FINAL REPORT

Geotechnical Investigation

Queensway Carleton Hospital Expansion

Submitted to:

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CA0033714.1722-Rev1

April 10, 2025

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1.0 INTRODUCTION

1.1 Context

This report presents the results of a geotechnical investigation carried out in support of a planned multi-building expansion of the Queensway Carleton Hospital (QCH) campus, in Ottawa Ontario. The project includes the design and construction of several new building elements directly attached or adjacent to existing hospital structures, a free-standing parking garage, and access road upgrades. The footprint areas of the planned improvements are presented on the Site Plan included as Figure 1 of this report.

Based on WSP's interpretation of the information obtained during the field investigations, a general description of the subsurface soil, bedrock and groundwater conditions is presented in this report. These conditions and available project details were then used to prepare geotechnical considerations and recommendations, including design and construction considerations for new building foundations. In preparing our fieldwork, testing plans and report outline for this assignment, we consulted historical reports and data, including WSP (formerly Golder) geotechnical reports and construction inspection records for existing onsite structures, and also government published mapping and information for the site area and region.

1.2 Limitations and Disclaimer

This report has been prepared at the request and for the sole use of QCH, according to the specific terms of the mandate given to WSP and described in our proposal dated April 10, 2024. The use of this report by any third party, as well as any decision based upon this report, is under that party's sole responsibility. WSP shall not be held financially or legally accountable for any possible claims or damages resulting from third-party decisions based on this report.

Furthermore, any opinions regarding conformity with national or local laws and regulations expressed in this report are only technical in nature; the report is not and shall not, in any case, be considered a legal opinion on any aspect of the site or project. Information in this report is only valid for the borehole locations as described, and it shall be recognized, as stated elsewhere, that conditions beyond borehole locations may change, potentially impacting on the findings and recommendations in this report.

Reference should be made to the standard Limitations of this Report, which follows the text and forms an integral part of this document.

1.3 Site and Project Description

The QCH site is located at and near 3045 Baseline Road in Ottawa, ON. As shown in Figure 1, seven campus facilities are proposed to be built or realigned. Details of each proposed feature are described below:

- A new free-standing Parking Garage (possibly 4 to 5 levels), rectangular in shape, approximately 35 x 75 m, is to be built north of the existing parking garage, on the northwestern side of the site. A grassy area with trees and paved bike lanes currently exists within the structural footprint area. Borehole elevations in the area (current investigation only) vary between 77.2 and 76.6 meters above sea level (masl), as per the CGVD28 geodetic datum. WSP understands that no underground parking levels are planned.
- A new single-story Materials Management Addition, rectangular in shape, approximately 45 x 60 m, is to be built north and adjacent to the existing main hospital building, at the location of the existing Materials Loading Area. The borehole elevations in the area (current investigation only) vary between 75.0 and 75.8 masl.

- The new loading dock, with an “L-shape” approximately 22 x 22 m in area, may be upgraded or relocated slightly (but appears to stay at the same location as shown on conceptual development plans for the site). The Loading Dock is located southwest of the existing Cancer Centre, on the northeastern side of the campus. The new Loading Dock is to be connected on its western side to the proposed realigned Materials Loading Area. The borehole elevations in the area (i.e., current investigation only) vary between 74.9 and 75.0 masl.

- A new Emergency Department (ED) Addition, also ‘L’ shaped and approximately 60 x 100 m in area, is to be built east and adjacent to the main hospital building, at the location of the existing eastern paved entrance and paved access lane. An ambulance parking area is to be built northeast of the proposed Emergency Department addition. The borehole elevations in the area (i.e., current investigation only) vary between 77.5 and 77.8 masl.

- A new Urgent Care Centre (UCC) Addition, with a parallelogram shape approximately 25 x 25 m in area, is to be built adjacent to the main hospital building at its southeastern corner, at the location of the existing paved entrance. The borehole elevation in the area (i.e., the current investigation only) is approximately 78.9 masl.

- The existing Loading Area is planned to be realigned and moved north to make place for the new Materials Management Addition. The borehole elevation in the area (current investigation only) is approximately 77.6 masl.

- A new Road System, approximately 400 to 500 m long, is to be constructed along the western side of the campus, connecting John Sutherland Drive at the north to Baseline Road at the south. A grassy area with trees and paved bike lanes currently exists along the alignment. Borehole elevations in the area (i.e., current investigation only) vary between 75.8 and 79.7 masl.

1.4 Scope of Work

The scope of work for the subject geotechnical investigation included the following:

- A desktop study and review of existing geotechnical reports and published information for the site and surrounding area;

- Laying out and surveying borehole locations and elevations and obtaining utility locates at the site;

- Drilling of 16 exploratory boreholes, with bedrock coring at seven of the boreholes;

- Obtaining soil and bedrock core samples for inspection and possible testing;

- Installing casing at two locations for Vertical Seismic Profiling (VSP);

- Installing six groundwater monitoring wells (50mm diameter PVC casing, protective covers);

- Measuring water levels within the monitoring wells several days after drilling;

- Geotechnical and chemical laboratory testing of soil samples; and

- Preparation of this report which presents the factual results of the investigation and provides geotechnical considerations and recommendations related to the design and construction of the proposed hospital expansion.

2.0 SITE INVESTIGATION

2.1 Desktop Study

WSP reviewed available geological maps and databases, as well as all information made available during the initial project planning phase. WSP summarized all information in a Desktop Study Memorandum submitted on June 24, 2024. A copy of the memorandum is attached in Appendix E for reference.

In brief, the subsurface conditions described in historical reports include a layer of topsoil/fill underlain by silty sandy and/or clayey deposits with variable amounts of sand and silt. This material is underlain by sandy glacial till over the dolostone bedrock of the Beekmantown Group. Based on historic boreholes and monitoring well data, groundwater was encountered at depths ranging between 0.5 to 5 mbgs.

2.2 Geotechnical Fieldwork

The subject geotechnical investigation was carried out between July 8 and 19, 2024. Sixteen boreholes were advanced within the site as follows:

- Boreholes BH24-01 to 03 in the proposed Parking Garage area;
- Boreholes BH24-08, 09 and 11 in the proposed Materials Management Addition area;
- Boreholes BH24-06 and 07 in the proposed Permanent Loading Dock area;
- Boreholes BH24-12 to 14 in the proposed Emergency Department Addition area;
- Borehole BH24-15 in the proposed Urgent Care Centre Addition area;
- Borehole BH24-05 in the proposed realigned Loading Area, and
- Boreholes BH24-04, 10 and 16 along the proposed new Road System alignment.

Borehole locations are shown on Figure 1 – Borehole Location Plan.

The boreholes were advanced using Massenza MI3 and Geoprobe 7822DT drilling rigs, supplied and operated by Strata Drilling Group. Standard Penetration Tests (SPTs) were carried out in all boreholes at regular depth intervals, in general conformance with ASTM D 1586. Shear vane testing was conducted in soft to firm cohesive soil layers, where encountered. Soil samples were recovered using 50 mm outside diameter (OD) split-spoon samplers, driven using the SPT technique.

At boreholes BH24-01, 03, 06, 10, 11, 13 and 15, sampling continued into the bedrock using diamond coring techniques after auger refusal was encountered. Borehole BH24-10A was advanced 0.3 m south of BH24-10 for rock coring purposes only due to drilling issues at the original location.

Six monitoring wells were sealed into boreholes BH24-03, 06, 13 and 16 to allow for subsequent measurement of groundwater levels at the site. The monitoring wells were generally constructed and tagged according to O. Reg. 903 requirements. The monitoring wells are to remain until decommissioning during construction of the new facilities.

Two 62 mm (2.5’’) diameter PVC pipes were installed in boreholes BH24-01 and BH24-15 to allow for subsequent vertical seismic profiling (VSP). The theory, methodology, and results of VSP are presented in Appendix D.

All geotechnical fieldwork was supervised by WSP staff who located/surveyed boreholes, directed drilling operations and in situ testing, and logged the recovered soil and bedrock samples. Upon completion of drilling operations, all soil samples and rock cores obtained from the boreholes were transported to WSP’s Ottawa office for further examination and possible laboratory testing.

The laboratory soils testing program included the determination of natural moisture content, grain size distribution, and Atterberg (plasticity) limits. Eight soil samples were submitted to Eurofins for basic chemical analysis related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. Laboratory testing results are provided in Appendix C.

All borehole locations were selected, staked in the field, and subsequently surveyed by WSP personnel. The borehole coordinates and ground surface elevations were determined using a Trimble R10 GPS survey unit. The geodetic reference system used for the survey is the North American Datum of 1983 (NAD83). The borehole coordinates are based on the Modified Transverse Mercator (MTM Z9) coordinate system. The elevations are referenced to the Geodetic datum (CGVD28). The borehole general information is summarized in Table 1 below.

Table 1: Borehole General Information

Borehole No.	Coordinates: MTM Z9		Ground Surface Elevation (masl)	Refusal Depth (mbgs) [Elevation (m)]	Comments
	Northing (m)	Easting (m)			
BH24-01	5022053	358896	77.6	4.0 [73.6]	Rock coring to 10.4 m. VSP casing installed.
BH24-02	5022039	358957	77.2	3.3 [73.9]	-
BH24-03	5022027	358915	77.4	5.0 [72.4]	Rock coring to 10.1 m. Monitoring wells installed in overburden and rock.
BH24-04	5021961	358902	75.8	3.2 [72.6]	-
BH24-05	5022029	359004	77.6	4.7 [72.9]	-
BH24-06	5022009	359049	75.0	2.9 [72.1]	Rock coring to 6.3 m. Monitoring well installed in overburden.
BH24-07	5021991	359041	74.9	2.0 [72.9]	-
BH24-08	5021965	359018	75.8	1.7 [74.1]	-
BH24-09	5021951	359056	75.0	1.8 [73.2]	-
BH24-10	5021862	358938	79.3	6.1 [73.2]	Rock coring to 9.9 m.
BH24-11	5021943	359030	75.1	1.8 [73.2]	Rock coring to 5.3 m.
BH24-12	5021978	359121	78.8	7.8 [71.0]	-
BH24-13	5022010	359134	77.5	7.2 [70.3]	Rock coring to 11.2 m. Monitoring wells installed in overburden and rock.
BH24-14	5021983	359139	78.7	8.7 [70.0]	-
BH24-15	5021898	359154	78.9	10.0 [68.9]	Rock coring to 16.0 m.
BH24-16	5021739	358969	79.7	7.1 [72.6]	Monitoring well installed in the overburden.

3.0 SUBSURFACE CONDITIONS

3.1 General

The following section provides a general description of the major soil and bedrock layers encountered during the geotechnical investigation. It should be noted that the following discussion includes several simplifications for the purposes of discussing broadly similar soil strata and bedrock types. Boundaries between geological materials may be gradational, and variable across lateral distances. Subsurface conditions may vary between and beyond the borehole locations and the reader should refer to WSP's standard Limitations for geotechnical investigation reports attached in Appendix F. WSP should be contacted immediately if any new information is found that contradicts our findings, so we may update this report accordingly.

A detailed description of soil and bedrock stratigraphy encountered at each borehole location is shown on the borehole logs included in Appendix A. Please note that the factual descriptions shown in each borehole log take precedence over the generalized (and simplified) descriptions presented below.

It is to be noted that distinguishing between fill, a potential native granular deposit, and possibly reworked glacial till layers was difficult and, as such, may be imprecise in several of the boreholes.

3.2 Topsoil and Organics

A layer of topsoil and/or fill with organics was encountered at the ground surface at boreholes BH24-01 to 04, 13, 15 and 16. The measured layer thickness ranged from approximately 50 mm to 150 mm.

Topsoil and organic fills should be stripped from the construction area and stockpiled for possible use during site reinstatement after construction.

3.3 Existing Pavement Structures

A flexible pavement structure was encountered at boreholes BH24-05, 07 to 12, and 14.

A rigid (concrete-based) pavement structure was encountered at borehole BH24-06 in the existing Loading Dock area.

The existing pavement structures consisted of asphaltic concrete or concrete overlying a granular road base and subbase fill. The measured asphaltic concrete thickness ranged from 50 mm to 150 mm. Where found, the concrete thickness was approximately 150 mm.

The granular road fill comprised variable amounts of sand and gravel with a trace to some silt. The fill extended to depths ranging from approximately 0.20 m to 1.68 m below ground surface (mbgs).

Grain size distribution tests were conducted on seven samples of the fill layer and results are presented in Appendix A summary of the grain size distribution test results is also presented in the table below.

Table 2: Results of Grain Size Analysis – Existing Granular Road Fill

Borehole No.	Sample No.	Sample Depth (mbgs)	Grain Size Distribution		
			% Gravel	% Sand	% Fines
BH24-05	SS-1	0.00 – 0.61	45	45	10
BH24-06	SS-1A	0.00 – 0.08	41	47	12
BH24-06	SS-2	0.76 – 1.37	35	53	12
BH24-08	SS-1	0.13 – 0.61	28	53	19
BH24-11	SS-1A	0.10 – 0.30	26	56	18
BH24-12	SS-2	0.76 – 1.37	21	64	15
BH24-14	SS-1	0.30 – 0.76	38	45	17

*Fines refer to particles less than 0.075 mm in size (US Sieve No. 200).

Natural moisture content determination conducted on six samples of the pavement granular fill material yielded moisture contents ranging from about 2% to 6% (i.e., dry side of optimum per ASTM D 698).

3.4 General Fill

Fill (i.e., not part of a pavement structure) was encountered at boreholes BH24-01 to 04, 10, and 12 to 16, at depths ranging between 0 and approximately 1.5 mbgs. The fill thickness ranged between 0.4 m to 3.0 m at the borehole locations. The fill was described as a heterogeneous mixture of sand and silt, with variable amounts of clay and gravel (i.e., sand, sand and gravel, gravelly sand, silty sand, sandy silt, clayey silt, and silty clay depending on location).

Standard Penetration Tests (SPTs) were carried out within the fill and returned ‘N’ values ranging from 3 blows to over 50 blows per 0.3 m of penetration. Such values are indicative of very loose to very dense material, but most of the material is characterized as loose to compact.

Grain size distribution tests were conducted on six samples of the fill layer and results are presented in Appendix C. Grain size distribution boundaries in this report are based on the Unified Soil Classification System. A summary of the test results is also presented in the following table.

Table 3: Results of Grain Size Analysis – General Fill

Borehole No.	Sample No.	Sample Depth (m)	Grain Size Distribution			
			% Gravel	% Sand	% Silt	% Clay
BH24-03	SS-1B	0.15 – 0.61	39	45	16	
BH24-04	SS-1	0.00 – 0.61	21	43	36	
BH24-12	SS-2	0.76 – 1.37	16	54	30	
BH24-12	SS-4	2.29 – 2.90	27	42	23	8
BH24-13	SS-2	0.76 – 1.37	17	21	33	29
BH24-15	SS-1	0.00 – 0.61	38	47	15	

The natural moisture content of eight samples of the general fill material ranged from approximately 3% to 16% based on laboratory tests.

3.5 Cohesive (Clayey Silt, Silty Clay) Deposit

A layer of clayey silt to silty clay with variable amounts of sand was encountered at all boreholes except BH24-06 through 11. This layer was initially encountered at depths varying between 0.5 and 3.0 mbgs. Thickness varied between 1.7 m and 9.0 m as shown on the borehole logs.

At boreholes BH24-04, 05, 14 and 16, the upper portion of the deposit comprised stiff, brown to brownish grey material (i.e., inferred weathered crust) overlying firm to soft, brownish grey to grey silty clay. The weathered crust had a thickness ranging from 0.9 to 1.8 m. The weathered crust, or at least a clearly defined stiff brown undisturbed clay layer, was not observed in boreholes BH24-01 to 03, 12 and 13.

Twenty-eight shear vane tests were performed within the cohesive deposit and returned in-situ shear strengths ranging between 26 kPa to higher than 132 kPa, with the higher strengths occurring in the upper crust as noted. Sensitivity (i.e., ratio of undisturbed to remolded shear strength) ranged between 1 to 7. Clays with a sensitivity ratio of 4 to 8 are categorized as “sensitive” in classical soil mechanics, and “insensitive” when the ratio is less than 2. Soils with sensitivity greater than 8 should be treated with caution during construction as they may become very weak when disturbed.

Atterberg Limits and water content tests were conducted on eleven samples of the cohesive deposit. Results are presented in Appendix C and summarized below.

Table 4: Results of Atterberg Limits Test – Cohesive Deposit

Borehole No.	Sample	Depth (m)	Water content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Liquidity Index	USCS Symbol
BH24-01	SS-4	2.29 – 2.90	42	57	17	40	0.6	CH
BH24-02	SS-3	1.57 – 2.13	61	53	16	37	1.2	CH
BH24-04	SS-2	0.76 – 1.37	24	33	17	16	0.4	CI
BH24-04	SS-4	2.29 – 2.90	32	26	12	14	1.5	CL
BH24-05	SS-3	1.52 – 2.13	32	33	11	21	1.0	CI
BH24-12	SS-7	6.10 – 6.71	40	35	12	23	1.2	CI
BH24-13	SS-5	3.05 – 3.66	41	28	12	16	1.8	CL
BH24-14	SS-6	4.57 – 5.18	44	35	13	22	1.4	CI
BH24-15	SS-6	3.81 – 4.42	42	35	14	21	1.3	CI
BH24-15	SS-9	8.38 – 8.99	41	30	13	16	1.7	CL
BH24-16	SS-4	2.29 – 2.90	53	43	15	27	1.4	CI

The natural moisture content for sixteen samples of material ranged from 20% to 61% based on laboratory tests.

3.6 Glacial Till

Glacial till was encountered at boreholes BH24-01 to 03, 05 to 11, 13 and 16, at depths ranging between 0.3 m and 6.7 mbgs. In general, the glacial till comprised a heterogeneous mixture of clay, gravel, cobbles and boulders in a silt and sand matrix. The thickness of the till layer ranged from approximately 0.2 m to 4.8 m.

Standard penetration tests carried out within the glacial till reported ‘N’ values ranging from 0 to over 50 blows per 0.30 m of penetration, indicating a very loose to very dense state of packing, but more generally compact. In general, loose till was only encountered directly below the cohesive deposit or where the till was relatively thick.

Grain size distribution tests were conducted on eleven samples of the glacial till and the results are presented in Appendix C. A summary of the grain size distribution is also presented in the following table.

Table 5: Results of Grain Size Analysis – Glacial Till

Borehole No.	Sample No.	Sample Depth (m)	Grain Size Distribution			
			% Gravel	% Sand	% Silt	% Clay
BH24-03	SS-4	2.29 – 2.90	18	39	31	12
BH24-03	SS-6	3.81 – 4.42	1	40	59	
BH24-05	SS-4	2.29 – 2.90	7	31	55	7
BH24-06	SS-3	1.52 – 2.13	3	65	32	
BH24-07	SS-2	0.76 – 1.37	23	41	26	10
BH24-08	SS-2	0.76 – 1.37	33	31	31	5
BH24-09	SS-3	1.52 – 1.80	13	29	58	
BH24-10	SS-3	1.52 – 2.13	18	38	37	7
BH24-10	SS-5	3.05 – 3.66	14	37	42	7
BH24-13	SS-8	6.71 – 6.95	1	53	29	17
BH24-16	SS-6	4.57 – 5.18	13	39	39	9

The natural moisture content for seventeen samples of the till material varied between 8% and 22% based on laboratory tests.

3.7 Refusal and Bedrock

Auger, split-spoon sampler, and/or casing refusal were noted in all boreholes at depths ranging between approximately 1.7 mbgs to 10.0 mbgs. Refusal was encountered on either very dense glacial till, boulders or presumed bedrock.

Bedrock was confirmed by coring at boreholes BH24-01, 03, 06, 10, 11, 13, and 15, using rotary diamond drilling techniques and retrieving HQ-sized rock cores. Total cored lengths in these boreholes ranged from 3.3 m to 6.4m.

Cored rock samples were generally described as fresh, bedded, grey, fine to medium-grained, non- to slightly porous, medium strong to strong, dolostone with intermittent shale beds. The dolostone was interbedded with some fresh to moderately weathered, light brown, slightly to moderately porous sandstone observed at boreholes BH24-03 and 13 only.

Rock Quality Designation (RQD) values for the recovered rock core samples ranged from 0 % (very poor) to 100% (excellent); however, RQDs were typically in the fair to excellent range (i.e., above 40%).

The results of the UCS testing carried out on five samples of the bedrock indicated strengths ranging from 120 to 279 MPa. These results are characteristic of very strong bedrock overall, but localized rock strengths may vary widely. Many tests of cored specimens may be required to obtain a fulsome characterization of the rock mass, as it pertains to site-wide interpretations for built structures. Discontinuities (joins and fractures) also have a significant effect on strength of a rock mass and should be considered in these interpretations. Refer to Limitations in Appendix F for more information.

Excavations extended down to the dolomitic bedrock as part of previous construction projects at the site, revealed soil filled vertical joints or clefts in some locations. The infilling generally consisted of dense to very dense glacial till and the width of the joints was found to range between about 100 and 600 mm. Such fissures can create issues for foundation piling (extended driving, drift, reduced capacity, etc.) as described later in this report and in historical site records.

Photographs of retrieved rock core samples are provided in Appendix B for reference.

3.8 Groundwater

Monitoring wells were sealed into boreholes BH24-03, 06, 13 and 16 as part of the current investigation. The following table summarizes the measured groundwater levels.

Table 6: Summary of Groundwater Conditions

Monitoring Well	Ground surface Elevation (masl)	Groundwater Depth (mbgs)	Groundwater Elevation (masl)	Bottom of Well Elevation (masl)	Date
BH24-03 Shallow (Overburden)	77.4	4.6 (Dry) 4.6 (Dry)	72.8 72.8	72.8	2024-07-25 2024-08-07
BH24-03 Deep (Bedrock)	77.4	5.2 5.4	72.2 72.0	67.3	2024-07-25 2024-08-07
BH24-06 (Overburden)	75.0	1.9 2.0	73.1 73.0	72.1	2024-07-25 2024-08-07
BH24-13 Shallow (Overburden)	77.5	4.5 4.7	73.0 72.8	70.4	2024-07-25 2024-08-07
BH24-13 Deep (Bedrock)	77.5	5.0 5.3	72.5 72.2	66.3	2024-07-25 2024-08-07
BH24-16 (Overburden)	79.7	2.6 3.5	77.1 76.2	72.6	2024-07-25 2024-08-07

Groundwater levels are expected to fluctuate seasonally and over shorter periods of time. Higher groundwater levels should be expected during wet periods of the year, such as spring after the snowmelt or during periods of heavy rain.

3.9 Corrosion Testing

Soil samples from boreholes BH24-01, 03, 05, 08, 10, 13 and 15 were submitted to Eurofins Environmental Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. Test results are provided in Appendix C and are also summarized in the following table.

Table 7: Chemical Test Results (Corrosion Parameters)

Borehole No.	Sample Number	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
BH24-01	SA-3	1.52 – 2.13	0.006	0.01	0.26	8.19	3846
BH24-03	SA-5	3.05 – 3.66	0.002	0.01	0.14	8.55	7143
BH24-05	SA-5	3.05 – 3.66	0.019	0.03	0.49	8.29	2037
BH24-07	SA-3	1.52 – 1.96	0.064	0.04	1.06	8.54	943
BH24-08	SA-3	1.52 – 1.73	0.064	0.02	1.07	8.85	935
BH24-10	SA-4	2.29 – 2.90	0.025	0.01	0.7	8.87	1429
BH24-13	SA-4	2.29 – 2.90	0.003	0.01	0.16	8.32	6250
BH24-15	SA-7	5.33 – 5.94	0.369	0.06	4.49	7.74	223

4.0 DISCUSSION

4.1 General

This section of the report provides engineering guidance related to the geotechnical design aspects of the proposed hospital expansion project, based on our interpretation of the available information described herein and the project requirements. It should be noted that considerations and recommendations are intended for Designers. Contractors bidding on or undertaking construction works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedules, safety, equipment capabilities, etc. Reference should be made to the standard Limitations of this report, which form an integral part of this document and are provided in Appendix F.

4.2 Proposed Pavement Structures (Parking Garage, Loading Dock, New Road)

4.2.1 Pavement Design

4.2.1.1 Subgrade Preparation

At the proposed Parking Garage, the Realigned Loading area, and the ambulance parking area, the pavement subgrade comprises heterogeneous sandy fill and/or natural cohesive deposits (clayey silt, silty clay).

At the Permanent Loading Dock area, the pavement subgrade comprises heterogeneous sandy fill and/or glacial till (silty sand, sandy silt).

At the proposed New Road area, the pavement subgrade comprises heterogeneous sandy fill, native cohesive deposits (clayey silt, silty clay) and/or the glacial till (silty sand, sandy silt).

Deleterious materials, such as loose, disturbed soils, reworked cohesive fills, or soil containing organic material such as peat, should be removed beneath proposed paving areas. Geotechnical subgrade inspections are required during construction and shall confirm that deleterious materials are stripped, and exposed soils are suitable, relatively undisturbed or suitably recompacted, and cleared of ponded water, prior to placing engineered fill. Remedial work, such as soil replacement, should be carried out as directed by the Geotechnical Engineer. All stripping and earthwork activities must be performed in a manner consistent with good erosion and sediment control practices. Site soils may be susceptible to erosion and shall be protected according to best practices for temporary and permanent conditions. Heavy vehicle traffic should be limited from driving on exposed subgrade materials, and rutting damage shall be repaired at the direction of the Geotechnical Engineer.

Sections requiring grade raising should be backfilled using acceptable earth borrow (e.g., per OPSS.MUNI 206/212), Select Subgrade Material (per OPSS.MUNI 1010), existing site materials approved by the Geotechnical Engineer and/or Qualified Professional (QP) for reuse, or additional granular base if grade changes are minor. Fill material should be placed in 300 mm or thinner lifts as required for compaction to at least 95% of Standard Proctor Maximum Dry Density (SPMDD).

4.2.1.2 Pavement Drainage

The pavement subgrade should be crowned or sloped to promote subdrainage of the granular base and subbase layers towards perimeter swales or subdrainage piping connected to a positive frost-free outlet. Class 1 non-woven geotextile should be placed on top of the subgrade prior to the placement of the subbase pavement layer.

4.2.1.3 Flexible Pavement Structure

The following flexible pavement structures may be considered for the proposed roadway improvements and New Road areas, depending on anticipated traffic loadings. For the parking garage, it is assumed that the ground level will include a flexible pavement structure, while the higher levels will comprise post-tensioned concrete structural slabs.

Table 8: Flexible Pavement Structure Design

Pavement Component	Parking Garage Ground Level		Realigned Loading Area and New Road	Ambulance Parking Area
	Heavy Duty (with Truck Traffic)	Light Vehicles (Cars) Only	Heavy Duty (with Truck Traffic)	Heavy Duty (with Truck Traffic)
Hot Mix Asphalt	50 mm SP 12.5 60 mm SP 19	50 mm SP 12.5	60 mm SP 12.5 60 mm SP 19	50 mm SP 12.5 60 mm SP 19
OPSS.MUNI 1010 Granular A Base	200 mm	150 mm	300 mm	200 mm
OPSS.MUNI 1010 Granular B Subbase	400 mm	300 mm	400 mm	400 mm

*High density rigid Styrofoam insulation should be considered beneath parking garage ramps (and other areas as required) to provide increased frost protection.

The Performance Graded Asphalt Cement (PGAC) should consist of PG 58-34 for Traffic Category B. A “bump” of one to two grades should be considered when the pavement carries slow-moving or standing traffic, as recommended in the “MTO Superpave and SMA Guide”.

Construction should be carried out in conformance with procedures outlined in OPSS.MUNI 310 “Construction Specification for Hot Mix Asphalt”.

4.2.1.4 Rigid Pavement Structure

The existing Loading Dock pavement structure appears to be in good condition. If the loading dock is to be relocated, the following rigid (i.e., concrete) pavement structure could be considered:

Table 9: Rigid Pavement Structure Design

Pavement Component	Loading Dock
Portland Limestone Cement Concrete or equivalent	150 mm
High-Density Rigid Styrofoam Insulation	50 mm (minimum)
OPSS Granular A Base	300 mm
OPSS Granular B Subbase	900 mm

Construction should be carried out in conformance with procedures outlined in OPSS.MUNI 350 “Construction Specification for Concrete Pavement and Concrete Base”.

4.2.1.5 Compaction Requirements

Quality-controlled compaction of engineered fills and granular subbase and base materials will be essential for good performance of the roadway and parking/access ramp areas. Compaction should be carried out in conformance with procedures described in OPSS 501 “Construction Specification for Compacting” with compacted densities of the various materials being in accordance with Subsection 501.08.02 Method A. Granular base and subbase material should be uniformly compacted to at least 98% of SPMDD per ASTM D698.

4.2.1.6 Material Reuse

The existing granular road fills are not recommended for use in new pavement structures, given their relatively high fine content (i.e., >10% passing No. 200 sieve size). They can, however, be reused for general grade-raising purposes in selected areas.

Other existing fills (not part of a pavement structure), native cohesive material (silt, clay), and native glacial till should only be reused only for landscaping purposes due to relatively high fines content, the presence of plastic fines, and poor workability.

The excavated/reused soils must be free of any construction debris (such as old concrete, brick, or wood) and organic material. Also, any material to be reused should first be subjected to an environmental soil quality characterization per Provincial regulation O. Reg. 406/19 “On-Site and Excess Soil Management”.

4.3 Proposed Buildings and Additions (Emergency Department, Urgent care Center, Material Management)

4.3.1 Site Grading

At the time of writing this report no conceptual design information relative to site grading was available for the proposed hospital expansion areas.

It was assumed that the overall site grading would be similar to existing conditions and match with existing buildings on the site. Due settlement potential of the native cohesive deposit, any proposed grade raises of more than 1 m should be reviewed by WSP.

4.3.2 Frost Protection

All perimeter and exterior foundation elements in heated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior foundation elements are adjacent to surfaces that are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

As an alternative to earth cover, consideration could be provided to the use of rigid insulation. Additional guidance on insulation details can be provided if required. Also, refer to manufacturer design requirements for specific products.

If foundations need to be constructed during the winter months, foundation soils (i.e. subgrade, engineered fills, and backfill) must be protected from freezing temperatures using suitable construction techniques. The base of all excavations should be insulated from freezing temperatures immediately upon exposure, until heat can be supplied to the building interior and/or the foundations have sufficient earth cover to prevent freezing of the subgrade soils. Granular fills are not properly compatible in sub-freezing temperatures, nor when they contain ice pellets or snow; this must be avoided or addressed through appropriate protection systems.

4.3.3 Seismic Design

4.3.3.1 Liquefaction Potential

The very loose to compact silty sands (glacial till unit) that are present beneath the groundwater level may be susceptible to liquefaction during a significant earthquake event. Seismic liquefaction occurs when earthquake vibrations cause increases in pore water pressures within the soil. The presence of excess pore water pressures reduces effective stress and contact stress between the soil particles and reduces frictional resistance to shearing. Post-liquefaction settlement occurs when soil stabilizes into a denser more closely packed arrangement after an earthquake, potentially causing settlement at the ground surface depending on the thickness and depth of the layer. Excess pore pressures can also lead to a sudden temporary loss in strength, which can in some circumstances lead to lateral movements referred to as "lateral spreading" or "flow slides". These mechanisms are typically more important in geotechnical stability assessments of slopes, retaining walls, and along valley and shoreline features.

4.3.3.2 Liquefaction Assessment Methodology

For the purposes of this site and facility expansion, the liquefaction susceptibility of granular soils was evaluated by comparing the cyclic stress required to trigger liquefaction with available soil resistance. Liquefaction is predicted to occur when the available soil resistance is less than the cyclic stresses imposed (i.e., a factor of safety < 1.0).

The methodology used to assess liquefaction potential for this report is consistent with the “simplified” approach outlined in the CHBDC and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by a “design” earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out considering sampled soil characteristics and SPT N-values collected at the borehole locations, and groundwater data from the monitoring wells. The CRR profile with depth was calculated at borehole locations BH24-03 using the parameter $(N1)_{60cs}$, which is based on the SPT blow count obtained in the field, and corrected for overburden stress, sampler rod length, hammer energy efficiency, and total fines content.

4.3.3.3 Liquefaction Assessment Results

The liquefaction assessment was conducted using an earthquake magnitude (M) of 6.5 and peak ground acceleration of 0.394 g, which corresponds to a design earthquake having a 2% probability of being exceeded in 50 years (i.e., a 2,475-year return period) as outlined in the NBCC (2020).

The results of the analysis indicate that there are a few subsurface zones where liquefaction could potentially occur, but for the most part, these zones appear to be relatively thin and discontinuous beneath the site, and the soil is considered only marginally liquefiable. The magnitude of post-seismic settlement that may occur in these zones is estimated to be less than 35 mm. As such, there appears to be a small liquefaction risk potentially impacting on contemplated structural designs for the proposed hospital expansion.

The axial capacity of pile foundations deriving resistance from bedrock are not expected to be significantly affected by liquefaction effects from a typical earthquake event.

4.3.3.4 Seismic Site Classification

As outlined in the 2020 National Building Code of Canada (NBCC 2020), building foundations must be designed to resist a minimum earthquake force. In accordance with Tables 4.1.8.4.A and B of the NBCC 2020 and based on the results of the current and past geophysical testing, which included VSP testing at boreholes BH08-307, BH24-01 and 15, the average shear wave velocity in the dolomitic bedrock is higher than 1500 m/s. Therefore, Site Class A can be considered for the design of foundations resting on bedrock.

In the case of deep foundations with pile caps placed on the native soils at least 3 m above the bedrock, an X_{760} Site Designation is to be used according to NBCC 2020.

The VSP testing results are presented in Appendix D.

4.3.4 Foundation Design

At the proposed Parking Garage area (BH24-01 to 03), bedrock was encountered at depths ranging between about 3.3 to 5.0 mbgs. Concrete-filled steel pipe piles driven to bedrock are a potential design option and have been previously used to support QCH structures (see pile driving records in Appendix E). Other driven or bored pile options are also feasible. Shallow footings bearing on the natural cohesive deposit are not recommended due to concerns with settlement potential and sensitivity to disturbance. Additionally, a substantial, higher strength weathered crust was not encountered during the current investigation (or was too thin). Previous excavation and construction work explain the presence of considerable fill quantities, and the former crust, if any, was potentially removed and/or reworked at this location. Indeed, historic borehole logs for the site (summarized in the attached desktop study) describe a harder and stiffer cohesive deposit as compared to the present investigation findings.

If raft / combined footing designs are considered for the Parking Garage structure, we recommend that they be assessed for settlements and deformations using FEM software. Geotechnical input parameters may include an estimated subgrade reaction modulus of 3/B (MPa/m) for the cohesive deposit and 9/B (MPa/m) for the glacial till layer. Analysis of the full soil profile is required, and a detailed geotechnical design review must be conducted if these foundations options are taken.

At the proposed Materials Management Addition area (i.e., BH24-08, 09 and 11), bedrock was encountered at an approximate depth of 1.8 mbgs. Footings bearing directly on bedrock are therefore recommended here.

At the proposed Emergency Department and Urgent Care Centre Addition areas (i.e., BH24-12 to 15), bedrock was encountered at depths ranging between about 7.2 to 10.0 mbgs. Driven piles (e.g., concrete-filled concrete pipe, H-piles, etc.) or possibly bored piles / drilled shafts extending to (into) bedrock should be considered for design. Shallow foundations bearing on the natural cohesive deposit may be feasible for a lightly loaded one-story structure only, though given the highly sensitive nature and strength variability of the deposit, this is not recommended.

4.3.4.1 Footings on Bedrock

Based on the results of the subsurface investigation, the proposed Materials Management Addition may be supported by shallow spread/strip footings bearing on sound, slightly weathered to fresh dolostone found at elevations 74.0 to 73.1 masl in the boreholes.

For footings placed on sound competent bedrock, a factored Ultimate Limit State (ULS) bearing resistance of 1,500 kPa may be assumed for design. Serviceability Limit States (SLS) net bearing resistances do not generally apply to the design of foundations on the bedrock, provided the bedrock surface is properly cleaned of soil and weathered/fractured bedrock material at the time of construction.

For the ULS sliding resistance of a cast-in-place footing placed on bedrock, an unfactored sliding friction coefficient of 0.70 may be considered. In accordance with the 2023 Canadian Foundation Engineering Manual (CFEM 2023), a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying bedrock.

All bearing surfaces should be checked, evaluated, and approved at the time of construction by a Geotechnical Engineer who is familiar with the findings of this investigation and the design and construction of similar projects prior to placement of any concrete, back-fill, etc. Inspections shall confirm that zones of loose, weathered rock or weak shale seams are removed. If any fractures, joints, or voids are found within the exposed bedrock surface at the footing locations, they should be cleaned out (Hydrovac) and backfilled with U-Fill, or grouted.

4.3.4.2 Driven Piles

Based on the geotechnical investigation findings, the bedrock depth ranges from approximately 3.3 to 5.0 mbgs (73.9 to 72.4 masl) at the proposed Parking Garage location, and 7.2 to 10.0 mbgs (72.6 to 68.9 masl) at the Emergency Department and Urgent Care Centre Addition areas. Driven piles extending to bedrock, such as concrete-filled steel pipe piles as noted previously or possibly H-piles (with driving shoes for end protection), could be considered as a foundation option for these facilities.

The ULS factored geotechnical axial compression resistance for piles tipped on sound bedrock is calculated as 48 MPa (i.e., dolostone with UCS of 120 MPa and applying a resistance factor of 0.4 per CFEM 2023). This factored ULS value may be greater than the structural capacity of the pile, which will likely govern the design and should be checked by the Designer. For end-bearing piles on bedrock, SLS conditions generally do not govern design. The post-construction settlement of the structural elements, which derive their support from the piles driven to bedrock, should be negligible.

Based on the as-built pile driving records of the main building, the existing piles were driven to bedrock at elevations ranging between about 74.0 and 70.9 masl, except one pile which was driven deeper to elevation 64.0m. It is likely that the deeper pile was driven through a rock fracture, as previous geotechnical investigation reports noted that excavations down to bedrock during past construction projects revealed the existence of soil-filled vertical joints or clefts at a few locations.

Concrete, if required to fill piles, should be placed using the tremie method assuming that groundwater will be encountered above the bedrock. Installation should follow the requirements mentioned in OPSS 903

(Construction Specification for Deep Foundation) or similar applicable standards.

It is recommended that pile driving logs and possibly dynamic monitoring and capacity testing (known as PDA testing) be carried out by the Contractor during the piling operation for quality control documentation and to verify the transferred energy from the pile driving equipment and the load-carrying capacity of the piles. The Geotechnical Engineer can provide more information when details are known.

4.3.4.3 Bored Piles or Drilled Shafts

Deep bored/drilled shaft foundations embedded into the dolostone bedrock can potentially provide higher capacities than driven piles bearing directly on the bedrock surface.

Based on the current subsurface exploration program, which encountered little bedrock weathering, it is recommended that the upper 0.5 m of the bedrock be ignored for developing shaft capacities. This shall also be the minimum rock socket depth if end-bearing shafts are considered.

The following unfactored geotechnical resistances may be used to evaluate deep foundations.

End Bearing Resistance:

$$q_a = \sigma_c K_{sp} d$$

Where: q_a = allowable bearing pressure

σ_c = use the smallest between the average unconfined compressive strength of rock core (187 MPa) and the maximum compressive strength of concrete (assume 28 MPa at 28 days)

K_{sp} = empirical factor, use 0.2 for medium spacing

d = depth factor = $1+0.4\frac{L_s}{B_s} \leq 3$

L_s = length of socket

B_s = diameter of socket

Shaft Resistance:

$$Q_s = \pi B_s L_s q_s$$

Where: Q_s = Ultimate socket shear load

B_s = diameter of the socket

L_s = length of the socket

q_s = average unit shear resistance along the socket

$$\frac{q_s}{P_a} = b\left(\frac{q_u}{P_a}\right)^{0.5}$$

Where: q_s = unit socket shear

P_a = Atmospheric pressure, use 0.1013 MPa

b = an empirical factor, use 1.41, Table 9.17 CFEM 2023

q_u = smallest value of either the unconfined compressive strength of rock or concrete,

use 1.4 MPa (Concrete is the controlling value) $q_{u\ concrete} = 0.05f'_c$

For limit state design of the rock socketed foundations, it is recommended that the ultimate axial capacity for drilled shafts be calculated by tripling the allowable values above and then applying a geotechnical resistance factor of 0.4 and 0.3 for compression and tension (uplift) load cases, respectively.

Based on an inferred compressive strength of shaft concrete (i.e., assume 28 MPa strength at 28 days), a factored ultimate end-bearing resistance of 12 MPa may be assumed for preliminary shaft design. Since sound rock is expected to be present at the shaft base, it may only be necessary to penetrate the rock approximately 0.5m to develop full base resistance, and in this case shaft resistance in axial compression could be ignored.

Further geotechnical evaluations are required for rock sockets more than approximately 1.5 m deep (but depending on shaft diameter), to evaluate the combined total axial compressive capacity from the shaft and base resistance values. As noted in FHWA-NHI-10-016, the transfer of compression load via side shear resistance to the surrounding rock results in decreasing load with depth. The initial portion of the foundation load is transferred predominantly to side resistance, and the load transmitted to the shaft base may be small, reducing as the socket length to shaft diameter ratio increases. Full side resistance will eventually be mobilized and further increases in load must then be resisted by the base. In combining the side and base resistance of a socket, it is necessary to consider the rock type and quality and the resulting load transfer mechanism. The concrete-rock bond in the socket sidewall must not be broken if combined shaft and base resistances are to be used. Displacement required to mobilize maximum base resistance varies but is typically reached at approximately 4 to 5 percent of the shaft diameter for bearing in rock.

The Designer should note that shaft resistance will be required if uplift (e.g., wind) forces exist in the structure. Shaft base bonding generally does not contribute to uplift resistance.

For preliminary information purposes, a summary of factored side-shear resistances for two selected rock socket sizes and embedment depths is provided in Table 10 below. Rock socket lengths include an estimated 0.5 m of fractured/weathered rock which is ignored in the calculations. Final design values should be reviewed by WSP when more information is available.

Table 10: Summary of Factored Geotechnical Resistances for Bored Piles or Drilled Shafts in Rock

Rock Socket Diameter, m	Rock Socket Length, (m)	Rock Socket Shear Resistance at ULS	
		Compression, kN	Uplift, kN**
1.00	2.5	1,300	1,000
1.00	3.0	1,600	1,250
1.52	3.5	3,000	2,200
1.52	4.0	3,500	2,600

*Includes 0.5 m weathered rock ignored in resistance calculation.

**Poisson’s effect not considered. Reduction factor may apply depending on shaft design.

4.3.4.4 Lateral Resistance

If vertical piles are used to resist lateral loadings, then the horizontal reaction to the piles may be calculated from the expression (cohesionless soils only):

$$k_s = z\, n_h / d$$

Where: k_s = coefficient of horizontal subgrade reaction (MPa / m)

d = pile diameter or width (m)

n_h = constant of horizontal subgrade reaction (MPa / m), use 3,000 kN/m^3 for glacial till

z = depth below ground surface (m)

For cohesive soils, the horizontal reaction to the piles may be calculated from the following expression:

$$k_s = 67\,Su \,/\, d$$

Where: k_s = coefficient of horizontal subgrade reaction (MPa / m)

d = pile diameter or width (m)

Su = undrained shear strength, use 20 Kpa for natural cohesive deposit

Pile designs should be evaluated using geo-structural software, such as RS-Pile or L-Pile, to assess deflection and to evaluate moment and shear capacities.

In soils, group action for lateral loadings should be considered when the pile spacing in the direction of loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading using a reduction factor, R, as provided in the table below, where “d” is the pile diameter or width. The reduction factor should be less significant in rock-loading situations; in these situations, a value of 1.0 could be considered for spacings >4d.

Table 11: Horizontal Group Reduction Design Factors

Pile Spacing in Direction of Loading	Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The coefficient of horizontal subgrade reaction values may be used to calculate the lateral deflection of the pile (i.e., the SLS response), taking ground-structure interaction into account.

A more detailed assessment of the lateral resistance of the foundations to lateral loading should be undertaken once the number of piles and the layout of the pile group is known.

Battered piles could be used to resist horizontal loads, due to the generally weak lateral resistance offered by soils expected to be present near the pile cap.

4.3.4.5 Downdrag

When piles have been installed in or through a clay deposit that is subject to consolidation, from surcharge loadings for example, the resulting downward movement of the clay around the piles, as well as in any soils above the clay layers, induces downdrag forces in the piles through negative skin friction. Downdrag forces may also develop from post-earthquake liquefaction settlement, which may have limited effect at this site.

Downdrag forces, if any, should be deducted from the net pile capacity. Large downdrag forces are not expected for this project, based on our current understanding, but pile designs should be reviewed by the Geotechnical Engineer prior to construction.

4.3.4.6 Rock Anchors

The use of rock anchors to resist uplift forces on the foundations could be considered where additional uplift resistance is required.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes:

i) Failure of the steel tendon or top anchorage

ii) Failure of the grout/tendon bond

iii) Failure of the rock/grout bond, and

iv) Failure within the rock mass, or rock cone pull-out.

Potential failure modes i) and ii) are structural and are best addressed by a structural engineer.

For potential failure mode iii), the *factored* bond stress at the grout/rock interface may be taken as 1,000 kPa (or 1/30 of the compressive strength of the grout) for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance is calculated based on the weight of the potential mass of rock and soil which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \varphi \frac{\pi}{3} \gamma' D^3 \tan^2 \text{–} \theta$$

Where: Q_r = Factored uplift resistance of the anchor (kN);

φ = Geotechnical resistance factor (use 0.4);

$g^{/}$ = Effective unit weight of rock and soil (use 15 kN/m^3 below the groundwater level);

D = Anchor length in metres; and,

q = one-half of the apex angle of the rock failure cone (use 30°).

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 sin^2 \varphi + \ aD^2 \sin \varphi + bD^2 \sin \varphi + abD$$

Where: V = Volume of the truncated trapezoid failure zone (m^3);

D = Depth of anchor group (m);

a = Width of anchor group (m);

b = Length of the anchor group (m); and,

j = ½ of the apex angle of the rock failure cone, use 30°.

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \varphi\gamma' V$$

Where: Qr = Factored uplift resistance of the anchor (KN);

j = Geotechnical resistance factor, use 0.4;

$g^{/}$ = Effective unit weight of rock and soil, use 15 kN/m^3 below the water table; and,

V = Volume of truncated trapezoid (m^3).

It is recommended that proof load tests be carried out on any new anchors to confirm their resistance. The proof load tests should be carried out in accordance with the Post Tensioning Institute (PTI) Recommendations for Prestressed Rock and Soil Anchors (2004).

The Geotechnical Engineer or representative shall be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids.

Confirmation of sufficient embedment into the rock beneath the foundations should be carried out during construction to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

4.3.4.7 Interior Floor Slabs

In preparation for the construction of interior building floor slabs, all deleterious or otherwise loose, wet, or disturbed material should be removed from beneath the building footprint area. Provisions should be made for at least 250 mm of OPSS 1010 Granular A to form the base of the floor slab. Any bulk fill required to raise the grade up to the underside of the Granular A should consist of OPSS 1010 Granular B Type II. The under-slab fill should be placed in a maximum 300 mm thick lift and should be compacted to 100% of SPMDD using suitable vibratory compaction equipment.

The floor slabs should be structurally separate from the foundation walls and columns. Sawcut control joints should be provided at regular intervals and along column lines to minimize shrinkage cracking.

4.3.5 Permanent Drainage

Provision should be made for perimeter drainage around all exterior below-grade walls of the proposed buildings. Subfloor drainage should also be considered for any buildings with basements. Subdrainage should comprise perforated pipe (100 to 150 mm diameter suggested) in a surround of 19 mm clear stone, fully wrapped in geotextile. Subdrain pipes should discharge by gravity to an adjacent storm sewer or sump pit.

4.3.6 Foundation Wall Backfill

Foundation walls should be backfilled with free draining non-frost susceptible granular fill meeting the requirements of OPSS 1010 Granular B Type I materials. The backfill should be compacted to 95 % SPMDD using suitable compaction equipment. To reduce compaction-induced stresses, only light compaction rollers or plate tampers should be used within 1.0 m of the wall.

4.3.7 Lateral Earth Pressures

4.3.7.1 Static Loading

The lateral earth pressures acting on retaining walls will depend on the existing soil conditions, on the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and drainage conditions behind the walls. Seismic (earthquake) loading must also be considered in the design.

It is anticipated that excavations for the project will not extend deeper than 2.5 m below the existing ground surface. Based on the results of the drilling program, the excavations will be carried out within the existing fill, the natural cohesive deposit and possibly glacial till. The following parameters (unfactored) may be used for the design of retaining walls, including backfilled basement walls.

Table 12: Lateral Earth Pressures – Soil Parameters for static loading

Material	Unit Weight (kN/m^3)	Coefficients of static lateral earth pressure		
		Active, Ka	At rest, Ko	Passive, K_P
Cohesive Fill	17	0.39	0.56	2.56
Granular Fill	18	0.35	0.52	2.88
Natural Cohesive Deposit (Clayey Silt, Silty Clay)	17	1.0 (Short Term) 0.36 (Long Term)	1.0 (Short Term) 0.53 (Long Term)	2.77
Glacial Till (Silty Sand, Sandy Silt)	20	0.33	0.50	3.69
Granular A or Granular B Type II	22	0.27	0.43	3.85
Granular B Type I	22	0.31	0.47	3.53

Where the wall support and structure allow lateral yielding (e.g., unrestrained retaining walls), active earth pressures may be used in the design of the wall. Where the support does not allow lateral yielding, (i.e., foundation walls) at rest earth pressures should be assumed for design.

