A PRELIMINARY GEOTECHNICAL INVESTIGATION

TWO MULTI-STOREY TOWERS
500 FAMILLE- CÔTÉ AVE.
OTTAWA, ONTARIO

CLIENT CODE: **BATIMO101**
F/N: **GO-24-2537-00**

January 2025

CLIENT: **GROUPE EMD BATIMO**

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CONFIDENTIAL
Report presented to

Mr. Charbel Abou-Tayeh, P.Eng., GSC, PMP®

Project Director, Development & Construction
GROUPE EMD BATIMO

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1.0 INTRODUCTION

1.1 SUMMARY DESCRIPTION OF THE ASSIGNMENT

Groupe ABS Inc. (ABS) has been retained by Groupe Emd Batimo (Client) to conduct a preliminary geotechnical study to support the design for the proposed fourteen-storey, twelve-storey and four-storey buildings (Project) at 500 Famille-Côté Ave. (Site) in Ottawa, Ontario.

This study was carried out per the professional services proposal prepared by ABS on October 28, 2024 (Proposal Number: H242537). The proposal was accepted by the Client by means of signed back proposal on October 30, 2024.

This report is prepared specifically and exclusively for Groupe Emd Batimo (Client) and consultants potentially collaborating on the project. The use of this report or the reliance on it by a third party is the responsibility of such third party. Any modifications to the project must be reported to ABS so that the scope and relevance of the geotechnical study and the recommendations contained in this report can be reviewed and adjusted, if necessary. This report is subject to the limitations presented in Section 9.0 “Scope of the Report and Limitation of Liability”.

1.2 PROJECT UNDERSTANDING

ABS’s understanding of the Project is based on the Client’s email request and following correspondences between October 18 and 23, 2024 and based on the following documents that were received from the Client:

- Geotechnical Investigation – Proposed Long-Term Care Facility – Block 4 – 850 Champlain Street, Ottawa, Ontario – 2nd Revision, prepared by Paterson Group Inc. dated March 30, 2020;
- Geotechnical Investigation – Proposed Mixed-Use Development – Champlain Street at Jeanne D’Arc Boulevard, Ottawa, Ontario – 1st Revision, prepared by Paterson Group Inc. dated August 16, 2017;
- Phase I Environmental Site Assessment Update – 850 Champlain Street, Ottawa, Ontario, prepared by Paterson Group Inc., dated April 13, 2020;

It is understood that the site is currently a vacant land. It is also understood that the proposed project consists of a fourteen-storey building at the north of the Site, twelve-storey building on the south of the Site, and four-storey building at the west of the site. It is also understood that the proposed development will include two underground parking levels. At the time of writing this report, ABS was not provided with any structural or civil plans for the proposed Project. ABS understands that this is a pre-design geotechnical and environmental investigation and should, therefore, be considered preliminary in nature.

It is understood that the proposed multi storey buildings will be supported on deep foundation consists of driven piles to bedrock, such as steel H-piles or concrete filled close-end pipe piles, or caisson socketed into the bedrock. The proposed four-storey building will be supported on deep foundation as well.

It is important to emphasize that at the time of this report, ABS was not provided with any civil plans, structural drawings, or structural loads. Therefore, the recommendations provided in this report are preliminary in nature and will need to be reevaluated once the design loads are available.

1.3 SCOPE OF WORK

This report describes the subsurface conditions at this site and provide borehole location plans, records of borehole logs, and laboratory test results. Also, this report provides anticipated geotechnical conditions influencing the design and construction of the proposed multi-storey development.

2.0 SITE DESCRIPTION AND GEOLOGICAL FORMATION

2.1 SUMMARY OF SITE DESCRIPTION

The Site is located at the civic address 500 Famille-Côté Ave, Ottawa, Ontario. It is located in a residential neighborhood and bounded by Jeanne-d'Arc Boulevard from the north, Famille-Côté Ave from the east and Bilberry Drive from the west. A chain link fence separates the site from the pathway and the property to the south of the Site. The property is approximately 1.1 km to the South of the Ottawa River.

The site is currently a vacant land and was observed to be relatively flat. The Site was observed to be covered with vegetation cover consists of shrubs and hay growing towards the outskirts of the site. A few mature trees were observed on the north side of the site along Jeanne-d'Arc Boulevard. The site location is shown in Figure GEO-O1, Appendix 1.

2.2 GEOLOGICAL FORMATION

Based on published physiography maps of the area (Ontario Geological Survey) the Site is located within the Ottawa Valley Clay Plains. Surficial geology maps of the area indicate that the Site is located within fine-textured glaciomarine deposits composed of silt and clay, with minor sand and gravel. The surficial geology to the north of the Site towards Ottawa River consists of older alluvial deposits, colluvial deposits and organic deposits. To the south of the Site there is a Paleozoic bedrock formation.

Furthermore, based on the available well records from Ontario Well Records Map within proximity of the Site, the soil profile in the vicinity of the Site is anticipated to consist of a thick silty clay layer underlain by a till layer. Limited information was available about bedrock depth within proximity of the Site. The bedrock within the area is expected to comprise of limestone, dolostone, shale, arkose, and sandstone of Ottawa Group and Simcoe Group of Shadow Lake Formation.

3.0 FIELD INVESTIGATION AND TESTING

3.1 UNDERGROUND UTILITY CLEARANCE

Underground utility clearance was completed before commencing the fieldwork. Utility clearance requisitions were submitted to Ontario One Call (ON1Call) to obtain public utility locates. Public utility owners were informed, and all utility clearance documents were obtained before the commencement of drilling work. Private locates were completed by GFL Environmental.

3.2 SITE LAYOUT AND SURVEYING OPERATIONS

The Site was observed to be relatively flat. The borehole locations and surface elevations were surveyed by ABS with a GPS unit.

The locations and the coordinates of the boreholes in this study are presented on Figure GEO-01 in Appendix 1, as well as in the borehole log in Appendix 2.

3.3 FIELDWORK

The fieldwork was conducted between October 14 and 19, 2024, and consisted of drilling three boreholes. BH24-1 and BH24-2 were drilled to a maximum drilling depth of 27.43 meter below ground surface (mbgs) (El. 29.52 and 29.87 m, respectively), and BH24-3 was drilled to a maximum depth of 9.75 mbgs (El. 33.22 m).

The boreholes were drilled using a CME 55 rubber track-mounted drill rig, outfitted with hollow stem augers and NQ casing for wash boring. The equipment used for drilling was owned and operated by Forage Grenville Drilling of Grenville, Quebec. In BH24-1 and BH24-2, drilling proceeded with hollow stem augers down to 9.14 mbgs depth (El. 47.81 and 48.16 m) and then continued with wash boring using NQ casing afterward. In BH24-3, drilling was also conducted with hollow stem auger down to 9.75 mbgs (El. 47.24 m) followed by dynamic cone penetration test (DCPT) to refusal at 23.77 mbgs (El. 33.22 m).

Soil samples were obtained at 0.76 m intervals using a 51 mm outside diameter split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. SPT sampling was alternated with Field Vane Shear (FVS) test within the clayey silt deposit. The sampling interval was changed to 1.5 m sampling interval below 9.14 mbgs depth.

SPT sampler refusal and casing refusal were encountered at approximately 22.99 mbgs (El. 33.96 m) in BH24-1, and 24.54 mbgs (El. 32.76 m) in BH24-2. Drilling was continued with rock coring afterward using diamond drilling with NQ casing and wireline tooling in BH24-1 and BH24-2.

DCPT was only performed in BH24-3 to evaluate the soil consistency and continued to DCPT refusal at 23.77 mbgs (El. 33.22 m)

A 32 mm diameter standpipe monitoring well was installed in BH24-2 with screen installed in the overburden. Details and location information of the well are provided in Section 5.2 and summarized in Table 5.

The cored holes were sealed with bentonite holeplug and the boreholes were backfilled with auger cuttings and restored to the original ground surface. The approximate borehole locations are shown in Figure GEO-01, included in Appendix 1, and the borehole logs are provided in Appendix 2.

The field investigation, including drilling and sampling, was supervised on a full-time basis by ABS. All boreholes were logged during the drilling progress. All samples were labelled one by one as they retrieved. All soil samples were preserved in plastic bags to mitigate the risk of moisture loss during transportation to the geotechnical laboratory. Rock cores were laid and labelled in rock core boxes and transported to ABS labs. The Rock Quality Designation was immediately measured after coring to reduce the measurement errors caused by transportation induced damages to the rock cores.

4.0 LABORATORY WORK DESCRIPTION

The soil samples and rock cores collected during fieldwork were transported to ABS's laboratory for the purposes of analysis, identification, and classification. All samples underwent a thorough visual examination by a geotechnical engineer. Geotechnical laboratory testing was performed on representative soil samples to determine soil index properties that include grain-size analysis, Atterberg Limit, water content, and specific gravity. Mechanical properties such as consolidation characteristics of the silty clay and rock compressive strength were also measured using one-dimensional consolidation test and uniaxial rock compression test.

Eurofins Environment Testing in Ottawa carried out chemical tests on three representative soil samples including pH, chloride, sulphate, resistivity, electric conductivity, RedOx potential, and sulphide. Laboratory test results are included in Appendix 3.

All samples taken from the borehole that were not subjected to laboratory testing will be stored for a period of three (3) months from the date of the final report. Afterward, they will be destroyed unless a written notice regarding their disposition is provided to ABS in the meantime.

5.0 SOIL STRATIGRAPHY AND PROPERTIES

A summary of the soil stratigraphy at the Site based on the drilled borehole is summarized in the table below and detailed in the borehole log record presented in Appendix 2.

TABLE 1: SUMMARY OF THE STRATIGRAPHY ENCOUNTERED IN THE DEEP BOREHOLES

BOREHOLE	GROUND ELEVATION (m)	TOPSOIL DEPTH (m) [ELEVATION (m)]	SILTY CLAY DEPTH (m) [ELEVATION (m)]	TILL DEPTH (m) [ELEVATION (m)]	REFUSAL DEPTH (m) [ELEVATION (m)]	CRUSHED ROCK DEPTH (m) [ELEVATION (m)]	BEDROCK DEPTH (m) [ELEVATION (m)]
BH24-1	56.95	0.0 – 0.15 [56.95 – 56.80]	0.15 – 22.25 [56.80 – 34.7]	22.25 – 22.99 [34.7 – 33.96]	22.99 [33.96]	--	22.99 – 27.43 [33.96 – 29.52]
BH24-2	57.3	0.0 – 0.15 [57.3 – 57.15]	0.15 – 21.34 [57.15 – 35.96]	21.34 – 24.54 [35.96 – 32.76]	24.54 [32.76]	24.54 – 25.25 [32.76 – 32.05]	25.25 – 27.43 [32.05 – 29.87]
BH24-3	56.99	0.0 – 0.3 [56.99 – 56.69]	0.3 – 9.75 [56.69 – 47.24]	--	(DCPT) 23.77 [33.22]	--	--

5.1 SOIL STRATIGRAPHY

5.1.1 Topsoil

A thin layer of topsoil was observed in all boreholes. The topsoil observed to consist of silty clay with trace of organics and was brown to greyish brown with moisture content of moist.

5.1.2 Silty Clay

A silty clay layer was observed in all boreholes below the topsoil and extended to the till layer at depths ranging approximately 21.34 to 22.25 mbgs (El. 35.96 to 34.7 m), in BH24-1 and BH24-2, respectively. The silty clay was inferred to extent from 9.75 to 20.42 mbgs (El. 47.24 to 36.57 m) in BH24-3 based on DCPT blow counts.

The silty clay layer was observed to be desiccated (i.e., weathered and crusted) below the topsoil down to depths range between 1.37to 2.13 mbgs (El. 55.58 to 54.86 m). The weathered silty clay was observed to be brown with moisture content of moist.

Unweathered silty clay was observed in all boreholes and extended from 1.37 to 22.25 mbgs (El. 55.58 to 34.7 m) in BH24-1 and from 1.52 to 21.34 mbgs (El. 55.78 to 35.96 m) in BH24-2. In BH24-3, it was observed between 2.13 to 9.75 mbgs (El. 54.86 to 47.24 m) and inferred to continue to 20.42 mbgs (El. 36.57 m). The unweathered silty clay was observed to be grey with moisture content of moist to wet.

Two samples from the silty clay underwent hydrometer grain size analysis testing. Also, eight (8) samples from the silty clay layer underwent Atterberg Limit test and the samples were observed to have liquid limit (W_L) of 50% to 76%, plastic limit (W_P) of 26% to 39% and plasticity index (I_P) of 20% to 47% with natural moisture content (W_n) of 52% to 71% indicating that the silty clay layer has high plasticity and can be classified as CH to MH as per Unified Soil Classification System (USCS). The test results are summarized in Table 2. Test results of Atterberg Limit test are included in Appendix 3.

TABLE 2: GRAIN SIZE ANALYSIS AND ATTERBERG LIMIT TESTS SUMMARY – SILTY CLAY

BOREHOLE [SAMPLE ID]	DEPTH (mbgs) [ELEVATION (m)]	SIZE FRACTION (%)				ATTERBERG LIMIT (%)			I_L	W_N (%)	REMARKS
		GRAVEL	SAND	SILT	CLAY	W_L	W_P	I_P			
BH24-1 [SS8]	8.08 – 8.99 [48.87 – 47.96]	--	--	--	--	67	29	39	1.0	66	Clay of High Plasticity (CH) per USCS
BH24-1 [ST9]	10.7 – 11.3 [46.25 – 45.65]	0.0	1.0	44	55	--	--	--	--	62	Silt and Clay
BH24-1 [SS10]	13.7 – 14.3 [43.25 – 42.65]	--	--	--	--	55	29	26	1.0	55	Clay of High Plasticity (CH) per USCS
BH24-1 [SS11]	16.8 – 17.4 [40.15 – 39.55]	--	--	--	--	51	31	20	1.0	52	High Compressibility Silt (MH) per USCS
BH24-1 [SS12]	20.7 – 21.3 [36.25 – 35.65]	--	--	--	--	53	27	26	1.0	52	Clay of High Plasticity (CH) per USCS
BH24-2 [ST5]	4.6 – 5.2 [52.7 – 52.1]	0.0	1.0	49	60	--	--	--	--	72	Silt and Clay
BH24-2 [SS6]	6.1 – 6.7 [51.2 – 50.6]	--	--	--	--	76	30	47	0.9	71	Clay of High Plasticity (CH) per USCS
BH24-2 [SS7]	8.5 – 9.0 [48.8 – 48.3]	--	--	--	--	70	39	31	0.9	66	High Compressibility Silt (MH) per USCS
BH24-2 [SS8]	11.6 – 12.2 [45.7 – 45.1]	--	--	--	--	67	30	37	0.9	62	Clay of High Plasticity (CH) per USCS
BH24-2 [SS10]	17.7 – 18.3 [39.6 – 39.0]	--	--	--	--	50	26	23	1.1	51	Clay of High Plasticity (CH) per USCS

Note: W_L: Liquid Limit – W_P: Plastic Limit – I_P: Plasticity Index – W_n: Natural moisture content – G_s: Specific Gravity – I_L: Liquidity Index ($I_L = (W_n – W_P)/ I_P$) – USCS: Unified Soil Classification System.

The recorded SPT 'N' values within the weathered (crusted) silty clay ranged between 4 and 12 blows/300 mm. Pocket penetrometer test was performed on samples collected from the crusted clay and the inferred undrained shear strength within the crust ranged from 50 to 190 kPa, except for one sample recorded 390 kPa, indicating the weathered (crusted) silty clay has stiff to hard consistency in accordance with Canadian Foundation Engineering Manual (CFEM) 5th Edition 2023.

Within the unweathered silty clay, the recorded SPT 'N' values were very low and ranged from zero to 1 blows/300 mm. Most of the SPT spoon sampler had full penetration under weight of hammer (WOH); therefore, the SPT 'N' value was considered zero.

Field Vane Shear (FVS) test was conducted within the unweathered silty clay and the measured undrained shear strength ranged between 24 and 76 kPa indicating that the silty clay has soft to stiff consistency in accordance with CFEM. The sensitivity to disturbance of the silty clay was observed to be medium sensitivity (i.e., 2 < Sensitivity < 4).

Two (2) undisturbed soil samples were collected from approximate depths of 10.7 to 11.3 mbgs (El. 46.25 to 45.65 m) from BH24-1 and 4.6 to 5.2 mbgs (El. 52.7 and 52.1 m) from BH24-2. Two specimens were subjected to one-dimensional consolidation testing and the results are summarized in Table 3. In addition, specific gravity (G_s), and total unit weight of the saturated clay were measured.

TABLE 3: CONSOLIDATION PARAMETERS OF SILTY CLAY

BOREHOLE	SAMPLE	DEPTH (m) [ELEVATION (m)]	ESTIMATED P'_o (kPa)	P'_C (kPa)	OCR	C_r	C_c	e_o	G_s	ESTIMATED γ_t (kN/m³)
BH24-1	ST9	10.7 – 11.3 [46.25 – 45.65]	103	244	2.37	0.06	1.09	1.6	2.748	16.9
BH24-2	ST5	5.33 – 5.93 [52.7 – 52.1]	63	253	4.0	0.05	1.44	1.92	--	16.0

Note: P'_o: Effective overburden stress below ground surface – P'_c: Preconsolidation Pressure – OCR: Overconsolidation ratio - C_r: Recompression Index – C_c: Compression Index – e_o: Inital void ratio – G_s: Specific Gravity – γ_t: Total unit weight.

