipated groundwater flow rates within the subsoils and within the bedrock at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and istropic 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. The assumption regarding screen length and well diameter is considered to be met based on a screen length generally ranging from 1.5 to 3 m and a diameter ranging from 0.03 to 0.05 m.

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 Horslev 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 locations 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

The subject site generally consists of undeveloped, vacant land. An existing garage/storage building is located on the 6115 Flewellyn Road property. The property parcel of 5993 Flewellyn Road is cleared of trees and vegetation, where the property parcels comprising 6070 & 6115 Flewellyn Road are heavily treed with mature growth.

The site gradually slopes downward from the northwest to the southeast. The site also gradually slopes downward from the northeast and southwest to the central portion of the site, resulting in a shallow valley striking northwest - southeast. The subject site is bordered to the south by Flewellyn Road, to the west by residential dwellings, to the north by a residential development, and to the east by agricultural land and residential dwellings.

An existing stormwater management pond is present within the center of the site, adjacent to the west of the hydro corridor. However, the pond is owned by the City of Ottawa and is not part of the current development.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil overlying a loose to compact, brown silty sand to sandy silt deposit, followed by compact to dense glacial till, underlain by bedrock. The glacial till deposit was generally observed to consist of compact to dense brown silty sand with gravel, cobbles and trace clay.

A thin veneer of stiff, brown silty clay with some sand was observed in boreholes BH23-21 and BH26-21. The silty clay veneer was observed to extend to a maximum depth of 1.1 m below the existing ground surface.

A thin (0.3 m), localized occurrence of peat was encountered solely within test pit TP 12 and extended to a maximum depth of 0.8 m below the existing ground surface. The peat was not encountered within TP 11, located in close proximity to TP 12, nor was it encountered in any other test hole completed the geotechnical investigation.

Bedrock

Bedrock was cored in 11 boreholes to a maximum depth of 8.3 m below the bedrock surface, with an average RQD value ranging from 57 to 100%. This is indicative of a fair to excellent quality bedrock within the footprint of the proposed building. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Based on available geological mapping, the bedrock in this area consists of Paleozoic limestone of the Bobcaygeon Formation and an overburden drift thickness of 3 to 10 m depth.

Grain Size Distribution Testing

Grain size distribution testing (sieve analysis) was also completed on 12 selected soil sample. The results of the grain size analysis are summarized in Table 1 on the following page and presented on the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 1 – Summary of Grain Size Distribution Analysis

Test Hole Number	Sample	Gravel (%)	Sand (%)	Fines (%)	
				Silt (%)	Clay (%)
BH1-22	SS2	18.3	47.9	31.2	2.5
BH3-22	SS4	0.0	7.5	87.0	5.5
BH4-22	SS4	19.4	23.3	53.8	3.5
BH5-22	SS3	3.3	25.1	65.6	6.0
BH4-21	SS2 + SS3	6.5	24.2	69.3	
BH11-21	SS3	14.4	50.1	35.5	
BH14-21	SS2 + SS3	25.9	48.9	25.2	
BH19-21	SS2 + SS3	0.1	13.8	86.1	
BH24-21	SS2 + SS3	4.9	46.3	48.8	
BH35-21	SS4 + SS5	61.0	25.5	13.5	
BH37-21	SS3	0.0	64.2	35.8	
BH38-21	SS3 + SS4	0.0	21.0	79.0	

Permeameter Testing Results

In conjunction with a supplemental geotechnical investigation, Paterson completed site specific infiltration testing on September 7, 2022 using a Pask Permeameter in order to identify the infiltration potential of the underlying soils on site. A total of

24 permeameter tests were conducted at 12 locations to provide general coverage of the subject site. Preparation and testing of this investigation are in accordance with the Canadian Standards Association (CSA) B65-12-Annex E. Field saturated hydraulic conductivity (K_{fs}) values and estimated infiltration values are presented in Table 2 on the following page.

Field saturated hydraulic conductivity values were determined using the Engineering Technologies Canada (ETC) Ltd. Reference tables provided in the most recent ETC Past Permeameter User Guide dated July 2018. Infiltration rates have been determined based on approximate relationships provided by the Ontario Ministry of Municipal Affairs and Housing - Supplementary Guidelines to the Ontario Building Code, 1997 - SG-6 - Percolation Time and Soil Descriptions.

Table 2 – Summary of Field Saturated Hydraulic Conductivity Values and Infiltration Rates

Permeameter Test Location	Ground Surface Elevation (m)	Depth of Permeamete Testing (m)	Elevation of Permeameter Testing (m)	K_{fs} (m/sec)	Unfactored Infiltration Rate (mm/hr)	Soil Type
BH1-21	104.29	0.35	103.94	2.1×10^{-6}	56	Silty Sand
		0.60	103.69	1.9×10^{-6}	56	
BH2-21	107.19	0.30	106.89	6.4×10^{-6}	76	Silty Sand
		0.60	106.59	5.3×10^{-7}	39	
BH7-21	107.04	0.30	106.74	1.1×10^{-6}	47	Silty Sand
		0.60	106.44	1.6×10^{-6}	52	
BH11-21	104.98	0.30	104.68	2.7×10^{-6}	60	Silty Sand
		0.60	104.38	1.6×10^{-6}	52	Silty Sand to Sandy Silt
BH15-21	103.08	0.35	102.73	2.1×10^{-7}	31	Silty Sand to Sandy Silt
		0.55	102.53	$\leq8.1\times10^{-9}$	≤13	
BH17-21	104.42	0.30	104.12	5.9×10^{-6}	74	Silty Sand to Sandy Silt
		0.60	103.83	4.1×10^{-6}	67	
BH22-21	102.98	0.30	102.68	1.1×10^{-6}	47	Silty Sand
		0.60	102.38	1.6×10^{-6}	52	
BH23-21	102.38	0.30	102.08	5.3×10^{-7}	39	Silty Clay with Sand
		0.65	101.73	$\leq8.1\times10^{-9}$	≤13	
BH26-21	103.04	0.30	102.74	1.1×10^{-7}	26	Silty Clay with Sand
		0.60	102.44	1.1×10^{-7}	26	
BH29-21	102.31	0.30	102.01	5.3×10^{-7}	39	Silty Sand to Sandy Silt
		0.60	101.71	2.7×10^{-7}	33	
BH31-21	103.43	0.30	103.13	1.1×10^{-6}	47	Silty Sand to Sandy Silt
		0.60	102.83	1.4×10^{-7}	27	
BH37-21	103.54	0.30	103.24	5.3×10^{-6}	72	Silty Sand to Sandy Silt
		0.60	102.94	5.9×10^{-6}	74	

Note: Infiltration rates above do not include a safety correction factor.

The measured field saturated hydraulic conductivity (K_{fs}) values within the test holes are consistent with similar material Paterson has encountered on other sites and typical published values for silty sand, sandy silt and silty clay which typically range from $1x10^{-4}$ to $1x10^{-6}$, $1x10^{-6}$ to $1x10^{-8}$, $1x10^{-7}$ to $1x10^{-9}$ m/sec, respectively. The range in K_{fs} values is generally due to the variability in composition and consistency of the material encountered. It is important to note that the infiltration rates derived from the K_{fs} values in the table above are unfactored, and that a factor of safety will need to be applied prior to being considered for design purposes.

In accordance with the City of Ottawa Technical Bulletin IWSTB-2024-04, the opportunities for infiltration-based Low Impact Development (LID) measures at the subject site are constrained due to the silt content of the in-situ soils, the presence of shallow bedrock and the shallow/perched water table elevation within the overburden. As an alternative, the stormwater management strategy should target Best Management Practices (BMPs) best suited to mitigate the impacts of post-development site conditions, while maintaining consideration of site constraints.

Hydraulic Conductivity Values

Hydraulic conductivity (slug testing) values were recorded at each monitoring well location. The results are presented in Table 3 below.

Table 3 – Summary hydraulic conductivity values

Test Hole ID	Ground Surface Elevation (m)	Screened Interval (m)	K (m/sec)	Test Type	Soil Type/Bedrock
BH1-22	107.31	7.5 - 9.0	1.2×10^{-5}	Falling Head	Bedrock
			1.5×10^{-5}	Falling Head	
			1.6×10^{-5}	Falling Head	
			1.9×10^{-5}	Rising Head	
			1.5×10^{-5}	Rising Head	
BH2-22	103.58	7.5 - 9.0	8.9×10^{-6}	Falling Head	Bedrock
			9.1×10^{-6}	Rising Head	
BH3-22	102.25	7.5 - 9.0	6.0×10^{-5}	Falling Head	Bedrock
			6.6×10^{-5}	Rising Head	
BH3A-22	102.25	1.7 - 3.2	4.2×10^{-6}	Falling Head	Silty Sand to Sandy Silt & Glacial Till
			4.8×10^{-6}	Rising Head	
BH4-22	105.71	7.5 - 9.0	8.7×10^{-7}	Falling Head	Bedrock
			9.1×10^{-7}	Rising Head	
BH5-22	105.70	7.5 - 9.0	1.2×10^{-5}	Falling Head	Bedrock
			2.0×10^{-5}	Falling Head	
			1.4×10^{-5}	Rising Head	Bedrock
			1.5×10^{-5}	Rising Head	
HA1-22	106.78	0.4 – 0.7	2.2×10^{-5}	Falling Head	Silty Sand
			8.8×10^{-6}	Rising Head	
BH1-21	104.29	2.8 - 5.8	1.4×10^{-4}	Falling Head	Bedrock
			1.1×10^{-4}	Rising Head	

Table 3 – Summary hydraulic conductivity values

Test Hole ID	Ground Surface Elevation (m)	Screened Interval (m)	K (m/sec)	Test Type	Soil Type/Bedrock
BH2-21	107.19	2.6 - 5.6	$4.0x10^{-5}$	Falling Head	Bedrock
			$4.0x10^{-5}$	Falling Head	
			$3.9x10^{-5}$	Rising Head	
			$4.1x10^{-5}$	Rising Head	
BH3-21	108.41	2.7 - 5.7	$3.0x10^{-6}$	Falling Head	Bedrock
BH22A-21	102.98	7.2 - 10.2	$4.3x10^{-7}$	Falling Head	Bedrock
BH24-21	103.07	4.9 - 7.9	$6.0x10^{-5}$	Falling Head	Bedrock
			$7.3x10^{-5}$	Falling Head	
			$5.8x10^{-5}$	Rising Head	
			$5.7x10^{-5}$	Rising Head	
BH33-21	104.70	3.3 - 6.3	$1.6x10^{-4}$	Rising Head	Bedrock

Slug testing completed at the monitoring wells screened primarily in the silty sand to sandy silty layer (BH 3A-22, HA1-22) identified hydraulic conductivity values ranging from approximately $4.2x10^{-6}$ to $2.2x10^{-5}$ m/sec. These values are generally consistent with similar material Paterson has encountered on other sites and typical published values for silty sand to sandy silt, which typically range from $1x10^{-5}$ to $1x10^{-7}$ m/sec and is dependent on the ratio of sand to silt within the material.

The slug testing completed at the monitoring wells screened in bedrock identified hydraulic conductivity values ranging from approximately $4.3x10^{-7}$ to $1.6x10^{-4}$ m/sec. These values are generally consistent to with similar material Paterson has encountered on other sites and typical published values for limestone bedrock, which typically range from $1x10^{-5}$ to $1x10^{-10}$ m/sec and is dependent on the quality of the bedrock at a given location.

4.3 Groundwater

The groundwater levels were manually recorded within the monitoring wells and piezometers installed at each borehole. Data loggers were installed in all monitoring wells to record seasonal fluctuations and precipitation collected within the upper portion of the subsurface profile across the site. Where encountered, groundwater infiltration through the sidewalls of the test pits were recorded. The recorded groundwater levels are presented in Table 4 below, and are further noted on the Soil Profile and Test Data sheets in Appendix 1. The groundwater data recorded at the subject site to date is presented on Figures 2 to 13: Monitoring Well Water Elevations in Appendix 2.

Table 4 – Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH1-22	107.31	1.33	105.99	October 11, 2022
		1.35	105.97	October 28, 2022
		0.83	106.48	April 4, 2023
		1.35	105.96	May 31, 2023
BH1A-22	107.31	1.44	105.87	October 11, 2022
		1.43	105.88	October 28, 2022
		0.94	106.38	April 4, 2023
		1.46	105.86	May 31, 2023
BH2-22	103.58	1.52	102.06	October 11, 2022
		1.52	102.06	October 28, 2022
		0.59	102.99	April 4, 2023
		1.31	102.27	May 31, 2023
BH3-22	102.25	0.84	101.42	October 11, 2022
		0.61	101.64	October 28, 2022
		0.11	102.15	April 4, 2023
		0.93	101.32	May 31, 2023
BH3A-22	102.25	0.81	101.44	October 11, 2022
		0.40	101.85	October 28, 2022
		0.00	102.25	April 4, 2023
		0.99	101.26	May 31, 2023
BH4-22	105.71	3.62	102.10	October 11, 2022
		3.65	102.07	October 28, 2022
		3.08	102.64	April 4, 2023
		3.48	102.23	May 31, 2023
BH5-22	105.70	1.62	104.09	October 11, 2022
		1.64	104.06	October 28, 2022
		0.90	104.80	April 4, 2023
		1.56	104.14	May 31, 2023
HA1-22	106.78	0.31	106.48	October 11, 2022
		0.28	106.51	October 28, 2022
		0.14	106.64	April 4, 2023
		0.29	106.49	May 31, 2023
BH1-21*	104.29	1.22	103.07	January 11, 2022
		1.12	103.17	October 11, 2022
		1.01	103.28	October 28, 2022
		0.09	104.21	April 4, 2023
		0.97	103.33	May 31, 2023
BH2-21*	107.19	0.82	106.37	January 11, 2022
		1.16	106.03	October 11, 2022
		0.95	106.25	October 28, 2022

Table 4 – Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
		0.33	106.87	April 4, 2023
		0.87	106.32	May 31, 2023
BH3-21*	108.41	0.89	107.52	January 11, 2022
		0.90	107.51	October 11, 2022
		0.92	107.49	October 28, 2022
		0.52	107.89	April 4, 2023
		0.84	107.57	May 31, 2023
BH4-21	108.95	1.23	107.72	January 11, 2022
BH5-21	108.38	Dry	N/A	January 11, 2022
BH6-21	106.32	Dry	N/A	January 11, 2022
BH7-21	107.04	1.09	105.95	January 11, 2022
BH8-21	105.91	Dry	N/A	January 11, 2022
BH9-21	104.62	Blocked	N/A	January 11, 2022
BH10-21	105.70	2.83	102.87	January 11, 2022
BH11-21	104.98	1.32	103.42	January 11, 2022
BH12-21	104.05	1.58	102.73	January 11, 2022
BH13-21	103.54	1.44	101.96	January 11, 2022
BH14-21	103.28	1.37	101.91	January 11, 2022
BH15-21	103.08	0.92	102.16	January 11, 2022
BH16-21	104.19	1.32	102.87	January 11, 2022
BH17-21	104.42	1.25	103.17	January 11, 2022
BH18-21	105.06	1.40	103.66	January 11, 2022
BH19-21	101.85	1.04	100.81	January 11, 2022
BH20-21	102.25	1.71	100.54	January 11, 2022
BH21-21	102.92	Blocked	N/A	January 11, 2022
BH22A-21*	102.98	2.49	100.49	January 11, 2022
		2.61	100.37	October 11, 2022
		1.77	101.21	April 4, 2023
		2.72	100.26	May 31, 2023
BH23-21	102.38	Blocked	N/A	January 11, 2022
BH24-21*	103.07	0.67	102.40	January 11, 2022
		0.60	102.47	October 11, 2022
		0.46	102.61	October 28, 2022
		-0.03	103.10	April 4, 2023
		0.74	102.34	May 31, 2023
BH25-21	102.73	0.71	102.02	January 11, 2022
BH26-21	103.04	0.78	102.26	January 11, 2022
BH27-21	102.71	0.84	101.87	January 11, 2022
BH28-21	101.85	1.79	100.06	January 11, 2022
BH29-21	102.31	Blocked	N/A	January 11, 2022

Table 4 – Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH30-21	102.44	1.62	100.82	January 11, 2022
BH31-21	103.43	1.27	102.16	January 11, 2022
BH32-21	103.74	1.62	102.12	January 11, 2022
BH33-21*	104.70	1.84	102.86	January 11, 2022
		2.12	102.58	October 11, 2022
		1.98	102.72	October 28, 2022
		1.20	103.51	April 4, 2023
		2.22	102.49	May 31, 2023
BH34-21	102.65	Blocked	N/A	January 11, 2022
BH35-21	105.03	1.22	103.81	January 11, 2022
BH36-21	102.79	0.62	102.17	January 11, 2022
BH37-21	103.54	1.52	102.02	January 11, 2022
BH38-21	103.62	1.94	101.68	January 11, 2022
TP-1	105.94	Dry	-	November 20, 2020
TP-2	105.06	Dry	-	November 20, 2020
TP-3	102.10	Dry	-	November 20, 2020
TP-4	108.49	Dry	-	November 20, 2020
TP-5	108.36	1.28	107.08	November 20, 2020
TP-6	107.91	1.70	106.21	November 20, 2020
TP-7	106.31	2.24	104.07	November 20, 2020
TP-8	105.48	Dry	-	November 20, 2020
TP-9	104.47	Dry	-	November 20, 2020
TP-10	103.62	0.51	103.11	December 10, 2020
TP-11	103.01	0.89	102.12	December 10, 2020
TP-12	103.21	1.82	101.39	December 10, 2020
TP-13	104.30	0.61	103.69	December 10, 2020
TP-14	105.60	Dry	-	December 10, 2020
TP-15	106.80	2.28	104.52	December 10, 2020
TP-16	104.62	2.33	102.29	December 10, 2020
TP-17	103.90	1.78	102.53	December 10, 2020
TP-18	103.42	Dry	-	December 10, 2020

Notes:
-The ground surface elevation at each test hole location was surveyed using a handheld GPS and referenced to a geodetic datum
-* Denotes groundwater monitoring well

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered samples.

In addition to manual water level measurements, a groundwater monitoring program was carried out at the subject site. The groundwater monitoring program provides an overview of the variations in the monitoring well water levels based upon seasonal fluctuations. The monitoring wells were equipped with a submersible datalogger (TD-Diver, VanEssen Instruments) to accurately monitor fluctuations in the water levels. The datalogger was programmed to continuously measure and record water levels at a fixed rate of one (1) reading every 24 hours.

The monitoring program was undertaken from October 2022 to May 2023. The monitoring data was compared with Environment and Natural Resources Canada precipitation data from the Ottawa International Airport over the same timeframe as part of the monitoring program. The monitoring data is presented in Figures 2 to 13 in Appendix 2.

Upon review of the datalogger readings and manual measurements, the groundwater readings measured within the monitoring wells and the piezometers across the subject site varied from an elevation of 100.26 m to a maximum elevation of 108.1 m, generally decreasing with the topography of the site. Based on our analysis of the measured groundwater levels and the data logger groundwater readings, seasonal groundwater in piezometers and the monitoring wells varied between 0.6 to 2.8 m below ground surface and 0.0 to 3.7 m, respectively.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. It is expected that the proposed residential buildings will be founded on conventional style footings placed on a compact silty sand to sandy silt, compact to dense glacial till, and/or bedrock bearing surface.

It is anticipated that bedrock removal may be required in localized areas across the site for building construction and service installation. All contractors should be prepared for bedrock removal within the subject site.

Any loose or poor performing silty sand or sandy silt encountered at the underside of footing elevation should be proof-rolled and approved by the geotechnical consultant prior to the placement of the footings.

As the stiff, brown, silty clay layer was only encountered in two borehole locations and was only observed to a shallow depth. A 2 m permissible grade raise restriction is recommended for settlement sensitive structures placed over the silty clay deposit.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, peat and deleterious fill, such as those containing significant organic materials, or construction debris/remnants should be stripped from beneath, and within the lateral support zones, of any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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

Proof-Rolling

Where loose or disturbed silty sand or sandy silt 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, under the observation of Paterson.

Any poor performing areas noted during the proof-rolling operations should be removed and replaced with an approved fill.

Bedrock Removal

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

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.
As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

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

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

Vibration Considerations

Construction operations are 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. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipment. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining 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 several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines.

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 building.

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. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

To in-fill existing channels/ditches below building areas, roadways or other settlement sensitive structures, it is recommended to place Granular A, Granular B Type I or II, well graded blast rock (maximum 200 mm diameter) or select subgrade material. The backfill material should be placed under dry conditions, in above freezing temperatures and approved by the geotechnical consultant. The backfill should be placed in maximum 300 mm loose lifts and compacted to 98% of its SPMDD.

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

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. This material should be used structurally only to build up the subgrade for pavements.

Where the fill is open-graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction.

In-Filling Existing Ditches

Where existing ditches or channels are encountered, in-filling should be conducted according to the following methodology. In-filling the existing ditches and channels should be completed in a stepped fashion within the lateral support zone of the proposed buildings or other settlement-sensitive structures. The fill should consist of clean imported granular fill, such as OPSS Granular A or Granular B Type II material. The steps should have a minimum horizontal length of 1.5 m and minimum vertical height of 0.5 m, and should be compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Bearing resistance values are provided in Table 5 for footings placed on an undisturbed silty sand, glacial till or clean bedrock bearing surface.

Table 5 – Bearing Resistance Values

Bearing Surface	Factored Bearing Resistance Value at ULS (kPa)	Bearing Resistance Value at SLS or Allowable Bearing Pressure (kPa)
Compact Silty Sand to Sandy Silt	250	150
Compact to Dense Glacial Till	250	150
Engineered Fill (Granular A or Granular B Type II)	250	150
Clean Surface Sounded Bedrock	1000	-

Note: A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

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

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

Footings designed using the bearing resistance values at SLS provided in Table 1 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.

Lateral Support

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

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 a minimum of 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 heavily fractured, weathered bedrock bearing medium 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 to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements.

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

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class X_c** as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024 for foundations considered at this site. A higher seismic class may be applicable, provided the footings are within 3 m of the bedrock surface. However, this would need to be confirmed by performing a seismic shear wave velocity test at the subject site.

Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

Soils underlying the subject site are not susceptible to liquefaction. The granular soils below the USF elevation (sandy silt, silty sand and glacial till deposits) at the subject site have been evaluated for liquefaction potential in accordance with the “Liquefaction Resistance of Soils” publication prepared by Youd et al. (2001). Soils at the subject site were determined to have suitable factors of safety against liquefaction greater than the required factor of safety of 1.1 against liquefaction potential at all depths of overburden. This study is provided in Appendix 3.

5.5 Basement Slab

With the removal of all topsoil, peat and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction. Provision should be made for proof rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill.

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

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m3.However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

Lateral Earth Pressures

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

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the site area is 0.30 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Design

Car only parking areas, access and heavy traffic access areas are expected at this site. The subgrade material is anticipated to consist of silty sand to sandy silt, glacial till, compacted engineered fill or bedrock. The proposed pavement structures are presented in Tables 6,7 and 8.

Table 6 – Recommended Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, or bedrock.	

Table 7 – Recommended Pavement Structure – Local and Collector Roadways Without Bus Traffic

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** – HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
450	**SUBBASE** – OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, or bedrock.	

Table 8 – Recommended Pavement Structure – Roadways with Bus Traffic

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** – HL-8 or Superpave 19 Asphaltic Concrete
50	**Lower Binder Course** – HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
600	**SUBBASE** – OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, 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.

For residential driveways and car only parking areas, an Ontario Traffic Category A will be used. For local and collector roadways, an Ontario Traffic Category B should be used for design purposes. For roadways with bus traffic, an Ontario Traffic Category D should be used for design purposes.

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 I or 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.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, 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.

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 geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

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

Frost Susceptibility of Bedrock

When bedrock is encountered above the proposed founding depth and soil frost cover is less than 1.5 m, the frost susceptibility of the bedrock should be determined. This can be accomplished as follows:

- ☐ Drill supplemental coreholes within the bedrock in the vicinity of the foundations and assess the frost susceptibility.

- ☐ Examine service trench profiles extending in the bedrock in the vicinity of the foundations to determine if weathering is extensive.

If the bedrock is considered to be **non-frost susceptible**, the footings can be poured directly on the bedrock without any further frost protective measures.

If the bedrock is considered to be **frost susceptible**, the following measures should be implemented for frost protection:

- ☐ Option A – Sub-excavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.

- ☐ Option B – Use insulation to protect footings. It is preferable to pour footings on the insulation overlying weathered bedrock. However, due to potential undulating of the bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footing to be poured directly on the weathered bedrock.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. 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). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

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

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

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 be 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.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material for areas over a soil subgrade. However, the bedding thickness should be increased to 300 mm for areas over a bedrock subgrade, if encountered. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe. Reference should be made to drawings - OPSD 802.030, OPSD 802.031 & OPSD 802.033 for Rigid Pipe Bedding, Cover and Backfill on Type 1, 2 or 3 soil and bedrock excavation.

The cover material, which should consist of OPSS Granular A crushed stone, should extend from the spring line of the pipe to a minimum of 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) silty sand to sandy silt and glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet sub-excavated soil should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. All stones greater than 300 mm in their greatest dimension should be removed prior to reuse of site-generated glacial till.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of 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 98% of the SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be 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 by the use of 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 in 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 following soil samples were submitted for analytical testing:

❐ TP4 G3, sampled between geodetic elevations of 107.79 to 107.06 m
❐ TP16 G2, sampled between geodetic elevations of 104.27 to 103.66 m
❐ BH17-21 SS3, sampled between geodetic elevations of 103.14 to 102.68 m
❐ BH34-21 SS3, sampled between geodetic elevations of 101.14 to 100.54 m

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

6.8 Stormwater Management Pond

Based on the available drawings and discissions with the client, it is understood that two stormwater management ponds are proposed for the subject site. Paterson will be conducting supplemental geotechnical investigations in the near future in support of stormwater management pond design. Geotechnical recommendations in support of construction of the stormwater management ponds can be provided as part of detailed design, following completion of the supplemental geotechnical investigations.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

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

All excess soils generated by construction activities should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

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.

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.

O. Canton

Owen R. Canton, B.Eng.

LICENSED PROFESSIONAL ENGINEER
July 9, 2025
K. A. PICKARD
100531344
PROVINCE OF ONTARIO

Kevin A. Pickard, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5993, 6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 28, 2022

FILE NO. **PG5570**

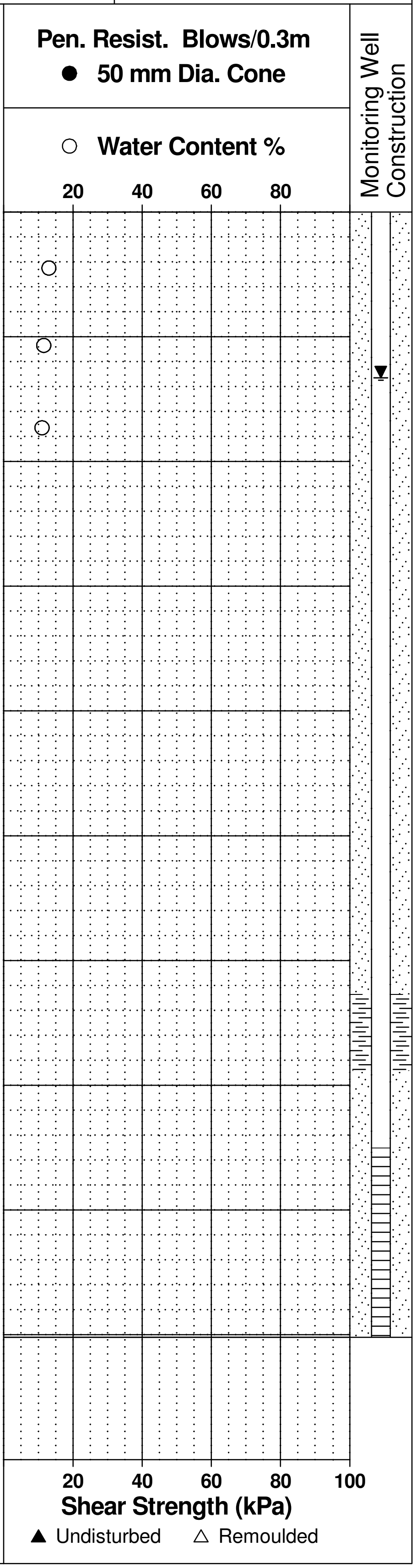
HOLE NO. **BH 1-22**

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30				
Loose to compact, brown **SILTY SAND,** trace gravel	0.60	AU	1		
GLACIAL TILL: Compact to dense, brown silty sand to sandy silt with gravel, cobbles and boulders	2.34	SS	2	45	17
		SS	3	14	65
BEDROCK: Excellent quality, grey limestone interbedded with dolostone	9.02	RC	1	100	89
		RC	2	100	100
		RC	3	100	100
		RC	4	98	98
		RC	5	100	100

End of Borehole

(GWL @ 1.33m - Oct. 11, 2022)

DEPTH (m)	ELEV. (m)
0	107.31
1	106.31
2	105.31
3	104.31
4	103.31
5	102.31
6	101.31
7	100.31
8	99.31
9	98.31



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5993, 6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

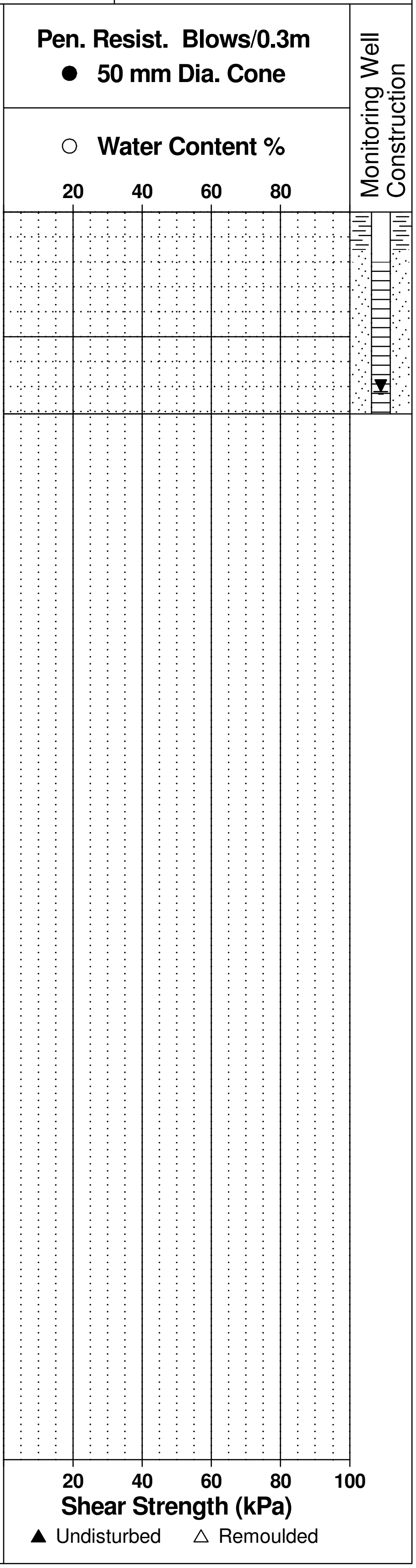
BORINGS BY CME-55 Low Clearance Drill

DATE September 28, 2022

FILE NO. PG5570

HOLE NO. BH 1A-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	107.31
OVERBURDEN						1	106.31
1.62							
End of Borehole Practical refusal to augering at 1.62m depth (GWL @ 1.44m - Oct. 11, 2022)							



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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

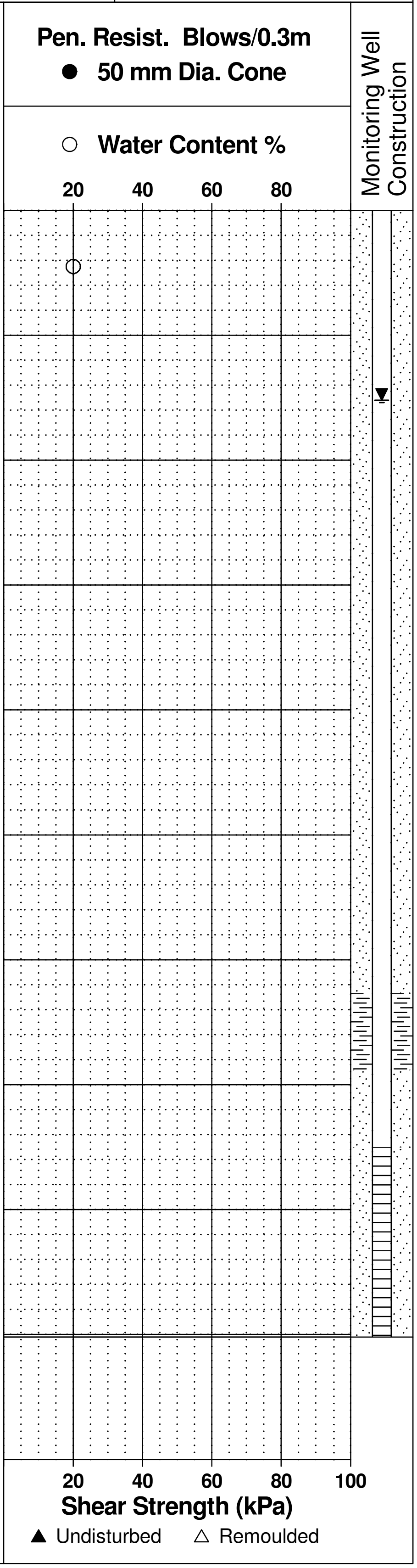
DATE September 28, 2022

FILE NO. **PG5570**

HOLE NO. **BH 2-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.30					
Compact, brown **SILTY SAND to SANDY SILT,** trace clay and gravel 0.76		AU	1		
BEDROCK: Good to excellent quality, grey limestone interbedded with dolostone		RC	1	100	77
		RC	2	100	97
		RC	3	100	100
		RC	4	100	100
		RC	5	100	97
		RC	6	100	100
9.02					
End of Borehole (GWL @ 1.52m - Oct. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	103.58
1	102.58
2	101.58
3	100.58
4	99.58
5	98.58
6	97.58
7	96.58
8	95.58
9	94.58



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DATUM Geodetic

REMARKS

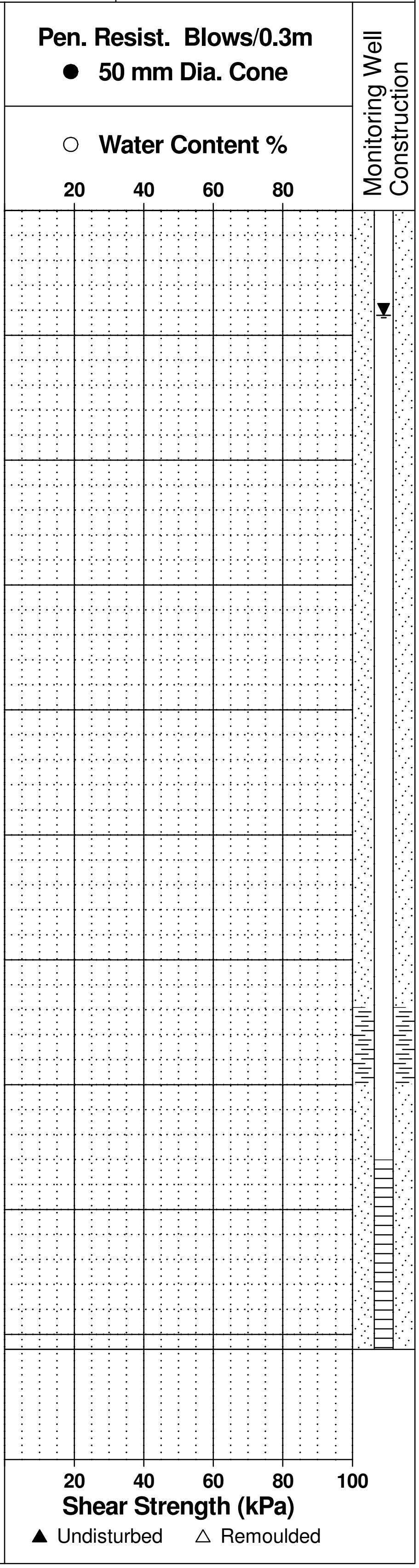
BORINGS BY CME-55 Low Clearance Drill

DATE September 29, 2022

FILE NO. **PG5570**

HOLE NO. **BH 3-22**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.25
TOPSOIL 0.28						
Compact, brown **SILTY SAND to SANDY SILT**	AU	1				
	SS	2	58	19	1	101.25
2.21	SS	3	58	17	2	100.25
GLACIAL TILL: Grey silty sand to sandy silt with gravel, cobbles and boulders, trace clay	SS	4	67	3		
3.43	SS	5	67	50+	3	99.25
BEDROCK: Excellent quality, grey limestone interbedded with doloston	RC	1	100	96	4	98.25
					5	97.25
	RC	2	100	98	6	96.25
	RC	3	100	100	7	95.25
	RC	4	100	100	8	94.25
9.12					9	93.25
End of Borehole (GWL @ 0.84m - Oct. 11, 2022)						



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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 29, 2022

FILE NO. PG5570

HOLE NO. BH 3A-22

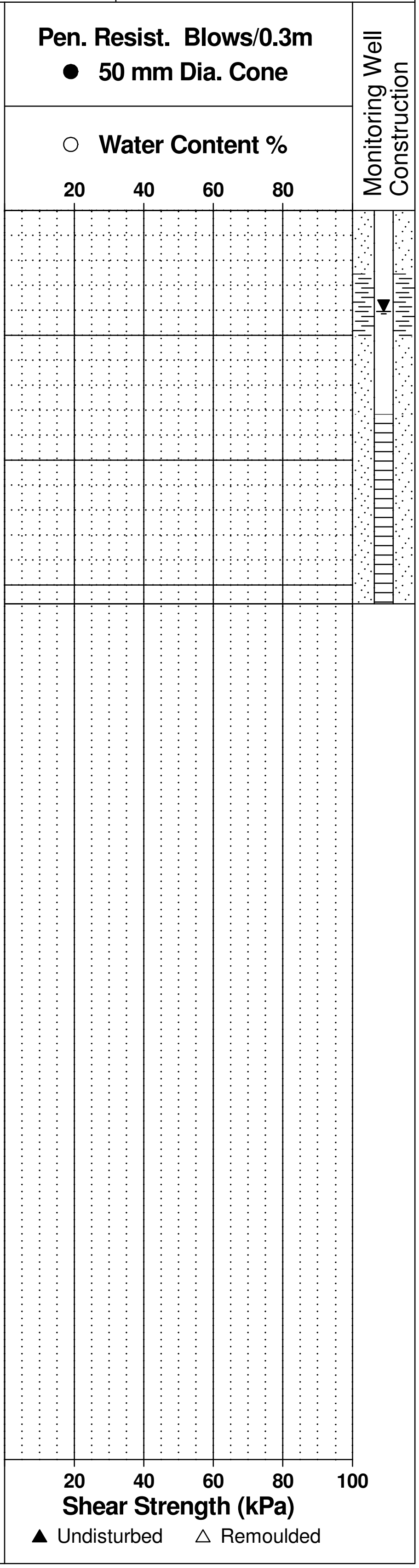
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.28	■				
Compact, brown **SILTY SAND to SANDY SILT** 2.20		AU	1		
GLACIAL TILL: Grey silty sand to sandy silt with gravel, cobbles and boulders, trace clay 3.15					

End of Borehole

Practical refusal to augering at 3.15m depth.

(GWL @ 0.81m - Oct. 11, 2022)

DEPTH (m)	ELEV. (m)
0	102.25
1	101.25
2	100.25
3	99.25



SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

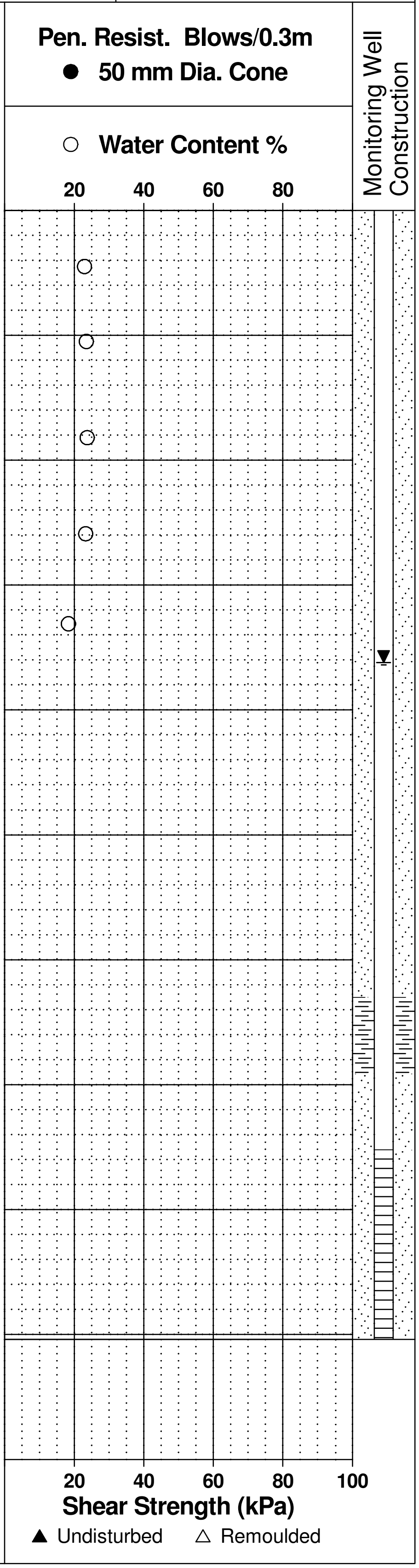
DATE September 29, 2022

FILE NO. PG5570

HOLE NO. BH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL (to 0.28)					
Compact, brown **SILTY SAND to SANDY SILT** (0.28 to 2.21)		AU	1		
		SS	2	75	22
		SS	3	75	21
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace clay - grey by 3.0m depth (2.21 to 3.61)		SS	4	67	17
		SS	5	57	45
BEDROCK: Good to excellent quality, grey limestone interbedded with dolostone (3.61 to 9.04)		RC	1	100	84
		RC	2	100	98
		RC	3	100	100
		RC	4	100	100
End of Borehole (GWL @ 3.62m - Oct. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	105.71
1	104.71
2	103.71
3	102.71
4	101.71
5	100.71
6	99.71
7	98.71
8	97.71
9	96.71



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DATUM Geodetic

REMARKS

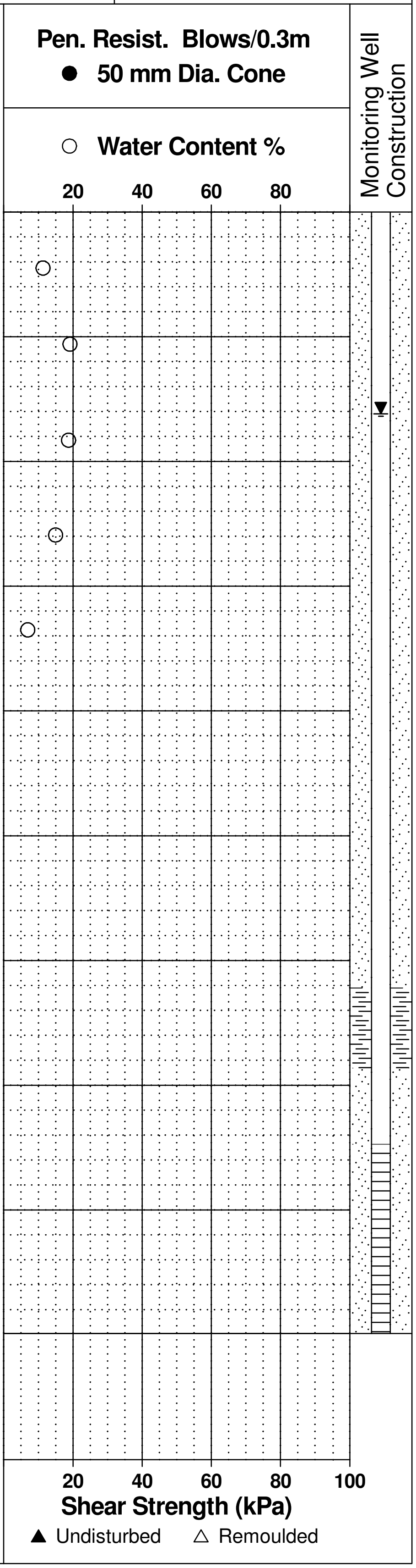
BORINGS BY CME-55 Low Clearance Drill

DATE September 30, 2022

FILE NO. **PG5570**

HOLE NO. **BH 5-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.70
TOPSOIL 0.28							
Compact, brown **SILTY SAND to SANDY SILT**		AU	1				
		SS	2	79	21	1	104.70
1.96		SS	3	71	29		
GLACIAL TILL: Compact to dense, brown silty sand to sandy silt, trace gravel 2.29						2	103.70
		SS	4	100	50+		
BEDROCK: Excellent quality, grey limestone interbedded with dolostone		RC	1	100	100		
						3	102.70
		RC	2	100	100		
						4	101.70
						5	100.70
		RC	3	100	100		
						6	99.70
		RC	4	100	100		
						7	98.70
						8	97.70
		RC	5	100	100		
8.99							
End of Borehole (GWL @ 1.62m - Oct. 11, 2022)							



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SOIL PROFILE AND TEST DATA

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REMARKS

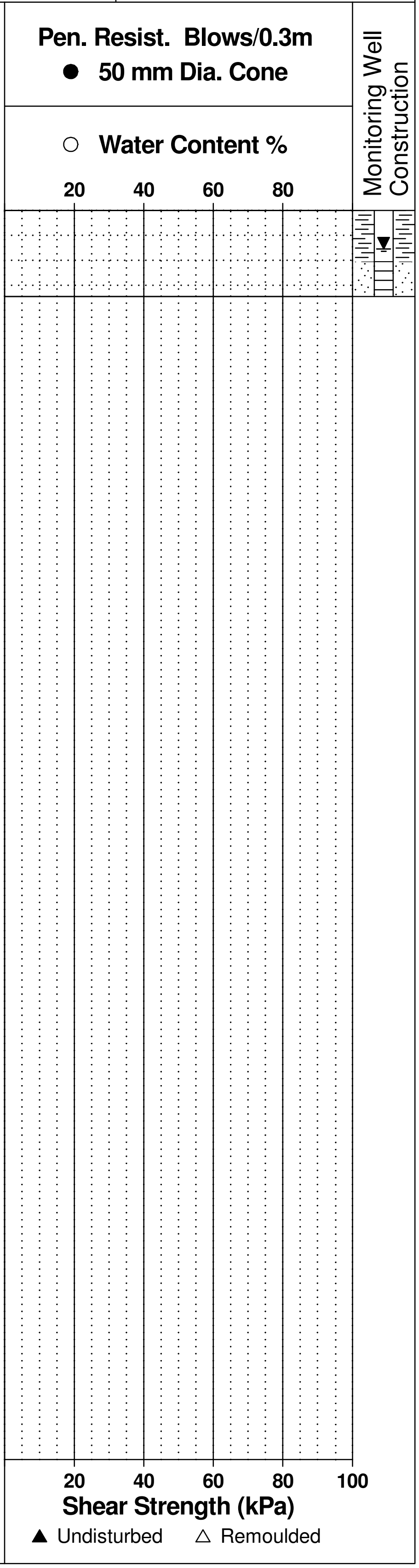
BORINGS BY Hand Auger

DATE September 28, 2022

FILE NO. PG5570

HOLE NO. HA 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	106.78
TOPSOIL 0.30	■						
Brown **SILTY SAND,** trace gravel 0.69	▥						
End of Hand Auger Hole (GWL @ 0.31m - Oct. 11, 2022)							