A minimum compaction and future traffic surcharge of 14 kPa, or as required by the Design Engineer, should be included in the lateral earth pressures for the structural design of the walls. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance equal to the height of the backfill above the base of the structure. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.

Select, free-draining granular fill should be used as a backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill.

The following equation can be used to calculate the lateral earth pressures for static loading conditions. It is assumed in this equation that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes, new lateral earth pressures will need to be calculated with different loading conditions (i.e., the soil above the wall treated as a surcharge).

$$\sigma_h = K(\gamma h + q)$$

Where: σ_h=Lateral earth pressure(kPa)

K=Earth pressure coefficient. Use 0.5 for foundation wall (restrained)

γ=The unit weight of soil used for backfilling behind the wall (use 22 kN/m^3 for compact granular material)

h=The depth to the point of interest or height of wall

q=The magnitude of any design surcharge at the ground surface

4.3.7.2 Seismic Loading

Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.

The horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the design PGA (i.e., k_h = 0.394). For structures which allow lateral yielding, k_h is taken as 0.5 times the design PGA (i.e., k_h = 0.197).

The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the K_{AE} obtained using the k_h values described above and assumed no vertical acceleration and wall to soil friction. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Table 13: Lateral Earth Pressures – Seismic loading parameters

Material	Seismic Earth Pressure Coefficients (Site Class A, 2% probability in 50 yrs)		
	Active, K_{AE} (Yielding)	Active, K_{AE} (Non-Yielding)	Passive, K_{PE}
Granular A or Granular B Type II	0.38	0.55	2.98
Granular B Type I	0.41	0.59	2.69

Lateral earth pressures will be higher under seismic loading conditions. If the foundation walls are intended to become partially flooded, then appropriate hydrostatic pressures should also be added to total earth pressure. The following equation provides the lateral earth thrust in case of an earthquake (per CHBDC, 2014):

$$P_{AE} = \frac{1}{2}\gamma H^2(1 - \mathrm{kV})K_{AE} \text{ and } P_{PE} = \frac{1}{2}\gamma H^2(1 - \mathrm{kV})K_{PE}$$

Where: P_{AE}=Resultant active earth thrust, including static and dynamic loads (kN/m)

P_{PE}= Resultant passive earth thrust, including static and dynamic loads (kN/m)

K_{AE}=Seismic active earth pressure coefficient, can be taken as 0.59 (Granular B Type 1, non-yielding wall)

K_{PE}= Seismic passive earth pressure coefficient, can be taken as 2.69 (Granular B Type 1, non-yielding wall)

k_V=Vertical acceleration coefficient (use 0)

The following equation provides the total active thrust height (h) from the base of the wall:

$$h = \frac{P_A\left(\frac{H}{3}\right) + \triangle P_{AE}(0.6H)}{P_{AE}}$$

Where: h = height at which the thrust acts form the base of the wall (m)

P_A = Static component of the lateral thrust, acting at 0.3H above bottom of the wall (kN/m)

ΔP_{AE} = Dynamic component of the lateral thrust, acting at 0.6H above the bottom of the wall

4.3.8 Excavations & Groundwater Control

It is anticipated that the majority of excavations for the project will be less than 2.5 mbgs. Based on the results of the drilling program, the excavations will be carried out within the existing fill, the cohesive deposit, and potentially glacial till. Temporary excavation slopes with a maximum inclination of 1V: 1H could be profiled in soils above the water table (assume Type 3 soils or better under OSHA regulations). For submerged soils, the slope would be 1V: 3H. Excavations would be feasible with conventional hydraulic excavating equipment. Excavations within the bedrock will require heavy hydraulic breakers and likely drill and blast techniques if extensive.

If extensive deeper excavations are required (for example for deep utilities or if basement levels are added to the building program) then these recommendations should be reviewed during detailed design based on the actual excavation locations, sizes, and depths.

Based on the groundwater levels measured in boreholes as part of the current and previous subsurface investigations (i.e., 0.5 to 5.4 m below ground surface), the excavations may encounter groundwater seepage, and dewatering control may be required during construction and excavation activities. Based on the expected conditions, it should be possible to manage groundwater seepage using properly filtered sumps, ditches, pumps, etc. However, this should be reviewed when more information and excavation details are known.

4.3.9 Site Servicing

The water and sewer services will need to be protected against freezing conditions. In Ottawa, water-bearing services are typically placed a minimum of 2.4 m below grade to provide protection from frost.

At least 150 mm of OPSS 1010 Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs during construction, it may be necessary to place a sub-bedding layer consisting of 300 mm of compacted OPSS 1010 Granular B Type II beneath the Granular A. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95 % of SPMDD (per ASTM D698). The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from the spring line of the pipe to at least 300 mm above the top of the pipe, should consist of OPSS 1010 Granular A or Granular B Type I with a maximum particle size of 25 mm. The cover material should be compacted to at least 95 % of SPMDD.

The backfill material should consist of Granular B. Where the trench will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfills should be placed in a maximum 300mm thick lift and should be compacted to at least 95 % of the material’s SPMDD using suitable vibratory compaction equipment.

4.4 Proposed Equipment Pads

It is understood that granular pads, founded on native soil will be installed to support some equipment at the site. The exact location of these equipment has not yet been finalized. However, the site preparation should follow the recommendations in the below section.

4.4.1 Site Preparation

The subsurface stratigraphy generally consists of topsoil over existing fill which is underlain by silty clay to clayey silt over glacial till, over dolostone with shale beds bedrock. As part of the site preparation, all topsoil, existing fill containing organics and rootlets, and other unsuitable materials should be removed from the footprint of the proposed pads. The exposed subgrade should be protected from disturbance of construction traffic and graded to quickly drain away surficial runoff from the project site.

Engineered fill conforming to OPSS Granular A or Granular B (Type I or II) with a maximum particle size of 26.5mm and less than 5% fines content (or other approved equivalent) should be used to reduce problems with frost adhesion and heaving. A Class II non-woven geotextile separator as per OPSS.MUNI 1860 (e.g., Terrafix 360R or approved equivalent) should be placed between the existing soil and free-draining granular fill to filter fines from water.

All of the backfill materials should be placed in maximum loose lifts of 200 mm and compacted to at least 95% of SPMDD at ±2% of OMC. Heavy construction equipment should be maintained at a distance of at least 1 m away from the edge of the excavation while the backfill soils are being placed.

4.5 Construction Considerations

4.5.1 Excavation Adjacent to Existing Structure

It is understood that some of the existing structures are supported on concrete-filled pipe piles driven to bedrock and concrete caissons with pile cap systems to support the superstructure. Additionally, some of the existing structures are supported on spread footings on rock. Pertinent pile driving records are included in the Desktop Study Memorandum, attached in Appendix E.

Excavations below existing pile caps, grade beams, and adjacent to existing pipe piles may be required for the project.

The Contractor is fully responsible for the detailed design and performance of the temporary shoring systems. The shoring method(s) chosen to support the excavation sides must consider soil and bedrock stratigraphy, the permissible movement of the shoring, the groundwater conditions, the methods adopted to manage the groundwater and construct the shoring systems, the potential ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities.

Given the expected relatively shallow depth of excavations (up to 2.5 mbgs), sloped excavations should be feasible in most areas. Where excavations are required adjacent to existing structures and utilities (and sufficient slopes cannot be provided) consideration may be given to the use of trench boxes or shoring to prevent deflection and movement of existing structures. If anchored temporary shoring is required, further guidance, at the design and construction stages, can be provided if required.

4.5.2 Corrosion and Cement Type

Soil samples from boreholes BH24-01, 03, 05, 08, 10, 13 and 15 were submitted to Eurofins Environmental Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. The results of this testing are provided in Appendix C and are summarized in the following table.

Table 14: Results of Basic Chemical Testing

Borehole No.	Sample Number	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
BH24-01	SA-3	1.52 – 2.13	0.006	0.01	0.26	8.19	3846
BH24-03	SA-5	3.05 – 3.66	0.002	0.01	0.14	8.55	7143
BH24-05	SA-5	3.05 – 3.66	0.019	0.03	0.49	8.29	2037
BH24-07	SA-3	1.52 – 1.96	0.064	0.04	1.06	8.54	943
BH24-08	SA-3	1.52 – 1.73	0.064	0.02	1.07	8.85	935
BH24-10	SA-4	2.29 – 2.90	0.025	0.01	0.7	8.87	1429
BH24-13	SA-4	2.29 – 2.90	0.003	0.01	0.16	8.32	6250
BH24-15	SA-7	5.33 – 5.94	0.369	0.06	4.49	7.74	223

The pH, resistivity and chloride concentration give an indication of the degree of corrosiveness of the sub-surface environment. Generally, the test results indicate a moderate to high potential for corrosion of exposed ferrous metal at the site which should be considered in the design of substructures.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater. Based on the standard A23.1-14 (CSA A23.1) by the Canadian Standards Association, the sulphate attack potential is considered low (i.e., less than moderate) on concrete structures at this site. Therefore, Type GU Portland cement should be acceptable for buried concrete substructures.

Corrosion effects on steel pipe piles depend on multiple factors, including ground chemistry, exposure to oxygenated environments, grader and quality of steel, coatings, etc. For preliminary considerations a sacrificial steel thickness of 5 mm could be assumed. However, it is strongly recommended that a metallurgical analysis be completed to confirm final design requirements. WSP can undertake a study if requested.

5.0 CLOSURE

We trust that this geotechnical report provided sufficient information to support the design and construction of the proposed development. WSP expects to be contacted if one of the assumptions made about the sign is changed. We remain available for any questions or concerns about the report.

Signature Page

WSP Canada Inc.

LICENSED PROFESSIONAL ENGINEER
A. A. KUITCHOUA PETKE
100542033
Apr10,2025
PROVINCE OF ONTARIO

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AKP/JSA/yj

https://wsponlinecan.sharepoint.com/sites/ca-ca00337141722/shared documents/06. deliverables/final report/rev-1/ca0033714.1722-reva-draft qch expansion geotechnical report-2025'04'07.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to WSP by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. WSP cannot be responsible for use of this report, or portions thereof, unless WSP is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without WSP's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, WSP may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of WSP's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that WSP interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: WSP will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of WSP's report. WSP should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of WSP's report.

During construction, WSP should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of WSP's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in WSP's report. Adequate field review, observation and testing during construction are necessary for WSP to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, WSP's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that WSP be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

FIGURES

Figure 1 - Borehole Location Plan

KEY MAP

SCALE 1:250,000

LEGEND

- BOREHOLE LOCATION
- BOREHOLE WITH VSP
- BOREHOLE WITH MONITORING WELL

PROPOSED DEVELOPMENT PLAN:

- NEW 1 LEVEL MATERIALS MANAGEMENT ADDITION
- NEW EMERGENCY DEPARTMENT ADDITION
- NEW PARKING GARAGE
- NEW ROAD
- RE-ALIGNED MATERIALS LOADING/UNLOADING AREA
- TEMPORARY LOADING DOCK
- URGENT CARE CENTRE ADDITION
- TOPOGRAPHIC CONTOUR 2 METRE
- OTHER SITE ELEMENTS

0 20 40 80

1:2,000 METRES

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. CONTAINS INFORMATION LICENSED UNDER THE OPEN GOVERNMENT LICENCE - ONTARIO
2. IMAGERY CREDITS: SOURCES: ESRI, HERE, GARMIN, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), (C) OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
3. COORDINATE SYSTEM: NAD 1983 UTM ZONE 18N

CLIENT

QUEENSWAY CARLETON HOSPITAL

PROJECT

GEOTECHNICAL INVESTIGATION
QUEENSWAY CARLETON HOSPITAL EXPANSION

TITLE

BOREHOLE LOCATION PLAN

CONSULTANT

YYYY-MM-DD	2024-08-02
DESIGNED	----
PREPARED	AS
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
CA0033714.1722	0001	A	1

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IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

25mm

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APPENDIX A

Borehole Logs

METHOD OF SOIL CLASSIFICATION

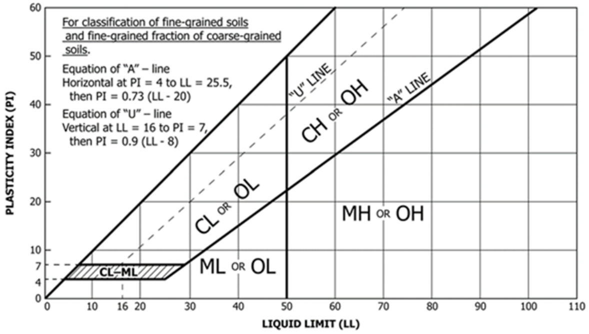
The WSP Canada Soil Classification[1] System is based on the Unified Soil Classification System (USCS) (after ASTM D2487)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10}xD_{60}}$	Organic Content [6,9]	USCS Group Symbol[3,5,7]	Primary Group Name[2]
INORGANIC (Organic Content <30% by mass)	**COARSE-GRAINED SOILS** (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Clean Gravels with <5% fines [3] (by mass)	Well Graded	≥4 (and)	≥1 to ≤3	≤30%	GW	Well-graded GRAVEL [4,6]
				Poorly Graded	<4 (and/or)	<1 or >3		GP	Poorly graded GRAVEL [4,6]
			Gravels with >12% fines [3] (by mass)	Below A Line	n/a			GM	SILTY GRAVEL [4,6]
				Above A Line	n/a			GC	CLAYEY GRAVEL [4,5,6]
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Clean Sands with <5% fines [7] (by mass)	Well Graded	≥6 (and)	≥1 to ≤3		SW	Well-graded SAND [6,8]
				Poorly Graded	<6 (and/or)	<1 or >3		SP	Poorly graded SAND [6,8]
			Sands with >12% fines [7] (by mass)	Below A Line	n/a			SM	SILTY SAND [6,8]
				Above A Line	n/a			SC	CLAYEY SAND [5,6,8]

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content [B,H]	USCS Group Symbol [A]	Primary Group Name [A]
				Dilatancy	Dry Strength	Shine Test	Thread Diameter (mm)	Toughness (of 3 mm thread)			
INORGANIC (Organic Content <30% by mass)	**FINE-GRAINED SOILS** (≥50% by mass is smaller than 0.075 mm)	SILTS (Nonplastic or PI and LL plot **below A-Line** on Plasticity Chart below) [C]	Liquid Limit <50 [D]	Rapid	None to Low	Dull to None	3 to >6	Low/can't roll 3 mm	<15%	ML	SILT [H]
				None to Slow	Low to Medium	Dull to Slight	3 to 6	Low	15% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50 [D]	None to V.Slow	Low to Medium	Slight	3 to 6	Low to Medium	<15%	MH	ELASTIC SILT[H]
				None	Medium to High	Dull to Slight	1 to 3	Low to Medium	15% to <30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot **above A-Line** on Plasticity Chart below) [A]	Liquid Limit <50 [D]	None to Medium Slow	Medium to High	Slight to Shiny	1 to 3	Medium	<15%	CL	LEAN CLAY [A,E,F,G,H]
				None to V.Slow	Medium to High	Slight to Shiny	1 to 3	Medium	15% to <30%	OL	ORGANIC CLAY [E,F,G]
			Liquid Limit ≥50 [D]	None	High to V.High	Shiny	<1	High	<15%	CH	FAT CLAY [E,F,G,H]
				None	High	Shiny	<1 to 1	High	15% to <30%	OH	ORGANIC CLAY [E,F,G]
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures	Relatively lightweight, possibly spongy. Some water may squeeze from sample. Some shrinkage may occur on air drying. Sand fraction may be visible. Low to high dilatancy. Thread weak near plastic limit. Low to medium dry strength.						30% to <75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat	Lightweight, spongy. Much water squeezes from sample. Shrinks considerably on air drying (i.e., very high water content). Plant structure identiable to altered.						75% to 100%		PEAT

Coarse-Grained Soil Note(s):

1. Based on the material passing the 75 mm sieve.
2. If field sample contains or drilling observations indicate cobbles or boulders or both, add, "with cobbles" or "with cobbles and boulders". Include notes on the depth(s) encountered, and sizes if possible.
3. Gravels with 5% to 12% fines require dual symbols:
   (GW-GM) Well-graded GRAVEL with silt,
   (GW-GC) Well-graded GRAVEL with clay,
   (GP-GM) Poorly graded GRAVEL with silt,
   (GP-GC) Poorly graded GRAVEL with clay.
4. If soil contains ≥15% sand, add "with sand" to Group Name.
5. If fines classify as CL-ML, use dual symbol (GC-GM) or (SC-SM) for Group Symbol.
6. If the soil has an organic content (OC) 15%≤OC<30% the prefix "Organic" should be added before the Group Name. If the soil has an organic content 3%≤OC<15% add "with organic fines" to Group Name. If the soil contains >0% to ≤3% organics, the descriptor "trace organics" may be added.
7. Sands with 5% to 12% fines require dual symbols:
   (SW-SM) Well-graded SAND with silt,
   (SW-SC) Well-graded SAND with clay,
   (SP-SM) Poorly graded SAND with silt,
   (SP-SC) Poorly graded SAND with clay.
8. If soil contains ≥15% gravel, add "with gravel" to Group Name.



Fine-Grained Soil Note(s):

A. If Atterberg limits plot above the A-line but in the 'hatched' area on the plasticity chart, soil is a (CL-ML) SILTY CLAY.
B. If the soil contains >0% to ≤3% organics, the descriptor "trace organics" may be added.
C. If fine-grained materials are nonplastic (i.e., a plastic limit (PL) cannot be measured), soil is a (ML) SILT.
D. If soil has a liquid limit (LL) >30% to <50%, the term 'medium plasticity' may be included in the description, but the Group Name/Symbol is not changed.
E. If soil contains 15% to <30% +No.200, add "with sand" or "with gravel".
F. If soil contains ≥30% +No.200 mainly sand, add "Sandy" to Group Name.
G. If soil contains ≥30% +No.200 mainly gravel, add "Gravelly" to Group Name.
H. If the soil has an organic content (OC) 3%≤OC<15% add "with organic fines" to Group Name.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

GRADATIONAL COMPONENT TERMS

% (by mass)	Term
≤ 5	Use “trace”
> 5 to ≤ 12	Use “few”
> 12 to <30	Use “little”
≥ 30 to <50	Use “some”
≥ 50	Use “mostly”

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); Nd:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven, pushed tube sampler, or geoprobe macro-core – note size
DS	Denison type sample
FS	Foil Sample
GS	Grab Sample
MC	Modified California Samples – note sample diameter and hammer weight
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split-spoon sampler (50 mm OD); larger sizes use MC
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT ‘N’ (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT ‘N’ in general accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT ‘N’ ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT ‘N’ value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT ‘N’ value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT ‘N’[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT ‘N’ in general accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT ‘N’ values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
NP	nonplastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

PROJECT: CA0033714.1722

LOCATION: N 5022052.54; E 358895.50

RECORD OF BOREHOLE: 24-01

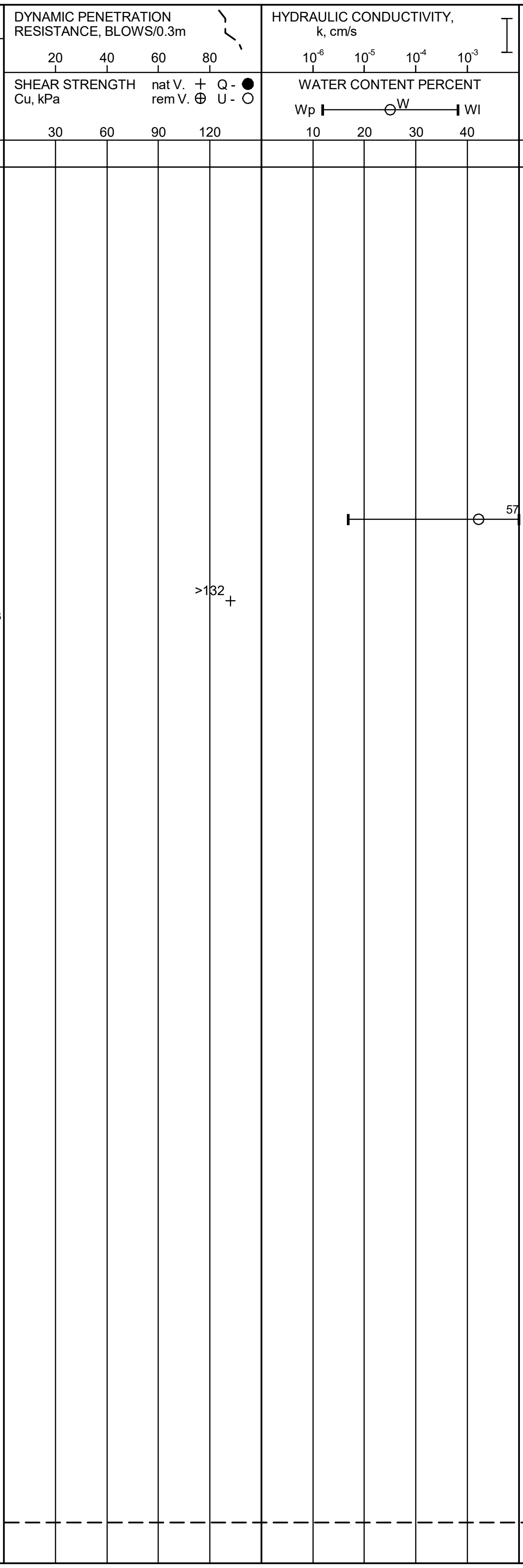
BORING DATE: July 17, 2024

DRILL RIG: Massenza MI3

SHEET 1 OF 2

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		77.62					
0	Washbore HW	TOPSOIL/FILL - (ML) Sandy SILT, some clay, organics; brown, rootlets; cohesive, moist, loose		0.00	1A	SS	5		
		FILL - (ML) CLAYEY Sandy SILT; brown, mottled; cohesive, w<PL, firm		0.15	1B				
1					2	SS	4		
		(CL/CH) CLAYEY SILT to SILTY CLAY, trace to some sand, trace gravel; brown, mottled, fissured; cohesive, w<PL to w>PL, soft to very stiff		76.25 1.37					
2					3	SS	4		
					4	SS	1		
3					5	SS	50/ 0.13		
		GLACIAL TILL with cobbles and boulders and/or WEATHERED BEDROCK		74.14 3.48					
4		BEDROCK		73.66 3.96					
5									
6									
7									
8									
9									
10		*CONTINUED NEXT PAGE*							



DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

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PROJECT: CA0033714.1722
LOCATION: N 5022052.54; E 358895.50

RECORD OF BOREHOLE: 24-01

BORING DATE: July 17, 2024
DRILL RIG: Massenza MI3

SHEET 2 OF 2
DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (30 60 90 120)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		BEDROCK	▨								
		END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-01.		67.23 10.39							
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-01

SHEET 1 OF 1

LOCATION: N 5022052.54 ;E 358895.50

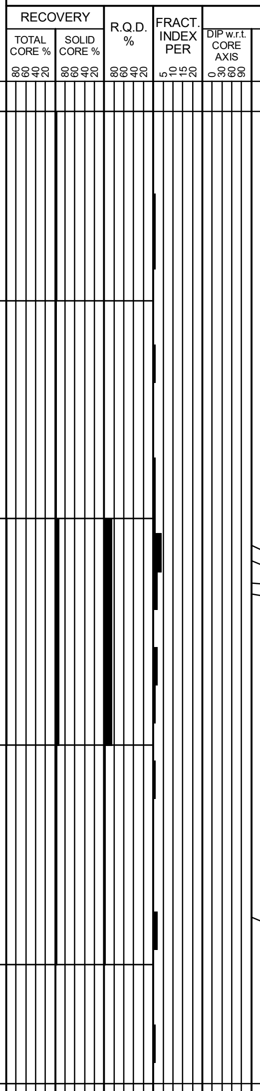
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 17, 2024
DRILL RIG: Massenza MI3
DRILLING CONTRACTOR: Strata

DATUM: Geodetic

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	FEATURES	PIEZOMETER
		Continued from Record of Borehole 24-01		73.66					
4		Fresh, bedded, grey, fine to medium grain, non-porous to slightly porous, medium strong to strong, DOLOSTONE, shale deds		3.96	1		BD,, BD,,		
5					2		BD,,		
6							BD,, BD,,		
7					3		BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,,		
8					4		BD,,		
9							BD,, BD,,		
10				67.23	5		BD,,		
		END OF DRILLHOLE		10.39					
11									
12									
13									



DEPTH SCALE
1 : 50

LOGGED:
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PROJECT: CA0033714.1722

LOCATION: N 5022039.10; E 358956.70

RECORD OF BOREHOLE: 24-02

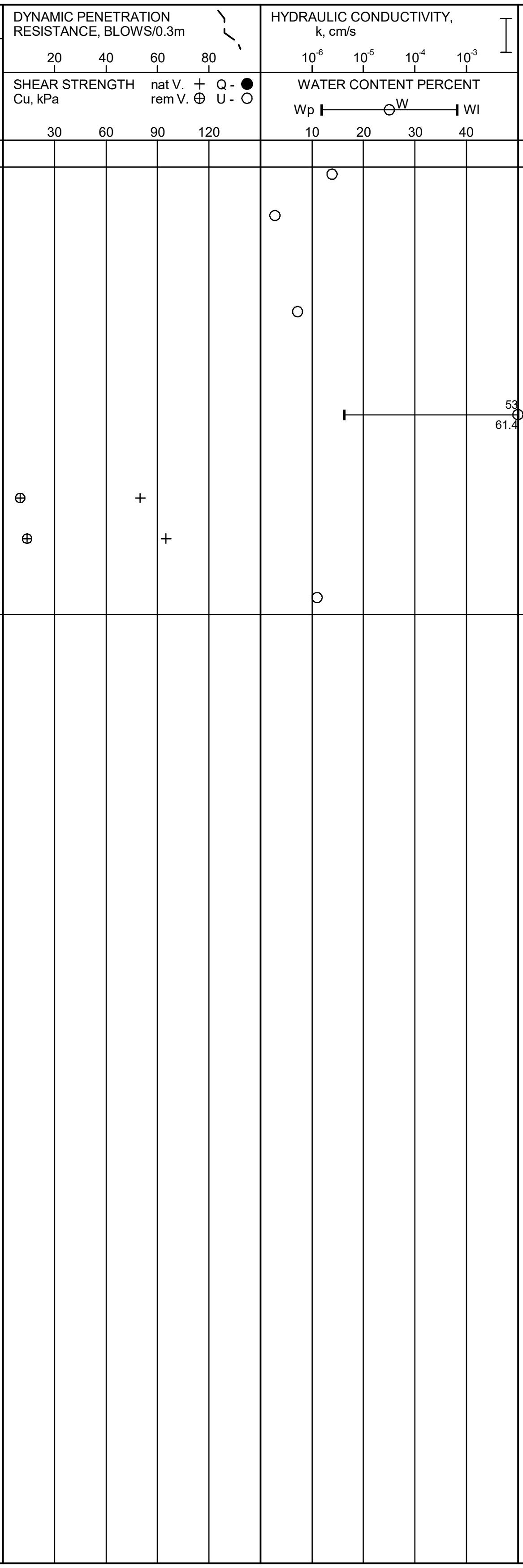
SHEET 1 OF 1

BORING DATE: July 8, 2024

DATUM: Geodetic

DRILL RIG: Massenza MI3

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		77.19					
0	Direct Push 108 mm I.D. DT45	TOPSOIL/FILL - (SW) SAND, some gravel, organics; dark brown; non-cohesive, moist, loose		0.00	1A	SS	21		
		FILL - (SW) gravelly SAND, some silt; brown grey; non-cohesive, moist, compact to loose		0.10	1B				
1		(CI/CH) SILTY CLAY, some sand, with sand seams; brown, mottled, fissured; cohesive, w~PL to w>PL, soft to stiff		76.12 1.07	2	SS	6		
2					3	SS	1		
3		(ML) Sandy SILT, some gravel, trace to some clay; brown-grey (GLACIAL TILL); non-cohesive, wet, compact		74.29 2.90	4	SS	50/ 0.10		
				73.89 3.30					
		END OF BOREHOLE							
4									
5									
6									
7									
8									
9									
10									



DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5022026.55; E 358915.00

RECORD OF BOREHOLE: 24-03

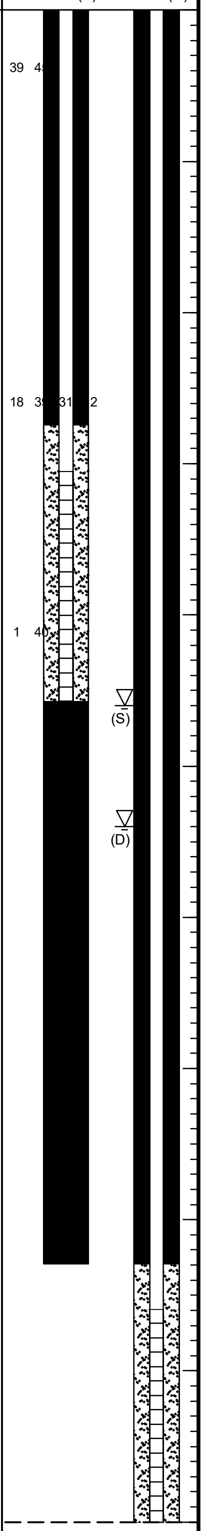
BORING DATE: July 8, 2024
DRILL RIG: Geoprobe 7822DT

SHEET 1 OF 2
DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
		GROUND SURFACE	77.42				
0	Direct Push 108 mm I.D. DT45	TOPSOIL/FILL - (SM) SILTY SAND, some gravel, trace clay, contains organics; dark brown; non-cohesive, moist, loose	0.00	1A	SS	20	
		FILL - (SW) SAND and GRAVEL, some silt; brown grey; non-cohesive, moist, compact	0.15	1B			39 4
1		(CL/CH) CLAYEY SILT / SILTY CLAY, some sand; brown, mottled, fissured; cohesive, w~PL, stiff to soft - Sand seams	76.81 0.61	2	SS	3	
2				3	SS	1	
		(SM) SILTY SAND, some gravel, some clay; brown (GLACIAL TILL); non-cohesive, moist to wet, loose to compact	75.13 2.29	4	SS	8	18 3 31 2
3				5	SS	14	
4		(SM) SILTY SAND, some clay, trace gravel; brown (GLACIAL TILL); non-cohesive, wet, loose - Contains cobbles and boulders	73.76 3.66	6	SS	4	1 40
		GLACIAL TILL with cobbles and boulders and/or WEATHERED BEDROCK	72.70 4.72	7	SS	50/ 0	
5		BEDROCK	72.37 5.05				
6							
7							
8							
9							
10							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 30 60 90 120
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION



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DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5022026.55; E 358915.00

RECORD OF BOREHOLE: 24-03

SHEET 2 OF 2

DATUM: Geodetic

BORING DATE: July 8, 2024

DRILL RIG: Geoprobe 7822DT

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 30 60 90 120	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									24-03 (S) 24-03 (D)
10				67.31 10.11							
		END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-03.									
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-03

SHEET 1 OF 1

LOCATION: N 5022026.55 ;E 358915.00

DRILLING DATE: July 9, 2024

DATUM: Geodetic

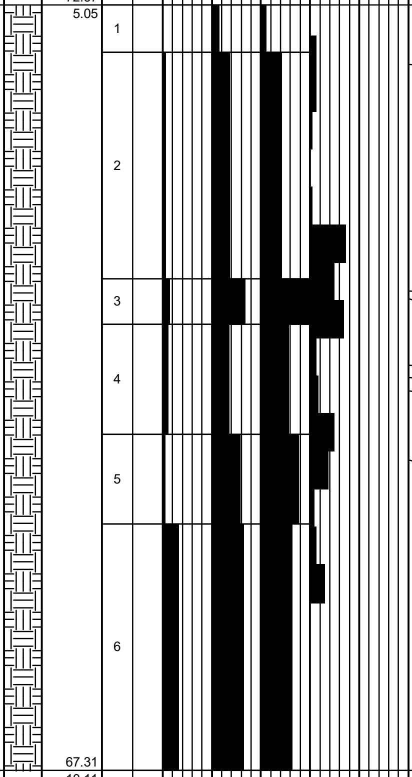
INCLINATION: -90° AZIMUTH: ---

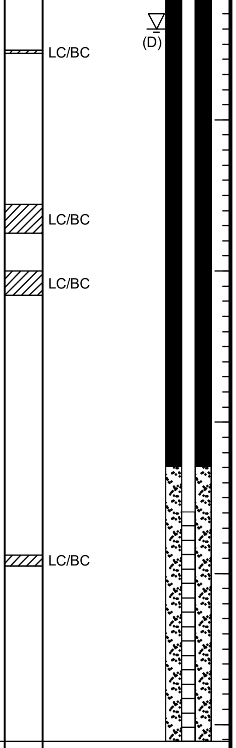
DRILL RIG: Geoprobe 7822DT

DRILLING CONTRACTOR: Strata

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE % / SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER (5 10 15 20)	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATH-ERING INDEX (W1 W2 W3 W4 W5 W6)	Diametral Point Load Index (MPa) (2 4 6)	FEATURES	PIEZOMETER 24-03 (S) / 24-03 (D)
		Continued from Record of Borehole 24-03		72.37														
		Fresh to moderately weathered, bedded, grey and light brown, fine to medium grain, non-porous to moderately porous, medium strong, DOLOSTONE and SANDSTONE UCS = 213 MPa		5.05	1													
					2						BD,, BD,, BD,, BD,,						LC/BC LC/BC	∇ (D)
6																		
7					3						BD,, FR,, BD,,						LC/BC	
					4						BD,, BD,, BD,, BD,, BD,,							
8					5						BD,, BD,, BD,, BD,, BD,,							
9					6						BD,, JN,, JN,,						LC/BC	
10				67.31 10.11							BD,,							
		END OF DRILLHOLE																
11																		
12																		
13																		
14																		
15																		





DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

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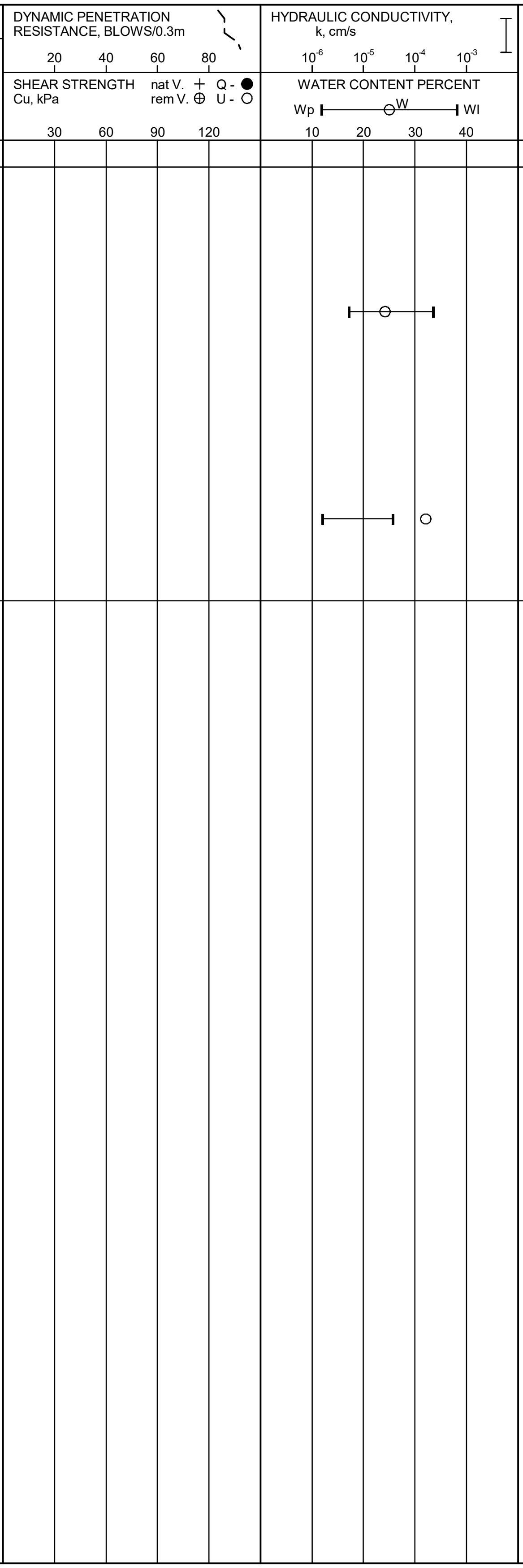
PROJECT: CA0033714.1722
LOCATION: N 5021961.02; E 358901.71

RECORD OF BOREHOLE: 24-04

BORING DATE: July 8, 2024
DRILL RIG: Geoprobe 7822DT

SHEET 1 OF 1
DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	75.81					
0	Direct Push 108 mm I.D. DT45	TOPSOIL/FILL - (CL/CH) SILTY CLAY, some gravel, trace sand, trace organics with rootlets; dark brown; cohesive, w<PL	0.00 0.08	1	SS	5		21 43
		FILL - (SP/SM) SILTY gravelly SAND, trace clay; brown grey; non-cohesive, moist, loose	75.20 0.61					
1		(CL/CI) CLAYEY SILT / SILTY CLAY, some sand; brown, mottled (WEATHERED CRUST); cohesive, w<PL, stiff to firm		2	SS	4		
				3A				
2		(CL/CI) SILTY CLAY, trace to some sand and gravel; brown-grey, some fissuring; cohesive, w~PL to w>PL, soft to firm	73.83 1.98	3B	SS	WH		
		- Grey		4	SS	WH		
3			72.61	5	SS	50/ 0		
		END OF BOREHOLE	3.20					
4								
5								
6								
7								
8								
9								
10								



DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5022029.18; E 359003.87

RECORD OF BOREHOLE: 24-05

SHEET 1 OF 1

DATUM: Geodetic

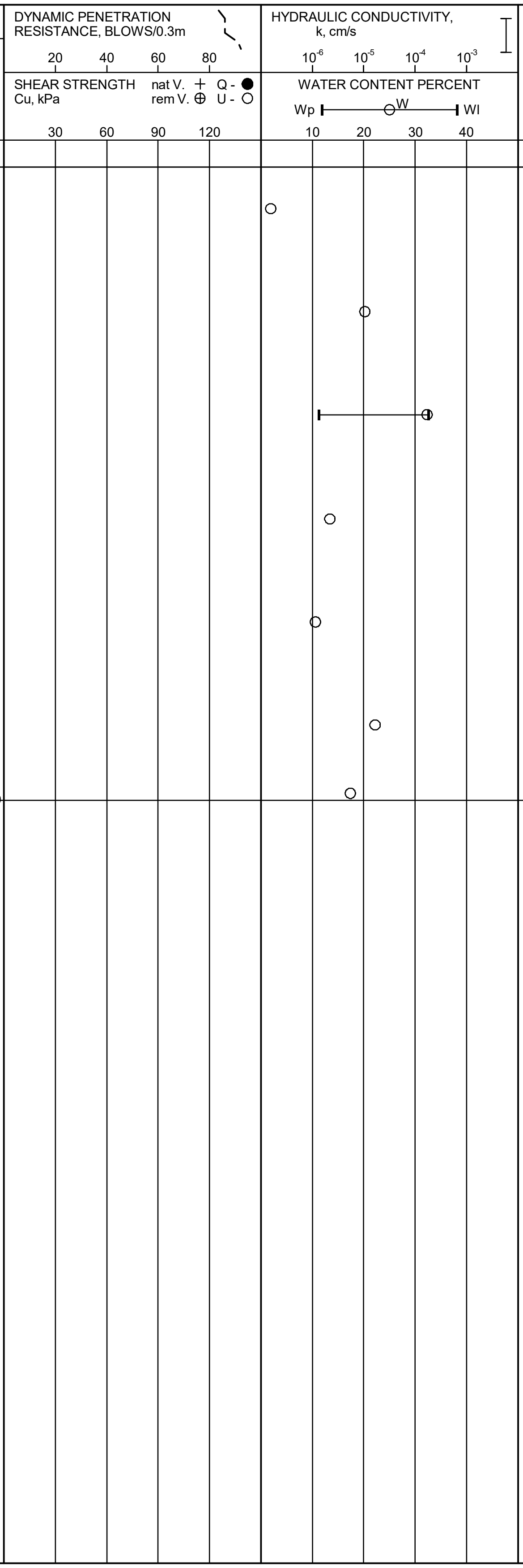
BORING DATE: July 8, 2024

DRILL RIG: Geoprobe 7822DT

BORING METHOD: Direct Push
108 mm I.D. DT45

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	77.55
ASPHALTIC CONCRETE (80 mm)	0.00
FILL - (SW) SAND and GRAVEL, trace silt; grey, crushed stone; (PAVEMENT STRUCTURE) non-cohesive, moist, compact	0.08
(CL/CI) CLAYEY SILT to SILTY CLAY, some sand; brown, some mottling (WEATHERED CRUST); cohesive, w<PL to w~PL, stiff	77.09 0.46
(SM/ML) Sandy SILT / SILTY SAND, fine layers of sandy silt, trace to some gravel, trace clay; brown grey, mottling present (GLACIAL TILL); non-cohesive, moist to wet, compact	75.26 2.29
END OF BOREHOLE	72.88 4.67

SAMPLES NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
1	SS	16	45 45
2	SS	6	
3	SS	4	
4	SS	10	7 31 55 7
5	SS	23	
6	SS	18	
7	SS	50/ 0.10	



ADDITIONAL LAB. TESTING

DEPTH SCALE METRES: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5022009.43; E 359052.81

RECORD OF BOREHOLE: 24-06

BORING DATE: July 18, 2024

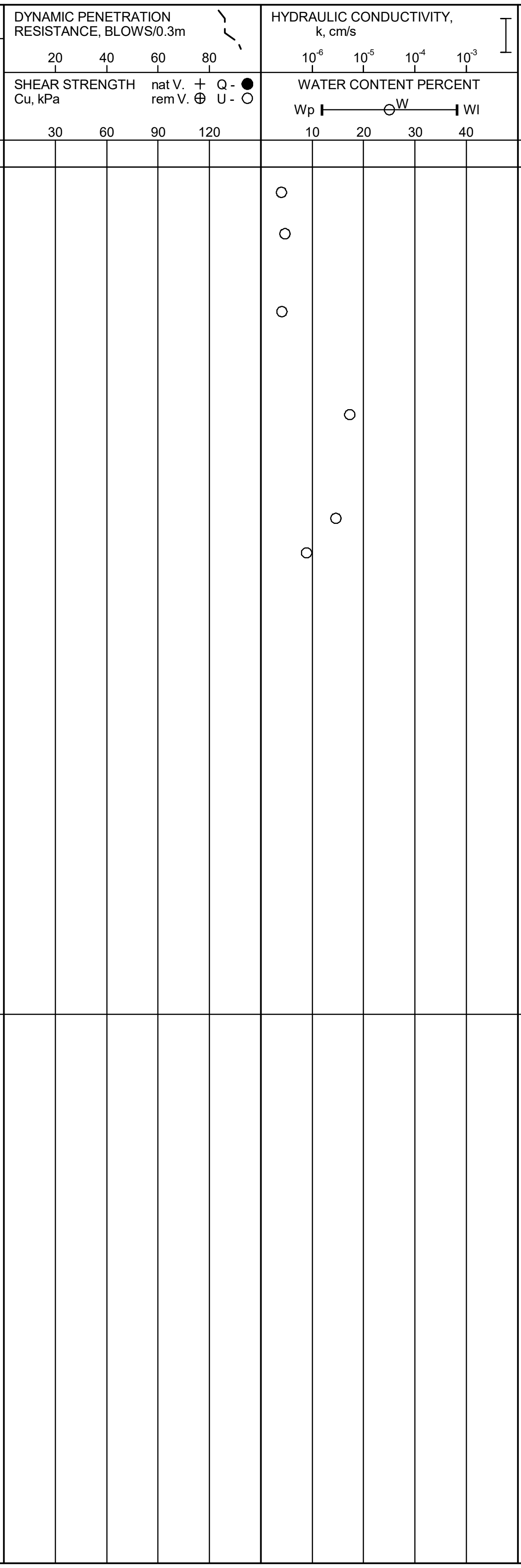
DRILL RIG: Massenza MI3

SHEET 1 OF 1

DATUM: Geodetic

BORING METHOD: Direct Push
108 mm I.D. DT45

DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	74.97				
CONCRETE (150 mm)	0.00	1A	SS	15	41 47 Benonite
FILL - (SP/GP) SAND and GRAVEL, some silt; grey brown; (PAVEMENT STRUCTURE) non-cohesive, moist	0.22	1B			
FILL - (SP/GP) gravelly SAND, some silt; brown; (PAVEMENT STRUCTURE) non-cohesive, moist, compact		2	SS	14	35 53 Sand
(SM) SILTY SAND, trace clay, trace gravel; grey (GLACIAL TILL); non-cohesive, moist, compact - Wet	73.45 1.52	3	SS	17	3 65 Aug. 7, 2024 Screen
		4A	SS	7	
(ML) gravelly Sandy SILT, some clay; grey, contains cobbles and boulders, contains rock fragments (GLACIAL TILL)	72.18 2.79 2.90	4B			Benonite
GLACIAL TILL with cobbles and boulders and/or WEATHERED BEDROCK	71.81 3.16				
BEDROCK					
END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-06.	68.72 6.25				



DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-06

SHEET 1 OF 1

LOCATION: N 5022009.43 ;E 359052.81

DRILLING DATE: July 18, 2024

DATUM: Geodetic

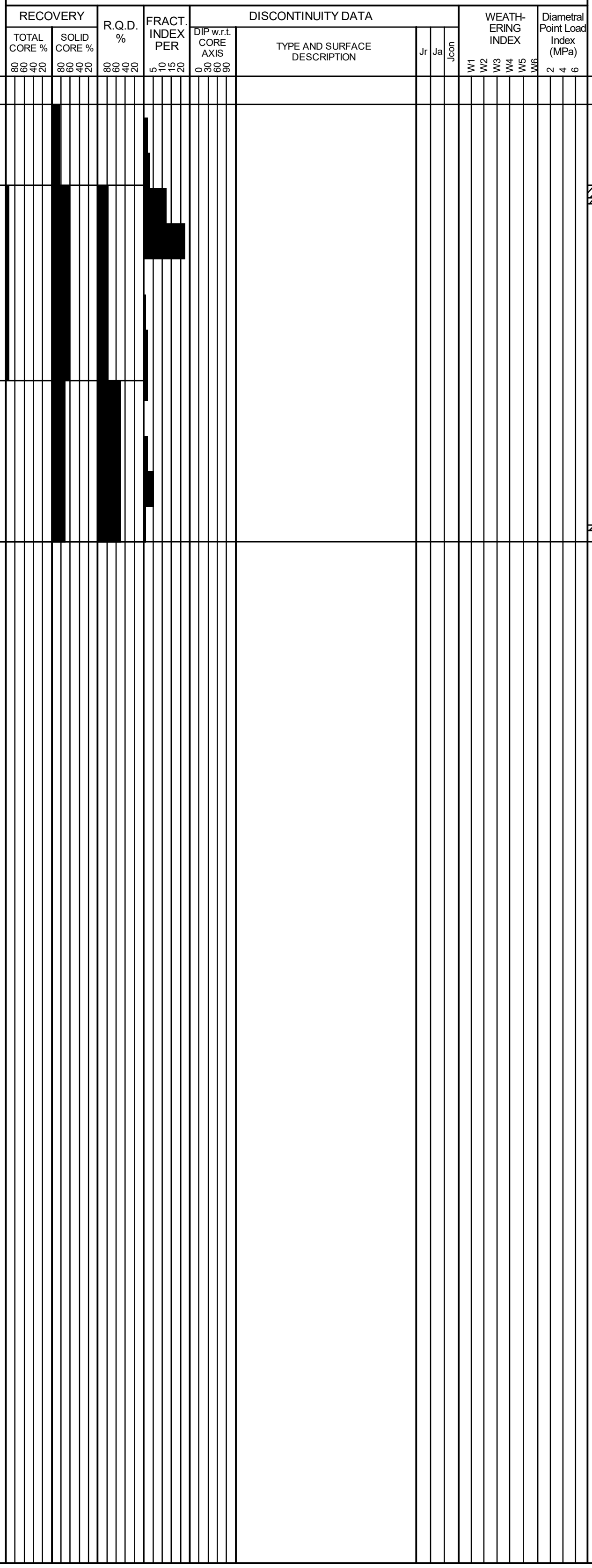
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Massenza MI3

DRILLING CONTRACTOR: Strata

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	FEATURES	PIEZOMETER
		Continued from Record of Borehole 24-06		71.81				
		Fresh, bedded, grey, fine to medium grain, non-porous to slightly porous, DOLOSTONE		3.16	1			
4					2		LC/BC LC/BC	
		UCS = 204 MPa						
5								
6					3		LC/BC	
				68.72				
		END OF DRILLHOLE		6.25				
7								
8								
9								
10								
11								
12								
13								



DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5021990.83; E 359040.51

RECORD OF BOREHOLE: 24-07

BORING DATE: July 10, 2024

DRILL RIG: Geoprobe 7822DT

SHEET 1 OF 1

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 30 60 90 120	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		GROUND SURFACE		74.94							
0	Direct Push 108 mm I.D. DT45	ASPHALTIC CONCRETE (100 mm)		0.00							
		FILL - (SW) gravelly SAND, some silt; grey, crushed stone; (PAVEMENT STRUCTURE) non-cohesive, moist, compact		0.10	1	SS	24				
		(SM/SP) SILTY gravelly SAND, trace clay; (GLACIAL TILL); non-cohesive, wet, very loose to compact		74.64 0.30							
1					2	SS	2				23 41 26 10
				72.98 1.96	3	SS	16				
2		END OF BOREHOLE									
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5021965.10; E 359017.53

RECORD OF BOREHOLE: 24-08

SHEET 1 OF 1

DATUM: Geodetic

BORING DATE: July 10, 2024

DRILL RIG: Geoprobe 7822DT

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (30 60 90 120)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		75.77							
0	Direct Push 108 mm I.D. DT45	ASPHALTIC CONCRETE (130 mm)		0.00							
		FILL - (SW) gravelly SAND, some silt; grey to grey-brown; (PAVEMENT STRUCTURE) non-cohesive, moist, dense		0.13	1	SS	30		○		28 53
		(SM/SP/GP) SILTY sandy GRAVEL, trace clay; grey-brown (GLACIAL TILL); non-cohesive, wet, loose to compact		75.16 0.61							
1					2	SS	9		○		33 31 31 5
					3	SS	50/ 0.05		○		
		END OF BOREHOLE		74.04 1.73							
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5021951.42; E 359055.52

RECORD OF BOREHOLE: 24-09

BORING DATE: July 10, 2024

DRILL RIG: Geoprobe 7822DT

SHEET 1 OF 1

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 30 60 90 120	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.96							
0	Direct Push 108 mm I.D. DT45	ASPHALTIC CONCRETE (80 mm)		0.00							
		FILL - (SW) gravelly SAND, some silt; grey, crushed stone; (PAVEMENT STRUCTURE) non-cohesive, moist, dense		0.08	1	SS	47				
1		(ML/SM) Sandy SILT to SILTY SAND, some gravel, some clay; grey (GLACIAL TILL); non-cohesive, moist to wet, compact to dense		74.05 0.91	2	SS	17				
					3	SS	50/ 0.13				13 29
2		END OF BOREHOLE		73.13 1.83							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

GTA-BHS 001 S:\CLIENTS\QUEENSWAY_CARLETON_HOSPITAL\QUEENSWAY_CARLETON_HOSPITAL_OTTAWA\02_DATA\GINT\QUEENSWAY_CARLETON_HOSPITAL_OTTAWA.GPJ GAL-MIS.GDT 12/6/24

PROJECT: CA0033714.1722

LOCATION: N 5021862.35; E 358938.20

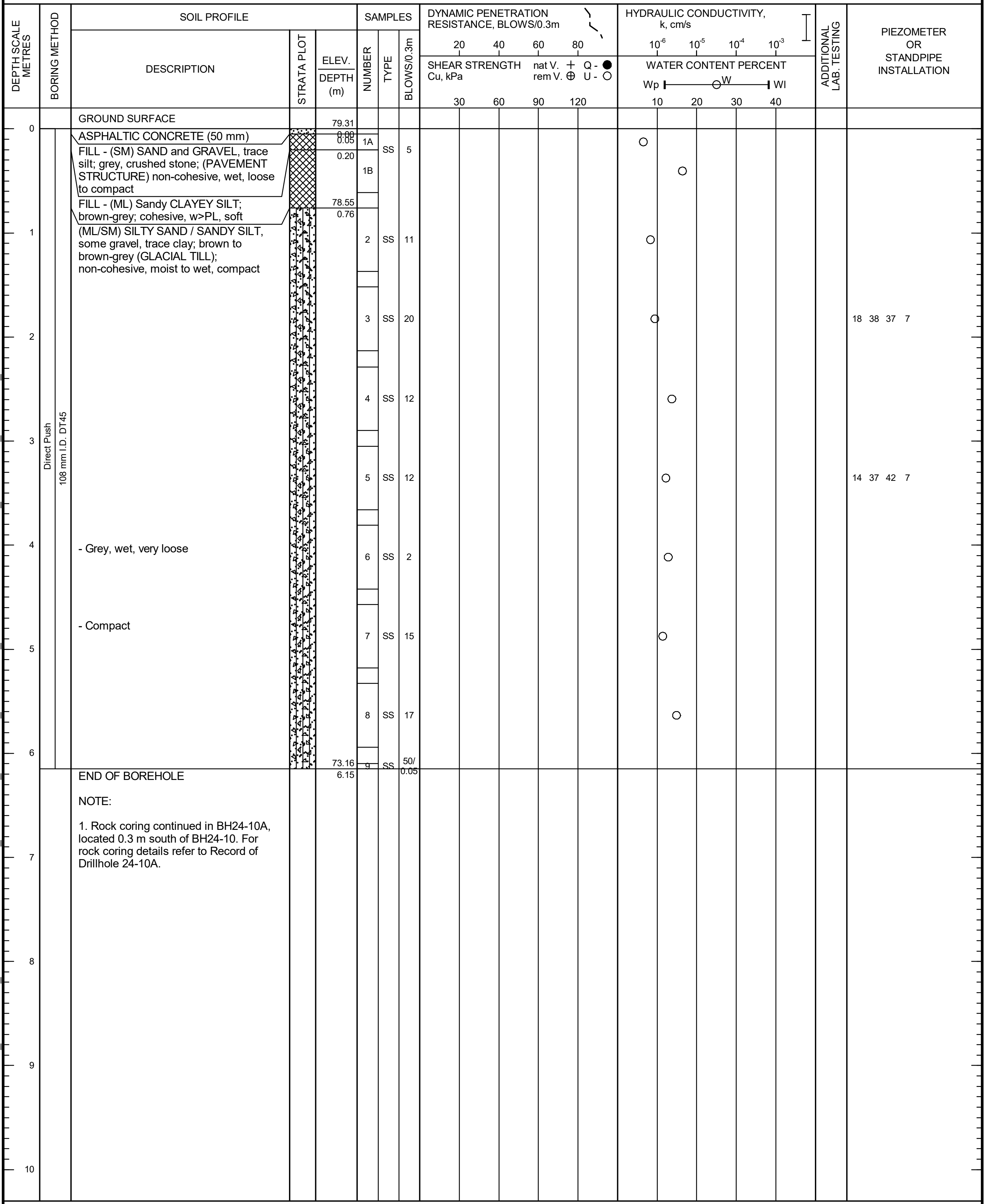
RECORD OF BOREHOLE: 24-10

BORING DATE: July 11, 2024

DRILL RIG: Geoprobe 7822DT

SHEET 1 OF 1

DATUM: Geodetic



BORING METHOD: Direct Push, 108 mm I.D. DT45

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	79.31				
0	ASPHALTIC CONCRETE (50 mm)	0.00				
	FILL - (SM) SAND and GRAVEL, trace silt; grey, crushed stone; (PAVEMENT STRUCTURE) non-cohesive, wet, loose to compact	0.05	1A	SS	5	
	FILL - (ML) Sandy CLAYEY SILT; brown-grey; cohesive, w>PL, soft	0.20	1B			
	(ML/SM) SILTY SAND / SANDY SILT, some gravel, trace clay; brown to brown-grey (GLACIAL TILL); non-cohesive, moist to wet, compact	78.55 0.76				
1			2	SS	11	
2			3	SS	20	18 38 37 7
3			4	SS	12	
			5	SS	12	14 37 42 7
4	- Grey, wet, very loose		6	SS	2	
5	- Compact		7	SS	15	
			8	SS	17	
6		73.16	9	SS	50/ 0.05	
	END OF BOREHOLE	6.15				

DEPTH SCALE: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

NOTE:

1. Rock coring continued in BH24-10A, located 0.3 m south of BH24-10. For rock coring details refer to Record of Drillhole 24-10A.