5.1.3 Sand and Gravel Till

A sand and gravel till layer with some clay and occasional boulders and cobbles was observed below the silty clay layer between 22.25 and 22.99 mbgs (El. 34.7 and 33.96 m) in BH24-1 and between 21.34 and 24.54 mbgs (El. 35.96 and 32.76 m) in BH24-2. The layer was also inferred based on DCPT blow counts in BH24-3 between 20.42 and 23.77 mbgs (El. 36.57 and 33.22 m). The layer was observed to be grey with moisture content of wet.

The recorded SPT 'N' value within the sand and gravel till in BH24-1 was 54 blows/300 mm, indicating that the till is dense in accordance with CFEM (2023). In BH24-2, coring started 21.95 mbgs (El. 35.35 m) through the bouldery till to 24.38 mbgs (El. 32.92 m). SPT spoon sampler refusal was encountered at 24.54 mbgs (El. 32.76 m).

5.1.4 Refusal and Bedrock

SPT sampler refusal was encountered in 22.99 and 24.54 mbgs (El. 33.96 and 32.76 m) in BH24-1 and BH24-2, respectively. Also, DCPT refusal was encountered at 23.77 mbgs (El. 33.22 m) in BH24-3.

Limestone bedrock was encountered and cored in BH24-1 and BH24-2 as described in Table 4. The bedrock was cored and sampled to approximately 4.44 m and 2.18 m into the bedrock in BH24-1 and BH24-2, respectively using diamond core NQ size.

During the core drilling, measurements including Total Core Recovery (TCR), Solid Core Recovery (SCR), and Rock Quality Designation (RQD) were carried out as part of the rock quality classification. TCR is defined as the sum of all recovered rock core pieces from a core run expressed as a percent of the total length of the core run. SCR is the total length of solid, full-diameter drill core recovered, taken as a percentage of the length of the core run. The RQD is defined as a percentage of the sum of the intact core pieces over 100 mm divided by the total length of core run. The TCR, SCR and RQD for the rock cores are presented in the borehole log records in Appendix 2.

Based on the retrieved rock cores from borehole, the bedrock was identified as Limestone interbedded with shale bedrock. It was observed to be weathered and fractured to intensely fractured with moderately close horizontal joints. In BH24-1, sound bedrock observed below 25.07 mbgs (El. 31.88 m), whereas in BH24-2, sound bedrock was not observed within the cored length.

The Limestone bedrock was observed to be strong, grey to dark grey, thinly bedded. It was observed to have poor to excellent quality based on RQD value of 20% to 83%. The rock cores are shown in Appendix 2.

TABLE 4: REFUSAL AND BEDROCK CORE SUMMARY

BOREHOLE ID	GROUND EL. (m)	BEDROCK SURFACE/REFUSAL EL. (m)	SOUND BEDROCK EL. (m)	CORE RUN ID	EL. (m)	TCR (%)	SCR (%)	RQD (%)	UCS (MPa)
BH24-1	56.95	33.96	31.88	RC14	33.96 – 32.52	93	89	49	136
				RC15	32.52 – 31.04	96	95	66	--
				RC16	31.04 – 92.52	99	99	83	--
BH24-2	57.3	32.76	--	RC12	32.76 – 32.05	Crushed rock			
				RC12	32.05 – 31.32	96	82	56	87
				RC14	31.32 – 29.87	79	66	20	--
BH24-3	56.99	33.22	DCPT Refusal						

Note: TCR: Total Core Recovery – SCR: Solid Core Recovery – RQD: Rock Quality Designation - UCS: Uniaxial Compressive Strength

5.2 GROUNDWATER

Groundwater was observed in open borehole during this investigation in BH24-1 and BH24-2 at 1.06 and 2.74 mbgs (El. 55.89 and 54.56 m), respectively. Also, wet samples were retrieved from depths ranging between 3 and 5.3 mbgs.

A standpipe monitoring well was installed in BH24-2 and its information is provided in Table 5. Groundwater level was taken on November 26, 2024.

The anticipated water level should be considered with caution as these conditions are only to those observed in the short term at the location and on the date specified in this report. It is important to emphasize that the groundwater level can be influenced by several factors, including, among others, precipitation, snowmelt, and modifications to the physical environment. Therefore, it can vary with seasons and years.

TABLE 5: MONITORING WELL SUMMARY

BOREHOLE ID	SCREEN INTERVAL (mbgs)	WATER LEVEL OBSERVATION			REMARKS
		INSTALLATION DATE	MEASUREMENT DATE	DEPTH (mbgs) [EL. (m)]	
BH24-1	--	--	Nov. 14, 2024	1.06 [55.89]	Measured in open borehole
BH24-2 MW	--	--	Nov. 19, 2024	2.74 [54.56]	Measured in open borehole
	6.1 – 9.14	Nov 19, 2024	Nov. 26, 2024	3.01 [54.29]	Measured in the well

5.3 CHEMICAL ANALYSIS

Chemical analyses were conducted by Eurofins Environment Testing in Ottawa to determine the pH, resistivity, sulphate, chloride, sulphide, electric conductivity, and RedOx potential of representative samples collected from the BH24-1 and BH24-2. The laboratory results for the chemical analysis are shown in Table 6 and included in Appendix 3.

TABLE 6: CHEMICAL TEST RESULTS SUMMARY

BOREHOLE ID	SAMPLE	DEPTH (MBGS)	pH	SULPHATE (%)	CHLORIDE (%)	RESISTIVITY (Ohm-cm)	REDOX (mV)	SULPHIDE (%)	ELECTRIC CONDUCTIVITY (mS/cm)
BH24-1	SS6	5.3 – 5.9	8.59	0.03	0.058	1282	114.7	0.01	0.78
BH24-2	SS9	14.6 – 15.2	8.57	0.04	0.128	685	102.6	0.04	1.46

6.0 DISCUSSIONS AND GEOTECHNICAL RECOMMENDATIONS

6.1 GENERAL

The recommendations provided within this report are based on our understanding of the project which is summarized in "Section 1.2" and on interpretation of factual information obtained from the borehole advanced during this subsurface investigation. It is understood that the proposed development consists of two multi-storey buildings and four-storey podium with two underground parking levels that cover the entire footprint of the three buildings.

There are several considerations, challenges and complexities involved in this project including:

- It is understood that this Project is currently in the pre-design stage. Therefore, it is important to emphasize that this report should be considered as preliminary in nature. ABS requests to be retained to review the contemplated foundation and earthworks designs once they become available and provide comments to ensure conformance with the general recommendations provided within this report.

- Excavation for the proposed development is expected to extend to a depth approximately 12 to 14 mbgs (El. 44.95 to 42.95 m). A temporary protection system (TPS) such as soldier piles and wood lagging, or secant wall will be required for the proposed excavation. The feasibility of either option shall be determined by the Client based on the pros and cons of each system.

- The native silty clay is sensitive and prone to disturbance and strength loss. If it is disturbed by over-excavation, remolding, equipment and foot traffic, or subjected to excess water, it will lose its initial strength and will need to be sub-excavated. Contractors should use excavation methods that minimize disturbance to the sensitive clay subgrades. This shall include the use of smooth-edged ditching buckets and avoiding construction traffic on subgrades. Also, it is recommended to use a 200 mm thick lean mix concrete mud-slab on the approved subgrade to protect the subgrade and provide an adequate working surface.

- Due to high fines content, the native silty clay subgrade is frost susceptible. If the construction schedule requires the work to be completed in winter conditions, care must be taken to prevent the subgrade from freezing using insulation or heating and hoarding. Subgrade shall be kept dry at all times, construction traffic on the subgrade shall be restricted to prevent disturbance. The subgrade shall be protected against frost until pouring the concrete mud slab.

- Current available guidelines entitled (Slope Stability Guidelines for Development Applications 2012) applicable in the City of Ottawa require slope stability assessment for slopes steeper than 1H:5V or 2 m high. The site was observed to be relatively flat with no significant change in grade. Also, the guidelines require global stability assessment for retaining walls of 1 m high or greater. No information was provided to ABS regarding the proposed grade raise and retaining walls. It is assumed there is no significant grade raise (i.e., < 1.0), and no retaining walls > 1 m high. Therefore, slope stability assessment was not conducted as part of this study.

- Steel H-piles or concrete filled close-end pipe piles driven to bedrock or caisson socketed into the bedrock can be used to support the proposed multi-storey buildings and the podium. The bedrock was encountered at approximately El. 33.96 to 32.05 m.

- It is not recommended to support the podium on a raft foundation on native silty clay. The reason is due to proximity of the proposed podium to the southern tower, stress interaction with the tower's foundation may result in overstressing the tower's deep foundation (i.e., the driven piles or the rock-socketed caissons) beyond its capacity. Therefore, this foundation was not considered in this report.

- The proposed structure will be designed under Part 4 of the Ontario Building Code (OBC 2012) and will, therefore, require a Seismic Site Class according to Table 4.1.8.4. Site Class was estimated using the weighted average SPT N-Value and undrained shear strength in the upper 30 m. The proposed structure can be designed using a seismic Site Class D.

- The silty clay deposit identified at the Site is a comprisable material and expected to consolidate over time in response to applied loads and grade raises. Also, lowering the groundwater level due to the influence of a permanent drainage, such as a perimeter drainage system may produce similar effect that may extend to the surrounding buildings. Therefore, it is recommended to seal the basement underground parking level and implement "bath-top" or tanking system to minimize the long-term settlement with the silty clay unit.

The purpose of this geotechnical investigation was to provide more information regarding the soil stratigraphy within the Site and to evaluate the mechanical properties of the native soils which will be used for the evaluation of the foundation and TPS design.

The comments made regarding the proposed mixed used commercial development are intended to highlight those aspects which could impact or affect the detail design of the foundation and Temporary Support System (TPS), for which special provisions may be required in the Contract Documents. Comments related to construction aspects are not intended to dictate construction equipment or methods. Relevant parties should make their own interpretation of the factual data presented in the report. Interpretation of the data presented may affect equipment selection, proposed construction methods, and scheduling of construction activities.

6.2 SITE PREPARATION

Excavations to accommodate the proposed two underground parking levels will extend through topsoil, and silty clay overburden down to approximately 7 to 8 mbgs (El. 49.95 to 48.95 m). Excavation needs to be performed with the support of Temporary Protection System (TPS) to protect the existing infrastructures, roadways and for public and workers safety.

The Site surrounding the excavation should be graded in the early stages of construction to provide positive control of surface water and directing it away from the excavation and subgrades. Appropriate provisions should be made for collection and disposal of groundwater, storm water and runoff including an adequate pumping system.

Excavation should extend down to the foundation level and at least 200 mm mud slab should be poured on approved subgrade. The mud slab will protect the subgrade, provide stable working floor, work as upward seepage barrier, and provide clean working space.

The excavated materials and any corresponding excess soil should be disposed off in accordance with all applicable environmental legislation. Excess soil management and evaluation of the environmental quality of subsoils is not within the scope of this geotechnical investigation.

6.2.1 Buried Services

Public and private utility owners should be notified prior to the commencement of any construction activities. Existing underground utilities in the vicinity of the proposed excavation should be reviewed before commencing any excavation works to identify potential damage hazards due to the proposed excavation. Existing utilities that are excavated or exposed as part of the construction will need to be supported and rerouted during the construction. Even with a shoring system, some inward movement of shoring is inevitable. This may cause slight ground settlement which may have an adverse effect on the existing buried utilities. The contractor shall inform owners of all existing utilities before proceeding with excavation. The utility owners may provide the permissible deformation that a particular utility may tolerate. Shoring shop drawings should be stamped by a professional engineer.

6.2.2 Excavation Impact on Adjacent Structures

The Designer and Contractor should account for the influence of the excavation on nearby structures and roadways. If the foundation influence zone intersects with the excavation, a case-specific review shall be provided by a structural engineer. The influence zone is defined by a 1H:1V outward and downward from the bottom of the footings. If any adjacent load bearing element is subject to undermining, then an Engineered Shoring system and/or underpinning program will need to be considered.

6.3 EXCAVATION

Excavation for the proposed two towers and podium with the proposed two underground levels is expected to extend to a depth between approximately 7 to 8 mbgs (El. 49.95 to 48.95 m). The base of the excavation is anticipated to be firm to soft silty clay. The excavation shall be conducted with the aid of temporary protection system (TPS).

All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), Regulations for Construction O.Reg. 213/91. The excavation of the overburden is expected to be performed using conventional hydraulic excavation equipment. The excavation for the proposed structures needs to be undertaken within the confines of a TPS or Engineered Shoring designed and installed in accordance with OHSA. The shoring will need to support the excavation sidewalls and act as a barrier against groundwater flow into the excavation. However, the removal of water within the shored excavation will still be required. Further discussion on the Engineered Shoring is provided in Section 6.3.2.

Local excavations shall be performed with specific reference to acceptable size slopes and stabilization requirements of OHSA. The general stratigraphy outlined herein can be considered an OHSA Type 3 Soil above groundwater and Type 4 Soil below groundwater. Local excavations should be conducted through a minimum 1H:1V or a flatter slope for Type 3 Soil and 3H:1V below groundwater (i.e., Type 4 Soil). The stability of the excavation side slopes is highly dependent on the Contractor's methodology and layout.

No surface surcharges should be placed closer to the edge of the excavation than a distance equals to twice the depth of the excavation, unless a TPS has been designed to accommodate such a surcharge.

6.3.1 Temporary Support System or Engineered Shoring

A TPS or Engineered Shoring system is required during both excavation and construction stages to protect the adjacent structures, roadways, utilities, and to protect the public safety. Engineered Shoring systems such as soldier pile and wood lagging, secant and/or tangent walls, permanent diaphragm walls are often used to support excavations through soil. The design of the Engineered Shoring system is the responsibility of the contractor. The Contractor should hire an experienced professional Geostructural Engineer to provide a detailed design for the Engineered Shoring system considering the space restrictions, estimated costs, and availability of materials. The Engineered Shoring designer must take into consideration the loads from any adjacent structures or infrastructure being retained, lateral earth pressures, groundwater pressure, seismic loading, construction surcharge loads, and pre-stressing loads or post tensioning loads on tiebacks. Also, it should consider the freeze-thaw action on the face of excavations, expansion and contraction of shoring elements, construction vibrations and compatibility with the design of proposed waterproofing and drainage systems for the sub-surface levels.

The TPS must be designed and constructed in accordance with OPSS.MUNI 539. TPS Performance level 1a with maximum 5 mm horizontal deflection can be considered adequate to support the nearby buildings, utilities, and roadways.

An anchoring and/or internal bracing system must be implemented into the shoring system to resist lateral earth pressure loadings. It is recommended that the protection system to be left in place and cut off in accordance with OPSS.MUNI 539.

A preconstruction survey is recommended at the outset of the project. The magnitude of ground movements adjacent to the excavation should be monitored throughout the construction. The performance level of the TPS and the threshold alert level for movement adjacent to the existing building or utilities should be determined by the shoring designer. Stockpiling of soil beside the excavations should be avoided. The weight of the stockpiled soil could lead to overstressing the shoring system.

It is recommended that the Client retain a Contractor and a Designer who have significant experience with excavations performed under similar soil conditions. Shop drawings should be submitted to the designers and reviewed by the geotechnical engineer well in advance of mobilization.

Fully mobilized (i.e., active lateral earth pressure coefficient) conditions should be considered. The lateral earth pressure coefficients for existing fill are given in Table 10 in Section 6.5 to assist Designer and Contractor with the design of the shoring system. Hydrostatic pressure build-up behind the shoring system should be considered in the design.

6.3.2 Soil Anchor (Tieback) Design Parameters

The following design parameters per the Limit States Design (LSD) method can be used for tiebacks. The soil anchors or tiebacks can be designed based on frictional stress between the grout and silty clay. The Ultimate Limit States (ULS) and Serviceability Limit States (SLS) bond stress values must be based on both performance and structural criteria. However, based upon typical published values, the unfactored ULS bond stress for straight shaft pressure-grouted anchors along the anchor bond zone values may be taking approximately 30 kPa for silt clay as per Ground Anchors and Anchored System (FHWA-IF-99-015).

Pressure grouting is an effective installation method in cohesive soils; however, post-grouting is a more effective method to obtain higher capacity. Post-grouting increases the radial stress acting on the grout body and cause irregular surface to be developed around the bond length that tends to interlock the grout and the ground. Delayed multiple grout injections to enlarge the grout body of straight shafted gravity grouted ground anchors can be used. Each injection is separated by one or two days. The high-pressure grout fractures the initial grout and wedges it outward into the soil enlarging the grout body. Further guidelines on shoring systems can be provided when needed.

As per Table 6.2 of CFEM (2023), it recommends a geotechnical resistance factor of 0.4 be applied to the empirical unfactored ULS values under typical Degree of Understanding of the site. The contractor shall conduct performance tests at the outset of the Project to verify the anchor capacities. Performance tests shall be performed on the first three production anchors installed and thereafter on a minimum of 2% of the remaining production anchors. Higher stress values may be used if performance load testing in the field is conducted to prove the capacities. If performance testing is carried out at the outset of the Project, then a resistance factor of Φ = 0.6 could be applied.

6.3.3 Subgrade Preparation

It is understood that the proposed towers and the podium will be supported on piles driven to bedrock or caissons socketed in the bedrock. It is also understood that the Client and the Designer may consider a raft foundation to support the proposed podium. However, two underground parking levels are assumed with the excavation extends down to 7 to 8 mbgs (El. 49.95 to 48.95 m). The anticipated soil at this depth is soft to firm silty clay.