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Prop. Residential Development - 6115 Flewellyn Road
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DATUM Geodetic

REMARKS

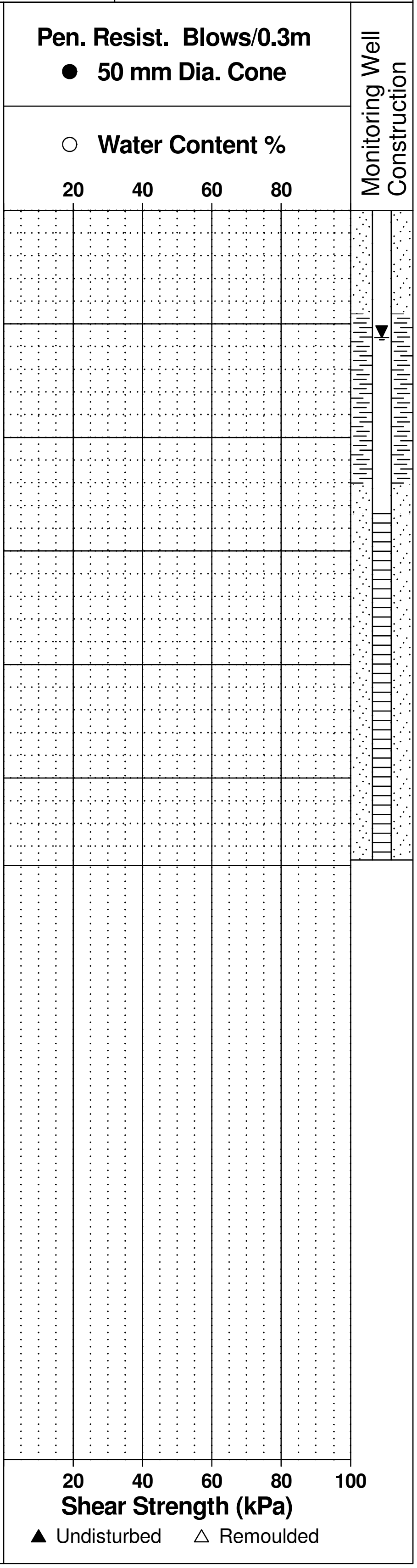
BORINGS BY Track-Mount Power Auger

DATE December 14, 2021

FILE NO. PG5570

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
						0	104.29
TOPSOIL 0.43							
Very loose, brown **SILTY SAND**		AU	1				
- some clay by 0.6m depth		SS	2	8	1	1	103.29
1.52							
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	25	23	2	102.29
2.19		SS	4	100	50+		
		RC	1	100	57		
						3	101.29
BEDROCK: Fair to excellent quality, grey limestone interbedded with dolostone		RC	2	100	68		
- 20mm thick mud seam at 3.4m depth						4	100.29
- 12mm thick mud at 3.7m depth		RC	3	100	98	5	99.29
5.77							
End of Borehole							
(GWL @ 1.22m - Jan. 11, 2022)							



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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

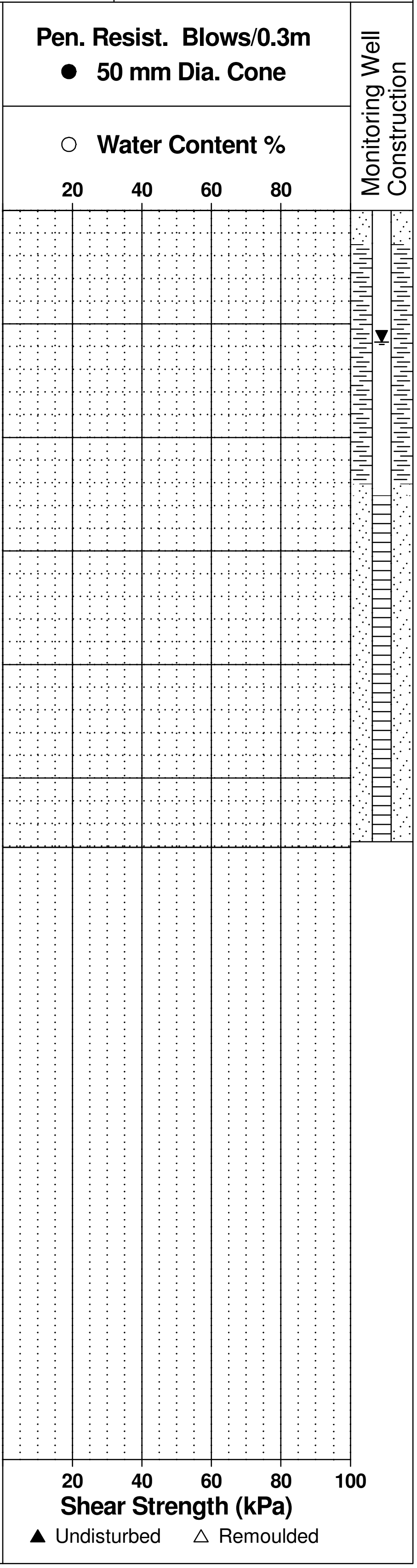
DATE December 14, 2021

FILE NO. **PG5570**

HOLE NO. **BH 2-21**

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Mulch	0.10				
TOPSOIL	0.51				
Compact, brown **SILTY SAND**	0.91	AU	1		
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	2.21	SS SS SS	2 3 4	75 75 0	12 50 50+
BEDROCK: Good to excellent quality, grey limestone - 12mm thick mud seam at 4.1m depth	5.61	RC RC RC	1 2 3	100 100 100	80 100 95
End of Borehole (GWL @ 0.82m - Jan. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	107.19
1	106.19
2	105.19
3	104.19
4	103.19
5	102.19



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

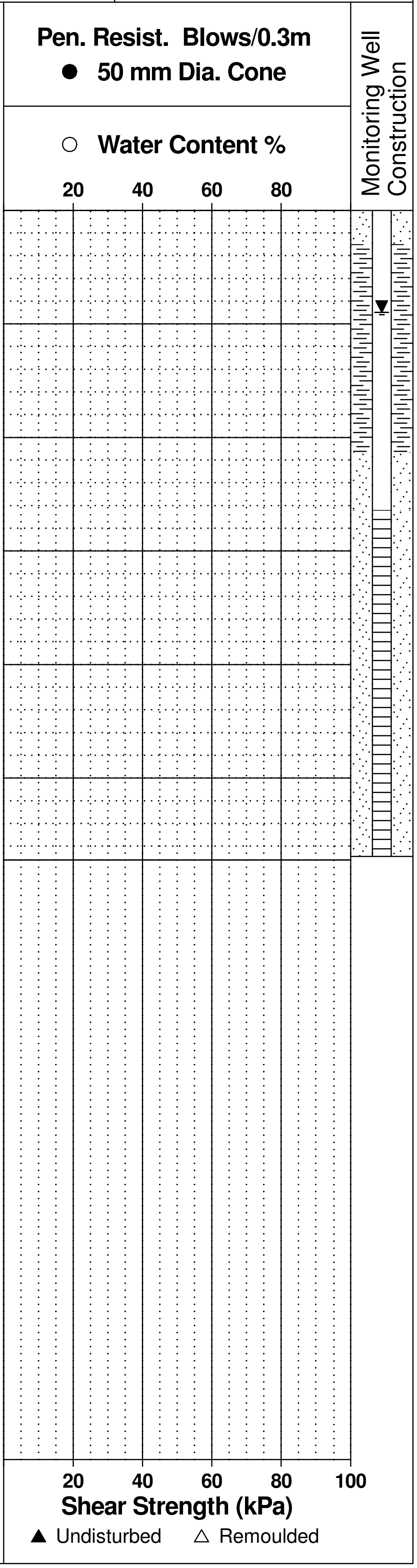
FILE NO. **PG5570**

HOLE NO. **BH 3-21**

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Mulch	0.08
TOPSOIL	0.43
Loose, brown **SILTY SAND**	0.63
Loose to compact, brown **SILTY SAND to SANDY SILT**	1.55
BEDROCK: Good to excellent, grey limestone interbedded with dolostone - 30mm thick mud seam at 4.3m depth	5.72
End of Borehole (GWL @ 0.89m - Jan. 11, 2022)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	0	50+
RC	1	100	100
RC	2	100	72
RC	3	100	100

DEPTH (m)	ELEV. (m)
0	108.41
1	107.41
2	106.41
3	105.41
4	104.41
5	103.41



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

FILE NO. PG5570

HOLE NO. BH 4-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	108.95
Mulch 0.10							
TOPSOIL 0.30		AU	1				
Compact, brown **SILTY SAND,** trace shells 0.60							
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders		SS	2	50	12	1	107.95
		SS	3	42	21	2	106.95
2.23							
End of Borehole Practical refusal to augering at 2.23m depth (GWL @ 1.23m - Jan. 11, 2022)							

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

FILE NO. PG5570

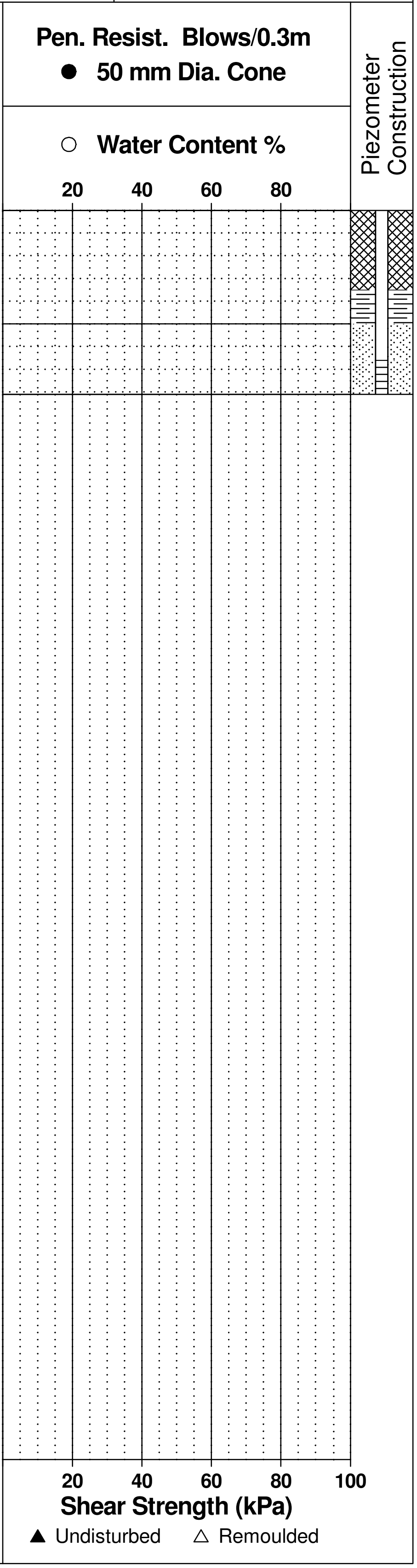
HOLE NO. BH 5-21

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	108.38
TOPSOIL	0.36	AU	1				
Loose, brown **SILTY SAND**	1.22	SS	2		4	1	107.38
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders	1.62	SS	3	0	50+		

End of Borehole

Practical refusal to augering at 1.62m depth

(BH dry - January 11, 2022)



SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

FILE NO. PG5570

HOLE NO. BH 6-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	106.32
TOPSOIL	0.41	AU	1				
Loose, brown **SILTY SAND,** trace clay	0.60						
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	1.55	SS	2	83	17	1	105.32

End of Borehole

Practical refusal to augering at 1.55m depth

(BH dry - January 11, 2022)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

FILE NO. PG5570

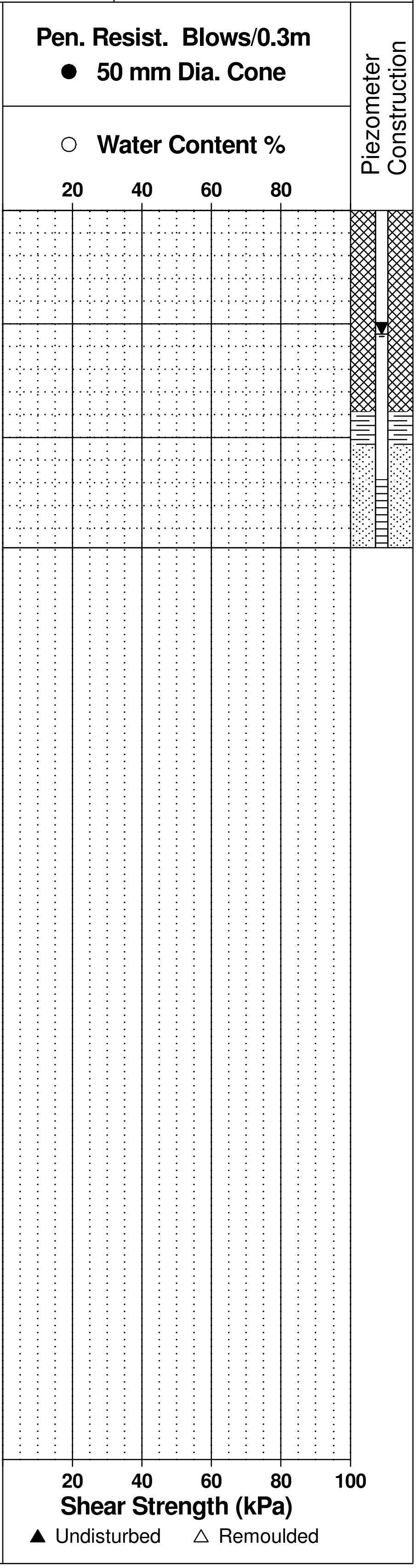
HOLE NO. BH 7-21

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Mulch	0.10	AU	1			0	107.04
TOPSOIL	0.41						
Loose, brown **SILTY SAND**	0.60						
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders		SS	2	100	19	1	106.04
		SS	3	67	44	2	105.04
	2.97	SS	4		52		

End of Borehole

Practical refusal to augering at 2.97m depth

(GWL @ 1.09m - Jan. 11, 2022)



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REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

FILE NO. PG5570

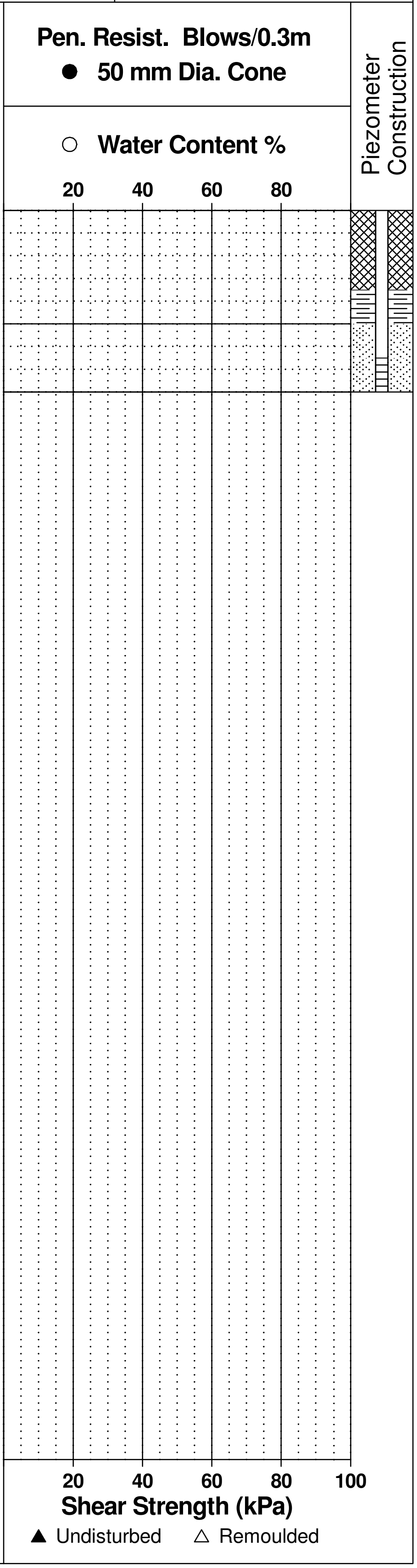
HOLE NO. BH 8-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	105.91
Mulch 0.05	AU	1				
TOPSOIL 0.38						
Loose, brown **SILTY SAND** 0.60						
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	SS	2	67	20	1	104.91
1.60	SS	3	0	50+		

End of Borehole

Practical refusal to augering at 1.60m depth

(BH dry - January 11, 2022)



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REMARKS

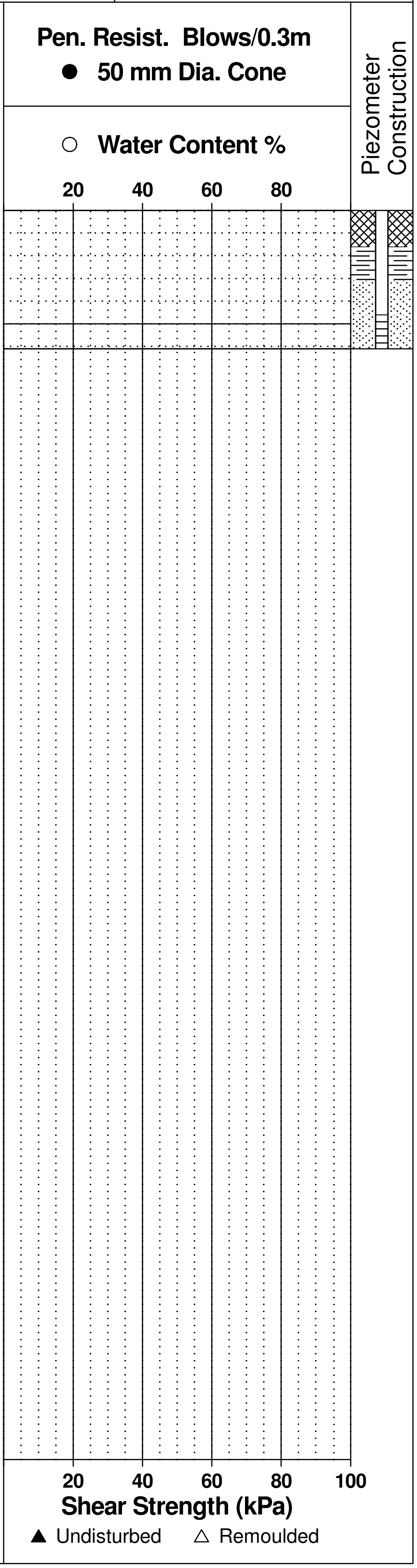
BORINGS BY Track-Mount Power Auger

DATE December 15, 2021

FILE NO. PG5570

HOLE NO. BH 9-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.62
TOPSOIL 0.36							
Loose, brown **SILTY SAND,** trace clay 0.69		AU	1				
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders 1.22		SS	2		50+	1	103.62
End of Borehole Practical refusal to augering at 1.22m depth (Piezometer damaged - Jan. 11, 2022)							



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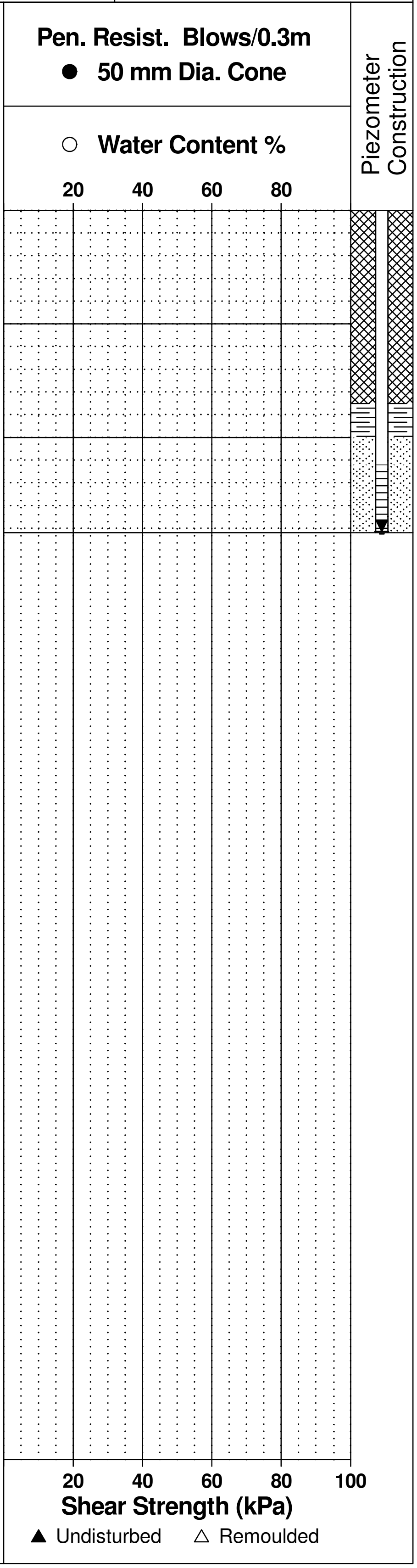
REMARKS
BORINGS BY Track-Mount Power Auger
DATE December 15, 2021
FILE NO. **PG5570**
HOLE NO. **BH10-21**

SOIL DESCRIPTION	DEPTH OF STRATUM	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.70
TOPSOIL	0.36						
Compact, brown **SILTY SAND**		AU	1				
		SS	2	67	23	1	104.70
		SS	3	67	16	2	103.70
	2.74	SS	4	64	25		
GLACIAL TILL: Compact to dense, grey silty sand with gravel, cobbles and boulders	2.84						

End of Borehole

Practical refusal to augering at 2.84m depth

(GWL @ 2.83m - Jan. 11, 2022)



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REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. **PG5570**

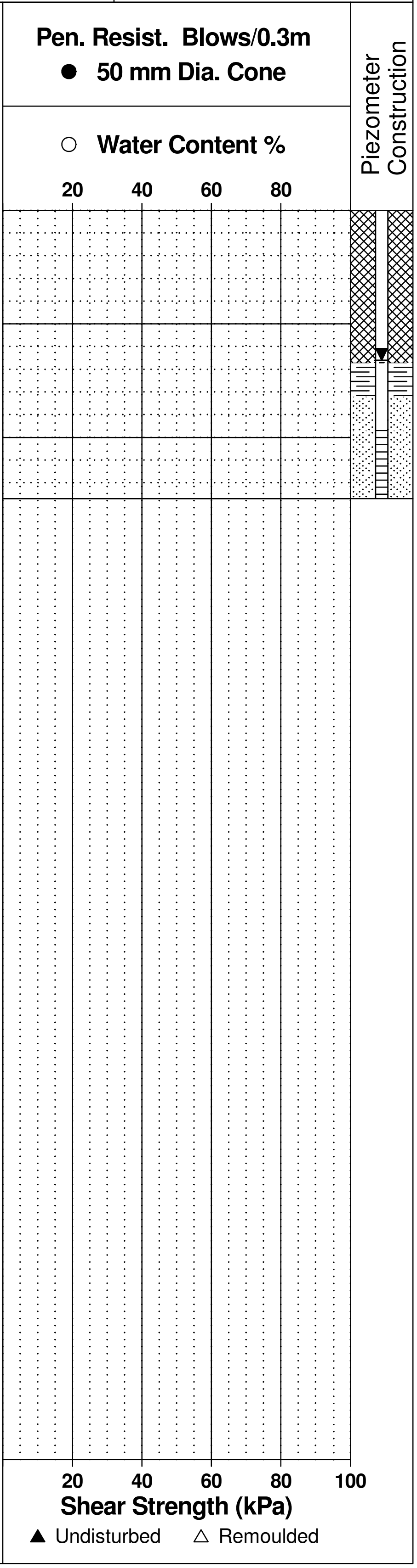
HOLE NO. **BH11-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.98
TOPSOIL 0.33							
Compact, brown **SILTY SAND** 0.66		AU	1				
Compact, brown **SILTY SAND to SANDY SILT** 1.12		SS	2	67	24	1	103.98
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders		SS	3	67	32	2	102.98
2.54		SS	4	80	50+		

End of Borehole

Practical refusal to augering at 2.54m depth

(GWL @ 1.32m - Jan. 11, 2022)



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REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. **PG5570**

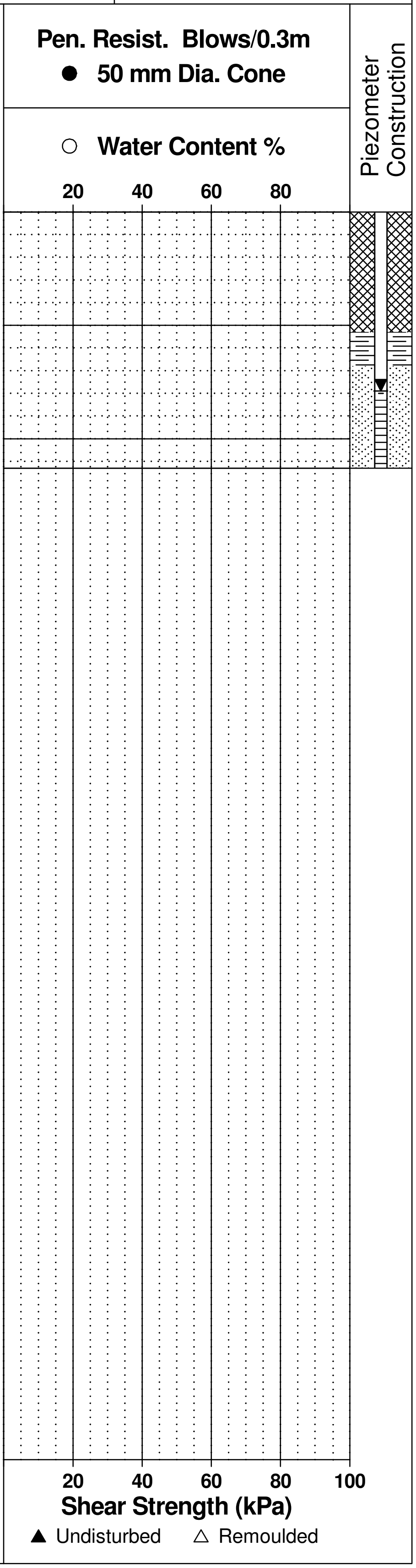
HOLE NO. **BH12-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.05
TOPSOIL 0.36						
Compact, brown **SILTY SAND** 0.69	AU	1				
Compact, brown **SILTY SAND to SANDY SILT** 1.45	SS	2	67	13	1	103.05
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders 2.26	SS	3	17	36	2	102.05

End of Borehole

Practical refusal to augering at 2.26m depth

(GWL @ 1.58m - Jan. 11, 2022)



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

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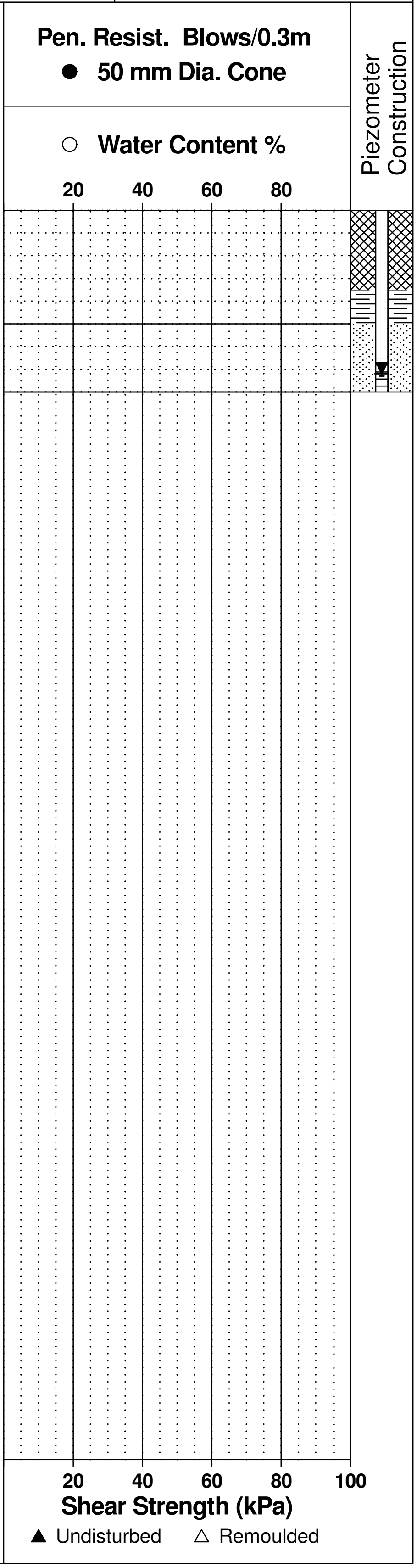
BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. PG5570

HOLE NO. BH13-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.54
TOPSOIL 0.36							
Loose, brown **SILTY SAND to SANDY SILT**		AU	1				
		SS	2	25	6	1	102.54
1.60		SS	3	0	50+		
End of Borehole Practical refusal to augering at 1.60m depth (GWL @ 1.44m - Jan. 11, 2022)							



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DATUM Geodetic

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BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. **PG5570**

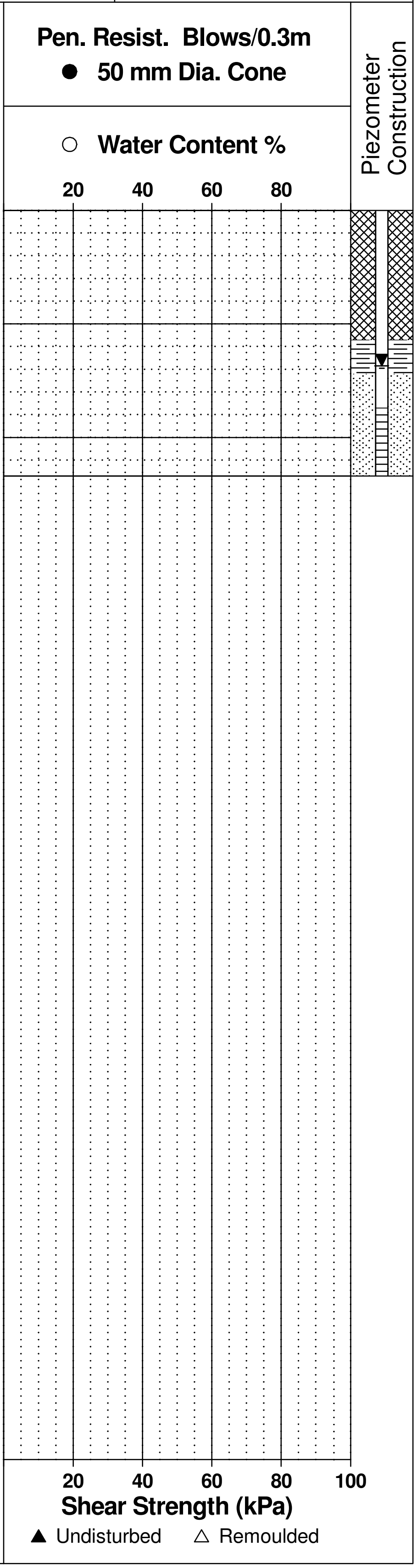
HOLE NO. **BH14-21**

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.28
TOPSOIL	0.36						
Loose, brown **SILTY SAND**	0.69	AU	1				
Loose, brown **SILTY SAND to SANDY SILT**	1.45	SS	2	67	6	1	102.28
GLACIAL TILL: Loose to dense, brown silty sand with clay, gravel, cobbles and boulders	2.34	SS	3	25	7	2	101.28
		SS	4	0	50+		

End of Borehole

Practical refusal to augering at 2.34m depth

(GWL @ 1.37m - Jan. 11, 2022)



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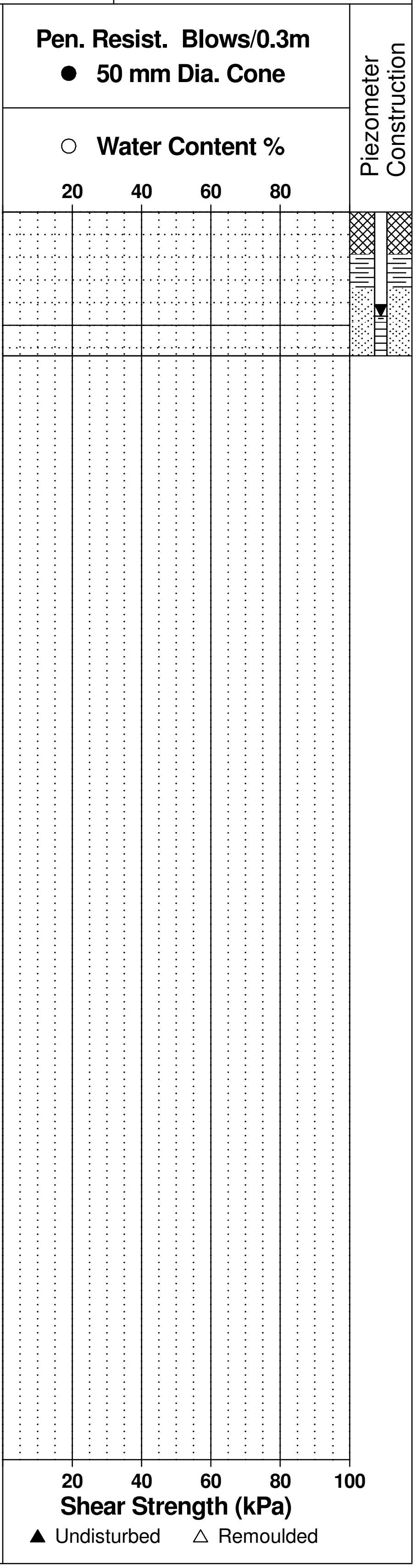
BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. PG5570

HOLE NO. BH15-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.08
TOPSOIL 0.30							
Compact, brown **SILTY SAND to SANDY SILT** 0.91		AU	1				
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders 1.27		SS	2	63	19	1	102.08
End of Borehole							
Practical refusal to augering at 1.27m depth							
(GWL @ 0.92m - Jan. 11, 2022)							



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BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. PG5570

HOLE NO. BH16-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.19
TOPSOIL 0.25							
Compact, brown **SILTY SAND,** trace gravel 0.69		AU	1				
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders 1.50		SS	2		22	1	103.19
End of Borehole Practical refusal to augering at 1.50m depth (GWL @ 1.32m - Jan. 11, 2022)							

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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DATE December 16, 2021

FILE NO. **PG5570**

HOLE NO. **BH17-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.42
TOPSOIL 0.28						
	AU	1				
Loose, brown **SILTY SAND to SANDY SILT**					1	103.42
	SS	2	75	5		
1.98	SS	3	91	11		
					2	102.42
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders 2.16						

End of Borehole

Practical refusal to augering at 2.16m depth

(GWL @ 1.25m - Jan. 11, 2022)

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. PG5570

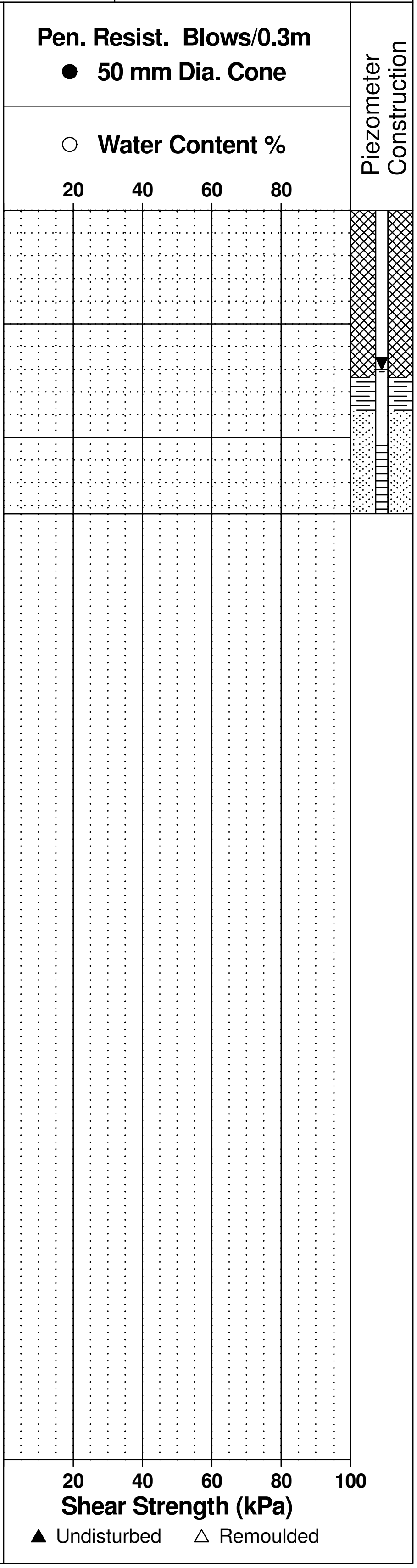
HOLE NO. BH18-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.06
TOPSOIL 0.30							
Compact, brown **SILTY SAND to SANDY SILT**		AU	1				
		SS	2	75	10	1	104.06
		SS	3	100	19	2	103.06
- grey by 2.0m depth 2.67		SS	4		50+		

End of Borehole

Practical refusal to augering at 2.67m depth

(GWL @ 1.40m - Jan. 11, 2022)



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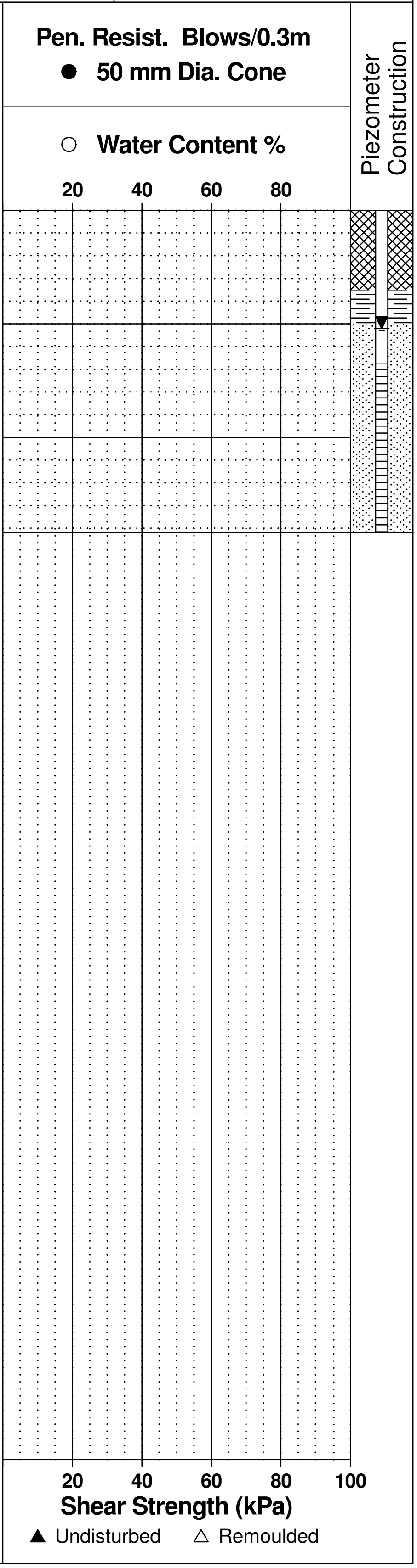
DATE December 16, 2021

FILE NO. **PG5570**

HOLE NO. **BH19-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	101.85
TOPSOIL 0.20	AU	1				
Compact, brown **SILTY SAND to SANDY SILT** 1.73	SS	2	50	12	1	100.85
Loose, brown **SILTY SAND,** some clay 2.59	SS	3	75	8	2	99.85
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders 2.84	SS	4	100	12		
End of Borehole Practical refusal to augering at 2.84m depth (GWL @ 1.04m - Jan. 11, 2022)						



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DATE December 17, 2021
FILE NO. **PG5570**
HOLE NO. **BH20-21**

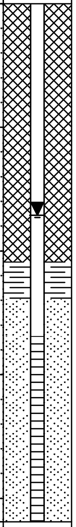
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.23				
Compact to loose, brown **SILTY SAND to SANDY SILT**		AU	1		
		SS	2	58	11
	2.44	SS	3	42	7
Interlayered grey **SANDY SILT** and grey **SILTY CLAY**	3.20	SS	4	75	3
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders		SS	5	67	23
	4.19	SS	6		50+
End of Borehole Practical refusal to augering at 4.19m depth (GWL @ 1.71m - Jan. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	102.25
1	101.25
2	100.25
3	99.25
4	98.25

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80


Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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DATE December 17, 2021

FILE NO. PG5570

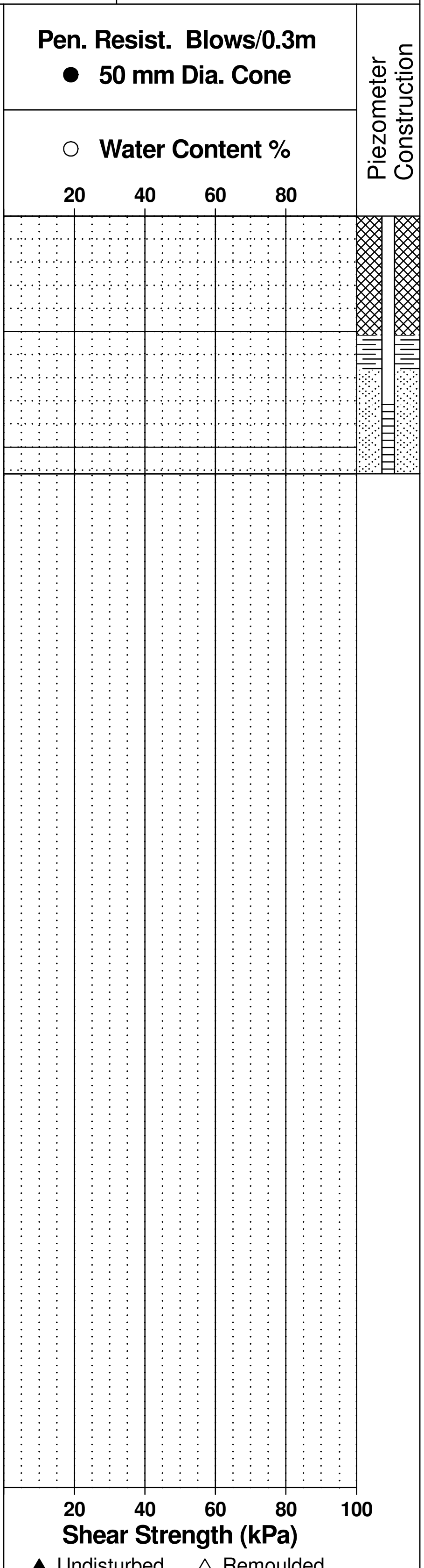
HOLE NO. BH21-21

SOIL DESCRIPTION	Layer depth (m)	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	102.92
TOPSOIL	0.25	■						
Loose, brown **SILTY SAND to SANDY SILT**	1.07	▥	AU	1				
			SS	2	42	36	1	101.92
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders	2.23	^^^	SS	3	50	71	2	100.92

End of Borehole

Practical refusal to augering at 2.23m depth

(Piezometer damaged - Jan. 11, 2022)



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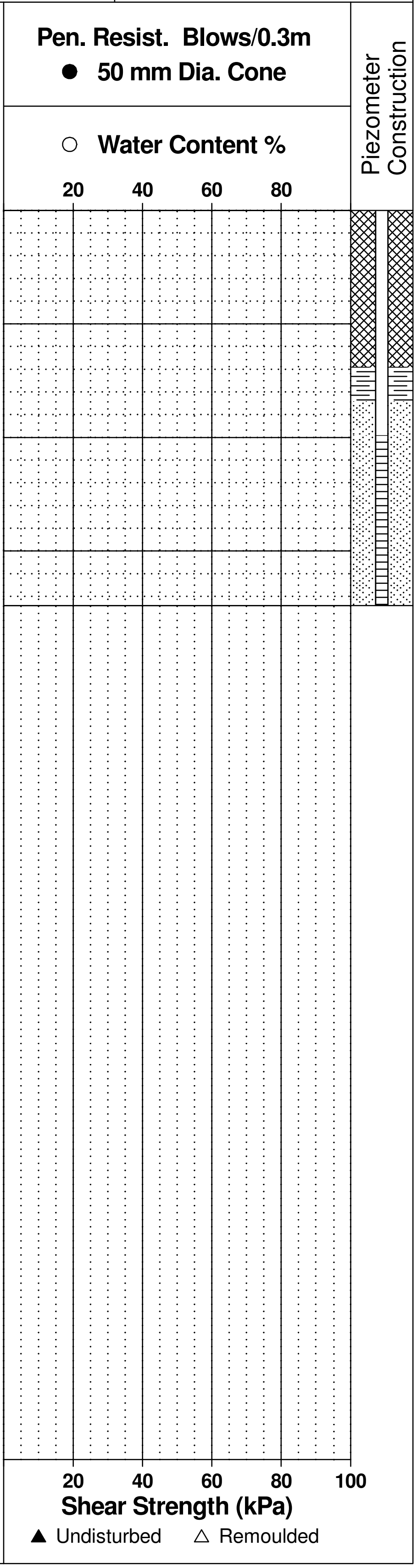
DATE December 20, 2021
FILE NO. PG5570
HOLE NO. BH22-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.98
TOPSOIL 0.20							
Loose, brown **SILTY SAND,** trace gravel 0.69		AU	1				
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders		SS	2	100	22	1	101.98
		SS	3	92	29	2	100.98
		SS	4	83	46	3	99.98
3.48		SS	5	50	50+		
End of Borehole Practical refusal to augering at 3.48m depth.							