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-10A

SHEET 1 OF 1

LOCATION: N ;E

DRILLING DATE: July 11, 2024

DATUM: Geodetic

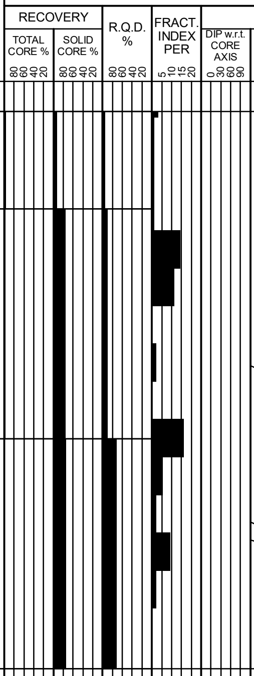
INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Strata

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	FEATURES	PIEZOMETER
		Continued from Record of Borehole 24-10										
		Fresh, bedded, grey, fine to medium grain, non-porous to slightly porous, medium strong, DOLOSTONE, shale beds		73.09 6.22	1		BD,, BD,,					
7 8					2		BD,, BD,, BD,, BD,,				LC/BC	
9					3		BD,, BD,, BD,, BD,, BD,, BD,,				LC/BC LC/BC	
10 11 12 13 14 15 16		END OF DRILLHOLE		69.41 9.90								



DISCONTINUITY DATA

WEATHERING INDEX: W1 W2 W3 W4 W5 W6

Diametral Point Load Index (MPa): 2 4 6

GTA-RCK 046 S:\CLIENTS\QUEENSWAY_CARLETON_HOSPITAL\QUEENSWAY_CARLETON_HOSPITAL_OTTAWA\02_DATA\GINT\QUEENSWAY_CARLETON_HOSPITAL_OTTAWA.GPJ GAL-MISS.GDT 12/6/24

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5021942.96; E 359030.46

RECORD OF BOREHOLE: 24-11

BORING DATE: July 10, 2024
DRILL RIG: Geoprobe 7822DT

SHEET 1 OF 1
DATUM: Geodetic

BORING METHOD: Direct Push 108 mm I.D. DT45

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 30 60 90 120
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 10 20 30 40

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	75.12					
0	ASPHALTIC CONCRETE (80 mm)	0.00					
	FILL - (SW) gravelly SAND, some silt; grey, crushed stone; (PAVEMENT STRUCTURE) non-cohesive, moist, dense	0.08	1A	SS	33		26 56
	FILL - (SP/SW) gravelly SAND, some silt; brown; (PAVEMENT STRUCTURE) non-cohesive, moist, dense to compact	74.82 0.30	1B				
1			2	SS	23		21 64
	(SM) SILTY SAND, some gravel; grey with rock fragments (GLACIAL TILL); non-cohesive, moist to wet, compact	73.44 1.68	3A 3B	SS	50/ 0		
2	BEDROCK	1.83					
5	END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-11.	69.86 5.26					

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-11

SHEET 1 OF 1

LOCATION: N 5021942.96 ;E 359030.46

DRILLING DATE: July 10, 2024

DATUM: Geodetic

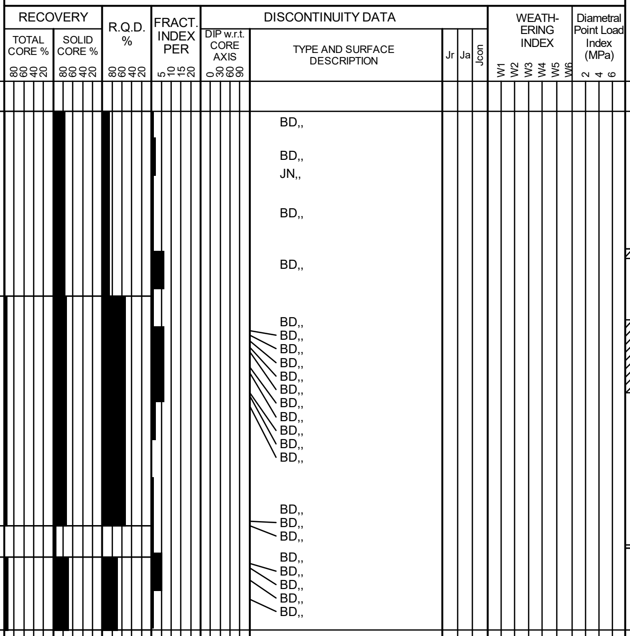
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Geoprobe 7822DT

DRILLING CONTRACTOR: Strata

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	FEATURES
	Continued from Record of Borehole 24-11	73.29			
2	Fresh, bedded, grey, fine to medium grain, non-porous to slightly porous, medium strong to strong, DOLOSTONE, trace shale	1.83	1	BD,, BD,, JN,, BD,,	
	UCS = 279 MPa			BD,,	LC/BC
3			2	BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,,	LC/BC
4				BD,, BD,, BD,,	
			3		LC/BC
5		69.86	4	BD,, BD,, BD,, BD,, BD,,	
	END OF DRILLHOLE	5.26			
6					
7					
8					
9					
10					
11					



DRILLING RECORD | SYMBOLIC LOG | FLUSH RETURN | PIEZOMETER

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5021978.11; E 359120.94

RECORD OF BOREHOLE: 24-12

BORING DATE: July 18, 2024

DRILL RIG: Massenza MI3

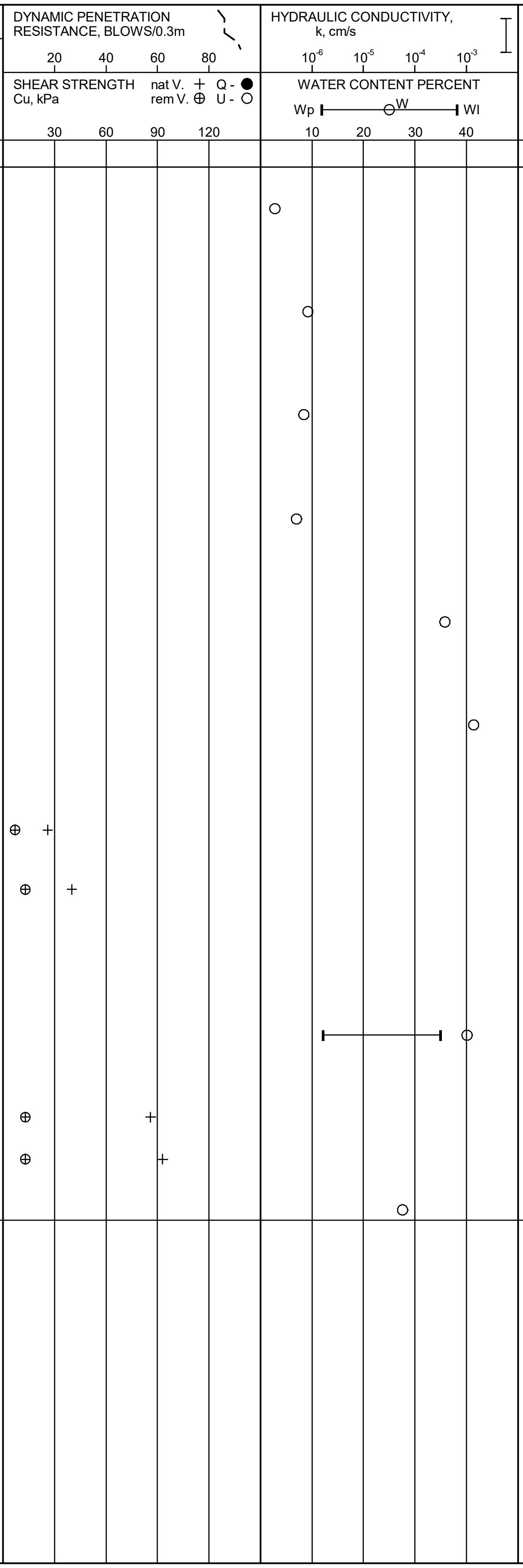
SHEET 1 OF 1

DATUM: Geodetic

BORING METHOD: Direct Push
108 mm I.D. DT45

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	78.85
ASPHALTIC CONCRETE (80 mm)	0.00
FILL - (SM) SAND and GRAVEL; grey-brown; (PAVEMENT STRUCTURE) non-cohesive, wet, dense	0.15
FILL - (SM/SP) SILTY SAND, some gravel; brown; non-cohesive, moist, compact	78.24 0.61
FILL - (ML) gravelly Sandy SILT, some clay; non-cohesive, moist, compact	77.33 1.52
FILL - (SP/SM) gravelly SILTY SAND, trace clay; brown; non-cohesive, moist, dense	76.56 2.29
(CL/CI) CLAYEY SILT to SILTY CLAY, some sand; grey; cohesive, moist, firm to soft	75.80 3.05
(CL/CI) SILTY CLAY, trace to some sand; grey; cohesive, wet, soft to stiff	73.07 5.78
END OF BOREHOLE	71.08 7.77

SAMPLES NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
1	SS	36	
2	SS	19	16 54
3	SS	11	
4	SS	33	27 42 23 8
5	SS	4	
6	SS	1	
7	SS	WH	
8	SS	WH	



DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5022009.72; E 359133.76

RECORD OF BOREHOLE: 24-13

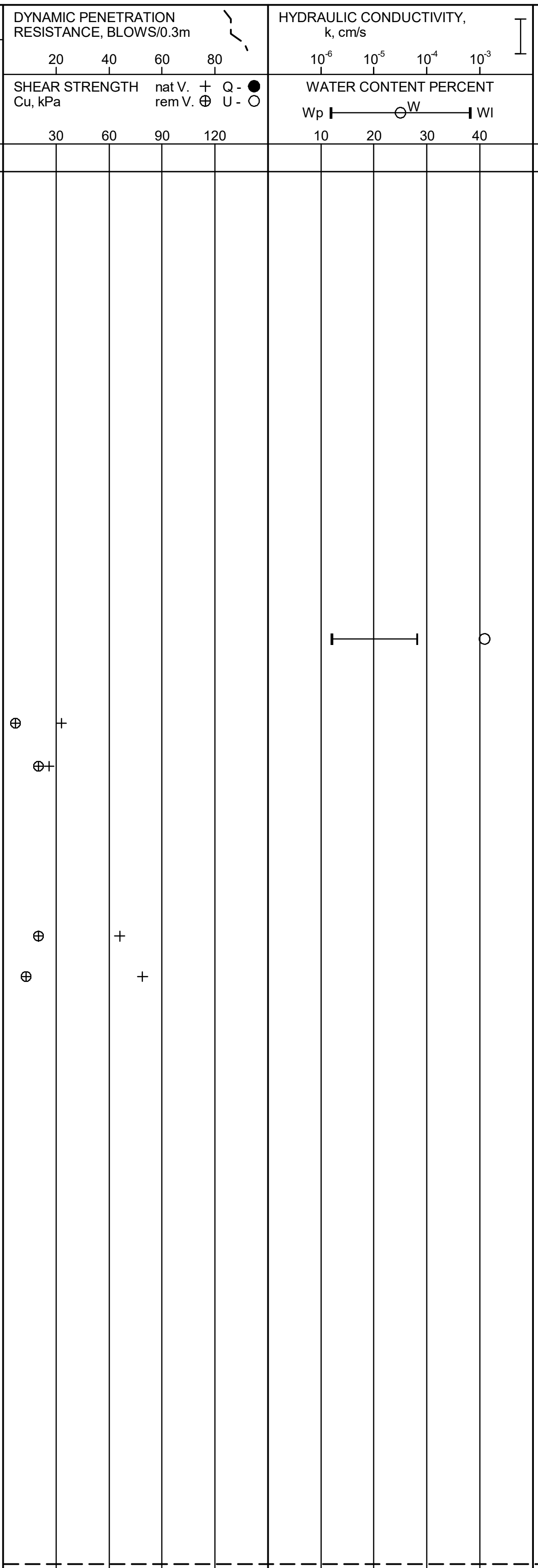
BORING DATE: July 19, 2024
DRILL RIG: Geoprobe 7822DT

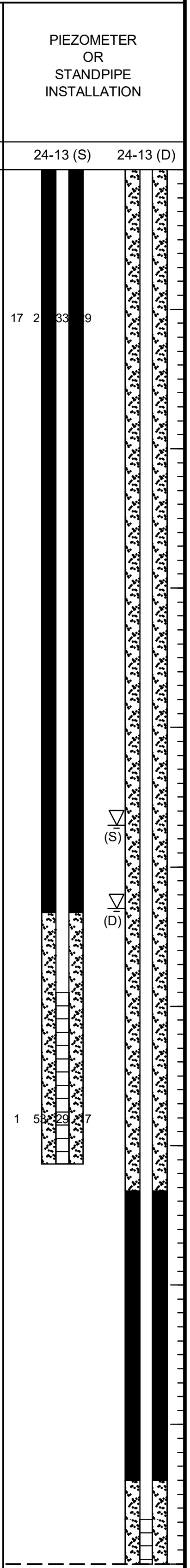
SHEET 1 OF 2
DATUM: Geodetic

BORING METHOD: Washbore HW

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	77.48
TOPSOIL (130 mm)	0.00
FILL - (SP) SAND, some silt, contains rootlets; grey brown; non-cohesive, very loose	0.13
FILL - (SM/ML) SILTY SAND to Sandy SILT, some clay with clay seams, some gravel, contains organics; grey-brown; non-cohesive, moist, very loose to loose	76.87 0.61
(ML/CL) Sandy SILT to CLAYEY SILT, trace clay, trace sand; grey with red mottling; cohesive, w<PL to w>PL, soft	75.19 2.29
(CL/CI) CLAYEY SILT, trace sand; grey-brown to grey; cohesive, w>PL, soft to stiff	74.43 3.05
(ML) CLAYEY SILT to Sandy SILT, some sand; grey; cohesive, w>PL, soft	71.38 6.10
(SM) SILTY SAND, some clay, trace gravel; grey (GLACIAL TILL); non-cohesive, wet, very dense	70.77 6.71
GLACIAL TILL with cobbles and boulders and/or WEATHERED BEDROCK	70.57 6.91
BEDROCK	70.32 7.16

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1A	SS	3
1B		
2	SS	4
3	SS	3
4	SS	WH
5	SS	WH
6	SS	WH
7	SS	WH
8	SS	>50





CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5022009.72; E 359133.76

RECORD OF BOREHOLE: 24-13

BORING DATE: July 19, 2024

DRILL RIG: Geoprobe 7822DT

SHEET 2 OF 2

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (30, 60, 90, 120) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10, 20, 30, 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									24-13 (S) 24-13 (D)
10		BEDROCK									
11				66.28 11.20							
		END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-13.									
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-13

SHEET 1 OF 1

LOCATION: N 5022009.72 ;E 359133.76

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 19, 2024

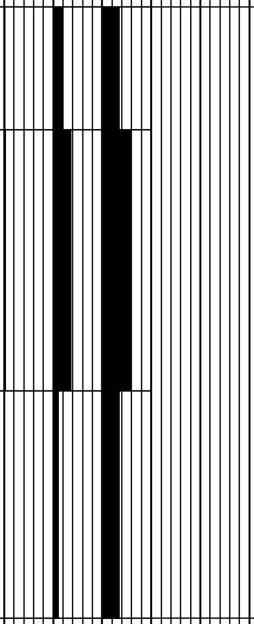
DRILL RIG: Geoprobe 7822DT

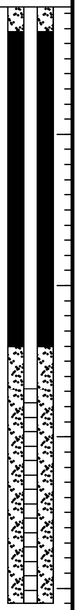
DRILLING CONTRACTOR: Strata

DATUM: Geodetic

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER (5 10 15 20)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATH-ERING INDEX (W1 W2 W3 W4 W5 W6)	Diametral Point Load Index (MPa) (2 4 6)	FEATURES	PIEZOMETER 24-13 (S)	PIEZOMETER 24-13 (D)
		Continued from Record of Borehole 24-13		70.32																
8 9 10 11		Fresh, bedded, grey, fine to medium grain, slightly porous, medium strong DOLOSTONE and SANDSTONE UCS = 120 MPa		7.16 66.28	1 2 3															
		END OF DRILLHOLE		11.20																





Depth scale marks: 8, 9, 10, 11, 12, 13, 14, 15, 16, 17

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5021983.09; E 359139.42

RECORD OF BOREHOLE: 24-14

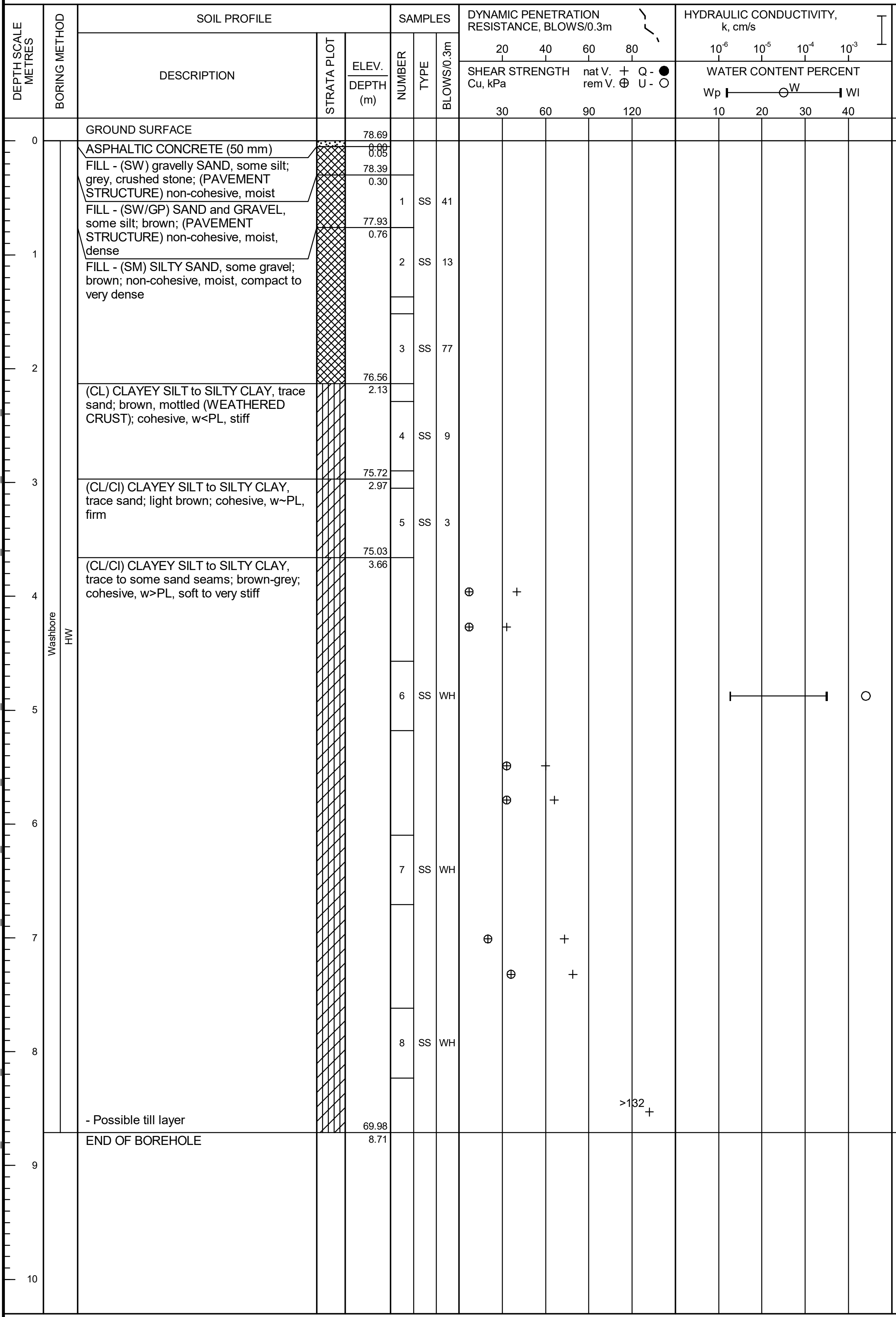
SHEET 1 OF 1
DATUM: Geodetic

BORING DATE: July 16, 2024
DRILL RIG: Massenza MI3

BORING METHOD: Washbore HW

DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)
GROUND SURFACE		78.69
ASPHALTIC CONCRETE (50 mm)		0.00
FILL - (SW) gravelly SAND, some silt; grey, crushed stone; (PAVEMENT STRUCTURE) non-cohesive, moist	▨	0.05
FILL - (SW/GP) SAND and GRAVEL, some silt; brown; (PAVEMENT STRUCTURE) non-cohesive, moist, dense	▨	78.39 0.30
FILL - (SM) SILTY SAND, some gravel; brown; non-cohesive, moist, compact to very dense	▨	77.93 0.76
(CL) CLAYEY SILT to SILTY CLAY, trace sand; brown, mottled (WEATHERED CRUST); cohesive, w<PL, stiff	▨	76.56 2.13
(CL/CI) CLAYEY SILT to SILTY CLAY, trace sand; light brown; cohesive, w~PL, firm	▨	75.72 2.97
(CL/CI) CLAYEY SILT to SILTY CLAY, trace to some sand seams; brown-grey; cohesive, w>PL, soft to very stiff - Possible till layer	▨	75.03 3.66
END OF BOREHOLE		69.98 8.71

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1	SS	41		38 45
2	SS	13		
3	SS	77		
4	SS	9		
5	SS	3		
6	SS	WH		
7	SS	WH		
8	SS	WH		



DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5021898.23; E 359154.04

RECORD OF BOREHOLE: 24-15

BORING DATE: July 15, 2024
DRILL RIG: Massenza MI3

SHEET 1 OF 2
DATUM: Geodetic

BORING METHOD: Washbore HW

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
GROUND SURFACE	78.89				
FILL - (SM) SAND, some organics; dark brown; non-cohesive, wet, compact	0.00				
FILL - (SW/GP) SAND and GRAVEL, some silt; grey to brown-grey; non-cohesive, moist, compact	0.05	1	SS	20	38 47
FILL - (ML/CL) CLAYEY SILT / SILTY CLAY, some sand, trace gravel; grey, slightly mottled and fissured; cohesive, w~PL, firm	77.98 0.91	2	SS	12	
(CL/CI) CLAYEY SILT / SILTY CLAY, trace sand, trace gravel, some sand seams; brown-grey; cohesive, w~PL to w>PL, stiff to soft	77.06 1.83	3	SS	7	
		4	SS	1	
		5	SS	2	
		6	SS	1	
(CL/CI) CLAYEY SILT / SILTY CLAY, trace sand; brown-grey; cohesive, w>PL, soft to very stiff	73.71 5.18	7	SS	WH	
		8	SS	WH	
- Grey - Sand seams, wet					
		9	SS	1	
- Possible till layer, wet and loose					
	68.94	10	SS	50/	

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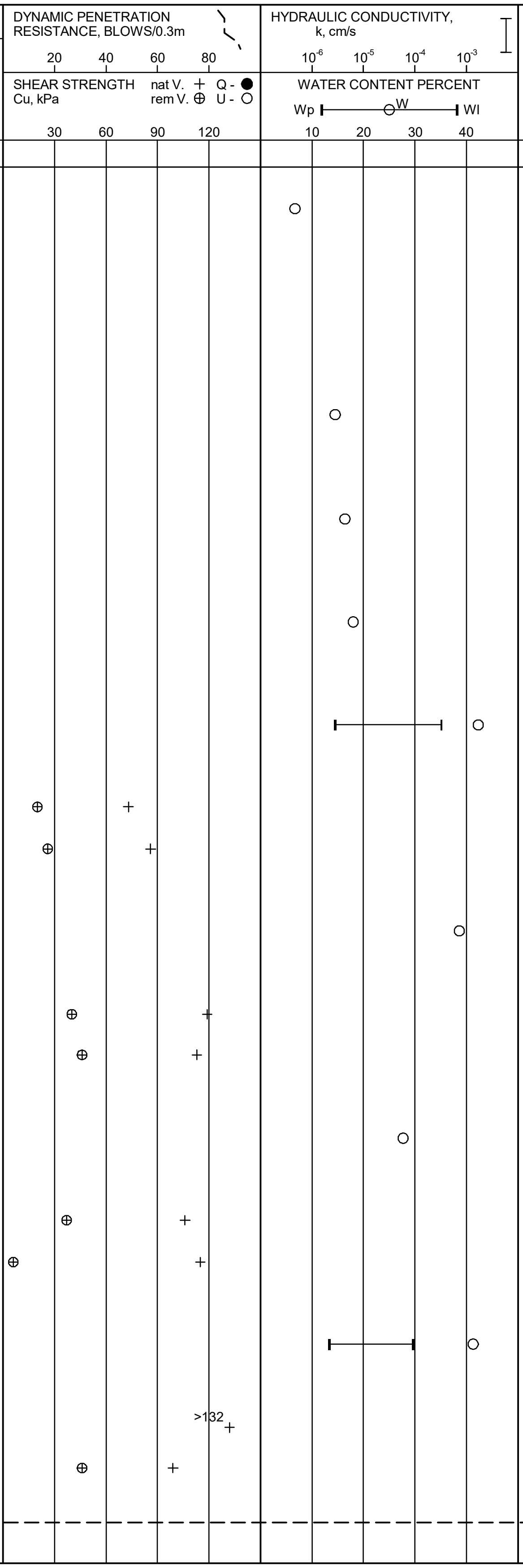


PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

PROJECT: CA0033714.1722

LOCATION: N 5021898.23; E 359154.04

RECORD OF BOREHOLE: 24-15

BORING DATE: July 15, 2024

DRILL RIG: Massenza MI3

SHEET 2 OF 2

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (30 60 90 120)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		BEDROCK		9.95			0.05				
11											
12											
13											
14											
15											
16		END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-15.		62.94 15.95							
17											
18											
19											
20											

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722

RECORD OF DRILLHOLE: 24-15

SHEET 1 OF 1

LOCATION: N 5021898.23 ;E 359154.04

DRILLING DATE: July 15, 2024

DATUM: Geodetic

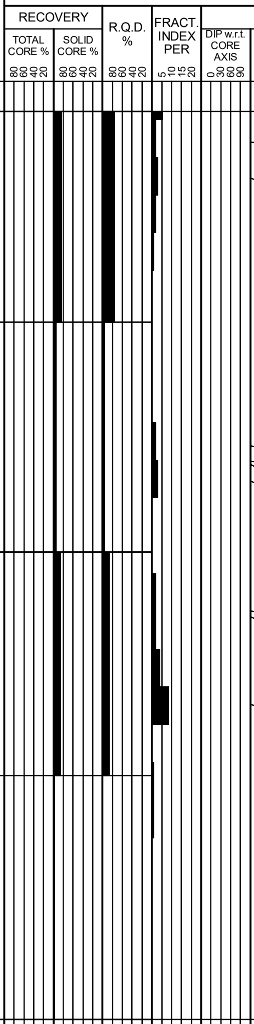
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Massenza MI3

DRILLING CONTRACTOR: Strata

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	FEATURES
	Continued from Record of Borehole 24-15	68.94 9.95			
10	Fresh, bedded, grey, fine to medium grain, non-porous to slightly porous, medium strong to strong, DOLOSTONE with shale beds		1	BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,,	LC/BC
11	UCS = 120 MPa				
12			2	BD,, BD,, BD,, BD,, BD,,	
13			3	BD,, BD,, BD,, BD,, BD,, BD,, BD,,	LC/BC
14–15		62.94 15.95	4	BD,, BD,,	
16	END OF DRILLHOLE				
17					
18					
19					



Jr Ja Jcon

WEATH-ERING INDEX W1 W2 W3 W4 W5 W6

Diametral Point Load Index (MPa) 2 4 6

SYMBOLIC LOG

FLUSH RETURN

DRILLING RECORD

PIEZOMETER

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

PROJECT: CA0033714.1722
LOCATION: N 5021739.24; E 358968.59

RECORD OF BOREHOLE: 24-16

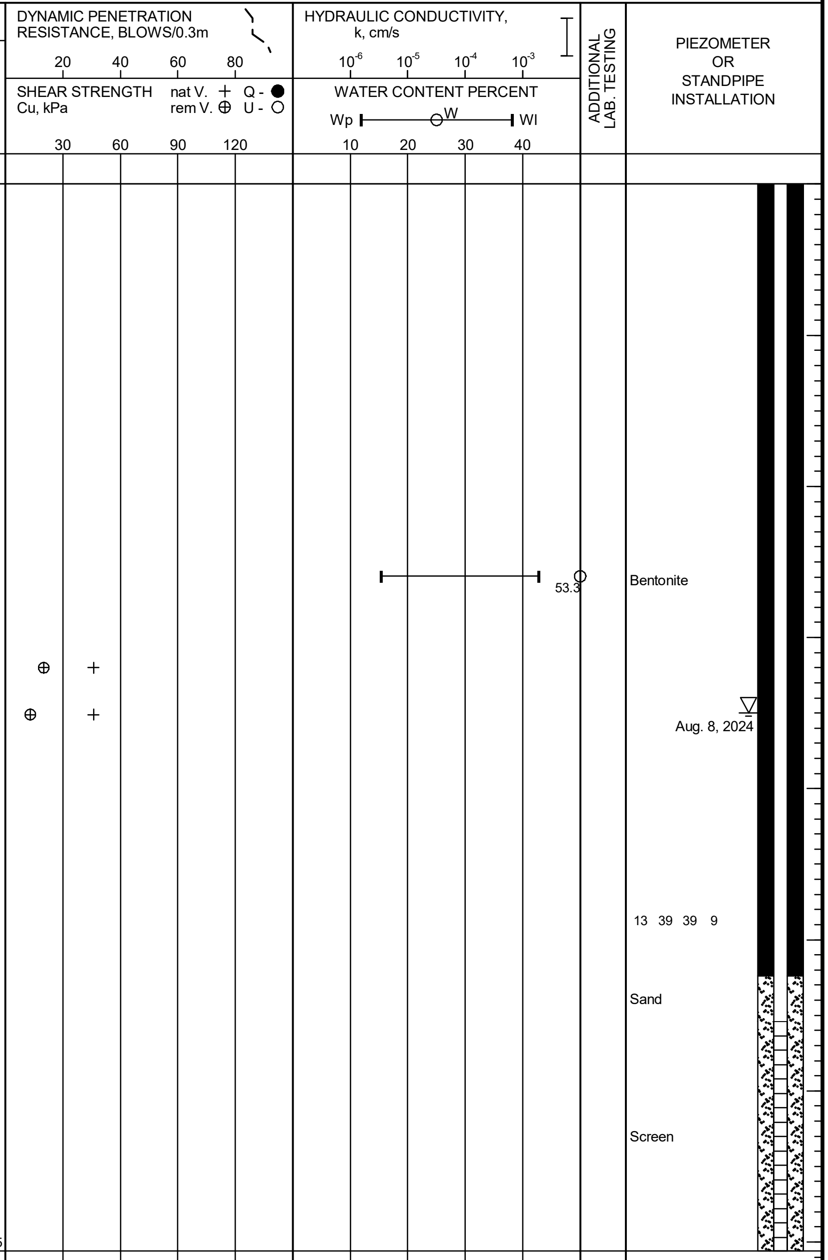
SHEET 1 OF 1
DATUM: Geodetic

BORING DATE: July 17, 2024
DRILL RIG: Massenza MI3

BORING METHOD: Washbore HW

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	79.71
FILL - (SW) SAND, some silt, some gravel, organics; brown; non-cohesive, moist, loose	0.00
FILL - (ML/CL) CLAYEY SILT / SILTY CLAY, some sand; dark brown, mottled, fissured; cohesive, w<PL, stiff	0.15
(CL) CLAYEY SILT / SILTY CLAY, some sand; brown, mottled, fissured (WEATHERED CRUST); cohesive, w<PL, stiff	79.10 0.61
(CL//CI) SILTY CLAY / CLAYEY SILT, trace sand; brown-grey; cohesive, w>PL, firm to soft	77.42 2.29
(SL/ML) SILTY SAND / Sandy SILT, some gravel, trace clay; grey (GLACIAL TILL); non-cohesive, wet, very loose to loose	76.05 3.66
(ML) Sandy SILT, some clay, some gravel; grey (GLACIAL TILL); non-cohesive, wet, loose	74.53 5.18
END OF BOREHOLE	72.65 7.06

NUMBER	TYPE	BLOWS/0.3m
1A	SS	6
1B		
2	SS	4
3	SS	2
4	SS	WH
5	SS	5
6	SS	WH
7	SS	5
8	SS	9
9	SS	50/ 0.05



DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

APPENDIX B

Rock Core Photos

24-01 (Dry)
Core Box 1 of 1

Elevation 73.7 m Top of Bedrock

Elevation 67.2 m End of Corehole

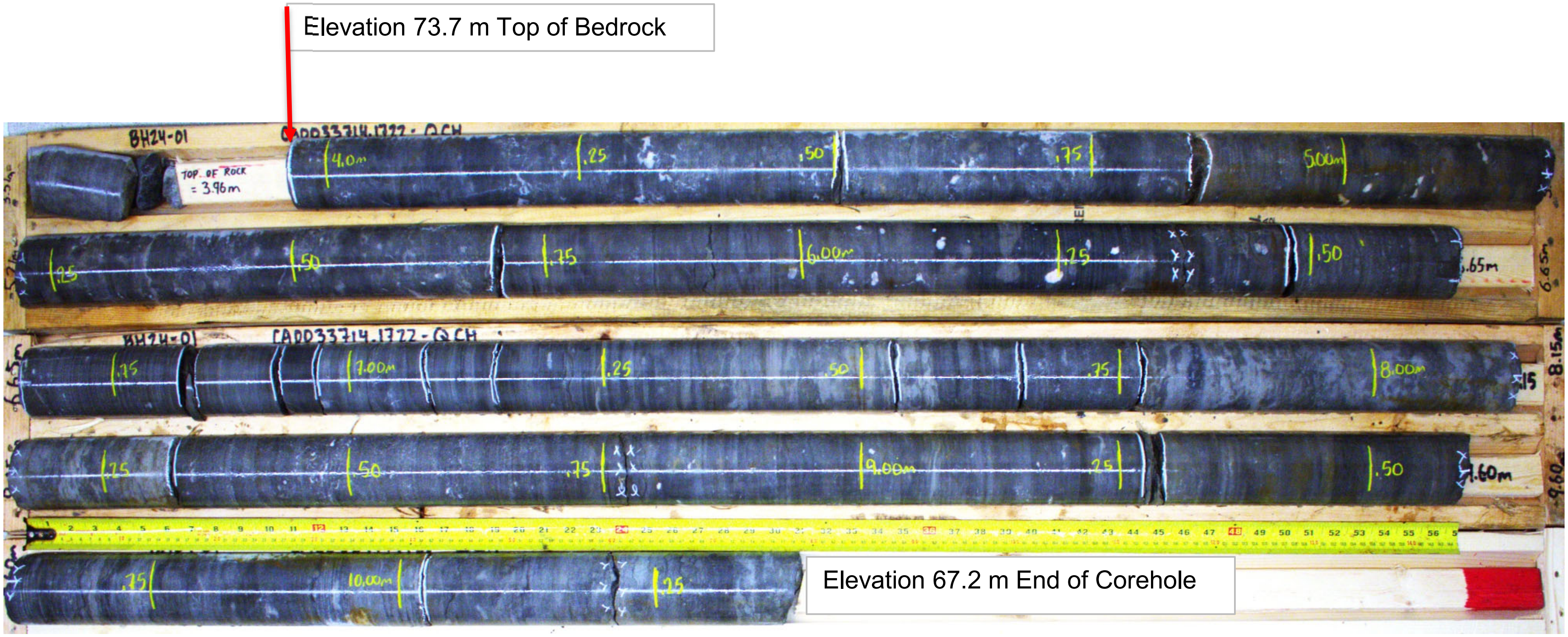
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A1

24-01 (Wet)
Core Box 1 of 1


Elevation 73.7 m Top of Bedrock

Elevation 67.2 m End of Corehole

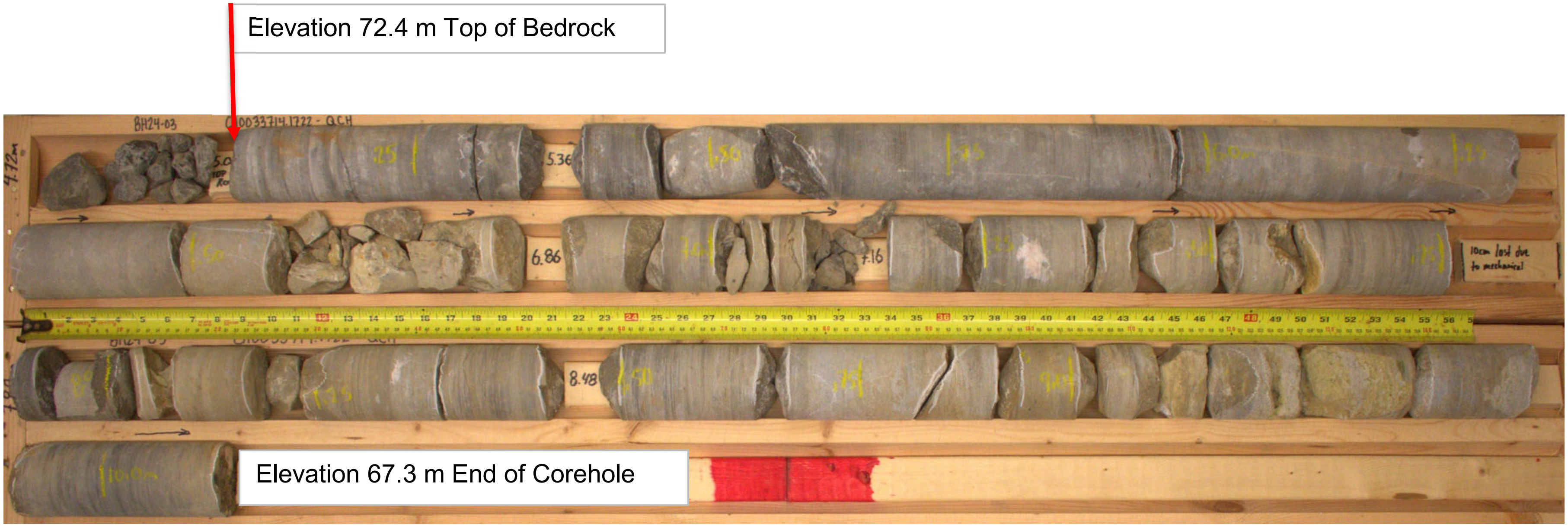
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A2

24-03 (Dry)
Core Box 1 of 1



Elevation 72.4 m Top of Bedrock

Elevation 67.3 m End of Corehole

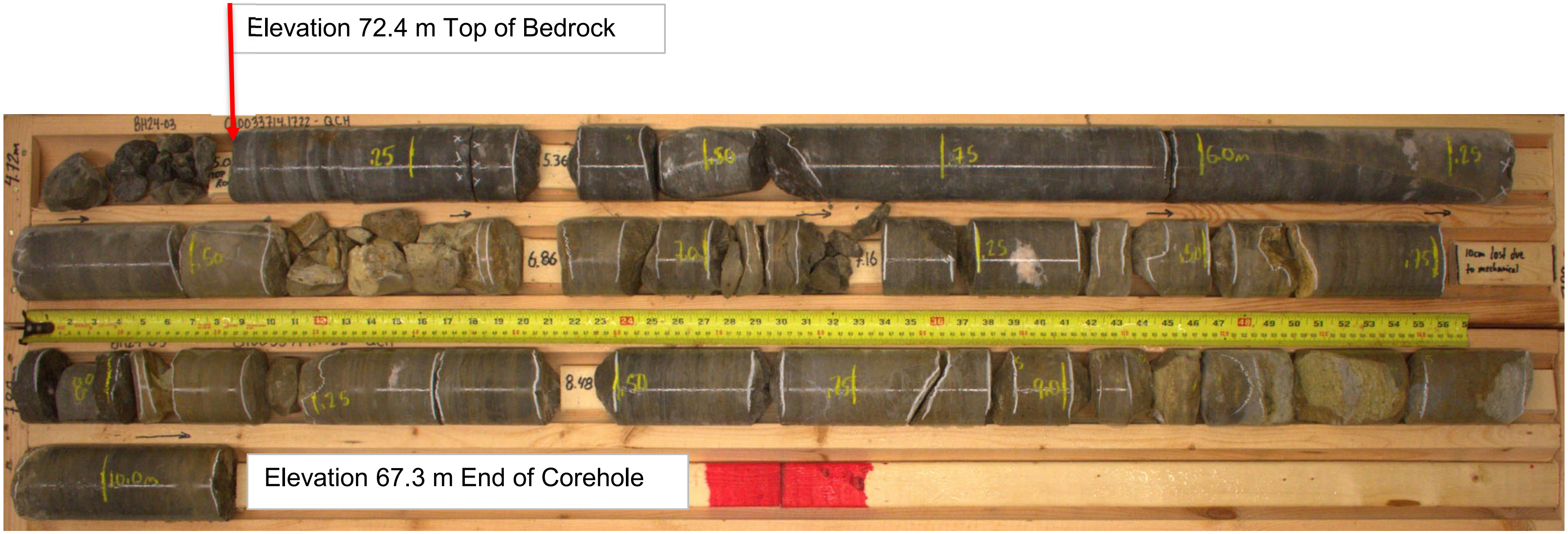
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A3