The native silty clay on this Site is sensitive and may lose its strength if it is disturbed or remolded by over-excavation, construction traffic, or exposed to excess water. Contractors should use excavation methods that minimize disturbance to the silty clay subgrade. Final excavations should be performed with a smooth-edged ditching bucket to expose native and undisturbed silty clay subgrade. A Geotechnical Engineer must review and approve foundation subgrade to ensure that the native silty clay subgrade is free of any organics, roots, fill, loose or disturbed soils and can support the design bearing pressure. Soft and loose spots and any identified local anomalies should be subexcavated and replaced with engineered fill in accordance with the comments in Section 6.7.1. The excavation should be sub-excavated to accommodate a 200 mm thick concrete mud slab (forms part of the water suppression system discussed in Subsection 6.6) to protect the approved subgrade surfaces and to provide a stable and clean dry working surface for construction operations.

6.3.4 Temporary Construction Dewatering

Groundwater was observed in open boreholes BH24-1 and BH24-2 and in the monitoring well in BH24-2. The water level was measured in open BH24-1 and BH24-2 at 1.06 and 2.74 mbgs (El. 55.89 and 54.56 m), respectively. It was also measured in the monitoring well in BH24-2 at 3.01 mbgs (El. 54.29 m). The observed groundwater levels were all above the base of the proposed excavation. Groundwater and surface runoff water due to seasonal changes and extreme weather events may seep and accumulate at the bottom of the excavations. Water quantities will depend on seasonal conditions, the depths of excavations, and the duration that excavation is left open.

Contractors should be prepared to handle any surface or groundwater infiltration by ditching, pumping and/or other methods to maintain dry working conditions. Recommendations for appropriate dewatering measures beyond conventional sump pump technique or other more intensive dewatering systems (e.g., well points or other specialized methods) to effectively lower the static groundwater level shall be provided by a specialized dewatering contractor. The excavated subgrade must be kept dry at all times (until the mud slab is poured) to minimize the disturbance of the subgrade. If a dewatering program is considered, groundwater should be lowered at least 0.5 m below the excavation level to avoid disturbing the silty clay during compaction and construction.

Any surface water infiltrating into the open excavation can be removed through conventional sump and pump methods. Existing utility trenches which join or intersect the excavations may act as a drain and supply off-Site water into the excavations. These should be plugged at the outset of construction in an attempt to mitigate this possibility.

Based on the O.Reg 63/13 and O.Reg 387/04, a Permit to Take Water (PTTW) Category 3 from the Ontario Ministry of the Environment, Conservation and Parks (MECP) will be required if the quantity of water to be pumped from the Site exceeds 400,000 L/day. For expected groundwater extraction between 50,000 and 400,000 L/day, an Environmental Activity and Sector Registry (EASR) permit is adequate. The volume of pumped water per day is a function of the size of the excavated area and the dewatered zone.

The hydraulic conductivity for the silty clay sand is expected to be in the range of $1x10^{-6}$ to $1x10^{-8}$ cm/s. The hydraulic conductivity values are estimated based on soil gradation analysis. In-situ percolation tests were not performed as part of this investigation. The provided hydraulic conductivity values can be used for the selection of the pump capacity for dewatering.

Further assessment of the dewatering requirements and the need for registration on the EASR or a PTTW will require a hydrogeological assessment and should be carried out by specialists experienced in this field.

It is the contractor responsibility to obtain a permit from the City of Ottawa to discharge the drained water into the municipal sewer system.

6.4 FOUNDATION

Based on our understanding of the proposed development, the following foundation options are feasible from a geotechnical perspective for both towers and the podium:

- Driven piles to bedrock (i.e., steel H-piles or concrete filled close-end pipe piles); or
- Rock-socketed cast-in-place concrete caissons.

6.4.1 Driven Piles

The piles shall be installed in accordance with OPSS 903 requirements. Based on our understanding, the proposed underside elevation of the pile cap is at approximately El. 44.95 to 42.95 m.

The soil stratigraphy included silty clay, bouldery till, crushed rock and limestone bedrock. Driving the piles through the bouldery rock will be challenging. It is recommended to perform an initial Wave Equation Analysis (WEAP Analysis) to assess the drivability of the proposed pile sections with respect to the hammer type, hammer energy, and the site soil conditions and to estimate the pile bearing capacity.

6.4.2 Axial Resistance

Steel H-piles or concrete filled close-end pipe piles driven to bedrock can be used to support the proposed buildings. The provided axial resistance values in this report are estimated based on geotechnical field test and observation during site investigation. The estimated axial geotechnical resistance was based on the recommendation of Section 9.10 of CFEM (2023).

The driven piles shall be driven to practical refusal on bedrock at approximate El. 31.88 m. Rock points/pile shoes will be required for all piles being seated into the bedrock. Relatively sound bedrock is estimated roughly at or below approximately 25.07 mbgs depth (El. 31.88 m).

Cores collected from the limestone bedrock were subjected to uniaxial compression test and the compressive strength ranged between 87 and 136 MPa as shown in Table 4. The geotechnical resistance at the Ultimate Limit State (ULS) was estimated following Ladanyi and Roy (1971) method discussed in Section 9.10.3 of CFEM (2006).

The Estimated **Factored** ULS geotechnical capacity including a geotechnical resistance factor ϕ_{gu} of 0.4 is provided in Table 7.

TABLE 7: PRELIMINARY FACTORED ULS GEOTECHNICAL CAPACITY

PILE TYPE	PILE DESIGNATION IMPERIAL (METRIC)	PRELIMINARY FACTORED ULS GEOTECHNICAL CAPACITY (kN)
H-PILE	HP 12x74 HP 310x110	900
CONCRETE FILLED CLOSED-END PIPE PILES	12 ¾” x 0.687” 324 mm x 17.45 mm	850

All piles shall be monitored during pile driving to ensure they have met the refusal criteria for the designated capacity. For design purposes, the settlement of piles driven to sound bedrock under SLS conditions is generally expected to be less than 10 mm to 15 mm (excluding the elastic deformation of the piles themselves).

The piling contractor will need to confirm the estimated pile capacity considering the driving energy of the proposed equipment using approved empirical methods at the outset of the Project. The piling contractor should carry out piling calculations according to Section 9.2 of CFEM 2023. Typical piling calculations would include the wave-equation analysis, Pile Driving Analysis (PDA) or other methods based on the contractor’s equipment. A geotechnical engineer must be retained to review and approve the piling calculations prior to mobilization and confirm the development of the necessary piling refusal criteria for use with this Project at the onset of piling operations.

Pile driving can induce excess pore water pressure within the silty clay immediately surrounding the pile resulting in reduction of the pile geotechnical resistance. The resistance, however, is expected to increase with time as the pore pressure dissipates after initial installation. It is recommended that a wait period of at least 7 days allowed before retaping for confirmation of the pile geotechnical resistance

ABS recommends that the installation of all piles be witnessed and reviewed by a geotechnical engineer or a qualified technician acting under the supervision of a geotechnical engineer on a full-time basis to verify the tip elevation, location, verticality, and to ensure that the design set criteria and the required pile capacity has been achieved. Pile splices will require inspection by a CWB welding inspector.

It is recommended that Pile Driving Analysis (PDA) be performed on one pile (i.e., a minimum of 10% of the piles) and be completed at onset and production stages of the Project. Firstly, at the onset of pile driving to confirm the set criteria established for this Project; and secondly, on any piles that are considered suspect.

6.4.3 Downdrag Force

Due to the presence of the soft silty clay layer, consolidation settlement is expected if grade raise is required. Downdrag forces need to be considered for deep foundation in areas where grades have increased from original. The downdrag forces can be estimated using the following formulas.

$$P_{neg.} = \alpha C\left(q'_{\circ} + \frac{\gamma' L_1}{2}\right) L_1 K$$

$$P_{pos.} = \alpha C\left(q'_{\circ}(L - L_1) + \frac{\gamma'(L^2 - {L_1}^2)}{2}\right) K$$

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{q'_{\circ}}{\gamma'}\right) - \frac{2q'_{\circ}}{\gamma'}$$

where:

$P_{neg.}$ = negative skin friction resistance (kN)
$P_{pos.}$ = Positive skin friction (kN)
α = coefficient relating the effective lateral pressure
C = caisson perimeter (m)
K = Lateral earth pressure coefficient; K = $K_{\circ} = 1 - \sin\phi$
L = Pile length (m)
L_1 = The distance to the neutral point (m)
q'_0 = Surcharge or fill pressure (kPa)

The downdrag load or negative skin friction on deep foundation can be factored by using a multiplier load factor of 1.25. The factored downdrag load should be added to the factored permanent loads to assess the effect of the downdrag. The factored dead loads and downdrag load should not exceed the factored structural resistance of a pile.

To reduce the downdrag acting on the piles, several options are available. Section (9.2.5.8.) of CFEM 2023 recommends applying bituminous or other viscous coating to the pile surface before installation.

If there are no permanent additional surcharge (dead loads) on the subgrade, downdrag forces can be neglected. However, if grade raise up to 1 m is envisioned, an unfactored negative skin friction of 140 kN should be considered.

6.4.4 Uplift Resistance

The unfactored uplift resistance of the proposed steel H-piles or concrete filled close-end pipe piles can be calculated using the following formula (Bowles 1997):

$$P_{up.} = \sum_{Z=0}^{L} Cq_s \, \Delta z + W_p$$

where:

- $P_{up.}$ = Unfactored uplift resistance (kN);
- C = Pile circumference (m);
- Δz = subdivided segment of the embedded length (m);
- W_p = Pile weight (kN); and
- q_s = Soil unit shaft friction.

The estimated unfactored uplift resistance based on the above formulas is approximately 320 kN per pile. CFEM 2023 recommends using a geotechnical resistance factor of 0.3 for deep foundation under tension.

Also, the dead weight of the structural elements can provide resistance to uplift and overturning forces. If higher uplift resistance is required, consideration should be given to increase the dead weight of the structure.

6.4.5 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading would be derived from the soil in front of the piles.

The lateral load response of a single pile may be estimated using the soil coefficient of lateral subgrade reaction that can be estimated as follows:

1. For cohesionless soils, the following formula given by (Terzaghi 1955) can be used.

$$k_h = n_h \frac{Z}{D}$$

where:

k_h = coefficient of horizontal subgrade reaction;
n_h = a soil type and condition-related factor given in the following table (kN/m^3). A value of 2,800 kN/m^3can be used for the till at this Site.
Z = depth (m); and
D = Pile diameter/width (m);

Since the estimated k_h value using the above formula increases significantly at greater depths, Bowles (1997) recommended using $(Z/D)^n$, where n is a fitting parameter ranged between 0.4 to 0.7.

2. For cohesive soil, the following formula by (Davisson 1972) can be used.

$$k_h = \frac{67\, c_u}{D}$$

where c_u is the undrained shear strength of the soil (kPa). An average undrained shear strength value of 30 kPa can be used at this Site.

The coefficient of horizontal subgrade reaction k_h may need to be reduced, based on the pile spacing, to account for pile group effects as per Section 9.9 of CFEM (2023).

TABLE 8: GROUP EFFECTS REDUCTION FACTORS FOR k_h (from NAVFAC (1986) Design Manual 7.02).

PILE SPACING IN DIRECTION OF LOADING	SUBGRADE REDUCTION FACTOR
8D	1.00
6D	0.70
4D	0.40
3D	0.25

6.4.6 Rock-Socketed Cast-in-Place Concrete Caissons

The use of liner or casing will be required to advance the caisson with minimal loss of ground since the overburden materials may not stand unsupported. It is also recommended that the casings be left-in-place as a permanent component of the caissons. Otherwise, if the casings are withdrawn during the pouring of concrete, there is a risk of creating defects due to movement of soil into the concrete. Additionally, it will be difficult to clean the bedrock socket/surface, even with the use of casings, unless the casings are socketed into the bedrock.

The axial resistance of caisson foundation can be derived based on either end-bearing resistance on well cleaned competent rock, sidewall of shaft shear resistance (end-bearing is excluded), or from both end-bearing resistance and shaft resistance. In this case, consideration must be given to the load-transfer behavior along the shaft.

To provide suitable fixity, the caisson should be provided with a minimum socket length equal to two (2) times the socketed diameter or 3 m, whichever is greater, into non-weathered sound bedrock. A minimum caisson diameter of 0.9 m or greater is recommended to facilitate inspection. The caisson socket shall advance through the weathered rock to the sound rock and the minimum socket length shall start from below the weathered rock depth in the sound rock. Sound bedrock was observed only in BH24-1 below El. 31.88 m. In BH24-2, sound bedrock could not be located with the coring depth. It is recommended to advance additional boreholes within the proposed building locations to determine the extent and quality of the bedrock during the detailed design phase.

The socket must be hydro vacuumed and cleaned thoroughly and inspected by a geotechnical engineer before pouring the concrete. End-bearing must not be considered if the base of the caisson was not hydro vacuumed and cleaned before pouring the concrete. If dewatering the sockets is not feasible, it should be planned to use tremie technique to construct the caissons under wet condition. In this case end-bearing must not be considered.

It should be noted that casing installation through the till or the crushed rock will be difficult. The foundation installation contractor should be made aware that chiseling/churn drilling or other methods will be required to advance the caissons through the till.

The structural resistance of the caissons must be checked by the structural designer. The actual socketed length required should be lengthened, if required, based on the lateral capacity, moment capacity and seismic analysis to satisfy the structural assessment.

The sedimentary limestone bedrock is strong to very strong with uniaxial compressive strength ranges from 87 MPa to 136 MPa. The caisson rock sockets will have to advance by rock coring, chisel/churn drilling, and/or a down-the-hole hammer technique.

6.4.7 Caisson Axial Resistance

Rock-socketed caissons should be designed based on the sidewall or shaft resistance of the rock socket. The estimated factored geotechnical capacity for 0.9 m diameter caisson and socket length of 1.8 m into sound bedrock is 2,000 kN. The factored geotechnical capacity was estimated following recommendations available in Section 9.10.4 of CFEM (2023). The estimated geotechnical resistance includes a geotechnical resistance factor of 0.4 per Table 6.2 of CFEM 2023.

If greater resistance is required, a deeper rock socket could be utilized. Alternatively, higher capacity may be considered if the caisson is designed based on end-bearing resistance. However, the rock-socket must be hydro vacuumed and cleaned and approved by the geotechnical engineer before pouring the concrete.

The SLS resistances do not apply to caissons socketed in the bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.4.8 Downdrag Force

Downdrag forces can be neglected if there is no grade raise envisioned. For up to 1 m grade raise, downdrag or negative skin friction should be considered for the caissons and is estimated to be 450 kN per caisson.

6.4.9 Uplift Resistance

Factored uplift resistance can be estimated based on the shaft capacity of the socket. A factored uplift resistance of 1,000 kN. This includes a geotechnical resistance factor of 0.2.

6.4.10 Resistance to Lateral Loads

Lateral loads on the proposed buildings can be geotechnically resisted through passive pressure developed along the socketed portion of the caissons into the bedrock. The Designer of the laterally loaded rock-socketed caissons must take into account the relative rigidity of the caissons with respect to the surrounding rock, the head fixity condition of the caissons, the structural capacity of the caissons to withstand bending moments and shear, the rock resistance that can be mobilized and the maximum tolerable lateral deflection at the top of the caissons.

Lateral loads resistance can be mainly derived from the passive pressure developed along the rock-socket portion. If soil subgrade reaction is required for the portion that is embedded in the soil, recommendations presented in Section 6.4.5 of this report is still valid.

ABS understands that the project is still in the pre-design phase and there is limited information available for lateral resistance design for the caissons. Additional recommendations can be provided during the detailed design phase once more information is available.

6.4.11 Floor Slab

Typical options for floor slab for underground parking would be a flexible asphalt pavement, a rigid free-floating slab on grade, or alternatively a structural slab. ABS was not provided with information with respect to design criteria for floor slab loadings and traffic loadings of the underground parking garage. Therefore, ABS has assumed that floor slabs are lightly loaded with no heavy racking or process machinery that require specific support.

A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 25 kPa. For design purposes and based upon a properly prepared native subgrade surface covered with 200 mm of Ontario Provincial Standard Specification (OPSS) 1010 Granular A, a typical preliminary modulus of subgrade reaction appropriate for the slab design would be approximately 25 MPa/m on engineered fill compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD). Alternative values would require additional analysis and testing.

For the unheated portions of the buildings or slabs that are exposed to cold temperatures, the slab subgrade shall be insulated. The insulation shall be load-bearing and spread below the slab for the entire width. It is the Designers' responsibility to determine the thickness of insulation based on the required R-value, equivalent to 1.8 m of earth material insulation value. The insulation shall extend beyond the slab thickenings to a distance equivalent to 1.8 m of earth material insulation value.

Subgrade preparation below floor slabs will involve the removal of all topsoil, organic matter and unsuitable soils to expose a competent native undisturbed silty clay subgrade. The subgrade for the slab-on-grade shall be prepared as per Section 6.3.3 of this report and shall be reviewed and approved by the Geotechnical Engineer

6.4.12 Grade Raise

No information was provided regarding any proposed grade raise for the Site. A **preliminary** grade raise of up to 1.0 m is allowed for this Site considering all the buildings will be supported on driven piles or caissons. If the type of the foundation is changed, then additional investigation and assessment may be required.

6.4.13 Frost Protection

Based on the Foundation Frost Penetration Depths for Southern Ontario (OPSD 3090.101), the frost penetration depth is 1.8 m below the existing ground surface.

Based on the subsurface investigation results, the native silt clay soil is classified as high frost-susceptible material. Frost susceptible materials are prone to frost heave and differential settlement. Therefore, all perimeter and exterior foundation elements, or interior foundation elements in unheated areas should be provided with a minimum of 1.8 m of earth cover or equivalent thermal rigid insulation for frost protection purposes. Frost protection depth can be reduced to 1.5 m for those buildings constantly heated during the cold season. The burial depth of water-bearing utility lines is typically 2.4 m below the ground surface.