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DATUM Geodetic

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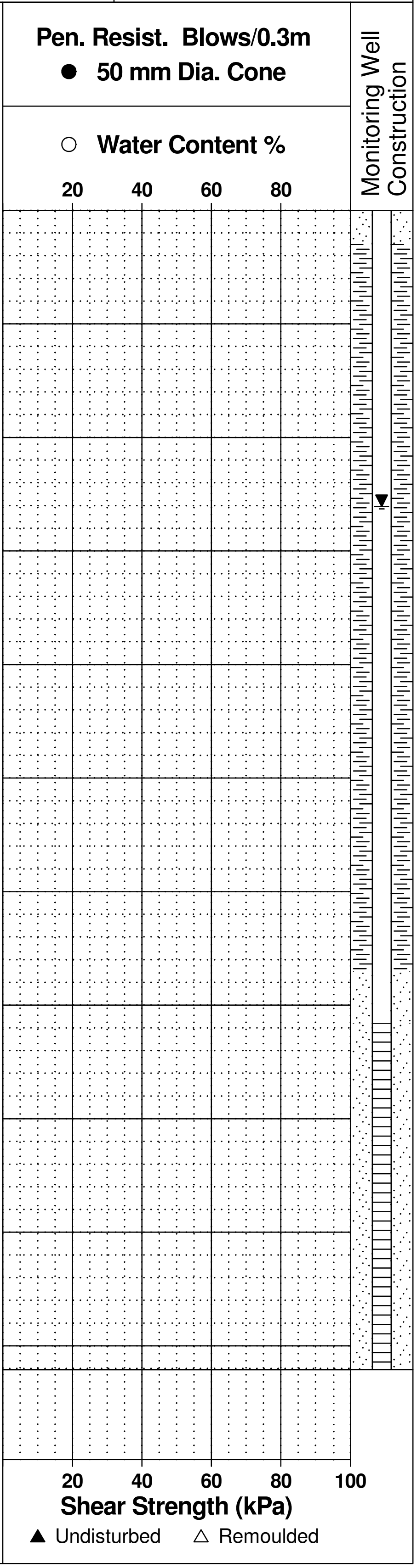
BORINGS BY Track-Mount Power Auger

DATE January 10, 2022

FILE NO. PG5570

HOLE NO. BH22A-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.98
TOPSOIL 0.20						
Loose, brown **SILTY SAND,** trace gravel 0.69	AU	1				
	SS	2	100	22	1	101.98
	SS	3	92	29	2	100.98
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	SS	4	83	46		
	SS	5	50	50+	3	99.98
	RC	1	77		4	98.98
	RC	2	14		5	97.98
5.97					6	96.98
	RC	3	100	94	7	95.98
BEDROCK: Excellent quality, grey dolostone interbedded with grey limestone	RC	4	100	100	8	94.98
	RC	5	100	100	9	93.98
10.21					10	92.98
End of Borehole (GWL @ 2.49m - Jan. 11, 2022)						



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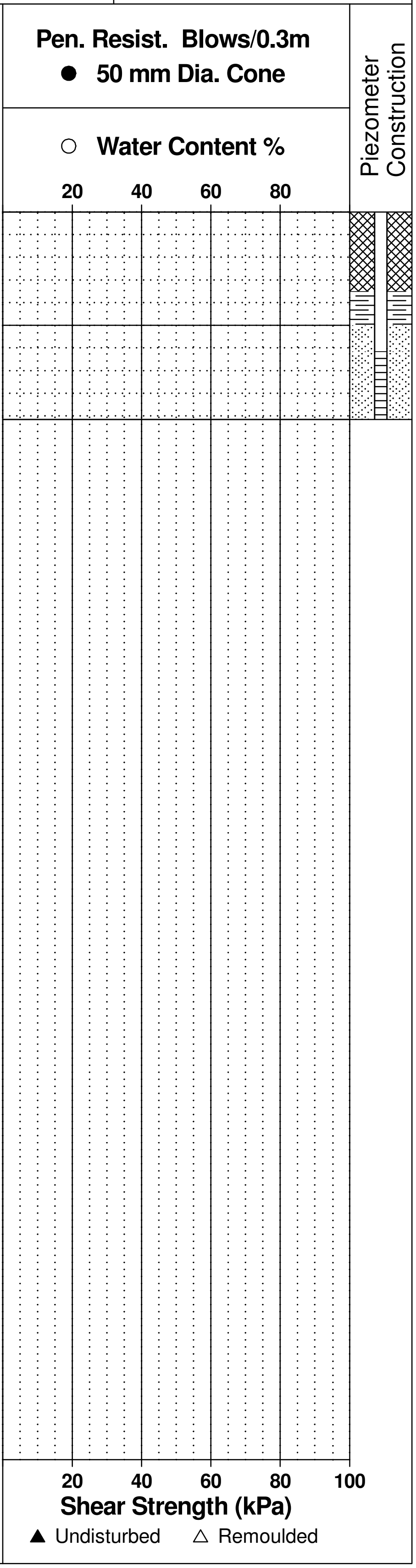
DATE December 20, 2021

FILE NO. PG5570

HOLE NO. BH23-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.38
TOPSOIL 0.28	■						
Stiff, brown **SILTY CLAY,** some sand 1.12	▨	AU	1				
		SS	2	25	32	1	101.38
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders, trace clay 1.83		SS	3	55			
End of Borehole Practical refusal to augering at 1.83m depth (Piezometer damaged - Jan. 11, 2022)							



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DATUM Geodetic

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BORINGS BY Track-Mount Power Auger

DATE December 20, 2021

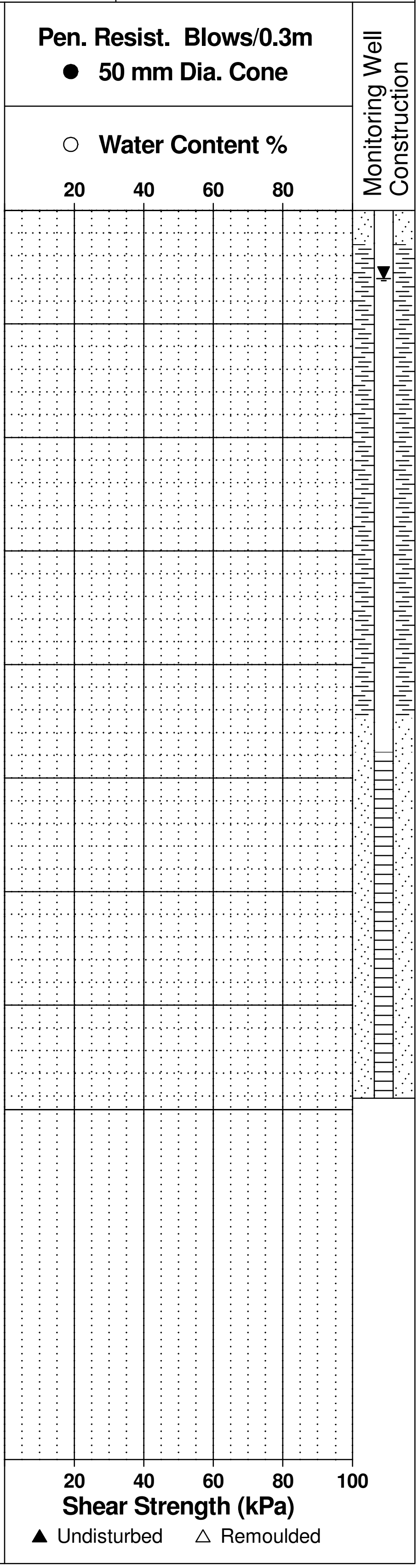
FILE NO. **PG5570**

HOLE NO. **BH24-21**

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.30
Loose to dense, brown **SILTY SAND to SANDY SILT**	1.83
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders - boulders cored from 2.46 to 4.42m depth	4.42
BEDROCK: Good to excellent quality, grey limestone interbedded with dolostone - 15mm thick mud seam at 5.25m depth	7.92
End of Borehole (GWL @ 0.67m - Jan. 11, 2022)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	8
SS	3	75	32
SS	4	50	50+
RC	1	100	
RC	2	19	
RC	3	100	81
RC	4	100	100
RC	5	100	100

DEPTH (m)	ELEV. (m)
0	103.07
1	102.07
2	101.07
3	100.07
4	99.07
5	98.07
6	97.07
7	96.07



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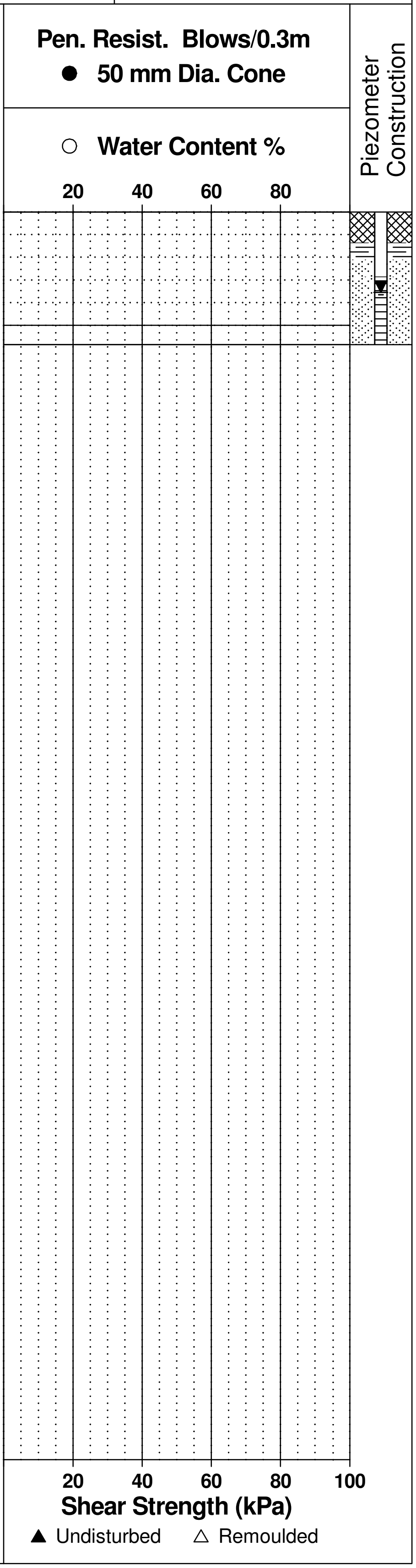
DATE December 21, 2021

FILE NO. **PG5570**

HOLE NO. **BH25-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.73
TOPSOIL 0.25							
Loose, brown **SILTY SAND,** trace clay and gravel		AU	1				
1.17		SS	2	71	50+	1	101.73
End of Borehole Practical refusal to augering at 1.17m depth (GWL @ 0.71m - Jan. 11, 2022)							



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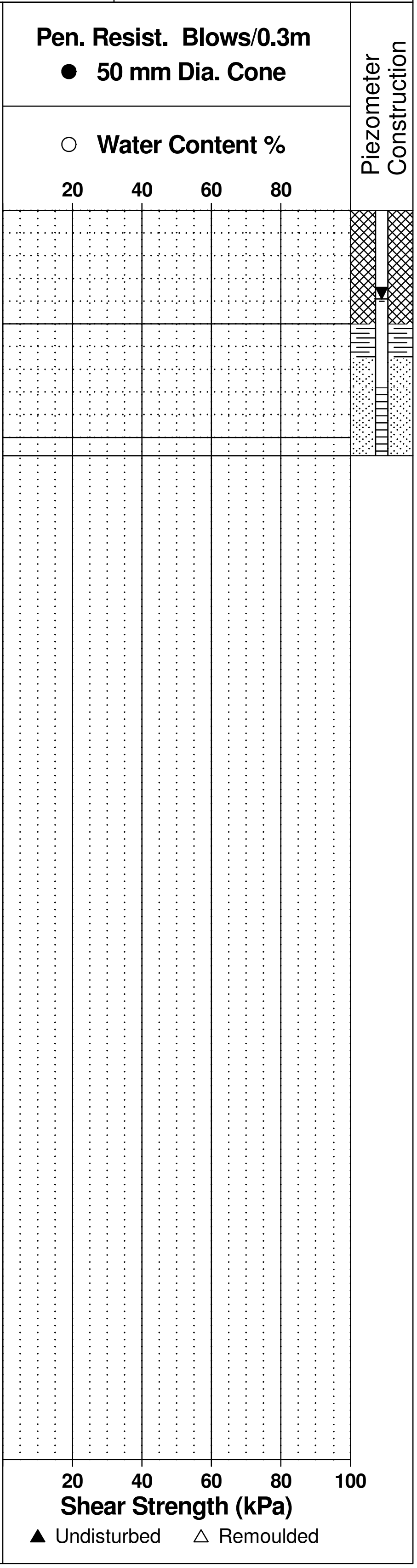
DATE December 21, 2021
FILE NO. PG5570
HOLE NO. BH26-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.04
TOPSOIL 0.25						
Stiff, brown **SILTY CLAY,** some sand 0.69	AU	1				
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders, trace clay	SS	2	40	50+	1	102.04
	SS	3	61	33	2	101.04
2.16						
End of Borehole Practical refusal to augering at 2.16m depth (GWL @ 0.78m - Jan. 11, 2022)						



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DATE December 21, 2021

FILE NO. PG5570

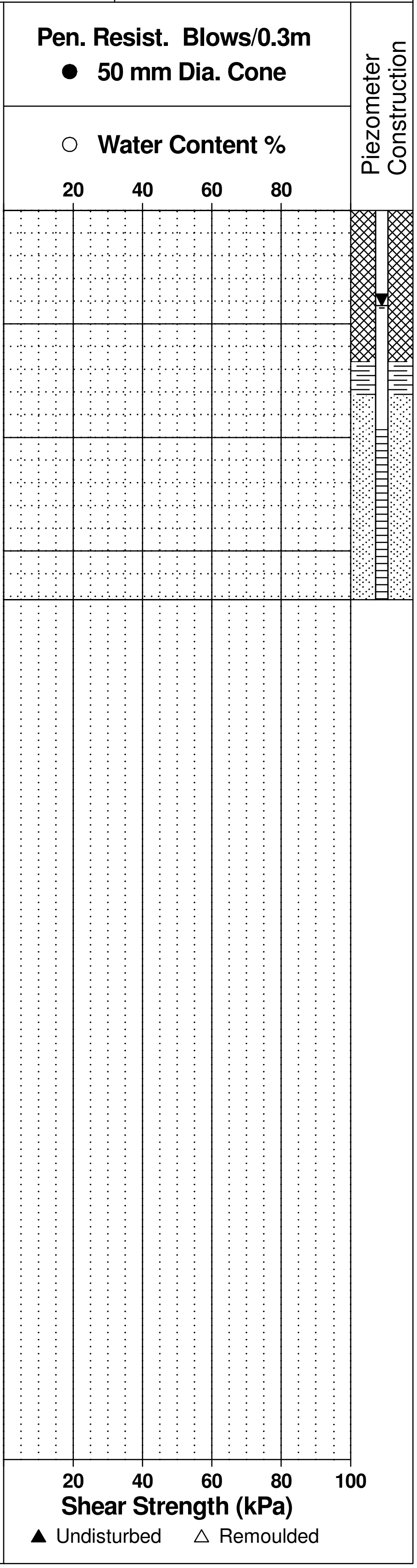
HOLE NO. BH27-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.71
TOPSOIL 0.28							
		AU	1				
Compact to loose, brown **SILTY SAND to SANDY SILT,** trace clay		SS	2	67	19	1	101.71
		SS	3	83	21	2	100.71
- grey by 2.4m depth		SS	4	50	9	3	99.71
3.12 **GLACIAL TILL:** Very loose, grey silty sand with gravel, cobbles and boulders 3.43		SS	5	86	3		

End of Borehole

Practical refusal to augering at 3.43m depth

(GWL @ 0.84m - Jan. 11, 2022)



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BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. **PG5570**

HOLE NO. **BH28-21**

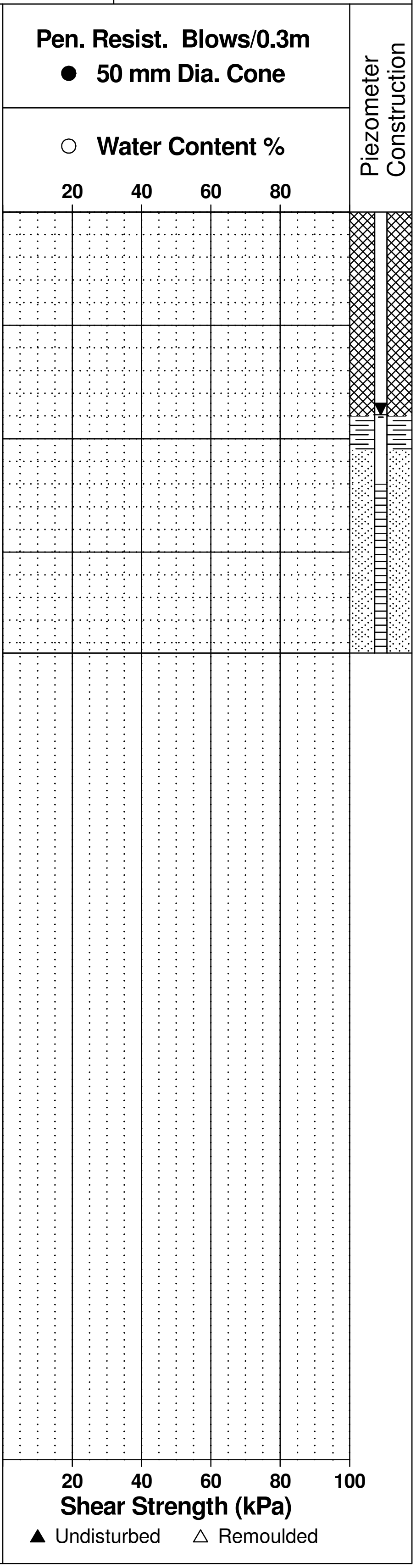
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30				
Loose, brown **SILTY SAND to SANDY SILT,** trace clay	2.21	AU	1		
		SS	2	42	7
		SS	3	58	8
Interbedded layers of grey **SILTY SAND** and grey **SILTY CLAY**	2.97	SS	4	100	2
GLACIAL TILL: Very loose, grey silty sand with clay, gravel and cobbles	3.89	SS	5	100	3
		SS	6		50+

End of Borehole

Practical refusal to augering at 3.89m depth

(GWL @ 1.79m - Jan. 11, 2022)

DEPTH (m)	ELEV. (m)
0	101.85
1	100.85
2	99.85
3	98.85



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BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. **PG5570**

HOLE NO. **BH29-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.31
TOPSOIL 0.28						
Loose to very loose, brown **SILTY SAND to SANDY SILT,** trace clay	AU	1				
	SS	2	50	9	1	101.31
	SS	3	67	8	2	100.31
- grey by 1.9m depth	SS	4	67	4	3	99.31
- intermittent layers of grey silty clay by 3.0m depth	SS	5	58	2		
3.96	SS	6	67			
End of Borehole						

Practical refusal to augering at 3.96m depth

(Piezometer damaged - Jan. 11, 2022)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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DATE December 21, 2021

FILE NO. **PG5570**

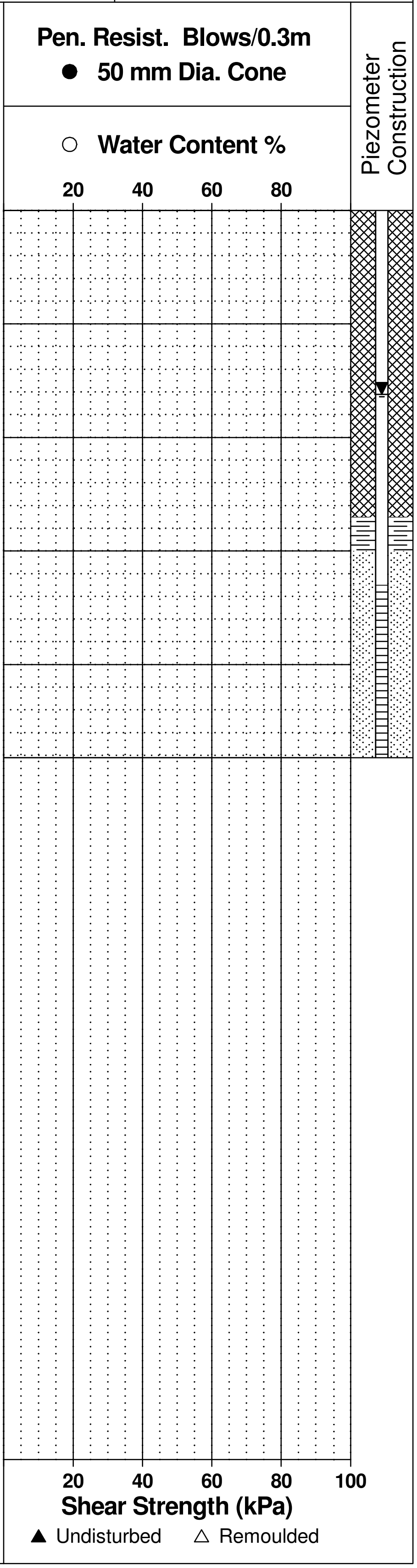
HOLE NO. **BH30-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.44
TOPSOIL 0.25						
Compact to very loose, brown **SILTY SAND to SANDY SILT,** trace clay - grey by 2.0m depth	AU	1				
	SS	2	50	12	1	101.44
	SS	3	33	10	2	100.44
	SS	4	92	1	3	99.44
3.45	SS	5	83	2		
GLACIAL TILL: Very loose to compact, grey silty sand with gravel, cobbles and boulders, trace clay	SS	6	33	24	4	98.44
4.82	SS	7	50	50+		

End of Borehole

Practical refusal to augering at 4.82m depth

(GWL @ 1.62m - Jan. 11, 2022)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. **PG5570**

HOLE NO. **BH31-21**

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL 0.36

Compact to loose, brown **SILTY SAND to SANDY SILT,** trace clay

- grey by 3.2m depth

4.72

GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders

6.12

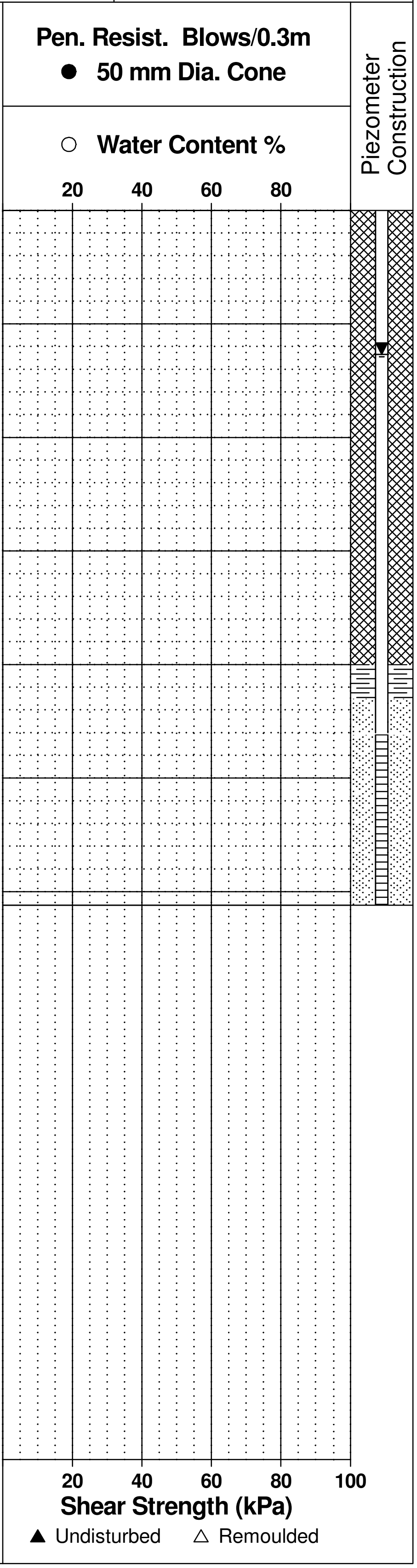
End of Borehole

Practical refusal to augering at 6.12m depth

(GWL @ 1.27m - Jan. 11, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	14
SS	3	50	22
SS	4	42	9
SS	5	58	5
SS	6	42	12
SS	7	58	37
SS	8		58
SS	9	0	50+

DEPTH (m)	ELEV. (m)
0	103.43
1	102.43
2	101.43
3	100.43
4	99.43
5	98.43
6	97.43



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. **PG5570**

HOLE NO. **BH32-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.74
TOPSOIL 0.15							
		AU	1				
Compact to dense, brown **SILTY SAND to SANDY SILT**		SS	2	67	39	1	102.74
- grey 1.4m depth		SS	3	67	26	2	101.74
2.21							
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders 2.36		SS	4	50	50+		

End of Borehole

Practical refusal to augering at 2.36m depth

(GWL @ 1.62m - Jan. 11, 2022)

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

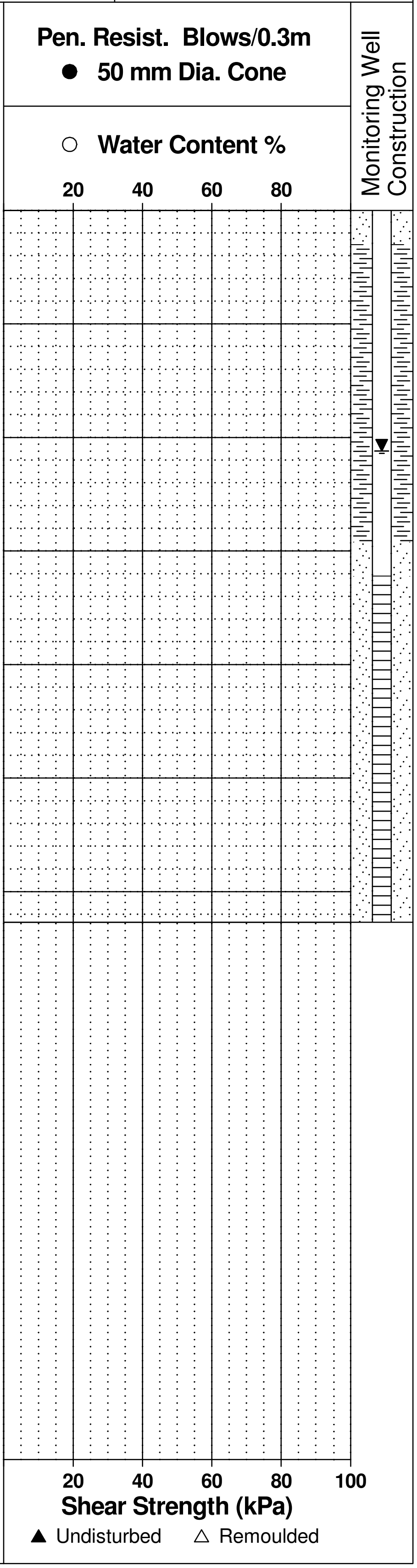
BORINGS BY Track-Mount Power Auger

DATE December 22, 2021

FILE NO. PG5570

HOLE NO. BH33-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.70
Compact, brown **SILTY SAND,** trace clay and organics	AU	1				
	SS	2	50	13	1	103.70
1.45						
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders	SS	3	8	11	2	102.70
	RC	1	30			
2.74						
BEDROCK: Good to excellent quality, grey limestone - 25mm thick mud seam at 3.7m depth - 30mm thick mud seam at 3.8m depth	RC	2	100	73	3	101.70
					4	100.70
	RC	3	95	85	5	99.70
	RC	4	100	100	6	98.70
6.27						
End of Borehole (GWL @ 1.84m - Jan. 11, 2022)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 22, 2021

FILE NO. PG5570

HOLE NO. BH34-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.65
TOPSOIL	0.25						
Compact to loose, brown **SILTY SAND to SANDY SILT**		SS	5	17	8		
		AU	1				
		SS	2	42	10	1	101.65
		SS	3	25	9		
	2.21					2	100.65
GLACIAL TILL: Very loose to loose, grey silty sand with gravel, cobbles and boulders, trace clay		SS	4	17	2		
						3	99.65
		RC	1	31		4	98.65
	5.21	RC	2	100	100	5	97.65
BEDROCK: Excellent quality, grey limestone interbedded with dolostone						6	96.65
	6.61	RC	3	100	100		
End of Borehole							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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Prop. Residential Development - 6115 Flewellyn Road
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 7, 2022

FILE NO. **PG5570**

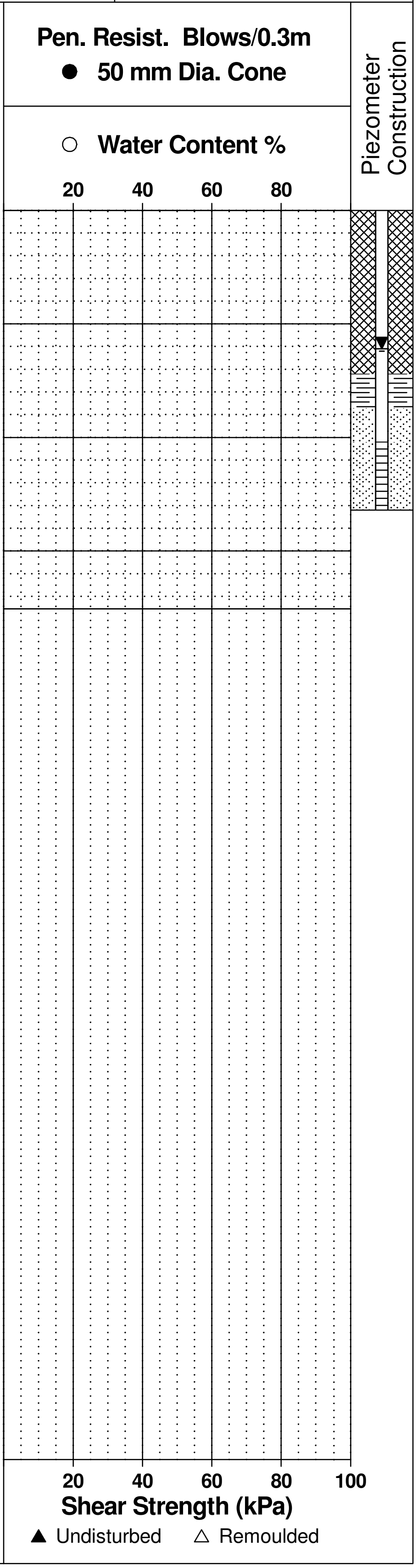
HOLE NO. **BH35-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.03
TOPSOIL 0.28		AU	1				
Loose to compact, brown **SILTY SAND to SANDY SILT**		SS	2	50	7	1	104.03
1.68		SS	3	50	25	2	103.03
GLACIAL TILL: Compact to very dense, grey silty sand with gravel, cobbles and boulders		SS	4	25	56	3	102.03
3.51		SS	5	67	50+		

End of Borehole

Practical refusal to augering at 3.51m depth.

(GWL @ 1.22m - Jan. 11, 2022)



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SOIL PROFILE AND TEST DATA

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Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE January 7, 2022
FILE NO. PG5570
HOLE NO. BH36-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.79
TOPSOIL (to 0.30)						
Compact, brown **SILTY SAND to SANDY SILT** (0.30 to 1.45)	AU	1				
	SS	2	42	15	1	101.79
GLACIAL TILL: Very dense to compact, brown silty sand with gravel, cobbles and boulders (1.45 to 2.90)	SS	3	60	50+		
					2	100.79
	SS	4	8	15		
End of Borehole (2.90)						

End of Borehole

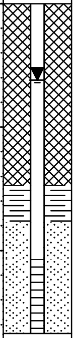
Practical refusal to augering at 2.90m depth.

(GWL @ 0.62m - Jan. 11, 2022)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

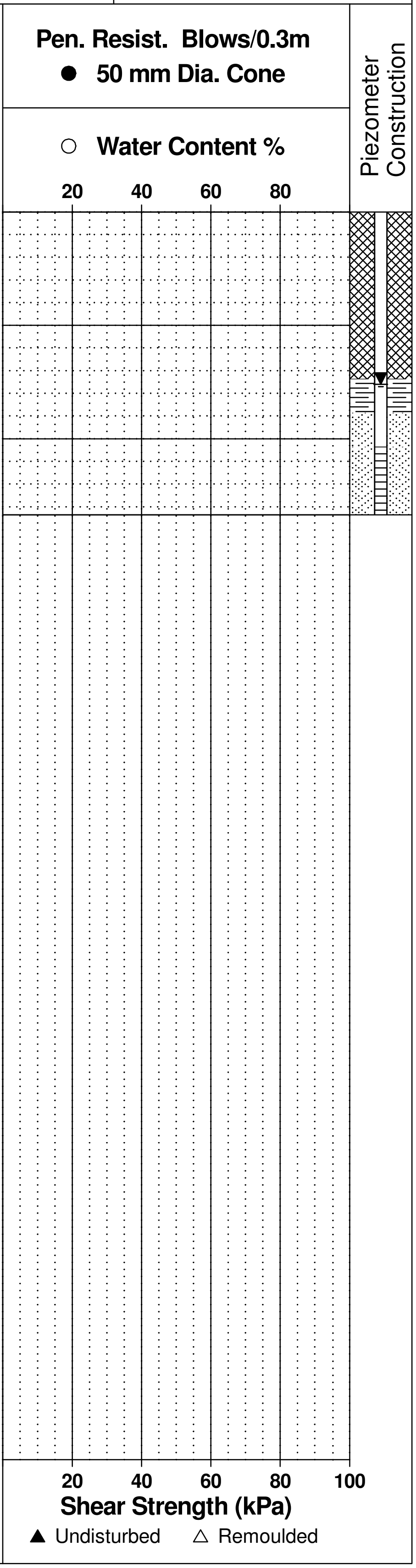
BORINGS BY Track-Mount Power Auger

DATE January 7, 2022

FILE NO. PG5570

HOLE NO. BH37-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.36						0	103.54
		AU	1				
Compact to dense, brown **SILTY SAND to SANDY SILT**		SS	2	42	22	1	102.54
2.21		SS	3	58	34	2	101.54
GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles and boulders 2.67		SS	4	50	50+		
End of Borehole Practical refusal to augering at 2.67m depth. (GWL @ 1.52m - Jan. 11, 2022)							



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SOIL PROFILE AND TEST DATA

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Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 7, 2022

FILE NO. **PG5570**

HOLE NO. **BH38-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.62
FILL: Crushed stone and gravel 0.15						
	AU	1				
Dense to compact, brown **SILTY SAND to SANDY SILT**	SS	2		32	1	102.62
	SS	3		24		
- grey by 2.0m depth					2	101.62
	SS	4	100	50+		
2.64						
End of Borehole						

Practical refusal to augering at 2.64m depth.

(GWL @ 1.94m - Jan. 11, 2022)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

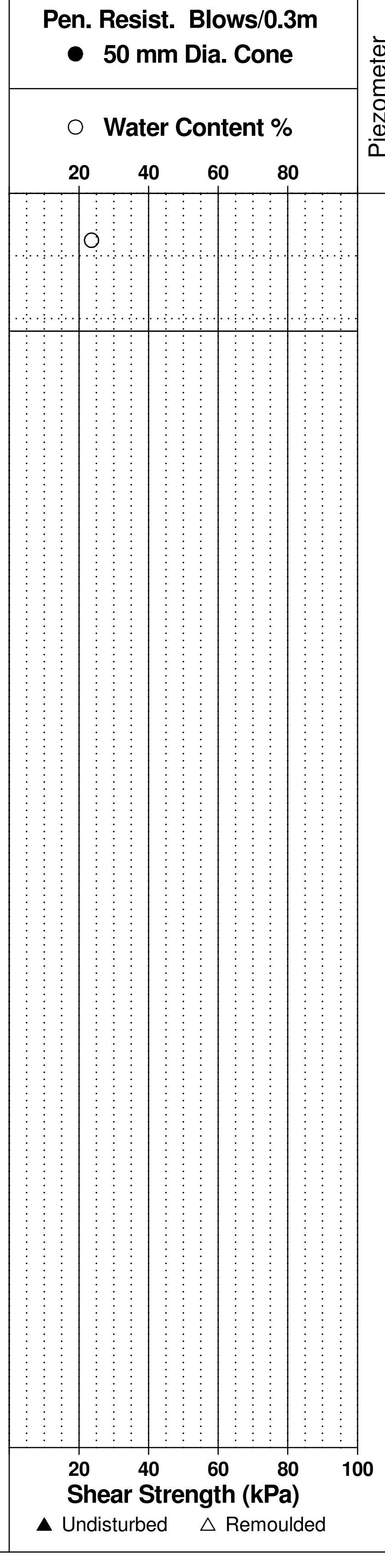
BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG5570

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	105.94	
TOPSOIL 0.15		G	1					
GLACIAL TILL: Brown silty sand with gravel and some clay 0.25		G	2					
BEDROCK Weathered interbedded limestone 0.44		G	3					
End of Test Pit								
Practical refusal to excavation at 0.44m depth								
(TP dry upon completion)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG5570

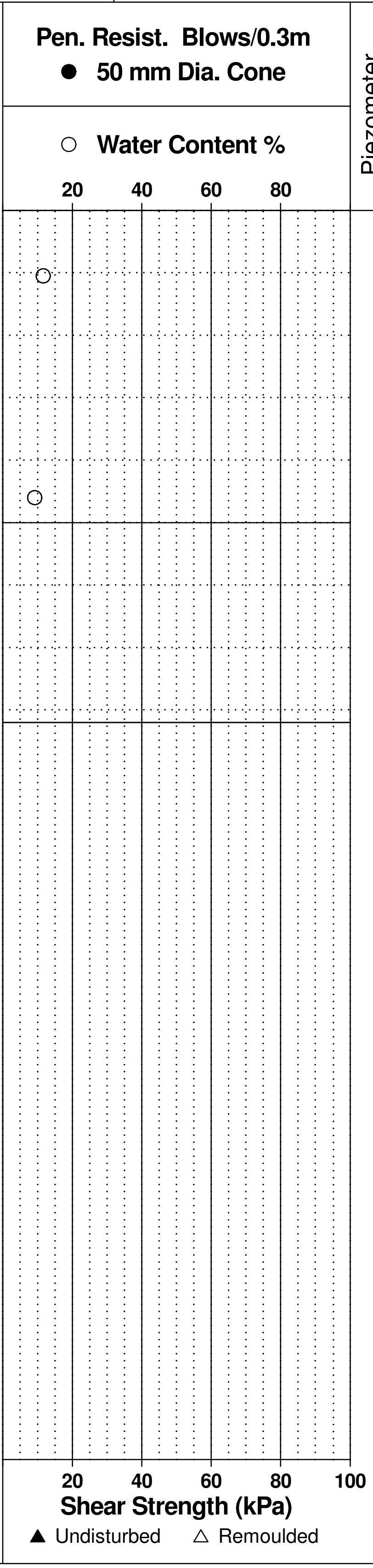
HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	105.06	
TOPSOIL 0.21		G	1					
Brown **SILTY SAND,** trace gravel 0.92		G	2					
						1	104.06	
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 1.64		G	3					

End of Test Pit

TP terminated on inferred bedrock surface at 1.64m depth

(TP dry upon completion)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

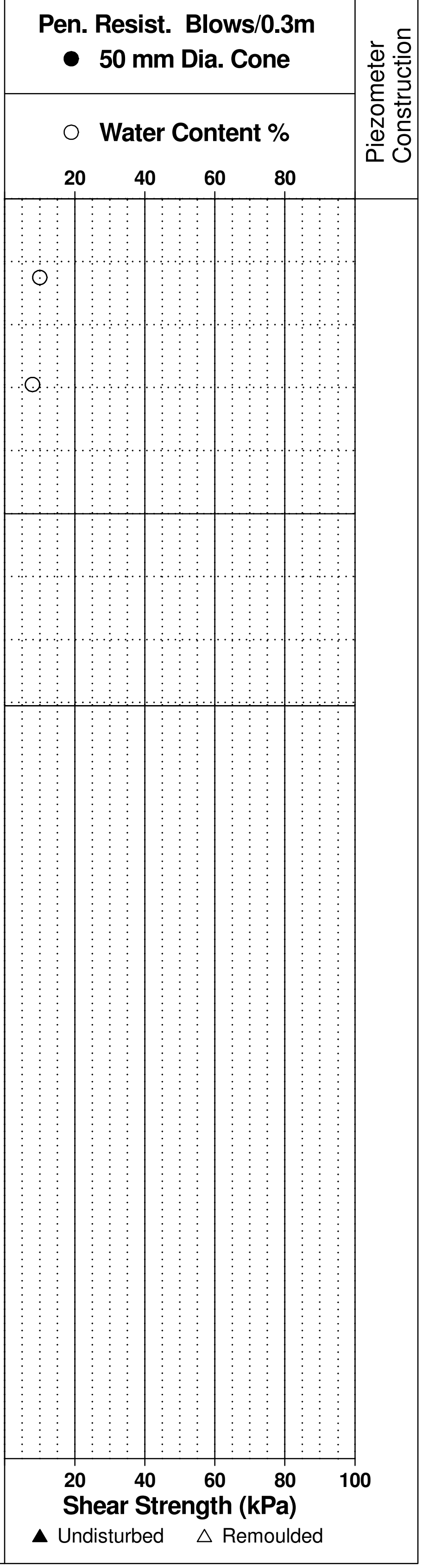
BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG5570

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.10
TOPSOIL 0.25		G	1				
Brown **SILTY SAND,** trace sea shells 0.59		G	2				
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 1.61		G	3			1	101.10
End of Test Pit TP terminated on inferred bedrock surface at 1.61m depth (TP dry upon completion)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

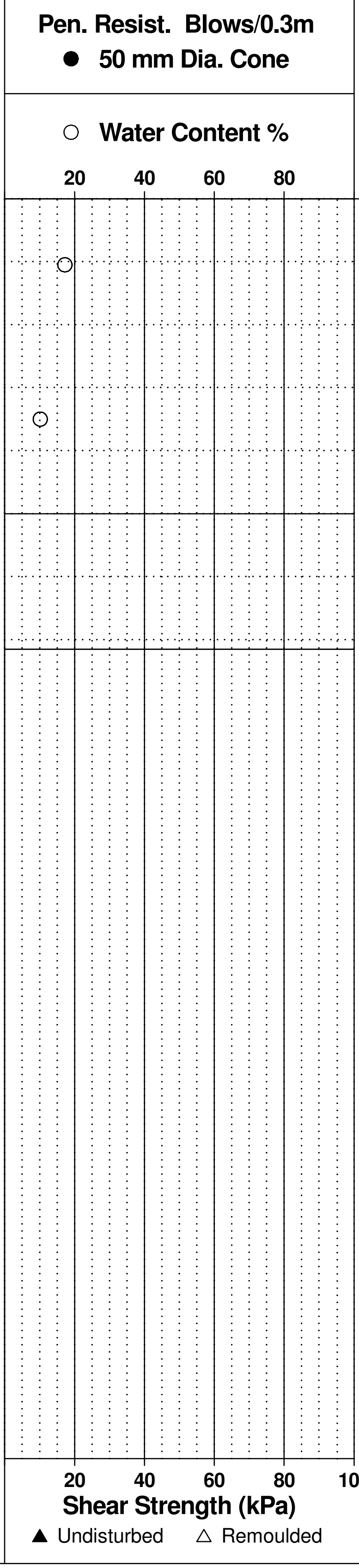
BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG5570

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	108.49	
TOPSOIL 0.21		G	1					
Brown **SILTY SAND,** trace gravel, cobble and organics 0.70		G	2					
GLACIAL TILL: Brown silty sand, some gravel, cobble, boulder, trace clay 1.43		G	3			1	107.49	
End of Test Pit Test Pit terminated on bedrock surface at 1.43m depth (TP dry upon completion)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5570**

REMARKS

HOLE NO. **TP 5**

BORINGS BY CME-55 Low Clearance Drill

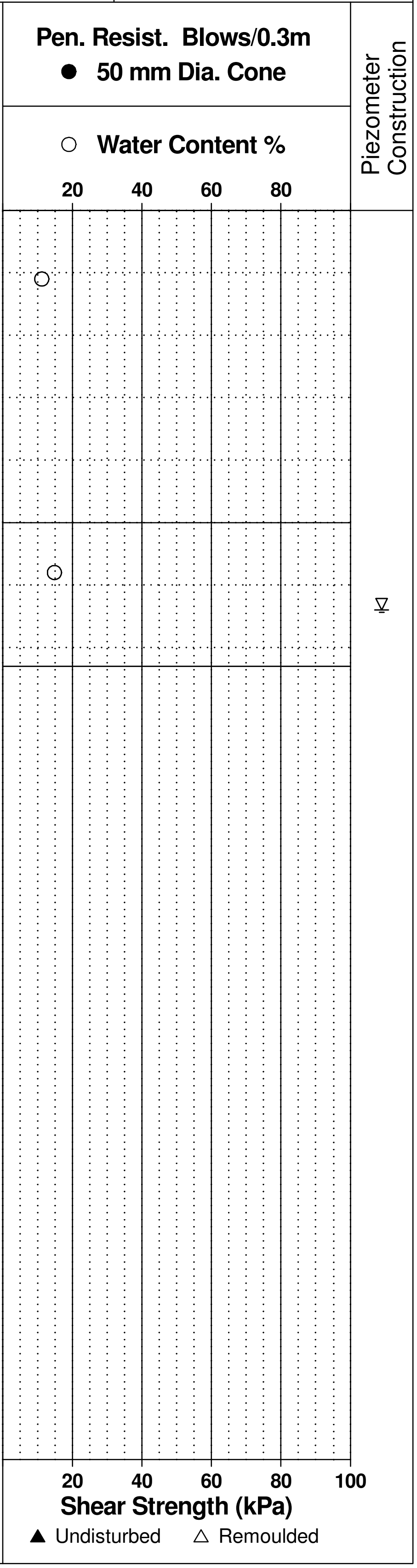
DATE November 20, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
TOPSOIL 0.22		G	1			0	108.36
Brown **SILTY SAND** 1.16		G	2			1	107.36
GLACIAL TILL: Brown silty sand, some gravel, cobble, and boulder 1.46		G	3				

End of Test Pit

TP terminated on inferred bedrock surface at 1.46m depth

(Groundwater infiltration at 1.28m - Nov 20, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG5570

HOLE NO. TP 6

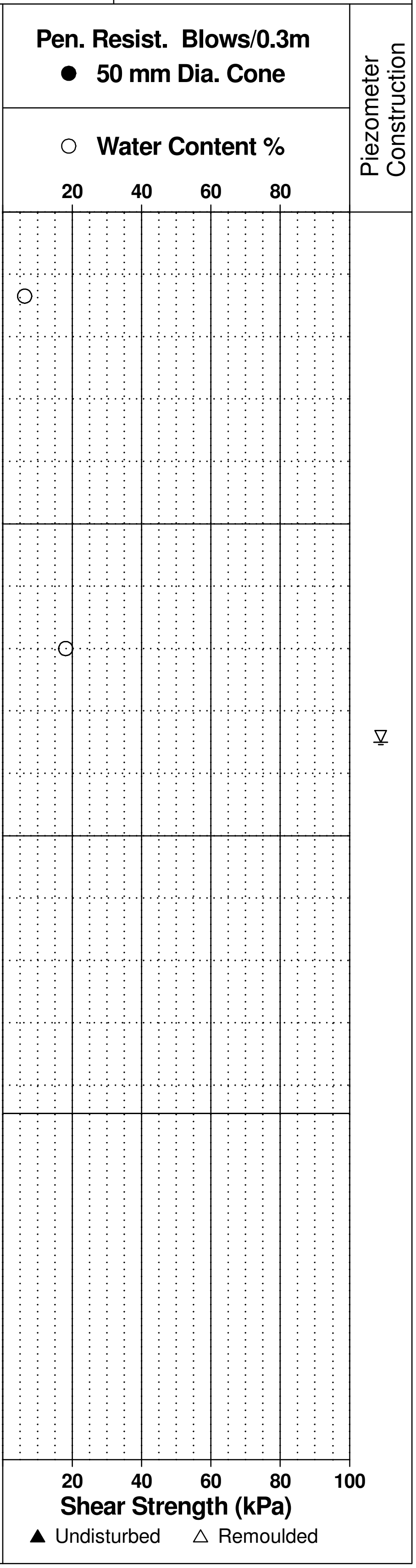
SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
TOPSOIL 0.27		G	1		
Brown **SILTY SAND,** trace cobble, boulders and seashells		G	2		
1.70		G	3		
BEDROCK: Weathered interbedded limestone 2.89		G	4		

End of Test Pit

TP terminated on inferred bedrock surface at 2.89m depth

(Groundwater infiltration at 1.70m - Nov 20, 2020)

