Geotechnical Investigation

Proposed Two New Apartment Buildings

1050 Tawadina Road, Ottawa, ON
(Revision 1)

Prepared For:

WestUrban Developments Ltd.
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Campbell River, BC, Canada
V9W 0E8

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1 Introduction

Englobe Corp. (Englobe) is pleased to present the findings of our geotechnical investigation for the proposed two new apartment buildings (Project) at 1050 Tawadina Road in Ottawa, Ontario (Site).

Englobe Corp. was retained by WestUrban Developments Ltd. (Client) to carry out a geotechnical investigation consisting of four (4) boreholes within the footprint of the proposed Project. A signed authorization to proceed with this investigation was provided by Mr. Cameron Salisbury of WestUrban Developments Ltd. on April 7, 2022, followed by purchase order number PO#157-1-052 on April 22, 2022. An additional MASW seismic survey was authorized by Mr. Salisbury on behalf of the Client on July 4, 2023.

This report is prepared for the sole use of the Client. The use of the report, or any reliance on it by any third party, is the responsibility of such third party. This report is subject to the limitations shown in Appendix A. It is understood that the Project will be performed in accordance with all applicable codes and standards present within its jurisdiction.

This geotechnical investigation has been undertaken in conjunction with a Phase I ESA by Englobe. The results of the Phase I are provided under separate cover.

2 Site Description and Project Understanding

The Site is located in an area of the former Rockcliffe Canadian Forces Base designated for residential development in Ottawa, ON. It is located at the municipal address of 1050 Tawadina Road in Ottawa, ON. The location of the Site is shown on the Site Location Map, Figure 1 in Appendix B. It is located approximately 0.5 km south of Sir George-Etienne Cartier Parkway and 1.25 km east of Aviation Parkway. The Site is bounded by future Tawadina Road from the north, Michael Stoqua Street from the east and Bareille Snow Street. The Site is currently vacant land covered with shrubs and bushes. Debris of previous structures included concrete, rebars, brick, and boulders were observed to cover most of the southeast part of the Site. At the west and south sides, the Site is bounded by vacant land subject to future development. To the east and southeast of the Site, newly constructed residential dwellings were observed.

Englobe's understanding of the Site and the Project is based on the following documents provided by the client:

- 'Design Update Package' dated September 16, 2022, prepared by FAAS and received from the Client on September 22, 2022;
- 'Site Plan', Drawing No. SPC.100, dated January 14, 2022, prepared by FAAS and received from the Client on July 11, 2023

- Landscape Plan Drawings, Drawing Nos. L1.0 to L2.1, dated January 14, 2022, prepared by FAAS and received from the Client on July 11, 2023; and
- Building Section Drawings, Drawing Nos. SPC.200 to SPC.202, dated January 14, 2022, prepared by FAAS and received from the Client on July 11, 2023.

The Site covers an approximate area of 7,179 m^2. The proposed development will consist of two proposed 9-storey L-shaped buildings, Building A and Building B in the in the northwest and southeast halves of the Site respectively, with one shared underground parking level between the two buildings. The proposed buildings will be separated by an existing easement which is integrated into a landscaped courtyard between the two buildings, with a single-storey amenities building located centrally in the courtyard.

Based on available information from previous geotechnical studies within the area and according to GeoOttawa database, the Ottawa Interceptor Outfall Sewer pipe crosses beneath the Site in the East – West direction within the existing easement. The Ottawa Interceptor Outfall Sewer pipe is a 2.4 m diameter concrete pipe constructed within the bedrock using a tunnelling construction technique at an approximate elevation of 36.25 to 35.25 masl. The elevation of the centreline of the pipe is approximately 52 m lower than the existing ground surface (~El. 88.6 masl).

At the time of preparation of this report, Englobe has not been provided with any structural drawings of the proposed development or proposed grading plans of the Site. Englobe should be retained during detailed structural design to review the proposed foundation drawings and grading plans once they become available to ensure conformance with the general recommendations provided within this report.

The following assumptions about the proposed Project were made by Englobe during the preparation of this geotechnical investigation:

- The two buildings and the underground parking will be founded on shallow foundations supported on bedrock at an approximate elevation below 85.0 m above sea level (masl);
- The proposed structure will be designed under Part 4 of the Ontario Building Code (OBC 2012) and will therefore require a Site Classification for seismic Site response according to Table 4.1.8.4.-A.
- There will be no existing or proposed retaining walls or slopes exceeding 1 m in height. Therefore, a slope stability assessment has not been conducted as part of this report.
- No significant global grade raises (i.e. > 0.5 m) are envisioned.

3 Scope of Work

Englobe's scope of the work for this assignment was outlined in our proposal (Ref No: P2203079.000 dated April 5, 2022) and was agreed to by the Client on April 7, 2021 by means of a signed services acceptance followed by a purchase order number PO#157-1-052 on April 22, 2022. Additional scope of work was outlined in email communication with the Client on June 27, 2023, and approved in email communication by the Client on July 4, 2023.

Our mandate consisted of the following activities in general:

- Retain a private utility subcontractor to provide public and private underground utility clearances at the proposed borehole locations;
- Retain a geotechnical drilling subcontractor to drill four (4) boreholes including the installation of one monitoring well;
- Supervise the fieldwork and log the subsoil and groundwater at the borehole locations based on the recovered samples;
- Perform geotechnical laboratory testing including selected index testing, and standard corrosion packages on selected soil samples;
- Conduct an MASW seismic survey to provide detailed Seismic Site Classification recommendations; and

  Preparation of this geotechnical investigation report summarizing the findings of the field and laboratory testing and providing geotechnical parameters and recommendations to assist designers.

4 Field Investigation and Laboratory Testing

4.1 Geotechnical Drilling

Background geotechnical information in the general area was reviewed from an earlier geotechnical investigation report for subdivision approval at former CFB Rockcliffe development by DST dated September 2015. Englobe visited the Site at the outset of the fieldwork to evaluate the Site conditions, accessibility and mark the proposed borehole locations. Utility clearance was carried out by Underground Service Locators (USL-1) on behalf of Englobe and all utility clearance documents were obtained before the commencement of drilling work.

The fieldwork was conducted on May 5, 2022 and consisted of drilling four boreholes that were advanced to a maximum approximate depth 5.7 m below the ground surface (mbgs). The boreholes were drilled using a CME-850 track-mounted drilling rig, outfitted with hollow stem augers. The equipment used for drilling was owned and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario.

Standard Penetration Testing (SPT) with a full-weight 107 kg hammer was performed at 0.75 m depth intervals. Disturbed soil samples were collected using a 51 mm diameter SPT split spoon sampler in addition to auger samples. The subsoils at the borehole location were logged by Englobe based on the samples that were recovered. Soil samples were labelled and submitted to Englobe's geotechnical laboratory for further visual examination and laboratory testing.

The bedrock was cored and sampled to approximately 3.0 m in BH22-2 and 22-3. Rock cores were obtained by diamond drilling and wireline tooling. Rock cores were retrieved in double-walled NQ coring methods.

A 51 mm diameter monitoring well was installed in borehole BH21-04. The well was protected in an above-ground protective casing. Details and location information of the well are provided in Section 5.3.

A summary of the new boreholes is shown in Table 4-1. The borehole locations are shown on the Borehole Location Plan attached as Figure 2 in Appendix B.