Should construction take place during winter, surfaces that support foundations or Engineered Fill must be protected by Contractors against freezing for the entire duration of construction or until adequate soil cover is in place. Backfill soils should not be placed in a frozen condition or placed on frozen subgrades.

6.4.14 Seismic Parameters

6.4.14.1 Seismic Site Classification

In accordance with the OBC-2012, structures designed under Part Four of the Code must be designed to resist a minimum earthquake force. This classification system is based on the average soil properties in the upper 30 m below the foundation level. Based upon the results of the drilling program, we recommend that this structure be designed to "Site Class D", with respect to Table 4.1.8.4.A of the OBC-2012, and subject to the limitations of the code.

6.4.14.2 Spectral Acceleration

The spectral acceleration values for different periods and the Peak Ground Acceleration (PGA) value for various cities and municipalities are specified in the National Building Code. The spectral acceleration data for different periods in the general vicinity of the site for a 2% chance of exceedance in 50 years (2475 years return period) are presented in the following table.

TABLE 9 : SEISMIC SPECTRAL RESPONSES FOR SITE CLASS D (2% IN 50 YRS)

Spectral Acceleration							
Sa (0.2)	Sa (0.5)	Sa (1.0)	Sa (2.0)	Sa (5.0)	Sa (10.0)	PGA (g)	PGV (m/s)
0.671	0.53	0.315	0.15	0.0417	0.0131	0.398	0.372

Note: Seismic Data from the Interpolation Tool of Natural Resources Canada website: https://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=45.483&longitude=-75.522&siteDesignation=XS&siteDesignationXS=D

The above table noted spectral responses are for reference only, and it may not indicate the critical spectrum for the proposed structure. Calculations are based on the 2020 hazard map. The structural engineer shall consider deriving design specific spectral responses.

6.4.15 Verification of Liquefaction Potential

Preliminary liquefaction susceptibility screening of the silty clay layers was performed using SPT test results following recommendations of the Canadian Foundation Engineering Manual (CFEM). It was concluded that the silty clay is not susceptible to liquefaction under "Typical Consequence" as per Section 18.6.3. of CFEM (2023).

6.5 LATERAL EARTH PRESSURES

The following lateral earth pressure parameters are provided to assist Contractors and Designers with the design of the proposed structures and temporary protection system or Engineered Shoring systems. The provided discussion below is for fully drained backfill. The Designer should consider any potential for hydrostatic pressure buildup. Compaction of backfill behind foundation wall and retaining structures can induce loads greater than the active or at-rest earth pressures. Therefore, the induced lateral earth pressure due to compaction should be added to the calculated earth pressure in accordance with Section 20.4 of CFEM (2023).

6.5.1 Static Conditions

The following static lateral earth pressure coefficients are recommended.

TABLE 10: LATERAL EARTH PRESSURE DESIGN PARAMETERS FOR BACKFILL AND NATIVE SOIL

MATERIAL	PARAMETER					
	ϕ' (°)	c' (kPa)	γ (kN/m³)	K_o	K_a	K_p
Granular A	34	0	22	0.44	0.28	3.54
Granular B II	32	0	22	0.47	0.31	3.25
SSM (Backfill)	30	0	20	0.5	0.33	3.00
Native Silty Clay	26	0	15	0.56	0.39	2.56
Till	32	--	19	0.47	0.31	3.25

Note: ϕ': effective friction angle – c': effective cohesion – γ: unit weight – K_o: coefficient of lateral earth pressure at rest – K_a: coefficient of active earth pressure – K_p: coefficient of passive earth pressure (for a vertical slope and a horizontal surface)

Static lateral earth pressure can be calculated by using the following equation:

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

where K is the lateral earth pressure coefficient. For yielding retaining walls, the active earth pressure coefficients, K_a, is recommended to be used. For non-yielding permanent walls, such as foundation walls, the at-rest, K_o, is recommended to be used for design. The resultant of the applicable static or at-rest force is assumed to act at *h* = 1/3H above the base of the wall where H is the Height of the wall. The unit of the retained soil "γ" is given in Table 10, and "q" is the value of any applied surcharge.

The above noted lateral pressure coefficients are calculated assuming the wall back angel is vertical and the backslope of the retained soil is horizontal. The wall-soil interaction angle is assumed to equal to 0.5 ϕ' as per CFEM. If temporary protection system is used, then designers should refer to CFEM for design assistance and a geotechnical engineer should be retained to perform the shoring design review.

6.5.2 Seismic Conditions

In accordance with Section 20.5 of CFEM (2023), soil retaining structures should be designed considering lateral forces due to seismic forces. Seismically induced lateral earth pressures may be calculated using Mononobe-Okabe method. Both geotechnical and geometric components are considered.

The total active earth pressure (P_{ae}) including static and dynamic loads can be expressed as follows:

$$P_{ae} = \frac{1}{2}\gamma H^2(1-k_v)K_{ae}$$

where:

H = Height of the wall,
K_{ae} = horizontal component of active earth pressure coefficient including effects of earthquake loading,
k_v = Vertical component of the earthquake acceleration typically a range of 2/3 x k_h to 1/3 k_h is considered, but a value closer to 2/3 x k_h is recommended.
k_h = Horizontal component of the earthquake acceleration.

For passive earthquake pressure (P_{pe}) the following equation can be used:

$$P_{pe} = \frac{1}{2}\gamma H^2(1-k_v)K_{pe}$$

where:

K_{pe} = horizontal component of passive earth pressure coefficient including effects of earthquake loading

The resultant active and passive earth pressures in the above equations include both the active pressures under static (Pa) and the passive earth pressure under static (P_p), respectively, as well as the incremental loads induced by earthquake forces (ΔP_{ae} and ΔP_{pe}).

$$\Delta P_{ae} = P_{ae} - P_a$$
$$\Delta P_{pe} = P_{pe} - P_p$$

The active and passive forces under static conditions (P_a and P_p) are assumed to act at a point of (0.3 x H) above the base, and the incremental loads induced by earthquake forces (ΔP_{ae} and ΔP_{pe}) are assumed to act near (0.6 x H) above the base, where H is the height of the wall. Therefore, the point of application ha and hp for Pae and P_{pe}, respectively may be calculated from the following equations:

$$h_a = [0.3H.P_a + 0.6H.\Delta P_{ae}]/P_{ae}$$
$$h_p = \left[0.3H.P_p + 0.6H.\Delta P_{pe}\right]/P_{pe}$$

The provided earth pressure coefficients for this Site are base one PGA of 0.398 based on Site Class D and probability of exceedance per annum of 0.000404.

The following soil parameters are presented to assist Designers in designing foundation walls for this Site under seismic conditions using the pseudo-static approach.

TABLE 11: COMBINED STATIC AND SEISMIC LATERAL EARTH PRESSURE COEFFICIENTS

MATERIAL	PARAMETERS				
	ϕ' (°)	c' (kPa)	γ(kN/m³)	K_{ae}	K_{pe}
Granular A	34	0	22	0.87	1.54
Granular B II	32	0	22	0.97	1.53
SSM (Backfill)	30	0	20	1.12	1.51
Native Silty Clay	26	0	18	1.48	1.88
Till	32	--	21	0.97	1.53

Note: K_{ae}: Horizontal component of active earth pressure coefficient including effects of earthquake loading – K_{pe}: Horizontal component of passive earth pressure coefficient including effects of earthquake loading (for a vertical slope and a horizontal surface)

The above noted lateral pressure coefficients are calculated assuming the wall back angel is vertical and the backslope of the retained soil is horizontal.

6.6 PERMANENT DRAINAGE

Consideration should be given to a water suppression system to lessen the effects of long-term dewatering of nearby existing structures.

A water suppression system will lessen the volume of groundwater taking and reduce the zone of influence of the long-term dewatering. The water suppression system can consist of the following:

- A 200 mm thick concrete mud slab placed over the undisturbed excavation bottom to create a horizontal hydraulic barrier.
- A double vertical bentonite membrane fastened to the shoring system extending from the top of the P-2 Level to the concrete mud slab. Ensure that the joints are staggered and taped for watertightness.
- Ensure that the raft slab is poured directly against the vertical bentonite membrane to create a better seal between these two elements.
- Fastened a composite drainage layer against the shoring system from finished grade to the top of the raft slab.
- It's expected that the foundation wall will be blind poured against the shoring system. The composite drainage layer will also behave as a protection layer for the waterproofing layer.
- Water stops should be installed at cold joints in the foundation walls and floor-wall joint.

- Drainage sleeves should be cast in the foundation wall at 3 m centres immediately above the top of the raft slab which will allow water infiltration to be directed to the sump pit via the interior perimeter drain within the granular layer.

6.7 BACKFILL

6.7.1 Engineered Fill

Any over excavation shall be leveled with lean concrete of the same strength as the foundation system. A 200 to 300 mm thick concrete mud slab placed over the undisturbed excavation bottom approved by the Geotechnical Engineer to create a horizontal hydraulic barrier.

All new fill soils that underlie footings, or other structural applications are considered as engineered fill. Engineered fill must meet the strict requirements as shown below:

- Typically, a crushed well-graded material such as an OPSS 1010 Granular A or Granular B Type II is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions;
- Engineered fill shall be placed in maximum loose lifts of 300 mm and adequately compacted to achieve 100% of its SPMDD. Engineered fill must have full-time compaction testing by a geotechnical personnel; and
- At a minimum, the engineered fill beneath foundations should extend laterally a distance of 0.3 m beyond the edge of the footings and then be sloped downward and outward at 1H:1V slope.

6.7.2 Engineered Backfill

The backfill placed against exterior foundations shall be free draining granular material meeting the grading requirements of an OPSS 1010 Granular B Type I, or Selected Subgrade Material (SSM), or equivalent granular material. The existing silty clay is not suitable for backfilling.

The exterior backfill should be placed and compacted as outlined below:

- Backfill should not be placed in frozen condition, or placed on a frozen subgrade;
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 0.3 m. Each lift should be uniformly compacted to achieve 95% of its SPMDD.
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98% of its SPMDD;
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95% of its SPMDD;

- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall;
- Entrance slabs should be founded on frost walls or alternatively have insulation details developed to prevent frost heaving at the building entrances; and
- In areas where the building backfill underlies pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

6.8 UNDERGROUND UTILITIES

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below the ground surface or as dictated by local applicable codes. If this depth is not achievable, equivalent thermal insulation should be provided. The contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in undisturbed silty clay above the groundwater could be classified as Type 3 Soil and sloped no steeper than 1H:1V or be shored. Below the groundwater level, the soils are considered to be Type 4 Soil and the excavation side slopes must be sloped from their bottom cut back at 3H:1V. Otherwise, lateral support for all excavations such as trench boxes should be used.

The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

The recommendations within this section are intended to be a supplement to, and not a replacement of the most recent local municipal requirements.

6.8.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS 1010 "Granular A" material and be placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed. All utility pipes and high amps electrical conduits shall receive a minimum of 150 mm bedding.
- The use of clear stone is not recommended for use as pipe bedding. The voids in the stone may result in a low gradient water flow and infiltration of fines from the surrounding soils and cover materials, causing settlement and loss of support to pipes and structures.
- The cover material should be a service sand material or an OPSS 1010 "Granular A". The dimensions should comply with the pertinent specification section.
- The bedding, spring line, and cover should be compacted to at least 98% of its SPMDD.

- All covers are to be compacted to 100% SPMDD if they are intersecting structural elements.
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.8.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 95% of its SPMDD.
- The backfill placed in the upper 0.3 m below the pavement subgrade elevation should be compacted to a minimum of 98% of its SPMDD.
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe; and
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction are as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

6.9 CEMENT TYPE AND CORROSION POTENTIAL

Two soil samples were submitted to Eurofins Environment Testing Ottawa for testing of chemical properties relevant to exposure of concrete elements to sulphate attacks as well as potential soil corrosivity effects on buried metallic structural elements. Test results are presented in Table 6 and the laboratory results for the chemical analysis are shown in Appendix 3.

Electrical resistivity, pH-value, and chloride concentration can provide an indication of the corrosion potential to buried steel elements in contact with subsurface environment. Using a corrosion nomograph proposed by King (1977) for buried metals and based on electrical resistivity results and pH-value, the corrosion potential for buried steel elements is within the moderately aggressive to aggressive range.

The American Water Works Association (AWWA) publication 'Polyethylene Encasement for Ductile-Iron Pipe Systems' ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. A soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the results obtained for the sample submitted, the Site soils are corrosive to ductile iron pipe. The corrosive effects of road de-icing salts should also be considered.

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) A23.1-04 and are given in Table 12 below.

TABLE 12: ADDITIONAL REQUIREMENT FOR CONCRETE SUBJECTED TO SULPHATE ATTACK

CLASS OF EXPOSURE	DEGREE OF EXPOSURE	WATER SOLUBLE SULPHATE IN SOIL SAMPLE (%)	CEMENTING MATERIAL TO BE USED
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.2 – 2.0	HS or HSb
S-3	Moderate	0.1 – 0.2	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for selected soil samples tested indicate a sulphate concentration of 0.03% to 0.04% in soil, as shown in Table 6, indicating a “moderate to low” risk for sulphate attack on concrete material. Therefore, Type GU Portland cement may be adequate to protect buried concrete elements in the subsurface conditions encountered.

7.0 PARKING LOT PAVEMENT STRUCTURE

The final level of the proposed new parking area was not provided to ABS at the time of this report. All organic matters and topsoil shall be subexcavated down to 0.3 to 0.5 m below the final surface level to accommodate the pavement structure for parking area and access roads.

The pavement structure shall be placed on a properly prepared native subgrade or engineered backfill. Subgrade preparation involves the removal of the organic matters and topsoil to expose native subgrade. The subgrade shall be prepared in accordance with Section 6.3.3 of this report. The final subgrade shall be proof-rolled to look for deflection, soft spots, or local anomalies. Typically, a heavy-duty steel drum roller or a loaded dump truck is sufficient for proof rolling. Proof rolling of proposed subgrades should be witnessed by geotechnical staff. Any non - performing areas should be sub-excavated and replaced with appropriate new borrowed fill materials. Appropriate fill materials would be free-draining non-frost susceptible soil similar to SSM, Granular 'B' Type I or Granular 'B' Type II materials compacted to 95% SPMDD under the direction of the geotechnical engineer. The upper 0.3 m backfill placed below the pavement subbase level should be compacted to a minimum of 98% of its SPMDD.

The pavement structure proposed in this design considers relatively low traffic movement of lightweight passengers to heavy fire trucks. The proposed pavement structure for light-weight vehicles parking area and access road is included in Table 13.

TABLE 13: PAVEMENT STRUCTURE

MATERIAL		THICKNESS (mm)	
		LIGHT DUTY	HEAVY DUTY
Surface	Superpave 12.5 mm, PG 58-34	50	40
Binder	Superpave 19 mm, PG 58-34	--	50
Base	OPSS 1010 Granular A	150	150
Subbase	OPSS 1010 Granular B Type II	300	450

The proposed pavement structures are designed for proof rolled subgrades. The base and subbase materials, i.e., Granular A for base and Granular B Type II for subbase, shall conform to OPSS 1010. Both base and sub-base should be compacted to 100% SPMDD. Asphalt layers should be compacted to comply with OPSS 310. The recommended Superpave 12.5 and 19 can be replaced with HL-3 and HL-8, respectively if required.

8.0 CONSTRUCTION CONSIDERATION

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. All bearing surfaces should be inspected and approved by experienced geotechnical personnel before placing the footings or concrete slabs.

In addition, an adequate level of construction monitoring should include laboratory and field tests during construction. This includes full-time compaction testing of engineered fill and part-time compaction testing of exterior foundation wall backfill with laboratory testing for the proposed fill soils for this Site. Also, periodic testing of concrete is required.

All backfilling shall comply with the OPSS.MUNI 501 unless the design recommendations included in this report exceed provisions of OPSS.MUNI 501.

The vibration should be kept at a minimal level to avoid soil disturbance and associated unexpected settlement to the nearby roadway, load bearing elements, and utilities. Also, the noise level should be kept at a tolerance level of noise per the municipality requirements. Vibration and deformation monitoring will be required throughout the construction.

9.0 SCOPE OF THE REPORT AND LIMITATION OF LIABILITY

The characteristics of the soil and rock described in this report are based on boreholes and exploration trenches conducted at a specific period and depict the nature of the site precisely where these boreholes were carried out. Thus, the characteristics between sampling points can vary significantly from the conditions encountered at the exact location where the samples were taken.

Furthermore, it should be noted that soil and rock formations may differ on the same site, and the boundaries between the various formations presented in this report should not be considered fixed. Groupe ABS Inc. cannot guarantee the accuracy of these boundaries, which depend on factors such as the number of boreholes or the sampling method.

Additionally, the properties of the soil and rock can be significantly altered after construction activities are carried out on the site or on adjacent sites. They can also be indirectly affected by exposure to freezing or weather conditions.

The groundwater conditions presented in this report apply solely to the study site. The groundwater levels indicated correspond only to the levels observed during the specified works, on the specified date and location. It should be noted that these conditions may vary depending on precipitation, snowmelt, or seasons. Moreover, construction activities or modifications to the physical conditions of the study site or adjacent sites can also alter groundwater conditions.

In this report, the descriptions of the sampled materials were conducted using commonly recognized methods of identification and classification in geotechnical engineering. These methods may involve judgment and interpretation. In practice, these descriptions are presumed to be accurate and correct.

The results of tests and analyses are valid only for the samples described in this report. The interpretation of field and laboratory results, as well as the recommendations provided, is applicable only to the study site and the information available for the project at the time of writing this report. They do not apply to any other project or site.