DEPTH (m)	ELEV. (m)
0	107.91
1	106.91
2	105.91



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5570

REMARKS

HOLE NO. TP 7

BORINGS BY CME-55 Low Clearance Drill

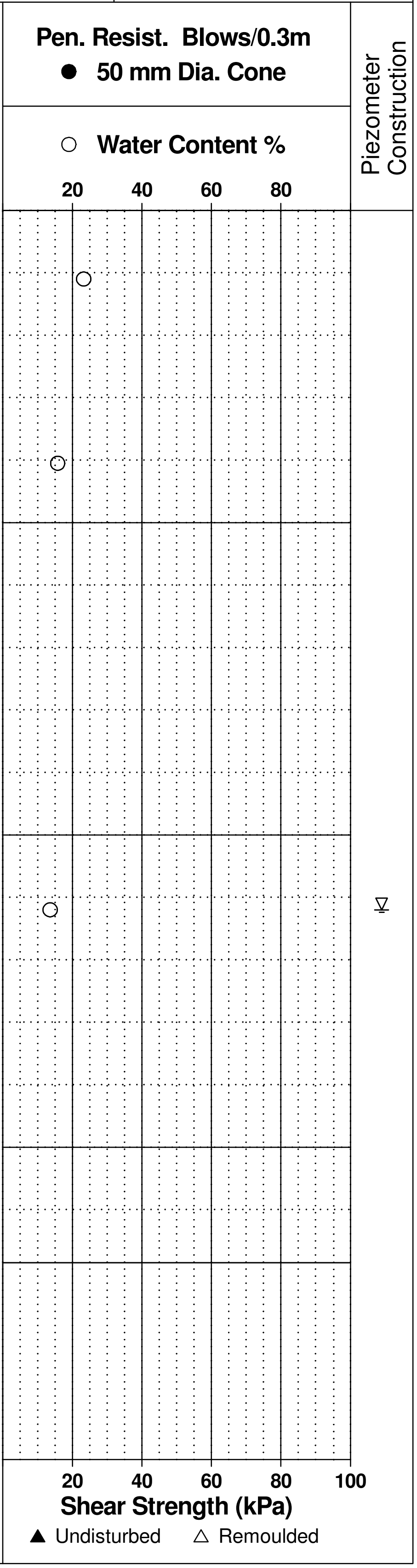
DATE November 20, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	106.31
TOPSOIL 0.22		G	1				
Brown **SILTY SAND,** trace clay 0.81		G	2				
						1	105.31
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders		G	3				
						2	104.31
		G	4				
3.37						3	103.31

End of Test Pit

TP terminated on inferred bedrock surface at 3.37m depth

(Groundwater infiltration at 2.24m - Nov 20, 2020)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. **PG5570**

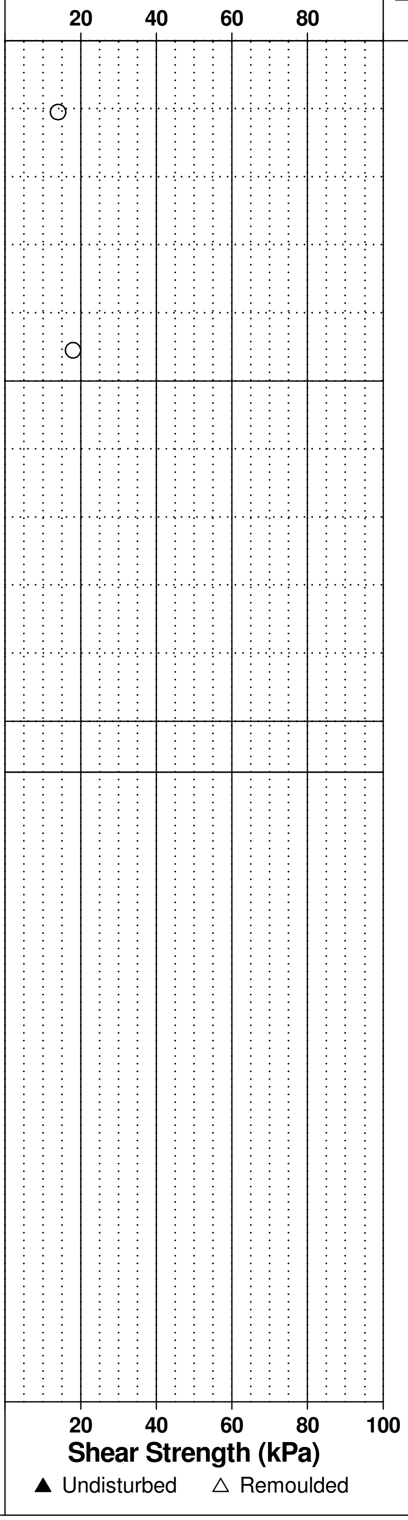
HOLE NO. **TP 8**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	105.48
TOPSOIL 0.21	■	G	1				
		G	2				
Brown **SILTY SAND,** trace clay and organics - increasing in silt content with depth						1	104.48
		G	3				
2.15						2	103.48
End of Test Pit TP terminated on inferred bedrock surface at 2.15m depth (TP dry upon completion)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

Piezometer Construction



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SOIL PROFILE AND TEST DATA

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REMARKS

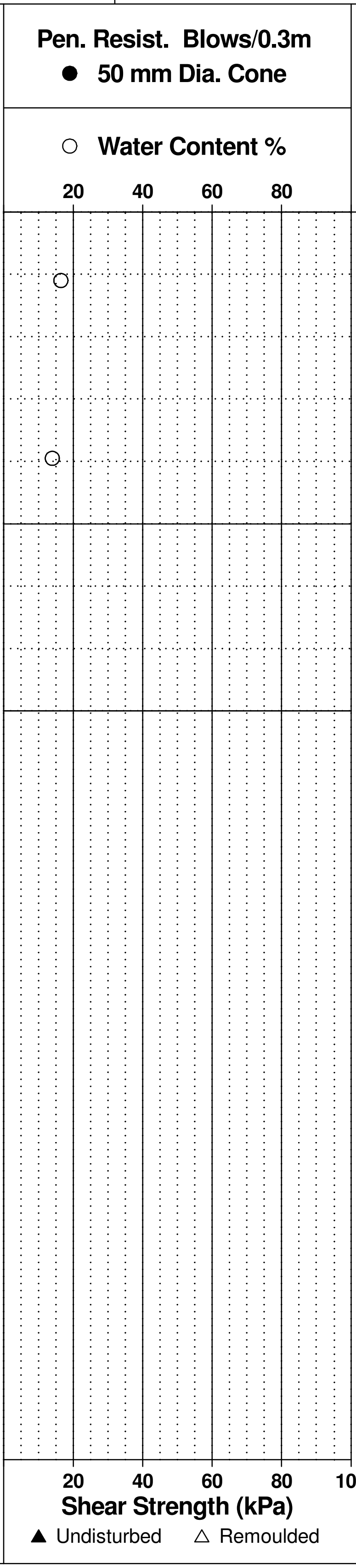
BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG5570

HOLE NO. TP 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	104.47	
TOPSOIL 0.22		G	1					
Brown **SILTY SAND,** trace organics 0.79		G	2					
GLACIAL TILL: Brown silty sand trace gravel, cobbles, and boulders 1.60		G	3			1	103.47	
End of Test Pit TP terminated on inferred bedrock surface at 1.60m depth (TP dry upon completion)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5570

REMARKS

HOLE NO. TP 10

BORINGS BY CME-55 Low Clearance Drill

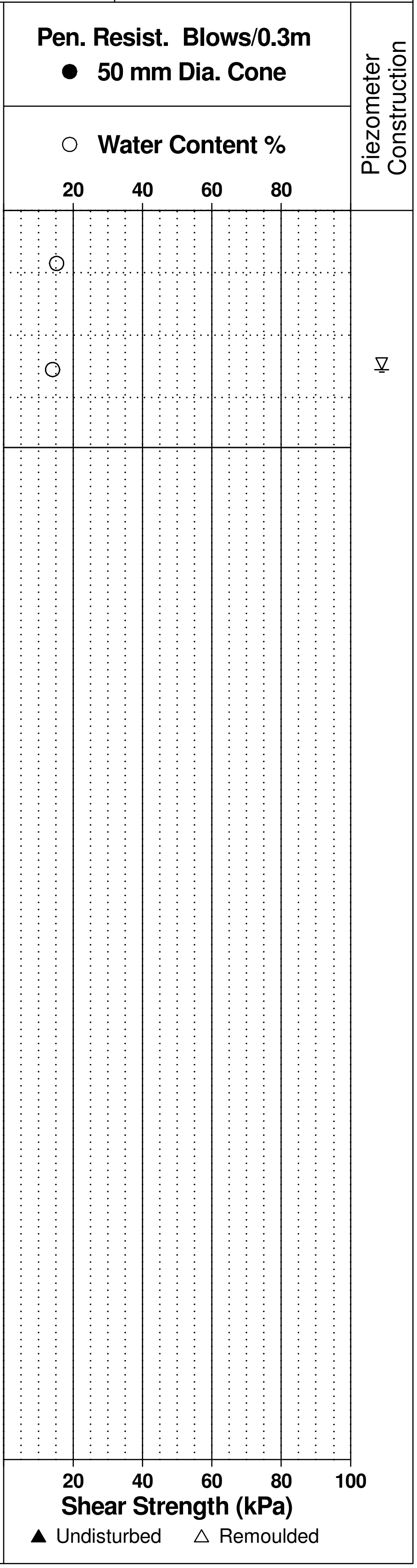
DATE December 10, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.62
TOPSOIL 0.17		G	1				
Brown **SILTY SAND,** trace gravel and cobbles 0.51		G	2				
GLACIAL TILL: Brown silty sand, with gravel, trace cobble and boulders 0.76		G	3				

End of Test Pit

TP terminated on inferred bedrock surface at 0.76m depth

(Groundwater infiltration at 0.51m - Dec 10, 2020)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5570

REMARKS

HOLE NO. TP 11

BORINGS BY CME-55 Low Clearance Drill

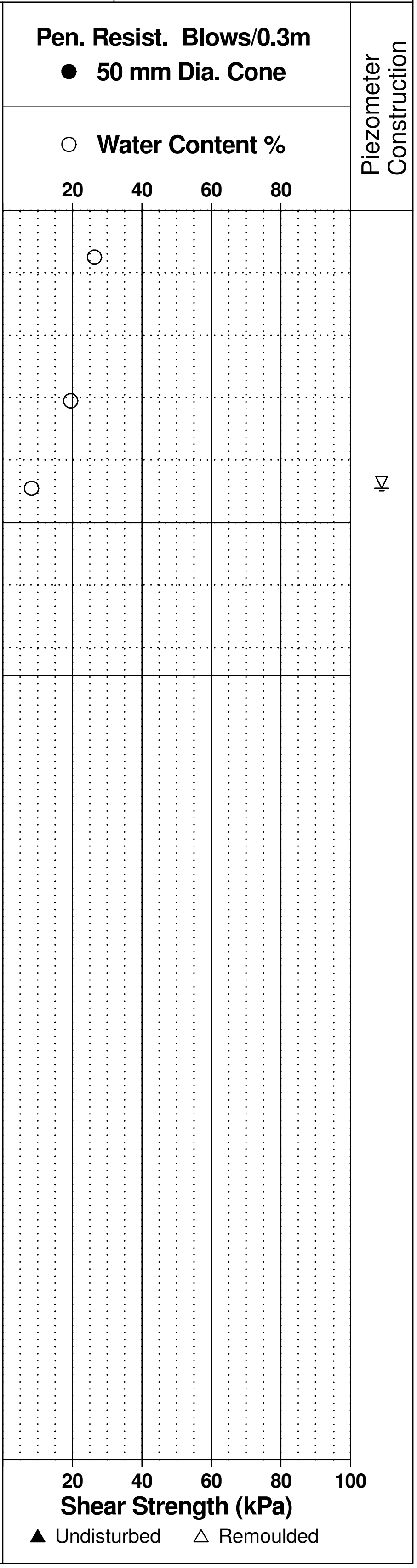
DATE December 10, 2020

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.01
TOPSOIL — 0.15	G	1				
Brown **SILTY SAND,** trace gravel — 0.89	G G	2 3				
GLACIAL TILL: Brown silty sand, with gravel, cobbles, and boulders — 1.49	G	1			1	102.01

End of Test Pit

TP terminated on inferred bedrock surface at 1.49m depth

(Groundwater infiltration at 0.89m - Dec 10, 2020)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6070 and 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 10, 2020

FILE NO. PG5570

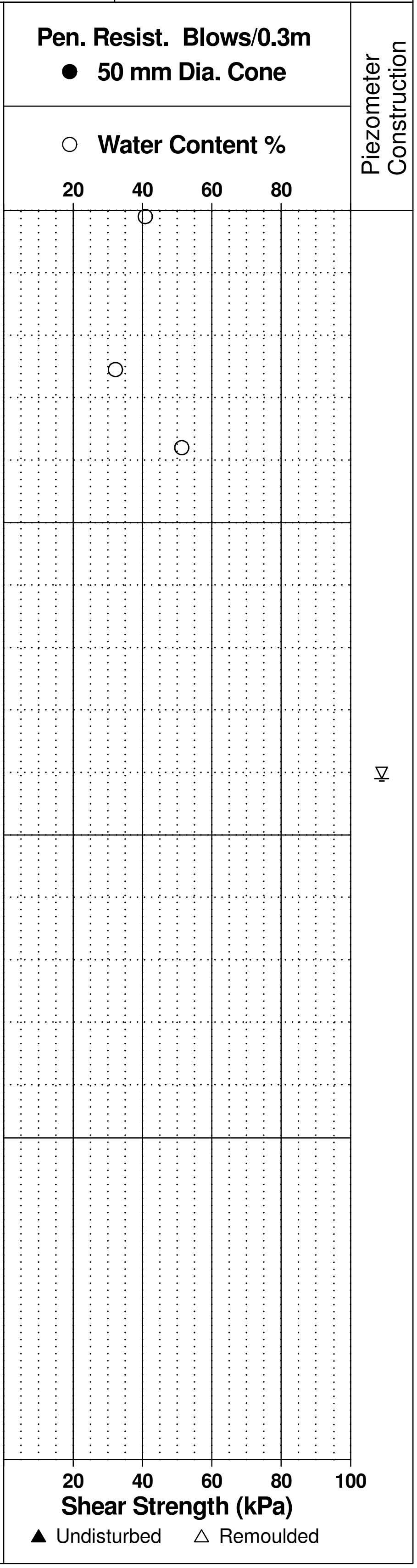
HOLE NO. TP 12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.02		G	1			0	103.21
Brown **SILTY SAND,** trace gravel 0.51		G	2				
Organic silt with **PEAT** fibers 0.76		G	3				
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 2.97		G	4			1	102.21
						2	101.21

End of Test Pit

TP terminated on inferred bedrock surface at 2.97m depth

(Groundwater infiltration at 1.82m - Dec 10, 2020)



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 10, 2020

FILE NO. PG5570

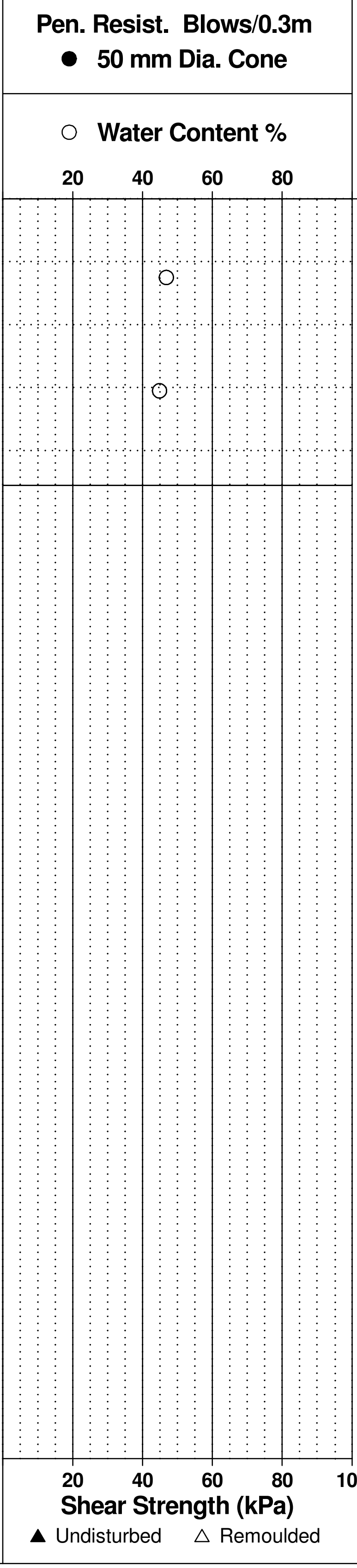
HOLE NO. TP 13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	104.30	
TOPSOIL 0.25		G	1					
Brown **SILTY SAND,** trace organics 0.61		G	2					⊻
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 0.91		G	3					

End of Test Pit

TP terminated on inferred bedrock surface at 0.91m depth

(Groundwater infiltration at 0.61m - Dec 10, 2020)



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5570

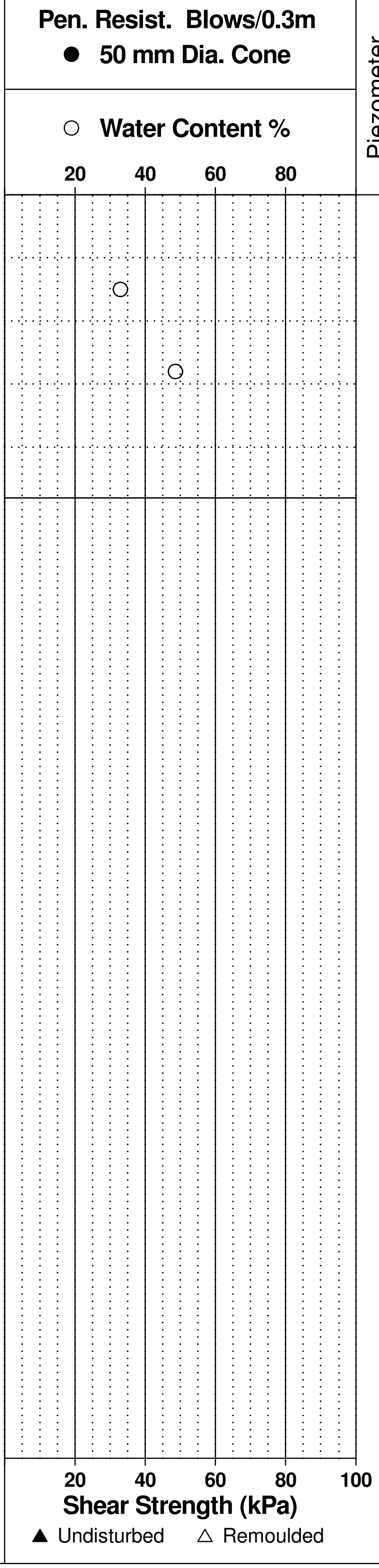
REMARKS

HOLE NO. TP 14

BORINGS BY CME-55 Low Clearance Drill

DATE December 10, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	105.60	
TOPSOIL 0.30			1					
Brown **SILTY SAND** 0.56			2					
GLACIAL TILL: Brown silty sand with gravel, cobbles, and boulders. 0.97			3					
End of Test Pit Practical refusal to excavation at 0.94m depth (TP dry upon completion)								



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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** December 10, 2020

FILE NO. PG5570

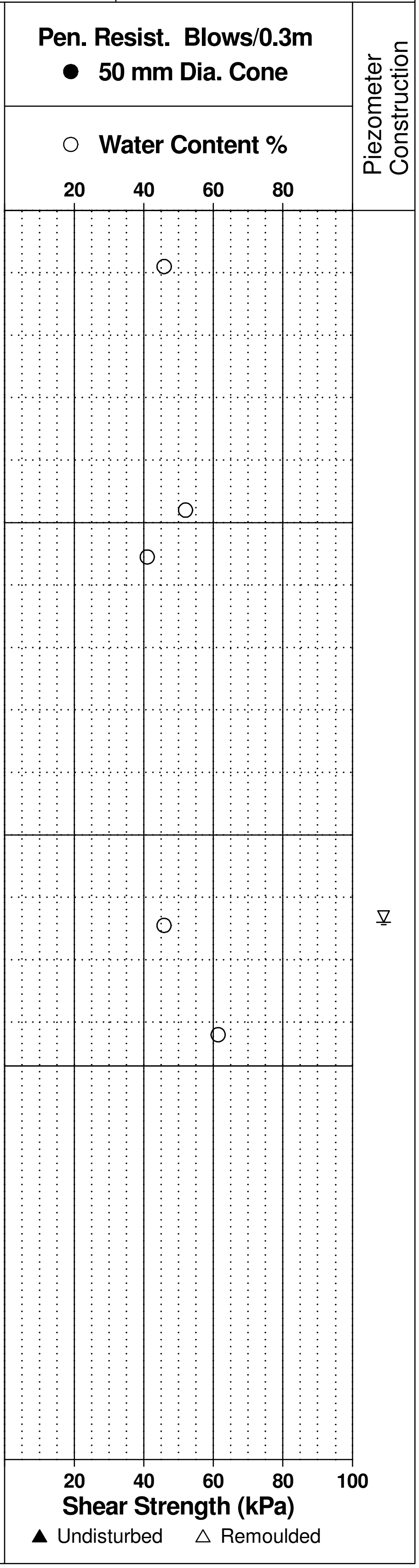
HOLE NO. TP 15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	106.80
TOPSOIL 0.18		G	1				
Brown **SILTY SAND**		G	2				
						1	105.80
		G	3				
		G	4				
						2	104.80
2.64		G	5				
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders 2.74		G	6				

End of Test Pit

TP terminated on inferred bedrock surface at 2.74m depth

(Groundwater infiltration at 2.28m - Dec 10, 2020)



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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 10, 2020

FILE NO. PG5570

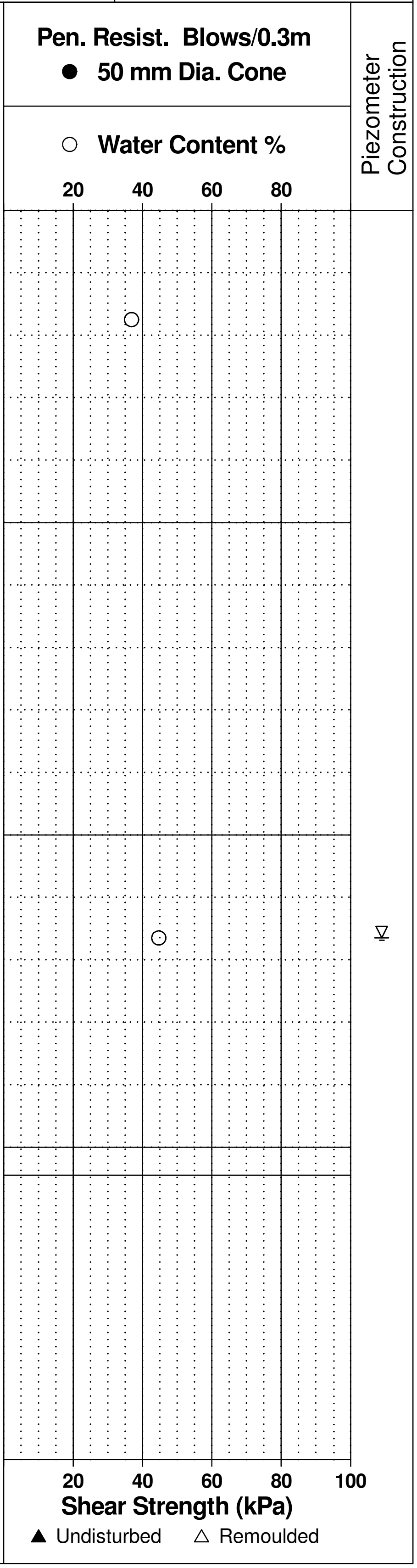
HOLE NO. TP 16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.62
TOPSOIL (to 0.35)	G	1				
Brown **SILTY SAND,** trace gravel (0.35 to 2.34)	G	2			1	103.62
					2	102.62
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders. (2.34 to 3.09)	G	3			3	101.62

End of Test Pit

TP terminated on inferred bedrock surface at 3.09m depth

(Groundwater infiltration at 2.33m - Dec 10, 2020)



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DATUM Geodetic

FILE NO. PG5570

REMARKS

HOLE NO. TP 17

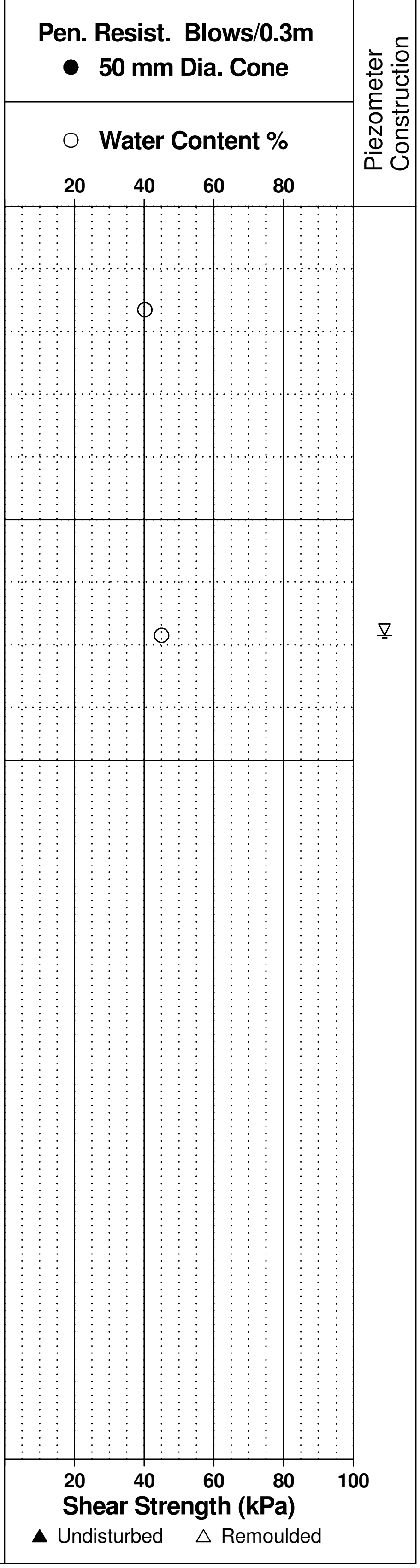
BORINGS BY CME-55 Low Clearance Drill **DATE** December 10, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.90
TOPSOIL 0.33		G	1				
Brown **SILTY SAND,** trace gravel 1.37		G	2			1	102.90
GLACIAL TILL: Brown silty sand, with gravel cobbles and boulders 1.78		G	3				

End of Test Pit

TP terminated on inferred bedrock surface at 1.78m depth

(Groundwater infiltration at 1.37m - Dec 10, 2020)



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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 10, 2020

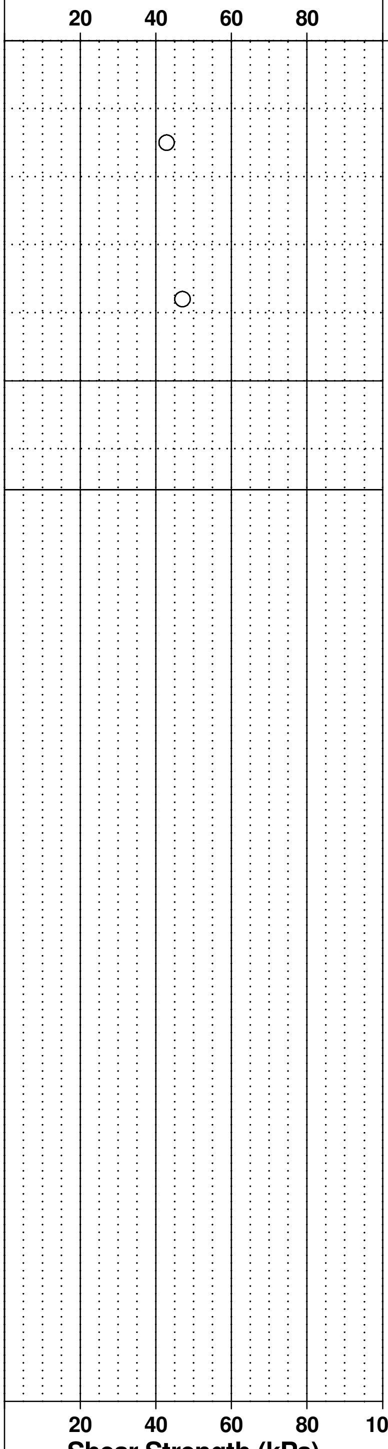
FILE NO. PG5570

HOLE NO. TP 18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.42
TOPSOIL		G	1				
0.30							
Brown **SILTY SAND,** some gravel		G	2				
		G	3			1	102.42
1.32							
End of Test Pit TP terminated on inferred bedrock surface at 1.32m depth (TP dry upon completion)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
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 = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

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)

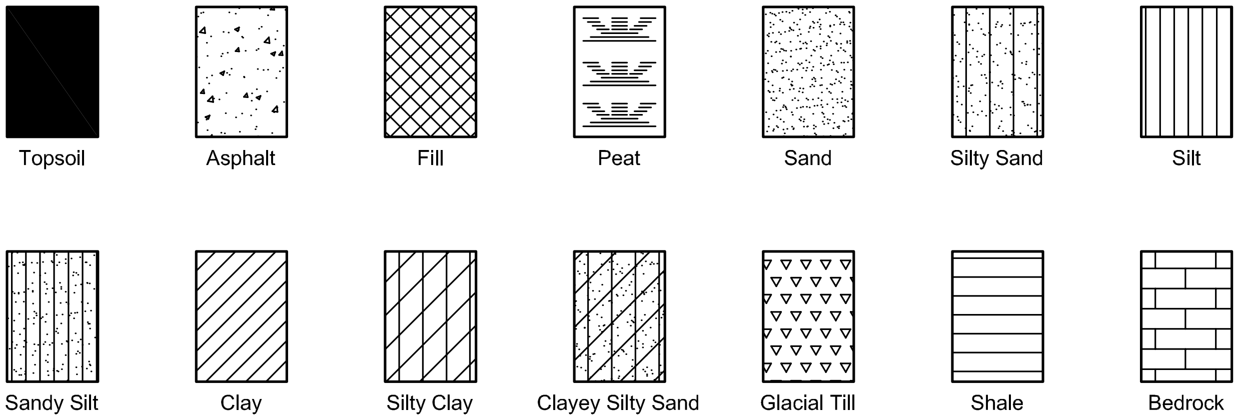
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

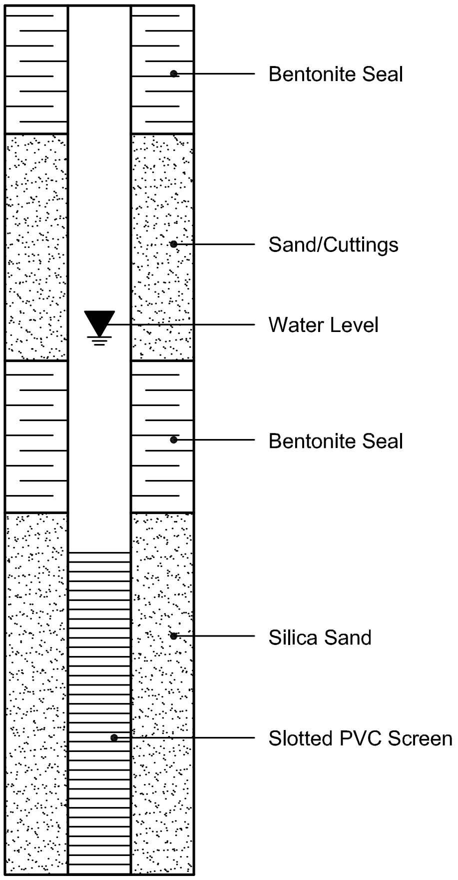
SYMBOLS AND TERMS (continued)

STRATA PLOT

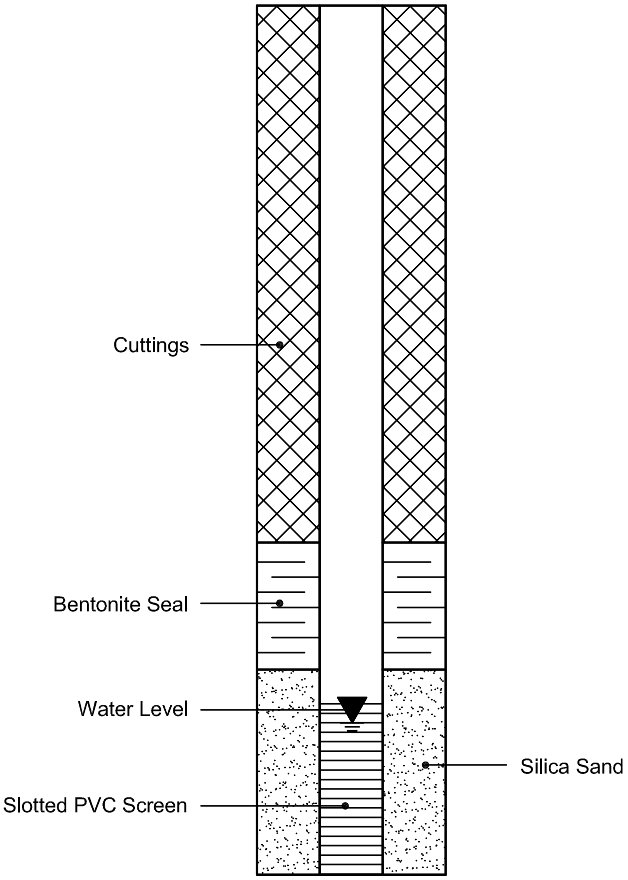


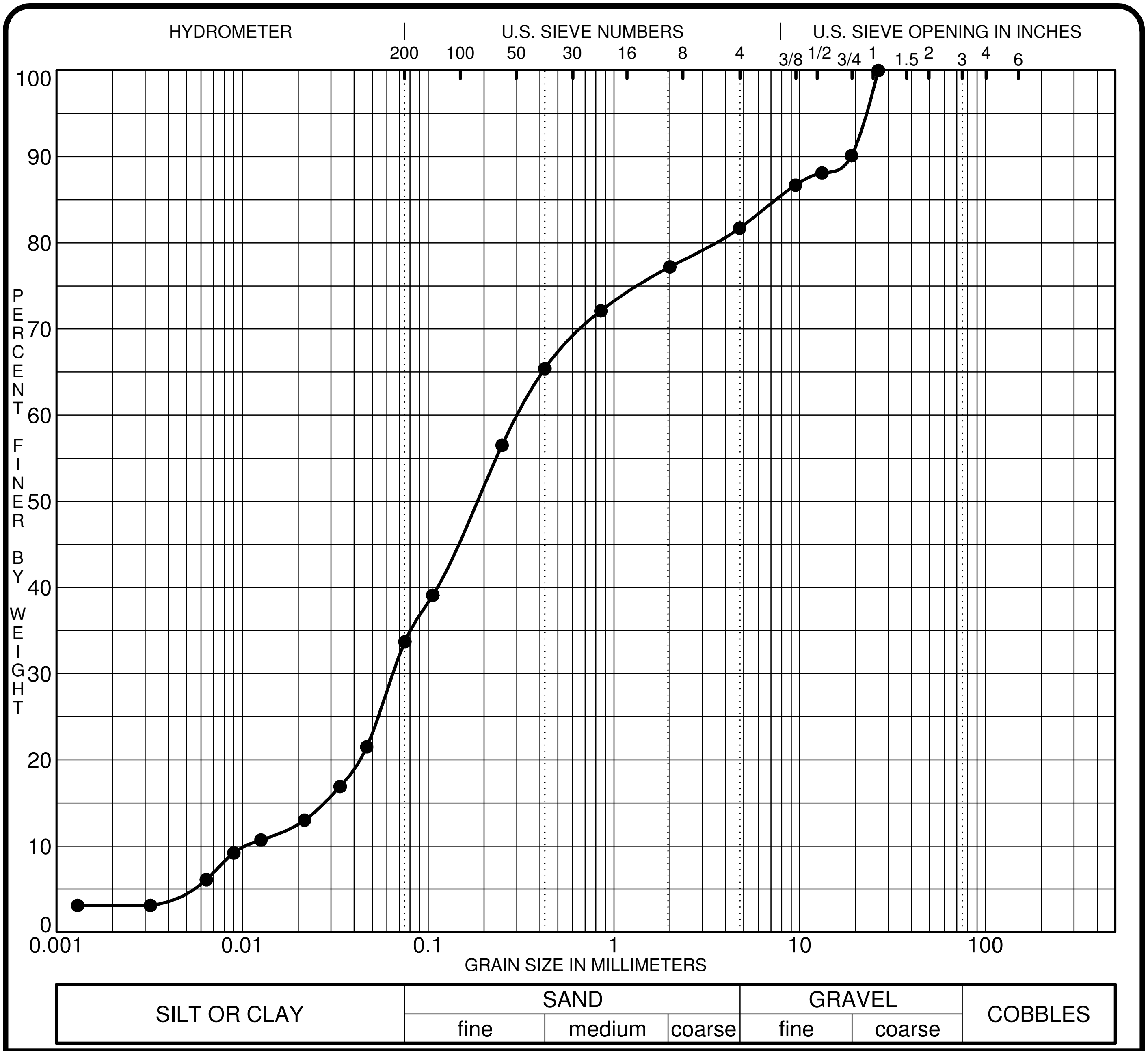
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-22**	**SS2**	**Glacial Till**					**1.27**	**28.6**
⌧									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-22**	**SS2**	**26.50**	**0.31**	**0.065**	**0.0108**	**18.3**	**48.0**	**31.2**	**2.5**
⌧										
▲										
★										

CLIENT **Caivan Communities**

FILE NO. **PG5570**

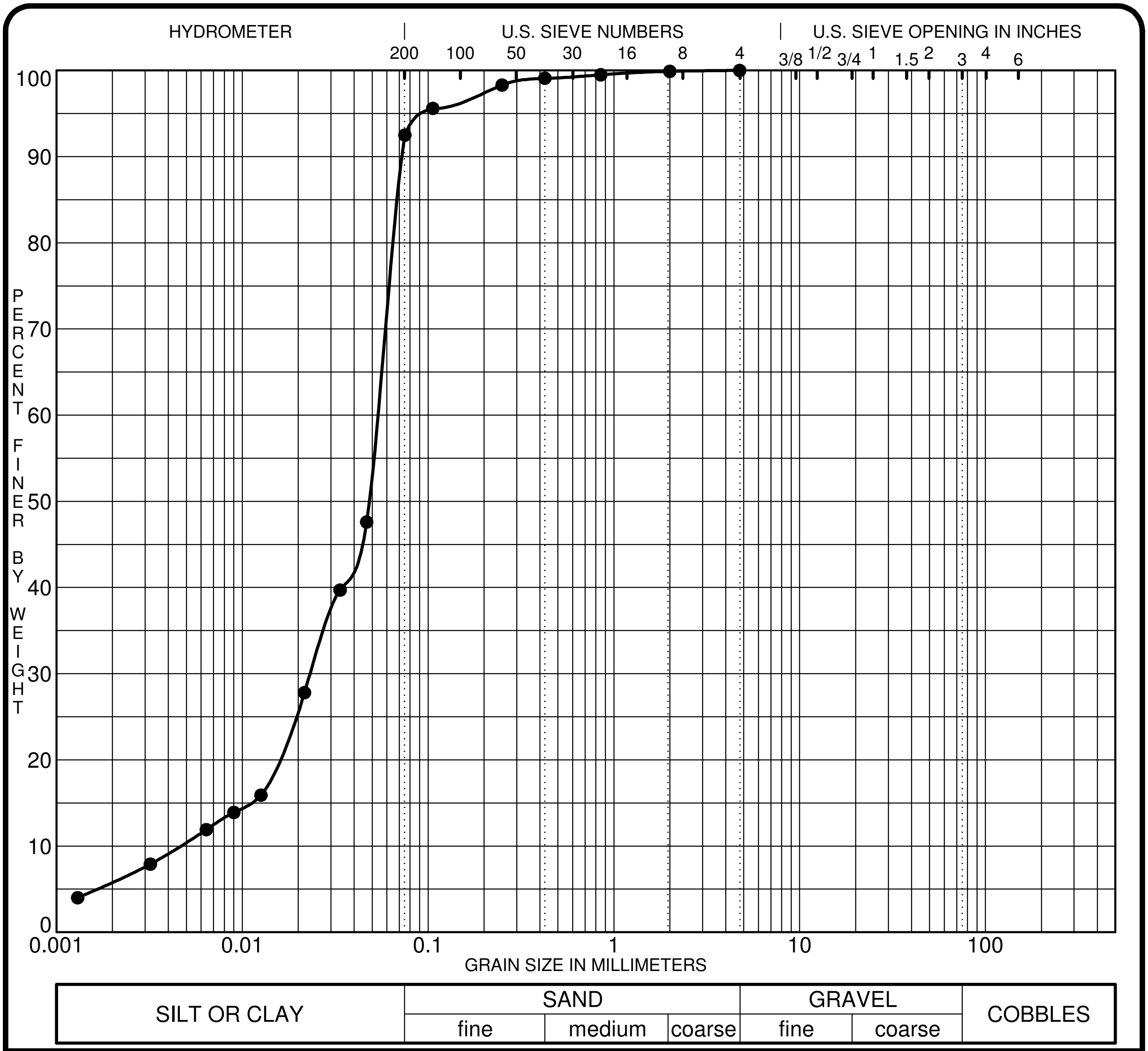
PROJECT **Geotechnical Investigation - 5993, 6070 and 6115 Flewellyn Road**

DATE **28 Sep 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



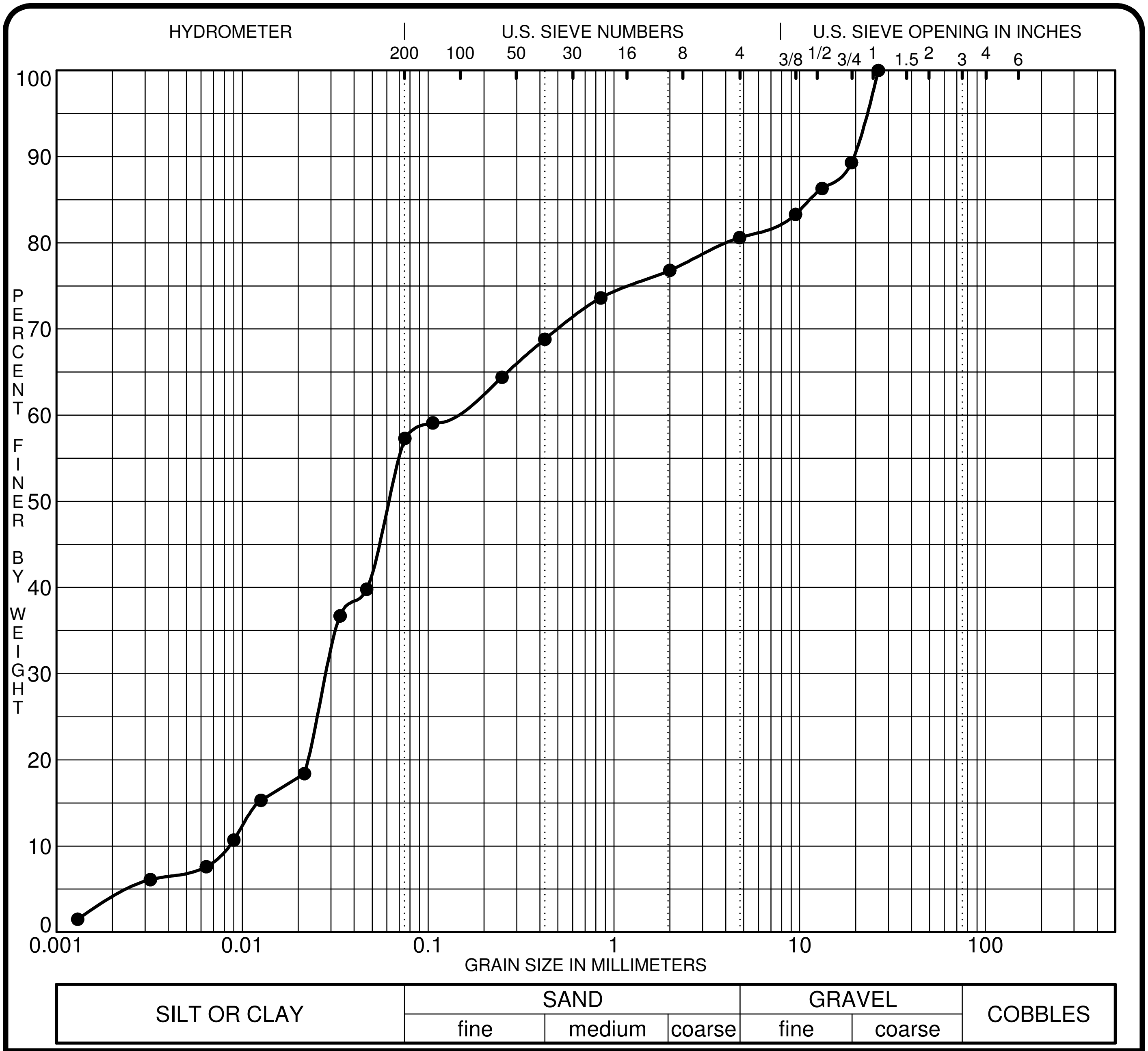
	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-22 SS4**	**Glacial Till**					**2.24**	**11.6**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-22 SS4**	**4.75**	**0.05**	**0.023**	**0.0046**	**0.0**	**7.5**	**87.0**	**5.5**
☒									
▲									
★									

CLIENT **Caivan Communities**
FILE NO. **PG5570**
PROJECT **Geotechnical Investigation - 5993, 6070 and 6115 Flewellyn Road**
DATE **29 Sep 22**

patersongroup **Consulting Engineers**
9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 4-22 SS4**	**Glacial Till**					**0.80**	**14.7**
⌧								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 4-22 SS4**	**26.50**	**0.12**	**0.029**	**0.0083**	**19.4**	**23.3**	**53.8**	**3.5**
⌧									
▲									
★									

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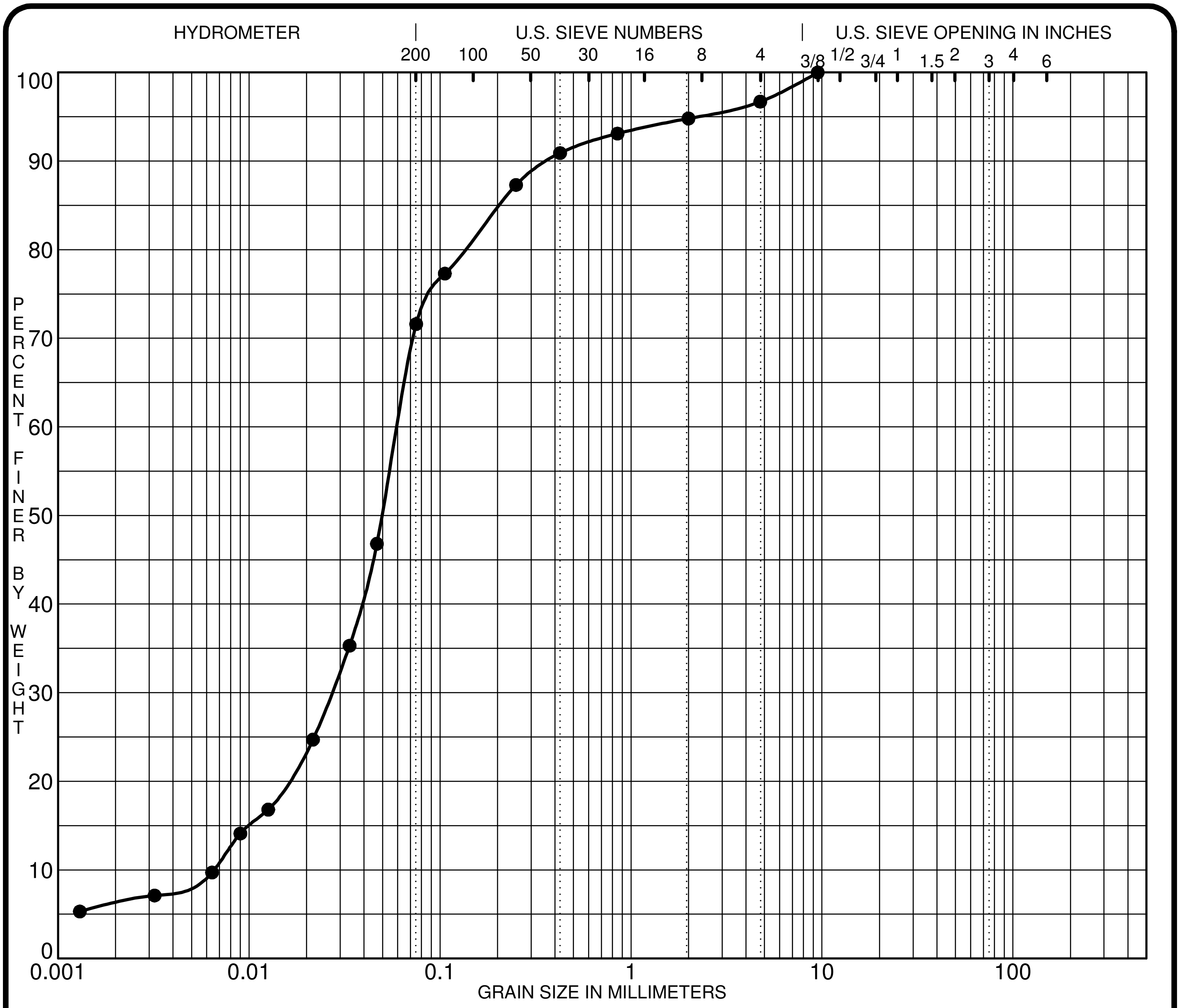

SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 5-22**	**SS3**	**Silty Sand/Sandy Silt**					**1.84**	**9.2**
⌧									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 5-22**	**SS3**	**9.50**	**0.06**	**0.027**	**0.0066**	**3.3**	**25.1**	**65.6**	**6.0**
⌧										
▲										
★										

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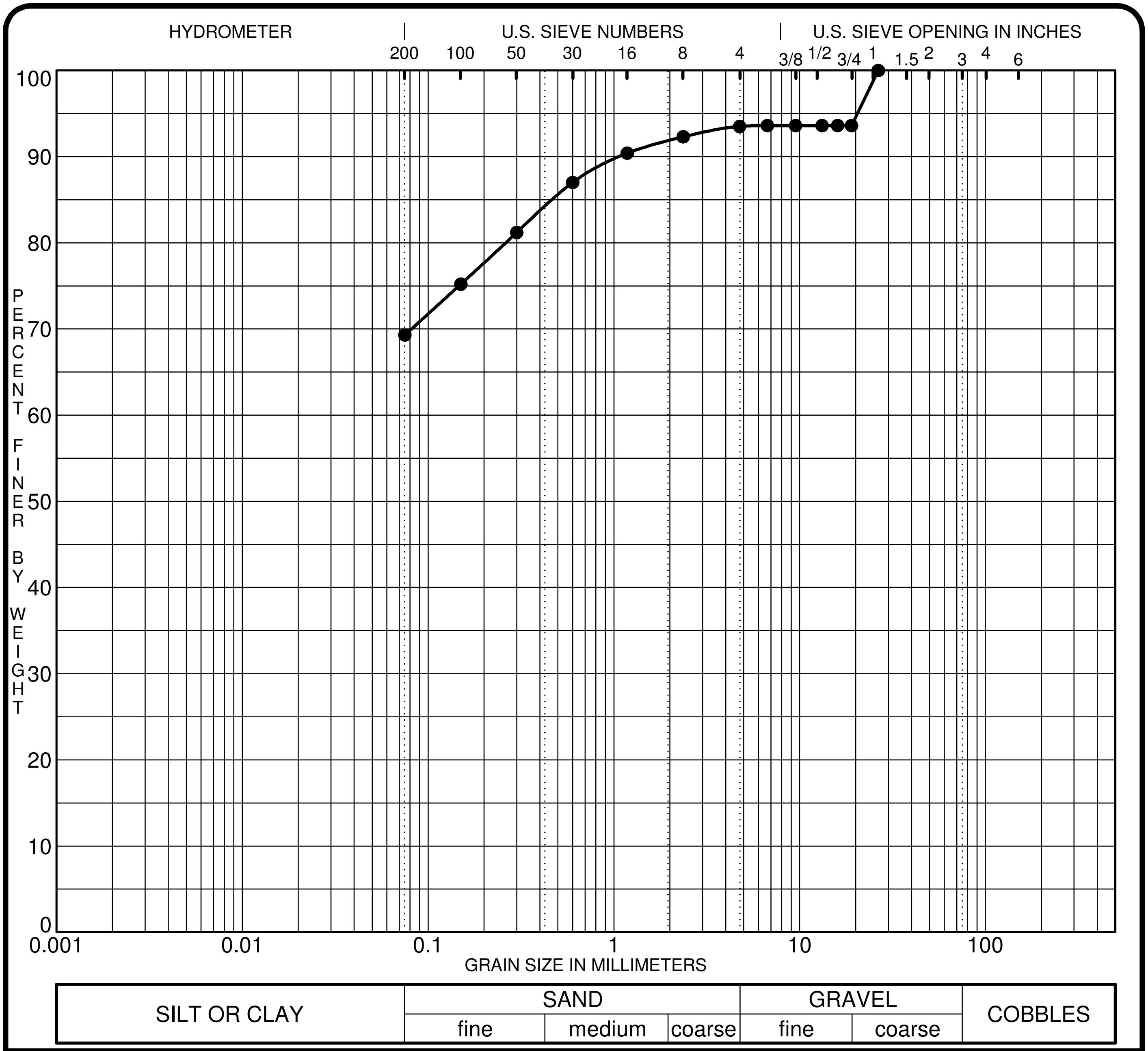
GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 4-21 SS2+SS3**	**GLACIAL TILL**					**0.40**	**2.5**
⊠								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 4-21 SS2+SS3**	**26.50**				**6.5**	**24.2**	**69.3**	
⊠									
▲									
★									

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PROJECT **Geotechnical Investigation - 5993, 6070 and 6115**
Flewellyn Road, Ottawa, On.