Table 4-1: Summary of New Borehole Depths

Borehole No.	Surface Elev. (masl)	Drilling Depth (mbgs)	Drilling Termination Criteria	Remarks
BH22-1	88.77	2.4	Auger refusal	--
BH22-2	89.66	5.0	~ 3.0 m confirmatory rock core	--
BH22-3	88.84	5.7	~ 3.0 m confirmatory rock core	--
MW22-4	88.61	2.1	Auger refusal	Monitoring well in the overburden

The boreholes were backfilled upon completion of drilling with bentonite and drill cuttings as applicable and restored to the existing ground level. In BH22-2 and 22-3, the bedrock cores were sealed with holeplug.

The subsurface stratigraphy encountered at the borehole locations was recorded by the Englobe representative based on the samples that were recovered and submitted to Englobe's laboratory for further visual examination and/or transported to external laboratories for testing. The boreholes were surveyed with a GPS unit to record their locations and elevations.

4.2 Laboratory Testing

Geotechnical laboratory testing was conducted on representative soil samples, and consisted of sieve analysis, and moisture contents. The results of sieve analyses and moisture content are shown on the borehole log records in Appendix C.

Paracel Laboratories Ltd., in Ottawa, Ontario carried out chemical tests on representative soil samples to determine the susceptibility to corrosion to ductile iron pipes and concrete attack parameters. The chemical parameters consisted of pH, chloride, sulphate, sulphide, resistivity, and redox potential). Laboratory test results are included in Appendix C.

4.3 Geophysical Testing

Englobe retained a geophysics subcontractor (Geophysics GPR International Inc.) to measure the shear wave velocity in the soils and bedrock using the Multi-Channel Analysis of Surface Waves (MASW) method. The testing was performed on July 19, 2023, and the resultant shear wave velocity profile is provided in Appendix E.

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5 General Description of Subsurface Conditions

In general, the soil stratigraphy within the borehole locations consisted of fill underlain by limestone bedrock.

Details of the subsurface conditions encountered in the boreholes are presented graphically on the borehole logs in Appendix C. The soil at the test locations were classified based on the results of the grain size analysis tests. General descriptions of the subsurface conditions found at the test locations are provided in the following subsections.

It is important to note that the soil descriptions presented below and in the Borehole Logs represent the soils encountered at the borehole locations only. They may vary between and beyond borehole locations, especially in previously excavated and/or filled areas.

5.1 Fill

Fill soils were encountered in all borehole locations. The depth of the fill ranged from approximately 1.8 m in BH22-2 to 2.4 mbgs in borehole BH22-3. This corresponds to approximate elevations ranging from 87.8 to 86.5 masl, respectively.

The fill soils were generally described as a sandy soil with varying silt and gravel content. Thin clayey silt fill layers with minor portions of sand and gravel were observed at the surface in BH22-2 and at the bottom of BH22-1. Organic materials (roots) were observed at the surface.

The fill in general was brown in colour and was observed to damp to moist with moisture content values ranged from 4.2% to 17.9%, on average 10.9%.

Four samples from the fill soils underwent grain-size analysis testing. The test results are summarized in Table 5-1 and presented in Appendix C.

Table 5-1: Summary of Particle Size Analysis Results of the Fill

Sample Tested	Sample Depth / Elev. (m)	Grain Size Analysis (%)				USCS Soil Classification*
		Gravel	Sand	Silt	Clay	
BH22-1 / SS-2	1.1 (87.7)	38	42	21		Silty Sand and Gravel
BH22-2 / SS-3(A&B)	1.7 (88.0)	8	56	36		Silty Sand
BH22-3 / SS-2	1.1 (87.7)	12	50	38		Silty Sand
BH22-3 / SS-3B	1.7 (87.1)	27	60	13		Gravelly Sand

*** USCS:** Unified Soil Classification System

The recorded SPT ‘N’ values for the range from 3 to 49 blows/300 mm, indicating the FILL has inconsistence compactness ranging from loose to dense.

5.2 Bedrock/Auger Refusals

Auger refusal was encountered in all boreholes. The bedrock was cored and sampled down to approximately 3.0 m in the bedrock using diamond core (NQ size) between El. 87.8 and 84.6 masl in BH22-2 and between El. 86.4 and 83.2 masl in BH22-3. The bedrock was observed to be strong, grey, limestone.

During the core drilling, measurements including Total Core Recovery (TCR) and Rock Quality Designation (RQD) were carried out for 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. The RQD is defined as percentage of the sum of the core pieces over 100 mm divided by the total length of core run. The TCR and RQD for the rock cores are presented in the borehole log records in Appendix C.

Based on the retrieved rock cores, the bedrock is observed to be moderately weathered near the bedrock surface to sound rock at deeper levels. The bedrock contained moderately spaced flat to vertical joint discontinuities.

The RQD-value of the first rock core (RC5) in BH22-3 was 42% indicating weathered rock. The RQD-value increases with depth and ranged between 61 to 100% indicating the bedrock is sound bedrock below an approximate El. 85.0 masl in BH22-3 and 87.8 masl in BH22-02.

Auger refusal without confirmatory rock coring was encountered on inferred bedrock/boulders in BH22-1 and MW22-4 at 2.4 m (El. 86.4 masl) and 2.1 m (El 86.5 masl) depths, respectively. Designers and Contractors are cautioned that these may represent refusal on a cobble and/or boulder as opposed to the bedrock surface.

Table 5-2 summarizes observations with respect to bedrock coring and auger refusal on inferred bedrock/boulders. Rock core photos are provided in Appendix C.

Table 5-2: Summary of Bedrock Observations

Borehole No.	Auger Refusal Elev. (masl)	Weathered Bedrock Elev. (masl)	Sound Bedrock Elev. (masl)	Bottom of Rock Core Elev. (masl)	Remarks
BH22-1	86.4	--	--	--	SPT sampler and Auger refusal on inferred bedrock/boulders
BH22-2	87.80	--	87.80	84.60	~ 3.0 m Bedrock coring
BH22-3	86.40	86.40	85.00	83.20	~ 3.0 m Bedrock coring
MW22-4	86.5	--	--	--	SPT sampler and Auger refusal on inferred bedrock/boulders

5.3 Groundwater

Water was not observed in any of open boreholes during and upon completion of drilling on May 5, 2022. The water level was measured in the monitoring well in MW22-4 on June 03, 2022 and the well was observed to be dry. Englobe returned to measure the water level in MW22-4 on August 16, 2023 and the groundwater level was noted at an approximate depth of 1.9 mbgs, corresponding to an elevation of El. 86.7 masl. Groundwater levels were also measured on August 16, 2023 in two wells on the Site which were not installed by Englobe and were noted at approximate depths of 1.8 and 4.6 mbgs respectively.

Based on previous investigations by Englobe, formerly DST, dated August 2006 for the overall Wateridge area, the groundwater level within the general proximity of the Site, based on records of monitoring wells installed approximately at 75.0 to 150.0 m distance from the Site, is expected to vary between approximate depths of 0.8 and 2.5 mbgs, which corresponds to approximate elevations between 87.4 and 80.4 masl.

Water levels are expected to fluctuate seasonally and in response to precipitation and snowmelt events.

Monitoring well details and water level observations are shown on the borehole log records provided in Appendix C.

It is important to emphasize that a hydrogeological investigation in support of PTTW, EASR application or dewatering volume estimate was not requested at the time of this geotechnical investigation.