24-03 (Wet)
Core Box 1 of 1



Elevation 72.4 m Top of Bedrock

Elevation 67.3 m End of Corehole

Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
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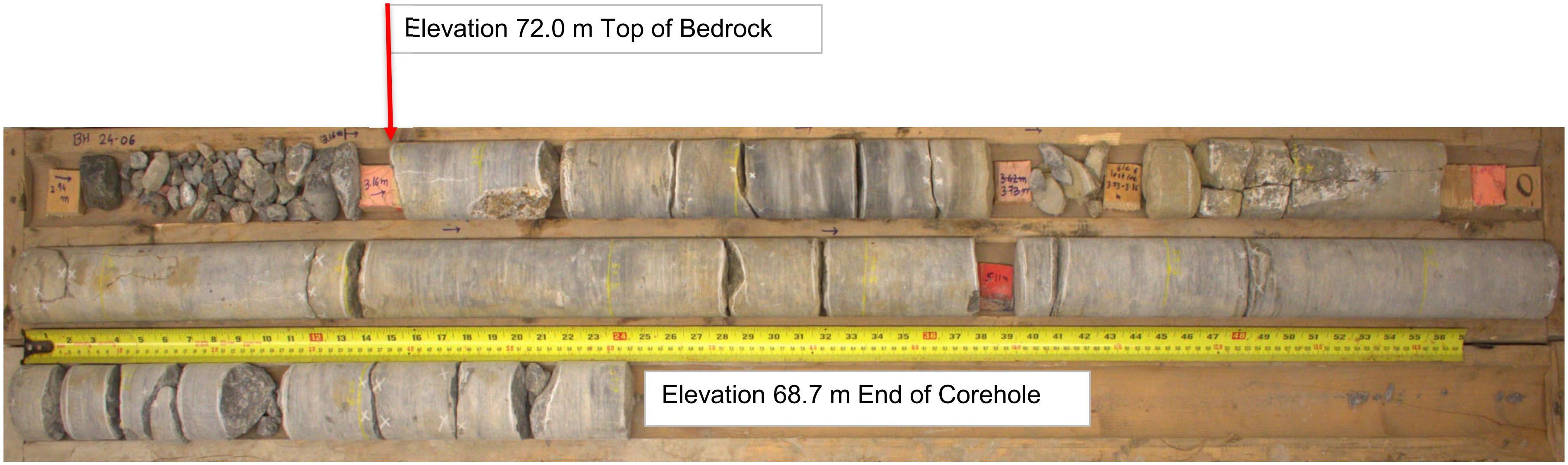
Figure A4

24-06 (Dry)
Core Box 1 of 1

Elevation 72.0 m Top of Bedrock

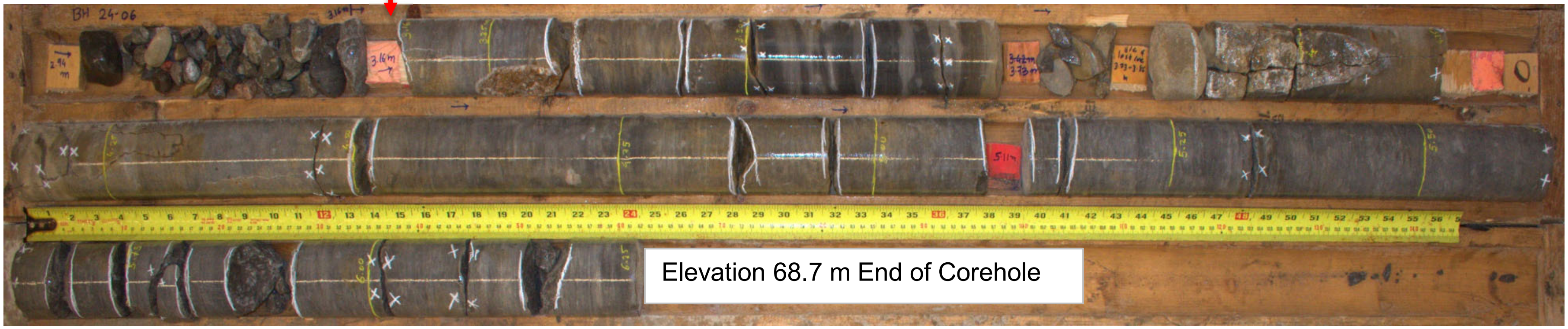


Elevation 68.7 m End of Corehole

Geotechnical Investigation Queensway Carleton Hospital Expansion 3045 Baseline Rd, Ottawa, ON K2H 8P4	Project No. CA0033714.1722 Drawn: BW Date: 2024-07-23 Checked: OB Review: JSA

Figure A5

24-06 (Wet)
Core Box 1 of 1



Elevation 72.0 m Top of Bedrock

Elevation 68.7 m End of Corehole

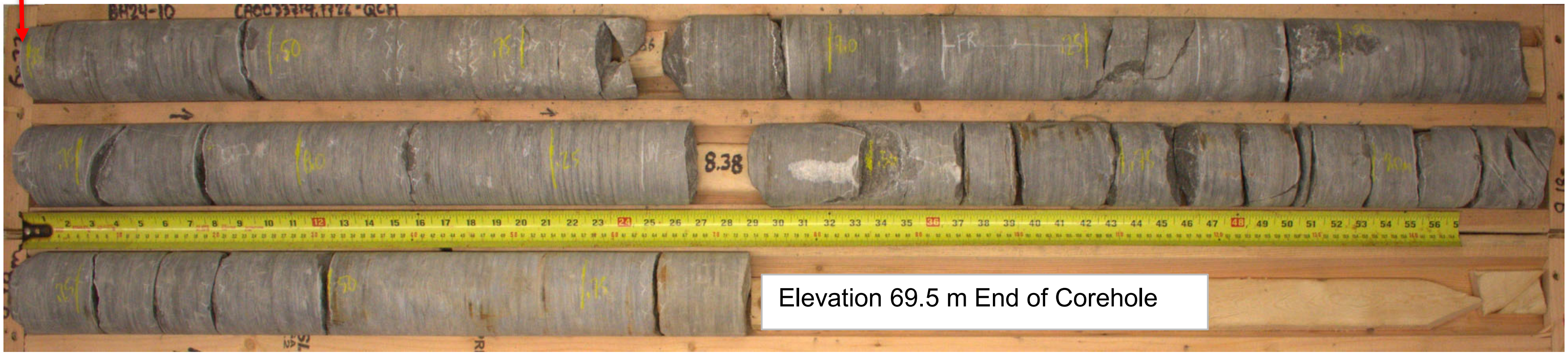
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No.	CA0033714.1722
Drawn:	BW
Date:	2024-07-23
Checked:	OB
Review:	JSA

Figure A6

24-10A (Dry)
Core Box 1 of 1



Elevation 73.2 m Top of Bedrock

Elevation 69.5 m End of Corehole

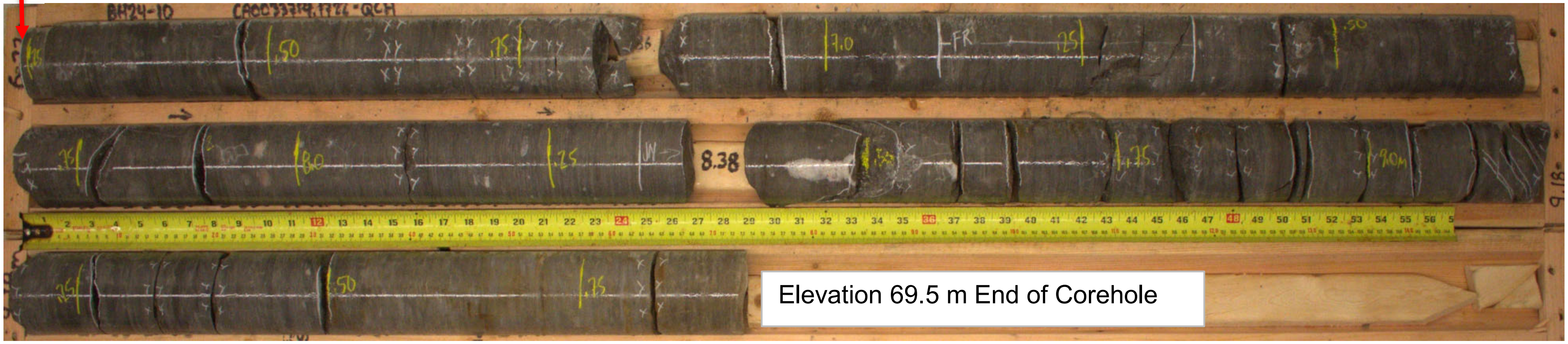
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A7

24-10A (Wet)
Core Box 1 of 1


Elevation 73.2 m Top of Bedrock

Elevation 69.5 m End of Corehole

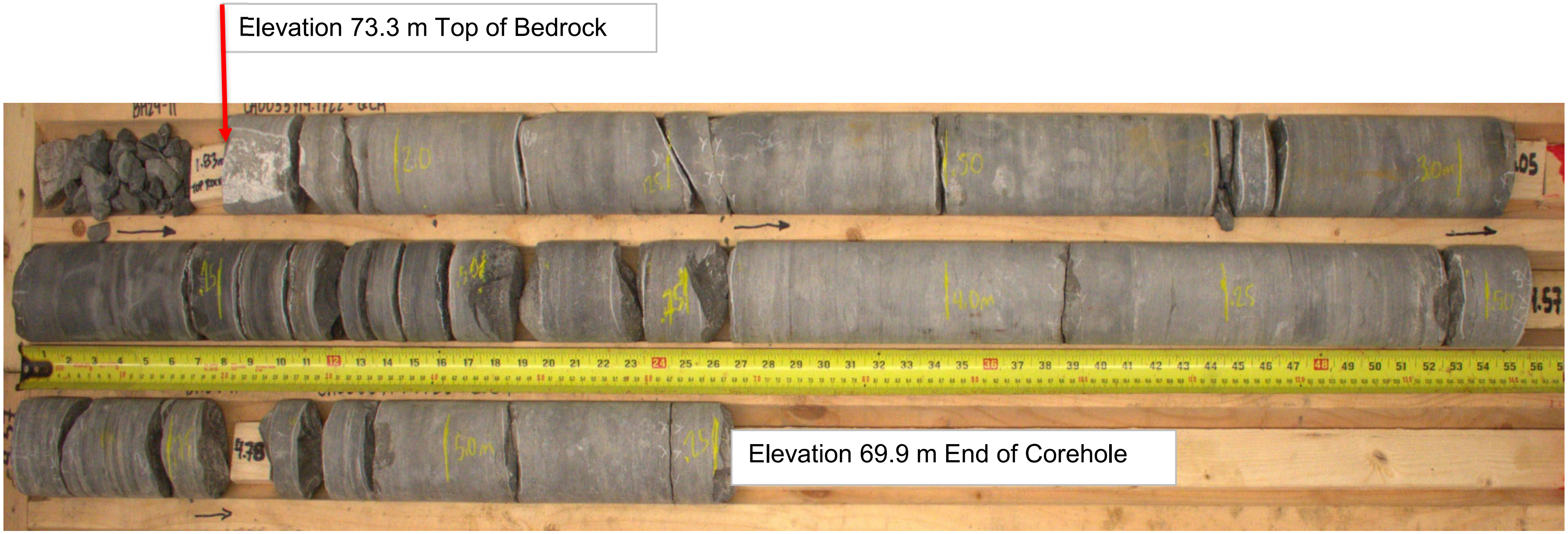
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A8

24-11 (Dry)
Core Box 1 of 1



Elevation 73.3 m Top of Bedrock

Elevation 69.9 m End of Corehole

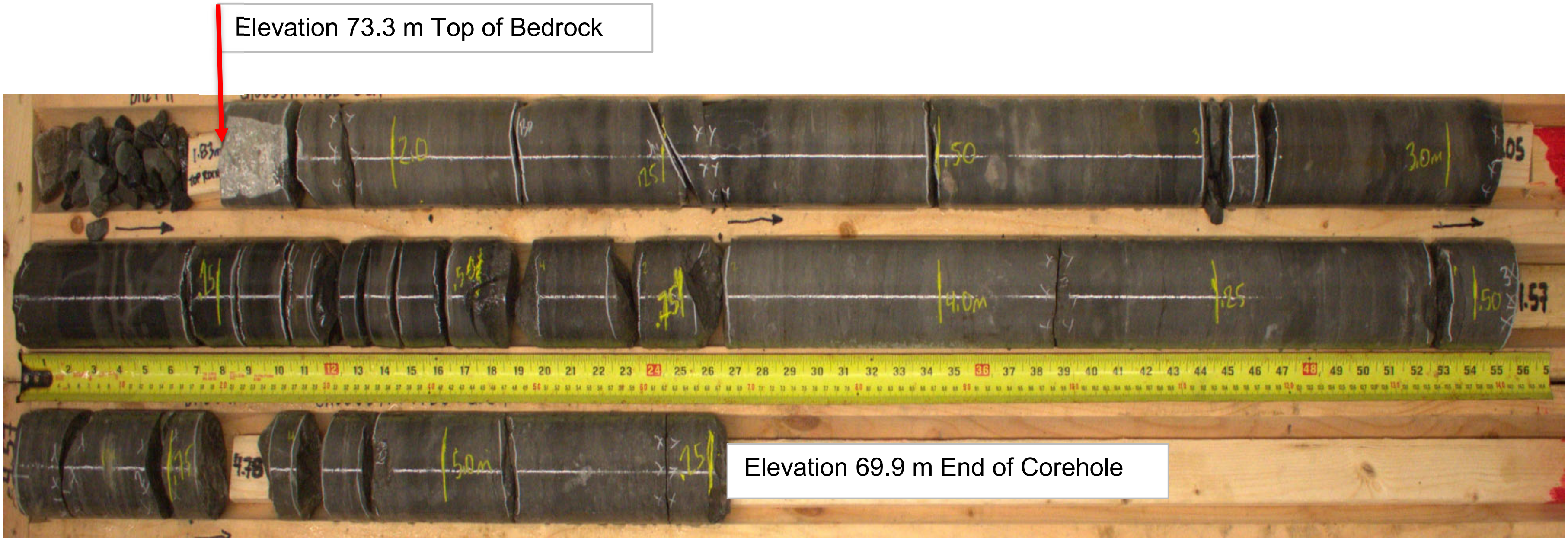
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A9

24-11 (Wet)
Core Box 1 of 1



Elevation 73.3 m Top of Bedrock

Elevation 69.9 m End of Corehole

Geotechnical Investigation

Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722

Drawn: BW

Date: 2024-07-23

Checked: OB

Review: JSA

Figure A10

24-13 (Dry)
Core Box 1 of 1


Elevation 70.3 m Top of Bedrock

Elevation 66.3 m End of Corehole

Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No.	CA0033714.1722
Drawn:	BW
Date:	2024-07-23
Checked:	OB
Review:	JSA

Figure A11

24-13 (Wet)
Core Box 1 of 1


Elevation 70.3 m Top of Bedrock

Elevation 66.3 m End of Corehole

Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A12

24-15 (Dry)
Core Box 1 of 1


Elevation 68.9 m Top of Bedrock

Elevation 62.9 m End of Corehole

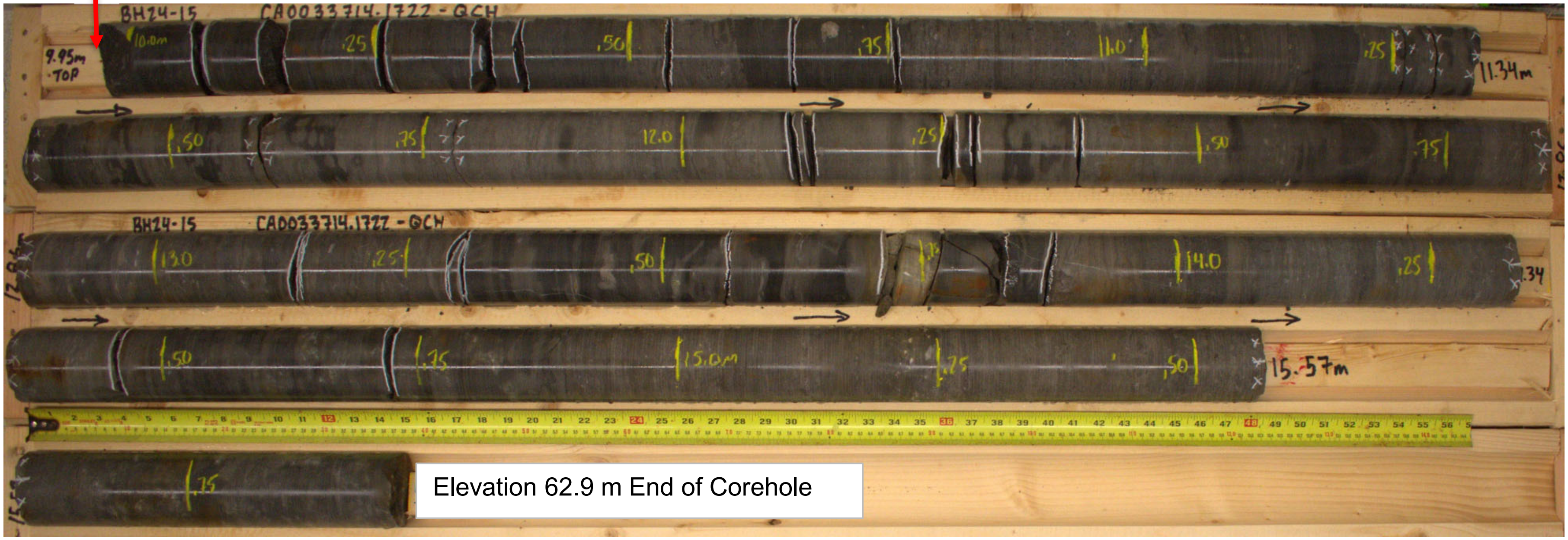
Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A13

24-15 (Wet)
Core Box 1 of 1



Elevation 68.9 m Top of Bedrock

Elevation 62.9 m End of Corehole

wsp

Geotechnical Investigation
Queensway Carleton Hospital Expansion

3045 Baseline Rd, Ottawa, ON K2H 8P4

Project No. CA0033714.1722
Drawn: BW
Date: 2024-07-23
Checked: OB
Review: JSA

Figure A14

APPENDIX C

Laboratory Results

TABLE 1

SUMMARY OF WATER CONTENT DETERMINATIONS (ASTM D2216/LS-701)

PROJECT NUMBER CA0033714.1722
PROJECT NAME QCH Expansion Geotechnical Investigation
DATE TESTED July 30, 2024

Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)
24-01	4	7'6"-9'6"	2.29-2.90	42.2%	24-06	1A	0'0"-0'3"	0.00-0.08	4.0%
					24-06	1B	0'4"-2'0"	0.10-0.61	4.7%
24-02	1A	0'0"-0'4"	0.00-0.10	13.9%	24-06	2	2'6"-4'6"	0.76-1.37	4.1%
24-02	1B	0'4"-2'0"	0.10-0.61	2.8%	24-06	3	5'0"-7'0"	1.52-2.13	17.3%
24-02	2	2'6"-4'6"	0.76-1.37	7.2%	24-06	4A	7'6"-9'2"	2.29-2.79	14.6%
24-02	3	5'0"-7'0"	1.52-2.13	61.4%	24-06	4B	9'2"-9'6"	2.79-2.90	8.9%
24-02	4	10'0"-10'10"	3.05-3.30	11.0%					
					24-08	1	0'5"-2'0"	0.13-0.61	3.1%
24-04	2	2'6"-4'6"	0.76-1.37	24.2%	24-08	2	2'6"-4'6"	0.76-1.37	13.5%
24-04	4	7'6"-9'6"	2.29-2.90	32.1%	24-08	3	5'0"-5'8"	1.52-1.73	14.2%
24-05	1	0'0"-2'0"	0.00-0.61	1.9%	24-10	1A	0'2"-0'8"	0.05-0.20	6.5%
24-05	2	2'6"-4'6"	0.76-1.37	20.2%	24-10	1B	0'8"-2'0"	0.20-0.61	16.4%
24-05	3	5'0"-7'0"	1.52-2.13	32.3%	24-10	2	2'6"-4'6"	0.76-1.37	8.3%
24-05	4	7'6"-9'6"	2.29-2.90	13.4%	24-10	3	5'0"-7'0"	1.52-2.13	9.4%
24-05	5	10'0"-12'0"	3.05-3.66	10.6%	24-10	4	7'6"-9'6"	2.29-2.90	13.7%
24-05	6	12'6"-14'6"	3.81-4.42	22.2%	24-10	5	10'0"-12'0"	3.05-3.66	12.2%
24-05	7	15'0"-15'4"	4.57-4.67	17.4%	24-10	6	12'6"-14'6"	3.81-4.42	12.8%
24-10	7	15'0"-17'0"	4.57-5.18	11.4%	24-15	4	7'6"-9'6"	2.29-2.90	16.4%
24-10	8	17'6"-19'6"	5.33-5.94	14.9%	24-15	5	10'0"-12'0"	3.05-3.66	18.0%
					24-15	6	12'6"-14'6"	3.81-4.42	42.3%
24-12	1	0'0"-2'0"	0.00-0.61	2.8%	24-15	7	17'6"-19'6"	5.33-5.94	38.6%
24-12	2	2'6"-4'6"	0.76-1.37	9.2%	24-15	8	22'6"-24'6"	6.86-7.47	27.7%
24-12	3	5'0"-7'0"	1.52-2.13	8.4%	24-15	9	27'6"-29'6"	8.38-8.99	41.3%
24-12	4	7'6"-9'6"	2.29-2.90	7.0%					
24-12	5	10'0"-12'0"	3.05-3.66	35.8%	24-16	4	7'6"-9'6"	2.29-2.90	53.5%
24-12	6	12'6"-14'6"	3.81-4.42	41.4%					
24-12	7	20'0"-22'0"	6.10-6.71	40.1%					
24-12	8	25'0"-25'6"	7.62-7.77	27.6%					
24-13	5	10'0"-12'0"	3.05-3.66	40.9%					
24-14	6	15'0"-17'0"	4.57-5.18	44.0%					
24-15	1	0'0"-2'0"	0.00-0.61	6.7%					
24-15	3	5'0"-7'0"	1.52-2.13	14.5%					

Tested By: cw
Checked By: MI

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TABLE 1
SUMMARY OF WATER CONTENT AND ATTERBERG LIMITS DETERMINATIONS
(D4318/LS-703&704)

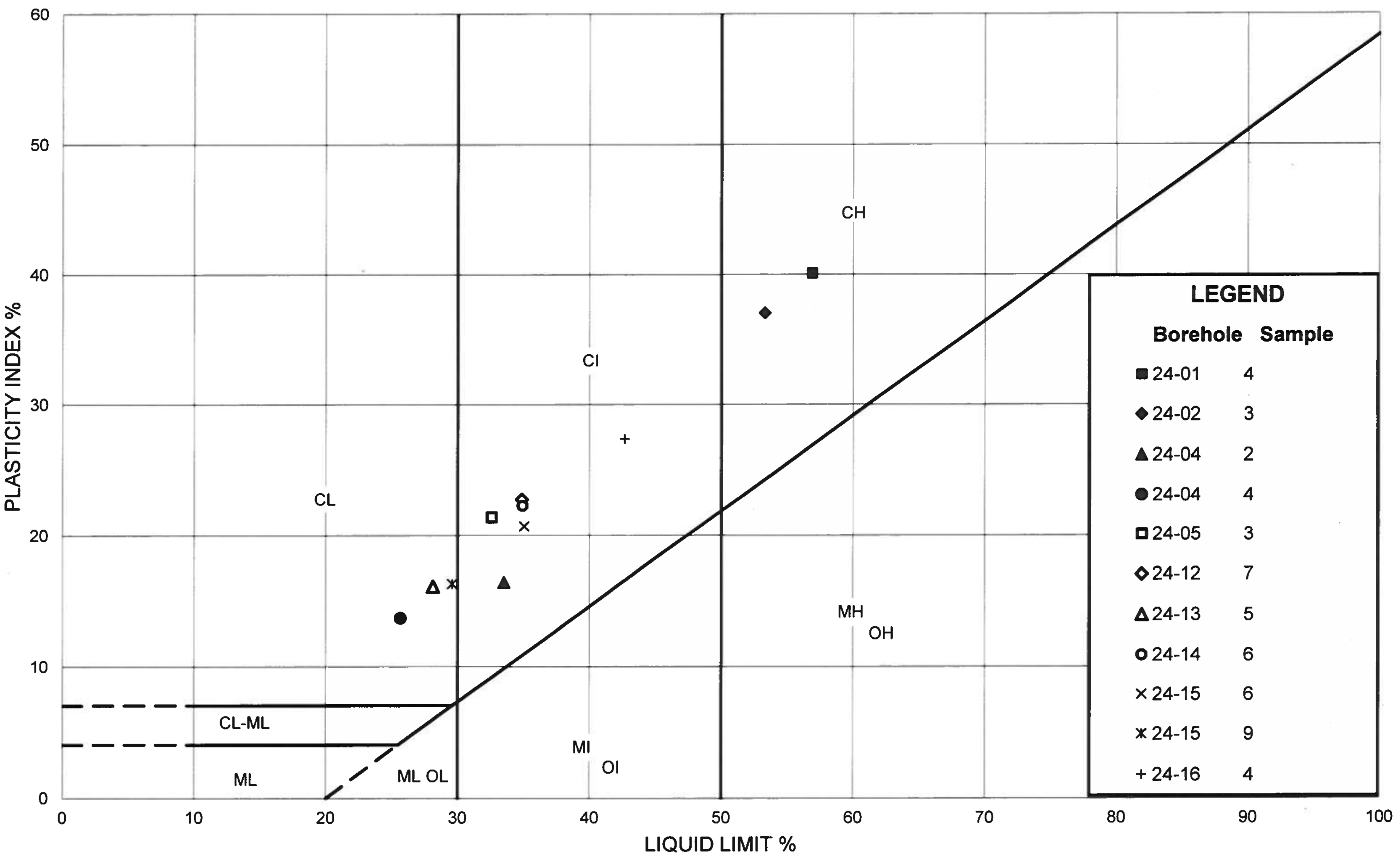
PROJECT NUMBER	CA0033714.1722
PROJECT NAME	QCH Expansion Geotechnical Investigation
DATE TESTED	August 14, 2024

Borehole No.	Sample No.	Depth (m)	Water Content (%)	Atterberg Limits W_L	W_P	LI	PI
24-01	4	2.29-2.90	42.20	56.9	16.8	0.6	40.1
24-02	3	1.57-2.13	61.40	53.3	16.2	1.2	37.1
24-04	2	0.76-1.37	24.20	33.5	17.1	0.4	16.4
24-04	4	2.29-2.90	32.10	25.7	12.0	1.5	13.7
24-05	3	1.52-2.13	32.30	32.6	11.2	1.0	21.4
24-12	7	6.10-6.71	40.10	34.9	12.1	1.2	22.8
24-13	5	3.05-3.66	40.90	28.1	12.0	1.8	16.1
24-14	6	4.57-5.18	44.00	34.9	12.6	1.4	22.3
24-15	6	3.81-4.42	42.30	35.1	14.4	1.3	20.7
24-15	9	8.38-8.99	41.30	29.6	13.3	1.7	16.3
24-16	4	2.29-2.90	53.50	42.7	15.3	1.4	27.4

Tested By: CW
Checked By: MI

wsp

V2021



wsp

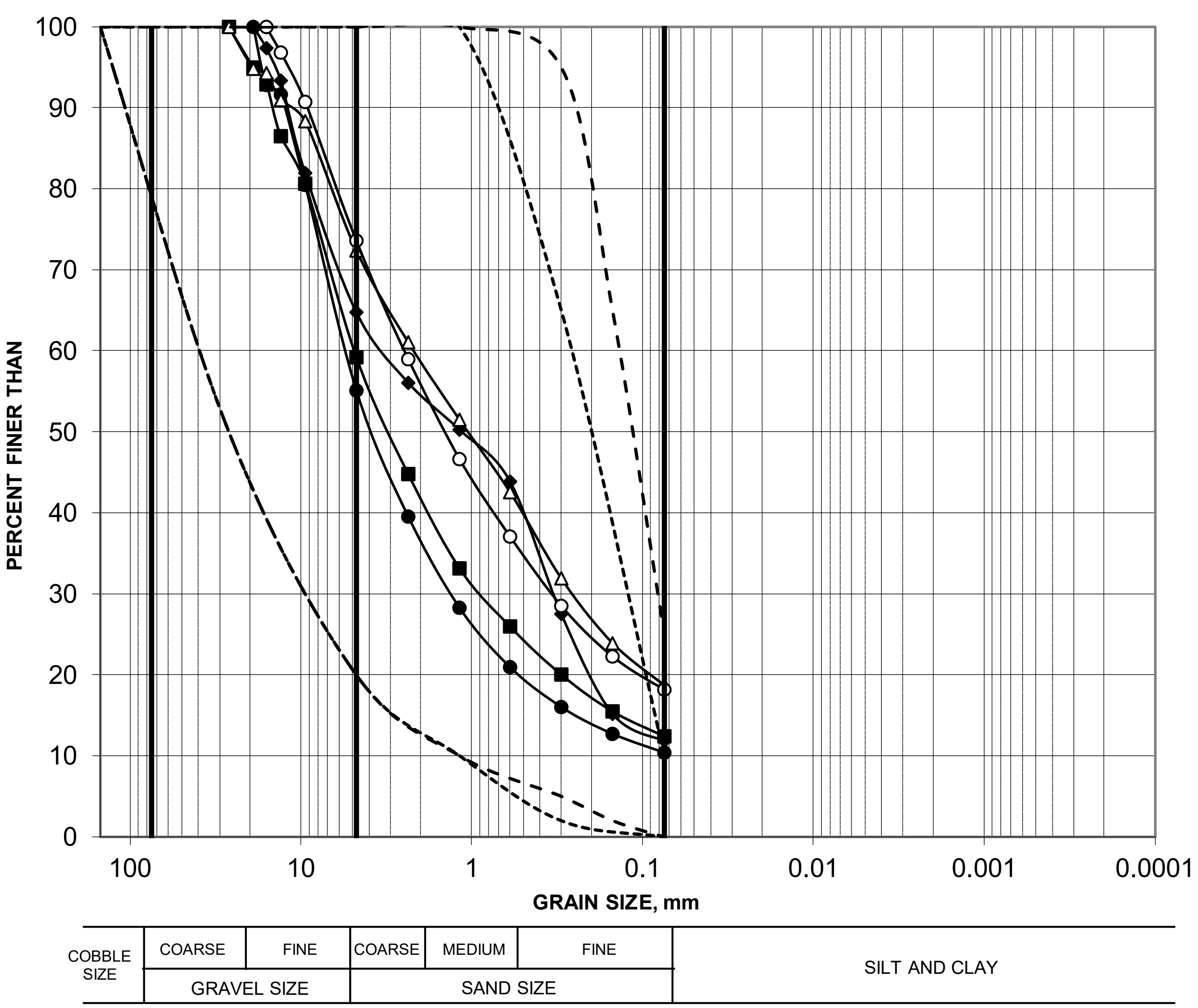
PLASTICITY CHART

Figure:
Project: CA0033714.1722
Created By: CW Checked By: MI

GRAIN SIZE DISTRIBUTION

FIGURE 1A

PAVEMENT STRUCTURE FILL



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—●— 24-05	1	0.00-0.61	45	45	10	
—■— 24-06	1A	0.00-0.08	41	47	12	
—◆— 24-06	2	0.76-1.37	35	53	12	
—△— 24-08	1	0.13-0.61	28	53	19	
—○— 24-11	1A	0.10-0.30	26	56	18	
------ OPSS 1010 - Gran B I						
– – – OPSS 1010 - SSM						

Project: CA0033714.1722

Created by: MI
Checked by: CW

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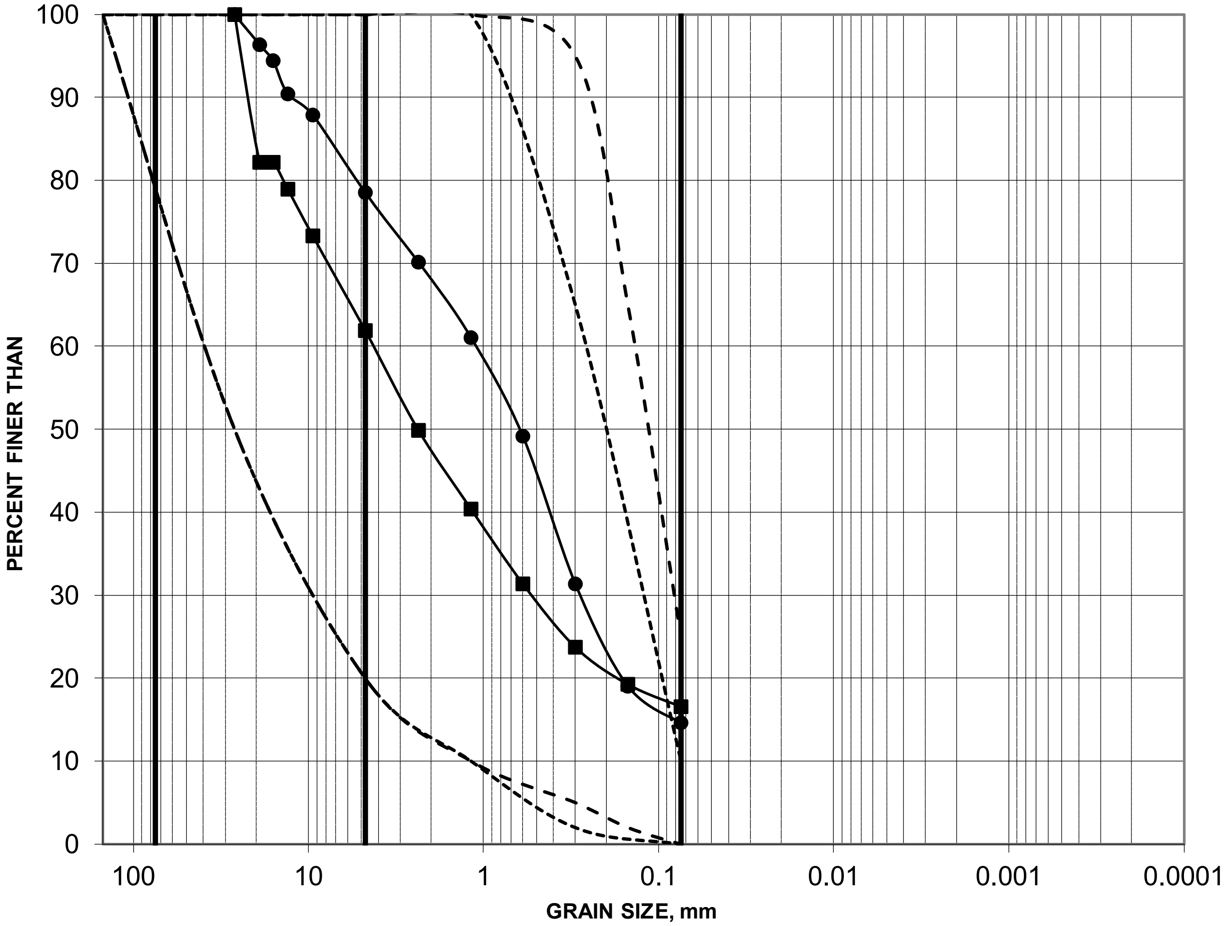
GRAIN SIZE DISTRIBUTION	FIGURE 1B

PAVEMENT STRUCTURE FILL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—●— 24-11	2	0.76-1.37	21	64	15	
—■— 24-14	1	0.30-0.76	38	45	17	
------ OPSS 1010 - Gran B I						
- - - OPSS 1010 - SSM						

Created by:	MI
Checked by:	CW

Project: CA0033714.1722

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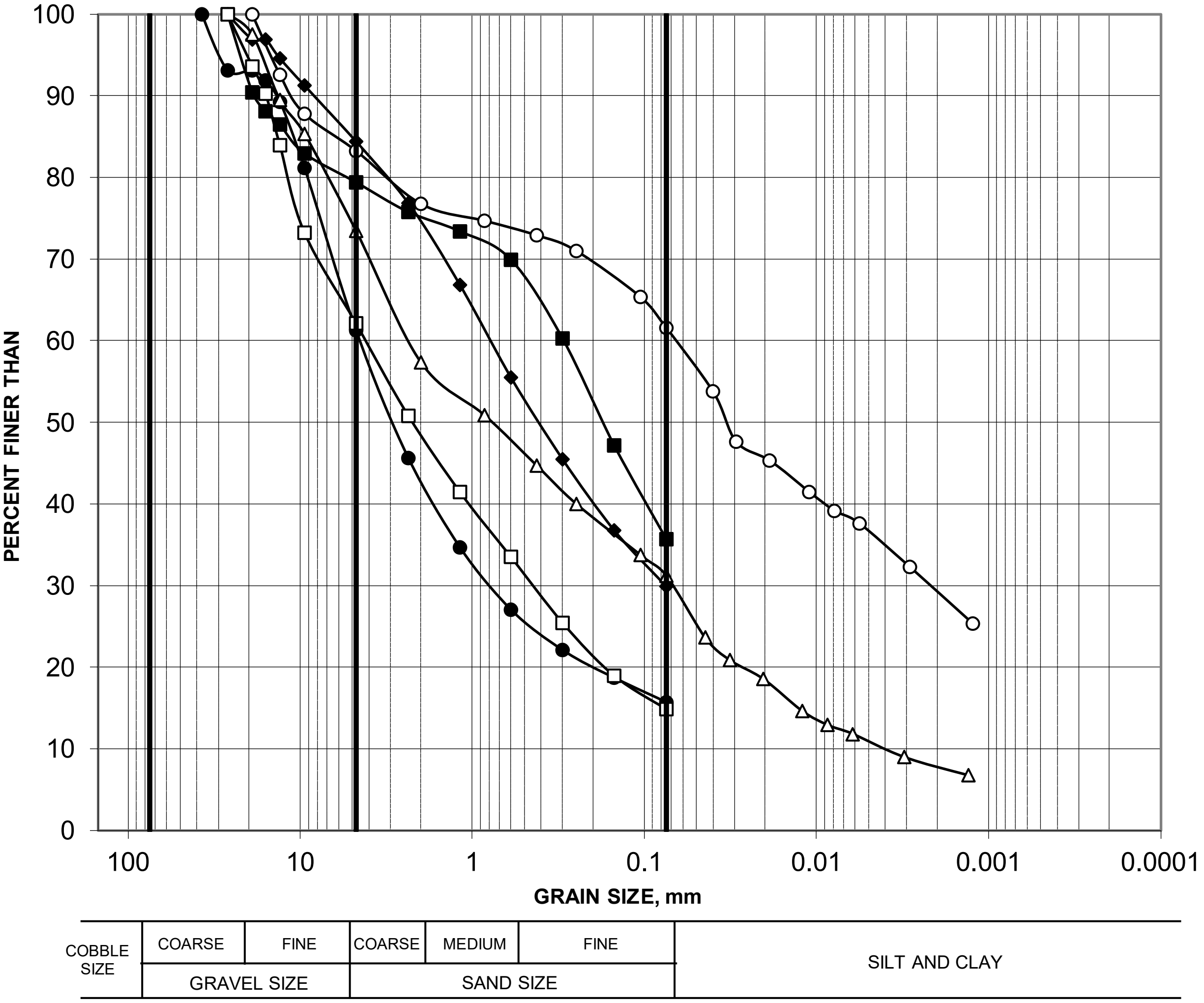
GRAIN SIZE DISTRIBUTION

FIGURE 2

FILL



				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—●—	24-03	1B	0.15-0.61	39	45	16	
—■—	24-04	1	0.00-0.61	21	43	36	
—◆—	24-12	2	0.76-1.37	16	54	30	
—△—	24-12	4	2.29-2.90	27	42	23	8
—○—	24-13	2	0.76-1.37	17	21	33	29
—□—	24-15	1	0.00-0.61	38	47	15	

Project: CA0033714.1722

Created by: MI
Checked by: CW

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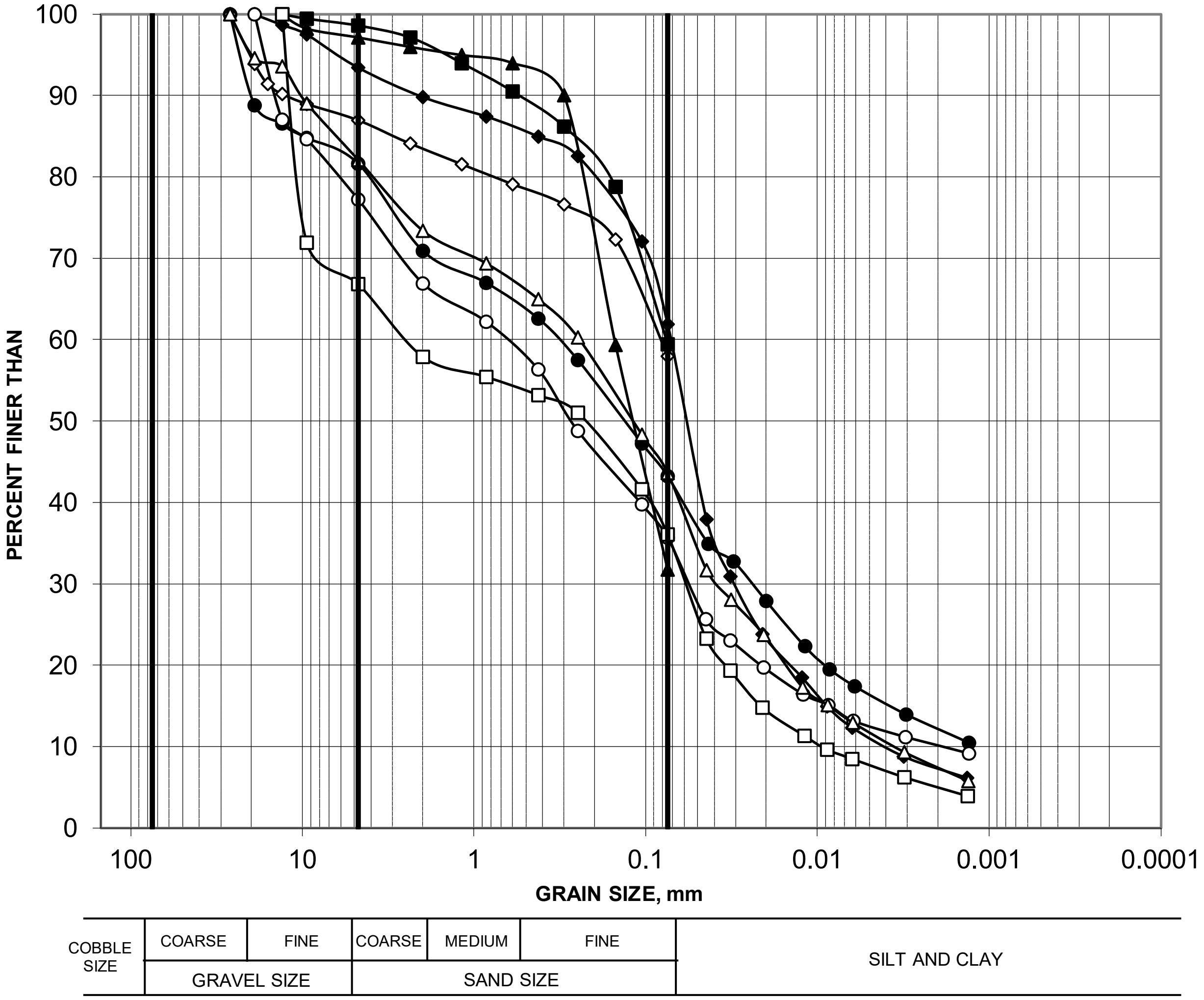
GRAIN SIZE DISTRIBUTION

FIGURE 3A

GLACIAL TILL



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
● 24-03	4	2.29-2.90	18	39	31	12
■ 24-03	6	3.81-4.42	1	40	59	
◆ 24-05	4	2.29-2.90	7	31	55	7
▲ 24-06	3	1.52-2.13	3	65	32	
○ 24-07	2	0.76-1.37	23	41	26	10
□ 24-08	2	0.76-1.37	33	31	31	5
◇ 24-09	3	1.52-1.80	13	29	58	
△ 24-10	3	1.52-2.13	18	38	37	7

Project: CA0033714.1722

Created by:	MI
Checked by:	CW

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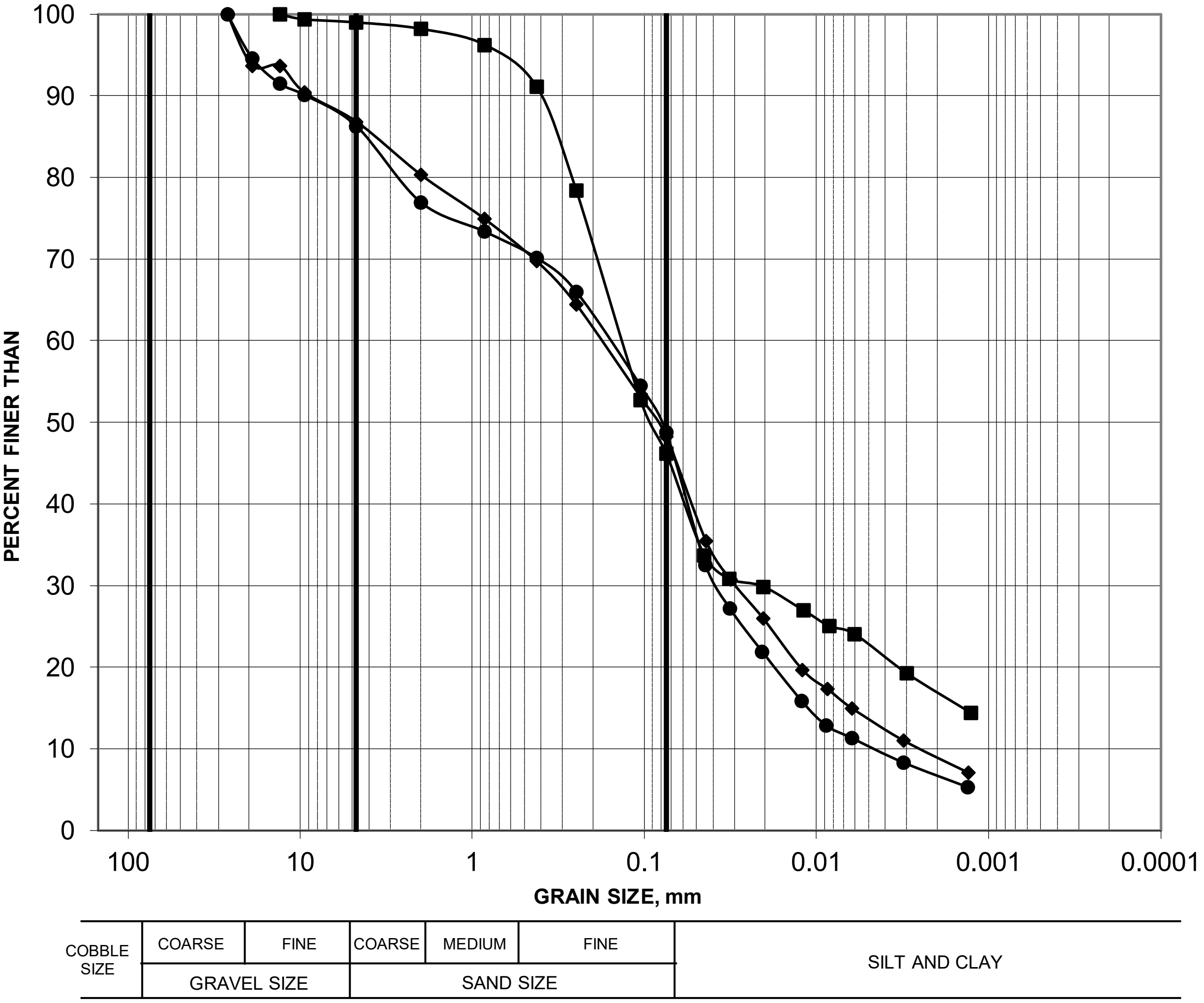
GRAIN SIZE DISTRIBUTION

FIGURE 3B

GLACIAL TILL



				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
●	24-10	5	3.05-3.66	14	37	42	7
■	24-13	8	6.71-6.95	1	53	29	17
◆	24-16	6	4.57-5.18	13	39	39	9

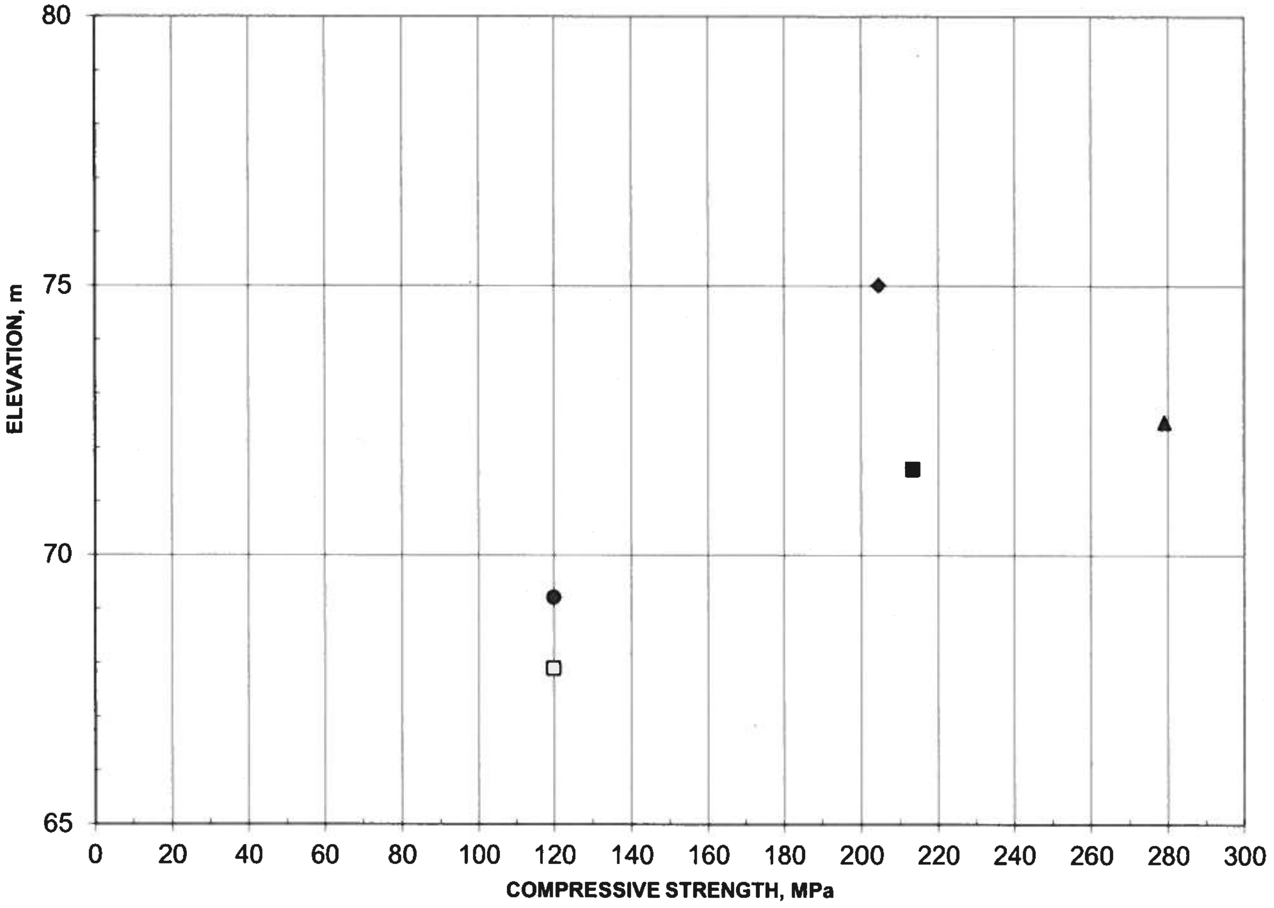
Project: CA0033714.1722

Created by: MI
Checked by: CW

https://wsponlinecan.sharepoint.com/Sites/Global-OttawaLab/Shared Documents/Active/CA0033714.1722/Figures/

ASTM D7012 - Method C
UNCONFINED UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE



	Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
■	24-03 RC1	5.8	2.6	2789	Dolostone	213	1
◆	24-06 RC1	4.7	2.6	2896	Dolostone	204	1
▲	24-11 RC1	2.7	2.6	2794	Dolostone	279	1
●	24-13 RC1	8.3	2.2	2735	Dolostone	120	1
□	24-15 RC1	11.0	2.5	2729	Dolostone	120	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical craking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

Created by: cw
Checked by:

Project: CA0033714.1722

https://wsponlinecan.sharepoint.com/Sites/Global-OttawaLab/Shared Documents/Active/CA0033714.1722/

eurofins | Environment Testing

Certificate of Analysis

Client: WSP Canada Inc.
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: M. Othmane Benkirane
PO#:
Invoice to: WSP Canada Inc.