DATE **15 Dec 21**

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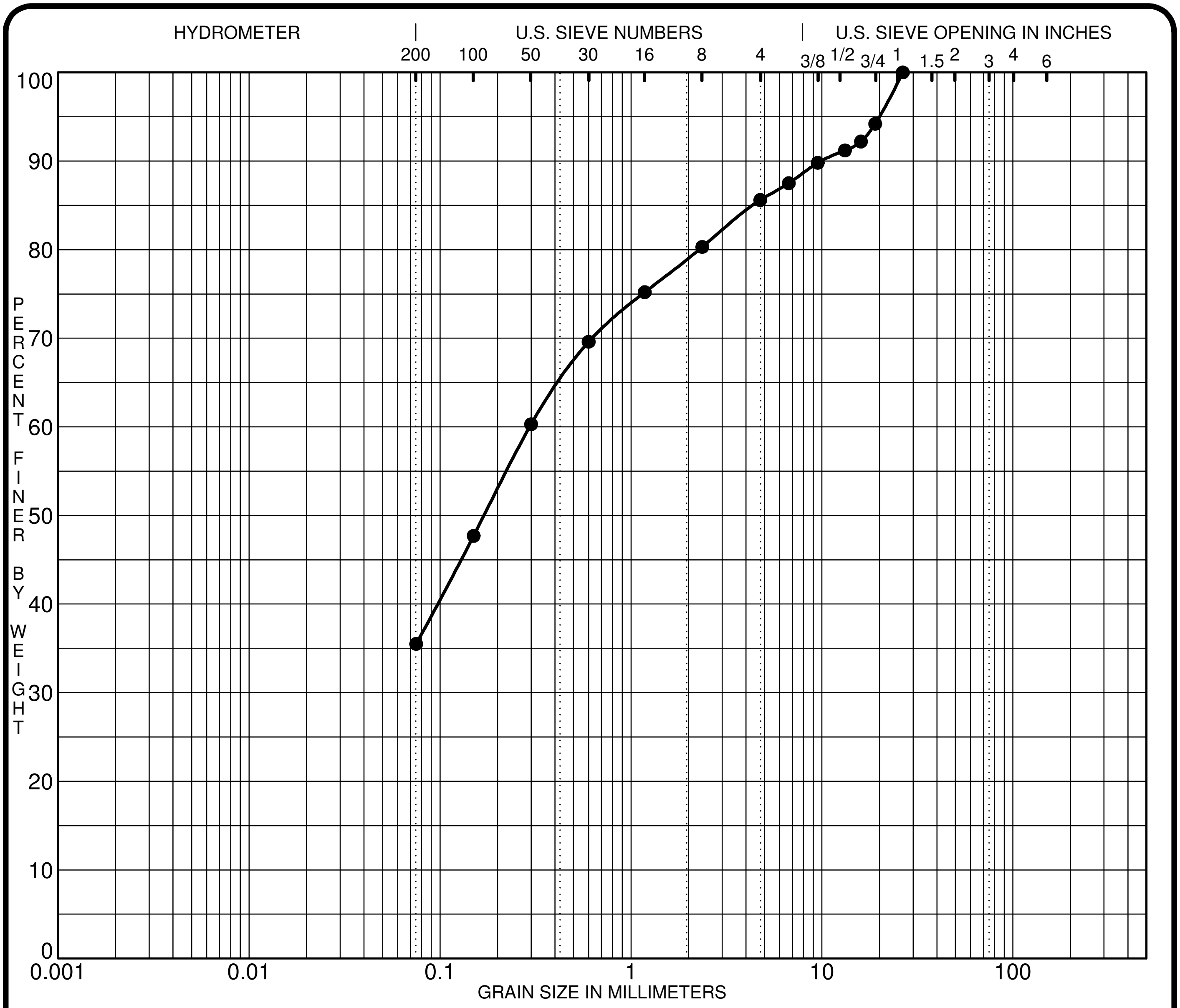
GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH11-21**	**SS3**	**GLACIAL TILL**					**0.87**	**24.2**
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH11-21**	**SS3**	**26.50**	**0.30**			**14.4**	**50.1**	**35.5**	
☒										
▲										
★										

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Flewellyn Road, Ottawa, On.

FILE NO. **PG5570**

DATE **16 Dec 21**

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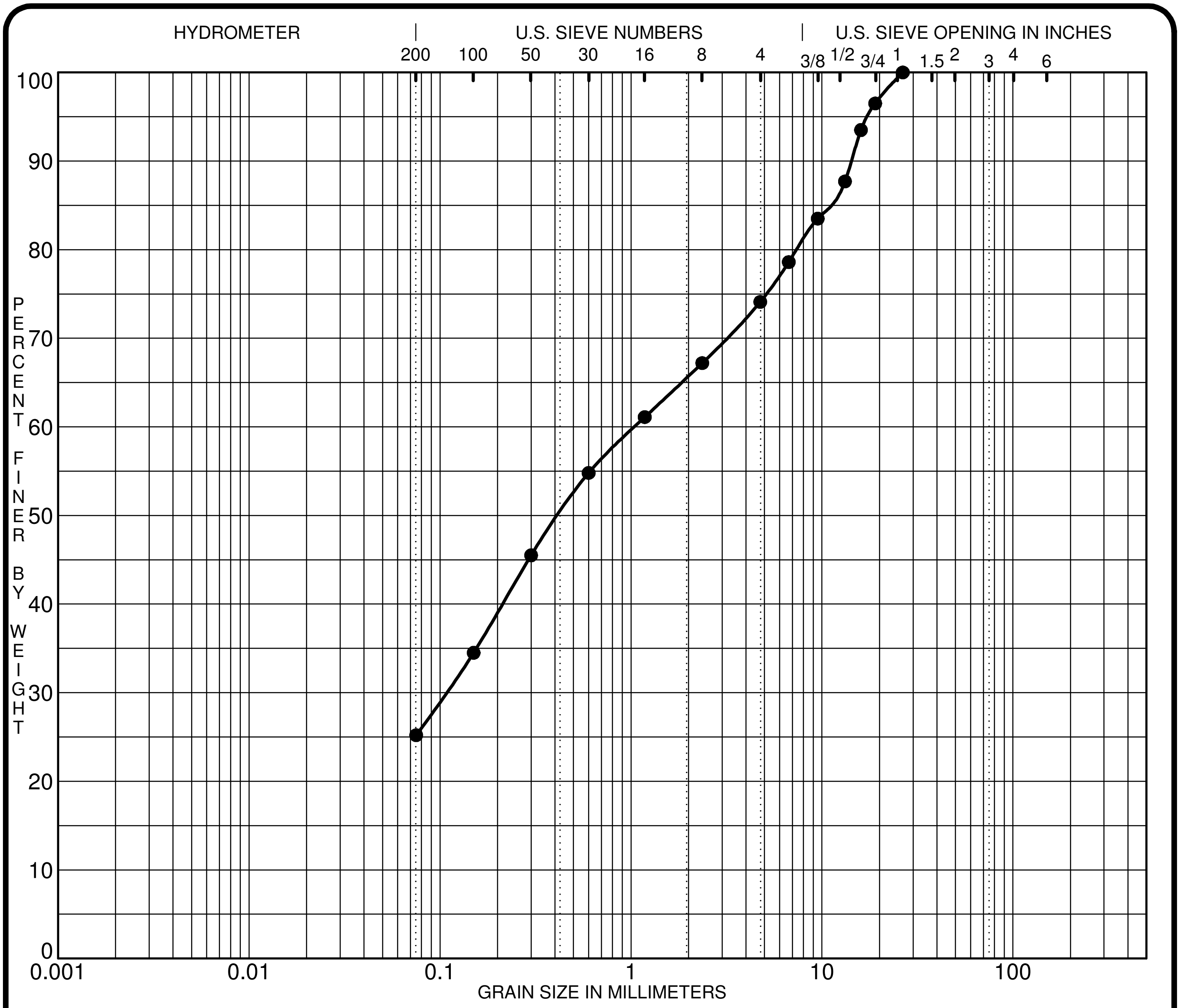
GRAIN SIZE DISTRIBUTION



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<tr><td>fine</td><td>medium</td><td>coarse</td><td>fine</td><td>coarse</td></tr>
</table>

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH14-21SS2+SS3**	**GLACIAL TILL**					**0.70**	**76.7**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH14-21SS2+SS3**	**26.50**	**1.05**	**0.107**		**25.9**	**48.9**	**25.2**	
☒									
▲									
★									

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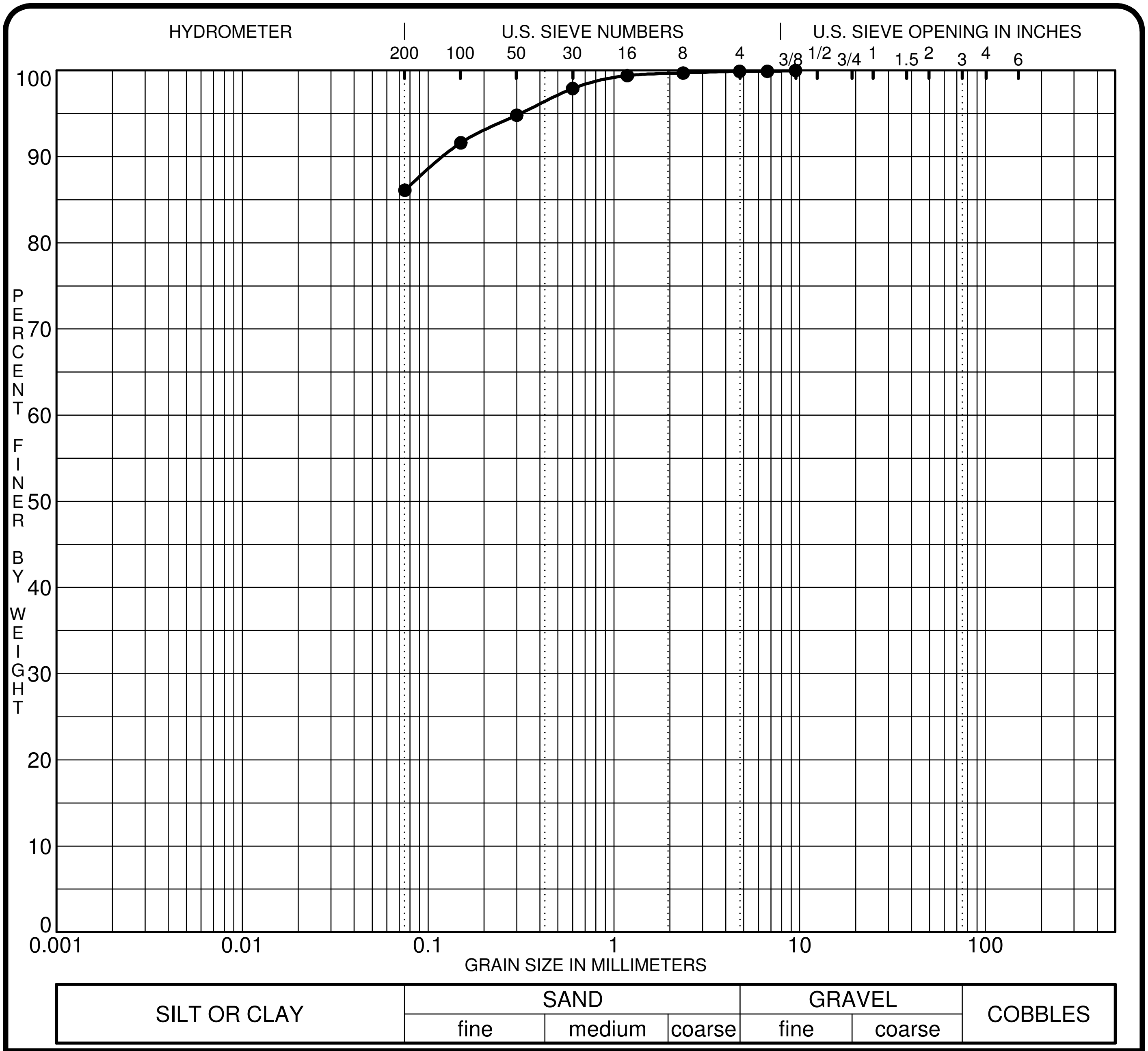
GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH19-21SS2+SS3**	**SILTY SAND/SANDY SILT**					**0.83**	**1.2**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH19-21SS2+SS3**	**9.50**				**0.1**	**13.8**	**86.1**	
☒									
▲									
★									

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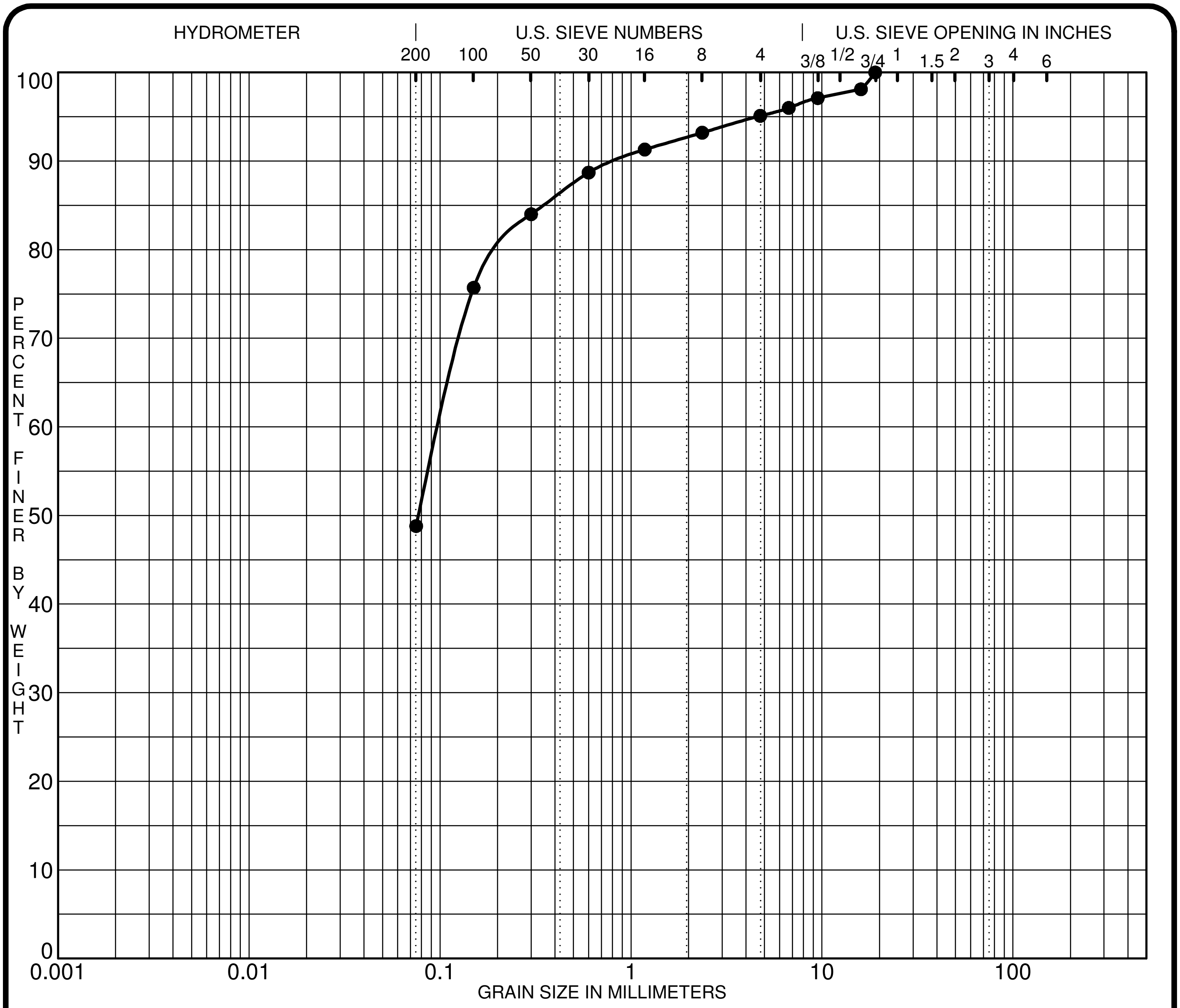
Flewellyn Road, Ottawa, On.

DATE **16 Dec 21**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



<table>
<tr><td rowspan="2">SILT OR CLAY</td><td colspan="3">SAND</td><td colspan="2">GRAVEL</td><td rowspan="2">COBBLES</td></tr>
<tr><td>fine</td><td>medium</td><td>coarse</td><td>fine</td><td>coarse</td></tr>
</table>

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH24-21SS3+SS4**	**GLACIAL TILL**					**1.10**	**3.5**
⌛								
▲								
★								

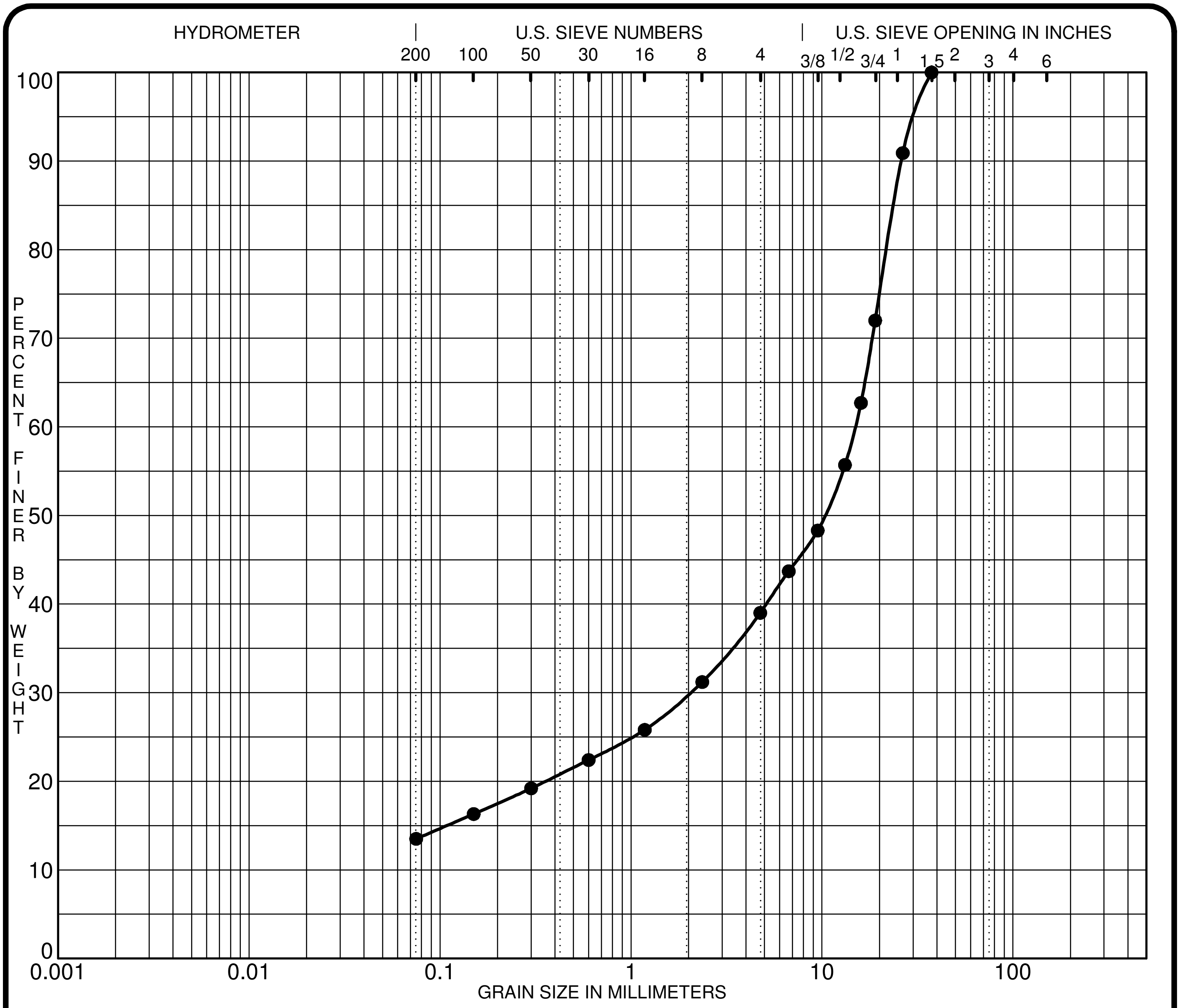
	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH24-21SS3+SS4**	**19.00**	**0.10**			**4.9**	**46.3**	**48.8**	
⌛									
▲									
★									

CLIENT	**Caivan Communities**	FILE NO.	**PG5570**
PROJECT	**Geotechnical Investigation - 5993, 6070 and 6115 Flewellyn Road, Ottawa, On.**	DATE	**20 Dec 21**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH35-21SS4+SS5**	**GLACIAL TILL**					**17.82**	**1100**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH35-21SS4+SS5**	**37.50**	**14.86**	**2.023**		**61.0**	**25.5**	**13.5**	
☒									
▲									
★									

CLIENT **Caivan Communities**

FILE NO. **PG5570**

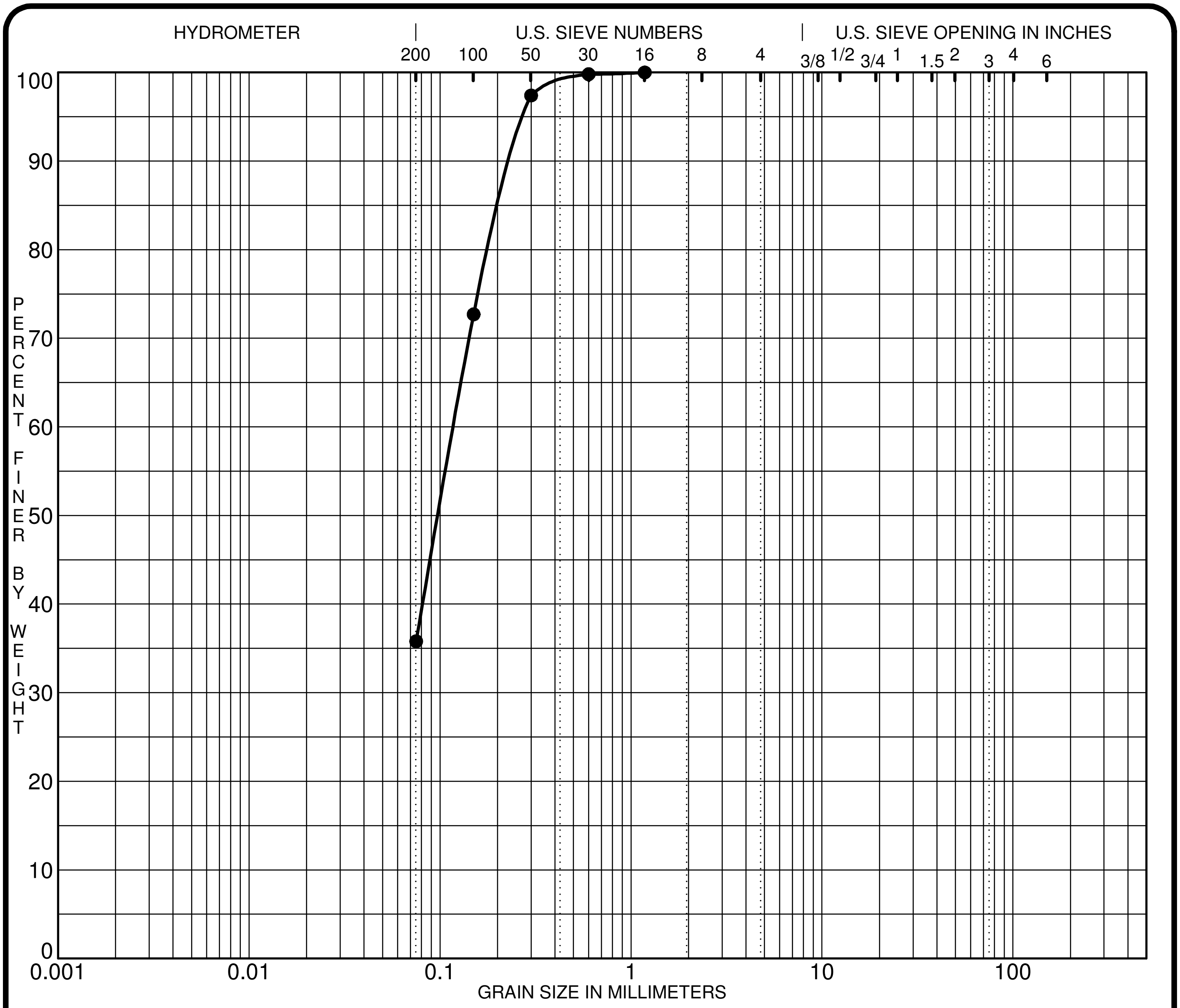
PROJECT **Geotechnical Investigation - 5993, 6070 and 6115**
Flewellyn Road, Ottawa, On.

DATE **7 Jan 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH37-21**	**SS3**	**SILTY SAND/SANDY SILT**					**0.65**	**2.4**
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH37-21**	**SS3**	**1.18**	**0.12**			**0.0**	**64.2**	**35.8**	
☒										
▲										
★										

CLIENT **Caivan Communities**

FILE NO. **PG5570**

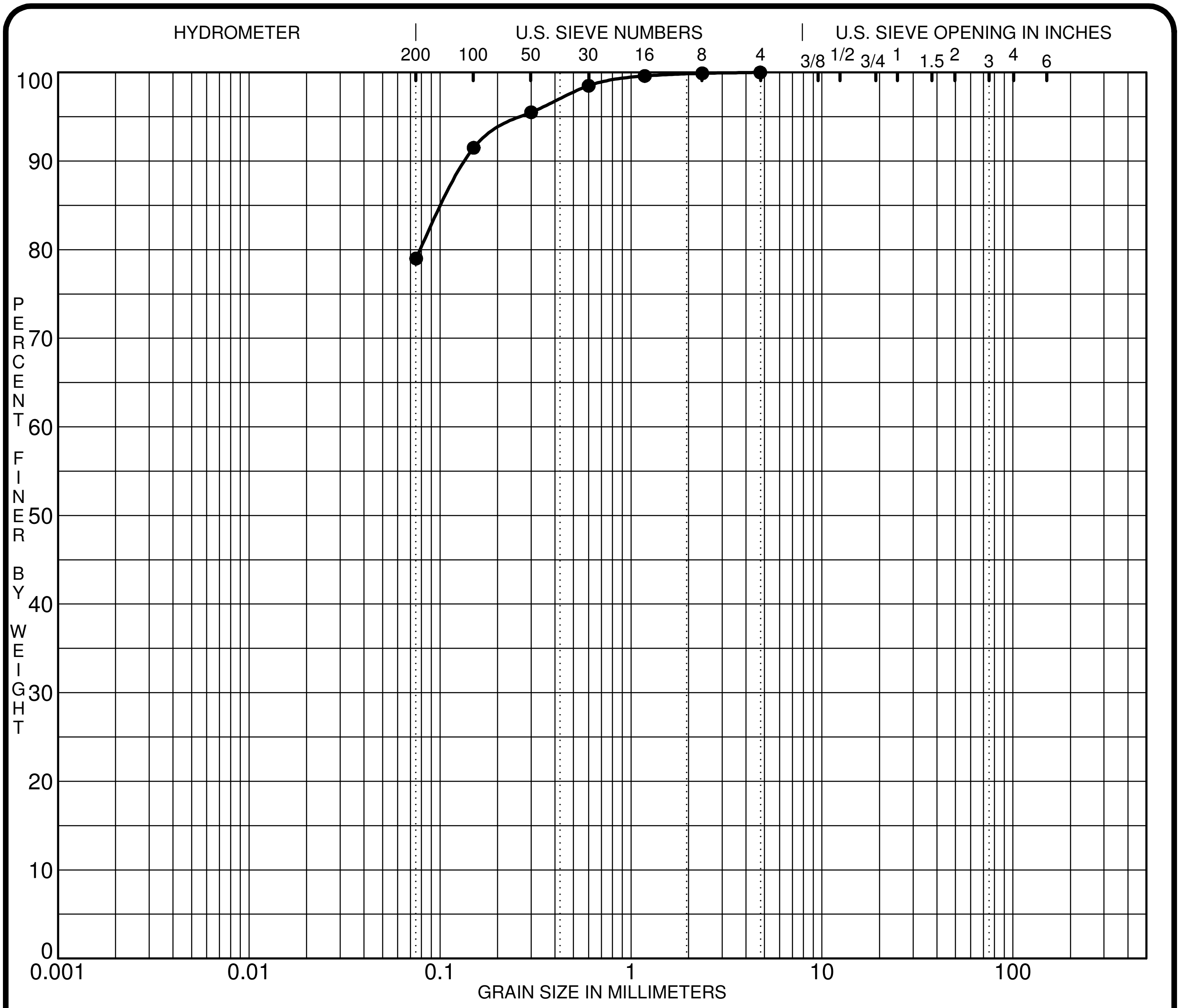
PROJECT **Geotechnical Investigation - 5993, 6070 and 6115**
Flewellyn Road, Ottawa, On.

DATE **7 Jan 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH38-21SS3+SS4**	**SILTY SAND/SANDY SILT**					**0.56**	**1.8**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH38-21SS3+SS4**	**4.75**				**0.0**	**21.0**	**79.0**	
☒									
▲									
★									

CLIENT **Caivan Communities**

FILE NO. **PG5570**

PROJECT **Geotechnical Investigation - 5993, 6070 and 6115**
Flewellyn Road, Ottawa, On.

DATE **7 Jan 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31285

Report Date: 27-Nov-2020

Order Date: 20-Nov-2020

Project Description: PG5570

	Client ID:	TP4-GR3	-	-	-
	Sample Date:	20-Nov-20 13:00	-	-	-
	Sample ID:	2047663-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.60	-	-	-
Resistivity	0.10 Ohm.m	93.8	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31363

Report Date: 17-Dec-2020

Order Date: 14-Dec-2020

Project Description: PG5570

	Client ID:	TPF-G2	-	-	-
	Sample Date:	11-Dec-20 15:30	-	-	-
	Sample ID:	2051099-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	82.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.33	-	-	-
Resistivity	0.10 Ohm.m	101	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

Certificate of Analysis

Report Date: 22-Dec-2021

Client: **Paterson Group Consulting Engineers**

Order Date: 17-Dec-2021

Client PO: 33505

Project Description: PG5570

	Client ID:	BH17-21 SS3	-	-	-
	Sample Date:	16-Dec-21 09:00	-	-	-
	Sample ID:	2151599-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.73	-	-	-
Resistivity	0.10 Ohm.m	48.9	-	-	-
Anions					
Chloride	5 ug/g dry	34	-	-	-
Sulphate	5 ug/g dry	24	-	-	-

	Client ID:	BH34-21 SS3	-	-	-
	Sample Date:	22-Dec-21 09:00	-	-	-
	Sample ID:	2152465-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	84.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.75	-	-	-
Resistivity	0.10 Ohm.m	81.3	-	-	-
Anions					
Chloride	5 ug/g dry	12	-	-	-
Sulphate	5 ug/g dry	9	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 – 13 MONITORING WELL WATER ELEVATIONS

TABLE 1 – MONITORING WELL WATER LEVEL MEASUREMENT SUMMARY

DRAWING PG5570-1 - TEST HOLE LOCATION PLAN

DRAWING PG5570-2 - BEDROCK CONTOUR PLAN

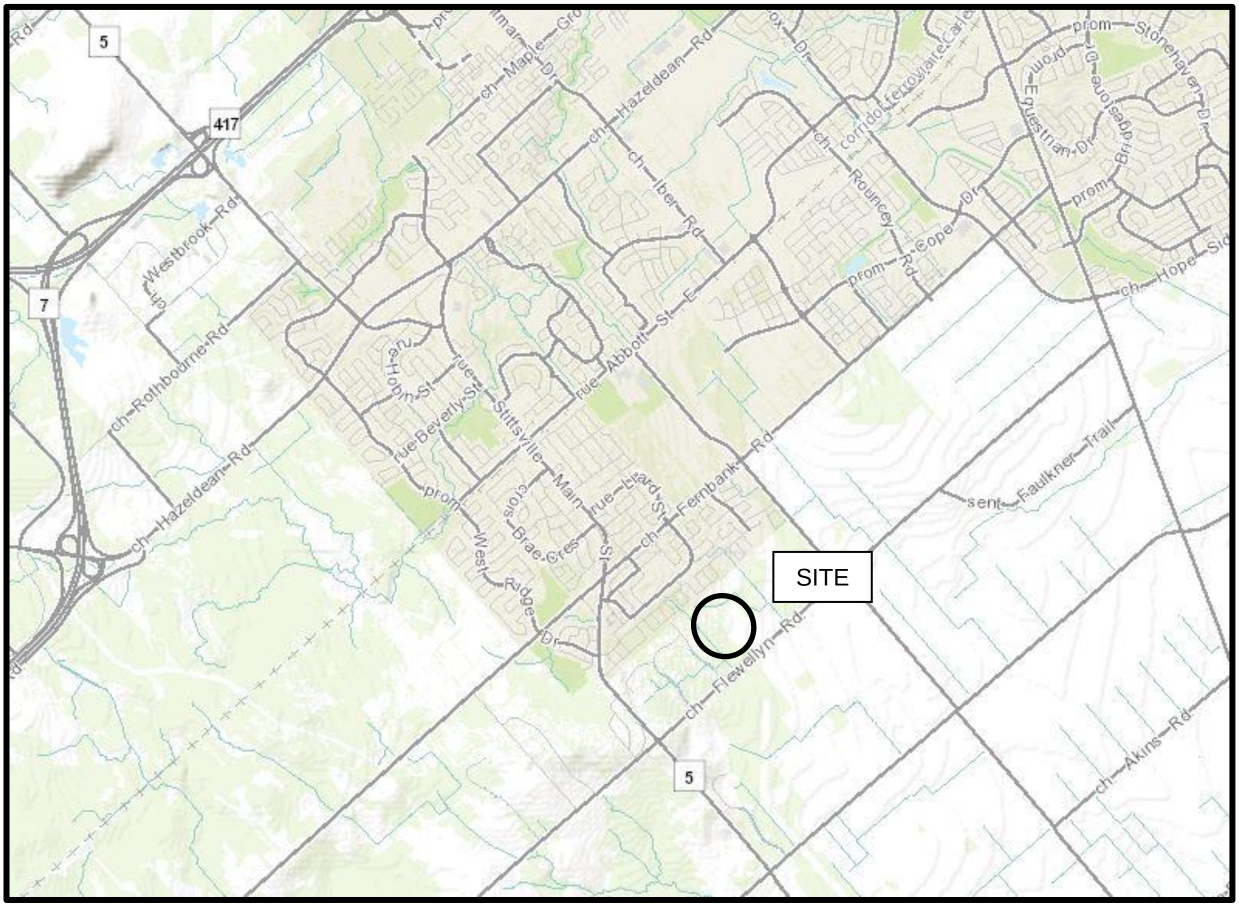


FIGURE 1

KEY PLAN

Figure 2: BH1-21 - Monitoring Well Water Elevations

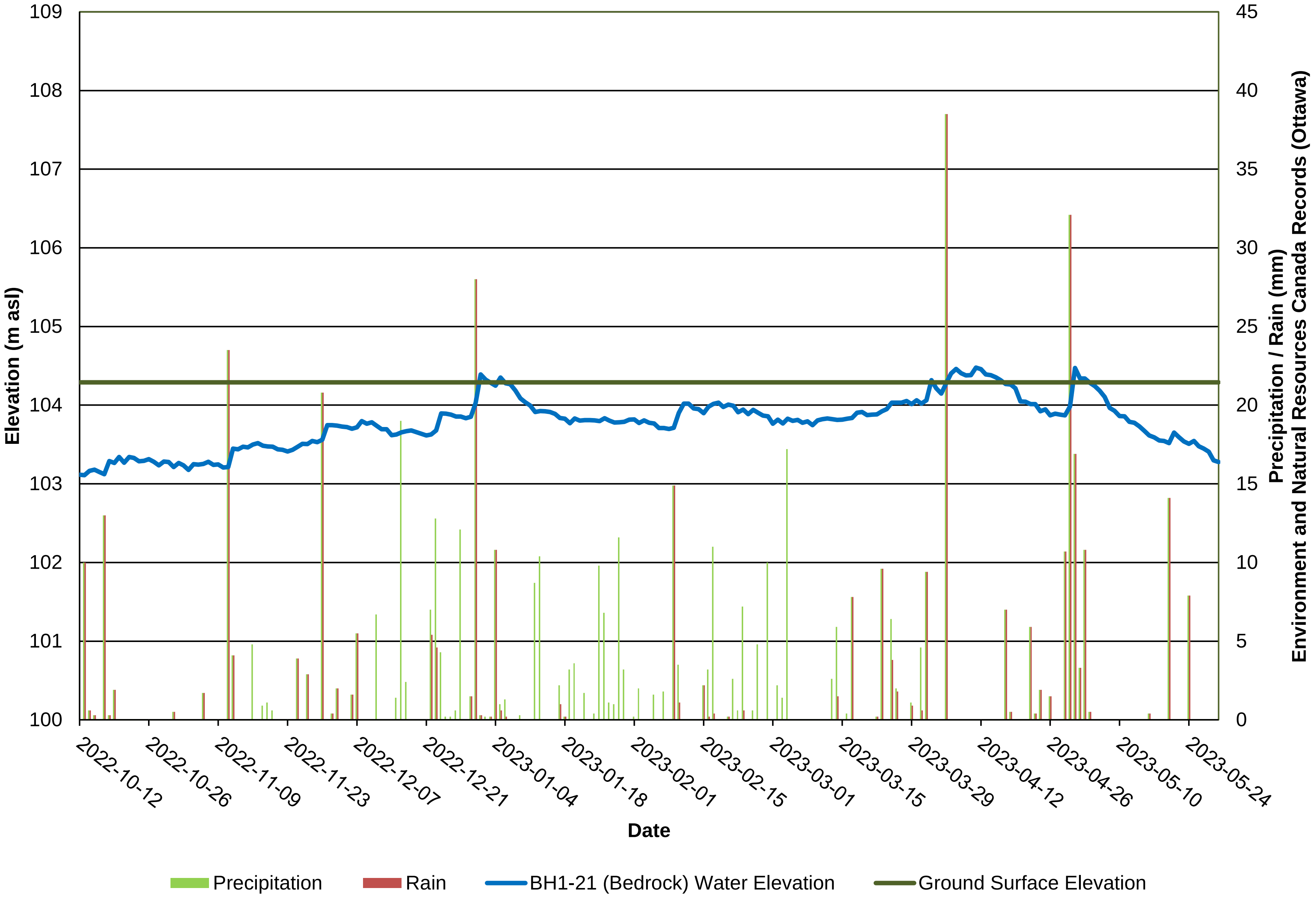


Figure 3: BH2-21 - Monitoring Well Water Elevations

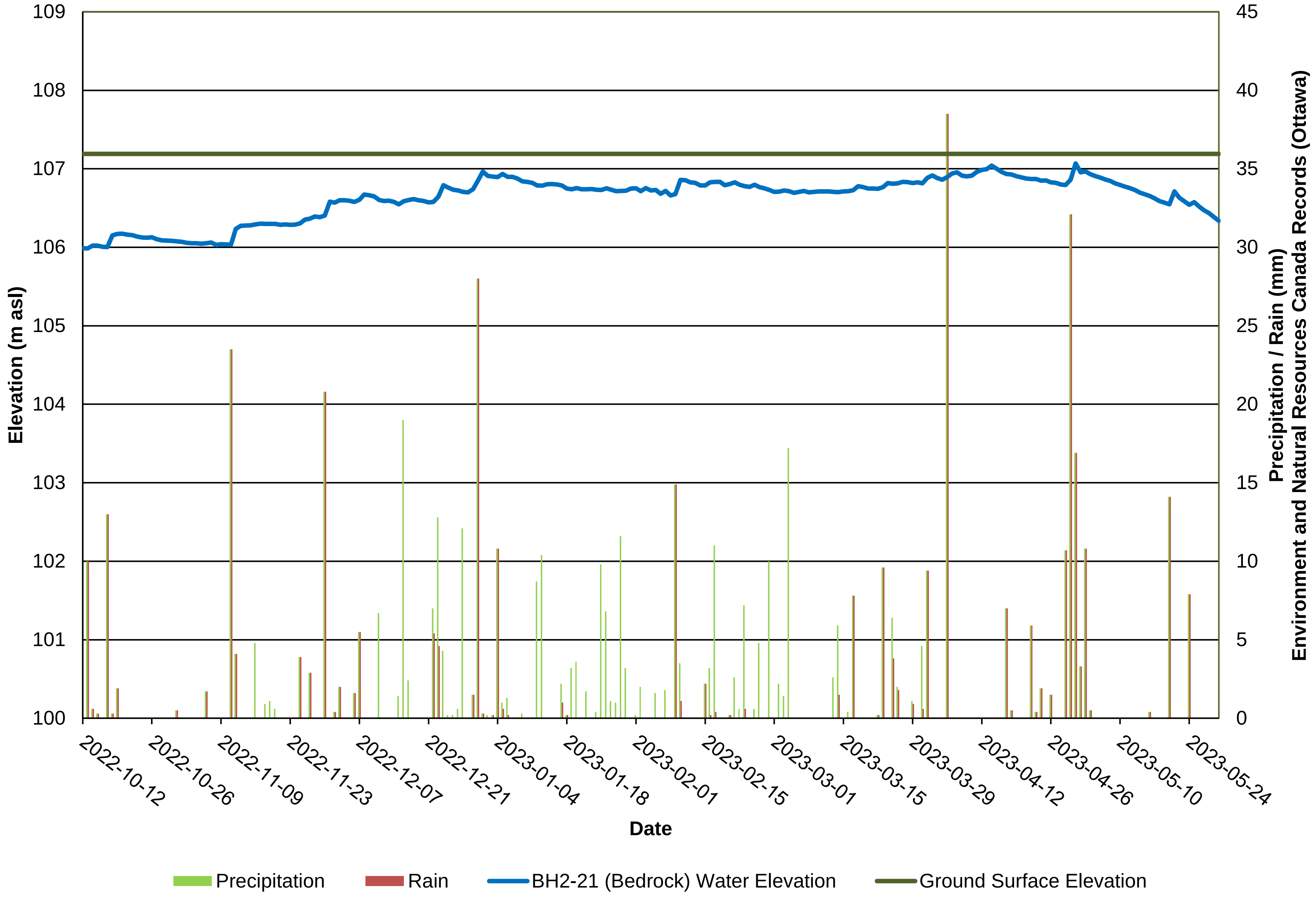
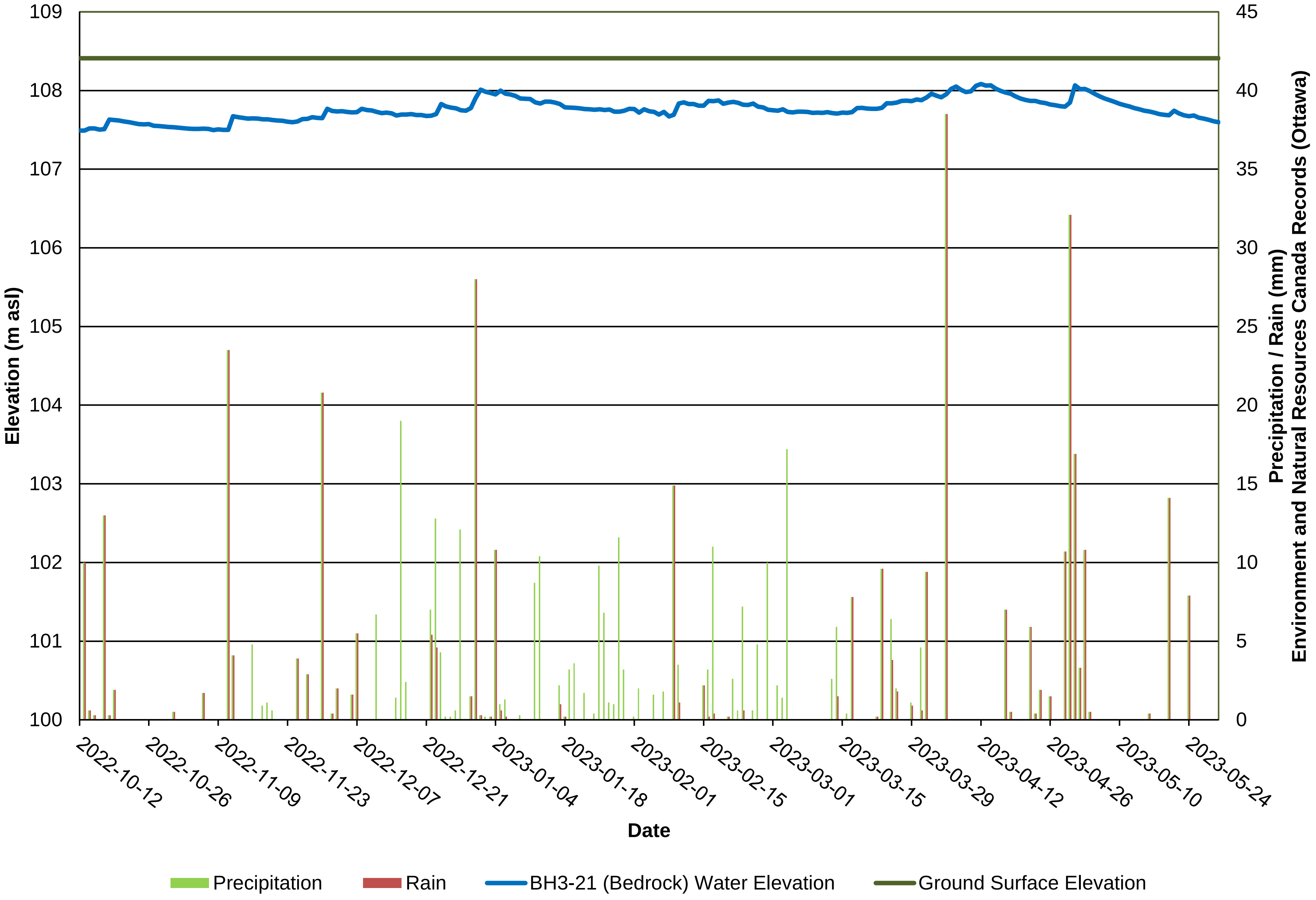