Table 5-3: Summary of Monitoring Well Observations

Monitoring Well No.	Location of Screen (masl)	Water Level Observation		
		Date	Depth (mbgs)	Elevation (masl)
MW22-4	88.0 to 86.5 (screen is located within the overburden)	June 3, 2022	Dry	--
		August 16, 2023	1.9	86.7
A366096-01	Well details unavailable (screen is located within overburden)	August 16, 2023	1.8	-
A366096-02	Well details unavailable (screen is located within bedrock)	August 16, 2023	4.6	-
BHMW 7	88.9 to 81.1 (screen is located within the overburden)	August 24, 2006	1.8	87.1
BHMW 8	82.9 to 74.3 (screen is located within the overburden)	August 24, 2006	2.5	80.4
BHMW10 OB	81.8 to 78.4 (screen is located within the overburden)	August 24, 2006	2.5	80.3
BHMW10 BR	70.2 to 66.8 (screen is located within the overburden)	August 24, 2006	5.0	77.8
BHMW11	85.4 to 82.1 (screen is located within the overburden)	August 24, 2006	0.8	87.4

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6 Discussion and Recommendations

Based on the results of geotechnical field and laboratory investigation performed, the following discussion is provided to assist the Client and their Designers with the development of foundation general arrangements and geotechnical design for the proposed Project in general. The recommendations provided within this report are based on our understanding of the proposed Project which is summarized above in Section 2 and are general in nature. If any of these understandings changes, Englobe should be contacted to assess the implications of those changes on the recommendations provided herein.

Based on the soil conditions encountered in the boreholes, and assuming that they are representative of the soil conditions across the Site, the most important geotechnical considerations for the design of the foundations for the proposed Project are expected to be the following:

- .
- **Bearing Capacity on Sound Bedrock:** Based on latest available drawings from the Client, the foundations will be founded on conventional pad and strip footings at an elevation of El. 87.20 masl, with an elevation of 85.23 El. in the parking ramp area. The foundations as shown will therefore be within in the existing uncontrolled FILL material. This FILL is not considered suitable founding material. All foundations must be founded on bedrock, or founded on new Engineered Fill or lean mix concrete in direct contact with bedrock. Englobe recommends raising the grade from the sound bedrock to the founding elevation using lean mix concrete. If lean mixed concrete is used below any footings it must extend a minimum of 0.3 m beyond the edge of the footing and then

downward at a 1H:1V. Recommended design bearing pressures on lean mix concrete would be the same as those for the bedrock, provided that the underlying subgrade has been approved by the Geotechnical Engineer. Consideration should also be given to extending the founding elevation of the footings to the sound limestone bedrock beneath the weathered zone. Any loose or unstable rock pieces should be removed from the footing zone of influence.

- **Ottawa Interceptor Outfall:** Ottawa Interceptor Outfall Sewer pipe crosses beneath the Site in the East – West direction within the existing easement. The Ottawa Interceptor Outfall Sewer pipe is a 2.4 m diameter concrete pipe constructed within the bedrock using a tunnelling construction technique at an approximate elevation of 36.2 to 35.2 masl. The centreline elevation of the pipe is approximately 52 m lower than the existing ground surface (~El. 88.6 masl). The proposed building will be supported on spread or strip footing system founded on the bedrock at an approximate El. 85.0 masl. Assuming absence of any major limestone bedrock faults and/or solution cavities and rubble zones under the footprint of the proposed buildings, a minimum of 40 m unweathered competent bedrock cover is expected to prevent stressing the existing interceptor outfall sewer pipe.
- **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. As part of the fieldwork program, Englobe has conducted geophysical testing to measure the shear wave velocity profile. Based on the results of the MASW testing, “Site Class A”, with respect to Table 4.1.8.4.A of the OBC-2012, is applicable to the design of the proposed buildings, provided they are founded in direct contact with sound bedrock and subject to the limitations of the code.
- **Temporary Construction Dewatering:** Excavation for the structure will penetrate through the fill into the bedrock. Groundwater and surface runoff water may infiltrate and accumulate at the bottom of the excavations due to seasonal changes and extreme weather events. It is expected that dewatering will be required during the construction stage for this building location to keep the excavation reasonably dry. Dewatering may be achievable with traditional sump and pump dewatering method. Application for Permit to Take Water from the Ministry of Environment is not required based on the observed level of water in the monitoring well.
- **Permanent Drainage and Waterproofing:** The excavations for the underground parking will extend below the existing bedrock surface resulting in water pooling at the proposed floor slab level. Therefore, permanent under-floor drainage and waterproofing are required. Exterior perimeter drains are not recommended for this Site. Full water proofing membranes such as a WR Meadows Mel-ROL PRECON, or an equivalent type product for walls and under-slab will be required. Water stops should be installed at cold joints in the foundation walls and floor-wall joints. Englobe also recommends the design of the building basement as a fully waterproof ‘bath-tub’ design (without

external perimeter drains) to avoid potential adverse impacts due to moisture movements in the immediate areas surrounding the proposed building footprint.

6.1 Site Preparation

Considering the proposed development will have one level of underground parking, all existing Fill and weathered rock are expected to be removed completely within the footprint of the proposed buildings, down to competent bedrock capable of supporting the structural loads of the proposed development. The existing Fill materials are heterogenous (i.e. contain demolition debris) in nature and not suitable for backfilling or Site grading.

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 storm water and runoff including an adequate pumping system.

6.1.1 Subgrade Preparation

The excavations for the foundations of the proposed two structures and floor slabs are generally expected to extend down to sound bedrock. Based on the recent boreholes the sound bedrock is expected to be encountered at approximate depths between 1.9 to 3.8 mbgs, corresponding to approximate elevations near El. 87.8 to 85.0 masl. It is our understanding that the final level of the finished ground floor will be approximately at the street level (~El. 90.2 masl). Based on the latest available drawings from the Client, the footings shown to be founded at an elevation of El. 87.20 masl, with an elevation of 85.23 El. In the parking ramp area. Therefore, moderate bedrock excavation will be required to achieve the desired elevations which is expected to generate manageable amount of excavated rock materials.

Subgrade preparation for footings founded on rock will involve the removal of all soils and weathered bedrock to expose a sound limestone bedrock. Any pieces of rock that can be manipulated by conventional excavation equipment should be removed, and as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed bedrock surface should be examined and approved by the Geotechnical Engineer to confirm the competency to support the design bearing pressures.

Confirmation of bedrock quality during construction will require the Contractor to perform probing of the bedrock using 50 mm diameter drill holes drilled to a depth of 1.5 m within the footings. These holes will need to be reviewed by the Geotechnical Engineer to confirm that no significant mud seams or voids exist at the footing location. If mud seams are found, localized areas of the footings may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly. The locations of these probe holes should be selected under the direction of the Geotechnical Engineer during construction. Contractors should plan for one probe per pad footing and a minimum or 1 probe every 6 m in strip footings.

6.1.2 Interference with Existing Underground Utilities

Designers should review the proposed excavation locations and compare them to the location of any existing underground utilities in their vicinity and address the potential impacts of the proposed development on all existing underground utilities in advance of construction. Existing utilities that are excavated or exposed as part of construction will need to be supported and rerouted around the building.

6.1.3 Protection of Adjacent Structures

Designers and Contractors should review the geometry, depth, and sloping requirements of all planned excavations. Currently, there are no structures adjacent to Site location, except the existing roadways and the residential units to the East of the Site on Michael Stoqua St. Proposed excavation dimensions should be compared to adjacent load bearing structures should they be available to ensure they are not undermined. Undermining is avoided by ensuring that no excavations penetrate below an imaginary line drawn outward and downwards 10H:7V below the toe or founding level of any load bearing structures. If the limit of not undermining adjacent structures cannot be satisfied, then an Engineered Shoring system and/or underpinning program will need to be considered.