The recommendations given in this report are primarily intended for the project design team. The number of boreholes needed to determine all subsurface conditions may exceed the number of boreholes conducted for design purposes. If the project design is modified, Groupe ABS Inc. should be consulted to ensure that the recommendations in this report are still valid. In the event of modifications to the recommendations, additional field or laboratory work may be necessary.

It is recommended that site visits be conducted by Groupe ABS Inc. as the work progresses to confirm, and if necessary, modify the interpretations or recommendations provided in this report. If such verifications are not possible, Groupe ABS Inc. will assume no responsibility for the geotechnical interpretation that third parties may make of this report, especially if the design is altered or if site conditions differ from those described in this report.

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APPENDIX 1

LOCATION OF BOREHOLES
FIGURE GEO-01

Source : ©2024 Google

Legend

- Study site
- Borehole

Issue date of plan: december 2024

ABS
8-850 Industrial Avenue, Ottawa
Phone: 514 448-2850
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Drawn by:	K. Si Moussa, Drafter
Reviewed by:	M. Al-Khazaali, P.Eng., Ph.D.
Reviewed by:	A. Lamrani, P.Eng., M.Eng.
Client:	Groupe EMD Batimo inc.
Title:	Borehole location plan
Project:	Two Multi-storey Towers
Location:	500, Famille-Côté Avenue, Orléans, Ontario
Scale:	1:1500
Client code:	BATIMO101
F/N:	GO-24-2537-00
Drawing #	1 / 1
Client ref.:	

0 15m 30m 45m 60m 75m

Dernière sauvegarde: 2024-11-25 10:38 | Format : ANSI full bleed B (11.00 x 17.00 pouces)
Chemin: Q:\BATIMO101\242537- 500 Famille-Côté Ave\GO\5 Deliverables\5.1 figures and logs\figures\CAD\EO24253700.dwg

Note: All indications in this drawing are located approximately, according to satellite images and/or chaining. The graphical registers are, for their part, georeferenced with the lot limits. It should be noted that only the surveys recorded by the surveyor are georeferenced. This information will be indicated in the legend.
This drawing should be read in accordance with the report that accompanies it.

APPENDIX 2

BOREHOLE RESULTS
BOREHOLE LOGS AND ROCK CORES

BOREHOLE LOG

Borehole N° BH24-1

Project Name:	**Two Multi-storey Towers**	CRN: **BATIMO101**	F/N: **GO-24-2537-00**
Client:	**Groupe EMD-Batimo inc.**	Geodesic Coordinates (NAD83 SCOPQ SCRS) Zone: 9	X : **381256.5** Y : **5038626.5** Z : **56.95**
Location:	**500, Famille-Côté Ave, Orléans, Ontario**		
Contractor:	**Forage Grenville Drilling**	Plan Number:	**GEO-1**
Type of Borehole:	**Auger**		
Borehole Diameter:	**203 mm**	Core Diameter:	Date of Borehole: **2024-11-14**
Field Technician:	**J. Brooks, tech.**	Prepared by: **K. Si Moussa, Drafter**	Depth of Borehole (m) : **27.43**

SAMPLE STATE
- Disturbed
- Intact (thin wall sampler)
- Lost
- Diamond core

TERMINOLOGY

"traces"	1-10 %
"some"	10-20 %
adjective (sandy...)	20-35 %
"and"	35-50 %

CLASSIFICATION

Clay	< 0,002 mm
Silt	0.002 to 0.08 mm
Sand	0.08 to 5 mm
Gravel	5 to 80 mm
Cobbles	80 to 300 mm
Blocks	> 300 mm

COMPACTION INDEX "N"

Very Loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

CONSISTENCY "Cu" (kPa)

Very Soft	< 12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	> 200

ROCK QUALITY DESIGNATION

QUALIFIER	% RQD
Very Poor	<25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

VISUAL CONTAMINATION (hydrocarbons)
- A : Absent
- D : Disseminated
- P : Pervasive

DEGREE OF PLASTICITY

	"W_L"	S_t = Cu/Cu_r
Low	< 30 %	< 2
Medium	30 - 50 %	2 - 4
High	> 50 %	4 - 8
Very high		8 - 16
Sensitive		> 16

CALIBER
- P : 148 mm
- N : 64 mm
- B : 51 mm

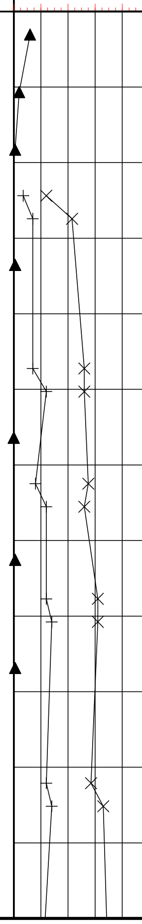
WATER LEVEL
- Water Infiltration
- Groundwater table

Date: **2024-11-19**
Depth(m) : **1.06**

STRATIGRAPHY / SAMPLES

Elevation (m) / Depth (m)	Stratigraphic Description	Type No	Caliber	State	Recovery	N, R or RQD	Blow Counts /15 cm	Lab Tests
56.95	Level							
56.80 / 0.00–0.15	Topsoil : Silty Clay, traces of organics, grey, damp.	SS-1	B	Disturbed	75	12	1-5-7-8	Su = 390 kPa
55.58 / 0.15–1.37	Native : Silty Clay, grey, damp, stiff (Weathered Crust).	SS-2	B	Disturbed	92	4	2-2-2-4	
1.37	Silty Clay, grey, damp to wet, soft to very soft (unweathered).	SS-3	B	Disturbed	100	1	1-1-1-1	Su =145 kPa
		FVS1						Cu=24kPa
		FVS2						Cur=7kPa
	Becomes soft.							Cu=43kPa
		SS-4	B	Disturbed	100	1	1-0-1-0	Cur=14kPa
		ST-5	B	Intact				
		FVS3						Cu=52kPa
		FVS4						Cur=14kPa
								Cu=52kPa
		SS-6	B	Disturbed	100	0	0-0-0-1	Cur=24kPa
		FVS5						Cu=55kPa
		FVS6						Cur=16kPa
								Cu=52kPa
		SS-7	B	Disturbed	100	1	0-0-1-1	Cur=24kPa
		FVS7						Cu=62kPa
		FVS8						Cur=24kPa
								Cu=62kPa
		SS-8	B	Disturbed	100	1	0-0-1-0	Cur=28kPa
		FVS9						Cu=57kPa
		FVS10						Cur=24kPa
								Cu=66kPa
		ST-9	B	Intact				Cur=28kPa

Water level: 52.38 m



LAB TESTS
- G : grain size analysis
- H : hydrometer test
- CA : chemical analysis
- LI : liquid limit
- PI : plastic limit
- w : water content
- Qu : rock comp. strength
- Cur : disturbed shear strength
- Cu : undrained shear strength
- Cc : consolidation coefficient
- k : permeability
- Dup : duplicate

Remark(s) : Su : Estimated undrained shear strength from pocket penetrometer.
GWL at 1.06 mbgs on November 19, 2024 in open borehole.
GWL at 4.57 mbgs on November 19, 2024 in casing.

Verified by: **M. Al-Khazaali, Ph.D., P.Eng.** Approved by: **M. Al-Khazaali, Ph.D., P.Eng.** Date of report 2024-12-12

ABS

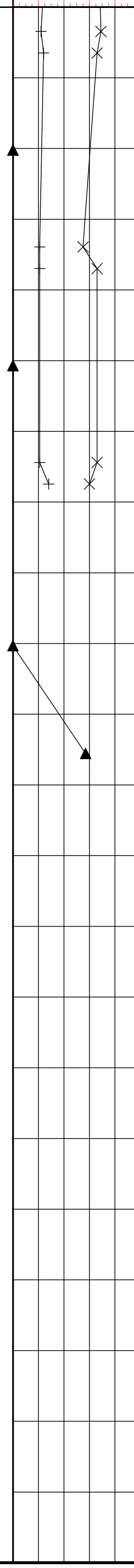
BOREHOLE LOG

Borehole N°
BH24-1

DEPTH (m)	DEPTH - ft	ELEVATION (m)	STRATIGRAPHIC DESCRIPTION	TYPE NO	SUB-SAMPLE	CALIBER	RECOVERY	N, R or RQD	BLOW COUNTS /15 cm Fragmentation (mm)	VOC (ppm)	VISUAL CONTAMINATION (hydrocarbons) A / D / P	WATER LEVELS	LAB TESTS
				FVS11 FVS12									Cu=69kPa Cur=22kPa Cu=66kPa Cur=24kPa
14				SS-10		B	100	0	0-0-0-2				
				FVS13 FVS14									Cu=55kPa Cur=21kPa Cu=66kPa Cur=21kPa
17				SS-11		B	100	0	0-0-0-0				
				FVS15 FVS16		B B							Cu=66kPa Cur=21kPa Cu=60kPa Cur=28kPa
21				SS-12		B	100	0	1-0-0-0				
		34.70 22.25	Till : Sand and Gravel, with cobbles and boulders, grey, moist to damp.	SS-13		B	67	57	30-37-17-19				
		33.96 22.99	Bedrock : Limestone interbedded with shale, grey to dark grey, Weathered, fractured, poor to fair quality based on RQD. Sound bedrock.	RC-14		B	89	*49*					
				RC-15		B	95	*66*					
				RC-16		B	99	*83*					
		29.52 27.43	END OF THE BOREHOLE										

Depth scale (m): 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33
Depth scale (ft): 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110

GRAPHIC



LAB TESTS

G : grain size analysis
H : hydrometer test
CA : chemical analysis
LI : liquid limit
PI : plastic limit
w : water content
Qu : rock comp. strength
Cur : disturbed shear strength
Cu : undrained shear strength
Cc : consolidation coefficient
k : permeability
Dup: duplicate

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BOREHOLE LOG

Borehole N° BH24-2

Project Name:	Two Multi-storey Towers	CRN: BATIMO101	F/N: GO-24-2537-00
Client:	Groupe EMD-Batimo inc.	Geodesic Coordinates (NAD83 SCOPQ SCRS) Zone: 9	X : 381258.4
Location:	500, Famille-Côté Ave, Orléans, Ontario		Y : 5038569.0
Contractor:	Forage Grenville Drilling		Z : 57.30
Type of Borehole:	Auger	Plan Number:	GEO-1
Borehole Diameter:	203 mm	Core Diameter:	Date of Borehole: 2024-11-14
Field Technician:	J. Brooks, tech.	Prepared by: K. Si Moussa, Drafter	Depth of Borehole (m) : 27.43

SAMPLE STATE

- Disturbed
- Intact (thin wall sampler)
- Lost
- Diamond core

TERMINOLOGY

"traces"	1-10 %
"some"	10-20 %
adjective (sandy...)	20-35 %
"and"	35-50 %

CLASSIFICATION

Clay	< 0,002 mm
Silt	0.002 to 0.08 mm
Sand	0.08 to 5 mm
Gravel	5 to 80 mm
Cobbles	80 to 300 mm
Blocks	> 300 mm

COMPACTION INDEX "N"

Very Loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

CONSISTENCY "Cu" (kPa)

Very Soft	< 12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	> 200

ROCK QUALITY DESIGNATION

QUALIFIER	% RQD
Very Poor	<25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

VISUAL CONTAMINATION (hydrocarbons)

- A : Absent
- D : Disseminated
- P : Pervasive

DEGREE OF PLASTICITY

	"W_L"	S_t = Cu/Cu$_r$
Low	< 30 %	< 2
Medium	30 - 50 %	2 - 4
High	> 50 %	4 - 8
Very high		8 - 16
Sensitive		> 16

CALIBER

- P : 148 mm
- N : 64 mm
- B : 51 mm

WATER LEVEL

Water Infiltration

Groundwater table

Date: 2024-10-19

Depth(m) : 2.74

LAB TESTS

G : grain size analysis
H : hydrometer test
CA : chemical analysis
LI : liquid limit
PI : plastic limit
w : water content
Qu : rock comp. strength
Cur : disturbed shear strength
Cu : undrained shear strength
Cc : consolidation coefficient
k : permeability
Dup : duplicate

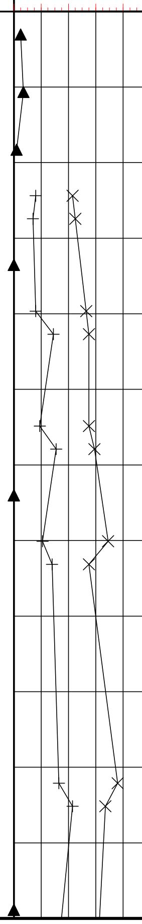
GRAPHIC: ▲ : N (standard pen.); △ : Nc (dynamic pen.); ▼ : Cu (laboratory); ▽ : Cur (laboratory); ✕ : Cu (field); + : Cur (field); W_p W_n W_l; 20 40 60 80

STRATIGRAPHY / SAMPLES

ELEVATION (m)	STRATIGRAPHIC DESCRIPTION
57.30	Level
0.00 / 57.15 / 0.15	Topsoil : Silty Clay, trace organics, greyish brown, damp, firm
	Native : Silty Clay, grey, moist, stiff (Weathered Crust)
55.78 / 1.52	Silty Clay, grey, moist, soft (unweathered)
	Becomes wet.

TYPE NO	CALIBER	RECOVERY	N, R or RQD	BLOW COUNTS /15 cm	LAB TESTS
SS-1	B	83	5	0-1-4-7	Su = 170 kPa
SS-2	B	100	7	2-3-4-4	Su = 190 kPa
SS-3	B	100	2	1-1-1-1	Su = 50 kPa
FVS1					Cu=43kPa Cur=16kPa
FVS2					Cu=45kPa Cur=14kPa
SS-4	B	100	0	0-0-0-1	
FVS3					Cu=53kPa Cur=16kPa
FVS4					Cu=55kPa Cur=29kPa
ST-5	B				
FVS5					Cu=55kPa Cur=19kPa
FVS6					Cu=59kPa Cur=31kPa
SS-6			0	0-0-0-1	
FVS7					Cu=69kPa Cur=21kPa
FVS8					Cu=55kPa Cur=28kPa
SS-7	B			0-0-1-1	
FVS9					Cu=76kPa Cur=33kPa
FVS10					Cu=67kPa Cur=43kPa
SS-8	B	100	0	0-0-0-1	

WATER LEVELS: 54.56 m



Remark(s) : Su : Estimated undrained shear strength from pocket penetrometer.
GWL at 2.74 mbgs on November 19, 2024 in open borehole.
GWL at 3.01 mbgs on November 26, 2024 in the monitoring well.

Verified by: M. Al-Khazaali, Ph.D., P.Eng. Approved by: M. Al-Khazaali, Ph.D., P.Eng. Date of report 2024-12-12

ABS

BOREHOLE LOG

Borehole N° BH24-2

DEPTH (m)	DEPTH - ft	ELEVATION (m)	STRATIGRAPHIC DESCRIPTION	TYPE NO	SUB-SAMPLE	CALIBER	RECOVERY	N, R or RQD	BLOW COUNTS /15 cm Fragmentation (mm)	LAB TESTS
				SS-8		B	100	0	0-0-0-1	
13–14	45			FVS11 FVS12						Cu=59kPa Cur=28kPa Cu=72kPa Cur=43kPa
15	50			SS-9		B	100	1	0-0-1-1	
16–17	55			FVS13 FVS14						Cu=62kPa Cur=21kPa Cu=55kPa Cur=24kPa
18	60			SS-10		B		0	0-0-0-1	
21	70	35.96 21.34	Till : Sand and gravel, some clay with cobbles and boulders, grey.	FVS15 FVS16						Cu=69kPa Cur=26kPa Cu=64kPa Cur=33kPa
24.54	80	32.76 24.54	Crushed rock.	SS-11		B			12 > 50	
25.25		32.05 25.25	Bedrock : Limestone interbedded with shale, grey to dark, unweathered, fractured, poor to fair quality based on RQD.	RC-12		B	82	56		
25.98	85	31.32 25.98		RC-13		B	66	20		
27.43	90	29.87 27.43	Intensely fractured. END OF THE BOREHOLE							

DEPTH (m): 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33

DEPTH - ft: 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110

VISUAL CONTAMINATION (hydrocarbons): A D P

VOC (ppm)

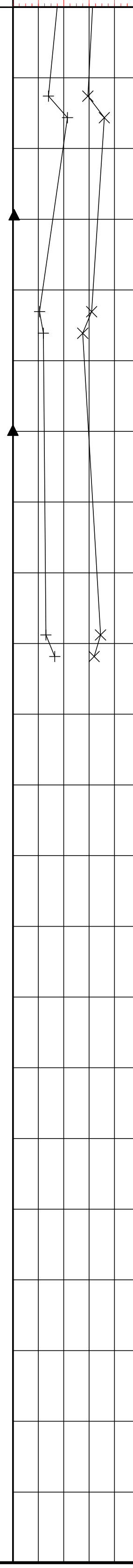
WATER LEVELS

GRAPHIC

▲ : N (standard pen.)
△ : Nc (dynamic pen.)
▼ : Cu (laboratory)
▽ : Cur (laboratory)
× : Cu (field)
+ : Cur (field)

W_p W_n W_l

20 40 60 80



LAB TESTS

G : grain size analysis
H : hydrometer test
CA : chemical analysis
LI : liquid limit
PI : plastic limit
w : water content
Qu : rock comp. strength
Cur : disturbed shear strength
Cu : undrained shear strength
Cc : consolidation coefficient
k : permeability
Dup : duplicate