Figure 4: BH3-21 - Monitoring Well Water Elevations



File: PH4625

Caivan (Stittsville South) Inc. and Caivan (Stittsville West) Ltd.
5993 and 6115 Flewellyn Road and 6030 and 6070 Fernbank Road
Monitoring Well Water Elevations

Figure 5: BH22A-21 - Monitoring Well Water Elevations

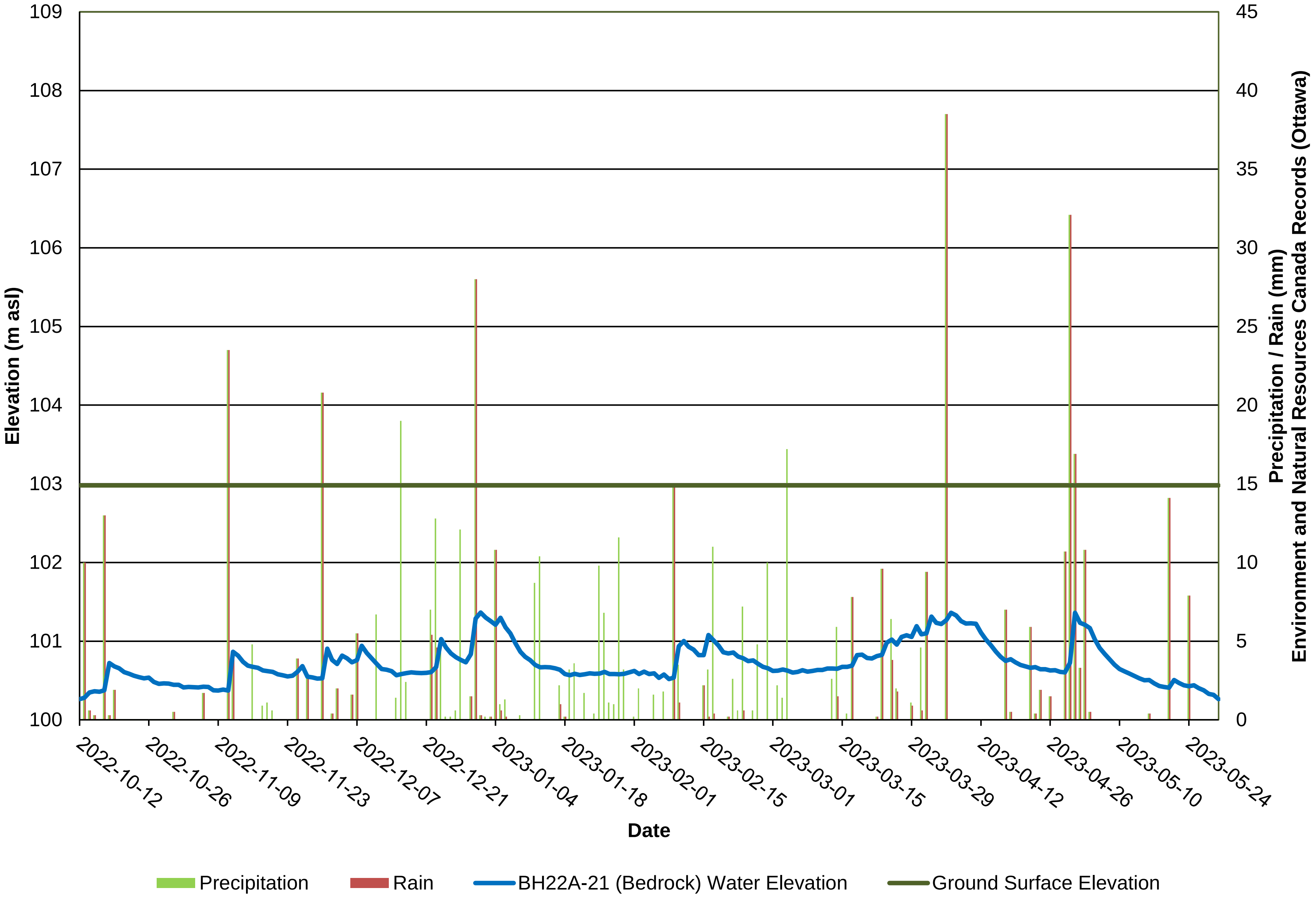
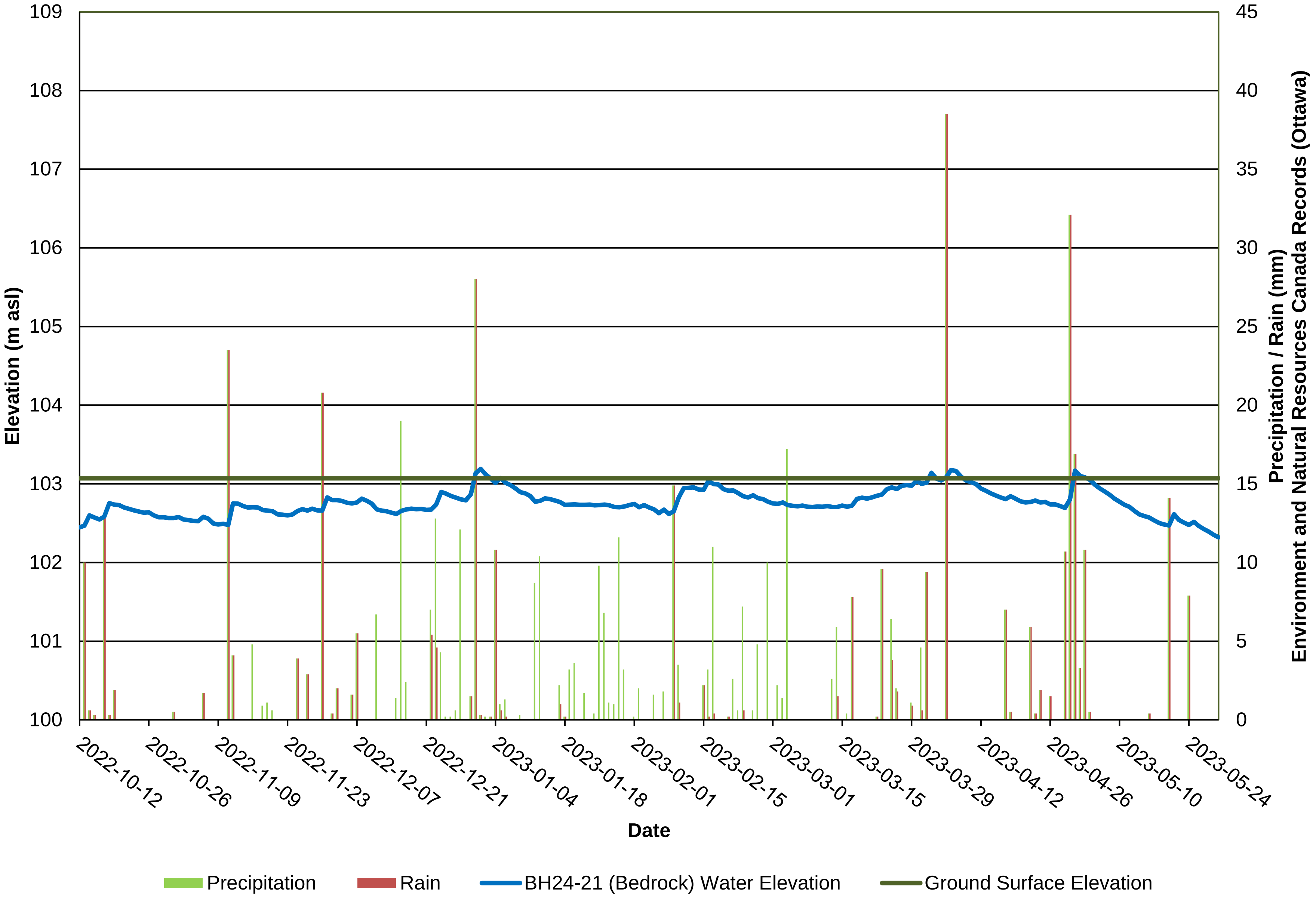


Figure 6: BH24-21 - Monitoring Well Water Elevations



PATERSON GROUP

Figure 7: BH33-21 - Monitoring Well Water Elevations

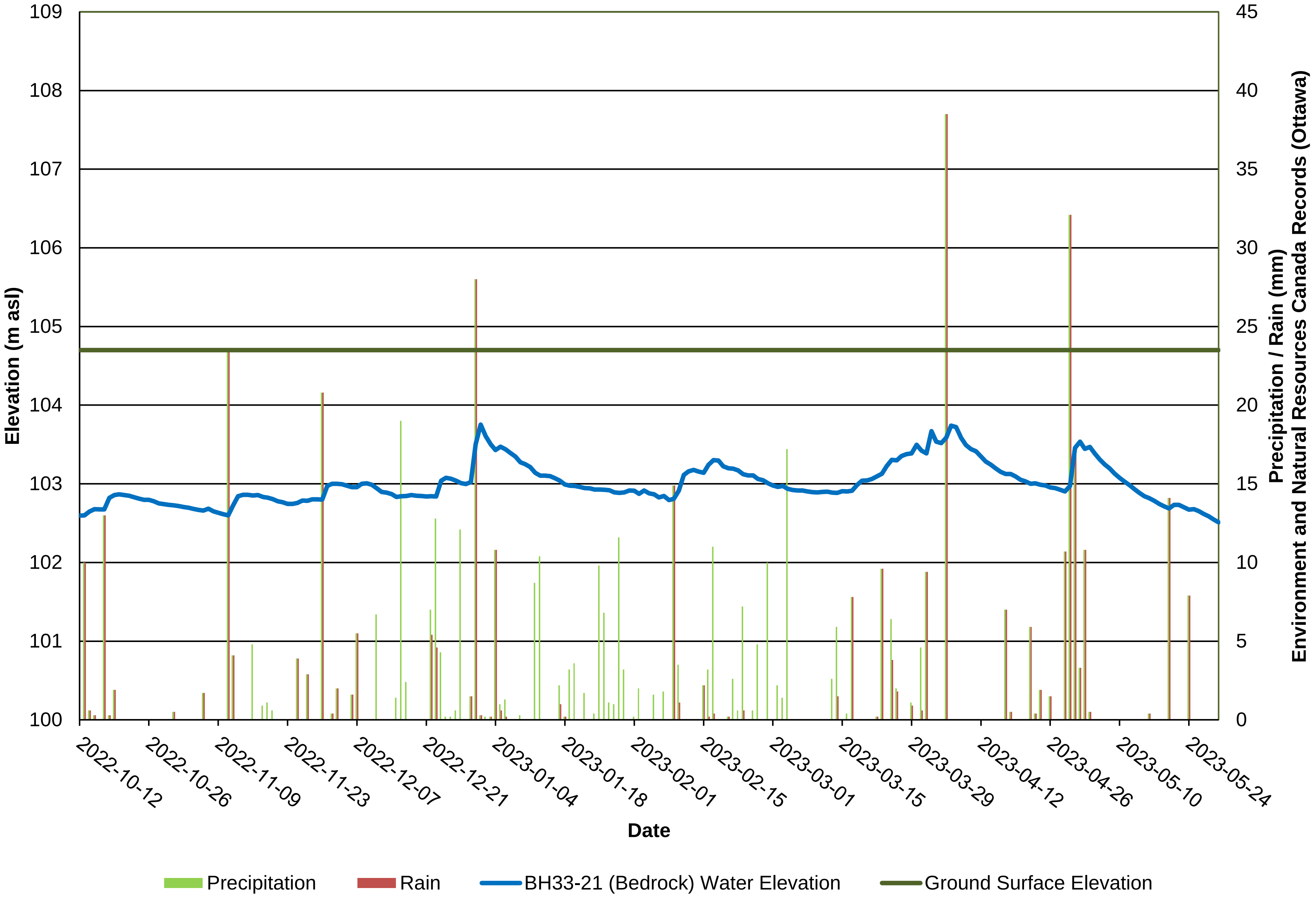
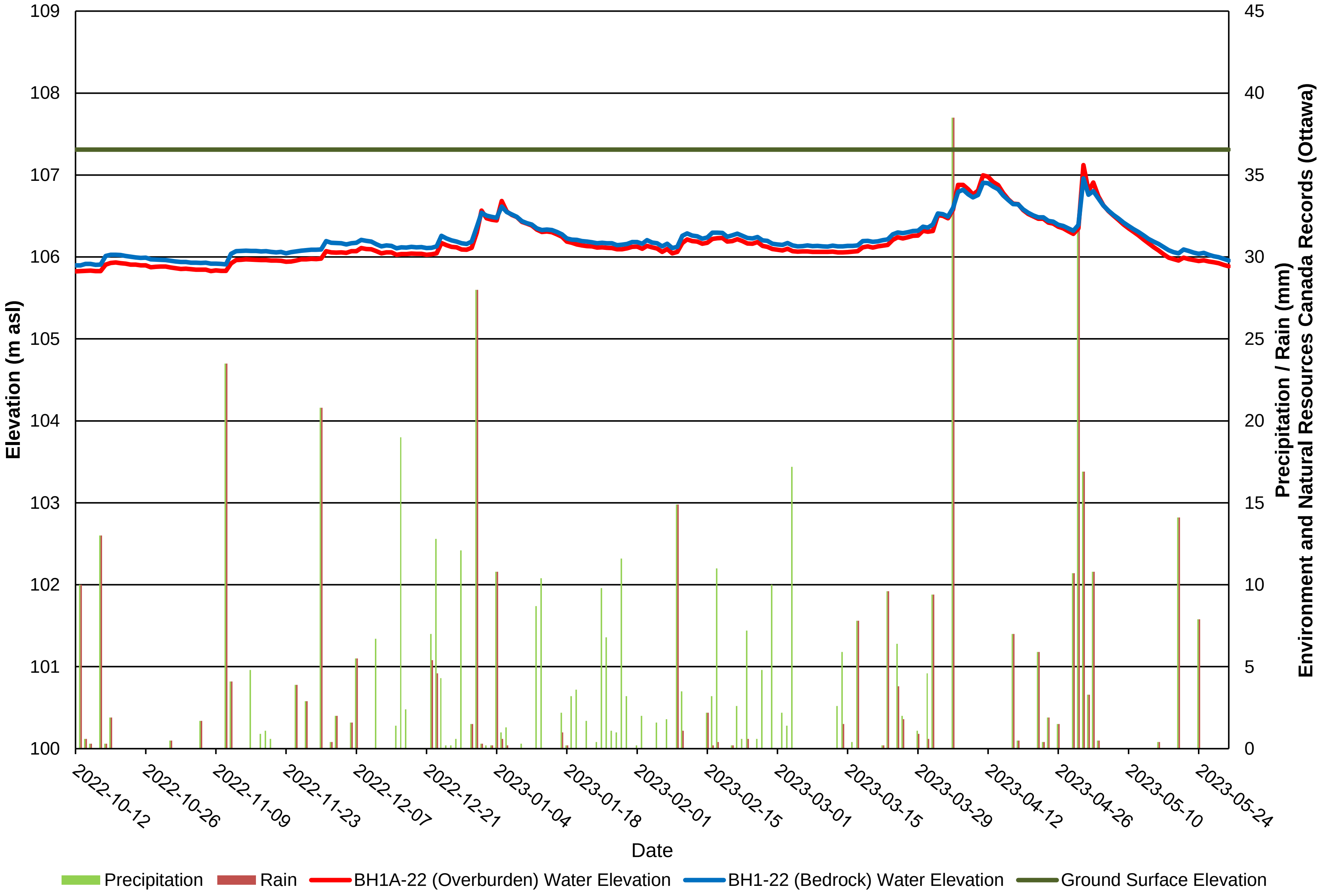


Figure 8: BH1-22 & BH1A-22 - Monitoring Well Water Elevations



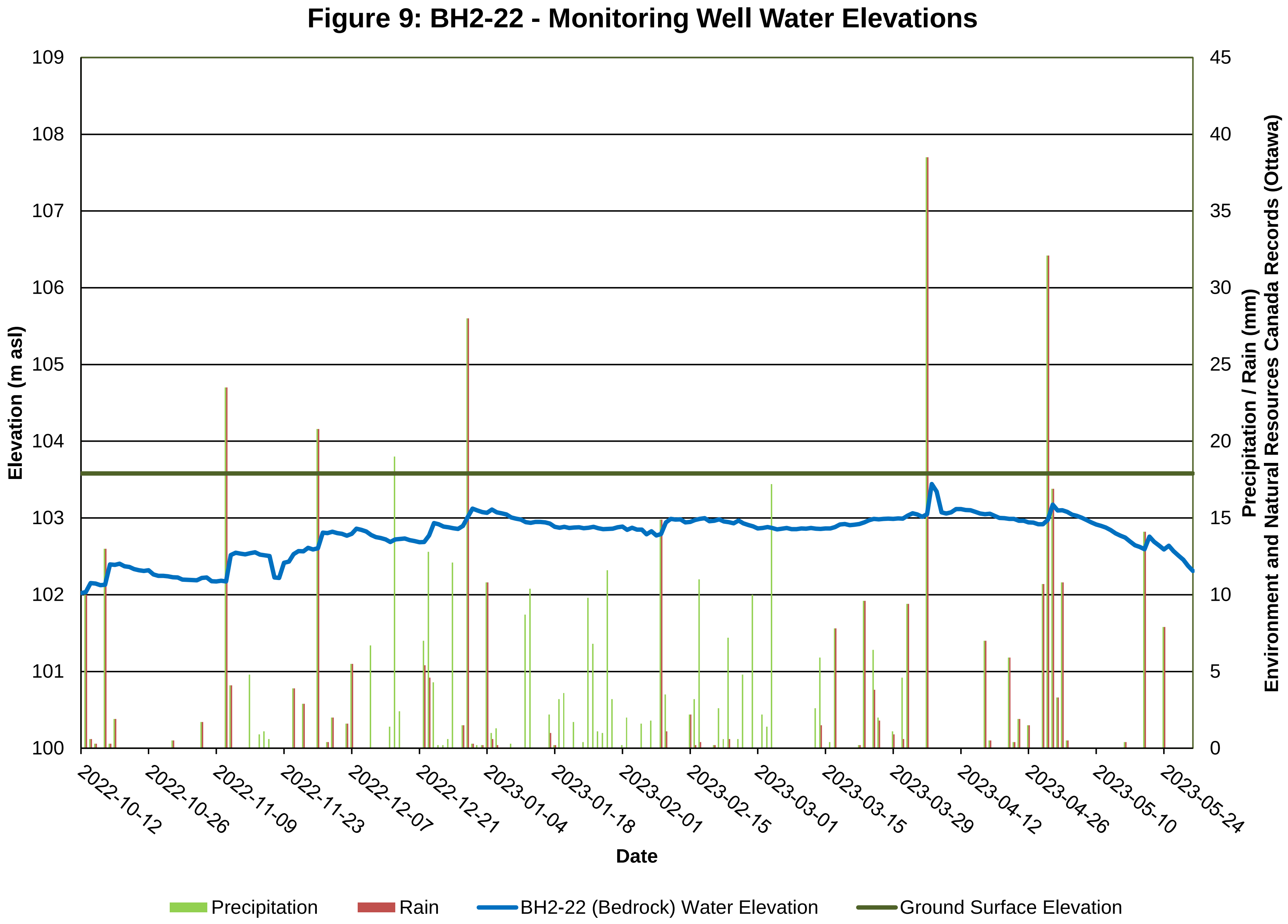
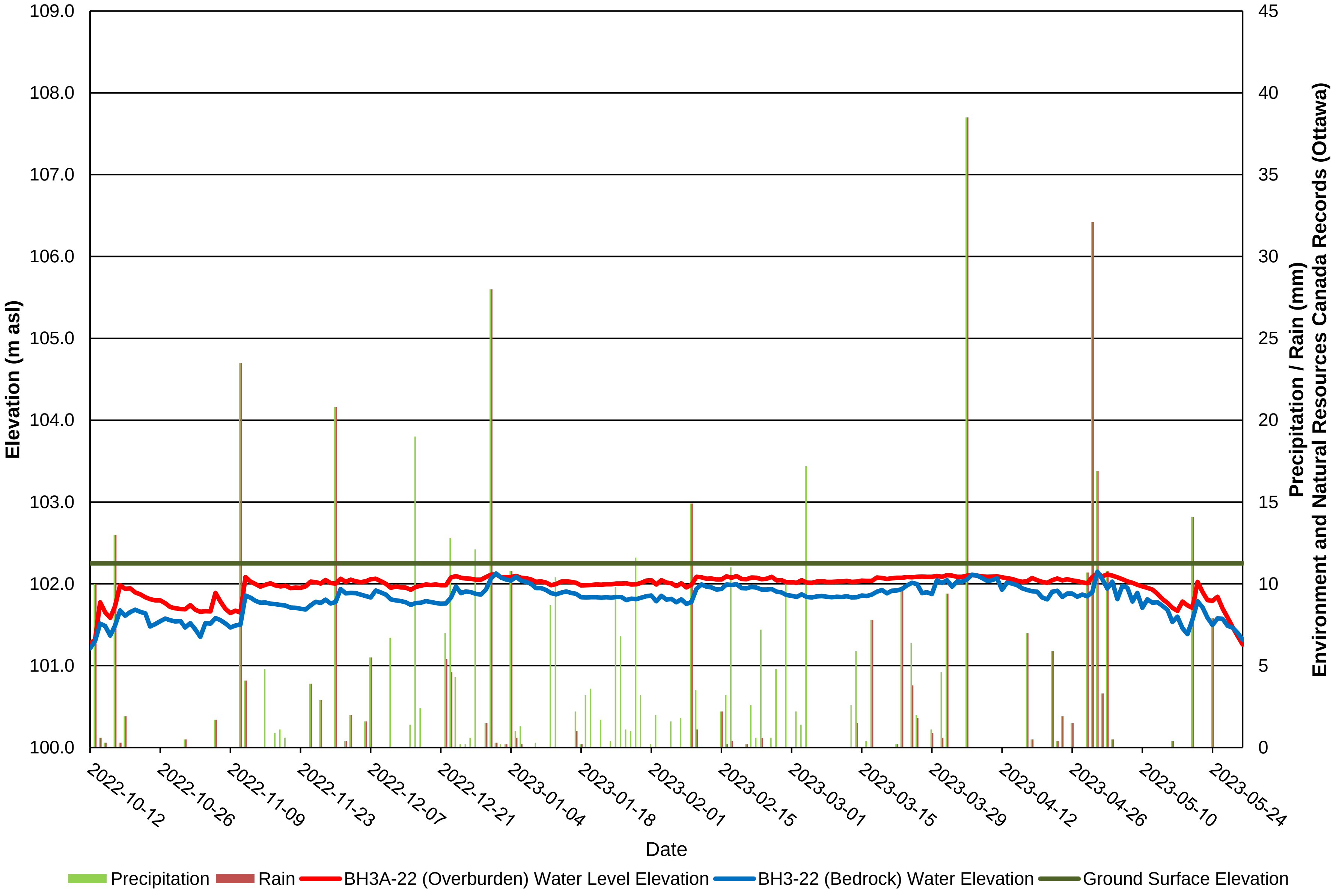


Figure 9: BH2-22 - Monitoring Well Water Elevations
Elevation (m asl)
109
108
107
106
105
104
103
102
101
100
Precipitation / Rain (mm)
Environment and Natural Resources Canada Records (Ottawa)
45
40
35
30
25
20
15
10
5
0
2022-10-12
2022-10-26
2022-11-09
2022-11-23
2022-12-07
2022-12-21
2023-01-04
2023-01-18
2023-02-01
2023-02-15
2023-03-01
2023-03-15
2023-03-29
2023-04-12
2023-04-26
2023-05-10
2023-05-24
Date
Precipitation
Rain
BH2-22 (Bedrock) Water Elevation
Ground Surface Elevation

Figure 10: BH3-22 & BH3A-22 - Monitoring Well Water Elevations



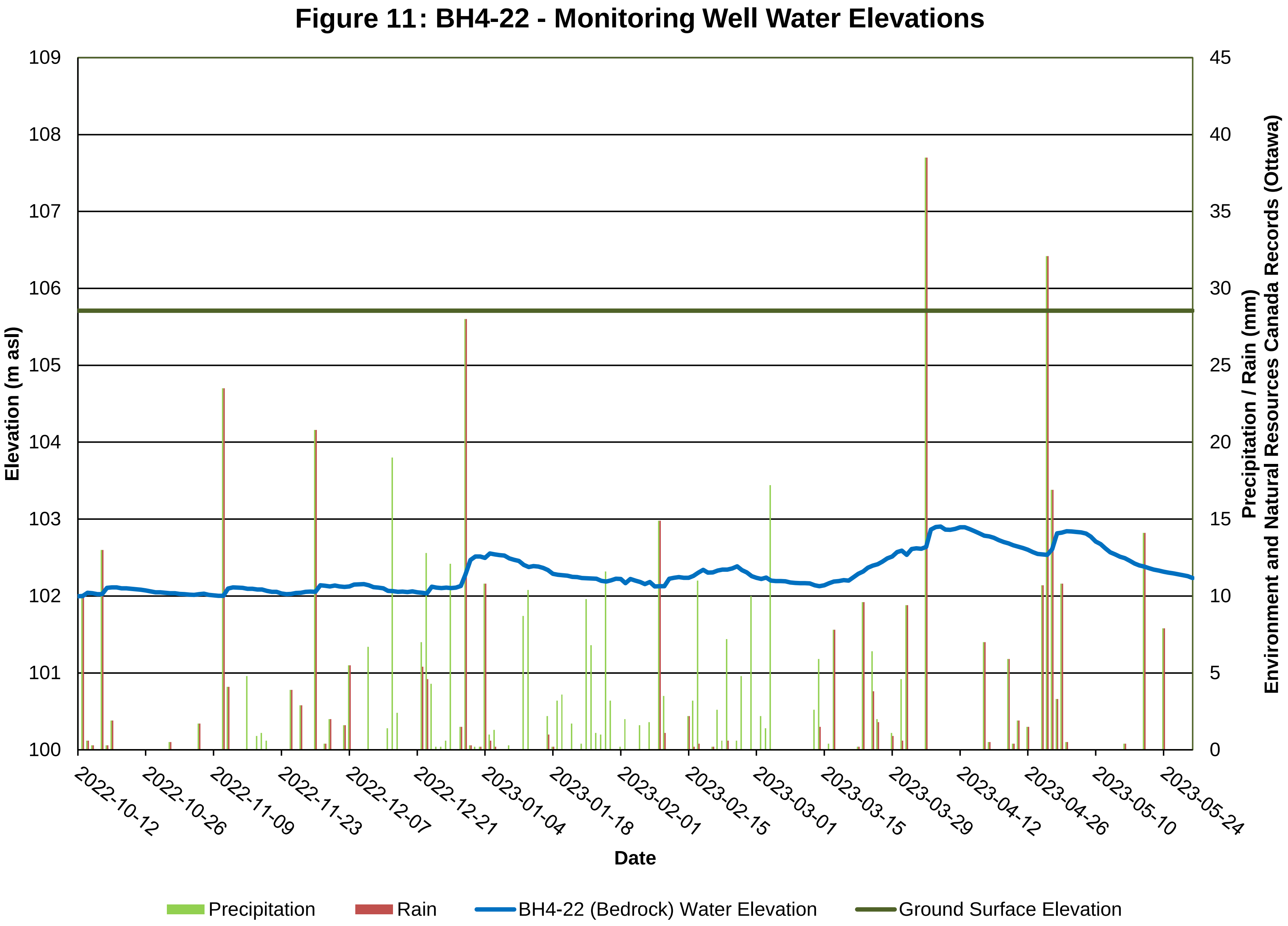


Figure 11: BH4-22 - Monitoring Well Water Elevations
Elevation (m asl)
109
108
107
106
105
104
103
102
101
100
Precipitation / Rain (mm)
Environment and Natural Resources Canada Records (Ottawa)
45
40
35
30
25
20
15
10
5
0
2022-10-12
2022-10-26
2022-11-09
2022-11-23
2022-12-07
2022-12-21
2023-01-04
2023-01-18
2023-02-01
2023-02-15
2023-03-01
2023-03-15
2023-03-29
2023-04-12
2023-04-26
2023-05-10
2023-05-24
Date
Precipitation
Rain
BH4-22 (Bedrock) Water Elevation
Ground Surface Elevation

Figure 12: BH5-22 - Monitoring Well Water Elevations

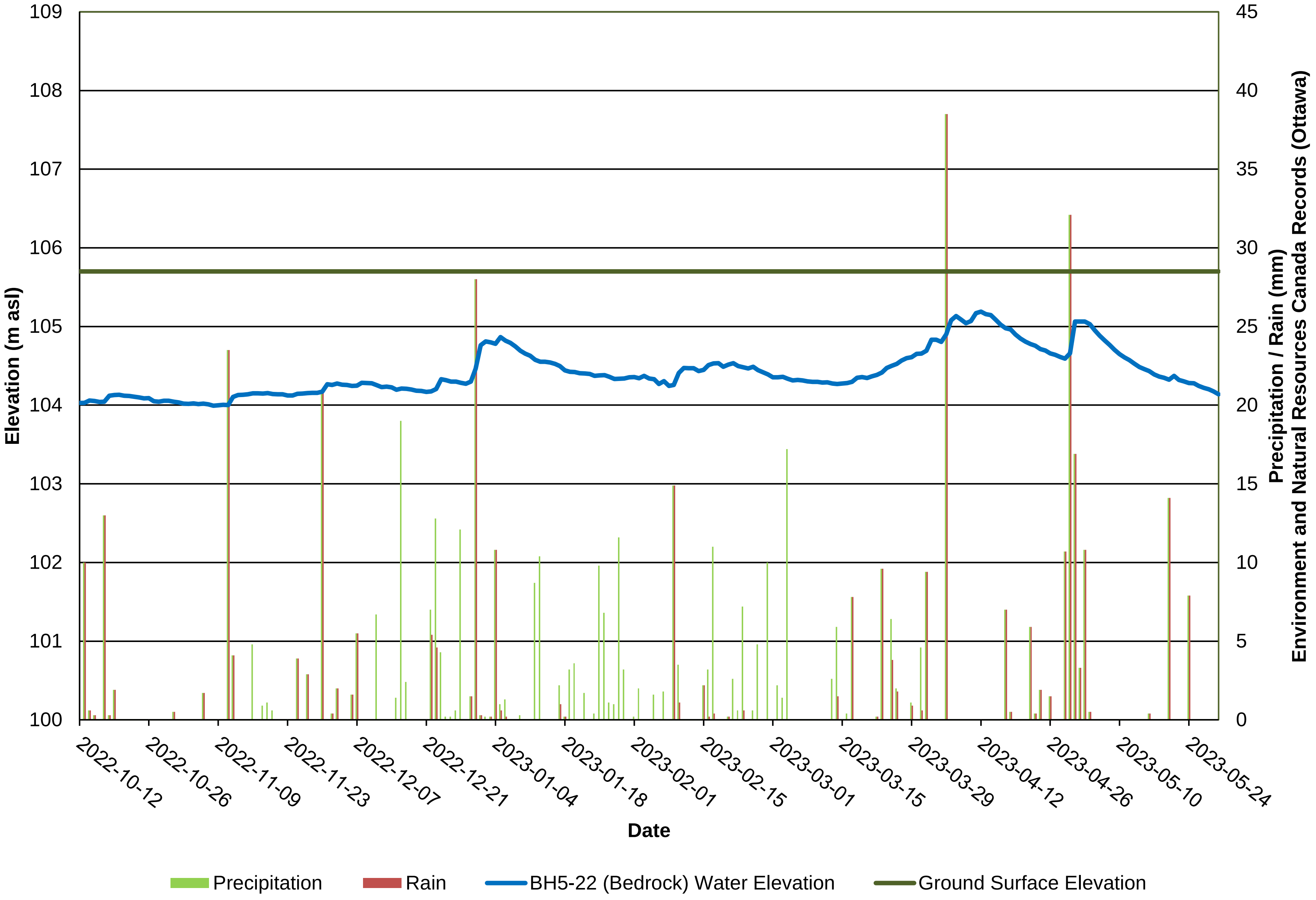


Figure 13: HA1-22 - Monitoring Well Water Elevations

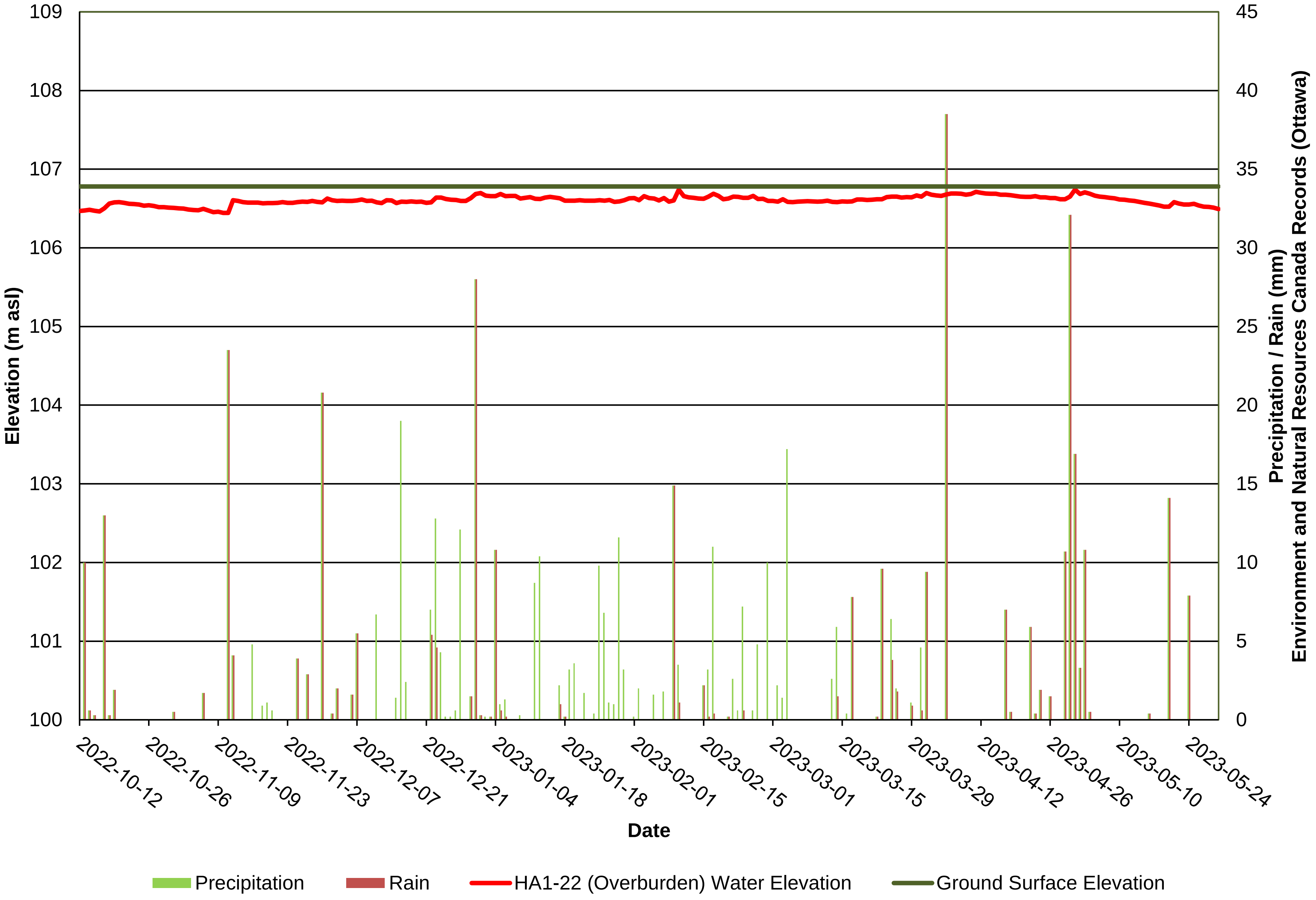


Table 1 - Monitoring Well Water Level Measurement Summary

Well ID		BH1-21	BH2-21	BH3-21	BH22A-21	BH24-21	BH33-21	HA1-22	BH1-22	BH1A-22	BH2-22	BH3-22	BH3A-22	BH4-22	BH5-22
Ground Surface Elevation (m asl)		104.29	107.19	108.41	102.98	103.07	104.7	106.78	107.31	107.31	103.58	102.25	102.25	105.71	105.7
Groundwater (GW) Measurements															
11-Jan-22	**GW Level (m bgs)**	1.22	0.82	0.89	2.49	0.67	1.84	Wells Were Not Installed At This Time							
	GW Elevation (m asl)	103.07	106.37	107.52	100.49	102.40	102.86								
11-Oct-22	**GW Level (m bgs)**	1.12	1.16	0.90	2.61	0.60	2.12	0.31	1.33	1.44	1.52	0.84	0.81	3.62	1.62
	GW Elevation (m asl)	103.17	106.03	107.51	100.37	102.47	102.59	106.48	105.99	105.87	102.06	101.42	101.44	102.10	104.09
28-Oct-22	**GW Level (m bgs)**	1.01	0.95	0.92	N/A	0.46	1.98	0.28	1.35	1.43	1.52	0.61	0.40	3.65	1.64
	GW Elevation (m asl)	103.28	106.25	107.49	N/A	102.61	102.72	106.51	105.97	105.88	102.06	101.64	101.85	102.07	104.06
04-Apr-23	**GW Level (m bgs)**	0.09	0.33	0.52	1.77	-0.03	1.20	0.14	0.83	0.94	0.59	0.11	0.00	3.08	0.90
	GW Elevation (m asl)	104.21	106.87	107.89	101.21	103.10	103.51	106.64	106.48	106.38	102.99	102.15	102.25	102.64	104.80
31-May-23	**GW Level (m bgs)**	0.97	0.87	0.84	2.72	0.74	2.22	0.29	1.35	1.46	1.31	0.93	0.99	3.48	1.56
	GW Elevation (m asl)	103.33	106.32	107.57	100.26	102.34	102.49	106.49	105.96	105.86	102.27	101.32	101.26	102.23	104.14

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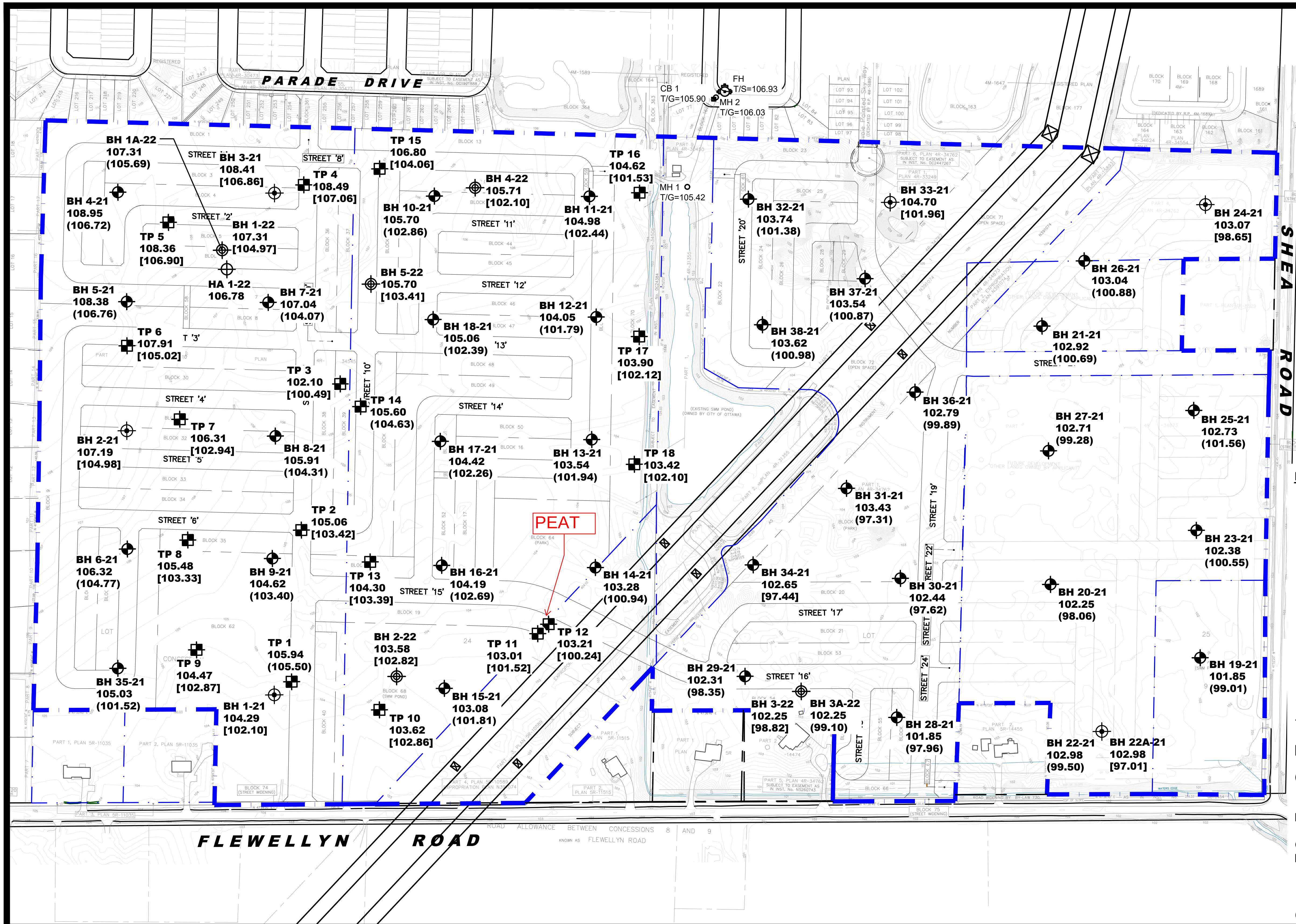

LEGEND:

⊕	HAND AUGER LOCATION (CURRENT INVESTIGATION)
◎	BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
●	BOREHOLE WITH PIEZOMETER LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)
▣	TEST PIT LOCATION\ (PATERSON GROUP REPORT; PG5570-1, JAN 3, 2021)
104.29	GROUND SURFACE ELEVATION (m)
[102.10]	BEDROCK SURFACE ELEVATION (m)
(102.39)	PRACTICAL REFUSAL TO AUGERING / EXCAVATION ELEVATION (m)

BASE PLAN PREPARED BY J.D. BARNES LIMITED.