6.2 Excavations

Based on Englobe's current understanding of the Project, we anticipate that the excavations will extend to an approximate elevation of El. 85.0 masl. Therefore, the proposed excavation will range from approximately 1.9 to 3.8 mbgs. Excavations will extend through the soils and into the limestone bedrock. Based on the excavation depth required, it is anticipated that excavations for the building will need to be performed with Engineered Shoring to avoid undermining the adjacent roadways.

6.2.1 Localized Shallow Sloped Excavations

The comments in this subsection are intended for small localized shallow excavations performed in soil. The following subsection is intended to discuss Engineered Shoring for the deeper building excavation.

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, amended. The comments within this subsection are intended to be in addition to, and not a replacement of the OHSA requirements.

Above the measured groundwater level, the soils are considered to be “Type 3 Soil” and as per OHSA, the excavation side slopes must be sloped from its bottom cut back at 1H:1V. Below the groundwater level, the soils are considered to be “Type 4 Soil” and as per OHSA, the excavation side slopes must be sloped from its bottom cut back at 3H:1V

Excavation through the soil below the groundwater level are anticipated to be more problematic. The sand to gravelly sand fill is anticipated to flow or run into the excavation. Therefore, as there are likely space restrictions, it is recommended the excavations be undertaken within the confines of an 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. Recommendations for appropriate dewatering measures beyond conventional sump pump techniques such as a positive dewatering system (e.g., well points or other specialized methods) to effectively lower the static groundwater level shall be provided by a specialized dewatering contractor.

For excavations into bedrock, the upper weathered rock zone will require back sloping depending on the degree of weathering. The bedrock quality and Site-specific requirements need to be assessed during construction by the Geotechnical Engineer. For planning purposes, a weathered bedrock is recommended to be treated as a “Type 2 Soil”. Sound rock would generally be self-supporting, however, as a precautionary measure, it should be back-sloped at 10V:1H. All rock excavations should be scaled, to remove loose rock fragments to ensure safe working conditions. All rock faces should be reviewed by a Geotechnical Engineer to look for loose pieces and wedge failures. Rock bolting for worker safety may be necessary depending on the layout and field condition at that time.

The stability of the excavation side slopes will be highly dependent on the Contractor’s methodology. No surface surcharges should be placed closer to the edge of the excavation than a distance equal to twice the depth of the excavation, unless an excavation support system has been designed to accommodate such a surcharge.

6.2.2 Engineered Shoring

Engineered Shoring systems through soil often include (but are not limited to): soldier piles and lagging, interlocking sheet piles, secant and/or tangent walls, permanent diaphragm walls, etc. The appropriate method should be selected by the Project Designers and Contractors considering the space restrictions, estimated costs, and availability of materials. Engineered Shoring systems must be designed by an experienced Professional Geostructural Engineer taking into consideration the following Site-specific aspects:

- Lateral earth pressures;

- Hydraulic pressures of the groundwater;
- Loads from any adjacent structures, or infrastructure being retained;
- Seismic loadings;
- Freeze-thaw action on the face of the excavations;
- Expansion and contraction of shoring elements;
- Pre-stressing loads or post tensioning loads on tie backs;
- Possible surcharge loads throughout construction (i.e., trucks, equipment, stockpiles, etc.);
- Vibrations induced by construction processes; and
- Compatibility with the design of proposed waterproofing and drainage systems for the sub-surface levels.

Soldier piles and sheet piling, if used would require predrilling to provide sufficient embedment to achieve toe fixity. It is expected that the Engineered Shoring systems would need to be provided with tie-back rock anchors to ensure their lateral stability. It is recommended that the Client retain Contractors and Designers who have significant experience with deep 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.

The preliminary lateral earth pressure parameters to assist Designers and Contractors with shoring designs through soil are discussed in Section 6.7 below.

6.2.3 Bedrock Excavation

For excavations into bedrock, upper weathered rock zone will require back sloping depending on the degree of weathering. The Site-specific bedrock quality and associated requirements need to be assessed during construction by the Geotechnical Engineer. For planning purposes, a weathered bedrock is recommended to be treated as a “Type 2 Soil”. Sound rock would generally be self-supporting, however, as a precautionary measure, it should be back-sloped at 10V:1H. All rock excavations should be scaled, to remove loose rock fragments to ensure safe working conditions. All rock faces should be reviewed by a Geotechnical Engineer to look for loose pieces and wedge failures.

Bedrock excavation will require pneumatic or hydraulic breakers such as hoe-rams or heavy rock excavation equipment capable of breaking and ripping sound limestone bedrock. Alternatively, controlled blasting techniques may need to be used, subject to the laws and blasting restrictions that are in effect for the area. Designers are referred to the OPSS.MUNI 120 and the City of Ottawa Special Provision F-1201 specifications for the use of explosives. In general, these documents require a blasting plan to be prepared by a Blasting Engineer. They also require conducting pre-blast surveys on nearby buildings, utilities, structures, water wells, and facilities likely to be affected by the blast. Vibration monitoring during the blasting in nearby structures or infrastructure is required.

6.3 Temporary Construction Dewatering

As discussed in Section 5.3, one monitoring well (MW22-4) was installed at the Site. The water level was recorded in MW22-4 on June 03, 2022 and was observed to be dry. Based on previous investigations by Englobe, formerly DST, dated August 2006 for the overall Wateridge area, the groundwater level within the general proximity of the Site, based records of monitoring wells installed approximately at 75.0 to 150.0 m distance from the Site, is expected to vary between approximate depths of 0.8 and 2.5 mbgs, which corresponds to approximate elevations near 87.4 and 80.4 masl. Given that excavations are expected to extend through the sandy fill into the bedrock to an approximate elevation El. 85.0 masl, groundwater and surface water seepage are expected in the excavations and will need to be adequately controlled.

Water quantities will depend on seasonal conditions, depths of excavations, presence and lateral extents of fractured rock zones, and the duration that excavations are left open. Groundwater will travel easily through the fill material and weathered rock surface. 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.

Effective groundwater control prior to and during construction and possibly permanently in this case are expected to be required. Recommendations for appropriate dewatering measures beyond conventional sump pump techniques such as a positive dewatering system (e.g., well points or other specialized methods) to effectively lower the static groundwater level shall be provided by a specialized dewatering contractor. A Permit to Take Water (PTTW) from the Ontario Ministry of Environment will be required if the quantity of water to be pumped from the Site exceeds 50,000 L/day. Based on observation made during the site investigation and observed water level in the MW22-04 and other available information to date, it is expected that PTTW is not required.

It should be realized that dewatering can cause ground settlement that extends laterally beyond the immediate area of dewatering. It is recommended that the contractor assess the likely impact on nearby existing structures, underground services, roadways, groundwater wells and use methods which will control the dewatering impact. A pre-construction survey documenting the conditions of nearby settlement-sensitive facilities/infrastructure be completed prior to start of construction.

6.4 Foundations

Englobe's original previous reporting dated November 03, 2022 provided bearing capacity recommendations based on proposed footings founded at an approximate elevation of El. 85.0 masl to ensure that the structures would be founded on sound bedrock to avoid any differential settlement behaviour. A factored bearing resistance of 1 MPa under Ultimate Limit State (ULS) conditions was recommended on sound bedrock.