Report Number: 3009915
Date Submitted: 2024-08-01
Date Reported: 2024-08-09
Project: CA0033714.1722
COC #: 915930

Dear Othmane Benkirane:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

Emma-Dawn
Ferguson
2024.08.09 16:11:43
-04'00'

APPROVAL: ______________________
Emma-Dawn Ferguson, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: https://directory.cala.ca/.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

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Certificate of Analysis

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Attention: M. Othmane Benkirane
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Invoice to: WSP Canada Inc.

Report Number: 3009915
Date Submitted: 2024-08-01
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Project: CA0033714.1722
COC #: 915930

				Lab I.D.	1738314	1738315	1738316	1738317
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2024-07-31	2024-07-31	2024-07-31	2024-07-31
				Sample I.D.	24-01 SA3	24-03 SA5	24-05 SA5	24-07 SA3
Group	**Analyte**	**MRL**	**Units**	**Guideline**				
Anions	Cl	0.002	%		0.006	0.002	0.019	0.064
	SO4	0.01	%		<0.01	0.01	0.03	0.04
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.26	0.14	0.49	1.06
	pH	2.00			8.19	8.55	8.29	8.54
	Resistivity	1	ohm-cm		3846	7143	2037	943

				Lab I.D.	1738318	1738319	1738320	1738321
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2024-07-31	2024-07-31	2024-07-31	2024-07-31
				Sample I.D.	24-08 SA3	24-10 SA4	24-13 SA4	24-15 SA7
Group	**Analyte**	**MRL**	**Units**	**Guideline**				
Anions	Cl	0.002	%		0.064	0.025	0.003	0.369
	SO4	0.01	%		0.02	<0.01	<0.01	0.06
General Chemistry	Electrical Conductivity	0.05	mS/cm		1.07	0.70	0.16	4.49
	pH	2.00			8.85	8.87	8.32	7.74
	Resistivity	1	ohm-cm		935	1429	6250	223

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

eurofins | Environment Testing

Certificate of Analysis

Client: WSP Canada Inc.
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: M. Othmane Benkirane
PO#:
Invoice to: WSP Canada Inc.

Report Number: 3009915
Date Submitted: 2024-08-01
Date Reported: 2024-08-09
Project: CA0033714.1722
COC #: 915930

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 464111 **Analysis/Extraction Date** 2024-08-08 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	6.20	98	90-110
Resistivity			
Run No 464118 **Analysis/Extraction Date** 2024-08-08 **Analyst** M B **Method** AG SOIL			
SO4	<0.01 %	109	70-130
Run No 464200 **Analysis/Extraction Date** 2024-08-09 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %	93	90-110

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX D

Vertical Seismic Profile Technical Memorandums

TECHNICAL MEMORANDUM

DATE August 12, 2024

Project No. CA0033714.1722

TO Othmane Benkirane
WSP Canada Inc.

CC

FROM Alex Bilson Darko, Christopher Phillips

**EMAIL alex.bilson.darko@wsp.com;
christopher.phillips@wsp.com**

VERTICAL SEISMIC PROFILING TEST RESULTS
QUEENSWAY CARLETON HOSPITAL, OTTAWA, ONTARIO

This memorandum presents the results of a Vertical Seismic Profiling (VSP) tests carried out at two locations located at Queensway Carleton Hospital, Ottawa, Ontario. The boreholes were drilled to depths of approximately 10.4 and 16.0 m (BH24-01 and BH24-15 respectively) below the existing ground surface and then cased with a 2.5-inch PVC pipe grouted in place.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth (Figure 1). The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole. The high-resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada (2020).

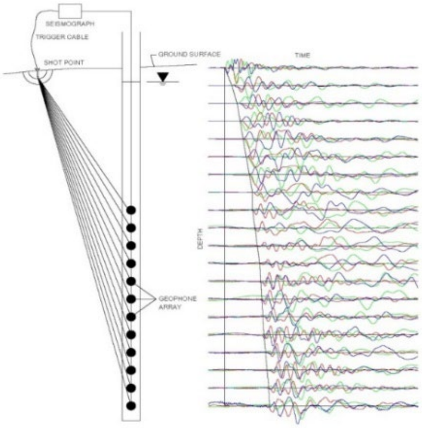


Figure 1: Layout and resulting time traces from a VSP survey.

Field Work

The field work was carried out on July 23rd, 2024, by personnel from the WSP Mississauga office. For the boreholes tested (BH24-01 and BH24-15), both compression and shear-wave seismic sources were used. The seismic source for the compression wave test consisted of a 10-lb. sledgehammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 2.4-metre-long, 150 by 150 mm wooden beam, weighted by a vehicle, and horizontally struck with a 10-lb. sledgehammer on opposite ends of the beam to induce polarized shear waves. Test measurements started at ground surface and were recorded in the boreholes with a 3-component receiver spaced at 1-metre intervals below the ground surface to the maximum depth of the casing. The source point was located at 2.1 m from the boreholes.

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The field crew actively monitored the noise levels before collecting data as nearby roads could create unwanted signal. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 milliseconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
2) Low Pass Filtering of data to remove spurious high frequency noise;
3) First break picking of the compression and shear-wave arrivals; and,
4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records from the borehole are presented in Figures 2 and 3 showing the first break picks of the compression wave followed by the shear wave arrivals overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

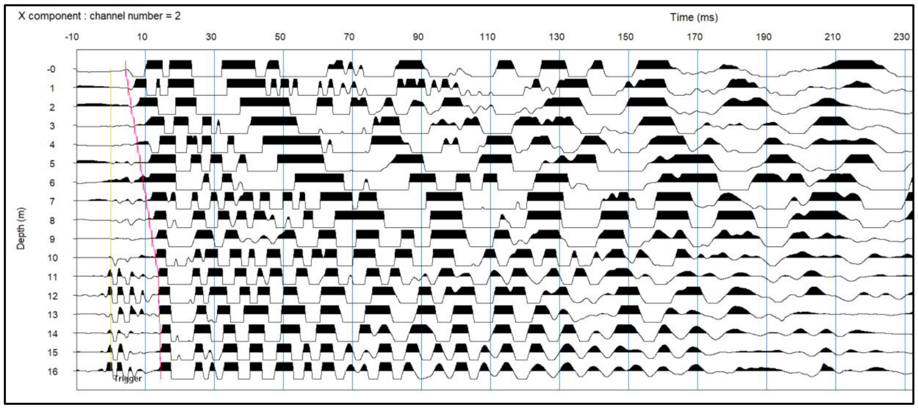


Figure 2: Example first break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of BH24-15.

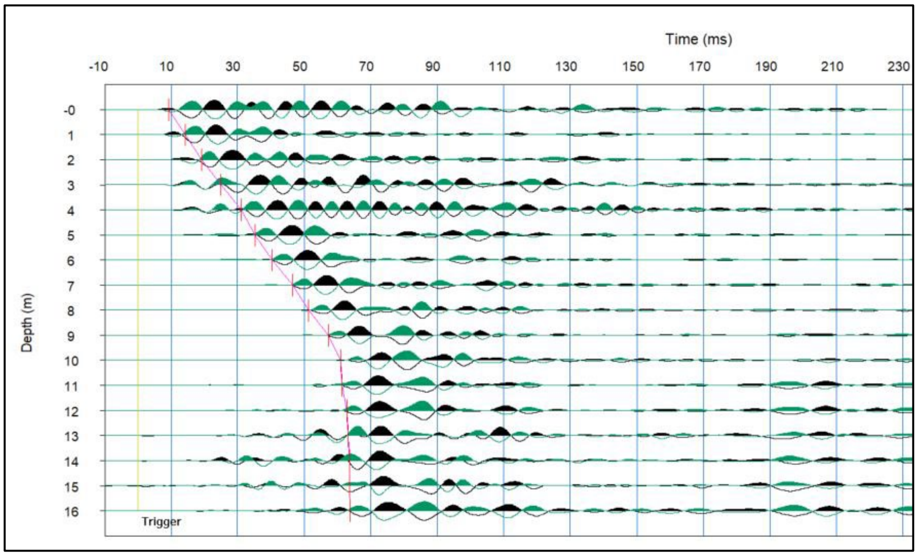


Figure 3: Example first break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of BH24-15.

Results

The VSP results for the boreholes are summarized in Tables 1 and 2. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Tables 1 and 2. The engineering moduli were calculated using an estimated bulk density of 1300-2600 kg/m^3 based on the borehole logs.

Closure

We trust that this technical memorandum meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience.

WSP Canada Inc.

Alex Bilson Darko, MSc
Geophysicist III, Experienced

Christopher Phillips, MSc, PGeo
Geophysicist VII, Senior Principal

ABD/CRP/

Attachments: Table 1 and 2– Shear Wave Profile

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BH24-01

Layer Depth (m)				Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.3	1.3	600	200	1300	0.44	52	150	399
1.3	2.3	600	190	1300	0.44	47	136	405
2.3	3.3	700	160	1300	0.47	33	98	593
3.3	4.3	3000	1800	2600	0.22	8424	20534	12168
4.3	5.3	3000	1800	2600	0.22	8424	20534	12168
5.3	6.3	3000	1800	2600	0.22	8424	20534	12168
6.3	7.3	3000	1800	2600	0.22	8424	20534	12168
7.3	8.3	3500	2100	2600	0.22	11466	27948	16562
8.3	9.3	3500	2100	2600	0.22	11466	27948	16562
9.3	10.3	3500	2100	2600	0.22	11466	27948	16562

Notes

1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

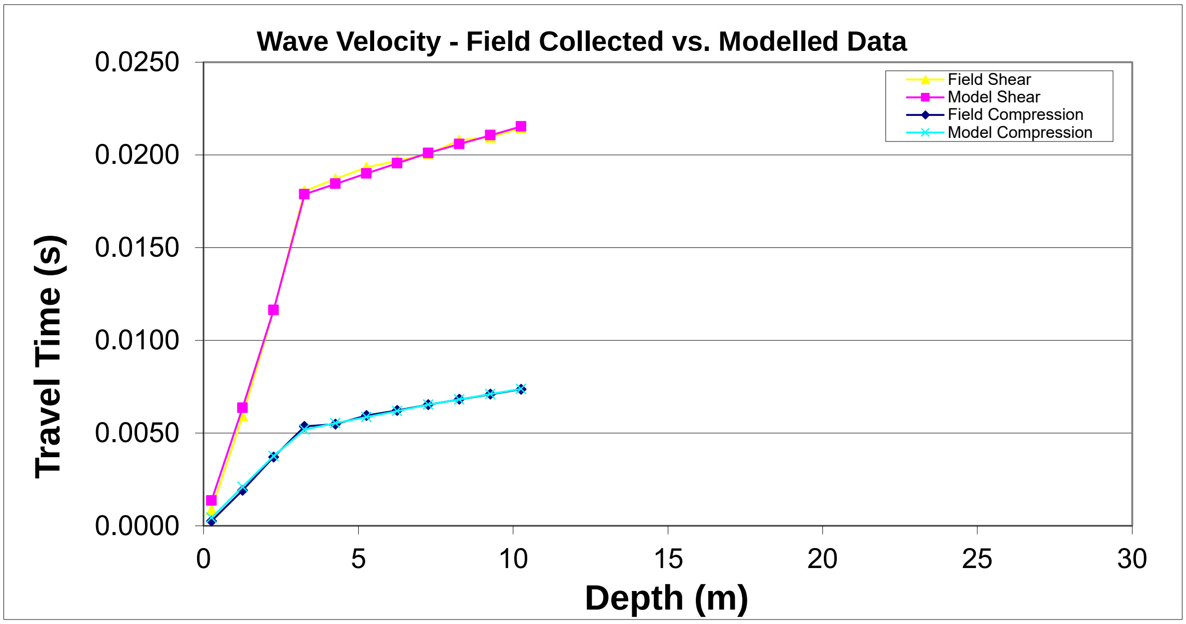
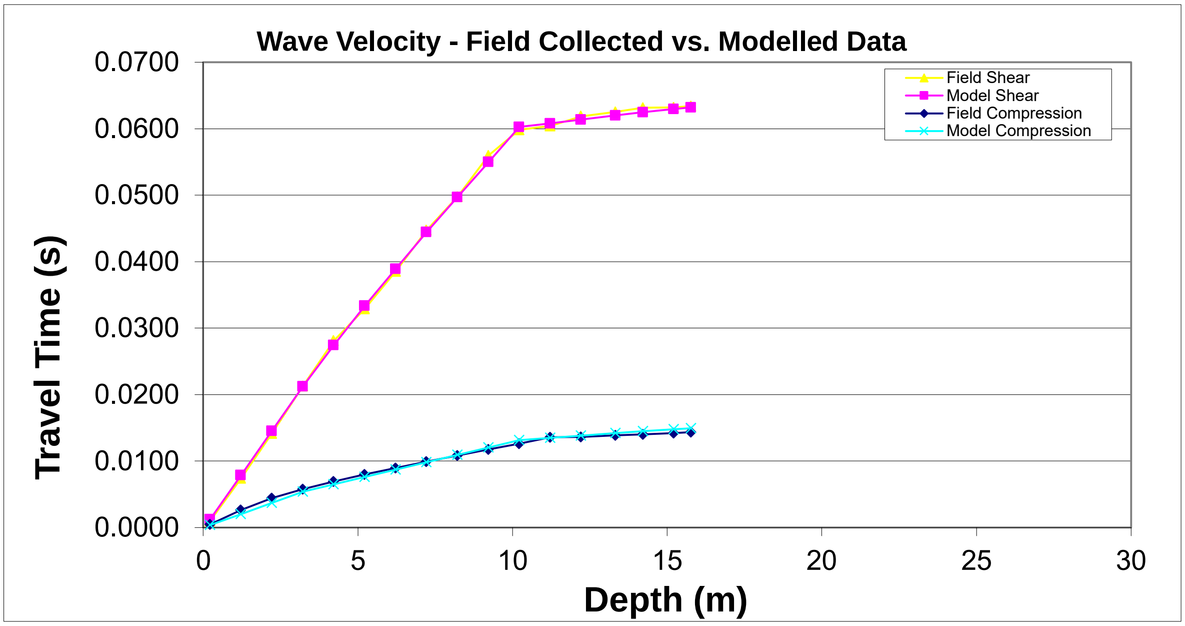


TABLE 2
SHEAR WAVE VELOCITY PROFILE AT BH24-15

Layer Depth (m)				Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.2	1.2	600	150	1300	0.47	29	86	429
1.2	2.2	600	150	1300	0.47	29	86	429
2.2	3.2	600	150	1300	0.47	29	86	429
3.2	4.2	900	160	1300	0.48	33	99	1009
4.2	5.2	900	170	1300	0.48	38	111	1003
5.2	6.2	900	180	1300	0.48	42	125	997
6.2	7.2	900	180	1300	0.48	42	125	997
7.2	8.2	900	190	1300	0.48	47	139	990
8.2	9.2	900	190	1300	0.48	47	139	990
9.2	10.2	900	190	1300	0.48	47	139	990
10.2	11.2	3000	1800	2600	0.22	8424	20534	12168
11.2	12.2	3000	1800	2600	0.22	8424	20534	12168
12.2	13.3	3000	1800	2600	0.22	8424	20534	12168
13.3	14.2	3000	1800	2600	0.22	8424	20534	12168
14.2	15.2	3500	2100	2600	0.22	11466	27948	16562
15.2	15.8	3500	2100	2600	0.22	11466	27948	16562

Notes

1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.



TECHNICAL MEMORANDUM

Golder Associates Ltd.
2390 Argentia Road
Mississauga, ON, Canada L5N 5Z7

Telephone: 905-567-4444
Fax Access: 905-567-6561

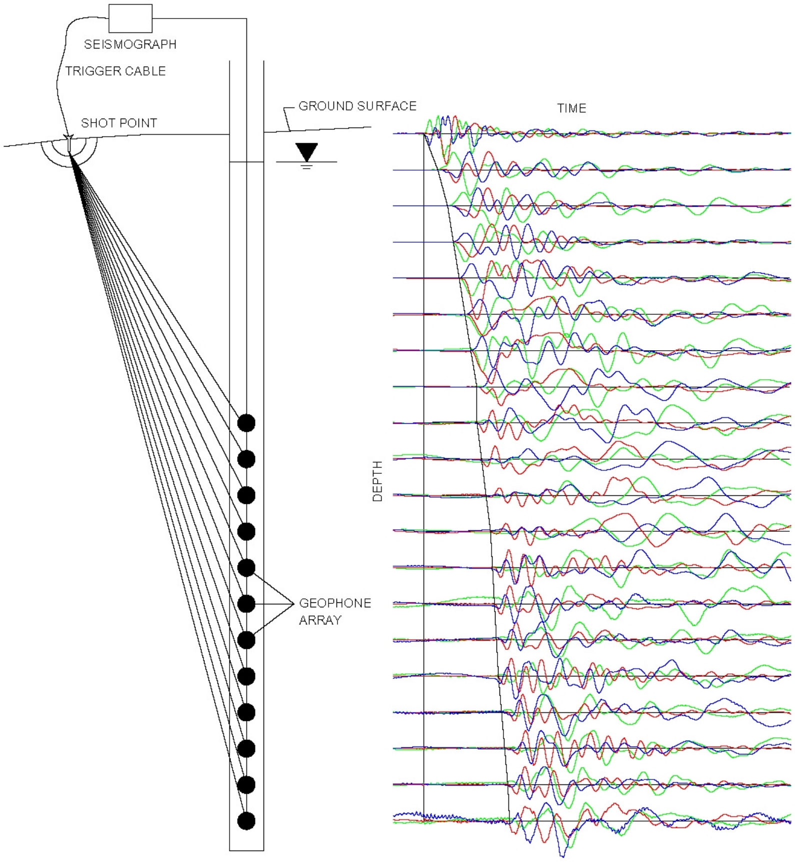
TO:	Michel St. Louis, GAL – Ottawa	**DATE:**	September 17, 2008
FROM:	Christopher Phillips, GAL – Mississauga Tom Flynn, GAL – Mississauga	**JOB NO:**	07-1121-0002
EMAIL:	cphillips@golder.com		
RE:	Vertical Seismic Profile Data Processing and Results – Queensway-Carleton Hospital		

This memorandum presents the processing and results of the vertical seismic profile (VSP) testing performed in Golder Borehole 08-307 located at the Queensway-Carleton Hospital by Golder Associates Ltd. (Golder) on August 28, 2008.

1.0 METHODOLOGY

Vertical seismic profiling is a single borehole geophysical method. Seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in the borehole at a known depth below ground surface. The active seismic source can be either compression or shear-wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada, 2005.



Example 1: Layout and resulting time traces from a VSP survey.

2.0 FIELD WORK

The field work was conducted on August 28, 2008, by Golder Associates personnel.

Both compression and shear-wave seismic sources were used and both were located in close vicinity to the borehole. The compression seismic source consisted of a 5.5 kilogram sledge hammer vertically impacted on a metal plate. The plate was located 2.2 metres from the borehole. The shear-wave seismic source used consisted of a 2.4 metre long, 150 millimetres by 150 millimetre wooden beam, weighted on the ground by a vehicle and horizontally struck with a 5.5 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was located 2.2 metres from the borehole.

Tests were conducted with the borehole geophone at 0.5m intervals, beginning at a depth of 1 metre below ground surface, to the maximum depth of the borehole (8 metres). A three component borehole geophone configuration was used to record the induced seismic events.

Data collected for each source were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. Data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 second window of data was collected for each seismic shot.

3.0 DATA PROCESSING

Processing of the VSP test results consisted of the following main steps:

1. Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
2. Low Pass Filtering (250 Hz) of data to remove spurious high frequency noise;
3. First break picking of the shear-wave arrivals;
4. Calculation of the average shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The quality of collected seismic data and the shear wave event ‘first break’ picks are presented on Figure 1 (below).

4.0 RESULTS

The VSP results are summarized in Table 1. Layer velocities, at 1 meter intervals, were calculated by best fitting a theoretical travel time model to the field collected data at 1 metre intervals. A plot of the match of the field to model data is presented in Figure 2. The depths presented on the tables are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented on Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole logs. We estimated a bulk density of 1750 kg/m^3 from the surface down to a depth of 3 mbgs which is the approximate depth of the dolomite bedrock as indicated in the borehole log (3.14 mbgs). Below this depth we estimated the bulk density to be 2000 kg/m^3. The shear-wave average velocities show an increase at 2 mbgs. This change in velocity correlates with the borehole log which indicates a shift from clay to glacial till.

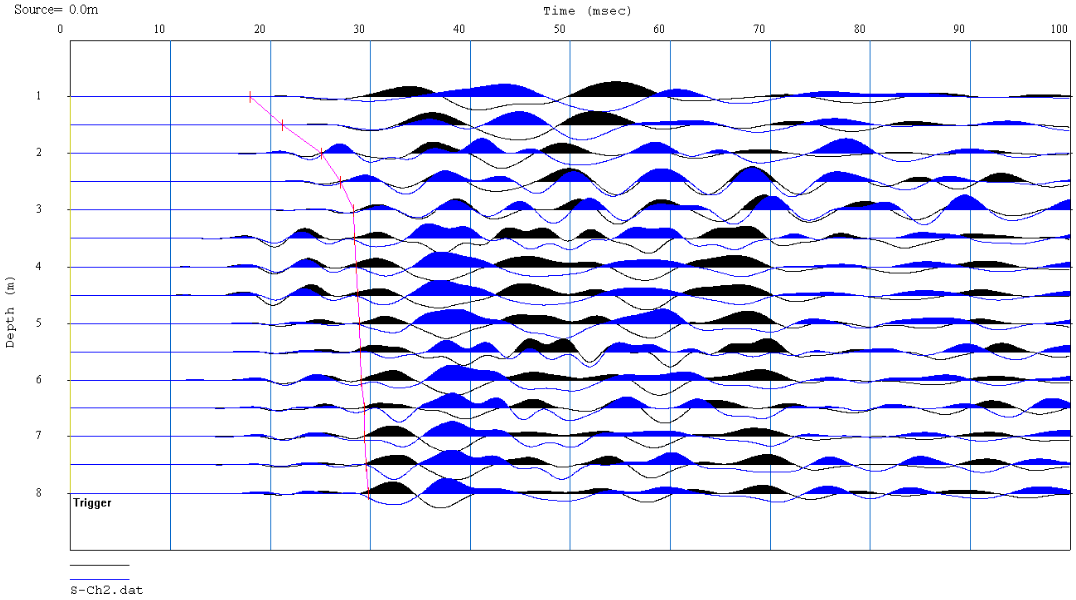


Figure 1: First break picking of S wave arrivals (red) along the seismic traces recorded at each receiver depth

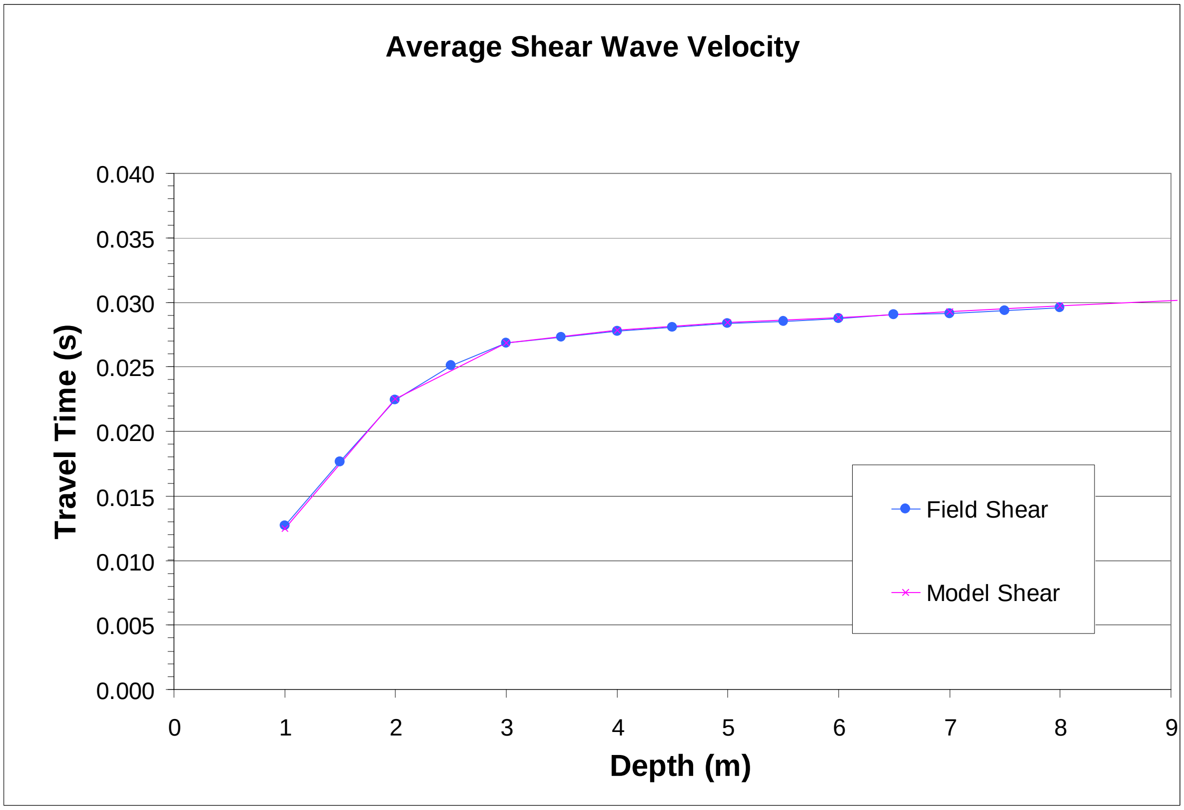


Figure 2: Comparison of Field and Model Calculated Shear Wave Travetimes

Table 1: Model Shear Wave Velocity Results

Layer Depth (m)			Estimated Bulk Density (kg/m^3)	Shear Modulus (MPa)
Top	**Bottom**	**Shear Wave**		
0.0	1.0	80	1750	11
1.0	2.0	100	1750	18
2.0	3.0	230	1750	93
3.0	4.0	1000	2000	2000
4.0	5.0	1800	2000	6480
5.0	6.0	2200	2000	9680
6.0	7.0	2200	2000	9680
7.0	8.0	2300	2000	10580
8.0	30.0	2500	2000	12500

The VSP results indicate an average shear-wave velocity, calculated from the time taken for the shear-wave to travel from the surface to a depth of 30 metres, of 780 m/s. The average velocity was calculated assuming that the velocity from 8 to 30 metres was the same as the velocity calculated at the bottom of the borehole (2500 m/s).

5.0 CLOSURE

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

PROFESSIONAL GEOSCIENTISTS
CHRISTOPHER R. PHILLIPS
PRACTISING MEMBER
1503
ONTARIO

Christopher Phillips, M.Sc., P.Geo
Senior Geophysicist

APPENDIX E

Desktop Study Memorandum

TECHNICAL MEMORANDUM

DATE June 24, 2024

Project No. CA0033714.1722

TO Mr. Peter Thompson
Queensway Carleton Hospital
3045 Baseline Road
Ottawa, Ontario K2H 8P4

GEOTECHNICAL DESKTOP STUDY
PROPOSED QUEENSWAY-CARLETON HOSPITAL EXPANSION

WSP Canada Inc. (WSP) has been retained to provide geotechnical engineering services for the proposed Queensway-Carleton Hospital expansion, located in Ottawa, Ontario (the Site).

It is understood that the Queensway Carleton Hospital (QCH or "Client" herein) requires geotechnical information to support design and construction of several new building elements attached or adjacent to existing hospital structures, a proposed free-standing parking garage, and access road upgrades, as shown on the attached Site Plan (Figure 1). The Site Plan also presents the location of 16 boreholes proposed by the Client, to be reviewed by WSP in the context of the proposed development and available historical information. This Technical Memorandum presents the results of our geotechnical desktop review and gap analysis, and our recommendations for the detailed investigation program.

WSP (including former Golder Associates Ltd. and McRostie Genest St-Louis acquisitions) previously completed several investigations within the Queensway-Carleton Hospital campus. The following information was considered most relevant:

- Report prepared by Golder Associates titled "Geotechnical Background Information 2009, Queensway-Carleton Hospital, Baseline Road, Ottawa, Ontario", dated August 10, 2009 (Report No. 07-1121-0002 (9000)). This Golder report also contains borehole logs from McRostie Genest report SF-1177A.

- Report prepared by Golder Associates titled "Report on Preliminary Geotechnical Investigation, Faculty Development Plan – Part 3A, Queensway-Carleton Hospital, Baseline Road, Ottawa, Ontario", dated October 29, 2008 (Report No. 07-1121-0002 (7000)).

The historical reports document 99 borehole and test pit logs completed on site between 1968 and 2008. A plan summarizing the historical borehole and test pit locations is attached as Figure 2. Borehole and test pit logs are appended.

In general, subsurface conditions beneath the QCH site include a layer of topsoil/fill underlain by silty sandy and/or clayey deposits with variable amounts of sand and silt. This material is underlain by sandy glacial till over the dolostone bedrock of the Beekmantown Group.

The following sections provide a discussion on each proposed expansion feature from a geotechnical perspective, including an information gap analysis and commentary on geotechnical investigation requirements.

Parking Garage

A new free-standing parking garage, rectangular in shape, approximately 35 x 75 m, is proposed to be built north of the existing parking garage. A grassy area with trees and paved bike lanes currently exists in the footprint area. Design review is required to determine the number of underground levels planned, if any.

WSP found records for five boreholes and test pits completed near the proposed footprint area:

- TP97-1: located slightly outside and north of the proposed area.
- TP97-2: located at the eastern limit of the proposed area.
- TP97-3: located in the western half of the proposed area.
- BH07-04: located slightly outside and southwest of the proposed area.
- BH07-06: located in the western half of the proposed area.

Based on the borehole and test pits logs, the ground elevation is mostly flat and varies between about 76.9 and 77.7 metres above mean sea level (masl), per a geodetic datum.

Subsurface materials are consistent with other areas of the QCH site.

Topsoil, where encountered, was approximately 150 to 450 mm thick.

Fill material was encountered in three of the historical testholes and consisted primarily of sand with variable amounts of silt and gravel. Fill extended to depths varying between approximately 0.6 and 0.9 m (76.4 to 76.0 masl). No SPT ‘N’ values were taken within the fill layer to assess relative compactness.

The (presumed native) granular deposit was encountered in three of the testholes and consisted primarily of compact silt and sand at varying amounts. It extended to depths varying between about 0.6 to 1.2 m (i.e., 76.9 to 75.9 masl). An SPT ‘N’ value of 19 was noted within this layer.

A cohesive soil deposit was reported in all five testhole records and was described as hard to firm clay to silty clay with loose sand seams. At the boreholes, the deposit extended to depths varying between about 3.7 and 4.1 m (i.e., 73.5 to 72.8 masl). Penetrometer readings were taken and reported undrained shear strengths of between 260 to 400 kPa. No in situ torque vane tests were taken, which more definitively measure the shear strengths of cohesive soils.

The glacial till, where encountered, was reported to contain very loose to loose silt and sand in varying amounts, with SPT ‘N’ values of between 2 and 5. Results are suggestive of potential disturbed material sampling; conditions should be tested and reaffirmed in the proposed updated study. The till deposit extended to a maximum depth of approximately 5.5 m.

Bedrock was cored and identified as grey dolostone and was encountered at depths approximately 4.4 and 5.5 m (i.e., 72.7 to 71.4 masl). Test pits extended to a maximum depth of 2.2 m without noting any refusal. Dolostone was core sampled at borehole BH07-04 to a depth of 7.7 m (69.2 masl), noting an RQD varying between 51 to 63% (Fair quality).

Groundwater was noted at a depth of 4.4 m (72.1 masl) at borehole BH07-04 during drilling (no piezometer reading available). No groundwater seepage was noted in any test pits.

The following table summarizes identified information gaps that are relevant to the geotechnical design of the proposed parking garage.

Table 1 – Information Gap Analysis – Parking Garage

Gap No.	Description of Information Gap	Degree of Importance
1	Available borehole and test pit logs date from 1997 and 2007. Information on any excavation, construction or grade raise that might have happened since then is not readily available. The exact field work methodology of past investigations is unknown. A refresh of the subsurface condition information at the site is recommended.	High
2	Qualities and characteristics of the overburden soil across the proposed area are limited. - Limited number of SPT 'N' values. - Overburden thickness across the proposed area is limited. - Further testing is recommended to assess the extent of very loose to loose soils.	Moderate
3	Limited information exists regarding the depth and characteristics of the bedrock underlying the site. - Only one borehole (BH07-04) included rock coring, however, the borehole is located outside the proposed area. - Bedrock was reported as only Fair quality; depth to Good to Excellent bedrock depth is unknown and may be applicable to the design of deep foundations for the structure.	Moderate
4	Groundwater levels/gradient across the proposed area are unknown. - The only water level depth available was taken in an open hole during drilling in 2007.	Moderate
5	Laboratory testing information is limited to non-existent. - No records of any grain sizes, Atterberg limits or UCS rock testing are available.	Moderate
6	Soil corrosion potential is unknown at the proposed area (sulphate damage to concrete elements and corrosion potential to buried steel elements).	Moderate
7	Accuracy of existing topographic information is unknown.	Low

As per the attached plan, three additional boreholes are proposed to be advanced around the location of the new parking garage.

Materials Management Addition

A new single story (one-level) Materials Management Addition, rectangular in shape, approximately 45 x 60 m, is proposed to be built north of the existing main hospital building, at the location of the existing materials loading area.

WSP found records of fourteen boreholes that were completed around the proposed area:

- BH72-16: located at the southern limit of the proposed area.
- BH91-1 to BH91-4: located along the eastern limit of the proposed area.
- BH98-3: located in the southwestern corner of the proposed area.
- BH04-08: located in the southwestern corner of the proposed area.
- BH04-09: located in the south center of the proposed area.
- BH04-10: located in the east side of the proposed area.
- BH04-11: located in the southwestern corner of the proposed area.
- BH04-12: located in the south center of the proposed area.
- BH04-13: located in the southeastern corner of the proposed area.
- BH05-8: located in the northeast side of the proposed area.
- BH07-10: located in the northeast side of the proposed area.

Based on the borehole logs, the ground elevation is mostly flat and varies between about 73.9 and 75.6 masl, except at borehole BH72-16 where the ground elevation was 77.6 masl, indicating that a cut might have happened in the area after 1972.

In general, the subsurface conditions consisted of topsoil or asphalt and/or fill, underlain by granular and/or cohesive soil deposits, underlain by glacial till and dolostone bedrock (typical site profile).

Topsoil, where encountered, was approximately 300 mm thick.

A pavement structure was encountered in most boreholes and extended to depths varying between about 0.9 to 2.0 m (73.9 to 72.4 masl). It comprised asphalt layers overlying crushed limestone fill and sand and gravel fill; clay and pieces of rock and crushed stone were noted in the fill. SPT ‘N’ values varying between about 8 and 92 were noted, indicating loose to very dense fill layers, but more generally compact (high “N” values likely indicative of stoney inclusions).

The granular deposit was encountered in two boreholes and consisted primarily of silt and sand in varying amounts. The deposit extended to a maximum depth of 1.5 m (i.e., 73.4 to 72.6 masl depending on location). An SPT ‘N’ value of 20 was noted within this layer, indicating it is a compact material.

The cohesive soil deposit was encountered in six of the noted boreholes and comprised very stiff to soft sandy clay to silty clay with very fine sand seams. The deposit extended to depths varying between approximately 1.9 to 4.6 m (i.e., 77.6 to 72.3 masl). A torque vane test was completed at borehole BH72-16 and reported an in situ undrained shear strength of approximately 70 kPa. The moisture content of material sampled in borehole BH72-16 varied between approximately 30 to 50% based on laboratory tests (inferred to be wetter than the plastic limit for this material).

The glacial till was encountered in five boreholes and was predominantly sandy textured according to the logs. The till extended to depths varying between approximately 1.6 and 5.4 m (i.e., 73.0 to 72.3 masl), and was described as medium dense to dense.

Bedrock sampled in borehole cores comprised grey dolostone and was encountered at depths varying between about 1.5 and 5.4 m (i.e., 73.1 to 72.2 masl). The bedrock was cored at boreholes BH72-16, BH04-10 to BH04-12, and BH05-8 to depths varying between about 3.1 to 6.9 m (71.1 to 70.6 masl), with reported core recovery ranging between 80 to 100%.

Groundwater levels were reported in boreholes BH72-16, BH98-3, BH04-09 to BH04-13 and BH05-8, and varied between elevation 72.8 to 73.1 masl, except at borehole BH72-16 which reported a groundwater elevation of 76.0 masl.

The following table summarizes identified information gaps that are relevant to the geotechnical design of the proposed materials management addition.

Table 2 – Information Gap Analysis – Materials Management Addition

Gap No.	Description of Information Gap	Degree of Importance
1	Available borehole logs date from 1972 to 2007. Information on any excavation, construction or grade raise that might have happened since then is not readily available. The exact field work methodology of past investigations is unknown. A refresh of the subsurface condition information at the site is recommended.	High
2	Qualities and characteristics of the overburden soil across the proposed area are limited. - Only one SPT ‘N’ value available for the granular deposit. - Only one shear strength test is available for the cohesive deposit. - Further testing is recommended to assess the extent of reported loose or soft soils.	Moderate
3	Limited information exists regarding depth and characteristics of the bedrock underlying the site. - No RQD values available. - The extent of weathered bedrock is unknown.	Moderate
4	Laboratory testing information is limited to non-existent. - No records of any grain sizes, Atterberg limits or UCS rock testing are available.	Moderate
5	Soil corrosion potential is unknown at the proposed area (sulphate damage to concrete elements and corrosion potential to buried steel elements).	Moderate
6	The reuse potential of existing pavement structure fills is unknown.	Low
7	Accuracy of existing topographic information is unknown.	Low

As per the attached plan, three additional boreholes are proposed to be advanced around the location of the new Materials Management Addition.

Temporary Loading Dock

A new Temporary Loading Dock, with an "L-shape" approximately 22 x 22 m in area, is proposed to be built southwest of the existing QCH Cancer Centre. The Temporary Loading Dock is to be connected to the west of the re-aligned materials loading area.

WSP found records of three boreholes that were completed around the area of this proposed structure:

- BH72-13: located slightly outside the northwestern corner of the proposed area.
- BH72-20: located at the northeastern corner of the proposed area.
- BH07-7: located at the northwestern corner of the proposed area.

Based on the borehole logs, the ground elevation varies between about 74.8 and 76.6 masl.

In general, the subsurface conditions at boreholes BH72-13 and BH72-20 comprised topsoil underlain by a cohesive deposit, granular deposit / glacial till layer, and dolostone bedrock. At borehole BH07-7 pavement structure was found directly overlying the bedrock.

Topsoil, where encountered, was clayey and approximately 300 mm thick.

The noted pavement structure extended to a depth of 2.2 m (72.6 masl) and comprised asphalt overlying crushed stone fill and compact to loose sand and gravel fill with pieces of crushed limestone.

A cohesive deposit comprising very stiff to soft sandy clay to silty clay with very fine sand seams was found to depths of between about 1.5 and 4.3 m (i.e., 74.6 to 72.2 masl). Torque (shear) vane tests were completed and reported undrained shear strengths of between 35 to 70 kPa for the in-situ material. Moisture content varied between approximately 25 to 55% in laboratory test samples.

The granular deposit was only encountered in borehole BH72-13 and consisted of loose to medium dense silty fine sand with little gravel. This material extended to a maximum depth of 3.3 m (72.8 masl) and is loose to compact based on a SPT 'N' value of 12.

Glacial till was only encountered in borehole BH72-20 and comprised dense sandy silty gravel. The till extended to a depth of 5.0 m (71.6 masl) and a SPT 'N' value of 40 was reported for this layer.

Dolostone bedrock was encountered at depths varying between about 2.2 and 5.0 m (i.e., 72.8 to 71.6 masl) and was cored to depths varying between about 3.7 to 6.5 m (i.e., 71.3 to 70.1 masl). Reported core recoveries varied between 87 and 100% and reported RQD varied between 47 and 100% (indicating Poor to Excellent quality material).

Groundwater elevations were measured in boreholes BH72-13 and BH72-20 between approximately 75.2 and 75.7 masl.

The following table summarizes identified information gaps that are relevant to the geotechnical design of the proposed temporary loading dock.

Table 3 – Information Gap Analysis – Temporary Loading Dock

Gap No.	Description of Information Gap	Degree of Importance
1	Available borehole logs date from 1972 to 2007. Information on any excavation, construction or grade raise that might have happened since then is not readily available. The exact field work methodology of past investigations is unknown. A refresh of the subsurface condition information at the site is recommended.	High
2	Qualities and characteristics of the overburden soil across the proposed area are limited. - Limited number of SPT ‘N’ values. - Available overburden thickness information across the proposed area is limited. All existing boreholes are towards the north of the proposed area. - Further testing is recommended to assess the extent of loose and soft soils.	Moderate
3	Limited information exists regarding depth and characteristics of the bedrock underlying the site. - All existing boreholes are towards the north of the proposed area.	Moderate
4	Groundwater levels/gradient across the proposed area are unknown. - Available water levels information date from 1972.	Moderate
5	Laboratory testing information is limited to non-existent. - No records of any grain sizes, Atterberg limits or UCS rock testing are available.	Moderate
6	Soil corrosion potential is unknown at the proposed area (sulphate damage to concrete elements and corrosion potential to buried steel elements).	Moderate
7	The reuse potential of existing pavement structure fills is unknown.	Low
8	Accuracy of existing topographic information is unknown.	Low

As per the attached plan, two additional boreholes are proposed to be advanced around the location of the temporary loading dock.

Emergency Department Addition

A new Emergency Department Addition, also ‘L’ shaped and approximately 60 x 100 m in area, is proposed to be built east of the main hospital building, at the location of the existing eastern paved entrance and paved access lane. An ambulance parking area is to be built northeast of the proposed Emergency Department Addition.

WSP found records of five boreholes that were completed around the proposed area:

- BH72-12: located at the northwestern corner of the proposed area.
- BH98-4: located slightly outside the southeastern corner of the proposed area.
- BH05-1 to BH05-3: located along the northern limit of the proposed area.

Based on the borehole logs, the ground elevation varies between about 76.6 and 78.6 masl.

Subsurface conditions consisted of topsoil or asphalt and/or fill, underlain by a cohesive soil deposit, glacial till, and bedrock.

Topsoil, where encountered, was clayey and approximately 150 to 300 mm thick.

The pavement structure was encountered in three boreholes and extended to depths varying between about 0.3 to 1.5 m (i.e., 76.7 to 76.4 masl). Pavement structure comprised asphalt overlying crushed stone fill and/or sand and gravel fill.

Clayey fill with topsoil was encountered in boreholes BH05-1 to BH05-3 to depths varying between about 1.5 to 3.0 m (i.e., 75.6 to 74.5 masl).

The cohesive deposit comprised hard to relatively soft sandy clay to silty clay with fine sand seams and extended to depths varying between 5.0 and 9.0 m (i.e., 72.6 to 69.6 masl). Torque (shear) vane tests were completed at select locations and reported undrained shear strengths varying between about 220 to 40 kPa. Pocket penetrometer readings in similar sampled materials widely varied between 400 and 20 kPa. Moisture content ranged between approximately 40 and 55% based on laboratory tests.

The glacial till was only encountered in borehole BH05-3 and consisted of very dense sandy textured material. The till was encountered at a depth of 9.0 m (69.0 masl) and was approximately 600 mm thick at this borehole location.

Dolostone bedrock was encountered at depths varying between approximately 5.0 and 9.6 m (i.e., 72.1 to 69.0 masl), and was core sampled to depths of approximately 6.5 to 11.1 m (i.e., 70.6 to 67.4 masl). Core recoveries varied from between 93 to 95%.

Groundwater elevations were measured in all five boreholes between approximately 74.6 and 75.7 masl.

The following table summarizes identified information gaps that are relevant to the geotechnical design of the proposed emergency department addition.

Table 4 – Information Gap Analysis – Emergency Department Addition

Gap No.	Description of Information Gap	Degree of Importance
1	Available borehole logs date from 1972 to 2005. Information on any excavation, construction or grade raise that might have happened since then is not readily available. The exact field work methodology of past investigations is unknown. A refresh of the subsurface condition information at the site is recommended.	High
2	Qualities and characteristics of the overburden soil across the proposed area are limited. - Extent of clay fill. - Further testing is recommended to assess the extent of soft soils.	Moderate
3	Limited information exists regarding depth and characteristics of the bedrock underlying the site. - The bedrock elevation appears variable across the proposed location.	Moderate
4	Laboratory testing information is limited to non-existent. - No records of any grain sizes, Atterberg limits or UCS rock testing are available.	Moderate
5	Soil corrosion potential is unknown at the proposed area (sulphate damage to concrete elements and corrosion potential to buried steel elements).	Moderate
6	The reuse potential of existing pavement structure fills is unknown.	Low
7	Accuracy of existing topographic information is unknown.	Low

As per the attached plan, three additional boreholes are proposed to be advanced around the location of the new emergency department addition.

Urgent Care Centre Addition

A new Urgent Care Centre addition (parallelogram shape, approximately 25 x 25 m) is proposed to be built at the southeastern corner of the main hospital building, at the location of the existing paved entrance.

WSP found records of two boreholes and a test pit that were completed around the proposed area:

- TP98-8: located at the southwestern corner of the proposed area.
- BH04-3: located at the western limit of the proposed area.

Based on the testhole logs, the ground elevation is mostly flat and varies between about 78.6 and 79.0 masl.

In general, the subsurface conditions consisted of topsoil and/or fill, underlain by a cohesive deposit.

The topsoil was encountered in TP98-8 and was approximately 250 mm thick.

The fill was encountered in both testholes and consisted of variable amounts of sand, clay, gravel and silt with debris. It extended to depths of 2.3 and 2.4 m (76.7 to 76.2 masl). Variable SPT 'N' values of 47, 12, 6, 11, and 10 were noted within this layer.

The cohesive deposit comprised stiff to soft clay to silty clay and extended to depths of approximately 3.1 to 5.1 m (i.e., 75.5 to 73.9 masl). A torque vane test was completed and reported an undrained shear strength of approximately 45 kPa. Pocket penetrometer readings in recovered split spoon samples varied widely between 335 to 25 kPa. Moisture content was approximately 40% based on laboratory tests.

At borehole BH04-3, auger refusal on probable bedrock was noted at approximately 9.5 m depth (69.5 masl). Test pit TP98-8 was terminated at 3.1 m depth (75.5 masl) without reaching refusal.

A groundwater level elevation of 75.9 masl was measured at borehole BH04-3.

The following table summarizes identified information gaps that are relevant to the geotechnical design of the proposed urgent care centre addition.