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

SCALE: 1:4000

0 50 100 150 200 250m

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PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
6	UPDATED TO NEW CONCEPTUAL PLAN	03/07/2024	KP
5	UPDATED TO NEW CONCEPTUAL PLAN	28/08/2023	KP
4	UPDATED CLIENT'S NAME AND SITE ADDRESS	12/06/2023	KP
3	UPDATED SITE BOUNDARY	13/02/2023	KP
2	BH 1-22 - BH 5-22 & HA1-22 ADDED TO PLAN	10/03/2022	KP

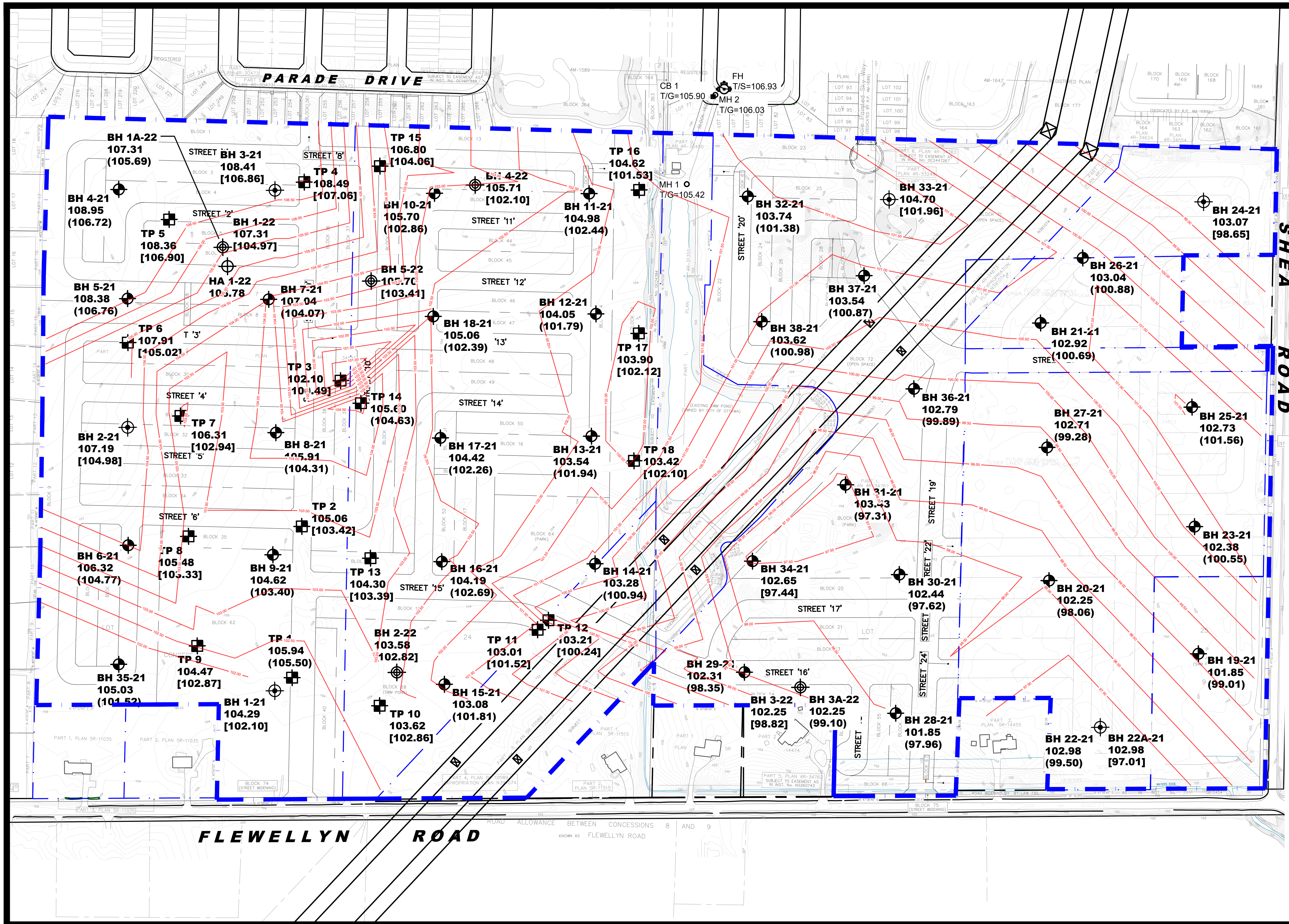
CAIVAN (STITTSVILLE SOUTH) INC. & CAIVAN (STITTSVILLE WEST) LTD.
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
OTTAWA, 5993 & 6115 FLEWELLYN ROAD & 6030 & 6070 FERNBANK ROAD ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:4000
Date: 01/2022
Drawn by: YA
Report No.: PG5570-2, REVISION 4
Checked by: KP
Dwg. No.: PG5570-1
Approved by: DJG
Revision No.: 6

11x17



LEGEND:

⌖	HAND AUGER LOCATION (CURRENT INVESTIGATION)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
⊕	BOREHOLE WITH PIEZOMETER LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)
■	TEST PIT LOCATION\ (PATERSON GROUP REPORT; PG5570-1, JAN 3, 2021)
97.50	BEDROCK CONTOUR (m)
104.29	GROUND SURFACE ELEVATION (m)
[102.10]	BEDROCK SURFACE ELEVATION (m)
(102.39)	PRACTICAL REFUSAL TO AUGERING / EXCAVATION ELEVATION (m)

BASE PLAN PREPARED BY J.D. BARNES LIMITED.

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

SCALE: 1:4000

0 50 100 150 200 250m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
6	UPDATED TO NEW CONCEPTUAL PLAN	03/07/2024	KP
5	UPDATED TO NEW CONCEPTUAL PLAN	28/08/2023	KP
4	UPDATED CLIENT'S NAME AND SITE ADDRESS	12/06/2023	KP
3	UPDATED SITE BOUNDARY	13/02/2023	KP
2	BH 1-22 - BH 5-22 & HA1-22 ADDED TO PLAN	10/03/2022	KP

CAIVAN (STITTSVILLE SOUTH) INC. & CAIVAN (STITTSVILLE WEST) LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
OTTAWA, 5993 & 6115 FLEWELLYN ROAD & 6030 & 6070 FERNBANK ROAD ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:4000
Date: 01/2022
Drawn by: YA
Report No.: PG5570-2, REVISION 4
Checked by: KP
Dwg. No.: PG5570-2
Approved by: DJG
Revision No.: 6

11x17

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APPENDIX 3

Liquefaction Resistance of Soils – Youd et al. (2001)

LIQUEFACTION RESISTANCE OF SOILS: SUMMARY REPORT FROM THE 1996 NCEER AND 1998 NCEER/NSF WORKSHOPS ON EVALUATION OF LIQUEFACTION RESISTANCE OF SOILS[a]

By T. L. Youd,[1] Member, ASCE, and I. M. Idriss,[2] Fellow, ASCE

ABSTRACT: Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Professors H. B. Seed and I. M. Idriss developed and published a methodology termed the ''simplified procedure'' for evaluating liquefaction resistance of soils. This procedure has become a standard of practice throughout North America and much of the world. The methodology which is largely empirical, has evolved over years, primarily through summary papers by H. B. Seed and his colleagues. No general review or update of the procedure has occurred, however, since 1985, the time of the last major paper by Professor Seed and a report from a National Research Council workshop on liquefaction of soils. In 1996 a workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) was convened by Professors T. L. Youd and I. M. Idriss with 20 experts to review developments over the previous 10 years. The purpose was to gain consensus on updates and augmentations to the simplified procedure. The following topics were reviewed and recommendations developed: (1) criteria based on standard penetration tests; (2) criteria based on cone penetration tests; (3) criteria based on shear-wave velocity measurements; (4) use of the Becker penetration test for gravelly soil; (4) magnitude scaling factors; (5) correction factors for overburden pressures and sloping ground; and (6) input values for earthquake magnitude and peak acceleration. Probabilistic and seismic energy analyses were reviewed but no recommendations were formulated.

INTRODUCTION

Over the past 25 years a methodology termed the ''simplified procedure'' has evolved as a standard of practice for evaluating the liquefaction resistance of soils. Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Seed and Idriss (1971) developed and published the basic ''simplified procedure.'' That procedure has been modified and improved periodically since that time, primarily through landmark papers by Seed (1979), Seed and Idriss (1982), and Seed et al. (1985). In 1985, Professor Robert V. Whitman convened a workshop on behalf of the National Research Council (NRC) in which 36 experts and observers thoroughly reviewed the state-of-knowledge and the state-of-the-art for assessing liquefaction hazard. That workshop produced a report (NRC 1985) that has become a widely used standard and reference for liquefaction hazard assessment. In January 1996, T. L. Youd and I. M. Idriss convened a workshop of 20 experts to update the simplified procedure and incorporate research findings from the previous decade. This paper summarizes recommendations from that workshop (Youd and Idriss 1997).

To keep the workshop focused, the scope of the workshop was limited to procedures for evaluating liquefaction resistance of soils under level to gently sloping ground. In this context, liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soils. Important postliquefaction phenomena, such as residual shear strength, soil deformation, and ground failure, were beyond the scope of the workshop.

The simplified procedure was developed from empirical evaluations of field observations and field and laboratory test data. Field evidence of liquefaction generally consisted of surficial observations of sand boils, ground fissures, or lateral spreads. Data were collected mostly from sites on level to gently sloping terrain, underlain by Holocene alluvial or fluvial sediment at shallow depths (<15 m). The original procedure was verified for, and is applicable only to, these site conditions. Similar restrictions apply to the implementation of the updated procedures recommended in this report.

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). Increased pore-water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs, the soil stratum softens, allowing large cyclic deformations to occur. In loose materials, the softening is also accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground. In moderately dense to dense materials, liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following liquefaction of moderately dense granular materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressures dissipate.

CYCLIC STRESS RATIO (CSR) AND CYCLIC RESISTANCE RATIO (CRR)

Calculation, or estimation, of two variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on a soil layer, expressed in terms of CSR; and (2) the capacity of the soil to resist liquefaction, expressed in terms of CRR. The latter variable has been termed the cyclic stress ratio or the cyclic stress ratio required to generate liquefaction, and has been given different symbols by different writers. For example, Seed and Harder (1990) used the symbol $CSR\ell$, Youd (1993) used the symbol CSRL, and Kramer

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[a] Workshop participants are listed on page 311.

[1] Prof., Brigham Young Univ., Provo, UT 84602.

[2] Prof., Univ. of California at Davis, Davis, CA 95616.

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(1996) used the symbol CSR_L to denote this ratio. To reduce confusion and to better distinguish induced cyclic shear stresses from mobilized liquefaction resistance, the capacity of a soil to resist liquefaction is termed the CRR in this report. This term is recommended for engineering practice.

EVALUATION OF CSR

Seed and Idriss (1971) formulated the following equation for calculation of the cyclic stress ratio:

$$\text{CSR} = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (1)$$

where a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake (discussed later); g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively; and r_d = stress reduction coefficient. The latter coefficient accounts for flexibility of the soil profile. The workshop participants recommend the following minor modification to the procedure for calculation of CSR.

For routine practice and noncritical projects, the following equations may be used to estimate average values of r_d (Liao and Whitman 1986b):

$$r_d = 1.0 - 0.00765z \quad \text{for } z \leq 9.15 \text{ m} \qquad (2a)$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \qquad (2b)$$

where z = depth below ground surface in meters. Some investigators have suggested additional equations for estimating r_d at greater depths (Robertson and Wride 1998), but evaluation of liquefaction at these greater depths is beyond the depths where the simplified procedure is verified and where routine applications should be applied. Mean values of r_d calculated from (2) are plotted in Fig. 1, along with the mean and range of values proposed by Seed and Idriss (1971). The workshop participants agreed that for convenience in programming spreadsheets and other electronic aids, and to be consistent with past practice, r_d values determined from (2) are suitable for use in routine engineering practice. The user should understand, however, that there is considerable variability in the flexibility and thus r_d at field sites, that r_d calculated from (2) are the mean of a wide range of possible r_d, and that the range of r_d increases with depth (Golesorkhi 1989).

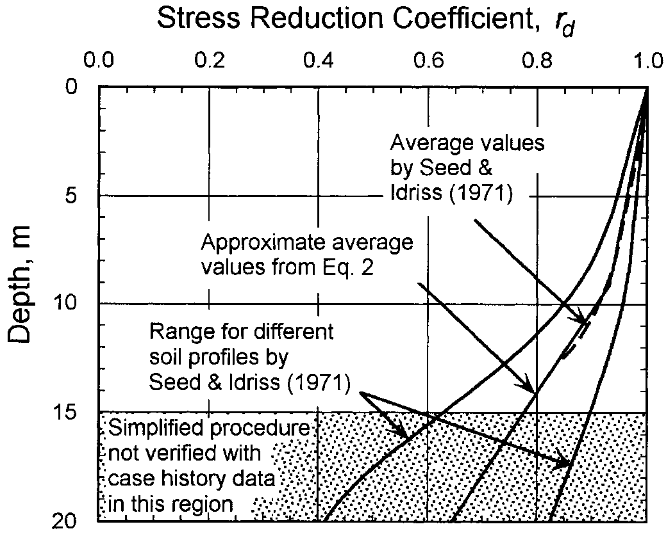


FIG. 1. r_d versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean-Value Lines Plotted from Eq. (2)

For ease of computation, T. F. Blake (personal communication, 1996) approximated the mean curve plotted in Fig. 1 by the following equation:

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \qquad (3)$$

where z = depth beneath ground surface in meters. Eq. (3) yields essentially the same values for r_d as (2), but is easier to program and may be used in routine engineering practice.

I. M. Idriss [Transportation Research Board (TRB) (1999)] suggested a new procedure for determining magnitude-dependent values of r_d. Application of these r_d require use of a corresponding set of magnitude scaling factors that are compatible with the new r_d. Because these r_d were developed after the workshop and have not been independently evaluated by other experts, the workshop participants chose not to recommend the new factors at this time.

EVALUATION OF LIQUEFACTION RESISTANCE (CRR)

A major focus of the workshop was on procedures for evaluating liquefaction resistance. A plausible method for evaluating CRR is to retrieve and test undisturbed soil specimens in the laboratory. Unfortunately, in situ stress states generally cannot be reestablished in the laboratory, and specimens of granular soils retrieved with typical drilling and sampling techniques are too disturbed to yield meaningful results. Only through specialized sampling techniques, such as ground freezing, can sufficiently undisturbed specimens be obtained. The cost of such procedures is generally prohibitive for all but the most critical projects. To avoid the difficulties associated with sampling and laboratory testing, field tests have become the state-of-practice for routine liquefaction investigations.

Several field tests have gained common usage for evaluation of liquefaction resistance, including the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements (V_s), and the Becker penetration test (BPT). These tests were discussed at the workshop, along with associated criteria for evaluating liquefaction resistance. The participants made a conscientious attempt to correlate liquefaction resistance criteria from each of the various field tests to provide generally consistent results, no matter which test is applied. SPTs and CPTs are generally preferred because of the more extensive databases and past experience, but the other tests may be applied at sites underlain by gravelly sediment

TABLE 1. Comparison of Advantages and Disadvantages of Various Field Tests for Assessment of Liquefaction Resistance

Feature (1)	Test Type SPT (2)	CPT (3)	V_s (4)	BPT (5)
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

or where access by large equipment is limited. Primary advantages and disadvantages of each test are listed in Table 1.

SPT

Criteria for evaluation of liquefaction resistance based on the SPT have been rather robust over the years. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ plot reproduced in Fig. 2. $(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure of approximately 100 kPa (1 ton/sq ft) and a hammer energy ratio or hammer efficiency of 60%. The normalization factors for these corrections are discussed in the section entitled Other Corrections. Fig. 2 is a graph of calculated CSR and corresponding $(N_1)_{60}$ data from sites where liquefaction effects were or were not observed following past earthquakes with magnitudes of approximately 7.5. CRR curves on this graph were conservatively positioned to separate regions with data indicative of liquefaction from regions with data indicative of nonliquefaction. Curves were developed for granular soils with the fines contents of 5% or less, 15%, and 35% as shown on the plot. The CRR curve for fines contents <5% is the basic penetration criterion for the simplified procedure and is referred to hereafter as the ''SPT clean-sand base curve.'' The CRR curves in Fig. 2 are valid only for magnitude 7.5 earthquakes. Scaling factors to adjust CRR curves to other magnitudes are addressed in a later section of this report.

SPT Clean-Sand Base Curve

Several changes to the SPT criteria are recommended by the workshop participants. The first change is to curve the trajectory of the clean-sand base curve at low $(N_1)_{60}$ to a projected intercept of about 0.05 (Fig. 2). This adjustment reshapes the clean-sand base curve to achieve greater consistency with CRR curves developed for the CPT and shear-wave velocity procedures. Seed and Idriss (1982) projected the original curve through the origin, but there were few data to constrain the curve in the lower part of the plot. A better fit to the present empirical data is to bow the lower end of the base curve as indicated in Fig. 2.

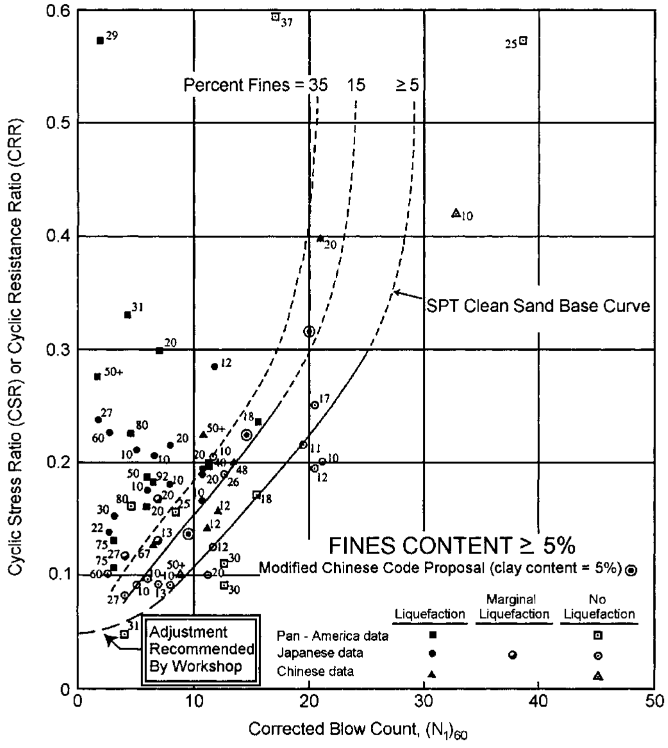


FIG. 2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et al. 1985)

At the University of Texas, A. F. Rauch (personal communication, 1998), approximated the clean-sand base curve plotted in Fig. 2 by the following equation:

$$\mathrm{CRR}_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (4)$$

This equation is valid for $(N_1)_{60} < 30$. For $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as nonliquefiable. This equation may be used in spreadsheets and other analytical techniques to approximate the clean-sand base curve for routine engineering calculations.

Influence of Fines Content

In the original development, Seed et al. (1985) noted an apparent increase of CRR with increased fines content. Whether this increase is caused by an increase of liquefaction resistance or a decrease of penetration resistance is not clear. Based on the empirical data available, Seed et al. developed CRR curves for various fines contents reproduced in Fig. 2. A revised correction for fines content was developed by workshop attendees to better fit the empirical database and to better support computations with spreadsheets and other electronic computational aids.

The workshop participants recommend (5) and (6) as approximate corrections for the influence of fines content (FC) on CRR. Other grain characteristics, such as soil plasticity, may affect liquefaction resistance as well as fines content, but widely accepted corrections for these factors have not been developed. Hence corrections based solely on fines content should be used with engineering judgment and caution. The following equations were developed by I. M. Idriss with the assistance of R. B. Seed for correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \quad (5)$$

where α and β = coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for FC} \leq 5\% \quad (6a)$$

$$\alpha = \exp[1.76 - (190/\mathrm{FC}^2)] \quad \text{for } 5\% < \text{FC} < 35\% \quad (6b)$$

$$\alpha = 5.0 \quad \text{for FC} \geq 35\% \quad (6c)$$

$$\beta = 1.0 \quad \text{for FC} \leq 5\% \quad (7a)$$

$$\beta = [0.99 + (\mathrm{FC}^{1.5}/1{,}000)] \quad \text{for } 5\% < \text{FC} < 35\% \quad (7b)$$

$$\beta = 1.2 \quad \text{for FC} \geq 35\% \quad (7c)$$

These equations may be used for routine liquefaction resistance calculations. A back-calculated curve for a fines content of 35% is essentially congruent with the 35% curve plotted in Fig. 2. The back-calculated curve for a fines contents of 15% plots to the right of the original 15% curve.

Other Corrections

Several factors in addition to fines content and grain characteristics influence SPT results, as noted in Table 2. Eq. (8) incorporates these corrections

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad (8)$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction

TABLE 2. Corrections to SPT (Modified from Skempton 1986) as Listed by Robertson and Wride (1998)

Factor (1)	Equipment variable (2)	Term (3)	Correction (4)
Overburden pressure	—	C_N	$(P_a/\sigma'_{vo})^{9.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

factor for rod length; and C_S = correction for samplers with or without liners.

Because SPT N-values increase with increasing effective overburden stress, an overburden stress correction factor is applied (Seed and Idriss 1982). This factor is commonly calculated from the following equation (Liao and Whitman 1986a):

$$C_N = (P_a/\sigma'_{vo})^{0.5} \qquad (9)$$

where C_N normalizes N_m to an effective overburden pressure σ'_{vo} of approximately 100 kPa (1 atm) P_a. C_N should not exceed a value of 1.7 [A maximum value of 2.0 was published in the National Center for Earthquake Engineering Research (NCEER) workshop proceedings (Youd and Idriss 1997), but later was reduced to 1.7 by consensus of the workshop participants] Kayen et al. (1992) suggested the following equation, which limits the maximum C_N value to 1.7, and in these writers' opinion, provides a better fit to the original curve specified by Seed and Idriss (1982):

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a) \qquad (10)$$

Either equation may be used for routine engineering applications.

The effective overburden pressure σ'_{vo} applied in (9) and (10) should be the overburden pressure at the time of drilling and testing. Although a higher ground-water level might be used for conservatism in the liquefaction resistance calculations, the C_N factor must be based on the stresses present at the time of the testing.

The C_N correction factor was derived from SPT performed in test bins with large sand specimens subjected to various confining pressures (Gibbs and Holtz 1957; Marcuson and Bieganousky 1997a,b). The results of several of these tests are reproduced in Fig. 3 in the form of C_N curves versus effective overburden stress (Castro 1995). These curves indicate considerable scatter of results with no apparent correlation of C_N with soil type or gradation. The curves from looser sands, however, lie in the lower part of the C_N range and are reasonably approximated by (9) and (10) for low effective overburden pressures [200 kPa (<2 tsf)]. The workshop participants endorsed the use of (9) for calculation of C_N, but acknowledged that for overburden pressures >200 kPa (2 tsf) the results are uncertain. Eq. (10) provides a better fit for overburden pressures up to 300 kPa (3 tsf). For pressures >300 kPa (3 tsf), the uncertainty is so great that (9) should not be applied. At these high pressures, which are generally below the depth for which the simplified procedure has been verified, C_N should be estimated by other means.

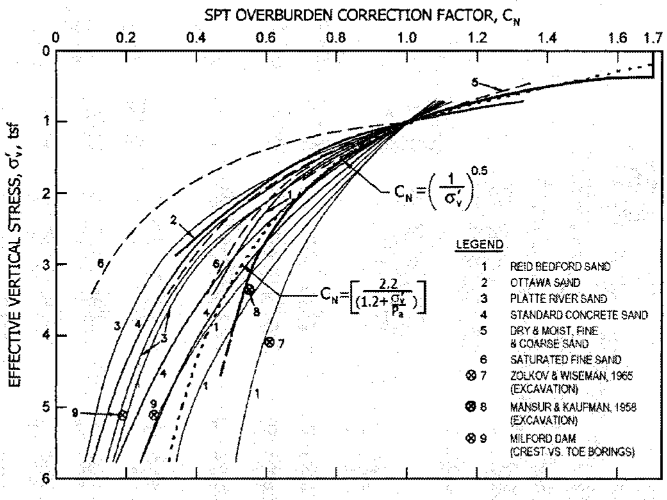


FIG. 3. C_N **Curves for Various Sands Based on Field and Laboratory Test Data along with Suggested** C_N **Curve Determined from Eqs. (9) and (10) (Modified from Castro 1995)**

Another important factor is the energy transferred from the falling hammer to the SPT sampler. An ER of 60% is generally accepted as the approximate average for U.S. testing practice and as a reference value for energy corrections. The ER delivered to the sampler depends on the type of hammer, anvil, lifting mechanism, and the method of hammer release. Approximate correction factors (C_E = ER/60) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 2. Because of variations in drilling and testing equipment and differences in testing procedures, a rather wide range in the energy correction factor C_E has been observed as noted in the table. Even when procedures are carefully monitored to conform to established standards, such as ASTM D 1586-99, some variation in C_E may occur because of minor variations in testing procedures. Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%. The workshop participants recommend measurement of the hammer energy frequently at each site where the SPT is used. Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate a C_E value for use in liquefaction resistance calculations. Use of good-quality testing equipment and carefully controlled testing procedures conforming to ASTM D 1586-99 will generally yield more consistent energy ratios and C_E with values from the upper parts of the ranges listed in Table 2.

Skempton (1986) suggested and Robertson and Wride (1998) updated correction factors for rod lengths <10 m, borehole diameters outside the recommended interval (65–125 mm), and sampling tubes without liners. Range for these correction factors are listed in Table 2. For liquefaction resistance calculations and rod lengths <3 m, a C_R of 0.75 should be applied as was done by Seed et al. (1985) in formulating the simplified procedure. Although application of rod-length correction factors listed in Table 2 will give more precise $(N_1)_{60}$ values, these corrections may be neglected for liquefaction resistance calculations for rod lengths between 3 and 10 m because rod-length corrections were not applied to SPT test data from these depths in compiling the original liquefaction case history databases. Thus rod-length corrections are implicitly incorporated into the empirical SPT procedure.

A final change recommended by workshop participants is the use of revised magnitude scaling factors rather than the original Seed and Idriss (1982) factors to adjust CRR for earthquake magnitudes other than 7.5. Magnitude scaling factors are addressed later in this report.

CPT

A primary advantage of the CPT is that a nearly continuous profile of penetration resistance is developed for stratigraphic interpretation. The CPT results are generally more consistent and repeatable than results from other penetration tests listed in Table 1. The continuous profile also allows a more detailed definition of soil layers than the other tools listed in the table. This stratigraphic capability makes the CPT particularly advantageous for developing liquefaction-resistance profiles. Interpretations based on the CPT, however, must be verified with a few well-placed boreholes preferably with standard penetration tests, to confirm soil types and further verify liquefaction-resistance interpretations.

Fig. 4 provides curves prepared by Robertson and Wride (1998) for direct determination of CRR for clean sands (FC $\leq$ 5%) from CPT data. This figure was developed from CPT case history data compiled from several investigations, including those by Stark and Olson (1995) and Suzuki et al. (1995). The chart, valid for magnitude 7.5 earthquakes only, shows calculated cyclic resistance ratio plotted as a function of dimensionless, corrected, and normalized CPT resistance q_{c1N} from sites where surface effects of liquefaction were or were not observed following past earthquakes. The CRR curve conservatively separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction.

Based on a few misclassified case histories from the 1989 Loma Prieta earthquake, I. M. Idriss suggested that the clean sand curve in Fig. 4 should be shifted to the right by 10–15%. However, a majority of workshop participants supported a curve in its present position, for three reasons. First, purpose of the workshop was to recommend criteria that yield roughly equivalent CRR for the field tests listed in Table 1. Shifting the base curve to the right makes the CPT criteria generally more conservative. For example, for $(N_1)_{60} > 5$, q_{c1N}:$(N_1)_{60}$ ratios between the two clean-sand base curves, plotted in Figs. 4 and 2, respectively, range from 5 to 8—values that are slightly higher than those expected for clean sands. Shifting the CPT base curve to the right by 10 to 15% would increase those ratios to unusually high values ranging from 6 to 9. Second, base curves, such as those plotted in Figs. 2 and 4, were intended to be conservative, but not necessarily to encompass every data point on the plot. Thus the presence of a few points beyond the base curve should be allowable. Finally, several studies have confirmed that the CPT criteria in Fig. 4 are generally conservative. Robertson and Wride (1998) verified these criteria against SPT and other data from sites they investigated. Gilstrap and Youd (1998) compared calculated liquefaction resistances against field performance at 19 sites and concluded that the CPT criteria correctly predicted the occurrence or nonoccurrence of liquefaction with >85% reliability.

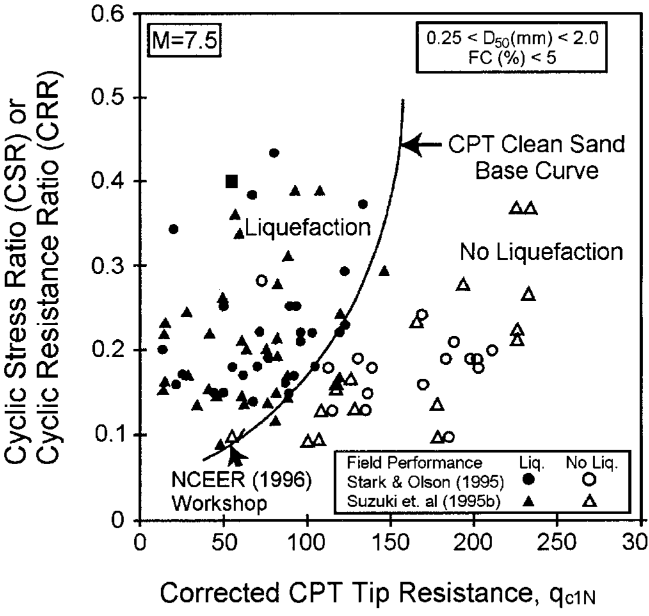


FIG. 4. Curve Recommended for Calculation of CRR from CPT Data along with Empirical Liquefaction Data from Compiled Case Histories (Reproduced from Robertson and Wride 1998)

The clean-sand base curve in Fig. 4 may be approximated by the following equation (Robertson and Wride 1998):

$$\text{If } (q_{c1N})_{cs} < 50 \quad CRR_{7.5} = 0.833[(q_{c1N})_{cs}/1{,}000] + 0.05 \qquad (11a)$$

$$\text{If } 50 \leq (q_{c1N})_{cs} < 160 \quad CRR_{7.5} = 93[(q_{c1N})_{cs}/1{,}000]^3 + 0.08 \qquad (11b)$$

where $(q_{c1N})_{cs}$ = clean-sand cone penetration resistance normalized to approximately 100 kPa (1 atm).

Normalization of Cone Penetration Resistance

The CPT procedure requires normalization of tip resistance using (12) and (13). This transformation yields normalized, dimensionless cone pentration resistance q_{c1N}

$$q_{c1N} = C_Q(q_c/P_a) \qquad (12)$$

where

$$C_Q = (P_a/\sigma'_{vo})^n \qquad (13)$$

and where C_Q = normalizing factor for cone penetration resistance; P_a = 1 atm of pressure in the same units used for σ'_{vo}; n = exponent that varies with soil type; and q_c = field cone penetration resistance measured at the tip. At shallow depths C_Q becomes large because of low overburden pressure; however, values >1.7 should not be applied. As noted in the following paragraphs, the value of the exponent n varies from 0.5 to 1.0, depending on the grain characteristics of the soil (Olsen 1997).

The CPT friction ratio (sleeve resistance f_s divided by cone tip resistance q_c) generally increases with increasing fines content and soil plasticity, allowing rough estimates of soil type and fines content to be determined from CPT data. Robertson and Wride (1998) constructed the chart reproduced in Fig. 5 for estimation of soil type. The boundaries between soil types 2–7 can be approximated by concentric circles and can be used to account for effects of soil characteristics on q_{c1N} and CRR. The radius of these circles, termed the soil behavior type index I_c is calculated from the following equation:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \qquad (14)$$

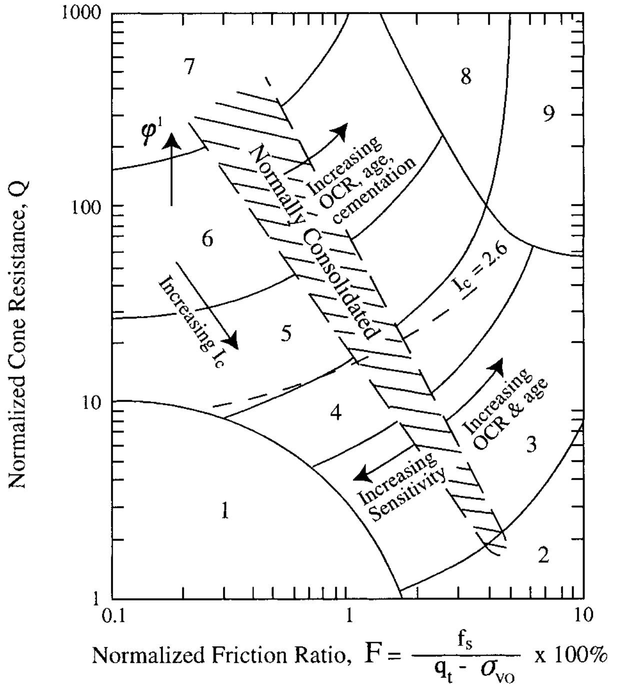
where

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \qquad (15)$$

and

$$F = [f_s/(q_c - \sigma_{vo})] \times 100\% \qquad (16)$$

The soil behavior chart in Fig. 5 was developed using an exponent n of 1.0, which is the appropriate value for clayey soil types. For clean sands, however, an exponent value of 0.5 is more appropriate, and a value intermediate between 0.5 and 1.0 would be appropriate for silts and sandy silts. Robertson and Wride recommended the following procedure for calculating the soil behavior type index I_c. The first step is to differentiate soil types characterized as clays from soil types characterized as sands and silts. This differentiation is performed by assuming an exponent n of 1.0 (characteristic of clays) and calculating the dimensionless CPT tip resistance Q from the following equation:



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

**Heavily overconsolidated or cemented*

FIG. 5. CPT-Based Soil Behavior-Type Chart Proposed by Robertson (1990)

$$Q = [(q_c - \sigma_{vo})/P_a][P_a/\sigma'_{vo}]^{1.0} = [(q_c - \sigma_{vo})/\sigma'_{vo}] \qquad (17)$$

If the I_c calculated with an exponent of 1.0 is >2.6, the soil is classified as clayey and is considered too clay-rich to liquefy, and the analysis is complete. However, soil samples should be retrieved and tested to confirm the soil type and liquefaction resistance. Criteria such as the Chinese criteria might be applied to confirm that the soil is nonliquefiable. The so-called Chinese criteria, as defined by Seed and Idriss (1982), specify that liquefaction can only occur if all three of the following conditions are met:

1. The clay content (particles smaller than 5 μ) is <15% by weight.
2. The liquid limit is <35%.
3. The natural moisture content is >0.9 times the liquid limit.

If the calculated I_c is <2.6, the soil is most likely granular in nature, and therefore C_q and Q should be recalculated using an exponent n of 0.5. I_c should then be recalculated using (14). If the recalculated I_c is <2.6, the soil is classed as nonplastic and granular. This I_c is used to estimate liquefaction resistance, as noted in the next section. However, if the recalculated I_c is >2.6, the soil is likely to be very silty and possibly plastic. In this instance, q_{c1N} should be recalculated from (12) using an intermediate exponent n of 0.7 in (13). I_c is then recalculated from (14) using the recalculated value for q_{c1N}. This intermediate I_c is then used to calculate liquefaction resistance. In this instance, a soil sample should be retrieved and tested to verify the soil type and whether the soil is liquefiable by other criteria, such as the Chinese criteria.

Because the relationship between I_c and soil type is approximate, the consensus of the workshop participants is that all soils with an I_c of 2.4 or greater should be sampled and tested to confirm the soil type and to test the liquefiability with other criteria. Also, soil layers characterized by an I_c > 2.6, but with a normalized friction ratio F < 1.0% (region 1 of Fig. 5) may be very sensitive and should be sampled and tested. Although not technically liquefiable according to the Chinese criteria, such sensitive soils may suffer softening and strength loss during earthquake shaking.

Calculation of Clean-Sand Equivalent Normalized Cone Penetration Resistance $(q_{c1N})_{cs}$

The normalized penetration resistance (q_{c1N}) for silty sands is corrected to an equivalent clean sand value $(q_{c1N})_{cs}$, by the following relationship:

$$(q_{c1N})_{cs} = K_c q_{c1N} \qquad (18)$$

where K_c, the correction factor for grain characteristics, is defined by the following equation (Robertson and Wride 1998):

$$\text{for } I_c \le 1.64 \quad K_c = 1.0 \qquad (19a)$$

$$\text{for } I_c > 1.64 \quad K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88 \qquad (19b)$$

The K_c curve defined by (19) is plotted in Fig. 6. For I_c > 2.6, the curve is shown as a dashed line, indicating that soils in this range of I_c are most likely too clay-rich or plastic to liquefy.

With an appropriate I_c and K_c, (11) and (19) can be used to calculate $CRR_{7.5}$. To adjust CRR to magnitudes other than 7.5, the calculated $CRR_{7.5}$ is multiplied by an appropriate magnitude scaling factor. The same magnitude scaling factors are used with CPT data as with SPT data. Magnitude scaling factors are discussed in a later section of this report.

Olsen (1997) and Suzuki et al. (1995) Procedures

Olsen (1997), who pioneered many of the techniques for assessing liquefaction resistance from CPT soundings, suggested a somewhat different procedure for calculating CRR from CPT data. Reasons for recommending the Robertson and Wride (1998) procedure over that of Olsen are the ease of application and the ease with which relationships can be quantified for computer-aided calculations. Results from Olsen's procedure, however, are consistent with results from the procedure proposed here for shallow (<15 m deep) sediment be-

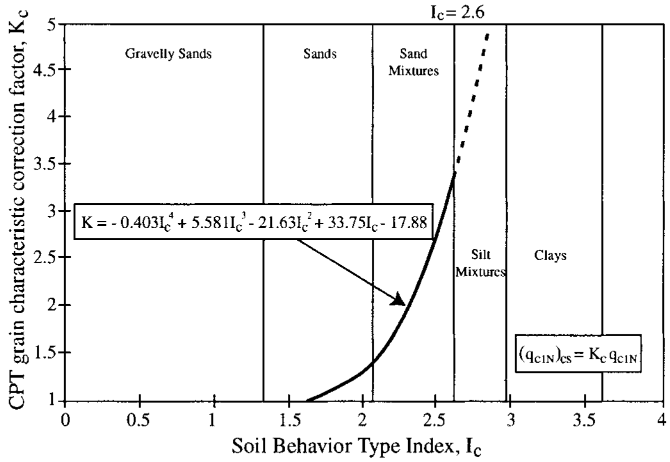


FIG. 6. Grain-Characteristic Correction Factor K_c for Determination of Clean-Sand Equivalent CPT Resistance (Reproduced from Robertson and Wride 1998)

neath level to gently sloping terrain. Olsen (1997) noted that almost any CPT normalization technique will give results consistent with his normalization procedure for soil layers in the 3–15 m depth range. For deeper layers, significant differences may develop between the two procedures. Those depths are also beyond the depth for which the simplified procedure has been verified. Hence any procedure based on the simplified procedure yields rather uncertain results at depths >15 m.

Suzuki et al. (1995) also developed criteria for evaluating CRR from CPT data. Those criteria are slightly more conservative than those of Robertson and Wride (1998) and were considered by the latter investigators in developing the criteria recommended herein.

Correction of Cone Penetration Resistance for Thin Soil Layers

Theoretical as well as laboratory studies indicate that CPT tip resistance is influenced by softer soil layers above or below the cone tip. As a result, measured CPT tip resistance is smaller in thin layers of granular soils sandwiched between softer layers than in thicker layers of the same granular soil. The amount of the reduction of penetration resistance in soft layers is a function of the thickness of the softer layer and the stiffness of the stiffer layers.

Using a simplified elastic solution, Vreugdenhil et al. (1994) developed a procedure for estimating the thick-layer equivalent cone penetration resistance of thin stiff layers lying within softer strata. The correction applies only to thin stiff layers embedded within thick soft layers. Because the corrections have a reasonable trend, but appear rather large, Robertson and Fear (1995) recommended conservative corrections from the $q_{cA}/q_{cB} = 2$ curve sketched in Fig. 7.

Further analysis of field data by Gonzalo Castro and Peter Robertson for the NCEER workshop indicates that corrections based on the $q_{cA}/q_{cB} = 2$ curve may still be too large and not adequately conservative. They suggested, and the workshop participants agreed, that the lower bound of the range of field data plotted by G. Castro in Fig. 7 provides more conservative K_H values that should be used until further field studies and analyses indicate that higher values are viable. The equation for the lower bound of the field curve is

$$K_H = 0.25[((H/d_c)/17) - 1.77]^2 + 1.0 \qquad (20)$$

where H = thickness of the interbedded layer in mm; q_{cA} and q_{cB} = cone resistances of the stiff and soft layers, respectively; and d_c = diameter of the cone in mm (Fig. 7).

V_s

Andrus and Stokoe (1997, 2000) developed liquefaction resistance criteria from field measurements of shear wave velocity V_s. The use of V_s as a field index of liquefaction resistance is soundly based because both V_s and CRR are similarly, but not proportionally, influenced by void ratio, effective confining stresses, stress history, and geologic age. The advantages of using V_s include the following: (1) V_s measurements are possible in soils that are difficult to penetrate with CPT and SPT or to extract undisturbed samples, such as gravelly soils, and at sites where borings or soundings may not be permitted; (2) V_s is a basic mechanical property of soil materials, directly related to small-strain shear modulus; and (3) the small-strain shear modulus is a parameter required in analytical procedures for estimating dynamic soil response and soil-structure interaction analyses.

Three concerns arise when using V_s for liquefaction-resistance evaluations: (1) seismic wave velocity measurements are made at small strains, whereas pore-water pressure buildup and the onset of liquefaction are medium- to high-strain phenomena; (2) seismic testing does not provide samples for classification of soils and identification of nonliquefiable soft clay-rich soils; and (3) thin, low V_s strata may not be detected if the measurement interval is too large. Therefore the preferred practice is to drill sufficient boreholes and conduct in situ tests to detect and delineate thin liquefiable strata, nonliquefiable clay-rich soils, and silty soils above the ground-water table that might become liquefiable should the water table rise. Other tests, such as the SPT or CPT, are needed to detect liquefiable weakly cemented soils that may have high V_s values.

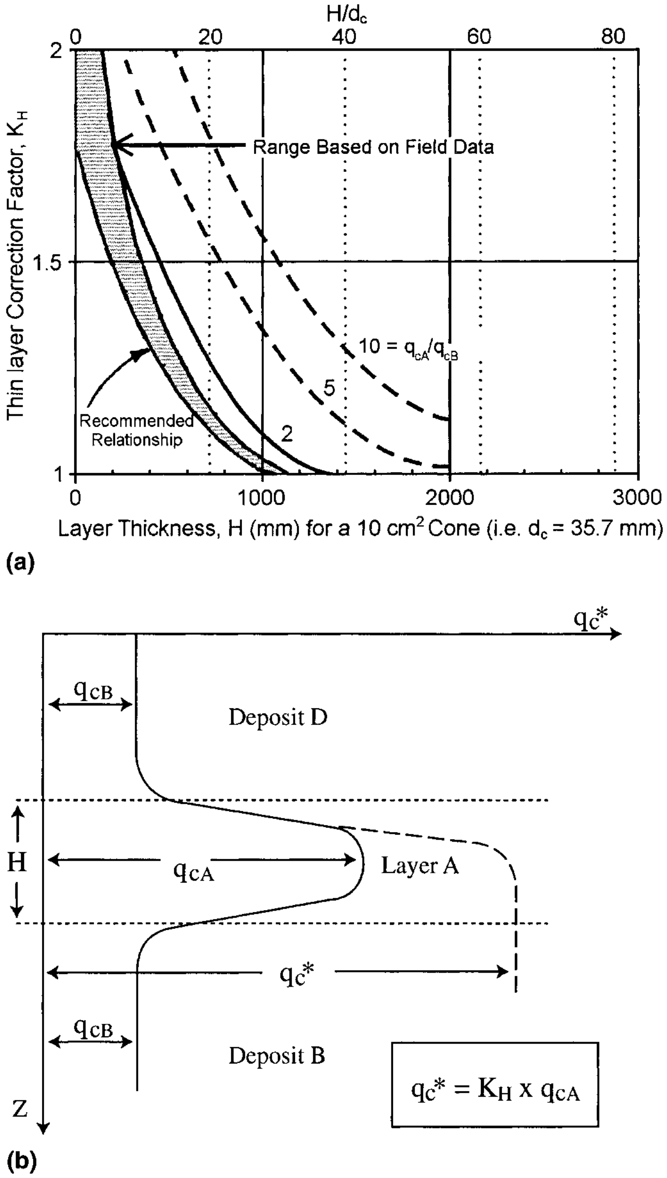


FIG. 7. Thin-Layer Correction Factor K_H for Determination of Equivalent Thick-Layer CPT Resistance (Modified from Robertson and Fear 1995)

V_s Criteria for Evaluating Liquefaction Resistance

Following the traditional procedures for correcting penetration resistance to account for overburden stress, V_s is also corrected to a reference overburden stress using the following equation (Sykora 1987; Kayen et al. 1992; Robertson et al. 1992):

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25} \qquad (21)$$

where V_{s1} = overburden-stress corrected shear wave velocity; P_a = atmospheric pressure approximated by 100 kPa (1 TSF);

and σ'_{vo} = initial effective vertical stress in the same units as P_a. Eq. (21) implicitly assumes a constant coefficient of earth pressure K'_o which is approximately 0.5 for sites susceptible to liquefaction. Application of (21) also implicitly assumes that V_s is measured with both the directions of particle motion and wave propagation polarized along principal stress directions and that one of those directions is vertical (Stokoe et al. 1985).

Fig. 8 compares seven CRR-V_{s1} curves. The "best fit" curve by Tokimatsu and Uchida (1990) was determined from laboratory cyclic triaxial test results for various sands with <10% fines and 15 cycles of loading. The more conservative "lower bound" curve for Tokimatsu and Uchida's laboratory test results is also shown as a lower bound for liquefaction occurrences. The bounding curve by Robertson et al. (1992) was developed using field performance data from sites in Imperial Valley, Calif., along with data from four other sites. The curves by Kayen et al. (1992) and Lodge (1994) are from sites that did and did not liquefy during the 1989 Loma Prieta earthquake. Andrus and Stokoe's (1997) curve was developed for uncemented, Holocene-age soils with 5% or less fines using field performance data from 20 earthquakes and over 50 measurement sites. Andrus and Stokoe (2000) revised this curve based on new information and an expanded database that includes 26 earthquakes and more than 70 measurement sites.

Andrus and Stokoe (1997) proposed the following relationship between CRR and V_{s1}:

$$\mathrm{CRR} = a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V^*_{s1} - V_{s1}} - \frac{1}{V^*_{s1}}\right) \tag{22}$$

where V^*_{s1} = limiting upper value of V_{s1} for liquefaction occurrence; and a and b are curve fitting parameters. The first parenthetical term of (22) is based on a modified relationship between V_{s1} and CSR for constant average cyclic shear strain suggested by R. Dobry (personal communication to R. D. Andrus, 1996). The second parenthetical term is a hyperbola with a small value at low V_{s1}, and a very large value as V_{s1} approaches V^*_{s1}, a constant limiting velocity for liquefaction of soils.

CRR versus V_{s1} curves recommended for engineering practice by Andrus and Stokoe (2000) for magnitude 7.5 earthquakes and uncemented Holocene-age soils with various fines contents are reproduced in Fig. 9. Also plotted and presented in Fig. 9 are points calculated from liquefaction case history information for magnitude 5.9–8.3 earthquakes. The three curves shown were determined through an iterative process of varying the values of a and b until nearly all the points indicative of liquefaction were bounded by the curves with the least number of nonliquefaction points plotted in the liquefaction region. The final values of a and b used to draw the curves were 0.022 and 2.8, respectively. Values of V^*_{s1} were assumed to vary linearly from 200 m/s for soils with fines content of 35% to 215 m/s for soils with fines content of 5% or less.