It is Englobe's understanding based on the latest available drawings from the Client dated May 30, 2023, that the foundations for the two proposed structures as currently designed extend to an elevation of El. 87.20 masl, with an elevation of El. 85.23 masl in the parking ramp area, where the soil would be a mixture of sound limestone bedrock and heterogeneous FILL material.

The in-situ FILL material is not considered suitable founding material and does not provide the required bearing capacity to support the proposed structures as currently designed. Any in-situ fill material must be removed, and it is recommended that any required grade raises above the sound bedrock subgrade be performed using lean mix concrete. If lean mix concrete is used below any footings, it must extend a minimum of 0.3 m beyond the edge of the footing and then outwards at a 1H:1V ratio. As currently designed, the founding elevation of the proposed conventional strip footings is up to 2.2 m above the recommended founding elevation of El. 85.0 masl provided by Englobe in our original Geotechnical Report. It is Englobe's strong recommendation that the founding elevation of the footings be adjusted to ensure that all footings are founded on sound limestone bedrock beneath the weathered zone.

6.4.1 Footings on Rock

For conventional pad and strip footings founded on sound limestone bedrock, a factored bearing resistance of 1 MPa under Ultimate Limit States (ULS) conditions is recommended on sound bedrock, according to CFEM (2006). This includes for a geotechnical resistance factor of $\Phi = 0.5$.

There is no corresponding design bearing pressure recommended under Serviceability Limit State (SLS) conditions for bedrock as settlement under the ULS condition is expected to be minimal. Designers should keep footing dimensions to a minimum of 1.0 m for pad footings, and 0.5 m for strip footings regardless of the bearing pressure being used.

Subgrade preparation for footings founded on rock will involve the removal of all soils and weathered rock to expose sound bedrock. Any pieces of rock that can be manipulated by conventional excavation equipment should be removed, as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed surface should be examined by the Geotechnical Engineer to assess its competency.

Confirmation of bedrock quality during construction will require probing of the bedrock at footing locations using 50 mm diameter holes drilled to a depth of 1.5 m within the footprint of footings. These holes will need to be reviewed by the Geotechnical Engineer to confirm that no significant mud seams or voids exist. If mud seams are found, localized areas of the footings may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly. The locations of these probe holes should be provided under the direction of the Geotechnical Engineer during construction.

6.4.2 Lean Mix Concrete

If the grade is required to be raised between the approved sound bedrock subgrade and the design footing elevation, then it is recommended to use a lean mix concrete, as opposed to with granular fill soils. If lean mixed concrete is used below any footings it must extend a minimum of 0.3 m beyond the edge of the footing and then downward at a 1H:1V. Recommended design bearing pressures on lean mix concrete would be the same as those for the bedrock, provided that the underlying subgrade has been approved by the Geotechnical Engineer.

6.5 Frost Protection

All footings for heated structures must be provided with a minimum of 1.5 m of earth cover, and 1.8 m of earth cover for unheated or isolated structures in the Ottawa area. Otherwise, an equivalent insulation detail as well as insulated concrete forms would be required in order to provide adequate protection against frost action during and following foundation construction. Where soil cover cannot be provided, an insulation detail should be designed or approved by a Geotechnical Engineer. Contractors must be aware that this detail may be such that the insulation may need to be placed below the footing and then the footing poured on top, and therefore pre-approval is recommended to ensure excavations and backfill are properly planned.

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.

6.6 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. Based on the seismic MASW survey results provided in Appendix E, the shear wave velocity values were calculated through the soil and rock, to estimate the Seismic Site Class. In the case that the footings are founded on intact bedrock, the structures can be designed to “Site Class A” with respect to Table 4.1.8.4.A of the OBC-2012, and subject to the limitations of the code.

6.7 Lateral Earth Pressures

The following preliminary lateral earth pressure parameters are provided to assist Contractors and Designers with the design of both permanent basement walls and temporary Engineered Shoring systems, if used. Designers will need to review if hydrostatic pressures are to be included in the earth pressure calculations based on the permanent drainage designs. If a fully waterproof 'bath-tub' design without perimeter drainage is being used, then hydrostatic pressures will need to be included in the design.

6.7.1 Static Conditions

The following Rankine earth pressure coefficients are being provided to assist Designers.

Table 6-1: Recommended Lateral Earth Pressure Coefficients for Static Conditions

Soil	Bulk Density 'γ' (kN/m³) *	Angle of Internal Friction, ϕ' (degrees)	Undrained Shear Strength, Su (kPa)	Rankin Earth Pressure Coefficients**		
				K_a	K_o	K_p
Existing Uncontrolled Cohesionless FILL Loose to compact	20	28	0	0.36	0.53	2.77
New Compacted Granular Backfill OPSS "Granular B, Type I"	22	30	0	0.33	0.50	3.00

** Only the bulk unit weight is being presented, Designers will need to assess whether bulk, saturated, and/or submerged unit weights should be used based on their design conditions.*
***Assumes level/flat backfill surface. If Engineered Shoring is used, then Designers should refer to CFEM-2006 for design assistance and a Geotechnical Engineer should be retained to perform shoring design review.*

For yielding retaining walls, the active earth pressure coefficients, K_a, is recommended to be used. For non-yielding permanent walls, such as basement 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 1/3H above the base of the wall where H is the Height of the wall.

6.7.2 Dynamic Conditions

Below grade walls subjected to lateral forces due to seismic forces can be designed using the pseudo-static approach using the Mononobe-Okabe equations, shown in Section 24.9 of the Canadian Foundation Engineering Manual 2006 (CFEM-2006). In these formulas, there are both geotechnical and geometric components.

The total active thrust under seismic loading (P_{ae}) is recommended to be expressed as follows:

$$P_{ae} = ½\, \gamma H^2(1- k_v)K_{ae}$$

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, typically Peak Ground Acceleration (PGA) or a factor thereof is used.

The Site Class-adjusted NBCC-2010 PGA for the Site is 0.242 at Site Class A, at a probability of exceedance per annum of 0.000404. This value was determined using the NBCC-2010 Seismic Hazard Calculation document which is attached in Appendix E.

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 (P_a) and the passive earth pressure under static (P_p), respectively, as well as the increased force due to seismic forces. The active and passive forces under static conditions are assumed to act at a point of (0.3 x H) above the base and the seismic force is assumed to act near (0.6 x H) above the base, where H is the height of the wall. Therefore, the point of application for P_{ae} and P_{pe} may be calculated from the following equations:

$$h_a = [(0.33H.P_a) + (0.6H.P_e)]/P_{ae}$$

$$h_p = [(0.33H.P_p) + (0.6H.P_e)]/ P_{pe}$$

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

Table 6-2: Recommended Lateral Earth Pressure Coefficients under Dynamic Conditions

Soil	Bulk Density 'γ' (kN/m³) *	Angle of Internal Friction, ϕ' (degrees)	Undrained Shear Strength, Su (kPa)	Mononobe Okabe Earth Pressure Coefficients**	
				K_{ae}	K_{pe}
Existing Uncontrolled Cohesionless FILL Loose to compact	20	28	0	**0.56**	**1.80**

Soil	Bulk Density 'γ' (kN/m^3) *	Angle of Internal Friction, ϕ' (degrees)	Undrained Shear Strength, Su (kPa)	Mononobe Okabe Earth Pressure Coefficients**	
				K_{ae}	K_{pe}
New Compacted Granular Backfill OPSS "Granular B, Type II"	22	30	0	**0.52**	**1.91**

** Only the bulk unit weight is being presented, Designers will need to assess whether bulk, saturated, and/or submerged unit weights should be used based on their design conditions.*
***Assumes level/flat backfill surface. If Engineered Shoring is used, then Designers should refer to CFEM-2006 for design assistance and the Geotechnical Engineer should be retained to perform shoring design review.*

6.8 Floor Slabs

Based on the design traffic condition in the proposed underground parking lot, designers will need to decide what type of floor will be necessary in the parking garage. Typical options would be a flexible asphalt pavement, a rigid free-floating slab on grade, or alternatively a structural slab.