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BOREHOLE LOG

ABS

Borehole N° BH24-3

Project Name:	**Two Multi-storey Towers**	CRN: **BATIMO101**	F/N: **GO-24-2537-00**
Client:	**Groupe EMD-Batimo inc.**	Geodesic Coordinates (NAD83 SCOPQ SCRS) Zone: 9	X: **381237.4** Y: **5038583.6** Z: **56.99**
Location:	**500, Famille-Côté Ave, Orléans, Ontario**		
Contractor:	**Forage Grenville Drilling**	Plan Number:	**GEO-1**
Type of Borehole:	**Auger**		
Borehole Diameter:	**203 mm**	Core Diameter:	
		Date of Borehole:	**2024-11-14**
Field Technician:	**J. Brooks, tech.**	Prepared by: **K. Si Moussa, Drafter**	Depth of Borehole (m): **24.50**

SAMPLE STATE
- Disturbed
- Intact (thin wall sampler)
- Lost
- Diamond core

TERMINOLOGY

"traces"	1-10 %
"some"	10-20 %
adjective (sandy...)	20-35 %
"and"	35-50 %

CLASSIFICATION

Clay	< 0,002 mm
Silt	0.002 to 0.08 mm
Sand	0.08 to 5 mm
Gravel	5 to 80 mm
Cobbles	80 to 300 mm
Blocks	> 300 mm

COMPACTION INDEX "N"

Very Loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

CONSISTENCY "Cu" (kPa)

Very Soft	< 12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	> 200

ROCK QUALITY DESIGNATION

QUALIFIER	% RQD
Very Poor	<25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

VISUAL CONTAMINATION (hydrocarbons)
- A : Absent
- D : Disseminated
- P : Pervasive

DEGREE OF PLASTICITY

	"W_L"	S_t = Cu/Cu$_r$
Low	< 30 %	< 2
Medium	30 - 50 %	2 - 4
High	> 50 %	4 - 8
Very high		8 - 16
Sensitive		> 16

CALIBER
- P : 148 mm
- N : 64 mm
- B : 51 mm

WATER LEVEL
- Water Infiltration
- Groundwater table

Date:
Depth(m) :

GRAPHIC legend: ▲ : N (standard pen.); △ : Nc (dynamic pen.); ▼ : Cu (laboratory); ▽ : Cur (laboratory); ✕ : Cu (field); + : Cur (field); W_p, W_n, W_l; scale 20 40 60 80

LAB TESTS legend: G : grain size analysis; H : hydrometer test; CA : chemical analysis; LI : liquid limit; PI : plastic limit; w : water content; Qu : rock comp. strength; Cur : disturbed shear strength; Cu : undrained shear strength; Cc : consolidation coefficient; k : permeability; Dup : duplicate

Elevation / Depth (m)	Stratigraphic Description	Type No	Sub-sample	Caliber	State	Recovery	N, R or RQD	Blow counts /15 cm	Lab tests
56.99	Level								
0.00 / 56.69 / 0.30	Topsoil : Silty Clay, trace organic, brownish grey, damp, firm.	SS-1		B	Disturbed	75	5	0-2-3-4	Su = 125 kPa
	Native : Silty Clay, grey, damp to moist, firm to soft (Weathered Crust)	SS-2		B	Disturbed	83	7	1-3-4-4	Su = 175 kPa
54.86 / 2.13		SS-3		B	Disturbed	100	3	1-1-2-2	Su = 80 kPa
	Silty Clay, grey, wet, very soft (unweathered)	SS-4		B	Disturbed	100	0	0-0-0-1	
		SS-5		B	Disturbed	100	0	0-0-0-1	
		SS-6		B	Disturbed	100	0	0-0-0-1	
		ST-7		B	Intact				
		SS-8		B	Disturbed	100	0	0-0-0-1	
		SS-9		B	Disturbed	100	0	0-0-0-0	
		SS-10		B	Disturbed	100	0	0-0-0-1	
		SS-11		B	Disturbed	100	0	0-0-0-0	
		SS-12		B	Disturbed	100	0	0-0-0-0	
		SS-13		B	Disturbed	100	0	0-0-0-0	
47.24 / 9.75	Inferred silty Clay.						2, 2, 3, 3, 3, 4, 4		

Depth scale: 1–11 m; 5–35 ft.

Remark(s) : Su : Estimated undrained shear strength from pocket penetrometer.

Verified by: **M. Al-Khazaali, Ph.D., P.Eng.** Approved by: **M. Al-Khazaali, Ph.D., P.Eng.** Date of report 2024-12-12

ABS

BOREHOLE LOG

Borehole N°
BH24-3

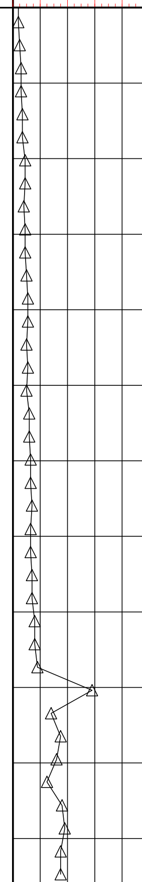
DEPTH (m)	DEPTH - ft	ELEVATION (m)	STRATIGRAPHIC DESCRIPTION	SYMBOL	TYPE NO	SUB-SAMPLE	CALIBER	STATE	RECOVERY	N, R or RQD	BLOW COUNTS /15 cm Fragmentation (mm)	VOC (ppm)	VISUAL CONTAMINATION (hydrocarbons) A	D	P	WATER LEVELS	LAB TESTS
				▨						4							
				▨						4							
				▨						5							
13				▨						6							
				▨						6							
	45			▨						7							
14				▨						7							
				▨						9							
				▨						9							
15				▨						8							
	50			▨						9							
				▨						9							
				▨						10							
16				▨						11							
				▨						11							
	55			▨						10							
17				▨						11							
				▨						10							
				▨						12							
18				▨						12							
	60			▨						13							
				▨						13							
19				▨						14							
				▨						13							
				▨						13							
	65			▨						14							
20				▨						14							
		36.57 / 20.42	Inferred Till.	▨						16							
										16							
21										18							
	70									58							
										28							
22										35							
										32							
										25							
23	75									36							
										38							
										35							
										35							
24		33.22 / 23.77	END OF BOREHOLE (DCPT refusal).														
	80																
25																	
26	85																
27																	
	90																
28																	
29	95																
30																	
	100																
31																	
32	105																
33																	
	110																

GRAPHIC

▲ : N (standard pen.)
△ : Nc (dynamic pen.)
▼ : Cu (laboratory)
▽ : Cur (laboratory)
× : Cu (field)
+ : Cur (field)

W_p W_n W_l

20 40 60 80



LAB TESTS

G : grain size analysis
H : hydrometer test
CA : chemical analysis
LI : liquid limit
Pl : plastic limit
w : water content
Qu : rock comp. strength
Cur : disturbed shear strength
Cu : undrained shear strength
Cc : consolidation coefficient
k : permeability
Dup : duplicate

Retrieved Rock Cores

Client:	**Groupe EMD-Batimo Inc.**	Project No.: **GO-24-2537-00**	
Project:	**Proposed Multi-Storey Mixed-Use Development at 500 Famille-Cote Ave, Ottawa, ON**	Client No.: **BATIMO101**	
Site:	**500 Famille-Cote Ave, Ottawa, ON**	Contractor: **Forage Grenville Drilling**	
Field Tech:	**Jim Brooks**		

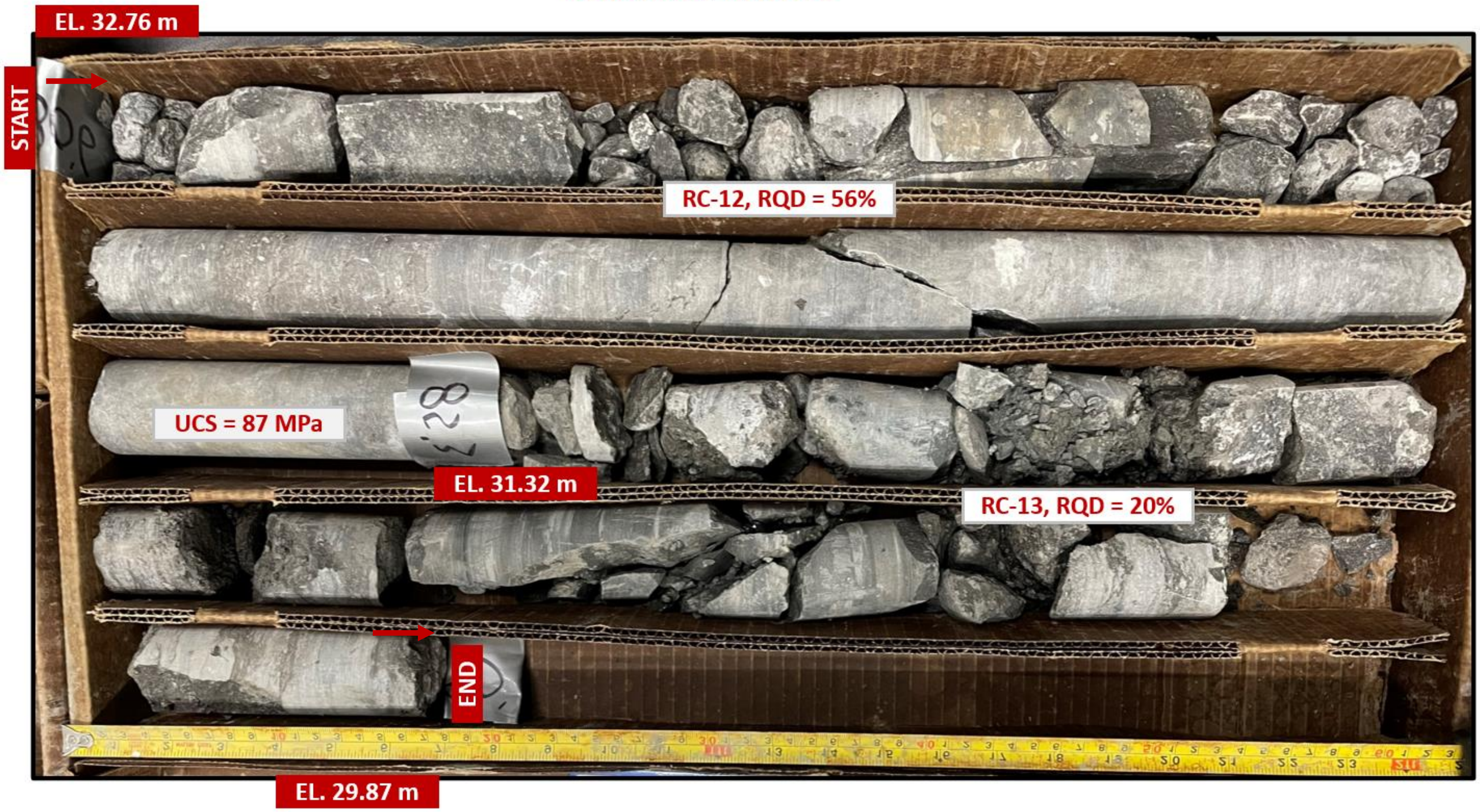
Borehole: BH24-1

Prepared by: Alpesh R. Senghani | Verified by: Mohammed Al-Khazaali | Date: 2024-12-16

Retrieved Rock Cores

Client:	**Groupe EMD-Batimo Inc.**	Project No.: **GO-24-2537-00**	
Project:	**Proposed Multi-Storey Mixed-Use Development at 500 Famille-Cote Ave, Ottawa, ON**	Client No.: **BATIMO101**	
Site:	**500 Famille-Cote Ave, Ottawa, ON**	Contractor: **Forage Grenville Drilling**	
Field Tech:	**Jim Brooks**		

Borehole: BH24-2



Prepared by: Alpesh R. Senghani | Verified by: Mohammed Al-Khazaali | Date: 2024-12-16

APPENDIX 3

GEOTECHNICAL LABORATORY TESTS
TEST REPORTS

GRANULOMETRIC ANALYSIS AND SEDIMENTOMETRY

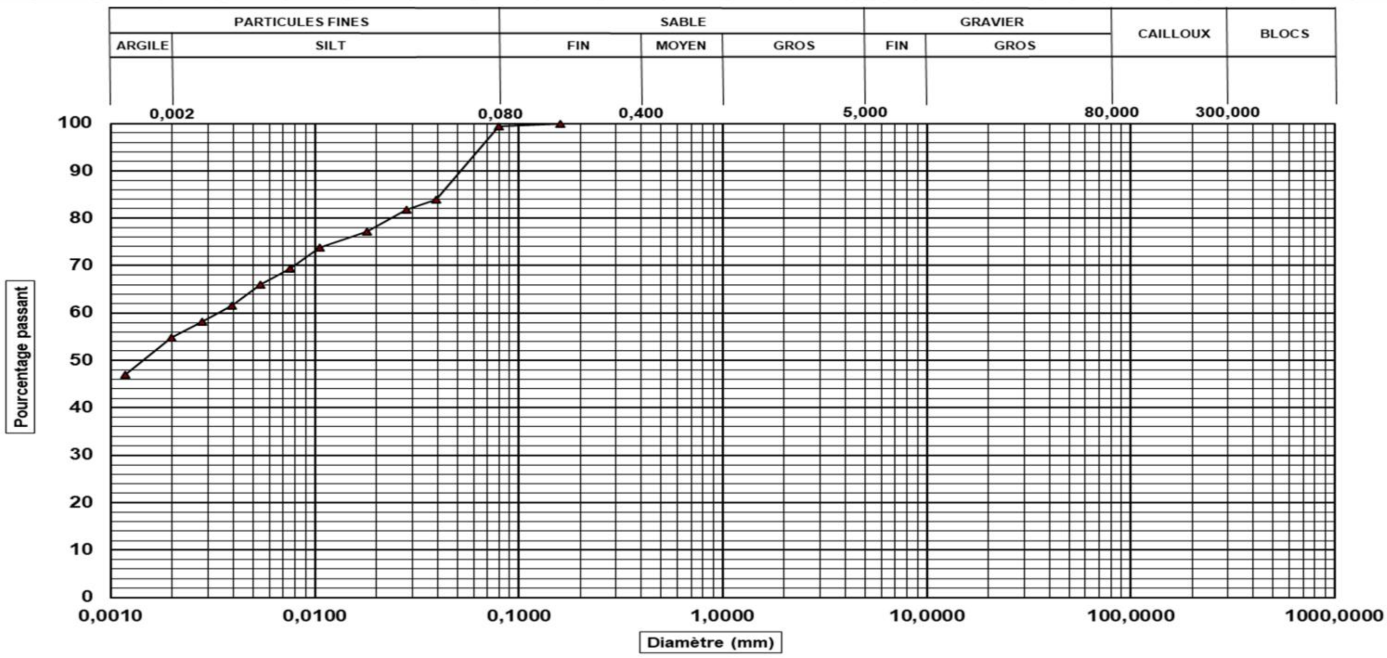
LC 21-040 & BNQ 2501-025

Project :	500 Family-Side Ave,	Customer :	Batimo Development Inc.
Project # :	GO24253700	Client #:	BATIMO101
V/D:		Date :	2024-11-26

Sampling

Material Type:	Natural soil	Lab No.:	344 246
Collected by:	Jim Brooks	Taken on:	2024-11-26
Location of the sample:	BH24-1 Shelby Tube (ST9)	Caliber:	Natural soil
		Depth:	10.70 - 11.30 m

Granulometric curve



PERCENTAGES OF GRANULOMETRIC FRACTIONS					
Fine Fraction		**Coarse Fraction**			
Clay (%)	**Silt (%)**	**Sand (%)**	**Gravel (%)**	**Pebbles (%)**	**Blocks (%)**
54.7	44.7	0.6	0.0	0.0	0.0

D10 (mm)	D30 (mm)	D60 (mm)	Cu	Cc	Wn (%)
N / A	N / A	0.003	N / A	N / A	N / A

TABLE OF PASSING PERCENTAGES

(mm)	(%)	(mm)	(%)	(mm)	(%)
80.00	100	2.500	100	0.01807	77.2
56.00	100	1.250	100	0.01057	73.9
40.00	100	0.630	100	0.00760	69.4
31.50	100	0.315	100	0.00543	66.0
20.00	100	0.160	100	0.00390	61.6
14.00	100	0.080	99.4	0.00279	58.2
10.00	100	0.03935	84.0	0.00199	54.9
5.00	100	0.02807	81.7	0.00118	47.0

SAMPLE DESCRIPTION

Drs	2,700 (Estimated)
Sample	BH24-1 ST-9
Depth	10.76 - 10.86 m
Nomenclature	
Clay and silt, traces of sand.	