Table 5 – Information Gap Analysis – Urgent Care Centre Addition

Gap No.	Description of Information Gap	Degree of Importance
1	Available borehole and test pit logs date from 1998 to 2004. Information on any excavation, construction or grade raise that might have happened since then is not readily available. The exact field work methodology of past investigations is unknown. A refresh of the subsurface condition information at the site is recommended.	High
2	Qualities and characteristics of the overburden soil across the proposed area are limited. - Limited number of SPT 'N' values. - Overburden thickness across the proposed area is limited. All existing boreholes are towards the west of the proposed area. - Further testing is recommended to assess the extent of soft soils.	Moderate
3	Limited information exists regarding depth and characteristics of the bedrock underlying the site. - Auger refusal depth available for one borehole only. - No rock coring was conducted.	Moderate
4	Groundwater levels/gradient information across the proposed area is limited.	Moderate
5	Laboratory testing information is limited to non-existent. - No records of any grain sizes, Atterberg limits or UCS rock testing are available.	Moderate
6	Soil corrosion potential is unknown at the proposed area (sulphate damage to concrete elements and corrosion potential to buried steel elements).	Moderate
7	Accuracy of existing topographic information is unknown.	Low

As per the attached plan, one additional borehole is proposed to be advanced around the location of the new urgent care centre addition.

Realigned Loading Area

The existing loading area is planned to be moved north to make place for the new materials management addition.

WSP found records of one borehole that was completed around the proposed area:

- BH07-12: located at the center of the proposed area.

Based on the borehole log, the ground elevation is approximately 77.4 masl.

In general, the subsurface conditions are typical for the site and consist of topsoil overfill, underlain by a cohesive deposit, and a granular deposit.

The topsoil was approximately 50 mm thick.

The fill consisted of sand and gravel to 0.6 m (76.8 masl) and loose sandy silt with some clay and trace gravel to 1.3 m (76.1 masl). An SPT ‘N’ value of 9 was noted within this layer.

The cohesive deposit consisted of a very stiff to stiff silty clay with sand seams. It extended to a depth of about 2.7 m (74.7 masl).

The granular deposit consisted of loose to dense silty fine sand and extended to auger refusal at 4.0 m depth (73.4 masl).

Auger refusal on probable bedrock was noted at approximately 4.0 m (73.4 masl) depth.

A groundwater level elevation of 74.2 masl was measured in borehole BH07-12 (open hole measurement during drilling).

The following table summarizes identified information gaps that are relevant to the geotechnical design of the proposed urgent care centre addition.

Table 6 – Information Gap Analysis – Realigned Loading Area

Gap No.	Description of Information Gap	Degree of Importance
1	Available borehole log dates from 2007. Information on any excavation, construction or grade raise that might have happened since then is not readily available. The exact field work methodology of past investigations is unknown. A refresh of the subsurface condition information at the site is recommended.	High
2	Qualities and characteristics of the overburden soil across the proposed area are limited. - Limited number of SPT 'N' values. - Overburden thickness information across the proposed area is limited. - Further testing is recommended to assess the extent of loose soils.	Moderate
3	Limited information exists regarding depth and characteristics of the bedrock underlying the site. - Auger refusal depth available for one borehole only. - No rock coring was conducted.	Moderate
4	Groundwater levels/gradient information across the proposed area is limited.	Moderate
5	Laboratory testing information is limited to non-existent. - No records of any grain sizes, Atterberg limits or UCS rock testing are available.	Moderate
6	Soil corrosion potential is unknown at the proposed area (sulphate damage to concrete elements and corrosion potential to buried steel elements).	Moderate
7	Accuracy of existing topographic information is unknown.	Low

As per the attached plan, one additional borehole is proposed to be advanced around the location of the realigned loading area.

New Road System

A new road system, approximately 400 to 500 m long, is proposed to be built on the west side of the QCH complex, connecting John Sutherland Drive at the north to Baseline Road at the south. A grassy area with trees and paved bike lanes exists at the proposed area.

No available borehole records were found around the proposed area.

As per the attached plan, three additional boreholes are proposed to be advanced along the location of the new road.

General Findings

Based on the 2008 Golder report, corrosivity analyses on samples of the cohesive deposit and glacial till from boreholes BH08-302 and BH08-307 indicate that concrete made with Type GU Portland cement should be acceptable. The past performance of older existing foundations exposed at the time of the 2008 investigation would support this expectation. The results also indicate moderate levels of corrosivity for buried ferrous metals with significant variations over the site possibly affected in part by de-icing chemicals in parking areas. It is to be noted that the 2008 boreholes are not in the immediate location of any of the newly proposed features and that the corrosivity levels noted at these boreholes might not be representative of the entire QCH site

Based on that same report, point load index tests carried out of the dolostone cores retrieved from the 2008 investigation resulted in an average UCS of about 153 MPa, indicating a very strong R5 bedrock. It is to be noted that the 2008 boreholes are not in the immediate location of any of the newly proposed features. Based on the 2009 Golder report, previous site investigations revealed soil-filled vertical joints or clefts at a few locations in the dolomitic bedrock. The infilling generally consisted of dense to very dense glacial till and the widths of the joints were found to range from about 100 to 600 mm. It was noted that removal of the bedrock at the site would require drill and blast techniques.

Based on seismic Vertical Soil Profiling (VSP) completed in 2008 at borehole BH08-307, the shear wave velocity of the bedrock increases from about 1000 m/s at a depth of about 3 m to 2500 m/s at a depth of about 8 m. A Site Class A was given for footing type foundations bearing directly on the dolostone bedrock. Overburden shear wave velocities must be considered for shallower structures (e.g., slabs, pile caps) bearing on overburden.

Based on the 2009 Golder report, the site has been considerably reworked at several locations over the years due to construction activities for additions to the QCH campus, which, according to the report, would explain the presence of areas with a significant amount of fill deposits.

The 2009 Golder report indicated that one-storey slab-on-grade structures can generally be supported on conventional spread footings within the natural undisturbed clay soils. Heavier structures would require footings bearing on bedrock or deep foundation elements such as end bearing piles or caissons. The 2008 Golder report discussed geotechnical recommendations for the construction of a one- to four-level Surgical Addition to be built adjacent to the main existing building at its northwestern corner. The report noted that the existing structure was supported on concrete filled pipe piles driven to bedrock. The slab on the grade of the existing structure was based at a geodetic elevation of 79.25 m. The Surgical Addition was to be supported on footings placed directly on the dolostone bedrock. An excavation below the existing pile caps and adjacent to the existing pipe piles would have been required. The as-built pile driving records (McRostie Genest St-Louis, 1973-74), attached at the end of this document, indicate that the piles of the existing main building were driven to elevations varying between about 70.9 to 74.0 m, except pile 347A which was driven deeper to elevation 64.0 m.

Recommended Geotechnical Program

To close the above noted information gaps and provide geotechnical recommendations, a geotechnical exploration program should be carried out for the design and construction of the proposed features. The 16-borehole plan proposed by the Client is considered adequate. Indeed, given the “age” of the available historical subsurface information, and given how scattered and inconsistent the information is across multiple boreholes and investigations, WSP could not justify reducing the number of proposed boreholes and the extent of the scope of work. The extent of the expansion project across the QCH site, the presence of potentially significant amounts of fill, the presence of loose and soft soils, the variability of the bedrock profile, the limited information on the groundwater level and the lack of geotechnical laboratory information are all important factors to be considered and to be properly assessed for an efficient and safe design.

The proposed drilling program includes:

- Preparation of a health and safety plan.

- Request of public and private locates. This will include a site visit to layout the boreholes as per the plan. The boreholes may be moved slightly from their original position to avoid drilling through underground services, to facilitate drilling setup and/or to minimize damage to existing features (landscaping, sidewalks, pavement, etc.)

- The mobilization of a geotechnical drilling rig and qualified personnel.

- The drilling and sampling of 16 boreholes to auger refusal. Six boreholes will include 3 to 4.5 m rock coring to sample and verify the bedrock condition at the location of the new parking garage and building additions.

- Soil will be sampled at regular depth intervals and in-situ testing including SPT and shear vane testing will be completed in accordance with standard industry practices. Shelby tubes may be advanced in soft to firm clay soils, if encountered, to collect relatively undisturbed samples. The entire field program will be supervised by a qualified member of WSP's geotechnical staff.

- Monitoring wells will be installed in up to 8 boreholes (6 in overburden soil, 2 in bedrock) to determine local groundwater levels following completion of the drilling program. The water levels will be allowed to stabilize after drilling for a period of approximately 2 weeks before groundwater readings are taken. Levels in existing monitoring wells, if any, will also be checked.

- Laboratory testing program to obtain site-specific parameters required for geotechnical design recommendations, including physical and chemical properties of site soils. Chemical testing (sulphate content, pH, soil resistivity, and chloride content) will be carried out on three selected soil samples from the site to determine the potential for sulphate attack and appropriate cement types per CSA A23.1, as well as the potential for corrosion of buried steel elements (e.g. AWWA rating system).

Given the depth variability of the bedrock profile and considering the distance between all newly proposed project features and the historical VSP borehole BH08-307, additional shear wave velocity testing at the site may prove beneficial, especially if basement levels and/or deep foundations are to be considered for the building additions. If a proposed feature is not to be found directly on bedrock, the soil profile may dominate seismic behavior. The materials must therefore be accurately characterized. Indeed, based on the available information on the subsurface conditions at the Site, it is expected that Seismic Site Classes E to C would apply, depending on the location. Site-specific shear wave velocity measurements are required per NBCC (2020) and OBC (2019) for Site Classes A and B. WSP proposes to include additional VSP testing to the scope of work, at up to two of the sixteen proposed boreholes (BH24-03 and BH24-15), for an additional cost of $6,000 per borehole, to potentially justify a higher Site Class. Client approval of a scope change is required, as this testing was not anticipated prior to the desktop review. VSP testing requires that a 2'' PVC pipe be installed and grouted in place in a borehole, with the pipe being encased in at least 6 m of rock to provide adequate results.

Environmental considerations are outside of the geotechnical scope of work. These include the presence of rare or endangered species at the site and the presence of or proximity to Areas of Potential Environmental Concern (APECs) which may contain contaminated soils or groundwater. During drilling, WSP will make note of any potential contaminant indicators in the sampled material, such as discolouration, staining, sheens, odours, etc. Species at risk assessments and environmental site assessments can be completed by WSP if required, separately from the aforementioned proposed geotechnical investigation.

Closure

We trust that this desktop study provides sufficient information for your present requirements. If you have any questions concerning this study, please do not hesitate to contact the undersigned.

Yours truly,

WSP Canada Inc.

Othmane Benkirane, MASc., ing., P.Eng.	J. Stephen Ash, P.Eng. P. Geo.
Geotechnical Engineer	*Senior Principal Geotechnical Engineer*

OB/SA/yj

Attachments: Figure 1 – Site Plan
Figure 2 - Historical Borehole Site Plan, Project No. 07-1121-0002, prepared by Golder, dated August 05, 2009
Relevant Historical Testhole Logs
As-Built Pile Driving Records, Report SF-1177B Part A and B, prepared by McRostie Genest St-Louis, dated March 13, 1974

https://wsponline-my.sharepoint.com/personal/yashika_jindal_wsp_com/documents/desktop/othmane/ca0033714.1722 - qch expansion - geoetchnical desktop study_jsa rev 2.docx

KEY MAP

SCALE 1:250,000

LEGEND

⊕	PROPOSED BOREHOLE LOCATION
—	SITE LAYOUT

0 20 40 80

1:2,000 METRES

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. CONTAINS INFORMATION LICENSED UNDER THE OPEN GOVERNMENT LICENCE - ONTARIO
2. IMAGERY CREDITS: SOURCES: ESRI, HERE, GARMIN, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), (C) OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
3. COORDINATE SYSTEM: NAD 1983 UTM ZONE 18N

CLIENT
QUEENSVIEW CARLTON HOSPITAL

PROJECT
QUEENSVIEW CARLETON HOSPITAL EXPANSION PROJECT

TITLE
SITE PLAN

CONSULTANT

wsp

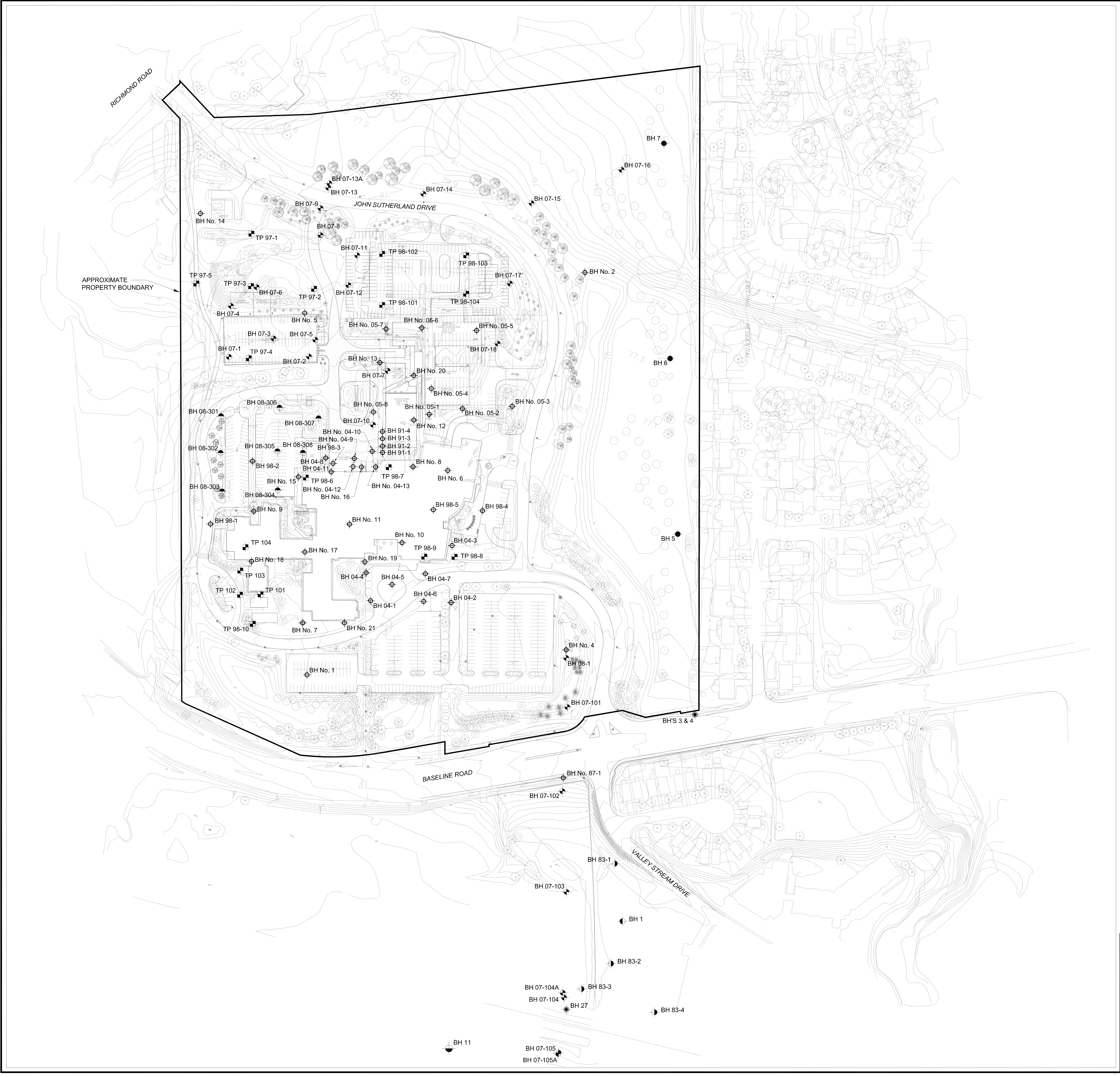
YYYY-MM-DD	2024-06-04
DESIGNED	----
PREPARED	CGE
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
CA0033714.1722	0001	A	1

PATH: S:\Clients\City_of_Ottawa\Queensway_Carleton_Hospital\99_PROJ\CA0033714_1722\40_PROD\0001_BHPlan\CA0033714_1722-0001-BG-0001.mxd PRINTED ON: 2024-06-04 AT: 10:46:01 AM

25mm

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B



LEGEND

◓	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD. REPORT No. 07-1121-0002-7000 (OCT. 2008)
◍	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD. REPORT No. 07-1121-0002-1 (JUNE 2007)
◒	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 931-2007
◉	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 851-2546
◑	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 831-2310
◐	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 73711
●	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 72766 (JUNE 1972)
⊕	BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATIONS BY McROSTIE GENEST ST-LOUIS
▚	TEST PIT LOCATION IN PLAN, PREVIOUS INVESTIGATIONS BY McROSTIE GENEST ST-LOUIS

REFERENCE

BASE PLAN SUPPLIED BY PARKIN ARCHITECTS LIMITED

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 07-1121-0002-9000

20 0 20 40
SCALE 1:1000 METRES

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
0	01/01/01	XXX	REV_DESC	XXX		

PROJECT
GEOTECHNICAL BACKGROUND INFORMATION 2009
QUEENSWAY-CARLETON HOSPITAL
OTTAWA, ONTARIO

TITLE

SITE PLAN

Golder Associates
Ottawa, Ontario

PROJECT No. 07-1121-0002		
DESIGN		
CADD	J.M.	5 AUG. 09
CHECK	M.St.L	11 AUG. 09
REVIEW	G.S.W.	11 AUG. 09

FILE No.0711210002-9000-02.dwg
SCALE 1:1,000 REV.

FIGURE 2

& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

BASELINE WEST OF SIOUX CRES.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZERO) 253.1' DATE FEB. 9, 1972

HOLE / FORAGE No. 12

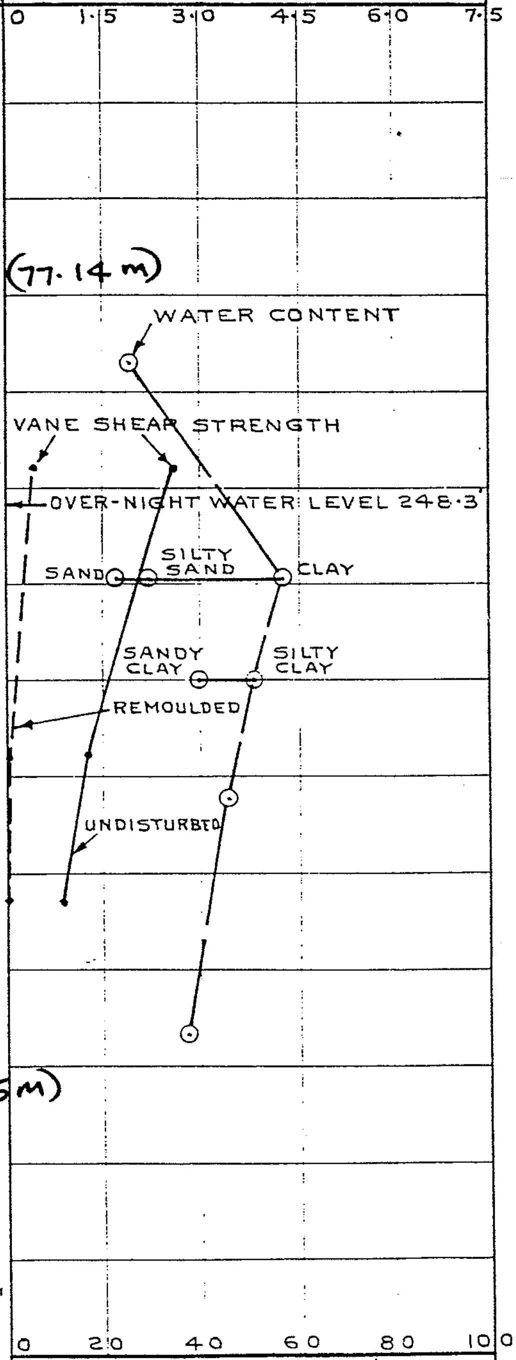
NOTES SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd	Sample No. Echantillon	DESCRIPTION OF SOIL DU SOL Ground Surface – Niveau du Sol	Depth in Feet Profondeur-Pied	Elevation Niveau
				CLAYEY TOPSOIL	0'	253.1' (77.14 m)
	8.0,8.2,8.5	29	12_1	HARD SANDY BROWNISH GRAY CLAY	1.0'	252.1'
	3.4,3.6,4.0 3.8,3.6,3.8 4.2,4.4,4.2 2.8,2.6,3.0 R_0.4		12_2	STIFF SANDY BROWNISH GRAY CLAY WITH SOME 1/32" TO 1" FINE SAND LAYERS & A 1/2" SILTY SAND LAYER	2.5'	250.6'
	0.5,1.0,1.2	1	12_3	MEDIUM SOFT SILTY GRAY CLAY WITH A 1" SANDY GRAY CLAY LAYER	7.5	245.6'
	0.5,0.8,0.6	2	12_4	SOFT SANDY GRAY CLAY	9.0	244.1'
		5	12_5			
2" DROP & 50% OF WATER LOST AT EL. 234.5'				DOLOMITE ROCK CORE RECOVERY 93%	16.4	236.7' (72.15m)
				BOTTOM OF HOLE	21.4	231.7'

~~PROBING OR~~ VANE TEST – ~~SONDAGE OU~~ ESSAI AU MOULINET

MARTEAU----HAMMER
CHUTE LIBRE---DROP
NO CASING SANS TUBAGE
BARRE-----DIA. ROD

~~BLOWS/FOOT OR~~ SHEAR STRENGTH K.S.F.
~~COUPS/PIED OU~~ RÉSISTANCE AU CISAILLEMENT K/PD.2



WATER CONTENT
% TENEUR EN EAU
NATURAL / NATURELLE ⊙
LIQUID LIMIT / LIMITE DE LIQUIDITÉ ☐
PLASTIC LIMIT / LIMITE DE PLASTICITÉ △

PLATE / PLAQUE No. 6 (1991) ~~12~~

R = REMOULDED-REMANIE
CR = CORE RECOVERY / CAROTTE RECUPEREE

McROSTIE SETO GENEST
& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

BASELINE WEST OF SIOUX CRES.

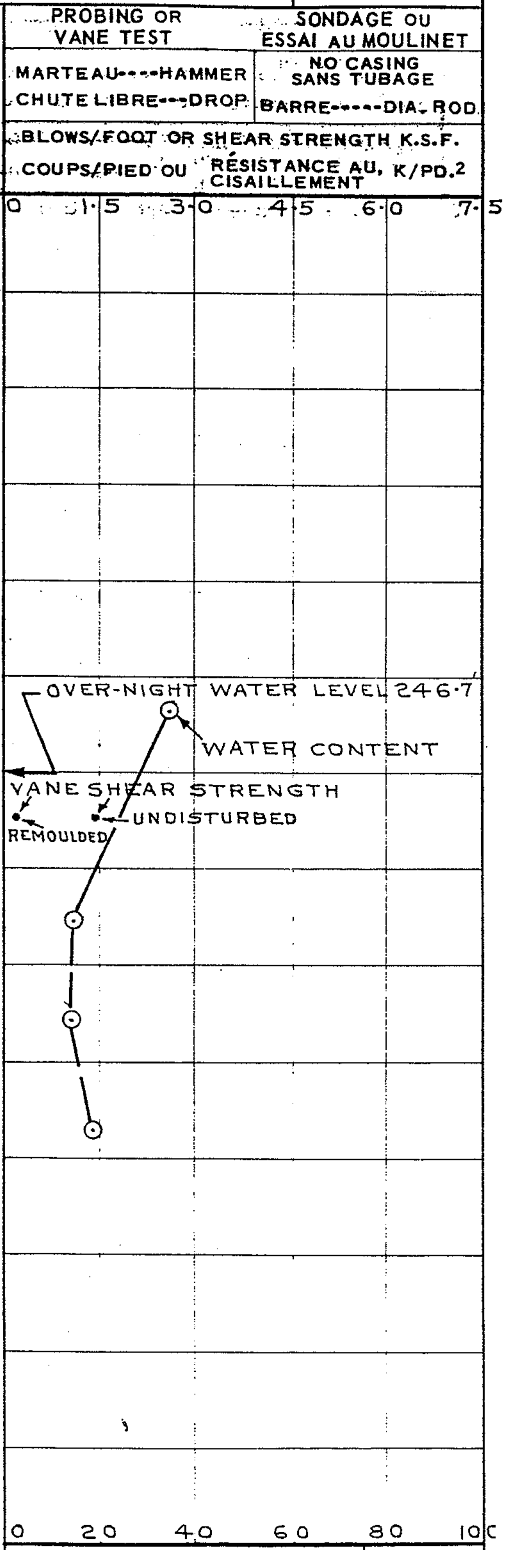
ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZERO) 249·8′

DATE FEB. 9, 1972

HOLE / FORAGE No. 13

NOTES SEE PLATE No.2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd.	Sample No. Echantillon	DESCRIPTION OF SOIL DU SOL	Depth in Feet Profondeur-Pied	Elevation Niveau
				Ground Surface – Niveau du Sol	0′	249·8′
				CLAYEY TOPSOIL	1·0′	248·8′
	6·6, 7·2, 7·4	13	13_1	VERY STIFF BROWNISH GRAY CLAY WITH SOME SAND & A LITTLE TOPSOIL	5·0′	244·8′
			13-2	LOOSE TO MEDIUM DENSE SILTY FINE SAND WITH A LITTLE GRAVEL		
		12	13_3			
		5 for 6″	13_4		10·9′	238·9′
1″ DROP & ALL WATER LOST AT EL. 235·5′				DOLOMITE ROCK CORE RECOVERY 87%	15·9′	233·9′
				BOTTOM OF HOLE		



% WATER CONTENT / TENEUR EN EAU
NATURAL / NATURELLE ⊙
LIQUID LIMIT / LIMITE DE LIQUIDITÉ ⊡
PLASTIC LIMIT / LIMITE DE PLASTICITÉ △

PLATE / PLAQUE No. 13

R = REMOULDED-REMANIE
CR = CORE RECOVERY / CAROTTE RECUPEREE

MCROSTIE SETO GENEST
& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

BASELINE RD. WEST OF SIOUX CRES.

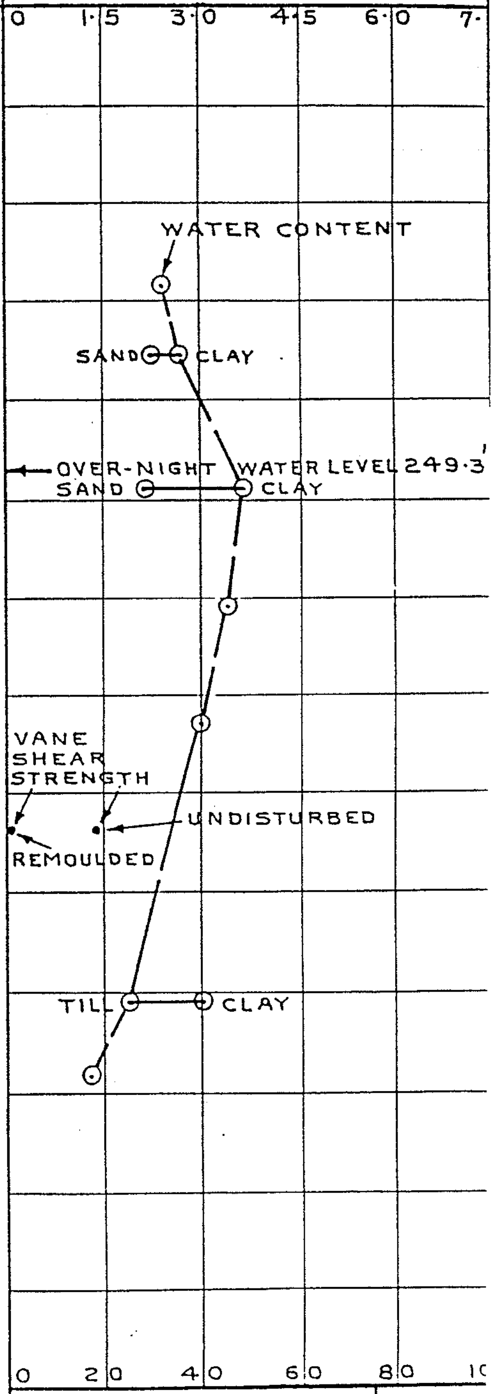
ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZERO) 254·7' DATE MAR. 27, 1972

HOLE / FORAGE No. 16

NOTES SEE PLATE No.

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd.	Sample No. Echantillon	DESCRIPTION OF SOIL DU SOL — Ground Surface – Niveau du Sol	Depth in Feet Profondeur - Pied	Elevation Niveau
					0'	254·7'
		12	16-1	VERY STIFF SANDY BROWNISH GRAY CLAY WITH SOME SILT	2·5'	252·2'
		10	16-2	VERY STIFF SANDY BROWNISH GRAY CLAY WITH SOME SILT & WITH A FEW 1/8" TO 1/4" CLAYEY FINE SAND POCKETS & LAYERS	5·0'	249·7'
		4	16-3	STIFF SILTY BROWNISH GRAY CLAY WITH SOME VERY FINE SAND & WITH A 1½" CLAYEY FINE & VERY FINE SAND LAYER	7·5'	247·2'
		2 1 for 18"	16-4 16-5	SOFT SILTY GRAY CLAY WITH A LITTLE VERY FINE SAND	15·0'	239·7'
	1·6, 1·6, 1·6 R-0·0		16-6	TILL WITH A FEW 1/4" TO 3/4" SILTY GRAY CLAY LAYERS	17·5'	237·2'
		20 for 4"	16-7	DENSE GRAVELLY TILL	17·8'	236·9'
				DOLOMITE ROCK CORE RECOVERY 90%	22·8'	231·9'
				BOTTOM OF HOLE		

PROBING OR VANE TEST — SONDAGE OU ESSAI AU MOULINET
MARTEAU----HAMMER
CHUTE LIBRE---DROP
NO CASING SANS TUBAGE
BARRE-----DIA. ROD
BLOWS/FOOT OR SHEAR STRENGTH K.S.F.
COUPS/PIED OU RÉSISTANCE AU CISAILLEMENT K/PD.2



WATER CONTENT
%TENEUR EN EAU
NATURAL NATURELLE ⊙
LIQUID LIMIT LIMITE DE LIQUIDITÉ ☐
PLASTIC LIMIT

PLATE / PLAQUE No 16

R = REMOULDED-REMANIE

McROSTIE SETO GENEST
& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

BASELINE RD. WEST OF SIOUX CRES.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) NIVEAU DU SOL (PROFONDEUR ZERO) 251.3' DATE MARCH 28, 1972

HOLE FORAGE No. 20

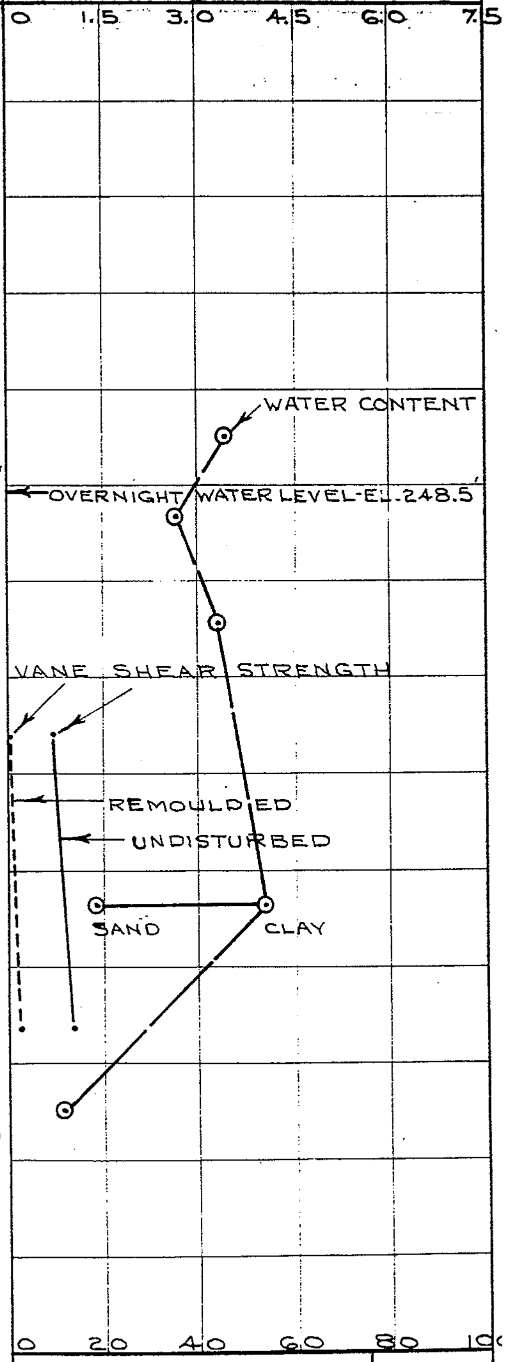
NOTES SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd	Sample No. Echantillon	DESCRIPTION OF SOIL DU SOL Ground Surface – Niveau du Sol	Depth in Feet Profondeur - Pied	Elevation Niveau
				CLAYEY TOPSOIL	0'	251.3'
		18	20-1	SANDY BROWNISH GRAY CLAY	1.0'	250.3'
		4	20-2	MEDIUM SOFT SANDY BROWNISH GRAY CLAY	2.5'	248.8'
	1.0, 1.5, 1.2	3	20-3			
	0.5, 0.5, 0.6 0.6, 0.5, 0.5 1.0, 0.6, 0.6 0.8, 0.6, 0.6 0.8, 0.8, 0.8 R-0.0		20-4	MEDIUM SOFT TO SOFT SILTY GRAY CLAY WITH A LITTLE VERY FINE SAND & WITH SOME 1/32" TO 3/4" FINE SAND LAYERS	6.5'	244.8'
		40	20-5	DENSE SANDY GRAVEL WITH A 1" SILTY TILL LAYER	14.3'	237.0'
				DOLOMITE ROCK CORE RECOVERY 95%	16.3'	235.0'
				BOTTOM OF HOLE	21.3'	230.0'

GROUND FROZEN TO EL. 248.8'

2" SOFT DRILLING AT EL. 234.5'

~~PROBING OR~~ VANE TEST
~~SONDAGE OU~~ ESSAI AU MOULINET
MARTEAU----HAMMER
CHUTE LIBRE---DROP
NO CASING SANS TUBAGE
BARRE-----DIA. ROD
~~BLOWS/FOOT OR~~ SHEAR STRENGTH K.S.F.
~~COUPS/PIED OU~~ RÉSISTANCE AU CISAILLEMENT K/PD.2



WATER CONTENT
%TENEUR EN EAU
NATURAL NATURELLE ⊙
LIQUID LIMIT LIMITE DE LIQUIDITÉ ☐
PLASTIC LIMIT LIMITE DE PLASTICITÉ △

PLATE PLAQUE No. 20

R = REMOULDED-REMANIE
CR = CORE RECOVERY CAROTTE RECUPEREE

QWEENSWAY CARLETON HOSPITAL ELECT. ROOM	B.M. (ELEV. 74.980m)geodetic: Ground	BOREHOLE No. 91-1
MILLS ROSS ARCHITECTS	floor of existing boiler room.	Project No: E-6591
START DATE: 15/05/91 -		ELEVATION 75.55 (m)

SAMPLE TYPE ■ DISTURBED-AUGER ◿ SHELBY TUBE ☒ SPLIT SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0			■			TOPSOIL		75.6
			■			75.25		
1.0			■					74.6
			■			gray CLAY		
2.0			■					73.6
	power auger refusal-		■			Bottom of hole 72.70		
3.0								72.6
4.0								71.6

McROSTIE GENEST ST-LOUIS
Ottawa, Canada

COMPLETION DEPTH 2.9 m

COMPLETE 15/05/91

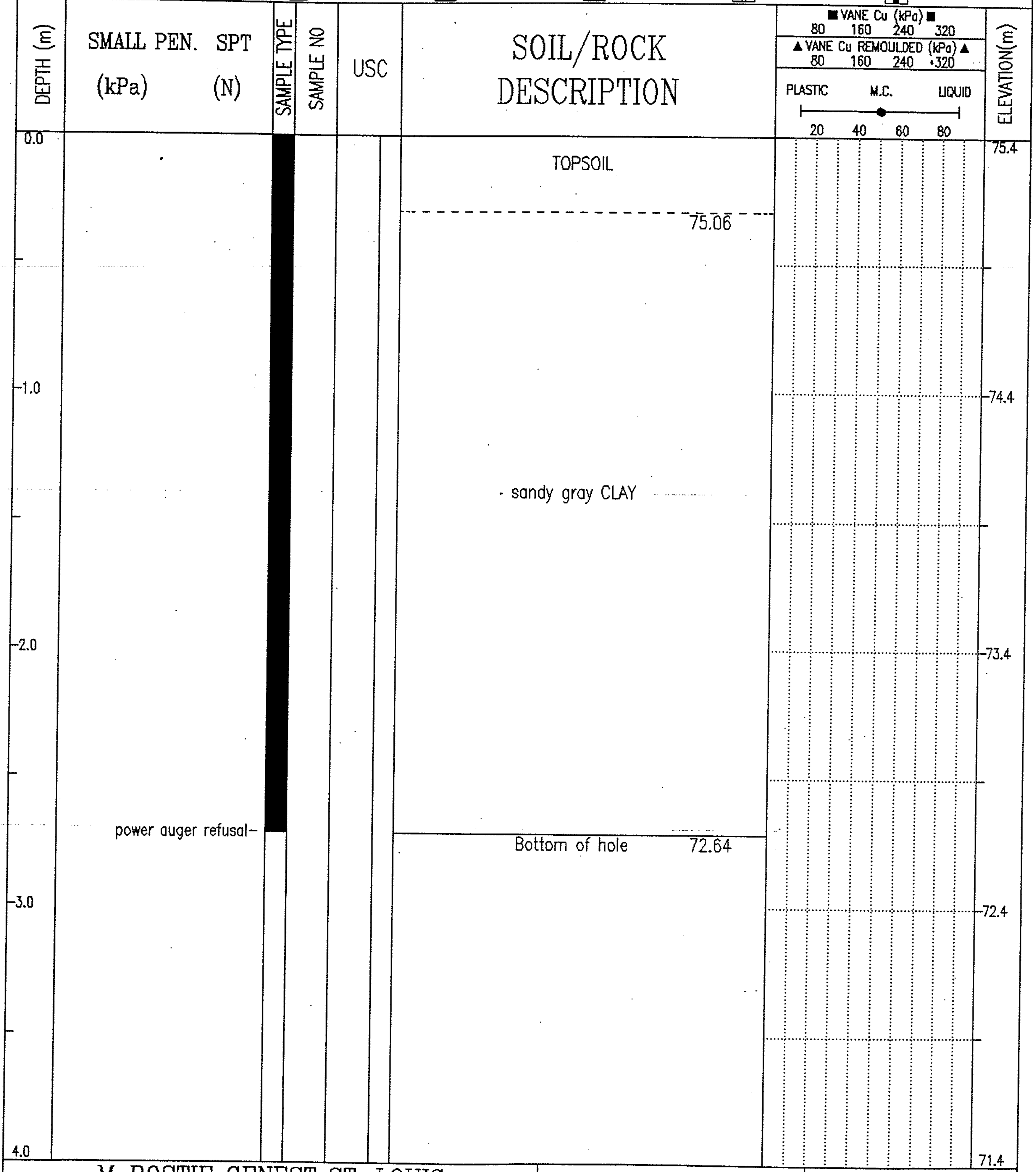
LOGGED BY JML

DWG NO. 2

Page 1 of 1

QWEENSWAY CARLETON HOSPITAL ELECT. ROOM	B.M. (ELEV. 74.980m)geodetic: Ground	BOREHOLE No. 91-2
MILLS ROSS ARCHITECTS	floor of existing boiler room.	Project No: E-6591
START DATE: 15/05/91 –		ELEVATION 75.36 (m)

SAMPLE TYPE ■ DISTURBED-AUGER ◸ SHELBY TUBE ☒ SPLIT SPOON ▤ PROBING ◫ NO RECOVERY ▮ CORE



McROSTIE GENEST ST-LOUIS Ottawa, Canada	COMPLETION DEPTH 2.7 m	COMPLETE 15/05/91
	LOGGED BY JML	DWG NO. 3
		Page 1 of 1

QWEENSWAY CARLETON HOSPITAL ELECT. ROOM	B.M. (ELEV. 74.980m)geodetic: Ground floor of existing boiler room.	BOREHOLE No. 91-3
MILLS ROSS ARCHITECTS		Project No: E-6591
START DATE: 15/05/91 -		ELEVATION 75.30 (m)

SAMPLE TYPE ■ DISTURBED-AUGER ◺ SHELBY TUBE ☒ SPLIT SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0			■			TOPSOIL		75.3
			■			75.00		
1.0			■			sandy gray CLAY		74.3
2.0			■					73.3
3.0	power auger refusal-		■			Bottom of hole 72.21		72.3
4.0								71.3

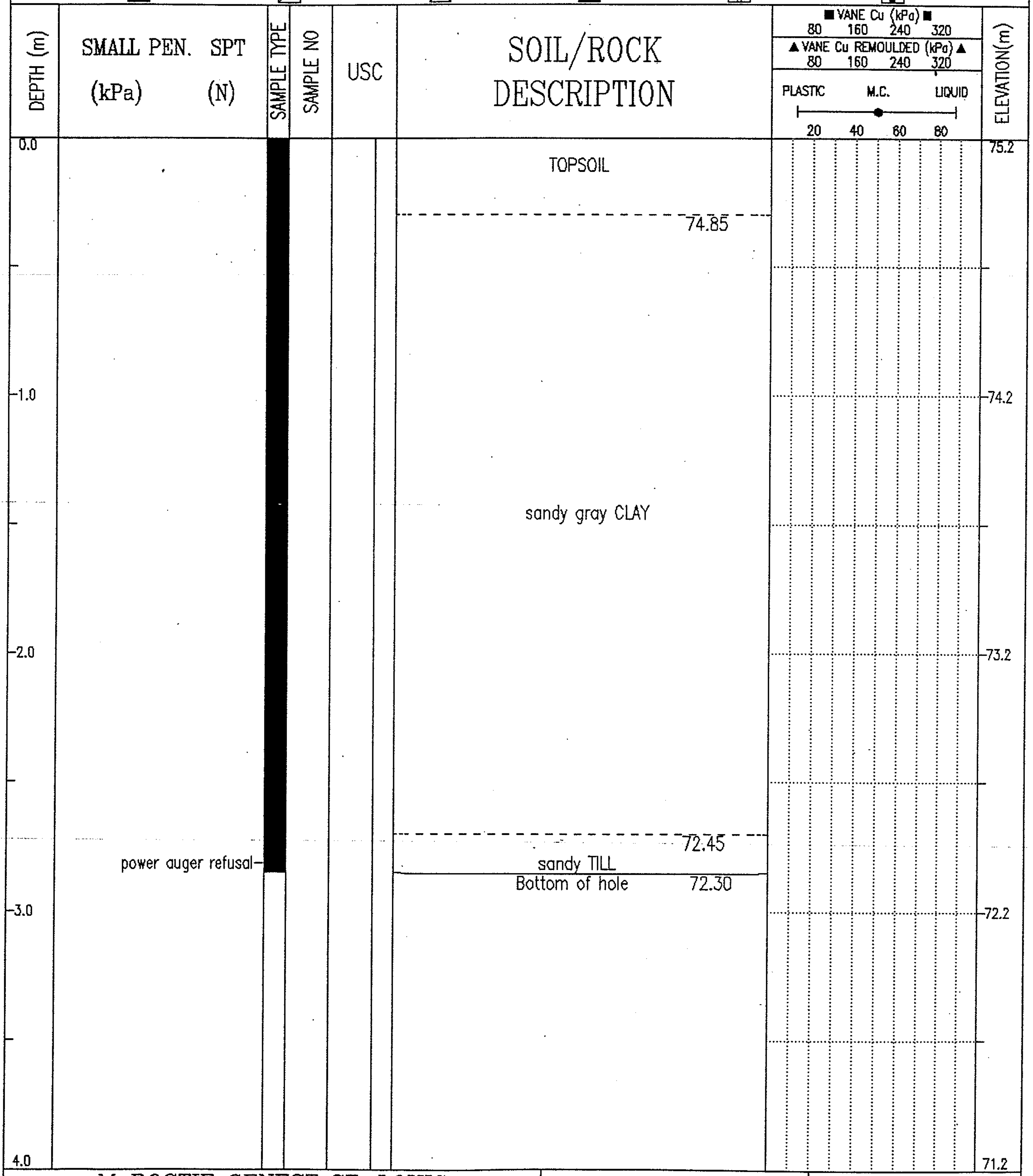
McROSTIE GENEST ST-LOUIS
Ottawa, Canada

COMPLETION DEPTH 3.1 m	COMPLETE 15/05/91
LOGGED BY JML	DWG NO. 4

Page 1 of 1

QWEENSWAY CARLETON HOSPITAL ELECT. ROOM	B.M. (ELEV. 74.980m)geodetic: Ground	BOREHOLE No. 91-4
MILLS ROSS ARCHITECTS	floor of existing boiler room.	Project No: E-6591
START DATE: 15/05/91 -		ELEVATION 75.15 (m)

SAMPLE TYPE ■ DISTURBED-AUGER ◿ SHELBY TUBE ☒ SPLIT SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE



McROSTIE GENEST ST-LOUIS Ottawa, Canada	COMPLETION DEPTH 2.9 m	COMPLETE 15/05/91	
	LOGGED BY JML	DWG NO. 5	Page 1 of 1

McROSTIE GENEST ST-LOUIS
& Associates Ltd.
Consulting Engineers
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No. 97-1

Date : JAN. 14, 1997

ALZHIEMERS FACILITY
QUEENSWAY CARLETON HOSPITAL

ELEV. 77.67	DEPTH in metres	DESCRIPTION	REMARKS
		TOPSOIL	sides stable
77.22	0.45		
		medium dense silty fine SAND	PENETROMETER READINGS 400 kPa
76.87	0.80		
76.67	-- 1 --		
		very stiff	400 kPa
		sandy brownish gray CLAY	
			395 kPa
			345 kPa
75.67	-- 2 --		345 kPa no water seepage
75.47	2.2		
		Bottom of pit	
	NOTE:	ALL TEST PITS DUG WITH RUBBER TIRED BACKHOE (CASE 580C)	
		B.M. (ELEV 74.98m) geodetic: Ground floor of existing boiler plant.	
			Plate No. 2

McROSTIE GENEST ST-LOUIS
& Associates Ltd.
Consulting Engineers
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.
97-2

Date : JAN. 14, 1997

ALZHIEMERS FACILITY
QUEENSWAY CARLETON HOSPITAL

ELEV. 77.18	DEPTH in metres	DESCRIPTION	REMARKS
77.03	0.15	TOPSOIL	sides stable
		medium dense silty very fine SAND	
76.58	0.60		PENETROMETER READINGS
		very stiff	260,260,260 kPa
76.18	-- 1 --	fissured sandy brownish gray CLAY	305 kPa
			305 kPa
			305 kPa
			305 kPa no water seepage
75.18	-- 2 --	Bottom of pit	

Plate No.
3

McROSTIE GENEST ST-LOUIS
& Associates Ltd.
Consulting Engineers
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No. 97-3

Date : JAN. 14, 1997

ALZHIEMERS FACILITY
QUEENSWAY CARLETON HOSPITAL

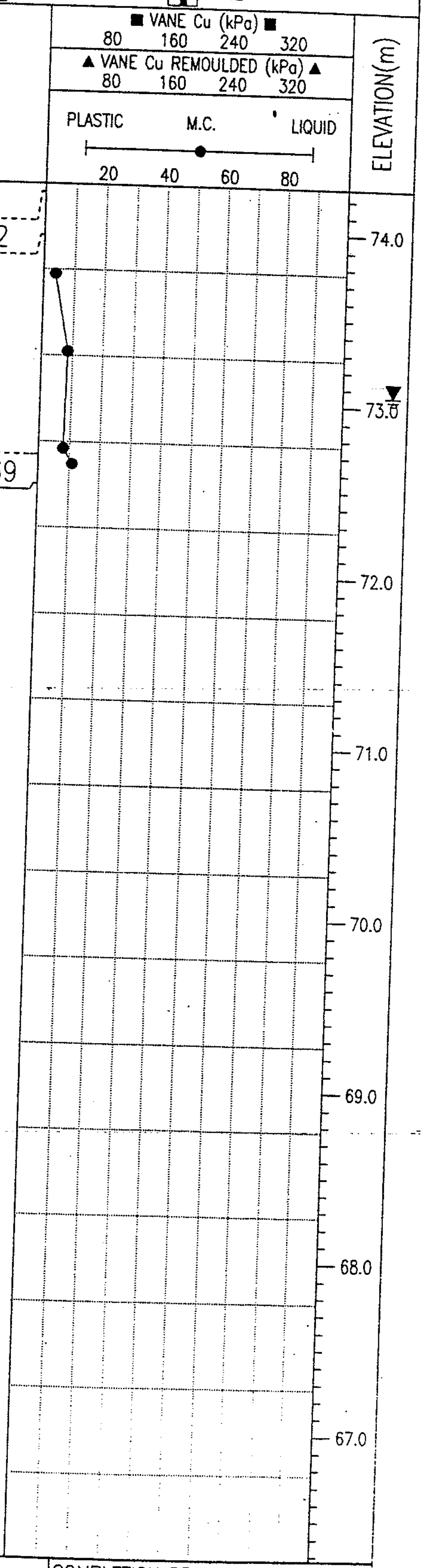
ELEV. 77.03	DEPTH in metres	DESCRIPTION	REMARKS
76.88	0.15	FILL - topsoil	sides stable
		FILL fine sand	
76.43	0.60		
		TOPSOIL	
76.13 76.03	0.90 -- 1 --		PENETROMETER READINGS 375 kPa
		very stiff sandy brownish gray CLAY	345 kPa 345 kPa 325 kPa
75.03	-- 2 --		no water seepage
		Bottom of pit	