Englobe was not provided with any design criteria for floor slab loadings and traffic loadings for the floor slab of the underground parking garage, therefore we have 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 be a maximum value of 24 kPa. If larger slab loadings are envisioned, then Englobe should be retained to perform additional consulting in regard to design of the floor slab. 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 30 MPa/m on Engineered Fill and compacted to 100% of its Standard Proctor Maximum Dry density (SPMDD). Alternative values would require additional analysis and testing.

A capillary moisture barrier consisting of a layer of either 19 mm clear stone or an OPSS 1010 "Granular A" at least 200 mm thick should underlie the slab. This layer should be compacted to 100% of its SPMDD and placed on approved subgrade surfaces.

If floor coverings are to be used, vapour barriers are also recommended to be incorporated beneath the slab. Floor toppings may be impacted by curing and moisture conditions of the concrete. Floor finish manufacturer's specifications and requirements should be consulted, and procedures outlined in the specifications should be followed.

Subgrade preparation below floor slabs will involve the removal of all soils and weathered bedrock to expose an intact limestone bedrock. Any pieces of rock that can be easily manipulated by conventional excavation equipment should be removed, as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed bedrock surface should be examined and approved by the Geotechnical Engineer.

Any new fill used to raise the grade between the approved bedrock subgrade and the floor slab should be considered as Engineered Fill and should be placed in strict conformance with the requirements in Section 6.12.1.

6.9 Resistance of Foundation Uplift

Resistance to foundation uplift or overturning forces can be provided by considering the dead weight of the structures and backfill soils, increasing the dead weight of the structure using additional concrete elements, or with the use of additional rock anchors.

In the case that grouted rock anchors are considered, rock anchors may be designed based on a frictional stress between grout and intact bedrock. Based upon typical published values and conservative approach, Englobe recommends that a conservative allowable working stress value of 400 kPa be used to calculate the length of the required bond zone. The bond zone must be entirely within sound bedrock.

Designing in accordance with the Limit States Design (LSD) method, Designers may take the approach that working stress value is approximately equivalent to the SLS value. The ULS and SLS must be based upon both performance and structural criteria. However, based upon typical published values, the unfactored ULS values may be approximately 1,400 kPa to more than 2,100 kPa. As per CFEM-2006, a geotechnical resistance factor of Φ=0.3 should be applied to the empirical unfactored ULS values. Higher stress values may be available; however, performance load testing in the field will be required to prove the capacities. If performance testing is carried out at the outset of the Project, then a resistance factor of Φ=0.4 could be applied.

In order to mobilize the shear stress in the rock, the load at the top of the anchor must be properly transferred through the upper bedrock to the bond zone to prevent progressive grout fail and ensure proper performance. Therefore, a “free length” is required through the foundation element, the weathered rock zone, and down to the bond zone.

The mass of rock mobilized by a rock anchor may be assumed to be based upon a 60-degree cone drawn upward from a point located at the lower one-third point of the bond zone and spaced such that the theoretical cones do not overlap. Designers should review the spacing of anchors and take into account of any overlapping cones (i.e. avoid doubling-up on rock mass calculations for overlapping cones). The bulk unit weight of bedrock may be assumed to be approximately 26 kN/m^3. The corresponding buoyant unit weight would be approximately 16 kN/m^3. It is recommended that Designers consider the water level to be near the surface, and therefore, use submerged unit weights for the rock mass calculations.

6.10 Corrosion Potential of Soils

Analytical testing was carried out on two soil sample collected from the boreholes (BH22-02 and BH22-03) to determine corrosion potential of the subsurface soils. The selected soil samples were tested for pH, resistivity, chlorides, sulphides, sulphates and redox potential. The test results are summarized in the following table and presented in Appendix D.

Table 6-3: Corrosion Parameter Results

Parameter	Tested Value	
	BH22-02, SS2	MW21-02, SS3A/SS4
pH	6.97	7.33 (SS4)
Chloroide (%)	0.0007	0.0011 (SS4)
Sulphate (%)	0.0291	0.0048 (SS4)
Resistivity (Ohm-cm)	2200	4600 (SS4)
Sulphide (%)	< 0.04	< 0.04 (SS3A+SS4)
Redox Potential (mV)	375*	370 (SS3A+SS4)

*Sample holding time was exceeded prior to analysis

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 not considered to be moderately corrosive to ductile iron pipe.

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) A23.1-04 and are given in Table 6-4 below.

Table 6-4: 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	0.20 - 2.0	HS or HSb
S-3	Moderate	0.10 - 0.20	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for selected soil samples tested indicate a sulphate concentration of maximum of a 0.0291% in soil, as shown in Table 6-3. indicating a “moderate” risk for sulphate attack on concrete material.

6.11 Waterproofing and Permanent Drainage

Under floor drainage is recommended for this structure based on groundwater level which is above the basement floor slab. The building basement can be designed as a fully waterproof ‘bath-tub’ design (without external perimeter drains to avoid potential adverse impacts due to moisture movements in the immediate areas around the proposed building footprint.

Full water proofing membranes such as a WR Meadows Mel-ROL PRECON or equivalent type product for walls and under-slab will be required. These types of membranes adhere to the concrete and provide a waterproof seal between the membrane and poured concrete. Their installation would require that excavations be planned large enough for safe worker accesses on the exterior of the foundation wall to allow installation. Water stops should be installed at cold joints in the foundation walls and floor-wall joint.

Under floor drainage systems should be placed at a minimum 4.5 m spacing between drains, running in one direction, and set at a minimum of 0.45 m below the underside of floor slabs.

6.12 Backfill

All new fill soils that underlie floor slabs, footings, in building interiors, or other structural applications are considered as Engineered Fill and must be treated as follows:

6.12.1 Engineered Fill

All new fill soils that underlie floor slabs, footing, or other structural applications is considered as Engineered Fill. For this Project, Engineered Fill may be required to raise the grade between the approved intact bedrock subgrade and floor slabs. Engineered Fill must meet the strict requirements as shown below:

- The proposed material must be tested for grain size and Proctor and reviewed and approved by the Geotechnical Engineer before being considered as Engineered Fill. Typically, a crushed well-graded material such as an OPSS 1010 “Granular A” or “Granular B Type II” type material is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions;
- Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade approved by the Engineer. Any deficient areas should be repaired prior to placement;
- Engineered Fill should 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 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. Designers and contractors are cautioned that the resultant excavation can be quite large if a significant thickness of Engineered Fill is required.