Remarks

Prepared by: Sofiane Azeggagh
Sofiane Azeggagh

Verified by:
Mohammed Al-Khazaali

Date: 2024-12-17

N/Ref.: Comes from 1C

GRANULOMETRIC ANALYSIS AND SEDIMENTOMETRY

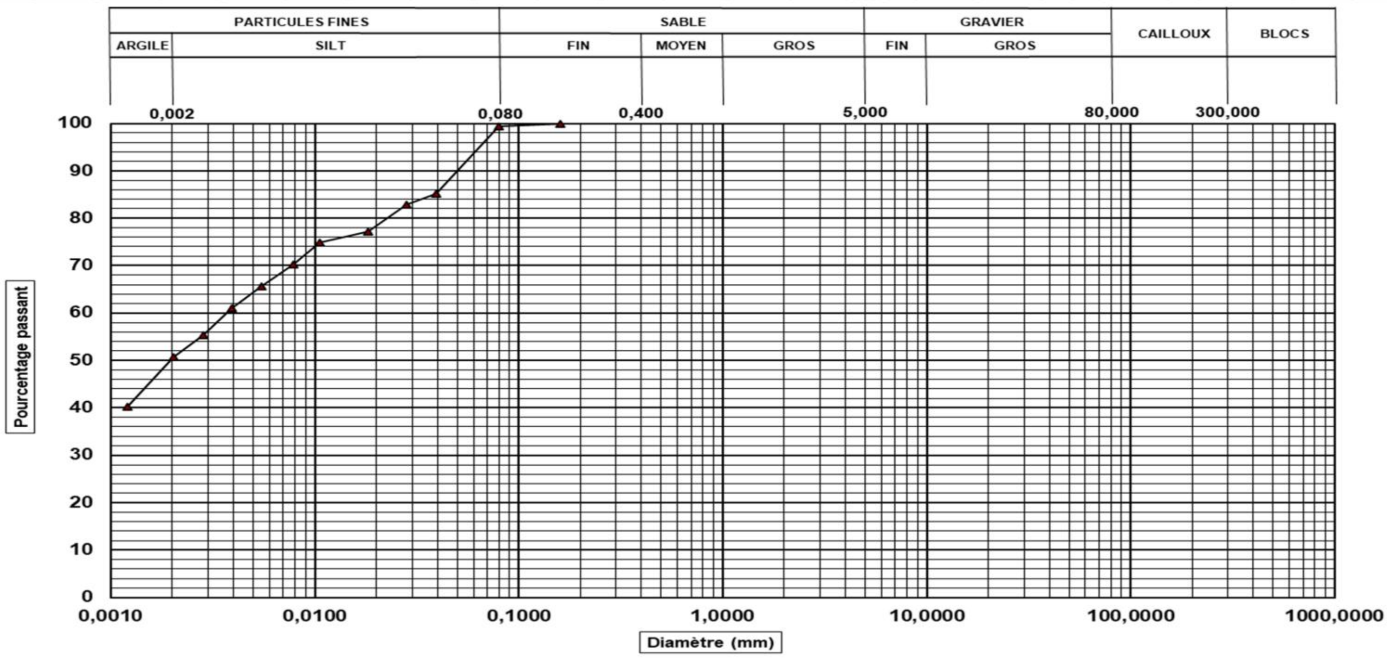
LC 21-040 & BNQ 2501-025

Project :	500 Family-Side Ave,	Customer :	Batimo Development Inc.
Project # :	GO24253700	Client #:	BATIMO101
V/D:		Date :	2024-11-26

Sampling

Material Type:	Natural soil	Lab No.:	344 248
Collected by:	Jim Brooks	Taken on:	2024-11-26
Location of the sample:	BH24-2 Shelby Tube (ST5)	Caliber:	Natural soil
		Depth:	4.60 - 5.20 m

Granulometric curve



PERCENTAGES OF GRANULOMETRIC FRACTIONS					
Fine Fraction		Coarse Fraction			
Clay (%)	Silt (%)	Sand (%)	Gravel (%)	Pebbles (%)	Blocks (%)
50.4	49.0	0.6	0.0	0.0	0.0

D10 (mm)	D30 (mm)	D60 (mm)	Cu	Cc	Wn (%)
N / A	N / A	0.004	N / A	N / A	N / A

TABLE OF PASSING PERCENTAGES

(mm)	(%)	(mm)	(%)	(mm)	(%)
80.00	100	2.500	100	0.01821	77.2
56.00	100	1.250	100	0.01062	74.9
40.00	100	0.630	100	0.00781	70.3
31.50	100	0.315	100	0.00548	65.7
20.00	100	0.160	100	0.00393	61.0
14.00	100	0.080	99.4	0.00283	55.3
10.00	100	0.03948	85.2	0.00203	50.7
5.00	100	0.02817	82.9	0.00121	40.3

SAMPLE DESCRIPTION

Drs	2,700 (Estimated)
Sample	BH24-2 ST5
Depth	5.06 - 5.12 m
Nomenclature	
Clay and silt, traces of sand.	

Remarks

Prepared by: Sofiane Azeggagh
Sofiane Azeggagh

Verified by:
Mohammed Al-Khazaali

Date: 2024-12-17

N/Ref.: Comes from 1C

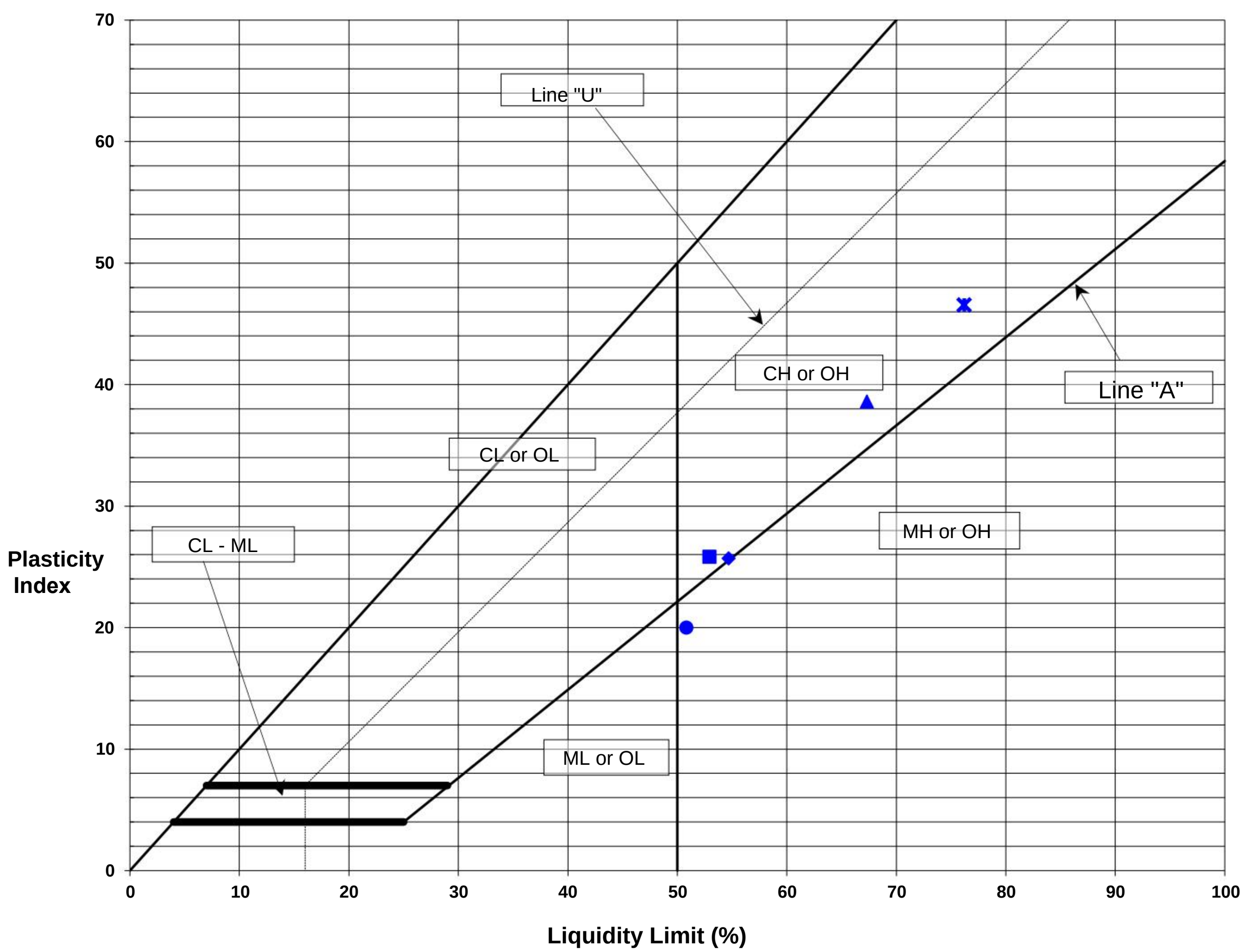
PLASTICITY CHART FOR SOILS

Client: Batimo Development Inc.

N/A: GO24253700

PROJECT :

CLIENT NUMBER: BATIMO101



RESULTS "BNQ 2501-092"									
Legend	Borehole	Sample No.	Depth (m)	Wn	LL	PL	PI	LI	Classification
▲	**BH24-1**	**SS8**	**8.08 - 8.99**	**65.4%**	**67%**	**29%**	**39%**	**1.0**	**CH or OH**
◆	**BH24-1**	**SS10**	**13.70 - 14.30**	**54.9%**	**55%**	**29%**	**26%**	**1.0**	**CH or OH**
●	**BH24-1**	**SS11**	**16.80 - 17.40**	**51.6%**	**51%**	**31%**	**20%**	**1.0**	**MH or OH**
■	**BH24-1**	**SS12**	**20.70 - 21.30**	**51.9%**	**53%**	**27%**	**26%**	**1.0**	**CH or OH**
*	**BH24-2**	**SS6**	**6.10 - 6.70**	**70.8%**	**76%**	**30%**	**47%**	**0.9**	**CH or OH**

Remarks

Prepared by : Sofiane Azeggagh *Sofiane Azeggagh*

Date : 2024-12-11

N/Ref.: Comes from 1C

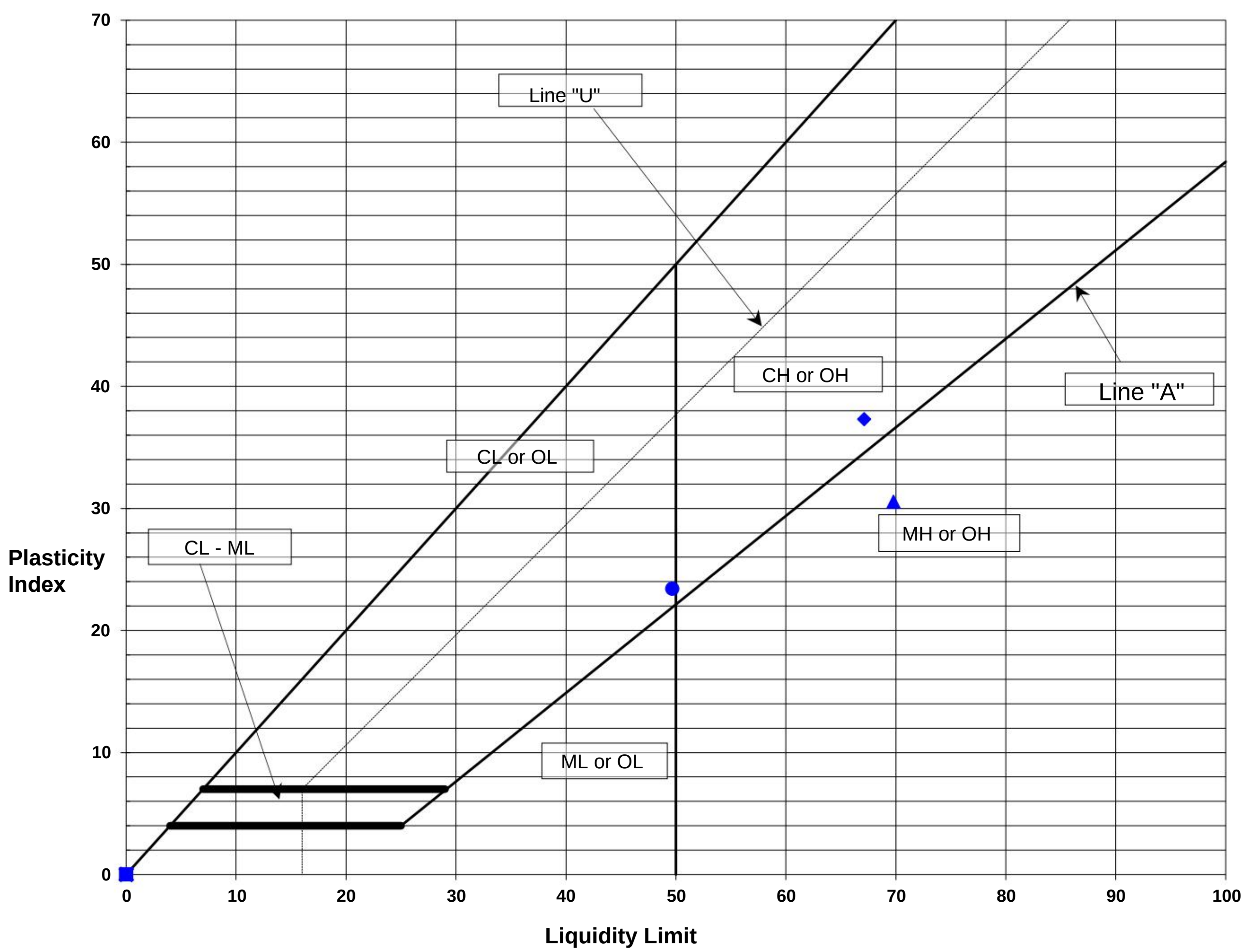
PLASTICITY CHART FOR SOILS

CLIENT : Batimo Development Inc.

N/A: GO24253700

PROJECT :

CLIENT NUMBER: BATIMO101



RESULTS "BNQ 2501-092"									
Legend	Borehole	Sample No.	Depth (m)	Wn	LL	PL	PI	LI	Classification
▲	**BH24-2**	**SS7**	**8.50 - 9.00**	**65.8%**	**70%**	**39%**	**31%**	**0.9**	**MH or OH**
◆	**BH24-2**	**SS8**	**11.60 - 12.20**	**61.9%**	**67%**	**30%**	**37%**	**0.9**	**CH or OH**
●	**BH24-2**	**SS10**	**17.70 - 18.30**	**51.4%**	**50%**	**26%**	**23%**	**1.1**	**CL or OL**
■									
*									

Remarks

Prepared by : Sofiane Azeggagh *Sofiane Azeggagh*

Date : 2024-12-11

N/Ref.: Comes from 1C

Description and extraction of a thin-walled tube sample

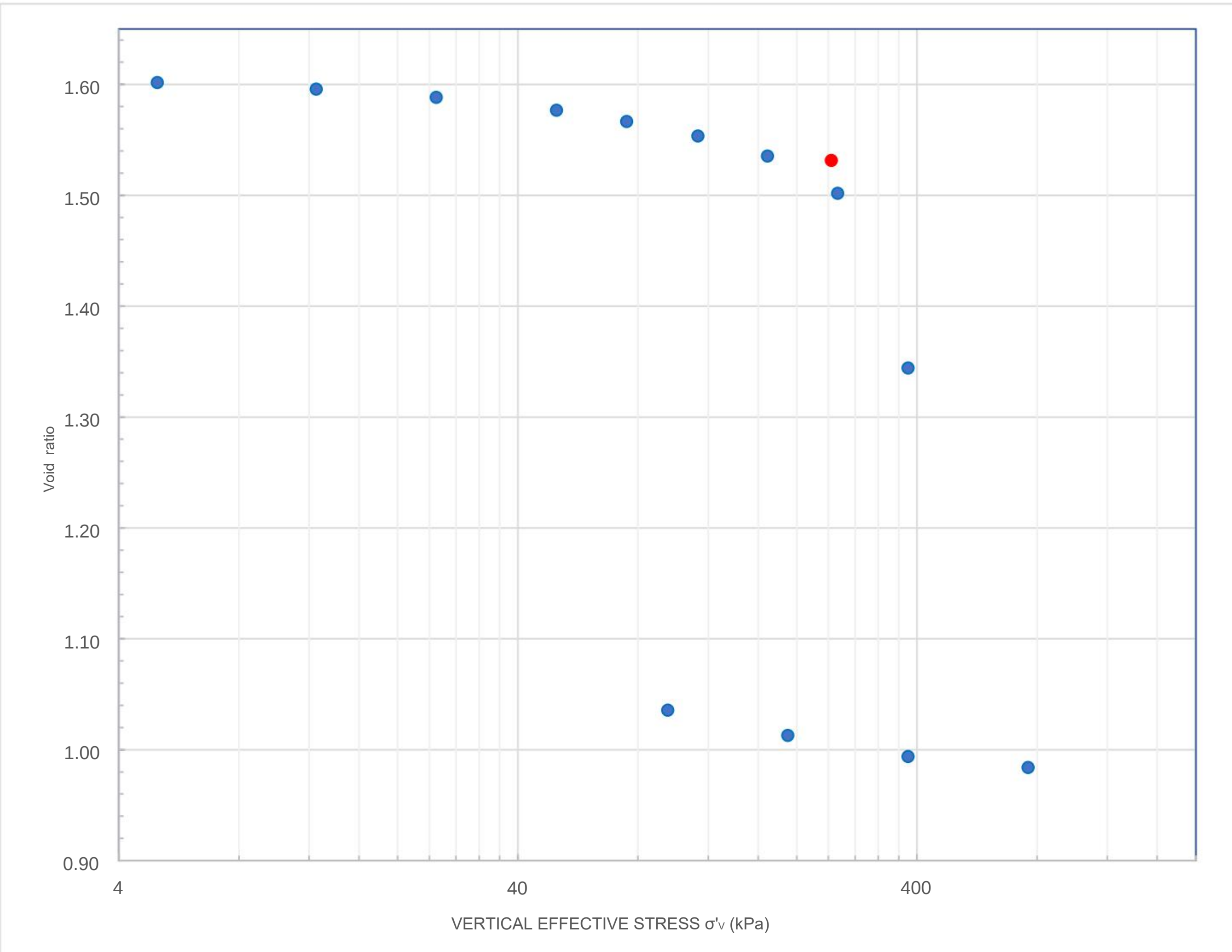
LC 22-300 Standard

Project:	500 Family-Side Ave,	Project # :	GO24253700
Client	Batimo Development Inc.	Customer #:	BATIMO100
Extracted on:	04-12-2024	Borehole:	BH24-1
Extracted by:	Manon	Sample #:	ST9
No lab. :	344 246	Depth :	10.70 - 11.30 m