Plate No.
4

QUEENSWAY-CARLETON HOSPITAL	B.M.(ELEV 74.98m)geodetic: Ground floor of existing boiler plant.	BOREHOLE NO: 98-3
CLIENT: QUEENSWAY-CARLETON HOSPITAL		PROJECT NO: E - 7671
START DATE: 98/06/11		ELEVATION: 74.27 m

SAMPLE TYPE ■ REMOULDED ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ◫ NO RECOVERY ▮ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION
0.0					ASPHALT
					FILL – crushed limestone 74.22
		19 12/15cm	☒	1	73.97 FILL coarse sand with some gravel
1.0		24 13/15cm	☒	2	
WL					Water level june 15/98 elev 73.06m
		7/15cm 5/3cm	☒	3 3A	
					medium dense sandy TILL 72.69
2.0					Bottom of hole 72.52 Power auger refusal
3.0					
4.0					
5.0					
6.0					
7.0					
8.0					



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

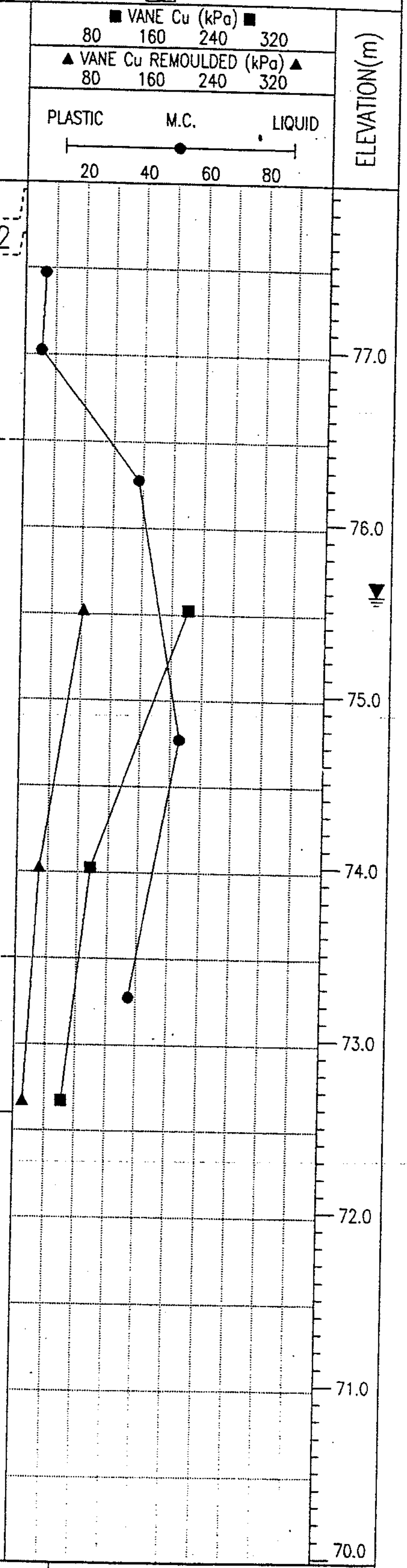
LOGGED BY: JML	COMPLETION DEPTH: 1.75 m
REVIEWED BY: E.S	COMPLETE: 98/06/11
4	Page 1 of 1

98/06/22 10:18AM (STANDARD)

QUEENSWAY-CARLETON HOSPITAL	B.M.(ELEV 74.98m)geodetic: Ground floor of existing boiler plant.	BOREHOLE NO: 98-4
CLIENT: QUEENSWAY-CARLETON HOSPITAL		PROJECT NO: E – 7671
START DATE: 98/06/11		ELEVATION: 77.97 m

SAMPLE TYPE ■ REMOULDED ◿ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯▯ NO RECOVERY ▮ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION
0.0					ASPHALT
					FILL – crushed limestone 77.92
		15/15cm 33	☒	1	77.67
					FILL coarse sand and gravel with traces of topsoil
1.0		13/15cm 11	☒	2	
	170,210,220	5 4/15cm	☒	3	76.47
2.0					
WL					Water level june 15/98 elev 75.59m
					hard to stiff brownish gray CLAY
3.0	20,20,20	2/45cm	☒	4	
4.0					
		wt of hammer /45cm	☒	5	73.47
5.0					stiff silty gray CLAY
					Bottom of hole 72.57
6.0					
7.0					
8.0					



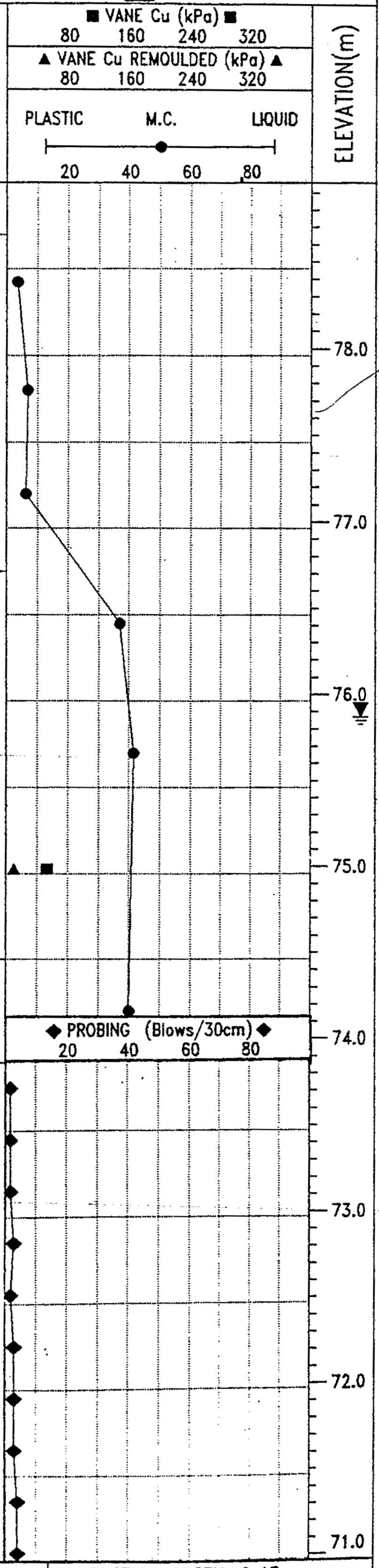
McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 5.4 m
	REVIEWED BY: E·S	COMPLETE: 98/06/11
	5	Page 1 of 1

98/06/22 10:15AM (STANDARD)

QUEENSWAY CARLETON HOSPITAL - 2004	B.M.(ELEV 77.25m)geodetic: Spindle top	BOREHOLE NO: 04-3
	of hydrant north of oxygen tanks.	PROJECT NO: E-8690
START DATE: 04/08/05		ELEVATION: 78.96 m

SAMPLE TYPE ■ REMOULDED ◸ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION
0.0					FILL - sand and gravel
		47 35/15cm 50/10cm	☒	1	78.66
	split barrel refusal				
1.0		12 6	☒	2	FILL sand, gravel and clay with some topsoil
		11 10	☒	3	
2.0					
	335,310,335	17 28	☒	4	76.71
					Water level Aug.10/04 elev 75.87m
3.0	WL 70,70,95	4 4	☒	5	stiff brownish gray CLAY
4.0					
	25,25,25	2 2	☒	6	soft silty gray CLAY 74.46
5.0			▤		PROBING 73.86
6.0			▤		SOIL with low resistance to penetration
7.0			▤		
8.0			▤		Borehole continued 70.96



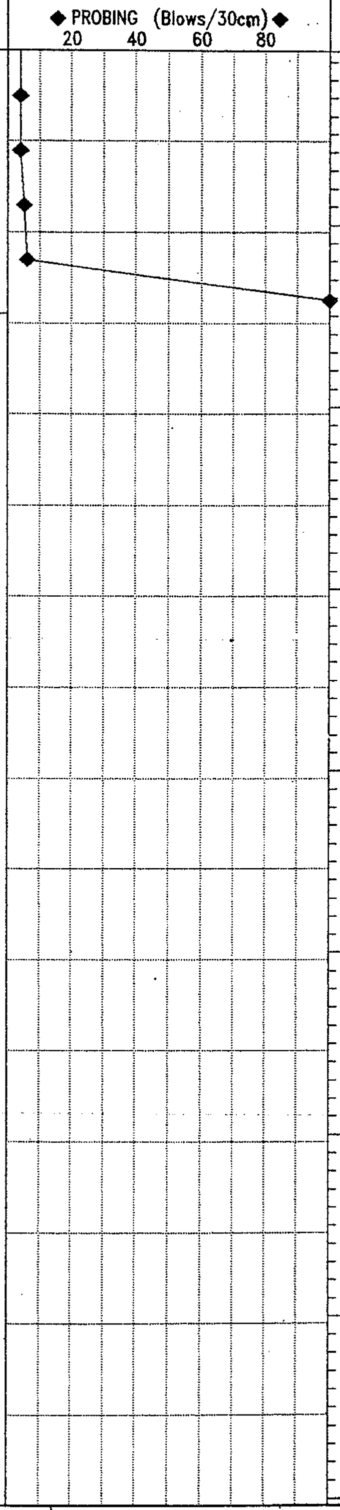
McROSTIE GENEST ST-LOUIS	LOGGED BY: JML	COMPLETION DEPTH: 9.45 m
Ottawa, Canada	REVIEWED BY: E.S.	COMPLETE: 04/08/05
	Fig. No: 5	Page 1 of 2

04/08/17 11:58AM (STANDARD)

QUEENSWAY CARLETON HOSPITAL - 2004	B.M.(ELEV 77.25m)geodetic: Spindle top	TEST HOLE NO: 04-3
	of hydrant north of oxygen tanks.	PROJECT NO: E-8690
START DATE: 04/08/05		ELEVATION: 78.96 m

SAMPLE TYPE ■ REMOULDED-AUGER ◩ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH(m)	COMMENTS	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	◆ PROBING (Blows/30cm) ◆	ELEVATION(m)
8.0		▤		SOIL with low resistance to penetration		
9.0		▤				70.0
				Bottom of hole 69.51 Probing refusal		
10.0						69.0
11.0						68.0
12.0						67.0
13.0						66.0
14.0						65.0
15.0						64.0
16.0						63.0



McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 9.45 m
	REVIEWED BY: E.S.	COMPLETE: 04/08/05
	Fig. No: 6	Page 2 of 2

04/08/17 12:00PM (PROB-STD)

QUEENSWAY CARLETON HOSPITAL - 2004

START DATE: 04/08/05

B.M.(ELEV 77.25m)geodetic: Spindle top of hydrant north of oxygen tanks.

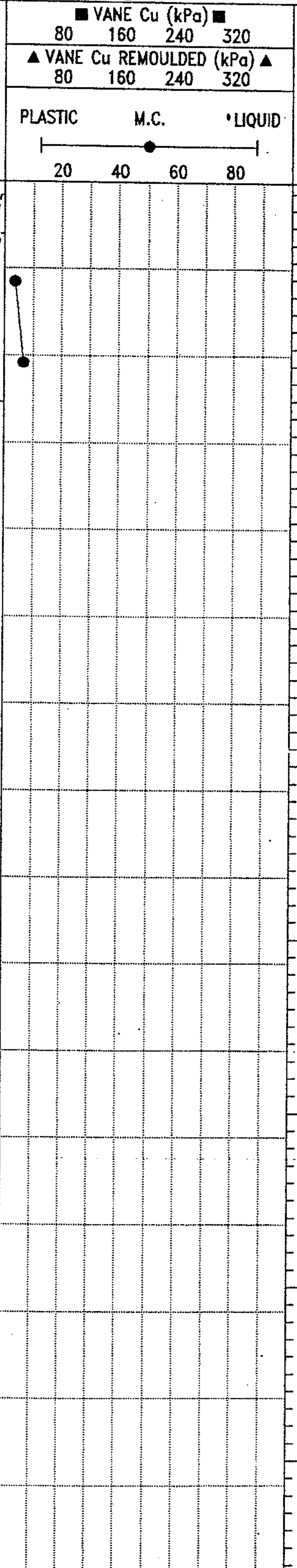
BOREHOLE NO: 04-8

PROJECT NO: E-8690

ELEVATION: 74.36 m

SAMPLE TYPE ■ REMOULDED-AUGER ⧄ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	ELEVATION(m)
0.0						FILL - asphalt	
						FILL - crushed limestone 74.28	74.0
		17 45	☒	1		74.06 FILL medium and coarse sand with some gravel & traces of clay	
1.0		15/15cm 50/13cm split-barrel refusal power auger refusal	☒	2			
						Bottom of hole 73.09 power auger refusal	73.0
2.0							72.0
3.0							71.0
4.0							70.0
5.0							69.0
6.0							68.0
7.0							67.0
8.0							



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML

REVIEWED BY: E.S.

Fig. No: 14

COMPLETION DEPTH: 1.27 m

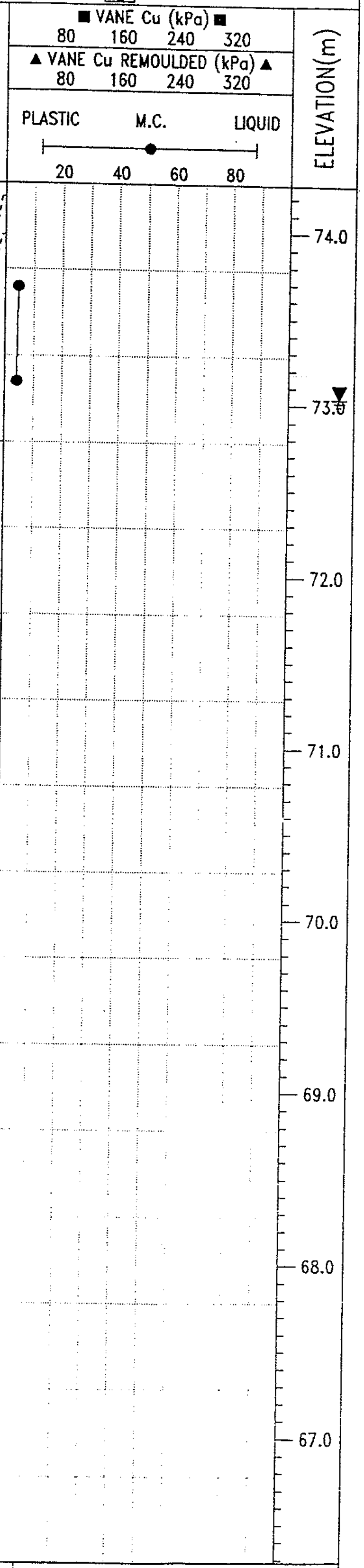
COMPLETE: 04/08/05

04/08/11 08:56AM (NQ-STD)

QUEENSWAY CARLETON HOSPITAL - 2004	B.M.(ELEV 77.25m)geodetic: Spindle top of hydrant north of oxygen tanks.	BOREHOLE NO: 04-9
		PROJECT NO: E-8690
START DATE: 04/08/03		ELEVATION: 74.28 m

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	
0.0						FILL - asphalt	
						FILL - crushed limestone	74.20
		16 17	☒	1		FILL medium and coarse sand with some gravel & pieces of broken rock	73.98
1.0		12 7/15cm 50/5cm	☒	2			
	WL split barrel refusal power auger refusal					Water level Aug.10/04 elev 73.04m	
						Bottom of hole power auger refusal	72.78
2.0							
3.0							
4.0							
5.0							
6.0							
7.0							
8.0							

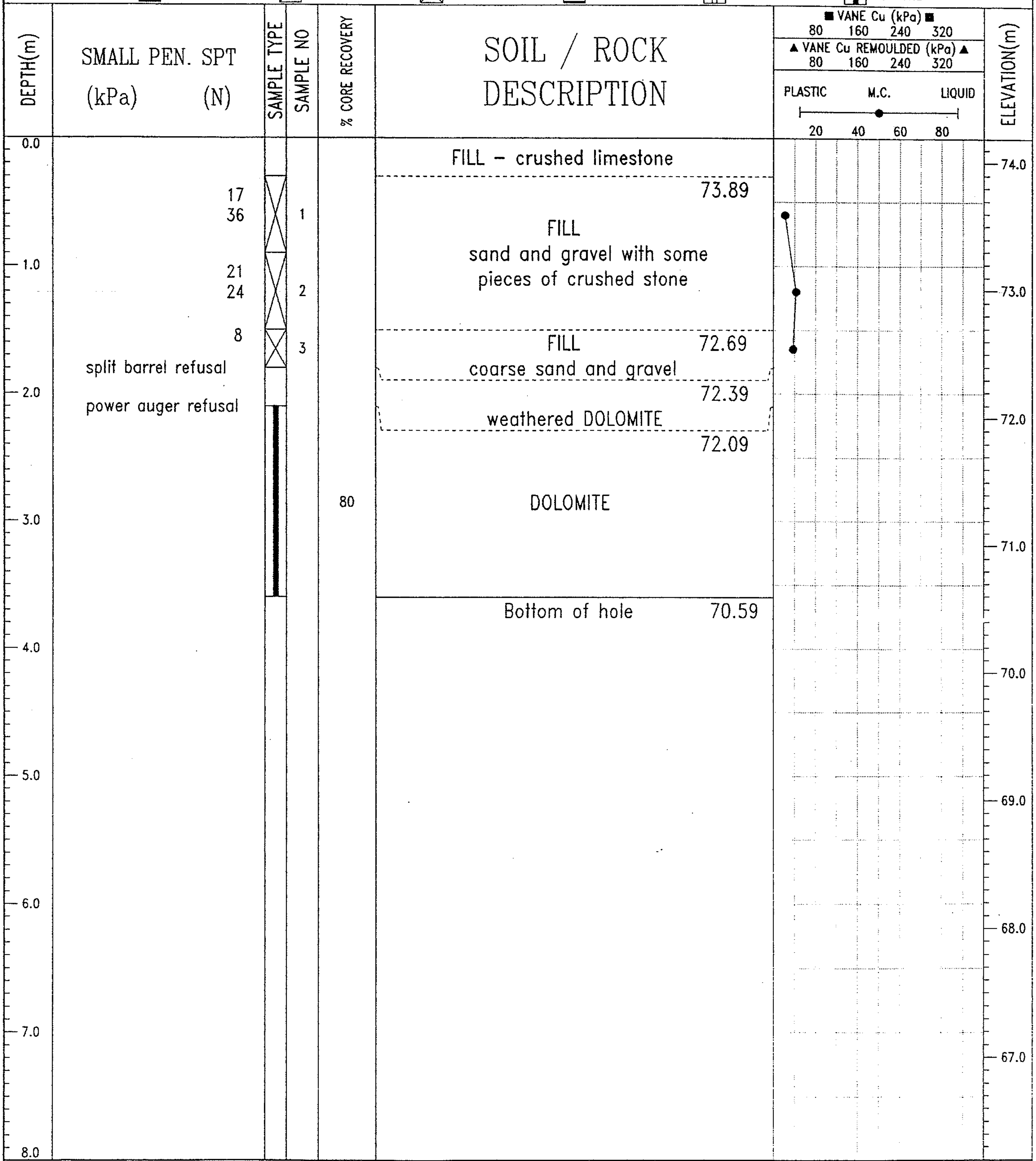


McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 1.5 m
	REVIEWED BY: E.S.	COMPLETE: 04/08/03
	Fig. No: 15	Page 1 of 1

04/08/11 08:56AM (NQ-STD)

QUEENSWAY CARLETON HOSPITAL – 2004	B.M.(ELEV 77.25m)geodetic: Spindle top of hydrant north of oxygen tanks.	BOREHOLE NO: 04-10
		PROJECT NO: E-8690
START DATE: 04/08/03		ELEVATION: 74.19 m

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE



SMALL PEN. SPT (N)	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	ELEVATION
			FILL – crushed limestone	73.89
17 36	1		FILL sand and gravel with some pieces of crushed stone	
21 24	2			72.69
8	3		FILL coarse sand and gravel	72.39
split barrel refusal				
power auger refusal			weathered DOLOMITE	72.09
		80	DOLOMITE	
			Bottom of hole	70.59

McROSTIE GENEST ST-LOUIS	LOGGED BY: JML	COMPLETION DEPTH: 3.6 m
Ottawa, Canada	REVIEWED BY: E.S.	COMPLETE: 04/08/03
	Fig. No: 16	Page 1 of 1

04/08/23 07:45AM (NO-STD)

QUEENSWAY CARLETON HOSPITAL – 2004

START DATE: 04/08/16

B.M.(ELEV 77.25m)geodetic: Spindle top of hydrant north of oxygen tanks.

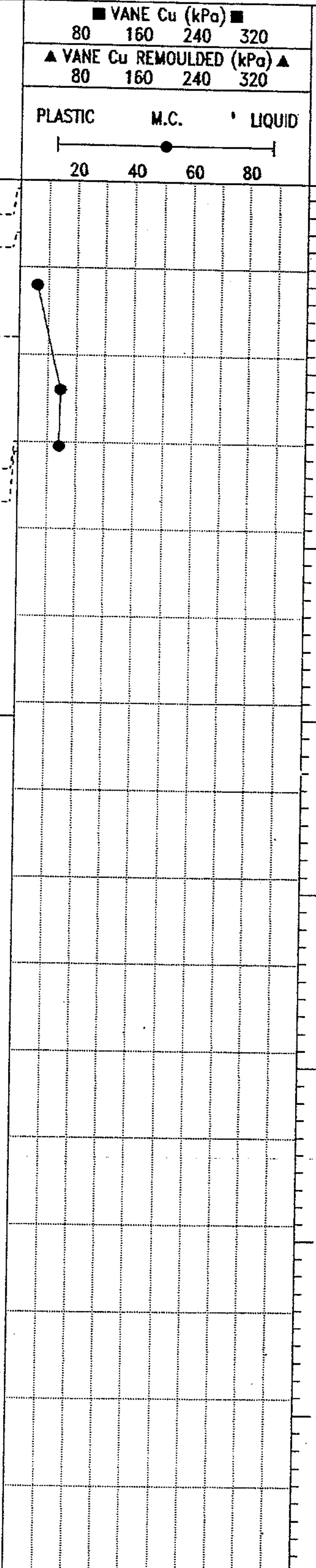
BOREHOLE NO: 04-11

PROJECT NO: E-8690

ELEVATION: 74.09 m

SAMPLE TYPE ■ REMOULDED–AUGER ◪ SHELBY TUBE ☒ SPLIT–SPOON ▤ NW–CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	ELEVATION(m)
0.0						FILL – asphalt	74.0
						FILL – crushed limestone 74.04	
		8 10	☒	1		FILL 73.79 sand with some gravel	
1.0 ▼		2/15cm 20 11/15cm	☒	2		medium dense 73.19 silty fine SAND with traces of gravel	73.0 ▼
	WL	50/5cm	☒	3		Water level Aug.17/04 elev 72.92m	
	split barrel refusal		▮			sandy TILL 72.56	
2.0			▮		100	72.53 DOLOMITE	72.0
3.0						Bottom of hole 71.01	71.0
4.0							70.0
5.0							69.0
6.0							68.0
7.0							67.0
8.0							



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML

REVIEWED BY: E.S.

Fig. No: 17

COMPLETION DEPTH: 3.08 m

COMPLETE: 04/08/16

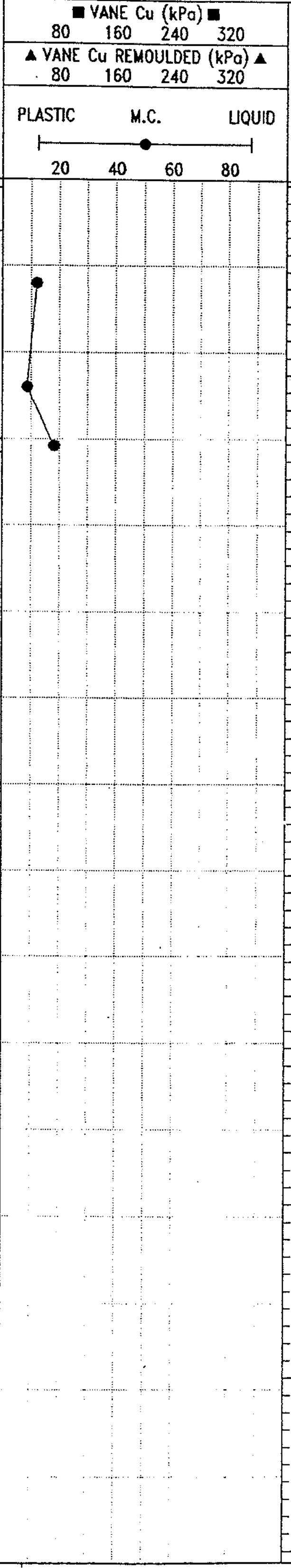
Page 1 of 1

04/08/20 03:44PM (NQ-STD)

QUEENSWAY CARLETON HOSPITAL - 2004
B.M.(ELEV 77.25m)geodetic: Spindle top of hydrant north of oxygen tanks.
BOREHOLE NO: 04-12
PROJECT NO: E-8690
START DATE: 04/08/16
ELEVATION: 73.93 m

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	ELEVATION(m)
0.0						FILL - asphalt	
						FILL - crushed limestone 73.88	
		8 11	☒	1		73.63 FILL SAND with some gravel and traces of clay	
1.0		3/15cm 18 9/15cm	☒	2		Water level Aug.17/04 elev 72.78m	73.0
	WL	50/8cm	☒	3		sandy TILL 72.43	
	split barrel refusal		▮			72.35	
2.0			▮		100	DOLOMITE	72.0
3.0			▮			Bottom of hole 70.83	71.0
4.0							70.0
5.0							69.0
6.0							68.0
7.0							67.0
8.0							66.0



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

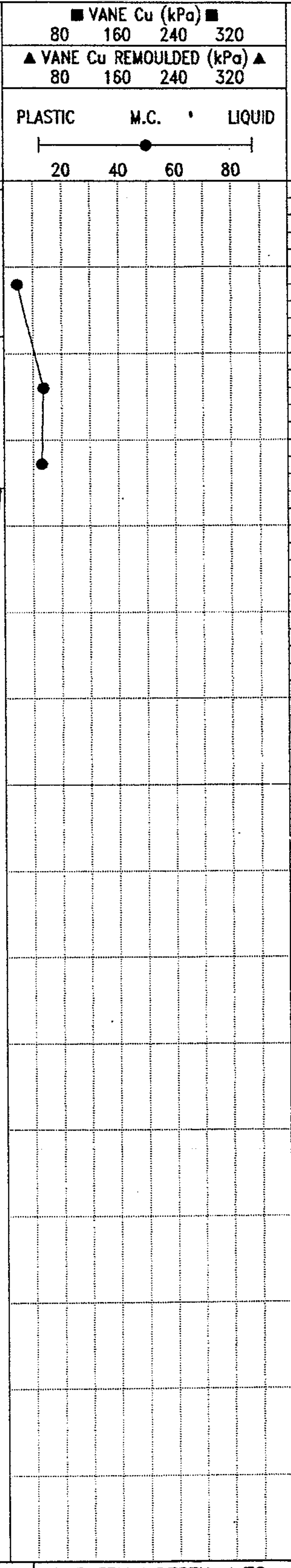
LOGGED BY: JML
REVIEWED BY: E.S.
Fig. No: 18
COMPLETION DEPTH: 3.1 m
COMPLETE: 04/08/16
Page 1 of 1

04/08/20 03:44PM (NQ-STD)

QUEENSWAY CARLETON HOSPITAL - 2004	B.M.(ELEV 77.25m)geodetic: Spindle top of hydrant north of oxygen tanks.	BOREHOLE NO: 04-13
		PROJECT NO: E-8690
START DATE: 04/08/16		ELEVATION: 74.5 m

SAMPLE TYPE ■ REMOULDED-AUGER ⧄ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	ELEVATION(m)
0.0						FILL - asphalt	
		5/15cm 23 9/15cm	☒	1		74.45 FILL crushed limestone	74.0
1.0		16 14	☒	2		73.60 FILL sand and gravel with pieces of crushed stone, traces of clay and topsoil	
	WL	2/15cm 50/13cm	☒	3		Water level Aug.17/04 elev 73.09m	73.0
	split barrel refusal						
2.0						Bottom of hole 72.72	
							72.0
3.0							
							71.0
4.0							
							70.0
5.0							
							69.0
6.0							
							68.0
7.0							
							67.0
8.0							



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML	COMPLETION DEPTH: 1.78 m
REVIEWED BY: E.S.	COMPLETE: 04/08/16
Fig. No: 19	Page 1 of 1

04/08/20 03:44PM (NO-STD)

QUEENSWAY - CARLETON CANCER CENTRE	B.M.(ELEV 77.25m)geod.: Spindle top of	BOREHOLE NO: 05-1
	hyd. on west side of main road to boiler	PROJECT NO: E-8941
START DATE: 05/12/13	plant north of propane tanks.	ELEVATION: 76.97 m

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY
	11 9	☒	1	
	2 4	☒	2	
85,95,70	2 2	☒	3	
50,65,65 WL	2 2	☒	4	
50,50,50	2 2	☒	5	
50,50,50	2 2	☒	6	
25,25,25	2 2	☒	7	

SOIL / ROCK DESCRIPTION

ASPHALT

FILL - sand and gravel 76.89

76.67

FILL
clay with some topsoil

75.47

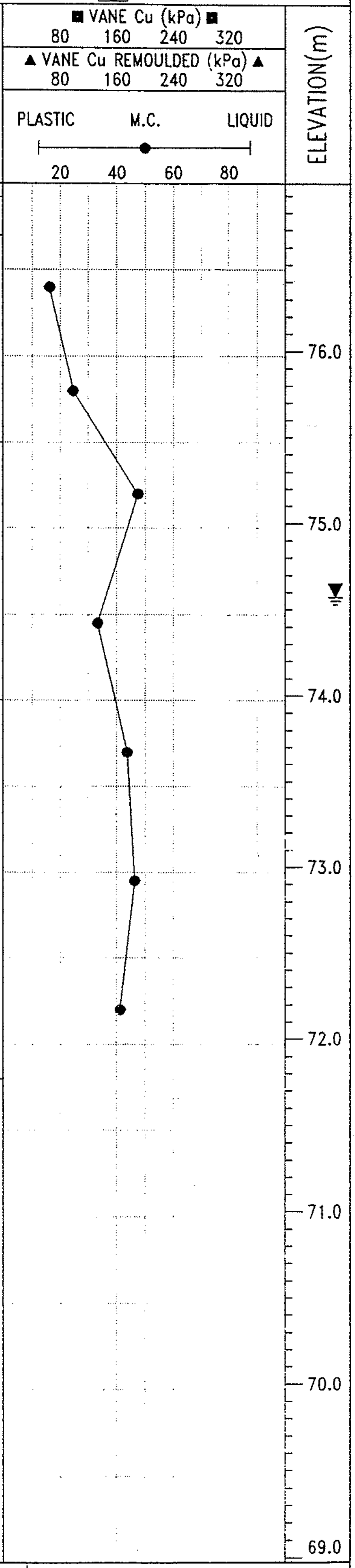
stiff to
medium soft
silty brownish gray CLAY

Water level Dec.14/05 elev 74.57m

73.97

soft
silty gray CLAY

Bottom of hole 71.77
Power auger refusal



DEPTH(m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0

LOGGED BY: JML	COMPLETION DEPTH: 5.2 m
REVIEWED BY: E.S.	COMPLETE: 05/12/13
Fig. No: 2	Page 1 of 1

McROSTIE GENEST ST-LOUIS
Ottawa, Canada

06/01/09 10:41AM (NQ-STD)

QUEENSWAY – CARLETON CANCER CENTRE

START DATE: 05/12/13

B.M.(ELEV 77.25m)geod.: Spindle top of hyd. on west side of main road to boiler plant north of propane tanks.

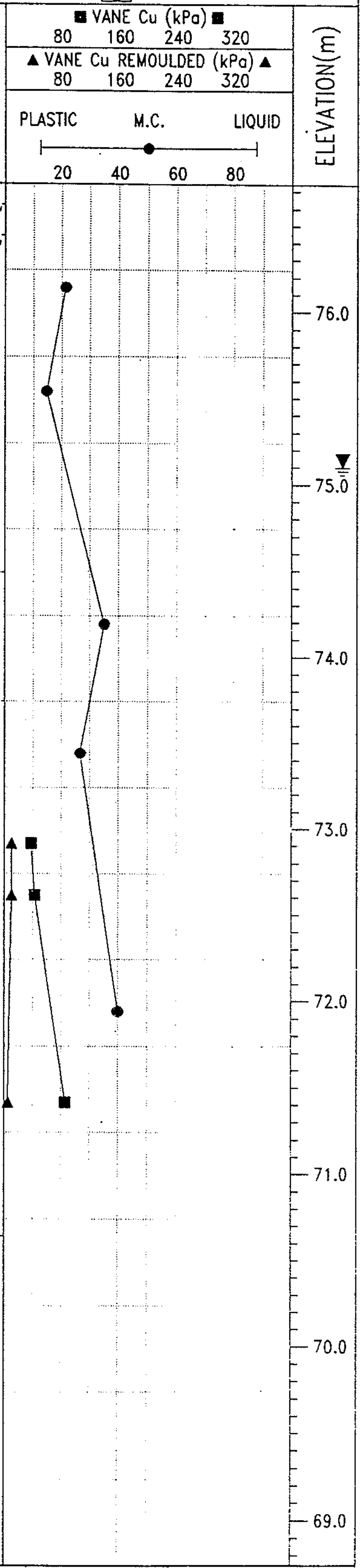
BOREHOLE NO: 05-2

PROJECT NO: E-8941

ELEVATION: 76.74 m

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ☰ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION
0.0						ASPHALT
						FILL – sand and gravel 76.62
		23 9	☒	1		76.44
1.0		4 5	☒	2		FILL clay with traces of gravel
	WL	2 3	☒	3		Water level Dec.14/05 elev 75.10m
2.0						
	70,65,65	2 2	☒	4		stiff to 74.49 medium soft silty brownish gray CLAY
3.0	50,50,65	2 2	☒	5		73.74
4.0						soft silty gray CLAY
	25,50,25	2 2	☒	6		
5.0						
6.0	split barrel refusal	50/10cm	☒	7		Bottom of hole 70.64
7.0						
8.0						



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML

REVIEWED BY: E.S.

Fig. No: 3

COMPLETION DEPTH: 6.1 m

COMPLETE: 05/12/13

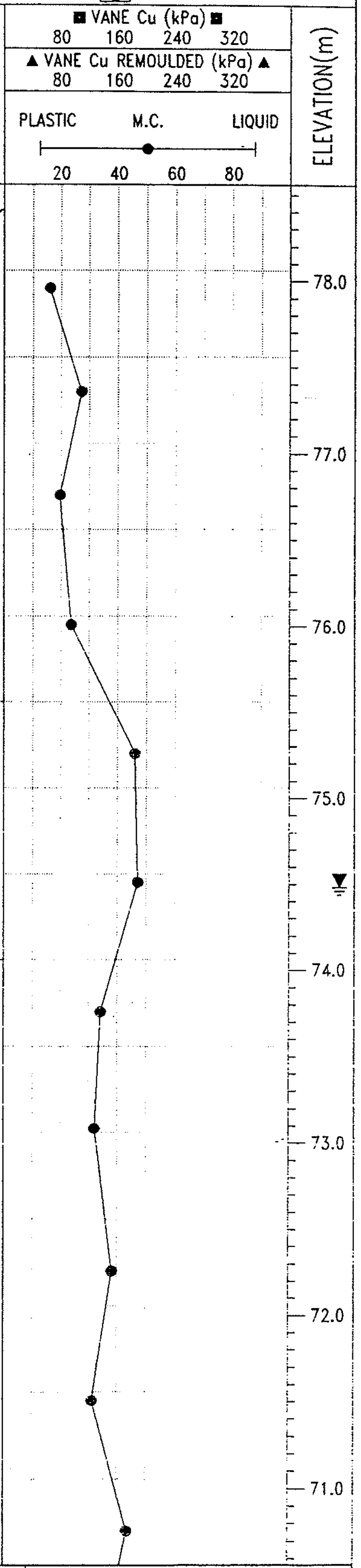
Page 1 of 1

06/0?/09 10:44AM (NO-STD)

QUEENSWAY - CARLETON CANCER CENTRE	B.M.(ELEV 77.25m)geod.: Spindle top of	BOREHOLE NO: 05-3
	hyd. on west side of main road to boiler	PROJECT NO: E-8941
START DATE: 05/12/12	plant north of propane tanks.	ELEVATION: 78.56 m

SAMPLE TYPE ■ REMOULDED-AUGER ◺ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION
0.0						FILL - topsoil 78.41
		5 5	☒	1		FILL brownish gray clay
1.0		4 4	☒	2		
		4 4	☒	3		
2.0		13 16	☒	4		
3.0		4 3	☒	5		75.56 stiff to medium soft silty brownish gray CLAY
4.0	WL	2 2	☒	6		Water level Dec.14/05 elev 74.48m
		2 2	☒	7		74.06 soft silty gray CLAY
5.0		2 2	☒	8		
6.0		2 2	☒	9		
7.0		2 2	☒	10		
8.0		2 2	☒	11		Borehole continued 70.56



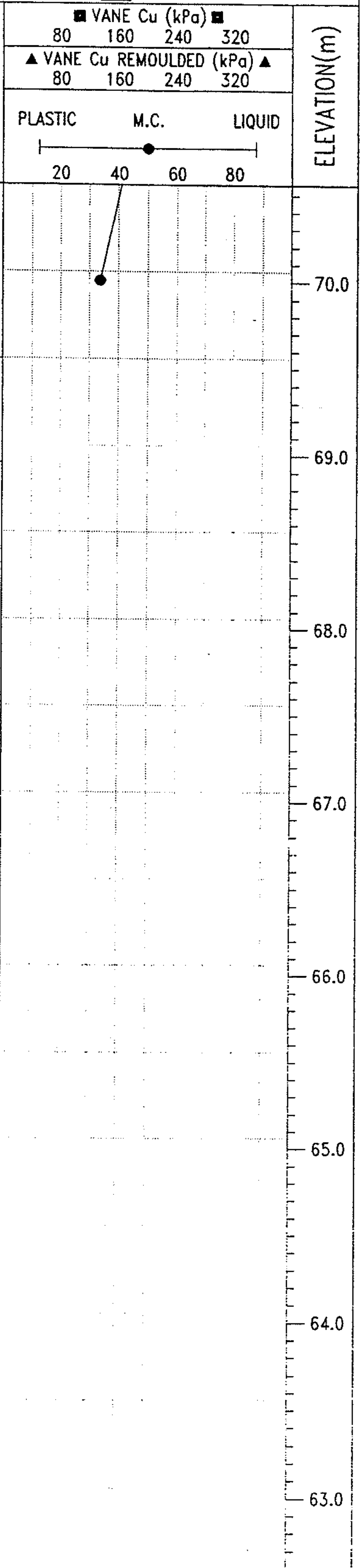
McROSTIE GENEST ST-LOUIS	LOGGED BY: JML	COMPLETION DEPTH: 11.12 m
	REVIEWED BY: E.S.	COMPLETE: 05/12/12
Ottawa, Canada	Fig. No: 4	Page 1 of 2

06/01/07 10:06AM (NO-STD)

QUEENSWAY - CARLETON CANCER CENTRE	B.M.(ELEV 77.25m)geod.: Spindle top of	BOREHOLE NO: 05-3
	hyd. on west side of main road to boiler	PROJECT NO: E-8941
START DATE: 05/12/12	plant north of propane tanks.	ELEVATION: 78.56 m

SAMPLE TYPE ■ REMOULDED-AUGER ◿ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION
8.0			☒	11		soft silty gray CLAY
		2 2	☒	12		
9.0	split barrel refusal	50/5cm	☒	13		69.56 sandy TILL
	power auger refusal					
10.0			▮		95	68.96 DOLOMITE
11.0						Bottom of hole 67.44
12.0						
13.0						
14.0						
15.0						
16.0						



McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 11.12 m
	REVIEWED BY: E.S.	COMPLETE: 05/12/12
	Fig. No: 5	Page 2 of 2

06/01/07 10:07AM (NQ-STD)

QUEENSWAY - CARLETON CANCER CENTRE

START DATE: 05/12/12

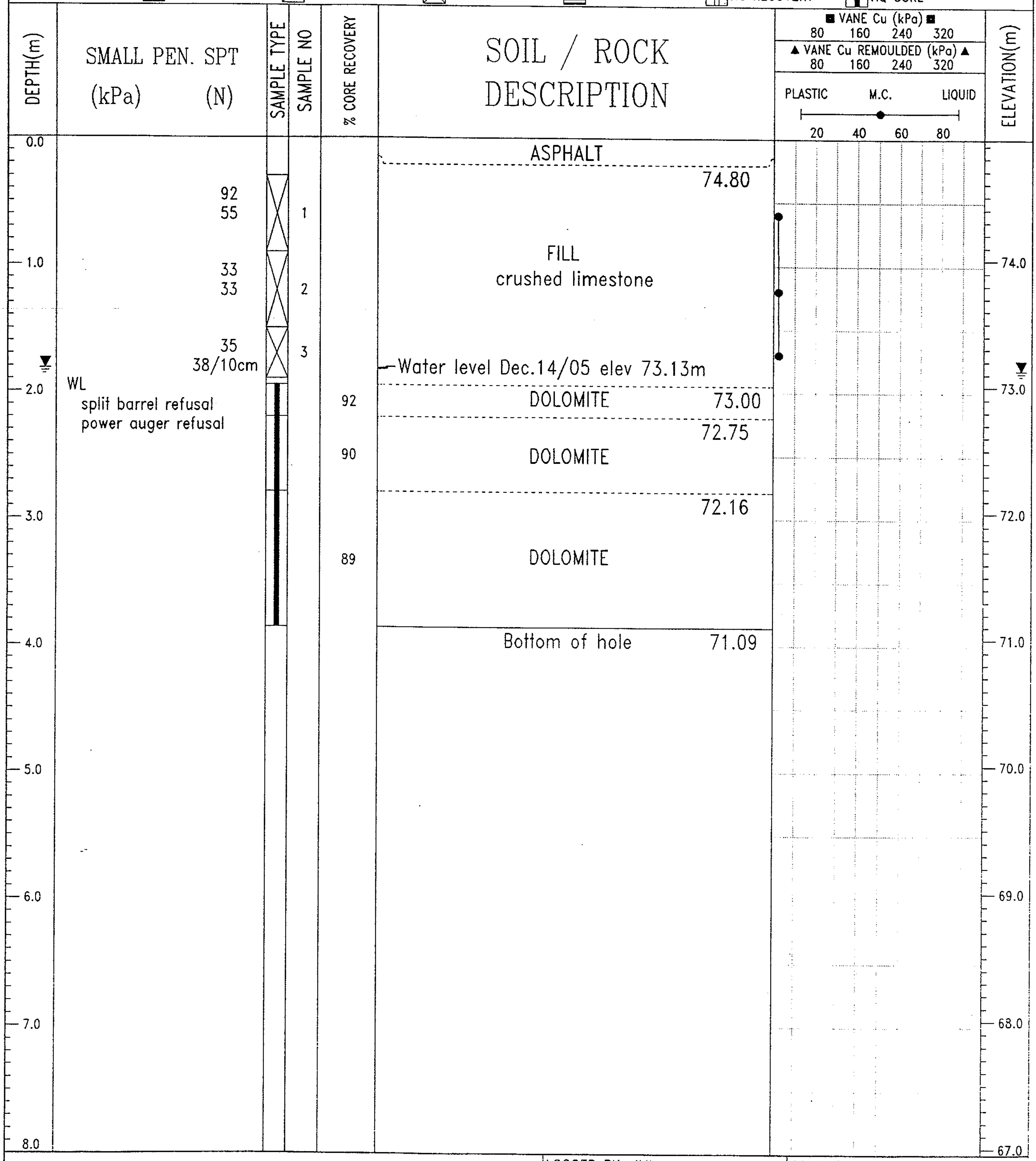
B.M.(ELEV 77.25m)geod.: Spindle top of hyd. on west side of main road to boiler plant north of propane tanks.

BOREHOLE NO: 05-8

PROJECT NO: E-8941

ELEVATION: 74.95 m

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE



McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML

REVIEWED BY: E.S.

Fig. No: 10

COMPLETION DEPTH: 3.86 m

COMPLETE: 05/12/12

Page 1 of 1

06/01/07 10:12AM (NG-STD)

PROJECT: 07-1121-0002

RECORD OF BOREHOLE: 07-4

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: February 1, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10⁻⁶ 10⁻⁵ 10⁻⁴ 10⁻³ / WATER CONTENT PERCENT Wp ⊢ O W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		76.92							
	Power Auger 200mm Diam. (Hollow Stem)	Dark brown silty fine sand, some gravel, trace clay (FILL)		76.72 0.20							
		Brown sand and gravel, pieces of concrete (FILL)		76.39 0.53							
		Grey silty sand, some gravel (FILL)		76.11							
		Brown sand and gravel (FILL)									
1		Dark brown SILTY CLAY with sand		75.85 1.07							
		Very stiff grey brown SILTY CLAY with sand seams (Weathered Crust)		75.40	1	50 DO	11				
		Loose grey brown SILTY fine SAND, trace clay		75.24 1.68							
2		Firm to stiff grey brown SILTY CLAY with sand seams		74.02	2 3	50 DO 50 DO	4 1				
3		Firm grey SILTY CLAY with sand seams		72.80 2.90	4	50 DO	1				
4		Loose grey SILTY SAND, some gravel, trace clay (GLACIAL TILL)		71.41 4.12	5 6 7	50 DO 50 DO 50 DO	2 6 >100				∇
5	Rotary Drill NQ Core	Grey DOLOMITE BEDROCK		69.20 5.51	8	NQ RC	DD	T.C.R. (%) 92 S.C.R. (%) 84 R.Q.D. (%) 63			
7					9	NQ RC	DD	T.C.R. (%) 100 S.C.R. (%) 67 R.Q.D. (%) 51			
		End of Borehole		7.72							Water level in open hole at elev. 72.14m during drilling
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: R.I.

CHECKED: J.L.

PROJECT: 07-1121-0002

RECORD OF BOREHOLE: 07-6

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: February 7, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface	77.13				
Power Auger 200mm Diam. (Hollow Stem)	Grey brown sand and gravel (FILL)	0.00				Gravel
	Grey brown sandy silt, trace gravel and organic matter (FILL)	76.72 0.41				
	Grey brown SANDY SILT	76.37 0.76	1	50 DO	19	
	Mottled brown CLAY, occasional sand seams	75.91 1.22	2	50 DO	7	
	Very stiff grey brown SILTY CLAY, some fine sand seams	75.00 2.13	3	50 DO	3	Native Backfill
			4	50 DO	3	
	Very loose grey SANDY SILT, trace gravel (GLACIAL TILL)	73.47 3.66	5	50 DO	2	Silica Sand Standpipe
	End of Borehole Auger Refusal	72.69 4.44	6	50 DO	>100	Note: Unable to find standpipe in gravel surfaced parking lot. No water level taken.

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.W.M.

CHECKED: J.L.

PROJECT: 07-1121-0002
LOCATION: See Site Plan
SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 07-10

BORING DATE: February 1, 2007

SHEET 1 OF 1
DATUM: Geodetic
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		Ground Surface		74.93							
0	Power Auger 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE									
		Grey crushed stone (FILL)		0.10 74.40							
		Brown sand and gravel, trace to some clay layers (FILL)		0.53 74.17							
		Grey brown silty clay, trace sand (FILL)		0.76 73.91	1	50 DO	6				
1		Loose brown sand and gravel (FILL)		73.76							
		Loose grey SANDY SILT, trace clay		1.17 73.41							
		Firm grey SILTY CLAY with sand seams		1.52 73.00	2	50 DO	8				
2		End of Borehole Auger Refusal		1.93							Borehole dry upon completion of drilling
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: R.I.
CHECKED: J.L.

PROJECT: 07-1121-0002

RECORD OF BOREHOLE: 07-12

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: January 29, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		Ground Surface		77.38							
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL									
		Brown fine sand and gravel (FILL)		0.05	1	A.S.					
1		Loose brown sandy silt, some clay, trace gravel (FILL)		76.77 0.61	2	50 DO	9				
2		Stiff to very stiff grey brown SILTY CLAY with brown fine sand seams (Weathered Crust)		76.11 1.27	3 4	50 DO 50 DO	5 4				
3		Loose to dense grey SILTY fine SAND		74.64 2.74	5 6	50 DO 50 DO	15 >100				∇
4		End of Borehole Auger Refusal		73.42 3.96							Water level in open hole at elev. 74.18m upon completion of drilling
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: R.I.

CHECKED: J.L.