6.12.2 Exterior Foundation Wall Backfill

The backfill placed against exterior foundations should be a free draining granular material meeting the grading requirements of an OPSS 1010 "Granular B, Type I" or equivalent granular material. Exterior foundation backfill should be placed and compacted as outlined below:

- Backfill should not be placed in a frozen condition, or place 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;
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures, or alternatively wait until basement wall are tied together with the floor above before backfilling the exterior foundation wall;
- 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 a pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

6.12.3 Infiltration Chamber Backfill

It is Englobe's understanding that a proposed low impact development (LID) underground infiltration chamber will be installed in the southwest corner of the Subject Property west of Building B and south of Building A. The design of the infiltration chamber should be reviewed by a Geotechnical Engineer once drawings are available to ensure that the design conforms to the following recommendations:

- In areas where the building backfill underlies a pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.
- The invert depth of the infiltration facility should be founded with a minimum of 1.8 m of earth cover or the equivalent insulation detail installation in order to provide adequate protection against frost action.
- Typically, a minimum horizontal separation of 4.0 m should be maintained between infiltration facilities and building foundations. As the current separation of 1.9 m between the infiltration facility and the structure foundation does not meet the recommended minimum setback, clay backfill material should be incorporated into the exterior foundation backfill design between the infiltration facility and the structure foundation. This will create an impervious barrier preventing channelization of groundwater into the footprint of the proposed structure. Acceptable imported clay material may be used for the construction of the impervious barrier.
- Toronto and Region Conservation Authority (TRCA) and Credit Valley Conservation (CVC) guidelines suggest that a 1.0 m vertical separation be maintained between the bottom of an LID practice and the seasonal high groundwater table or bedrock surface. The groundwater level measured in MW22-04 on June 03, 2022 was observed to be dry, and was measured at 1.9 mbgs (EL. 86.7 masl) on August 16, 2023. Based on the current proposed design of the infiltration chamber, less than 1.0 m of separation may exist between the invert level of underground infiltration tank and the seasonal high groundwater table.

It is important to emphasize that groundwater levels are provided only as a general note to assist Designers. Infiltration testing and LID design are outside Englobe's scope of work at the time of this geotechnical investigation and Englobe has not been retained to conducted a hydrogeological investigation in support of LID design.

6.13 Underground Utilities

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

6.13.1 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 placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed;
- 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 pertinent specification section;
- The bedding, springline, and cover should be compacted to at least 98% of its SPMDD; and
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.13.2 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 98% of its SPMDD;
- The backfill placed in the upper 0.3 m below the pavement subgrade elevation should be compacted to a minimum of 100% of its SPMDD;
- Excavation backfill should attempt to match texture of the existing adjacent soils. If imported materials are used, side slopes with frost tapers are recommended. Frost tapers should be a back-slope of 10H:1V through the frost zone, (i.e.,1.8 m from finished grade);
- 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 is 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.

6.13.3 Clay Seals

Clay seals should be incorporated into the design of the any utility trenches. If clay seals are not used, then there is the potential for the trench to act as a drain and direct water into the proposed building footprint. The location of the clay seals should be at a frequency prescribed by the Civil Engineer, and at the property lines.

Ontario Provincial Standard Specifications (OPSS) 1205 and Drawings (OPSD) 802.095 are referred to both the Designers and Contractor for guidance on clay seals. Acceptable imported clay material may be used for the construction of the clay seals.

6.13.4 Ottawa Interceptor Outfall Sewer

Englobe, formerly DST, performed a geotechnical investigation for subdivision approval at the former CFB Rockliffe, Ottawa, Ontario dated September 2015. This study included preliminary evaluation of the influence of new subdivision developments on the existing sewer line considering the depth of the Ottawa Interceptor Outfall sewer pipe and assuming over 40 m thick competent rock cover, the anticipated rock RQD values, typical rock mass rating (RMR) determination, and existing/limited unconfined Compressive Strength (UCS) test results. Under these assumptions, the study concluded that the effect of the increase of the stress on the top of the bedrock due to grade raise and development can be considered negligible. The study suggested that the minimum bedrock crown to be maintain is 30 m or more between any intrusive work into the bedrock and the top of the sewer.

The study recommended that construction activities that will induce vibrations such as blasting will require vibration monitoring plan prepared by a professional engineer to ensure the integrity of the sewer is maintained during the construction activities. In addition, a pre-construction survey of the outfall sewer should be undertaken prior to the start of construction activities.

7

7 Monitoring During Construction

Englobe requests to be retained once the plans and specifications are finalized to review the documents and ensure the recommendations in this report are adequately addressed.

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. Based on our understanding of the scope of the Project, an adequate level of construction monitoring is as follows:

- Review and approval of all footing subgrades by geotechnical personnel prior to placement of lean concrete mud slabs;
- Confirmation of bedrock quality during construction using 1.5 m probe holes within the footings. These holes will need to be reviewed by the Geotechnical Engineer to confirm that no significant mud seams or voids exist;
- Review and approval of subgrades below the floor slab, prior to placement of lean concrete mud slabs;
- Laboratory testing and pre-approval of fill soils that are proposed to be used on Site;
- Full time compaction testing of Engineered Fill and part time compaction testing of exterior foundation wall backfill;
- Periodic testing of concrete;
- Vibration and settlement monitoring of adjacent Structures;

- Visual review of waterproofing membranes.

An important purpose of providing an adequate level of monitoring is to check that recommendations, based on data obtained at the discrete borehole locations, are relevant to other areas of the Site.

8 Closure

A description of limitations which are inherent in carrying out Site investigation studies is given in Appendix A and forms an integral part of this report.

We trust this report meets your present requirements. Should you have any questions, please do not hesitate to contact our office.

Appendix A
Limitations of Report

LIMITATIONS OF REPORT
GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that Englobe Consulting Engineers Inc. be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the boreholes.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid. Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others and Englobe Corp. cannot warranty their accuracy. Similarly, Englobe cannot warranty the accuracy of information supplied by the Client.

Appendix B
- Figure 1: Site Location Map
- Figure 2: Borehole Location Plan

Scale accurate when printed at 100% using paper size ANSI full bleed A (8.5 x 11.0 Inches)

Source:
Google Earth 2022

Note

1. This drawing shall be read in conjunction with the associated technical report.

Revision	Date	Issue	Approval
A	2022/07/08	Preliminary	

Client
WestUrban Developments Ltd.