	Length(m)	Depth (m)	Visual description	Water content	Atterberg Limits	Consolidation	Shear strenght at swedish cone	Others	Parts preserved	Depth (m)
TOP		10.7	Packed in two plastic bags held in place with black adhesive tape.							10.7
Rejection			length: 55cm					X		10.76
Hydrometer		10.8	Silty clay with traces of sand.					X		10.86
Gs		10.9	Homogeneous. Loss of 3 cm at the bottom of the tube due to Labvane test.					X		10.95
Part 2		11	Crack present at extraction at 10.84m (view photo)						X	11.05
Conso		11.1	Surface holes present at 10.84m, 10.93m, 11.00m, 11.07m. (View photo) Strong smell of machine oil			X				11.15
Part 3		11.2	Intact Consistency: Firm Disturbed consistency: Soft						X	11.22
Rej/Labv			Munsell color: GLEY1 4/ 4/1 10Y					X		11.25
		11.3								
BOTTOM										

CONSOLIDATION TEST LC 22-301 LC 22-301

CLIENT :	Batimo Development Inc.	**N/A:**	GO24253700
PROJECT :	BATIMO101	**CLIENT NUMBER :**	BATIMO101
SITE :	500 Family-Side Ave,	LAB **No .:**	344246
Borehole/sample:	BH24-1 Shelby ST9	**Depth:**	11.05 - 11.15
COLLECTED BY:	Jim Brooks	**TEST DATE :**	13-Dec-24



Geotechnical characteristics

Initial void index (e_0):	1.60	Recompression index (c_r):	0.06
Water content (w_0) (%):	61.8	Virgin Compression Index (c_c):	1.09
Wet density (kN/m3):	16.9	Preconsolidation stress (σ_{pc}) (kPa):	**244**
Initial saturation degree (Sr) (%):	100	Initial effective stress (σ_v) (kPa):	**89**

Prepared by : Carolane Croteau **Verified by:** Nadezda Zharkova, CPI **Date :** 2024-12-23

Description and extraction of a thin-walled tube sample

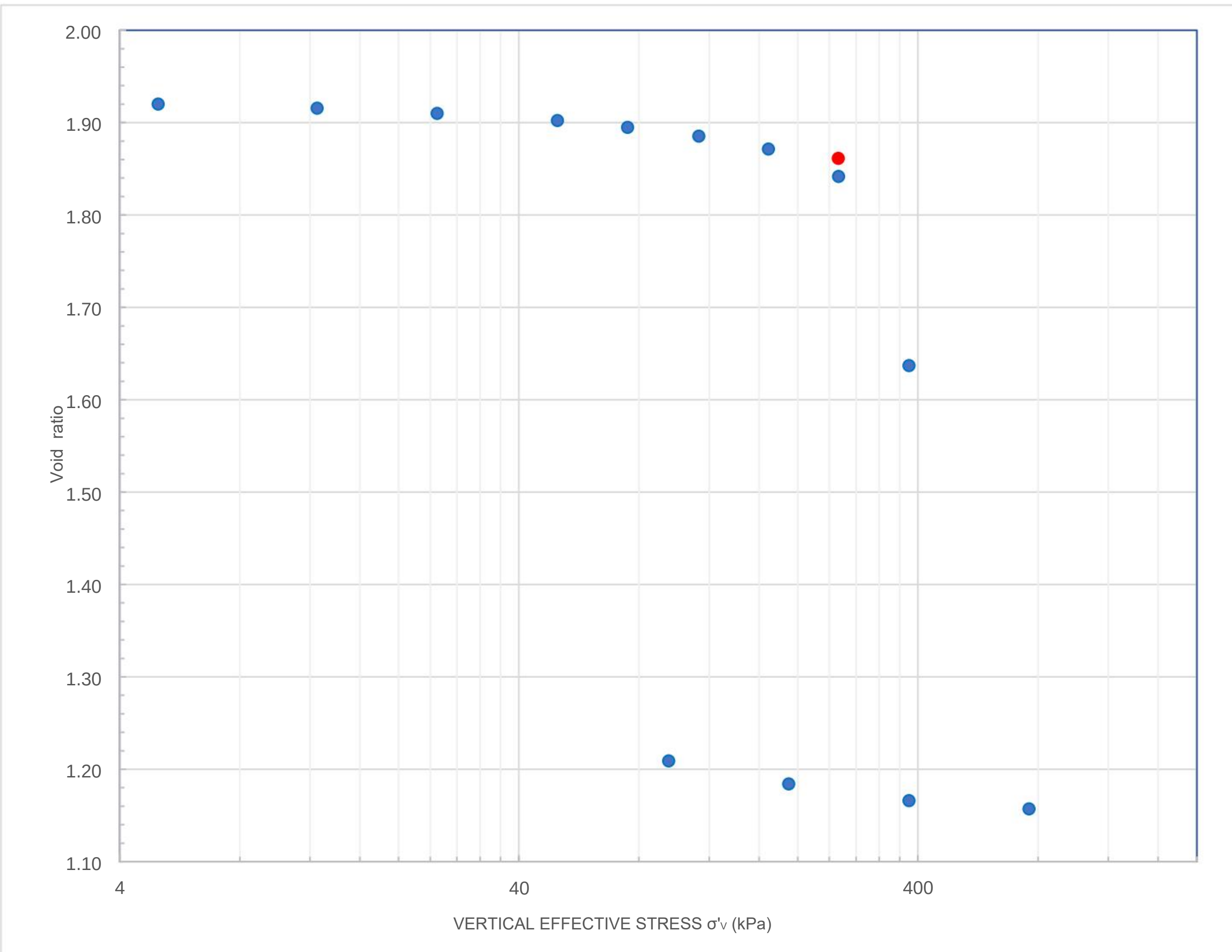
LC 22-300 Standard

Project:	500 Family-Side Ave,	Project # :	GO24253700
Client	Batimo Development Inc.	Client #:	BATIMO101
Extracted on:	2024-12-05	Borehole:	BH24-2
Extracted by:	Asma	Sample #:	Shelby Tube (ST5)
No lab. :	344 248	Depth :	4.60 - 5.20 m

Lenght (m)	Depth (m)	Visual description	Water content	Atterberg Limits	Consolidation	Shear strenght at swedish cone	Others	Parts preserved	Depth (m)
Top	4.6	Shelby tube packed in two plastic bags tied with adhesive tape.							4.6
Rejection		length: 55cm.					X (4.6 - 4.66)		4.66
3	4.7	Clay, a little silt with traces of sand.						X (4.66 - 4.76)	4.76
2	4.8	Moist, with an apparent consistency firm to stiff when intact and soft when disturbed.						X (4.76 - 4.86)	4.86
1	4.9	Color (Munsell): 5Y 4/1. Homogeneous sample showing no particular sedimentary characteristics overall. Clear cracks at 4.96 and 5.06m. (View photo)						X (4.86 - 4.96)	4.96
Conso	5				X (4.96 - 5.06)				5.06
Hydrometer	5.1						X (5.06 - 5.12)		5.12
Rejection							X (5.12 - 5.15)		5.15
	5.2								
	5.3								
Bottom									

CONSOLIDATION TEST LC 22-301 LC 22-301

CLIENT :	Batimo Development Inc.	**N/A:**	GO24253700
PROJECT :	BATIMO101	**CLIENT NUM :**	BATIMO101
SITE :	500 Family-Side Ave,	LAB **No .:**	344248
BOR/SAMPLE:	BH24-2 Shelby ST5	**DEPTH:**	4.96 - 5.06
COLLECTED BY:	Jim Brooks	**TEST DATE :**	16-Dec-24



Geotechnical characteristics

Initial void index (e_0):	1.92	Recompression index (c_r):	0.05
Water content (w_0) (%):	71.5	Virgin Compression Index (c_c):	1.44
Wet density (kN/m3):	16.0	Preconsolidation stress (σp_c) (kPa):	**253**
Initial saturation degree (Sr) (%):	100	Initial effective stress (σ_v) (kPa):	**52**

Prepared by : Carolane Croteau **Verified by:** Nadezda Zharkova, CPI **Date :** 2024-12-23

ROCK CORE TESTING

ASTM D 4543 and ASTM D 7012

O/R : GO24253700

Date : 2024-12-04

Customer:	Groupe EMD Batimo Inc.			Cust Ref:	BATIMO101		
Project :	500, Famille-Côté ave						
Part of work :	Borehole: BH24-1 and BH24-2						
ROCK CORE SAMPLING							
Sample reference number	344 240	344 243					
Borehole identification	BH24-1	BH24-2					
Identification of drilling course subsection	RC-14	RC-12					
Depth (mbgs)	23.5 - 23.7	25,80 - 25,90					
Rock core sampling done by	Customer	Customer					
LITHOLOGICAL DESPCRIPTION							
Type	Dolomite	Dolomite					
Presence of interbeds	No	No					
Intrusion	No	No					
COMPRESSION RESISTANCE							
Preparation of the extremities by sawing	Yes	Yes					
Evenness and angle	C	C					
Length after cuts (mm)	106,15	105,33					
Loading rate (MPa/s)	0,5	0,5					
Diameter of core (mm)	46,70	44,97					
Carrot weight (g)	493,40	442,20					
Height/diameter ratio	2,3	2,3					
Correction factor	1,0	1,0					
Load at breaking point (kN)	234	139					
Area (mm²)	1 713	1 588					
Compression resistance (MPa)	**136,6**	**87,5**					
Weight by volume (kN/m³)	26,6	25,9					
Volume weight (kg/m³)	2 711	2 640					
After-break appearance	Shattered	Shattered					

REMARQUES

Results are representative of the sample supplied by the customer.

Press (compression) Model : 1500 KN Serial number : 21005777

Prepared by : Hayet Souded, CPI Verified by: Catherine Haley, ing. Date : 2024-12-04

N/Ref.: LG027 (R07 2022-01-10) * Contractions used in the report: Compliant - C, Non-compliant - NC, Not available - N/D

Certificate of Analysis

Environment Testing

Client:	Groupe ABS 850 Industrial Ave (Suite B) Ottawa, ON K1G 4H3	Report Number:	3012834
Attention:	Mr. Mohammed Al-Khazaali	Date Submitted:	2024-11-26
PO#:		Date Reported:	2024-12-09
Invoice to:	Groupe ABS	Project:	GO24253700
		COC #:	231652

Dear Mohammed Al-Khazaali:

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:

APPROVAL: ______________________________

Addrine Thomas, Inorganics Supervisor

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.

Certificate of Analysis

eurofins | Environment Testing

Client:	Groupe ABS 850 Industrial Ave (Suite B) Ottawa, ON K1G 4H3	Report Number:	3012834
Attention:	Mr. Mohammed Al-Khazaali	Date Submitted:	2024-11-26
PO#:		Date Reported:	2024-12-09
Invoice to:	Groupe ABS	Project:	GO24253700
		COC #:	231652

Group	Analyte	MRL	Units	Guideline	1752607	1752608
				Lab I.D.	1752607	1752608
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2024-11-14	2024-11-14
				Sample I.D.	BH24-1 SS6	BH24-2 SS9
Anions	Cl	0.002	%		0.058	0.128
	SO4	0.01	%		0.03	0.04
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.78	1.46
	pH	2.00			8.59	8.57
	Resistivity	1	ohm-cm		1282	685
Moisture	Moisture-Humidite	0.1	%		41.4	36.2
Redox Potential	REDOX Potential		mV		114.7	102.6
Subcontract	S2-	0.01	%		0.01	0.04

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

Certificate of Analysis

Environment Testing

Client:	Groupe ABS 850 Industrial Ave (Suite B) Ottawa, ON K1G 4H3	Report Number:	3012834
Attention:	Mr. Mohammed Al-Khazaali	Date Submitted:	2024-11-26
PO#:		Date Reported:	2024-12-09
Invoice to:	Groupe ABS	Project:	GO24253700
		COC #:	231652

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 469355 **Analysis/Extraction Date** 2024-11-28 **Analyst** IP **Method** C SM2580B			
REDOX Potential	220 mV	100	97-103
Run No 469369 **Analysis/Extraction Date** 2024-11-28 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	6.18	99	90-110
Resistivity			
Run No 469417 **Analysis/Extraction Date** 2024-11-29 **Analyst** IP **Method** ASTM 2216			
Moisture-Humidite			80-120
Run No 469523 **Analysis/Extraction Date** 2024-12-02 **Analyst** M B **Method** AG SOIL			
SO4	<0.01 %	105	70-130
Run No 469614 **Analysis/Extraction Date** 2024-12-03 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %	96	75-125
Run No 469857 **Analysis/Extraction Date** 2024-12-05 **Analyst** AET **Method** SUBCONTRACT-SGS			

Guideline = *** = Guideline Exceedence**

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Certificate of Analysis

Eurofins Environment Testing

Client:	Groupe ABS 850 Industrial Ave (Suite B) Ottawa, ON K1G 4H3		Report Number:	3012834
			Date Submitted:	2024-11-26
			Date Reported:	2024-12-09
			Project:	GO24253700
Attention:	Mr. Mohammed Al-Khazaali		COC #:	231652
PO#:				
Invoice to:	Groupe ABS			

QC Summary

Analyte	Blank	QC % Rec	QC Limits
S2-			

Guideline = *** = Guideline Exceedence**

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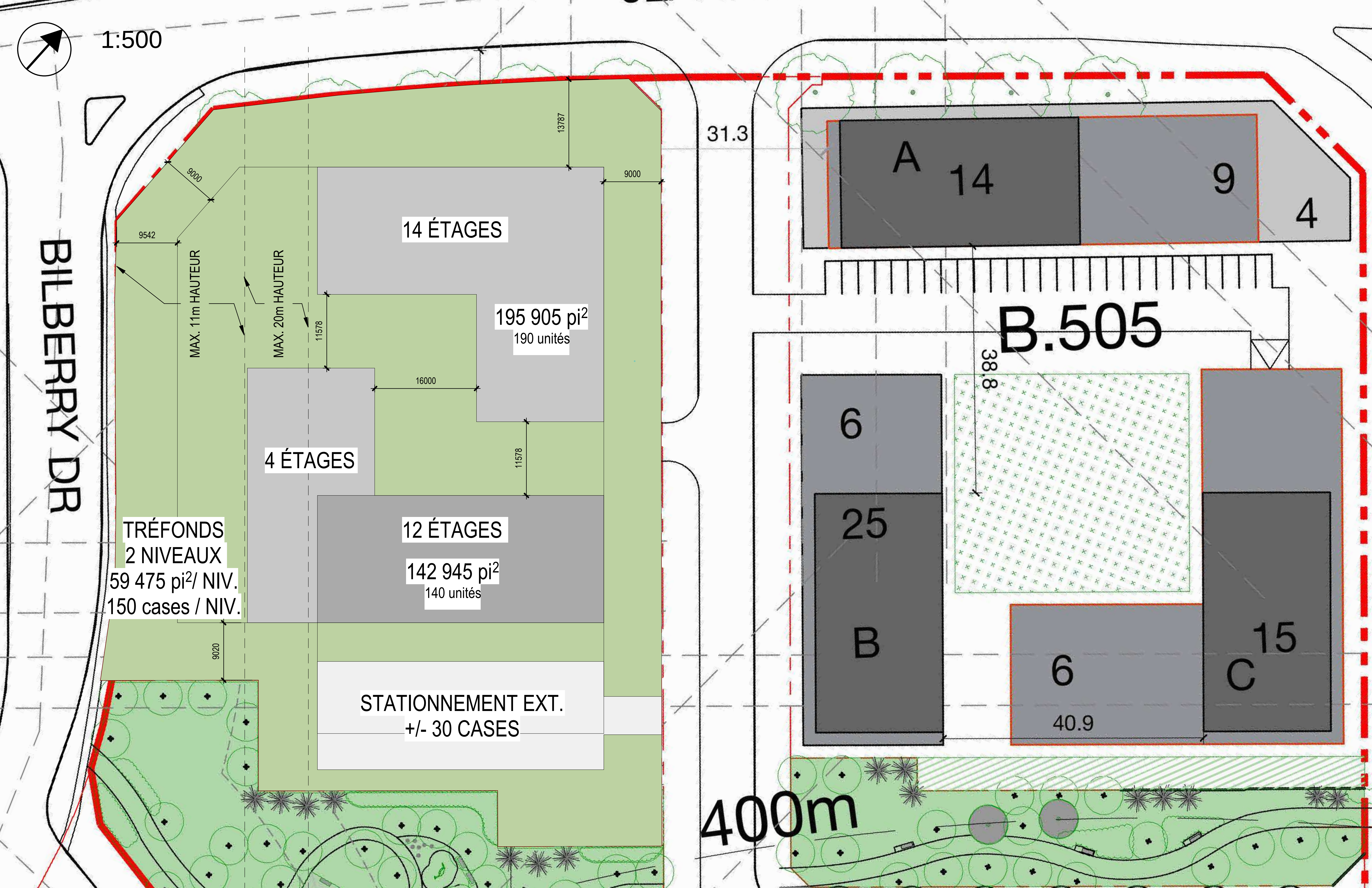
APPENDIX 4

DOCUMENTS RECEIVED FROM THE CLIENT


1:500
BILBERRY DR
14 ÉTAGES
195 905 pi²
190 unités
4 ÉTAGES
12 ÉTAGES
142 945 pi²
140 unités
TRÉFONDS
2 NIVEAUX
59 475 pi²/ NIV.
150 cases / NIV.
STATIONNEMENT EXT.
+/- 30 CASES
MAX. 11m HAUTEUR
MAX. 20m HAUTEUR
9000
9542
13787
9000
11578
16000
11578
9020
31.3
A
14
9
4
B.505
38.8
6
25
B
6
C
15
40.9
400m