PROJECT: 07-1121-0002-7000

RECORD OF BOREHOLE: BH 08-307

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: July 30, 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: 20 40 60 80; nat V. + Q - ●; rem V. ⊕ U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 10 20 30 40

ADDITIONAL LAB. TESTING

SOIL PROFILE

BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)
	Ground Surface	77.44
Power Auger, 200 mm Diam. (Hollow Stem)	Dark brown sandy clay and organics (FILL)	0.00
	Very stiff grey brown SILTY CLAY with silty fine sand seams (Weathered Crust)	77.14 0.30
	Loose to compact grey SANDY SILT, some gravel (GLACIAL TILL)	75.76 1.68
RD, NW		
Rotary Drill, HQ Core	Slightly weathered grey brown DOLOMITE BEDROCK, with brown sandstone seams	74.30 3.14
	Fresh, grey DOLOMITE BEDROCK, occasional brown sandstone layers and bands	72.87 4.57
	End of Borehole	68.80 8.64

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)
1	50 DO	8			
2	50 DO	4			
3	50 DO	13			
4	50 DO				
C1	HQ RC	DD	100	93	89
C2	HQ RC	DD	97	48	30
C3	HQ RC	DD	100	100	100
C4	HQ RC	DD	99	98	96
C5	HQ RC	DD	100	96	91
C6	HQ RC	DD	100	100	100

PIEZOMETER OR STANDPIPE INSTALLATION

Flush mount ∇ protective casing

Cement Grout

63 mm Diam Schedule 40 Pipe

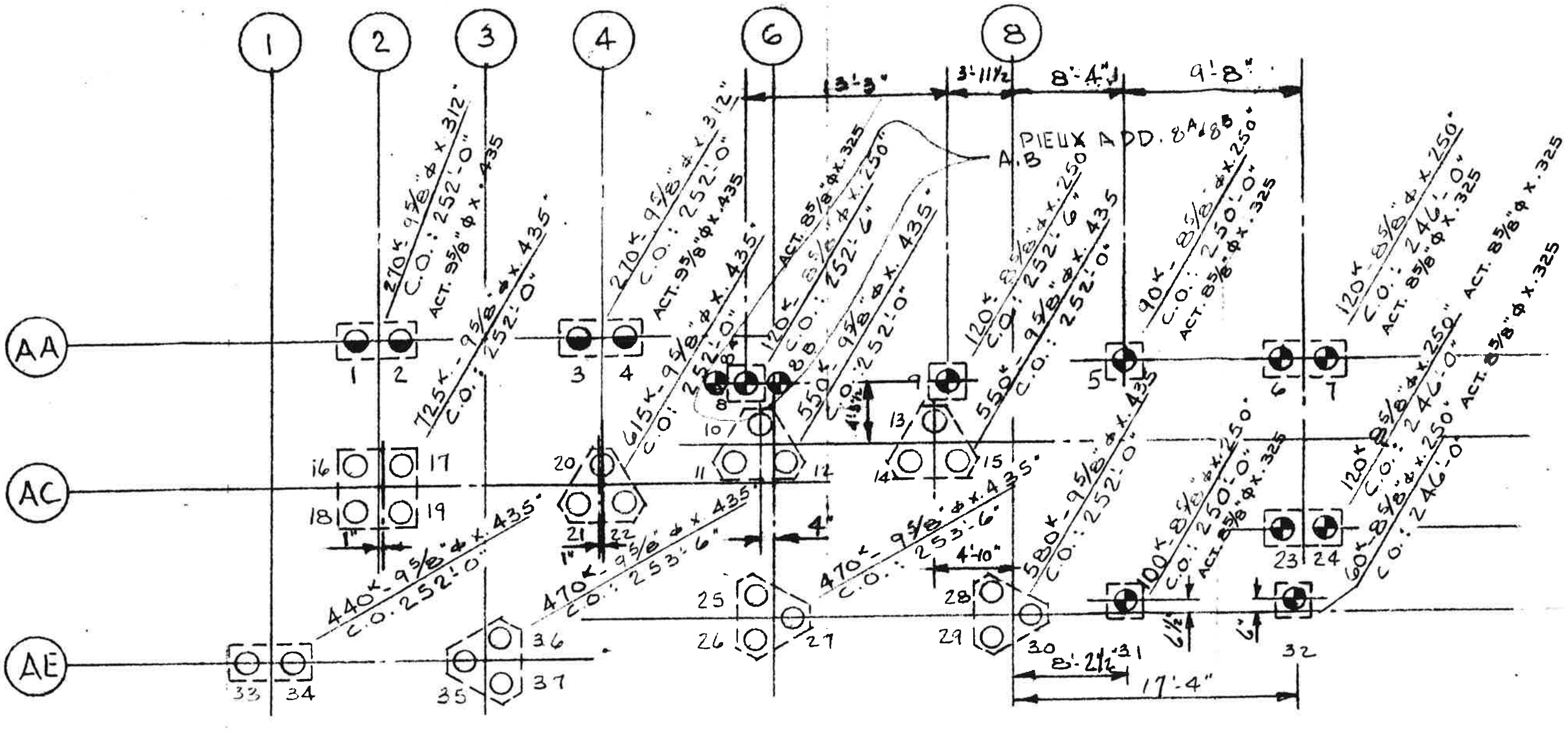
WL in standpipe at Elev. 76.88 m on Aug. 26, 2008

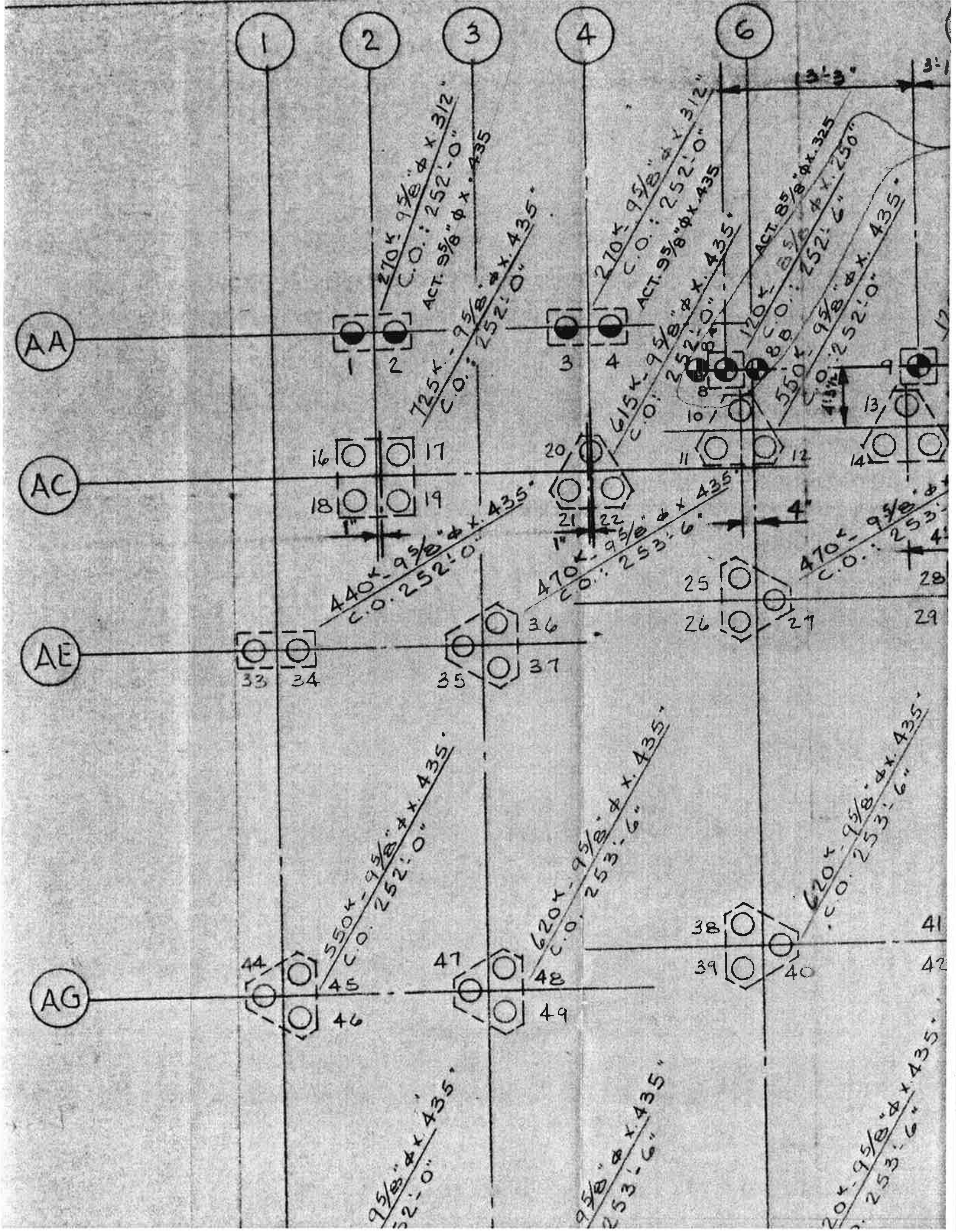
DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

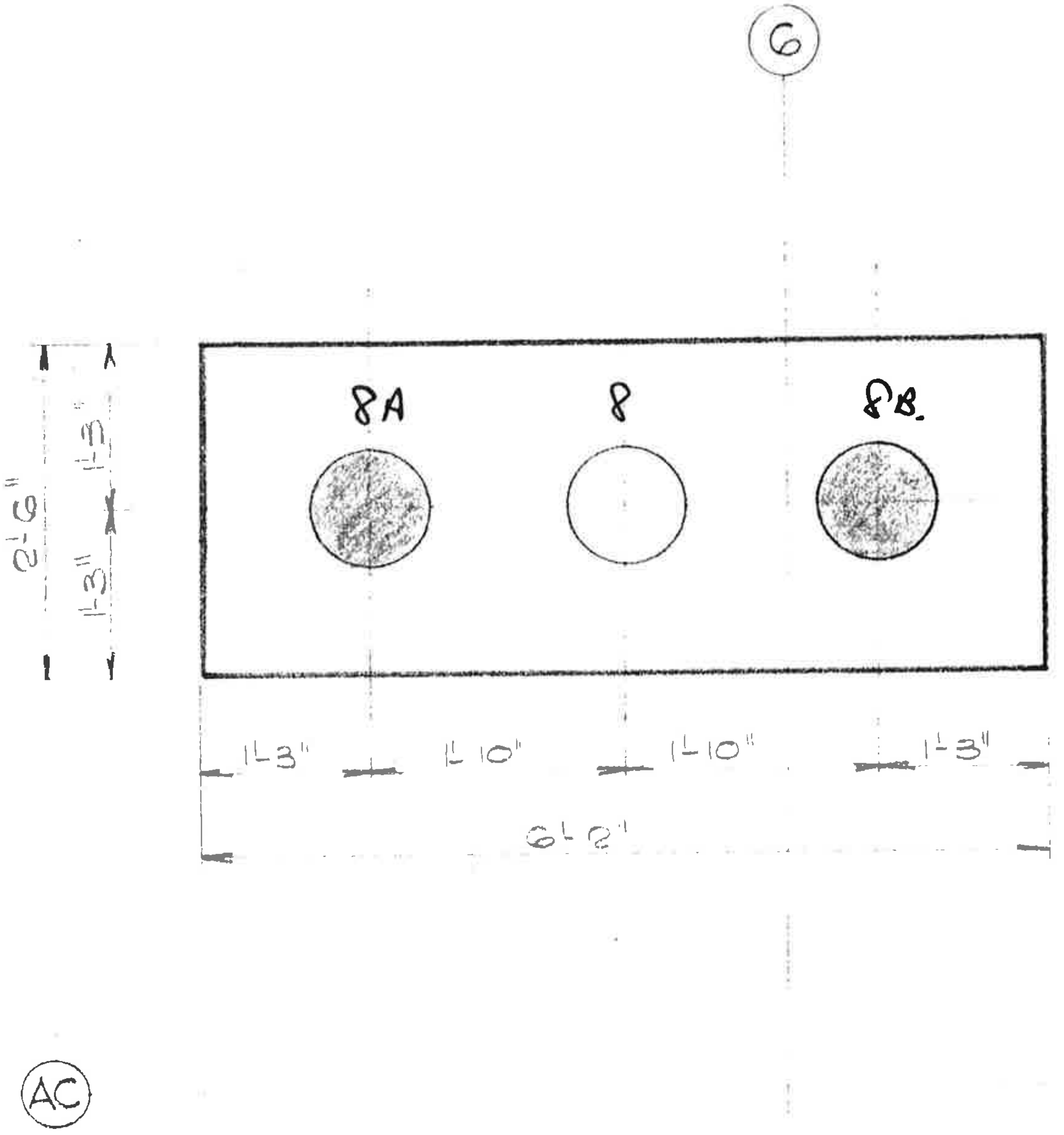
DEPTH SCALE
1 : 50

Golder Associates

LOGGED: P.A.H.
CHECKED: MAU


1
2
3
4
6
8
AA
AC
AE
3'-3"
3'-11½
8'-4"
9'-8"
PIEUX ADD. 8A & 8B
A.B
270K - 9⅝" Φ x .312
C.O.: 252'-0"
ACT. 9⅝" Φ x .435
725K - 9⅝" Φ x .435
C.O.: 252'-0"
270K - 9⅝" Φ x .312
C.O.: 252'-0"
ACT. 9⅝" Φ x .435
615K - 9⅝" Φ x .435
C.O.: 252'-0"
ACT. 8⅝" Φ x .325
120K - 8⅝" Φ x .250
C.O.: 252'-6"
550K - 9⅝" Φ x .435
C.O.: 252'-0"
120K - 8⅝" Φ x .250
C.O.: 252'-6"
550K - 9⅝" Φ x .435
C.O.: 252'-0"
90K - 8⅝" Φ x .250
C.O.: 250'-0"
ACT. 8⅝" Φ x .325
120K - 8⅝" Φ x .250
C.O.: 246'-0"
ACT. 8⅝" Φ x .325
120K - 8⅝" Φ x .250
C.O.: 246'-0"
ACT. 8⅝" Φ x .325
440K - 9⅝" Φ x .435
C.O.: 252'-0"
470K - 9⅝" Φ x .435
C.O.: 253'-6"
470K - 9⅝" Φ x .435
C.O.: 253'-6"
580K - 9⅝" Φ x .435
C.O.: 252'-0"
100K - 8⅝" Φ x .250
C.O.: 250'-0"
ACT. 8⅝" Φ x .325
60K - 8⅝" Φ x .250
C.O.: 246'-0"
8A
8B
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
1"
1"
4"
4'-10"
4'-3½"
6½"
6"
8'-2½"
17'-4"


1
2
3
4
6
3'-3"
3'-1
AA
AC
AE
AG
270k - 95/8"φ x .312"
C.O.: 252'-0"
ACT. 95/8"φ x .435
725k - 95/8"φ x .435
C.O.: 252'-0"
270k - 95/8"φ x .312"
C.O.: 252'-0"
ACT. 95/8"φ x .435
615k - 95/8"φ x .435
C.O.: 252'-0"
ACT. 85/8"φ x .325
120k - 85/8"φ x .250"
C.O.: 252'-6"
550k - 95/8"φ x .435
C.O.: 252'-0"
4'-3½"
1
2
3
4
5
7
8
9
10
11
12
13
14
16
17
18
19
20
21
22
1"
1"
4"
440k - 95/8"φ x .435
C.O. 252'-0"
470k - 95/8"φ x .435
C.O.: 253'-6"
470k - 95/8"φ x .435
C.O.: 253'-
4½
25
26
27
28
29
33
34
35
36
37
550k - 95/8"φ x .435
C.O. 252'-0"
620k - 95/8"φ x .435
C.O. 253'-6"
620k - 95/8"φ x .435
C.O. 253'-6"
38
39
40
41
42
44
45
46
47
48
49
95/8"φ x .435
252'-0"
95/8"φ x .435
253'-6"
20k - 95/8"φ x .435
253'-6"



REVISED PILE LAYOUT FOR 'P3'

1 Add 2-120^k capacity piles to existing single pile group at two locations shown shaded above

2 Cut off levels of piles are at elevation 252'-0"

3 Pile cap detail to be issued at a later date

QUEENSWAY CARLETON HOSPITAL	HANS L STUTZ ARCHITECT 1129 CARLING AVE OTTAWA ROBERT HALSALL & ASSOC LTD CONSULTING ENGINEERS 425 GLOUCESTER ST. OTTAWA	Oct 17th 73 S-103

07-1121-0002 (7000)

McROSTIE SETO GENEST
& Associates Ltd. – & Associés Ltée
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

noted
Seto

RIG NO. 2
PILE TYPE TUBE
PILE SIZE 8 5/8 & 9 5/8
HAMMER LBS. 3.500 DROP HT. 4'
VOL. BUCKET —
DRIVE TUBE —

DATE July 30/73 JOB NO. E-2910
JOB LOCATION QWY, CALL, HOSPITAL
JOB INSPECTOR K. BALLIS
JOB ENGINEER H. SETO
PILING CONTR. BENTA
PILING SUPT. C. LEVASSEUR

PILE NUMBER		THICK [redacted]	TONS	DRIVING DEPTH	PILE LENGTH	BOTT	FINAL PENETRATION			TUBE
BLOCK A+C	DIA	[redacted]	[redacted]			ELEV	BLOWS	REFU	RETAP	LENGTHS
OK 17 4	9 5/8	.435	110	16'-8"	13'-3"	238'-9"	5	0	0	30'-3"
OK 16 4	9 5/8	.435	110	16'-9"	13'-5"	238'-7"	5	0	0	21'-3"
OK 18 4	9 5/8	.435	110	16'-8"	13'-5"	238'-7"	5	0	0	21'-3"
OK 19 4	9 5/8	.435	110	16'-11"	13'-4"	238'-8"	5	0	0	22'-8"
OK 33 4	9 5/8	.435	110	16'-10"	13'-1"	238'-11"	5	0	0	22'-8"
OK 34 4	9 5/8	.435	110	16'-10"	13'-0"	239'-0"	5	0	0	22'-1"
OK 36 4	9 5/8	.435	110	16'-10"	15'-1"	238'-5"	5	0	0	22'-1"
OK 35 4	9 5/8	.435	110	16'-10"	14'-11"	238'-7"	5	0	0	20'-2"
OK 37 4	9 5/8	.435	110	16'-11"	15'-1"	238'-5"	5	0	0	20'-2"
OK 20 4	9 5/8	.435	110	12'-11"	9'-7"	242'-5"	5	0	0	19'-3"
OK 21 4	9 5/8	.435	110	14'-9"	11'-5"	240'-7"	5	1/16	0	19'-3"
OK 22 4	9 5/8	.435	110	14'-1"	10'-10"	241'-2"	5	0	0	21'-4"
OK 10 4	9 5/8	.435	110	14'-3"	10'-10"	241'-2"	5	1/16	0	21'-4"
OK 11 4	9 5/8	.435	110	13'-4"	9'-11"	242'-1"	5	0	0	21'-0"
OK 12 4	9 5/8	.435	110	13'-7"	10'-0"	242'-0"	5	0	0	15'-6"
OK 25 4	9 5/8	.435	110	13'-5"	11'-6"	242'-0"	5	0	0	15'-6"
OK 27 4	9 5/8	.435	110	13'-7"	11'-7"	241'-11"	5	0	0	15'-6"
OK 26 4	9 5/8	.435	110	13'-8"	11'-8"	241'-10"	5	0	0	19'-2"
OK 58 4	9 5/8	.435	110	18'-8"	15'-0"	237'-0"	5	0	0	21'-0"
OK 56 4	9 5/8	.435	110	18'-7"	14'-10"	237'-2"	5	1/16	0	19'-2"

COMMENTS TO DAY 20 TO DATE 22
July 31/73

Noted Seto

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Lteé
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. 2
PILE TYPE TUBE
PILE SIZE 8 5/8 & 9 5/8
HAMMER LBS. 3,500 DROP HT. 4' & 5'*
VOL. BUCKET —
DRIVE TUBE —

DATE JULY 31/73 JOB NO. E-2910
JOB LOCATION Q.Y.W. CARL. HOSPITAL
JOB INSPECTOR K. BALLIS
JOB ENGINEER H. SETO
PILING CONTR. BENTA
PILING SUPT. C. LEVASSEUR

Δ - ACTUALLY ONLY 85 TONS REQ'D.

BLOCK A&C PILE NUMBER	DIA	[redacted] THICK.	TONS	DRIVING DEPTH	PILE LENGTH	BOTT.	FINAL PENETRATION			TUBE LENGTHS
		[redacted]	[redacted]			ELEV.	REFUS	BLOWS	RETAP	
OK 57 4	9 5/8	.435	110	18'-7"	14'-11"	237'-1"	0	5	0	20'-0"
OK 45 4	9 5/8	.435	110	17'-8"	13'-10"	238'-2"	0	5	0	21'-8"
OK *44 4	9 5/8	.435	110	17'-9"	13'-10"	238'-2"	1/16	5	0	20'-0"
OK 46 4	9 5/8	.435	110	17'-10"	13'-11"	238'-1"	0	5	0	21'-6"
OK *48 4	9 5/8	.435	110	17'-9"	16'-0"	237'-6	1/16	5	0	21'-0"
OK 47 4	9 5/8	.435	110	17'-8"	15'-10"	237'-8"	0	5	0	18'-9"
OK 49 4	9 5/8	.435	110	17'-9"	15'-10"	237'-8"	0	5	0	18'-9"
OK 38 4	9 5/8	.435	110	14'-1"	12'-2"	241'-4"	0	5	0	21'-0"
OK *40 4	9 5/8	.435	110	14'-4"	12'-5"	241'-1"	1/16	5	0	17'-3"
OK *39 4	9 5/8	.435	110	13'-10"	12'-0"	241'-6"	1/16	5	0	17'-3"
OK 50 4	9 5/8	.435	110	16'-8"	14'-10"	238'-8"	0	5	0	23'-4"
OK *52 4	9 5/8	.435	110	17'-3"	15'-5"	238'-1"	1/16	5	0	19'-9"
OK *51 4	9 5/8	.435	110	16'-9"	14'-10"	238'-8	1/16	5	0	19'-10"
OK *4 4	9 5/8	.435	(85)Δ 110	12'-9"	9'-5"	242'-7"	0	5	0	23'-4"
OK *3 4	9 5/8	.435	(85)Δ 110	12'-8"	9'-4"	242'-8"	0	5	0	19'-9"
OK *1 4	9 5/8	.435	(85)Δ 110	16'-6"	13'-0"	239'-0"	0	5	0	19'-10"
OK *2 4	9 5/8	.435	(85)Δ 110	16'-6"	12'-11"	239'-1"	0	5	0	19'-10"
TODAY 17	CAST 3 CYLINDERS				Nos 4-5-6		REPRESENTS			
TODATE 39	PILE # 365 (TEST PILE #1)						4" SLUMP			
	XXX CEMENT 3/4 STONE						5,000		PSI	

COMMENTS FRANCON SUPPLIER TIME 2:00 PM

JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Engineers & Geologists
Soil Investigations
Inspection & Testing Services
Damage Claims

Offices & Laboratory
1479 Laperriere Ave.
Ottawa, Canada K1Z 7S8
Telephone (613) 728-3505

CONCRETE REPORT

REPORT NO. 2651

CLIENT McRostie, Seto, Genest & Assoc. Ltd.

PRELIMINARY DATE Aug. 8, 1973

ADDRESS 393 Bell Street, Ottawa

FINAL DATE August 29/73

JOB E-2910

AT

LOCATION IN WORK [illegible] Tube Piles

CONCRETE SAMPLED BY [illegible] Client

TIME 2:00 p.m.

DATE CYLINDERS CAST July 31, 1973

BY Client

TIME 2:00

CONCRETE BY Franecon

CLASS 5000

MIX DATA

CEMENT	WATER	CONCRETE TEMP. °F
FINE AGG.	ADMIXTURE	STONE SIZE 3/4 IN.
COARSE AGG.	PERCENT AIR	APPROXIMATE SLUMP 4 IN.

COMPRESSIVE STRENGTH

POUNDS PER SQUARE INCH

AGE AT TEST (DAYS)	7	28	28			
LAB. STORED SPECIMEN	3470	4350	4460			
JOB STORED SPECIMEN						
WEIGHT (LBS./CU. FT.)						
CYLINDERS MARKED	4 X	5 X	6 E			

CYLINDERS RECEIVED Aug. 1, 1973

TYPE OF BREAK

CONTRACTOR

REMARKS - PRELIMINARY:

REMARKS - FINAL:

COPIES TO:
Client - 3

B. F. Cumming

NOTED
Aseto

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Ltée
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. 2
PILE TYPE TUBE
PILE SIZE 8 5/8 & 9 5/8
HAMMER LBS. 3,500 DROP HT. 4'
VOL. BUCKET —
DRIVE TUBE —

DATE AUG, 2/73 JOB NO E-2910
JOB LOCATION Q. Y W. CARL. HOSPITAL
JOB INSPECTOR K. BALLIS
JOB ENGINEER H. SETO
PILING CONTR. BENTA
PILING SUPT. C. LEVASSEUR

BLOCK PILE NUMBER A&C		DIA	THICK	TONS	DRIVING DEPTH	PILE LENGTH	BOTT ELEV	FINAL PENETRATION			TUBE LENGTHS
								REFU	B.	RETAP	
OK	8 ✓	8 5/8	.325	60	13'-2"	10'-3"	242'-3"	1/16	5	0	25'-0"
OK	9 ✓	8 5/8	.325	60	13'-9"	10'-10"	241'-8"	0	5	0	21'-7"
OK	5 ✓	8 5/8	.325	60	14'-0"	8'-9"	241'-3"	0	5	0	19'-0"
OK	7 ✓	8 5/8	.325	60	14'-5"	5'-3"	240'-9"	0	5	0	19'-1"
OK	6 ✓	8 5/8	.325	60	14'-4"	5'-3"	240'-9"	0	5	0	22'-0"
OK	24 ✓	8 5/8	.325	60	14'-10"	5'-7"	240'-5"	0	5	0	18'-6"
OK	23 ✓	8 5/8	.325	60	14'-9"	5'-5"	240'-7"	0	5	0	19'-1"
OK	32 ✓	8 5/8	.325	60	15'-0"	5'-8"	240'-4"	1/16	5	0	25'-1"
OK	31 ✓	8 5/8	.325	60	15'-6"	9'-3"	240'-9"	1/16	5	0	25'-1"
OK	13 ✓	9 5/8	.435	110	13'-10"	10'-5"	241'-7"	0	5	0	21'-1"
OK	15 ✓	9 5/8	.435	110	14'-0"	10'-5"	241'-7"	0	5	0	20'-10"
OK	14 ✓	9 5/8	.435	110	13'-9"	10'-4"	241'-8"	0	5	0	21'-1"

TO DAY 12
TO DATE 51

PILES N^os 8-9-5-7-6-24
23-32 & 31 REQUIRES
.250 THICKNESS-ACTUAL .325.

COMMENTS

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Ltée
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. BENTA 212-110
PILE TYPE TUBE PILES
PILE SIZE 10 3/4 & 8 5/8
HAMMER LBS. 6.780 DROP HT. 2'-4'
VOL. BUCKET —
DRIVE TUBE —

DATE OCT, 23/73 JOB NO. E-2910
JOB LOCATION QWY CARL HOSPITAL
JOB INSPECTOR K. BALLIS
JOB ENGINEER N. I. McGILL
PILING CONTR. BENTA
PILING SUPT. L. BEVIGNY

PILE NUMBER	SIZE	THICK	TONS	DRIVING DEPTH	PILE LENGTH	BOTT ELEV	FINAL PENETRATION		TUBE LENGTH
							NO BLOW	PENET.	
347A	10 3/4	.575	110	43'-4"	40' 11" ~~43-5~~	210'-1"	5	0	60'-11"
347B	10 3/4	.575	110	21'-0"	18' 1" ~~20-7~~	232'-11"	5	0	36'-0"
8A	8 5/8	.325	60	9'-10"	9' 8" ~~10-2~~	242'-4"	5	0	21'-7"
8B	8 5/8	.325	60	9'-10"	9' 10" ~~10-4~~	242'-2"	5	0	21'-8"

TODAY 4
TO DATE 383

CAST 3 CYLINDERS N^{os} 51-52-53
REPRESENTS PILE N^{os} 426-427-428
429-444-445-452-453-446-441
432-433-420-421-408-409-396-397
384-385-374 & 375
5.000 PSI 3/4 STONE 3½ SLUMP
FRANCON SUPPLIER TIME 3:30
INV. # 321109
4 #6 REBARS ADDED TO ALL PILES.

COMMENTS

PAGE 3 OF 4

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Lteé
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. 2
PILE TYPE TUBE PILES
PILE SIZE 9 5/8 & 8 5/8
HAMMER LBS. 6.780 DROP HT. 2' 3' & 5'
VOL. BUCKET —
DRIVE TUBE —

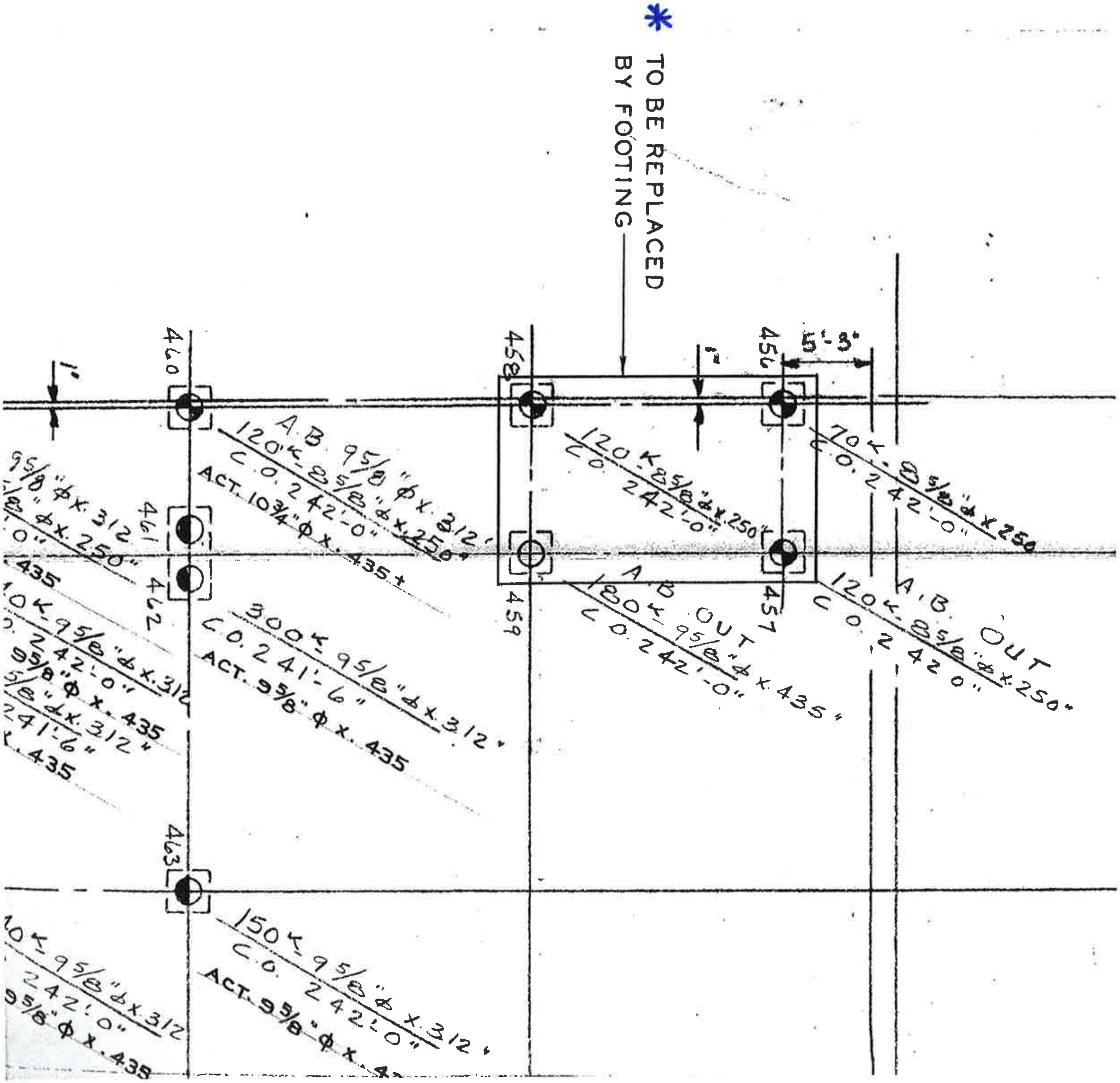
DATE SEPT. 28/73 JOB NO E-2910
JOB LOCATION QWY CARL. HOSPITAL
JOB INSPECTOR K. BALLIS
JOB ENGINEER N. I. Mc GILL
PILING CONTR. BENTA
PILING SUPT. C. LEVASSEUR

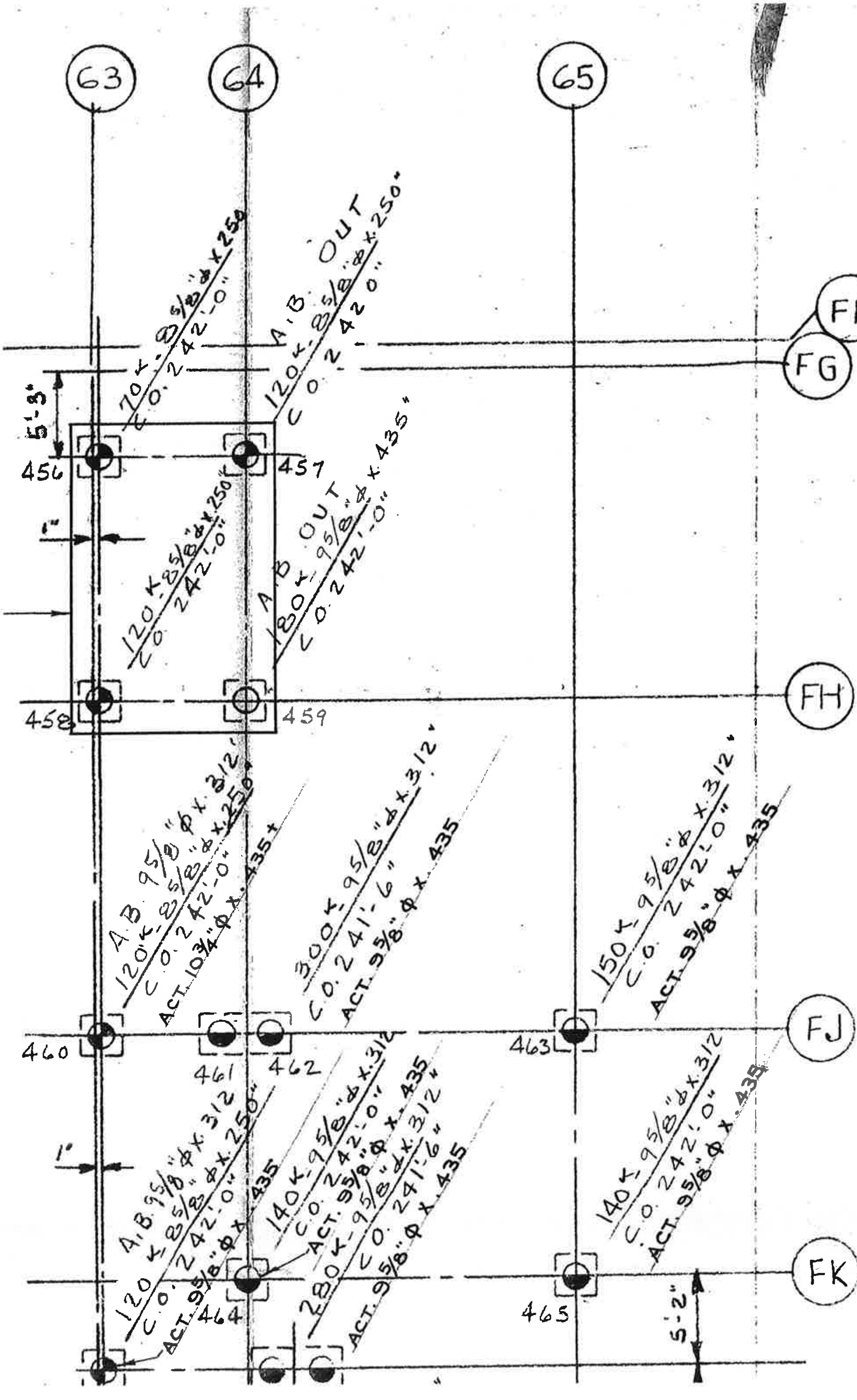
RETAPPING

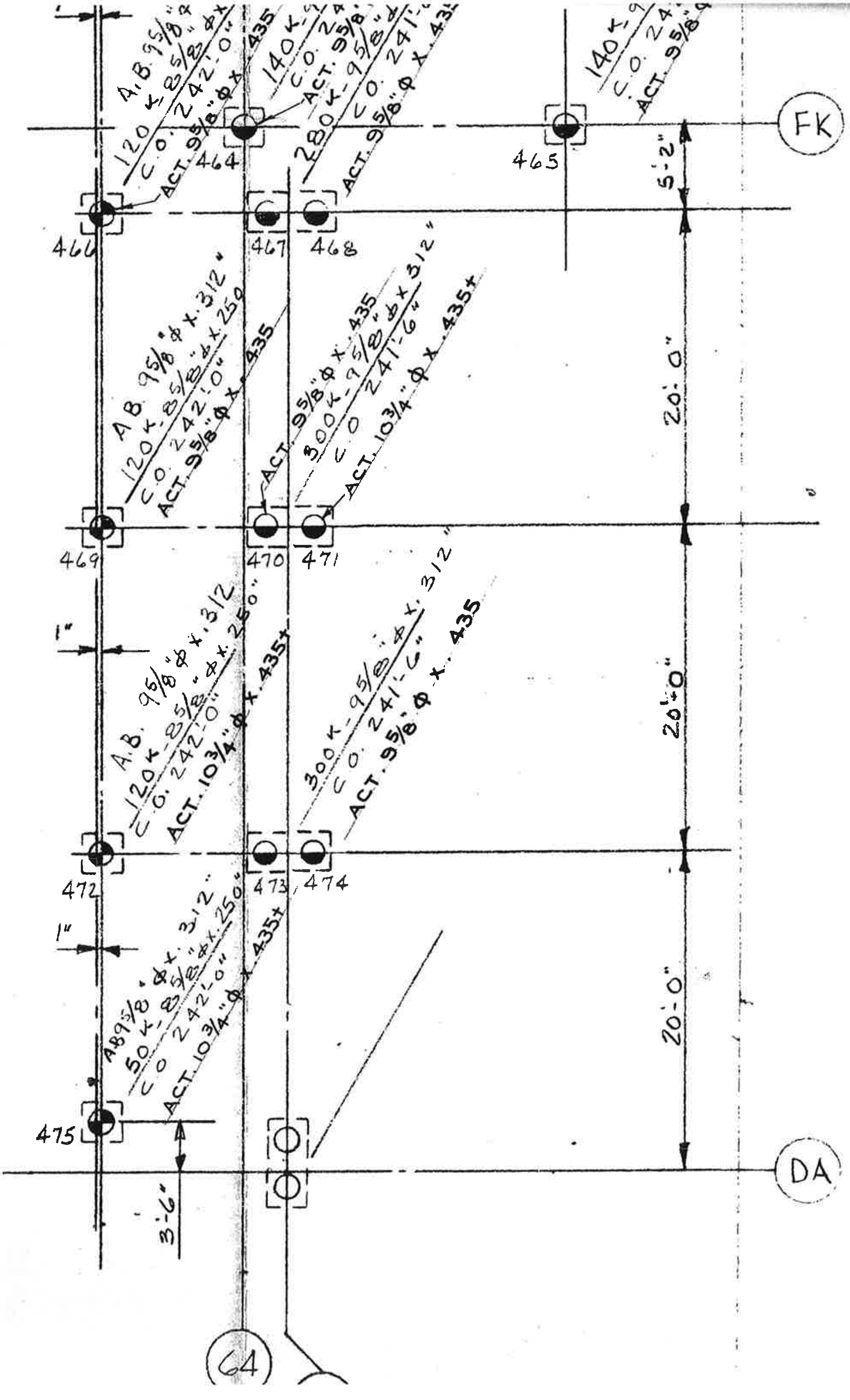
	PILE NUMBER		TIME START	TIME FINISH	DRIVING DEPTH	PILE LENGTH				
		SIZE	POURED	FOLLOW	DROP HT 5B	PENET	DROP HT. 5B	PENETRATION		
✓	83	9 5/8	✓	✓	2'	1/8	5'	0		
✓	81	9 5/8	✓	✓	2'	1/16	5'	1/4	0	
✓	86	9 5/8	✓	✓	2'	1/16	5'	0		
✓	85	9 5/8	✓	✓	2'	0	5'	0		
✓	84	9 5/8	✓	✓	2'	3/16	5'	1/16	1/16 0	
	8	8 5/8	✓	✓	2'	7/16	5'	1/4	TOP TUBE BUCKLED	
✓	9	8 5/8	✓	✓	2'	0	3'	0		
	240	8 5/8			2'	0	TUBE BEND			
	254	8 5/8			2'	1/8	3'	5/16	TUBE BEND	
	253	8 5/8			2'	0	3'	5/16	TUBE BEND	
	267	8 5/8		POURED	2'	1/8	2'	0	TUBE BEND	
	268	8 5/8			2'	1/8	2'	1/8	TUBE BEND	
/ Batter OK.	273	9 5/8			2'	0	4'	0		
✓	272	9 5/8			2'	1/16	4'	1/16	0	
✓	274	9 5/8			2'	0	4'	0		
✓	282	9 5/8	NO		2'	0	4'	0		
✓	284	9 5/8			2'	0	4'	0		
✓	283	9 5/8			2'	0	4'	0		
✓	305	9 5/8			2'	0	4'	0		

COMMENTS PILE N^os 8 - 240 - 254 - 253 - 267 & 268 CONTRACTOR STOP DRIVING DUE TO TOP OF TUBES BUCKLING & BENDING.

07-1121-0002 (7000)
QCH MRI-INFILL




63
64
65
FF
FG
FH
FJ
FK
70K-85/8"⌀x.250
CO. 242'-0"
A.B. OUT
120K-85/8"⌀x.250"
CO. 2420"
5'-3"
456
457
1"
120K-85/8"⌀x.250"
CO. 242'-0"
A.B. OUT
180K-95/8"⌀x.435"
CO. 242'-0"
458
459
A.B. 95/8"⌀x.312"
120K-85/8"⌀x.250"
CO. 242'-0"
ACT. 103/4"⌀x.435+
300K-95/8"⌀x.312"
CO. 241'-6"
ACT. 95/8"⌀x.435
150K-95/8"⌀x.312"
CO. 242'-0"
ACT. 95/8"⌀x.435
460
461
462
463
1"
A.B. 95/8"⌀x.312
120K-85/8"⌀x.250"
CO. 242'-0"
ACT. 95/8"⌀x.435
140K-95/8"⌀x.312
CO. 242'-0"
ACT. 95/8"⌀x.435
280K-95/8"⌀x.312
CO. 241'-6"
ACT. 95/8"⌀x.435
140K-95/8"⌀x.312
CO. 242'-0"
ACT. 95/8"⌀x.435
464
465
5'-2"


A.B. 95/8" ϕ x
120 k 85/8" ϕ x
C.O. 242'-0"
ACT. 95/8" ϕ x .435
140 k
C.O. 24
ACT. 95/8" ϕ
280 k 95/8" ϕ
C.O. 241'-
ACT. 95/8" ϕ x .435
140 k 9
C.O. 24
ACT. 95/8" ϕ
FK
464
465
466
467
468
5'-2"
AB 95/8" ϕ x .312"
120 k 85/8" ϕ x .250
C.O. 242'-0"
ACT. 95/8" ϕ x .435
ACT. 95/8" ϕ x .435
300 k 95/8" ϕ x .312"
C.O. 241'-6"
ACT. 103/4" ϕ x .435+
469
470
471
20'-0"
1"
A.B. 95/8" ϕ x .312
120 k 85/8" ϕ x .250"
C.O. 242'-0"
ACT. 103/4" ϕ x .435+
300 k 95/8" ϕ x .312"
C.O. 241'-6"
ACT. 95/8" ϕ x .435
20'-0"
472
473
474
1"
AB 95/8" ϕ x .312"
50 k 85/8" ϕ x .250"
C.O. 242'-0"
ACT. 103/4" ϕ x .435+
475
20'-0"
3'-6"
DA
64

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Lteé
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. BENTA 212-110
PILE TYPE TUBE PILES
PILE SIZE 9 5/8 & 10 3/4
HAMMER LBS. 6.780 DROP HT. 4'
VOL. BUCKET
DRIVE TUBE

DATE NOV, 15/73 JOB NO. E-2910
JOB LOCATION QWY. CARL. HOSPITAL
JOB INSPECTOR K. BALLIS
JOB ENGINEER N. I. McGILL
PILING CONTR. BENTA
PILING SUPT. L. SEVIGNY

PILE NUMBER	SIZE	THICK	TONS	DRIVING DEPTH	PILE LENGTH	BOTT ELEV.	FINAL PENETRATION № BLOWS	FINAL PENETRATION PENET.	TUBE LENGTH
		[redacted]	[redacted]						
463	9 5/8	.435	85	9'-1"	6'-6"	235'-6"	5	0	11'-1"
465	9 5/8	.435	85	10'-10"	7'-0"	235'-0"	5	0	15'-6"
~~456~~	~~10 3/4~~	~~.435~~	~~85~~		~~4'-5"~~	~~237'-7"~~	~~5~~	~~0~~	~~16'-1"~~
~~458~~	~~10 3/4~~	~~.435~~	~~85~~		~~4'-9"~~	~~237'-3"~~	~~5~~	~~0~~	~~16'-1"~~
460	10 3/4	.435	85	8'-5"	5'-4"	236'-8"	5	0	16'-1"
462	9 5/8	.435	85	8'-9"	5'-2"	236'-4"	5	0	13'-2"
461	9 5/8	.435	85	8'-5"	5'-1"	236'-5"	5	0	12'-11"
464	9 5/8	.435	85	9'-0"	6'-2"	235'-10"	5	0	12'-0"
466	9 5/8	.435	85	9'-0"	5'-10"	236'-2"	5	0	11'-7"
469	9 5/8	.435	85	9'-5"	6'-3"	235'-9"	5	0	11'-11"
472	10 3/4	.435	85	10'-0"	6'-6"	235'-6"	5	0	13'-4"
475	10 3/4	.435	85	9'-4"	7'-1"	234'-11"	5	0	21'-3"

TODAY 9
TO DATE

PILE N^os 456 & 458 PULLED OUT SHORT; NO LATERAL SUPPORT.

PILE N^os 456 - 457 - 458 & 459 ABANDONED.

PILE № 475 (8 5/8) DRIVEN SEPT 7/73 PULLED OUT & REPLACED WITH 10 3/4

COMMENTS

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Lteé
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. BENTA 212-110

PILE TYPE TUBE PILES

PILE SIZE 9 5/8 & 8 5/8 10 3/4

HAMMER LBS. 6.780 DROP HT. 3' 4'

VOL. BUCKET —

DRIVE TUBE —

DATE NOV, 14/73 JOB NO. E-2910

JOB LOCATION QWY CARL, HOSPITAL

JOB INSPECTOR K. BALLIS

JOB ENGINEER N. I. McGILL

PILING CONTR. BENTA

PILING SUPT. L. SEVIGNY

PILE NUMBER	SIZE	THICK	TONS	DRIVING DEPTH	PILE LENGTH	BOTT ELEV	FINAL PENETRATION		TUBE LENGTH
		[redacted]	[redacted]				Nº BLOWS	PENET.	
516	9 5/8	.435	85	20'-9"	19'-7"	233'-5"	5	0	30'-1"
517	9 5/8	.435	85	20'-11"	19'-11"	233'-1"	5	0	27'-2"
518	8 5/8	.325	65	19'-1"	20'-0"	233'-0"	5	0	34'-7"
519	8 5/8	.325	65	20'-7"	20'-3"	232'-9"	5	0	39'-10"
470	9 5/8	.435	85	10'-1"	6'-6"	235'-0"	5	0	12'-1"
471	10 3/4	.435	85	10'-0"	6'-5"	235'-1"	5	0	12'-3"
467	9 5/8	.435	85	9'-3"	5'-10"	235'-8"	5	0	10'-10"
468	9 5/8	.435	85	9'-2"	5'-10"	235'-8"	5	0	15'-4"
TO DAY	8								
TO DATE									

COMMENTS

McROSTIE SETO GENEST
& Associates Ltd. — & Associés Ltée
CONSULTING ENGINEERS — INGÉNIEURS CONSEILS
OTTAWA, ONT.

RIG NO. 2

PILE TYPE TUBE

PILE SIZE 9 5/8 ; 8 5/8

HAMMER LBS. 3.500 DROP HT. 4'

VOL. BUCKET —

DRIVE TUBE —

DATE SEPT 7/73 JOB NO. E-2910

JOB LOCATION QWY CARL, HOSPITAL

JOB INSPECTOR K. BALLIS

JOB ENGINEER N. I. McGILL

PILING CONTR. BENJA

PILING SUPT. C. LEVASSEUR

	PILE NUMBER	DIA	THICK	TONS	DRIVING DEPTH	PILE LENGTH	BOTT ELEV	FINAL PENETRATION			TUBE LENGTH
								REFU	BLOWS	RETAP	
OK	167	8 5/8	.325	60	11'-4"	8'-6"	233'-6"	0	5	/	18'-8"
	184X	9 5/8	.435	110	33'-4"	29'-10"	211'-8"	0	5	/	48'-6"
OK	375	8 5/8	.325	60	11'-9"	7'-4"	234'-2"	0	5	/	18'-1"
OK	374	8 5/8	.325	60	11'-10"	7'-8"	233'-10"	0	5	/	20'-10"
OK	362	9 5/8	.435	110	8'-10"	5'-11"	235'-7"	0	5	/	21'-0"
OK	363	9 5/8	.435	110	8'-10"	6'-0"	235'-6"	0	5	/	16'-2"
OK *	476	8 5/8	.325	60	10'-5"	7'-1"	234'-11"	0	5	/	15'-3"
OK	474	9 5/8	.435	85	11'-0"	6'-10"	234'-8"	0	5	/	18'-0"
OK.	473	9 5/8	.435	85	11'-0"	6'-9"	234'-9"	0	5	/	19'-8"
	TODAY	9									
	TO DATE	346									

* Replaced with 10¾" ϕ x .435 on Nov. 15/73

COMMENTS

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