The recommended curves shown in Fig. 9 are dashed above CRR of 0.35 to indicate that field-performance data are limited in that range. Also, they do not extend much below 100 m/s, because there are no field data to support extending them to the origin. The calculated CRR is 0.033 for a V_{s1} of 100 m/s. This minimal CRR value is generally consistent with intercept CRR values assumed for the CPT and SPT procedures. Eq. (22) can be scaled to other magnitude values through use of magnitude scaling factors. These factors are discussed in a later section of this paper.

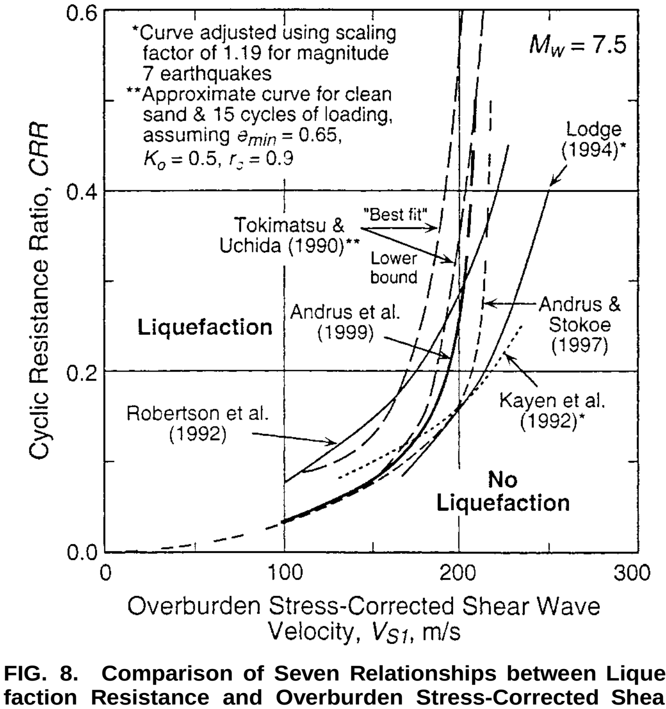


FIG. 8. Comparison of Seven Relationships between Liquefaction Resistance and Overburden Stress-Corrected Shear Wave Velocity for Granular Soils

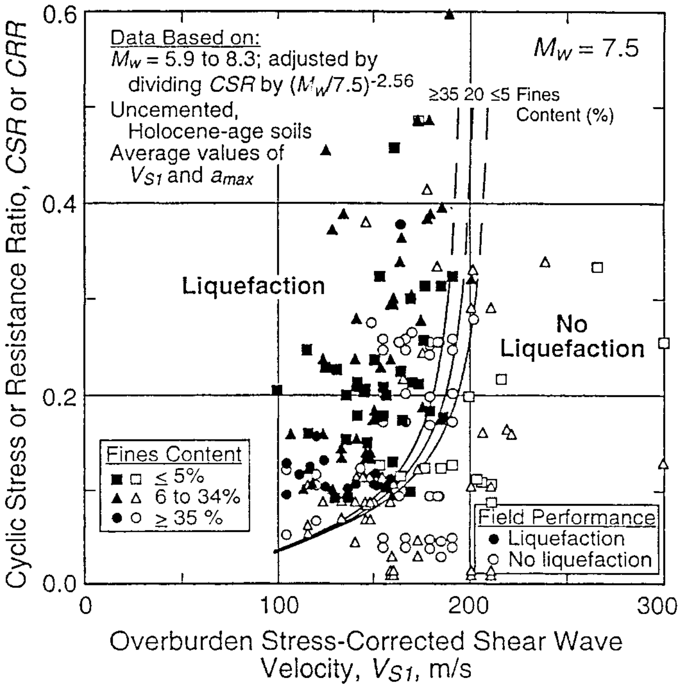


FIG. 9. Liquefaction Relationship Recommended for Clean, Uncemented Soils with Liquefaction Data from Compiled Case Histories (Reproduced from Andrus and Stokoe 2000)

BPT

Liquefaction resistance of nongravelly soils has been evaluated primarily through CPT and SPT, with occasional V_s measurements. CPT and SPT measurements, however, are not generally reliable in gravelly soils. Large gravel particles may interfere with the normal deformation of soil materials around the penetrometer and misleadingly increase penetration resistance. Several investigators have employed large-diameter penetrometers to surmount these difficulties; the Becker penetration test (BPT) in particular has become one of the more effectively and widely used larger tools. The BPT was developed in Canada in the late 1950s and consists of a 168-mm diameter, 3-m-long double-walled casing driven into the ground with a double-acting diesel-driven pile hammer. The hammer impacts are applied at the top of the casing and peneration is continuous. The Becker penetration resistance is defined as the number of blows required to drive the casing through an increment of 300 mm.

The BPT has not been standardized, and several different types of equipment and procedures have been used. There are currently very few liquefaction sites from which BPT data have been obtained. Thus the BPT cannot be directly correlated with field behavior, but rather through estimating equivalent SPT *N*-values from BPT data and then applying evaluation procedures based on the SPT. This indirect method introduces substantial additional uncertainty into the calculated CRR.

To provide uniformity, Harder and Seed (1986) recommended newer AP-1000 drill rigs equipped with supercharged diesel hammers, 168-mm outside diameter casing, and a plugged bit. From several sites where both BPT and SPT tests were conducted in parallel soundings, Harder and Seed (1986) developed a preliminary correlation between Becker and standard penetration resistance [Fig. 10(a)]. Additional comparative data compiled since 1986 are plotted in Fig. 10(b). The original Harder and Seed correlation curve (solid line) is drawn in Fig. 10(b) along with dashed curves representing 20% over- and underpredictions of SPT blow counts. These plots indicate that SPT blow counts can be roughly estimated from BPT measurements. These plots indicate that although SPT blow counts can be roughly estimated from BPT measurements, there can be considerable uncertainty for calculating liquefaction resistance because the data scatter is greatest in the range of greatest importance [*N*-values of 0–30 blows/300 mm (ft)].

A major source of variation in BPT blow counts is deviations in hammer energy. Rather than measuring hammer energy directly, Harder and Seed (1986) monitored bounce-chamber pressures and found that uniform combustion conditions (e.g., full throttle with a supercharger) correlated rather well with variations in Becker blow count. From this information, Harder and Seed developed an energy correction procedure based on measured bounce-chamber pressure.

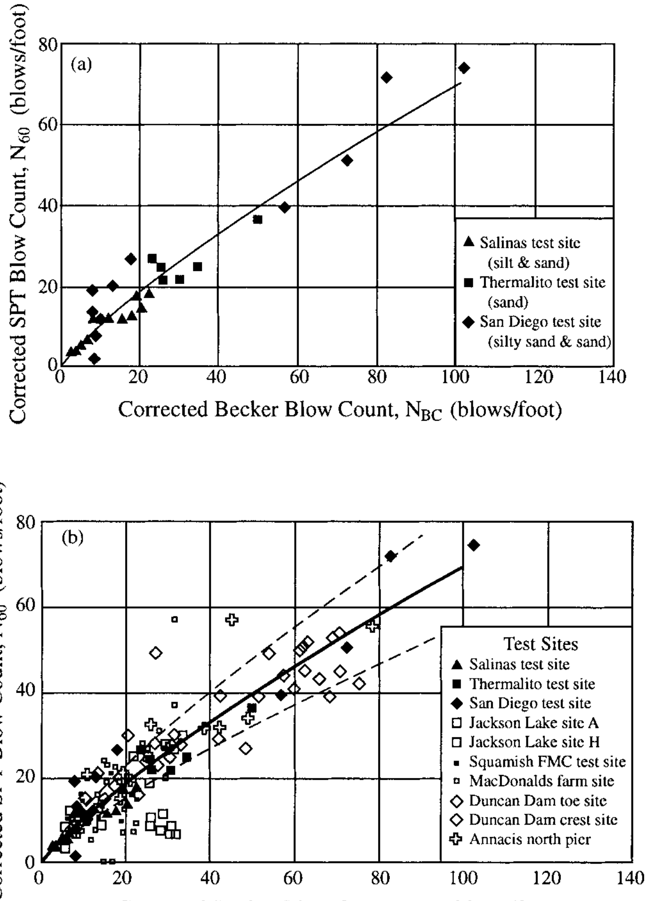


FIG. 10. Correlation between Corrected Becker Penetration Resistance N_{BC} and Corrected SPT Resistance N_{60}: (a) Harder and Seed (1986); (b) Data from Additional Sites (Reproduced from Harder 1997)

Direct measurement of transmitted hammer energy could provide a more theoretically rigorous correction for Becker hammer efficiency. Sy and Campanella (1994) and Sy et al. (1995) instrumented a small length of Becker casing with strain gauges and accelerometers to measure transferred energy. They analyzed the recorded data with a pile-driving analyzer to determine strain, force, acceleration, and velocity. The transferred energy was determined by time integration of force times velocity. They were able to verify many of the variations in hammer energy previously identified by Harder and Seed (1986), including effects of variable throttle settings and energy transmission efficiencies of various drill rigs. However, they were unable to reduce the amount of scatter and uncertainty in converting BPT blow counts to SPT blow counts. Because the Sy and Campanella procedure requires considerably more effort than monitoring of bounce-chamber pressure without producing greatly improved results, the workshop participants agreed that the bounce-chamber technique is adequate for routine practice.

Friction along the driven casing also influences penetration resistance. Harder and Seed (1986) did not directly evaluate the effect of casing friction; hence, the correlation in Fig. 10(b) intrinsically incorporates an unknown amount of casing friction. However, casing friction remains a concern for depths >30 m and for measurement of penetration resistance in soft soils underlying thick deposits of dense soil. Either of these circumstances could lead to greater casing friction than is intrinsically incorporated in the Harder and Seed correlation.

The following procedures are recommended for routine practice: (1) the BPT should be conducted with newer AP-1000 drill rigs equipped with supercharged diesel hammers to drive plugged 168-mm outside diameter casing; (2) bounce-chamber pressures should be monitored and adjustments made to measured BPT blow counts to account for variations in diesel hammer combustion efficiency—for most routine applications, correlations developed by Harder and Seed (1986) may be used for these adjustments; and (3) the influence of some casing friction is indirectly accounted for in the Harder and Seed BPT-SPT correlation. This correlation, however, has not been verified and should not be used for depths >30 m or for sites with thick dense deposits overlying loose sands or gravels. For these conditions, mudded boreholes may be needed to reduce casing friction, or specially developed local correlations or sophisticated wave-equation analyses may be applied to quantify frictional effects.

MAGNITUDE SCALING FACTORS (MSFs)

The clean-sand base or CRR curves in Figs. 2 (SPT), 4 (CPT), and 10 (V_{s1}) apply only to magnitude 7.5 earthquakes. To adjust the clean-sand curves to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed "magnitude scaling factors (MSFs)." These factors are used to scale the CRR base curves upward or downward on CRR versus $(N_1)_{60}$, q_{c1N}, or V_{s1} plots. Conversely, magnitude weighting factors, which are the inverse of magnitude scaling factors, may be applied to correct CSR for magnitude. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. Because the original papers by Seed and Idriss were written in terms of magnitude scaling factors, the use of magnitude scaling factors is continued in this report.

To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (FS) against liquefaction is written in terms of CRR, CSR, and MSF as follows:

$$FS = (CRR_{7.5}/CSR)MSF \tag{23}$$

where CSR = calculated cyclic stress ratio generated by the earthquake shaking; and $CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquakes. $CRR_{7.5}$ is determined from Fig. 2 or (4) for SPT data, Fig. 4 or (11) for CPT data, or Fig. 9 or (22) for V_{s1} data.

Seed and Idriss (1982) Scaling Factors

Because of the limited amount of field liquefaction data available in the 1970s, Seed and Idriss (1982) were unable to adequately constrain bounds between liquefaction and nonliquefaction regions on CRR plots for magnitudes other than 7.5. Consequently, they developed a set of MSF from average numbers of loading cycles for various earthquake magnitudes and laboratory test results. A representative curve developed by these investigators, showing the number of loading cycles required to generate liquefaction for a given CSR, is reproduced in Fig. 11. The average number of loading cycles for various magnitudes of earthquakes are also noted on the plot. The initial set of magnitude scaling factors was derived by dividing CSR values on the representative curve for the number of loading cycles corresponding to a given earthquake magnitude by the CSR for 15 loading cycles (equivalent to a magnitude 7.5 earthquake). These scaling factors are listed in column 2 of Table 3 and are plotted in Fig. 12. These MSFs have been routinely applied in engineering practice since their introduction in 1982.

Revised Idriss Scaling Factors

In preparing his H. B. Seed Memorial Lecture, I. M. Idriss reevaluated the data that he and the late Professor Seed used to calculate the original (1982) magnitude scaling factors. In so doing, Idriss replotted the data on a log-log plot and suggested that the data should plot as a straight line. He noted, however, that one outlying point had strongly influenced the original analysis, causing the original plot to be nonlinear and characterized by unduly low MSF values for magnitudes <7.5. Based on this reevaluation, Idriss defined a revised set of magnitude scaling factors listed in column 3 of Table 3 and plotted in Fig. 12. The revised MSFs are defined by the following equation:

$$MSF = 10^{2.24}/M_w^{2.56} \tag{24}$$

The workshop participants recommend these revised scaling factors as a lower bound for MSF values.

The revised scaling factors are significantly higher than the original scaling factors for magnitudes <7.5 and somewhat lower than the original factors for magnitudes >7.5. Relative to the original scaling factors, the revised factors lead to a reduced calculated liquefaction hazard for magnitudes <7.5, but increase calculated hazard for magnitudes >7.5.

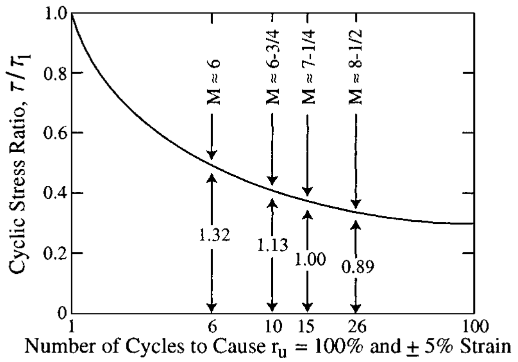


FIG. 11. Representative Relationship between CSR and Number of Cycles to Cause Liquefaction (Reproduced from Seed and Idriss 1982)

Ambraseys (1988) Scaling Factors

Field performance data collected since the 1970s for magnitudes <7.5 indicate that the original Seed and Idriss (1982) scaling factors are overly conservative. For example, Ambraseys (1988) analyzed liquefaction data compiled through the mid-1980s and plotted calculated cyclic stress ratios for sites that did or did not liquefy versus $(N_1)_{60}$. From these plots, Ambraseys developed empirical exponential equations that define CRR as a function of $(N_1)_{60}$ and moment magnitude M_w. By holding the value of $(N_1)_{60}$ constant in the equations and taking the ratio of CRR determined for various magnitudes of earthquakes to the CRR for magnitude 7.5 earthquakes, Am-

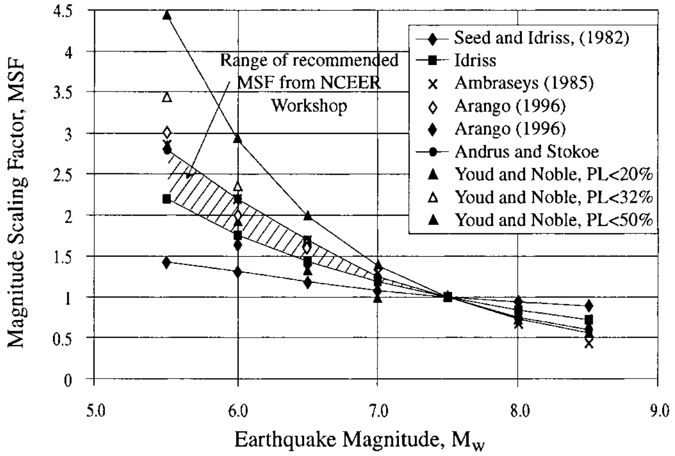


FIG. 12. Magnitude Scaling Factors Derived by Various Investigators (Reproduced from Youd and Noble 1997a)

TABLE 3. Magnitude Scaling Factor Values Defined by Various Investigators (Youd and Noble 1997a)

Magnitude, M (1)	Seed and Idriss (1982) (2)	Idriss[a] (3)	Ambraseys (1988) (4)	Arango (1996)		Andrus and Stokoe (1997) (7)	Youd and Noble (1997b)		
				Distance based (5)	Energy based (6)		$P_L < 20\%$ (8)	$P_L < 32\%$ (9)	$P_L < 50\%$ (10)
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.6	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00	—	—	1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.8?	—	—	0.73?
8.5	0.89	0.72	0.44	—	—	0.65?	—	—	0.56?

Note: ? = Very uncertain values.

[a] 1995 Seed Memorial Lecture, University of California at Berkeley (I. M. Idriss, personal communication to T. L. Youd, 1997).

braseys derived the magnitude scaling factors listed in column 4 of Table 3 and plotted in Fig. 12. For magnitudes <7.5, the MSFs suggested by Ambraseys are significantly larger than both the original factors developed by Seed and Idriss (column 2, Table 3) and the revised factors suggested by Idriss (column 3). Because they are based on observational data, these factors have validity for estimating liquefaction hazard; however, they have not been widely used in engineering practice.

For magnitudes >7.5, Ambraseys factors are significantly lower and much more conservative than the original (Seed and Idriss 1982) and Idriss's revised scaling factors. Because there are few data to constrain Ambraseys' scaling factors for magnitudes >7.5, they are not recommended for hazard evaluation for large earthquakes.

Arango (1996) Scaling Factors

Arango (1996) developed two sets of magnitude scaling factors. The first set (column 5, Table 3) is based on furthest observed liquefaction effects from the seismic energy source, the estimated average peak accelerations at those distant sites, and the seismic energy required to cause liquefaction. The second set (column 6, Table 3) was developed from energy concepts and the relationship derived by Seed and Idriss (1982) between numbers of significant stress cycles and earthquake magnitude. The MSFs listed in column 5 are similar in value (within about 10%) to the MSFs of Ambraseys (column 4), and the MSFs listed in column 6 are similar in value (within about 10%) to the revised MSFs proposed by Idriss (column 3).

Andrus and Stokoe (1997) Scaling Factors

From their studies of liquefaction resistance as a function of shear wave velocity V_s Andrus and Stokoe (1997) drew bounding curves and developed (22) for calculating CRR from V_s for magnitude 7.5 earthquakes. These investigators drew similar bounding curves for sites where surface effects of liquefaction were or were not observed for earthquakes with magnitudes of 6, 6.5, and 7. The positions of the CRR curves were visually adjusted on each graph until a best-fit bound was obtained. Magnitude scaling factors were then estimated by taking the ratio of CRR for a given magnitude to the CRR for magnitude 7.5 earthquakes. These MSFs are quantified by the following equation:

$$\text{MSF} = (M_w/7.5)^{-2.56} \tag{25}$$

MSFs for magnitudes <6 and >7.5 were extrapolated from this equation. The derived MSFs are listed in column 7 of Table 3, and plotted in Fig. 12. For magnitudes <7.5, the MSFs proposed by Andrus and Stokoe are rather close in value (within about 5%) to the MSFs proposed by Ambraseys. For magnitudes >7.5, the Andrus and Stokoe MSFs are slightly smaller than the revised MSFs proposed by Idriss.

Youd and Noble (1997a) Scaling Factors

Youd and Noble (1997a) used a probabilistic or logistic analysis to analyze case history data from sites where effects of liquefaction were or were not reported following past earthquakes. This analysis yielded the following equation, which was updated after publication of the NCEER proceedings (Youd and Idriss 1997):

$$\text{Logit}(P_L) = \ln(P_L/(1 - P_L)) = -7.0351 + 2.1738M_w - 0.2678(N_1)_{60cs} + 3.0265 \ln \text{CRR} \tag{26}$$

where P_L = probability that liquefaction occurred; $1 - P_L$ = probability that liquefaction did not occur; and $(N_1)_{60cs}$ = corrected equivalent clean-sand blow count. For magnitudes <7.5, Youd and Noble recommended direct application of this equation to calculate the CRR for a given probability of liquefaction. In lieu of direct application, Youd and Noble defined three sets of MSFs for use with the simplified procedure. These MSFs are for probabilities of liquefaction occurrence <20, 32, and 50%, respectively, and are defined by the following equations:

$$\text{Probability } P_L < 20\% \quad \text{MSF} = 10^{3.81}/M^{4.53} \text{ for } M_w < 7 \tag{27}$$

$$\text{Probability } P_L < 32\% \quad \text{MSF} = 10^{3.74}/M^{4.33} \text{ for } M_w < 7 \tag{28}$$

$$\text{Probability } P_L < 50\% \quad \text{MSF} = 10^{4.21}/M^{4.81} \text{ for } M_w < 7.75 \tag{29}$$

New Recommendation by Idriss

I. M. Idriss (TRB 1999) proposed a new set of MSFs that are compatible with, and are only to be used with, the magnitude-dependent r_d that he also proposed. These new MSFs have lower values than the revised MSFs listed in Table 3, but slightly higher values than the original Seed and Idriss (1982) MSFs. Because the proposed r_d and associated MSFs have not been published and the factors have not been independently verified, the workshop participants chose not to recommend the new r_d or MSFs at this time.

Recommendations for Engineering Practice

The workshop participants reviewed the MSFs listed in Table 3, and all but one (S. S. C. Liao) agree that the original factors were too conservative and that increased MSFs are warranted for engineering practice for magnitudes <7.5. Rather than recommending a single set of factors, the workshop participants suggest a range of MSFs from which the engineer is allowed to choose factors that are requisite with the acceptable risk for any given application. For magnitudes <7.5, the lower bound for the recommended range is the new MSF proposed by Idriss [column 3 in Table 3, or (23)]. The suggested upper bound is the MSF proposed by Andrus and Stokoe [column 7 in Table 3, or (26)]. The upper-bound values are consistent with MSFs suggested by Ambraseys (1988), Arango (1996), and Youd and Noble (1997a) for $P_L < 20\%$.

For magnitudes >7.5, the new factors recommended by Idriss [column 3 in Table 3; (25)] should be used for engineering practice. These new factors are smaller than the original Seed and Idriss (1982) factors, hence their application leads to increased calculated liquefaction hazard compared to the original factors. Because there are only a few well-documented liquefaction case histories for earthquakes with magnitudes >8, MSFs in that range are poorly constrained by field data. Thus the workshop participants agreed that the greater conservatism embodied in the revised MSF by Idriss (column 3, Table 3) should be recommended for engineering practice.

CORRECTIONS FOR HIGH OVERBURDEN STRESSES, STATIC SHEAR STRESSES, AND AGE OF DEPOSIT

Correction factors K_σ and K_α were developed by Seed (1983) to extrapolate the simplified procedure to larger overburden pressure and static shear stress conditions than those embodied in the case history data set from which the simplified procedure was derived. As noted previously, the simplified procedure was developed and validated only for level to gently sloping sites (low static shear stress) and depths less than about 15 m (low overburden pressures). Thus applications using K_σ and K_α are beyond routine practice and require specialized expertise. Because these factors were discussed at the workshop and some new information was developed, recommendations from those discussions are included here. These recommendations, however, apply mostly to liquefaction hazard analyses of embankment dams and other large structures. These factors are applied by extending (23) to include K_σ and K_α as follows:

$$\mathrm{FS} = (\mathrm{CRR}_{7.5}/\mathrm{CSR})\cdot \mathrm{MSF}\cdot K_\sigma \cdot K_\alpha \tag{30}$$

K_σ Correction Factor

Cyclically loaded laboratory test data indicate that liquefaction resistance increases with increasing confining stress. The rate of increase, however, is nonlinear. To account for the nonlinearity between CRR and effective overburden pressure, Seed (1983) introduced the correction factor K_σ to extrapolate the simplified procedure to soil layers with overburden pressures >100 kPa. Cyclically loaded, isotropically consolidated triaxial compression tests on sand specimens were used to measure CRR for high-stress conditions and develop K_σ values. By taking the ratio of CRR for various confining pressures to the CRR determined for approximately 100 kPa (1 atm) Seed (1983) developed the original K_σ correction curve. Other investigators have added data and suggested modifications to better define K_σ for engineering practice. For example, Seed and Harder (1990) developed the clean-sand curve reproduced in Fig. 13. Hynes and Olsen (1999) compiled and analyzed an enlarged data set to provide guidance and formulate equations for selecting K_σ values (Fig. 14). The equation they derived for calculating K_σ is

$$K_\sigma = (\sigma'_{vo}/P_a)^{(f-1)} \tag{31}$$

where σ'_{vo}, effective overburden pressure; and P_a, atmospheric pressure, are measured in the same units; and f is an exponent that is a function of site conditions, including relative density, stress history, aging, and overconsolidation ratio. The workshop participants considered the work of previous investigators and recommend the following values for f (Fig. 15). For relative densities between 40 and 60%, $f = 0.7–0.8$; for relative densities between 60 and 80%, $f = 0.6–0.7$. Hynes and Olsen recommended these values as minimal or conservative estimates of K_σ for use in engineering practice for both clean and silty sands, and for gravels. The workshop participants concurred with this recommendation.

K_α Correction Factor for Sloping Ground

The liquefaction resistance of dilative soils (moderately dense to dense granular materials under low confining stress) increases with increased static shear stress. Conversely, the liquefaction resistance of contractive soils (loose soils and moderately dense soils under high confining stress) decreases with increased static shear stresses. To incorporate the effect of static shear stresses on liquefaction resistance, Seed (1983) introduced a correction factor K_α. To generate values for this factor, Seed normalized the static shear stress τ_{st} acting on a plane with respect to the effective vertical stress σ'_{vo} yielding a parameter α, where

$$\alpha = \tau_{st}/\sigma'_{vo} \tag{32}$$

Cyclically loaded triaxial compression tests were then used to empirically determine values of the correction factor K_α as a function of α.

For the NCEER workshop, Harder and Boulanger (1997) reviewed past publications, test results, and analyses of K_α. They noted that a wide range of K_α values have been proposed, indicating a lack of convergence and a need for continued research. The workshop participants agreed with this assessment. Although curves relating K_α to α have been published (Harder and Boulanger 1997), these curves should not be used

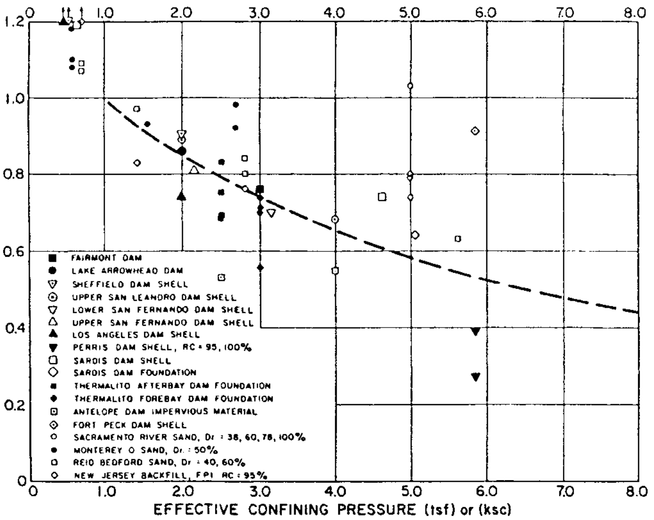


FIG. 13. K_σ**-Values Determined by Various Investigators (Reproduced from Seed and Harder 1990)**

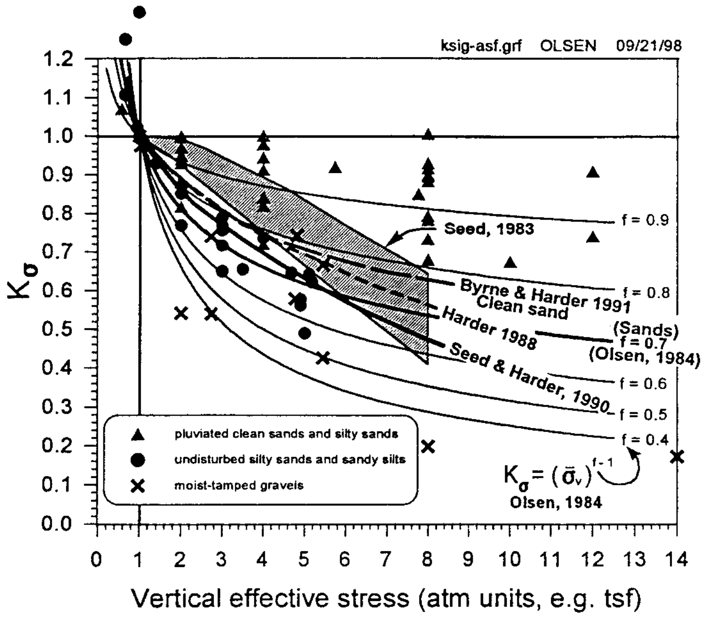


FIG. 14. Laboratory Data and Compiled K_σ **Curves (Reproduced from Hynes and Olsen 1999)**

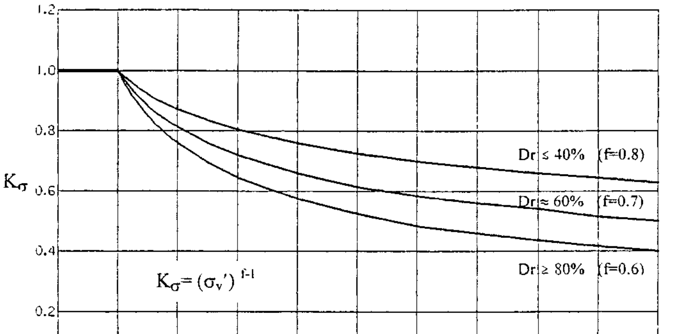


FIG. 15. Recommended Curves for Estimating K_σ **for Engineering Practice**

by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.

Influence of Age of Deposit

Several investigators have noted that liquefaction resistance of soils increases with age. For example, Seed (1979) observed significant increases in liquefaction resistance with aging of reconstituted sand specimens tested in the laboratory. Increases of as much as 25% in cyclic resistance ratio were noted between freshly constituted and 100-day-old specimens. Youd and Hoose (1977) and Youd and Perkins (1978) noted that liquefaction resistance increases markedly with geologic age. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and pre-Pleistocene sediments are generally immune to liquefaction. Although qualitative time-dependent increases have been documented as noted above, few quantitative data have been collected. In addition, the factors causing increased liquefaction resistance with age are poorly understood. Consequently, verified correction factors for age have not been developed.

In the absence of quantitative correction factors, engineering judgment is required to estimate the liquefaction resistance of sediments more than a few thousand years old. For deeply buried sediments dated as more than a few thousand years old, some knowledgeable engineers have omitted application of the K_σ factor as partial compensation for the unquantified, but substantial increase of liquefaction resistance with age. For man-made structures, such as thick fills and embankment dams, aging effects are minimal, and corrections for age should not be applied in calculating liquefaction resistance.

SEISMIC FACTORS

Application of the simplified procedure for evaluating liquefaction resistance requires estimates of two ground motion parameters—earthquake magnitude and peak horizontal ground acceleration. These factors characterize duration and intensity of ground shaking, respectively. The workshop addressed the following questions with respect to selection of magnitude and peak acceleration values for liquefaction resistance analyses.

Earthquake Magnitude

Records from recent earthquakes, such as 1979 Imperial Valley, 1988 Armenia, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe, indicate that the relationship between duration and magnitude is rather uncertain and that factors other than magnitude also influence duration. For example, unilateral faulting, in which rupture begins at one end of the fault and propagates to the other, usually produces longer shaking duration for a given magnitude than bilateral funding, in which slip begins near the midpoint on the fault and propagates in both directions simultaneously. Duration also generally increases with distance from the seismic energy source and may vary with tectonic province, site conditions, and bedrock topography (basin effects).

Question: Should correction factors be developed to adjust duration of shaking to account for the influence of earthquake source mechanism, fault rupture mode, distance from the energy source, basin effects, etc.?

Answer: Faulting characteristics and variations in shaking duration are difficult to predict in advance of an earthquake event. The influence of distance generally is of secondary importance within the range of distances to which damaging liquefaction effects commonly develop. Basin effects are not yet sufficiently predictable to be adequately accounted for in engineering practice. Thus the workshop participants recommend continued use of the generally conservative relationship between magnitude and duration that is embodied in the simplified procedure.

Question: An important difference between eastern U.S. earthquakes and western U.S. earthquakes is that eastern ground motions are generally richer in high-frequency energy and thus could generate more significant stress cycles and equivalently longer durations than western earthquakes of the same magnitude. Is a correction needed to account for higher frequencies of motions generated by eastern U.S. earthquakes?

Answer: The high-frequency motions of eastern earthquakes are generally limited to near-field rock sites. High-frequency motions attenuate or are damped out rather quickly as they propagate through soil layers. This filtering action reduces the high-frequency energy at soil sites and thus reduces differences in numbers of significant loading cycles. Because liquefaction occurs only within soil strata, duration differences on soil sites between eastern and western earthquakes are not likely to be great. Without more instrumentally recorded data from which differences in ground motion characteristics can be quantified, there is little basis for the development of additional correction factors for eastern localities.

Another difference between eastern and western U.S. earthquakes is that strong ground motions generally propagate to greater distances in the east than in the west. By applying present state-of-the-art procedures for estimating peak ground acceleration at eastern sites, differences in amplitudes of ground motions between western and eastern earthquakes are properly taken into account.

Question: Which magnitude scale should be used for selection of earthquake magnitudes for liquefaction resistance analyses?

Answer: Seismologists commonly calculate earthquake magnitudes using five different scales: (1) local or Richter magnitude M_L; (2) surface-wave magnitude M_s; (3) short-period body-wave magnitude m_b; (4) long-period body-wave magnitude m_B; and (5) moment magnitude M_w. Moment magnitude, the scale most commonly used for engineering applications, is the scale preferred for calculation of liquefaction

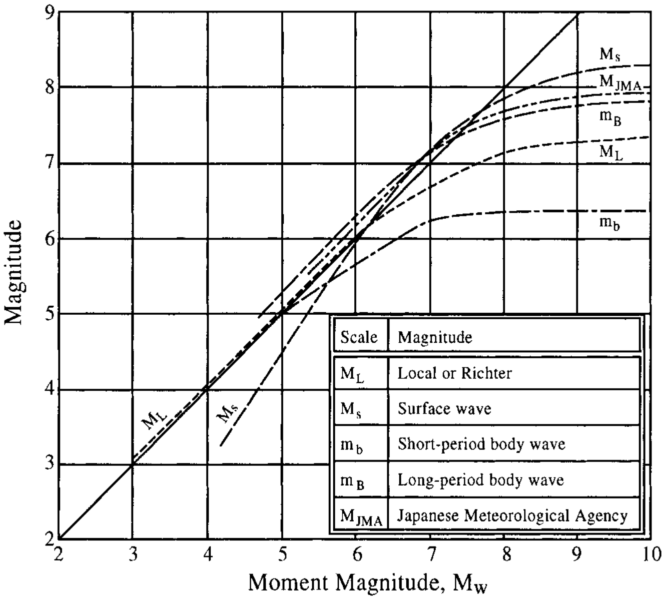


FIG. 16. Relationship between Moment M_w and Other Magnitude Scales (Reproduced from Heaton et al. Unpublished Report, 1982)

resistance. As Fig. 16 shows, magnitudes from other scales may be substituted directly for M_w within the following limitations—$M_L < 6$, $m_B < 7.5$, and $6 < M_s < 8$—m_b, a scale commonly used for eastern U.S. earthquakes, may be used for magnitudes between 5 and 6, provided m_b values are corrected to equivalent M_w values. The curves plotted in Fig. 16 may be used for this adjustment (Idriss 1985).

Peak Acceleration

In the simplified procedure, peak horizontal acceleration $a_{\max}$ is used to characterize the intensity of ground shaking. To provide guidance for estimation of $a_{\max}$, the workshop addressed the following questions.

Question: What procedures are preferred for estimating $a_{\max}$ at potentially liquefiable sites?

Answer: The following methods, in order of preference, may be used for estimating $a_{\max}$:

1) The preferred method for estimating $a_{\max}$ is through empirical correlations of $a_{\max}$ with earthquake magnitude, distance from the seismic energy source, and local site conditions. Several correlations have been published for estimating $a_{\max}$ for sites on bedrock or stiff to moderately stiff soils. Preliminary attenuation relationships have also been developed for a limited range of soft soil sites (Idriss 1991). Selection of an attenuation relationship should be based on such factors as region of the country, type of faulting, and site condition.

2) For soft sites and other soil profiles that are not compatible with available attenuation relationships, $a_{\max}$ may be estimated from local site response analyses. Computer programs such as SHAKE and DESRA may be used for these calculations (Schnabel et al. 1972; Finn et al. 1977). Input ground motions in the form of recorded accelerograms are preferable to synthetic records. Accelerograms derived from white noise should be avoided. A suite of plausible earthquake records should be used in the analysis, including as many as feasible from earthquakes with similar magnitudes, source distances, etc.

3) The third and least desirable method for estimating peak ground acceleration is through amplification ratios, such as those developed by Idriss (1990, 1991) and Seed et al. (1994). These factors use a multiplier or ratio by which bedrock outcrop motions are amplified to estimate surface motions at soil sites. Because amplification ratios are influenced by strain level, earthquake magnitude, and frequency content, caution and considerable engineering judgment are required in the application of these relationships.

Question: Which peak acceleration should be used: (1) the largest horizontal acceleration recorded on a three-component accelerogram; (2) the geometric mean (square root of the product) of the two maximum horizontal components; or (3) a vectorial combination of horizontal accelerations?

Answer: According to I. M. Idriss (oral discussion at NCEER workshop, 1996), where recorded motions were available, the larger of the two horizontal peak components of acceleration was used in the compilation of data used to derive the original simplified procedure. Where recorded values were not available, which was the circumstance for most sites, peak acceleration values were estimated from attenuation relationships based on the geometric mean of the two orthogonal peak horizontal accelerations. In nearly all instances where recorded motions were used, the peaks from the two horizontal records were approximately equal. Thus where a single peak was used, the peak and the geometric mean of the two peaks were about the same value. Based on this information, the workshop participants concurred that use of the geometric mean is consistent with the development of the procedure and is preferred for use in engineering practice. However, use of the larger of the two orthogonal peak accelerations yields a larger estimate of $a_{\max}$, is conservative, and is allowable. Vectorial accelerations are seldom calculated and should not be used. Peak vertical accelerations are generally much smaller than peak horizontal accelerations and are ignored for calculation of liquefaction resistance.

Question: Liquefaction usually develops at soil sites where ground motion amplification may occur and where sediment may soften, reducing motions as excess pore pressure develop. How should investigators account for these factors in estimating peak acceleration?

Answer: The recommended procedure is to calculate or estimate the $a_{\max}$ that would occur at the site in the absence of increased pore pressure or the onset of liquefaction. That peak acceleration incorporates the influence of site amplification, but neglects the influence of excess pore-water pressure.

Question: Should high-frequency spikes (periods <0.1 s) in acceleration records be considered or ignored?

Answer: In general, short-duration, high-frequency acceleration spikes are too short in duration to generate significant instability or deformation of granular structures, and should be ignored. By using attenuation relationships for estimation of peak acceleration, as noted above, high-frequency spikes are essentially ignored because few high-frequency peaks are incorporated in databases from which attenuation the relationships were derived. Similarly, ground response analyses programs such as SHAKE and DESRA generally attenuate or filter out high-frequency spikes, reducing their influence. Where amplification ratios are used, engineering judgment should be used to determine which bedrock acceleration is to be amplified.

ENERGY-BASED CRITERIA AND PROBABILISTIC ANALYSES

The workshop considered two additional topics: (1) liquefaction resistance criteria based on seismic energy passing through a liquefiable layer (Kayen and Mitchell 1997; Youd et al. 1997), and probabilistic analyses of case history data (Liao et al. 1988; Youd and Noble 1997b). Although probabilistic or risk analyses have been made for some localities and critical facilities, the workshop participants concluded that probabilistic procedures are still under development and not sufficiently formulated for routine engineering practice. Similarly, new energy-based criteria need to be independently tested before recommendations can be made for general practice. The workshop participants recommend that research and development continue on both of these relatively new and potentially useful procedures.

CONCLUSIONS

The participants in the NCEER workshop reviewed the state-of-the-art for evaluating liquefaction resistance and recommend several augmentations to that procedure. Specific recommendations, including procedures and equations, are listed in each section of this summary paper. Consensus conclusions from the workshop are:

1. Four field tests are recommended for routine evaluation of liquefaction resistance—the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity (V_s) measurements, and for gravelly sites the Becker penetration test (BPT). Criteria for each test were reviewed and revised to incorporate recent developments and to achieve consistency between resistances calculated from the various tests. Each test has its advantages and limitations (Table 1). the CPT provides the most detailed soil stratigraphy and robust field-data based liquefaction resistance curves now available. CPT testing

should always be accompanied by soil sampling for validation of soil type identification. The SPT has a longer record of application and provides disturbed soil samples from which fines content and other grain characteristics can be determined. Measured shear-wave velocities provide fundamental information on small-strain soil behavior that is useful beyond analyses of liquefaction resistance. V_s is also applicable at sites, such as landfills and gravelly sediments, where CPT and SPT soundings may not be possible or reliable. The BPT test is recommended only for gravelly sites and requires use of rough correlations between BPT and SPT, making the results less certain than other tests. Where possible, two or more test procedures should be applied to assure adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance.

2. The magnitude scaling factors originally derived by Seed and Idriss (1982) are overly conservative for earthquakes with magnitudes <7.5. A range of scaling factors is recommended for engineering practice, the lower end of the range being the new MSF recommended by Idriss (column 3, Table 3), and the upper end of the range being the MSF suggested by Andrus and Stokoe (column 7, Table 3). These MSFs are defined by (25) and (26), respectively. For magnitudes >7.5, the new factors by Idriss (column 3, Table 3) should be used. These factors, which are more conservative than the original Seed and Idriss (1982) factors, should be applied.
3. The K_σ factors suggested by Seed and Harder (1990) appear to be overly conservative for some soils and field conditions. The workshop participants recommend K_σ values defined by the curves in Fig. 14 or (31). Because K_σ values are usually applied to depths greater than those verified for the simplified procedure, special expertise is generally required for their application.
4. Procedures for evaluation of liquefaction resistance beneath sloping ground or embankments (slopes greater than about 6%) have not been developed to a level allowable for routine use. Special expertise is required for evaluation of liquefaction resistance beneath sloping ground.
5. Moment magnitude M_w should be used for liquefaction resistance calculations. Magnitude, as used in the simplified procedure, is a measure of the duration of strong ground shaking. The present magnitude criteria are conservative and should not be corrected for source mechanism, style of faulting, distance from the energy source, subsurface bedrock topography (basin effect), or tectonic region (eastern versus western U.S. earthquakes).
6. The peak acceleration $a_{\max}$ applied in the procedure is the peak horizontal acceleration that would occur at ground surface in the absence of pore pressure increases or liquefaction. Attenuation relationships compatible with soil conditions at a site should be applied in estimating $a_{\max}$. Relationships based on the geometric mean of the peak horizontal accelerations are preferred, but use of relationships based on peak horizontal acceleration is allowable and conservative. Where site conditions are incompatible with existing attenuation relationships, site-specific response calculations, using programs such as SHAKE or DESRA, should be used. The least preferable technique is application of amplification factors.

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WORKSHOP PARTICIPANTS

Chair: T. Leslie Youd, Brigham Young Univ., Provo, Utah; **Co-chair:** Izzat M. Idriss, Univ. of California at Davis, Davis, Calif.; Ronald D. Andrus, Clemson Univ. (formerly, National Institute of Standards and Technology, Gaithersburg, Md.; Ignacio Arango, Bechtel Corp., Oakland, Calif.; Gonzalo Castro, GEI Consultants, Inc., Winchester, Mass.; John T. Christian, Engineering Consultant, Boston; Richardo Dobry, Rensselaer Polytechnic Institute, Troy, N.Y.; W. D. Liam Finn, Univ. of British Columbia, Vancouver; Leslie F. Harder, Jr., California Department of Water Resources, Sacramento, Calif.; Mary Ellen Hynes, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.; Kenji Ishihara, Science Univ. of Tokyo, Tokyo; Joseph P. Koester, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.; Sam S. C. Liao, Parsons Brinckerhoff, Boston; William F. Marcuson III, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.; Geoffrey R. Martin, Univ. of Southern California, Los Angeles; James K. Mitchell, Virginia Tech, Blacksburg, Va.; Yoshiharu Moriwaki, Woodward-Clyde Consultants, Santa Ana, Calif.; Maurice S. Power, Geomatrix Consultants, San Francisco; Peter K. Robertson, Univ. of Alberta, Edmonton, Alberta; Raymond B. Seed, Univ. of California, Berkeley, Calif.; and Kenneth H. Stokoe II, Univ. of Texas, Austin, Tex.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

a, b = curve fitting parameters for use with V_s criteria for evaluating liquefaction resistance;
$a_{\max}$ = peak horizontal acceleration at ground surface;
C_B = correction factor for borehole diameter;
C_E = correction factor for hammer energy;
C_N = correction factor for overburden pressure applied to SPT;
C_Q = correction factor for overburden pressure applied to CPT;
C_R = correction factor for drilling rod length;
C_S = correction factor for split spoon sampler without liners;
$\mathrm{CRR}_{7.5}$ = cyclic resistance ratio for $M_w = 7.5$ earthquakes;
d_c = diameter of CPT tip;
F = normalized friction ratio;
f = exponent estimated from site conditions used in calculation of K_σ;
f_s = sleeve friction measured with CPT;
g = acceleration of gravity;
H = thickness of thin granular layer between softer sediment layers;
I_c = soil behavior type index for use with CPT liquefaction criteria;

K_c = correction factor for grain characteristics applied to CPT;
K_H = thin-layer correction factor for use with CPT;
K_α = correction factor for soil layers subjected to large static shear stresses;
K_σ = correction factor for soil layers subjected to large static normal stresses;
M_L = local or Richter magnitude of earthquake;
M_s = surface-wave magnitude of earthquake;
M_w = moment magnitude of earthquake;
m_B = long period body-wave magnitude of earthquake;
m_b = short period body-wave magnitude of earthquake;
N_m = measured standard penetration resistance;
$(N_1)_{60}$ = corrected standard penetration resistance;
$(N_1)_{60cs}$ = $(N_1)_{60}$ adjusted to equivalent clean-sand value;
n = exponent used in normalizing CPT resistance for overburden stress;
P_a = atmospheric pressure, approximately 100 kPa;
P_L = probability of liquefaction;
Q = normalized and dimensionless cone penetration resistance;
q_{c1N} = normalized cone penetration resistance;
$(q_{c1N})_{cs}$ = normalized cone penetration resistance adjusted to equivalent clean-sand value;
r_d = stress reduction coefficient to account for flexibility in soil profile;
V_s = measured shear-wave velocity;
V_{s1} = overburden-stress corrected shear-wave velocity;
V_{s1}^* = limiting upper value of V_{s1} for liquefaction occurrences;
z = depth below ground surface (m);
α, β = coefficients, that are functions of fines content, used to correct $(N_1)_{60}$ to $(N_1)_{60cs}$;
σ'_{vo} = effective overburden pressure;
τ_{av} = average horizontal shear stress acting on soil layer during shaking generated by given earthquake; and
τ_{st} = static shear stress acting on soil element due to gravitational forces.