Site
1050 Tawadina Road, Ottawa, ON

Report Title
Geotechnical Investigation

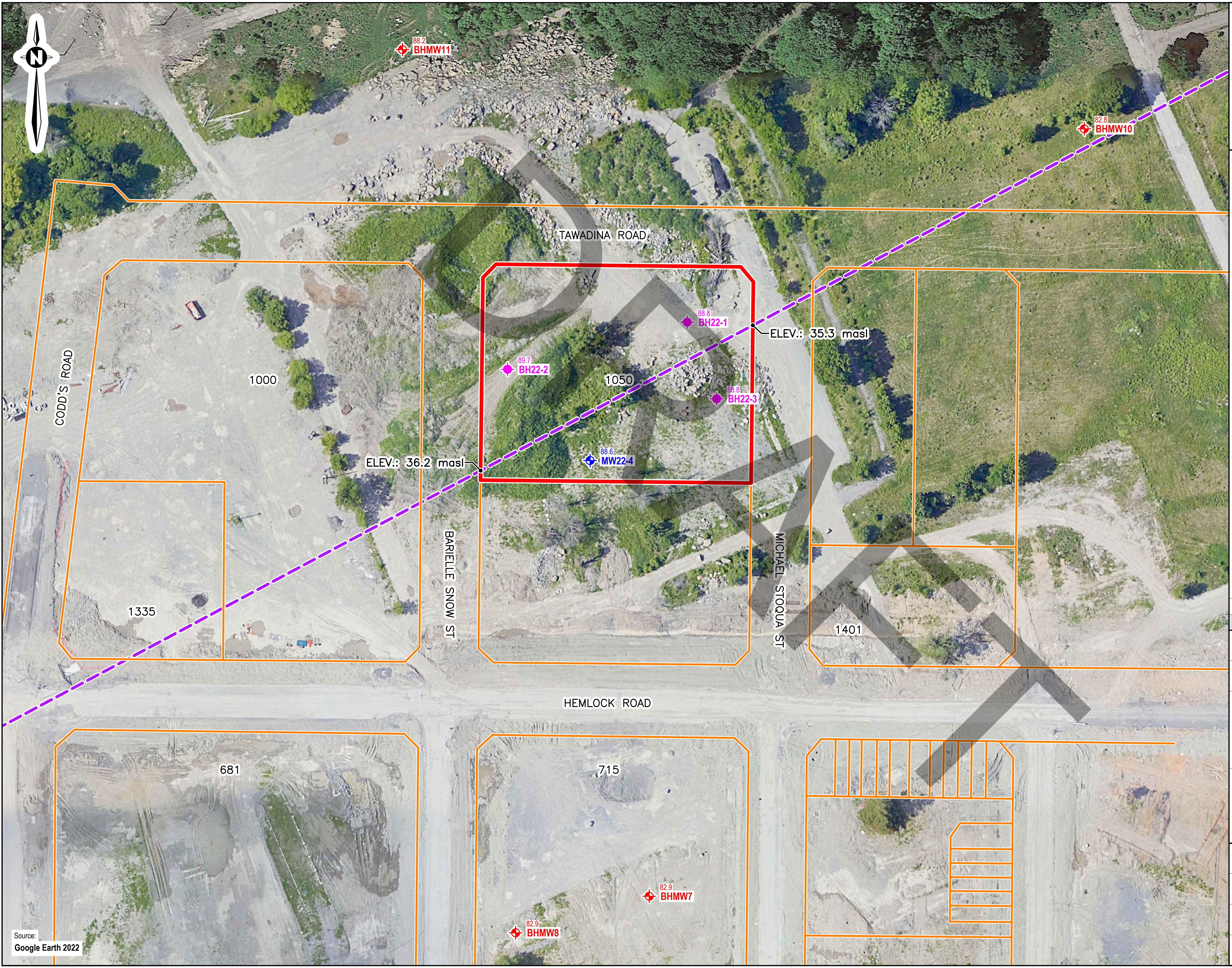
Drawing Title
Site Location Map

Designed By MA	Date July 2022
Drawn By JM	Project No. 02203079.000
Approved By	Figure No. 1
Scale As shown	

Friday, July 08, 2022 @ 13:33 by Joven Mendoza

Folder: D:\Joven OneDrive\OneDrive - EnGlobe Corp\Work\02203079 Tawadina Rd\Geotechnical Investigation\DWGs

Drawing: 1 Site Location.dwg



Source:
Google Earth 2022

ENGLOBE

Note

1. This drawing shall be read in conjunction with the associated technical report.

Legend

🟥	Approximate Site Boundary
🟧	Parcel / Future Development
▬ ▬	2400 Ø mm Ottawa Interceptor Outfall Sewer Pipe
⌖ 88.8	Borehole Location and Ground Elevation
⌖ 88.6	Monitoring Well Location and Ground Elevation
⌖ 82.8	Monitoring Well Location (DST, 2006)

1:1 250
0 10 20 30 40 50 m
0 1:1 5 cm
Scale accurate when printed at 100% using paper size ANSI full bleed B (11.0 x 17.0 Inches)

Revision	Date	Issue	Approval
A	2022/07/08	Preliminary	

Client
WestUrban Developments Ltd.

Site
1050 Tawadina Road, Ottawa, ON

Report Title
Geotechnical Investigation

Drawing Title
Site Plan

Designed By	MA	Scale	As shown
Drawn By	JM	Date	July 2022
Approved By		Project No.	02203079.000
Figure No.	2		

Drawing: 2 Site Plan.dwg Folder: D:\Joven OneDrive\OneDrive - EnGlobe Corp\Work\02203079 Tawadina Rd\Geotechnical Investigation\DWGs Friday, July 08, 2022 @ 13:33 by Joven Mendoza

Appendix C

- List of Symbols and Definitions
- Englobe Borehole Logs and Fence Diagram
- DST 2006 Monitoring Well Records
- Rock Core Photographs
- Sieve Gradation Analysis Results

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LIST OF SYMBOLS AND DEFINITIONS FOR GEOTECHNICAL SAMPLING AND COMMON LITHOLOGIES

The following is a reference sheet for commonly used symbols and definitions within this report and in any figures or appendices, including borehole logs and test results. Symbols and definitions conform to the standard proposed by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) wherever possible. Discrepancies may exist when comparing to third-party results using the Unified Soil Classification System (USCS).

PART A – SOILS

Standard Penetration Test (SPT) 'N'

The number of blows required to drive a 50-mm (2 in) split barrel sampler 300 mm (12 in). The standard hammer has a mass of 63.5 kg (140 lbs) and is dropped vertically from a height of 760 mm (30 in). Additional information can be found in ASTM D1586-11 and in §4.5.2 of the CFEM 4th Ed.

For penetration less than 300 mm, 'N' is recorded with the penetration that was achieved.

Non-Cohesive Soils

The relative density of non-cohesive soils relates empirically to SPT 'N' as follows:

Relative Density	'N'
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	> 50

Cohesive Soils

The consistency and undrained shear strength of cohesive soils relates empirically to SPT 'N' as follows:

Consistency	Undrained Shear Strength (kPa)	'N'
Very Soft	< 12	0 – 2
Soft	12 – 25	2 – 4
Firm	25 – 50	4 – 8
Stiff	50 – 100	8 – 15
Very Stiff	100 – 200	15 – 30
Hard	> 200	> 30

PART B – ROCK

The following parameters are used to describe core recovery and to infer the quality of a rockmass.

Total Core Recovery, TCR (%)

The total length of solid drill core recovered, regardless of the quality or length of the pieces, taken as a percentage of the length of the core run.

Solid Core Recovery, SCR (%)

The total length of solid, full-diameter drill core recovered, taken as a percentage of the length of the core run.

Rock Quality Designation, RQD (%)

The sum of the lengths of solid drill core greater than 100 mm long, taken as a percentage of the length of the core run. RQD is commonly used to infer the quality of the rockmass, as follows:

Rockmass Quality	RQD (%)
Very Poor	< 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	> 90

Weathering

The terminology used to describe the degree of weathering for recovered rock core is defined as follows, as suggested by the *Geological Society of London*:

Completely weathered: All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.

Highly weathered: More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.

Moderately weathered: Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present ether as a continuous framework or as core stone.

Slightly weathered: Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.

Fresh: No visible signs of weathering.

PART C – SAMPLING SYMBOLS

Symbol	Description
SS	Split spoon sample
TW	Thin-walled (Shelby Tube) sample
PH	Sampler advanced by hydraulic pressure
WH	Sampler advanced by static weight
SC	Soil core

PART D – IN-SITU AND LAB TESTING

SOIL NAMING CONVENTIONS

Particle sizes are described as follows:

Particle Size Descriptor		Size (mm)
Boulder		> 300
Cobble		75 – 300
Gravel	Coarse	19 – 75
	Fine	4.75 – 19
Sand	Coarse	2.0 – 4.75
	Medium	0.425 – 2.0
	Fine	0.075 – 0425
Silt		0.002 – 0.075
Clay		< 0.002

The principle constituent of a soil is written in uppercase. The minor constituents of a soil are written according to the following convention:

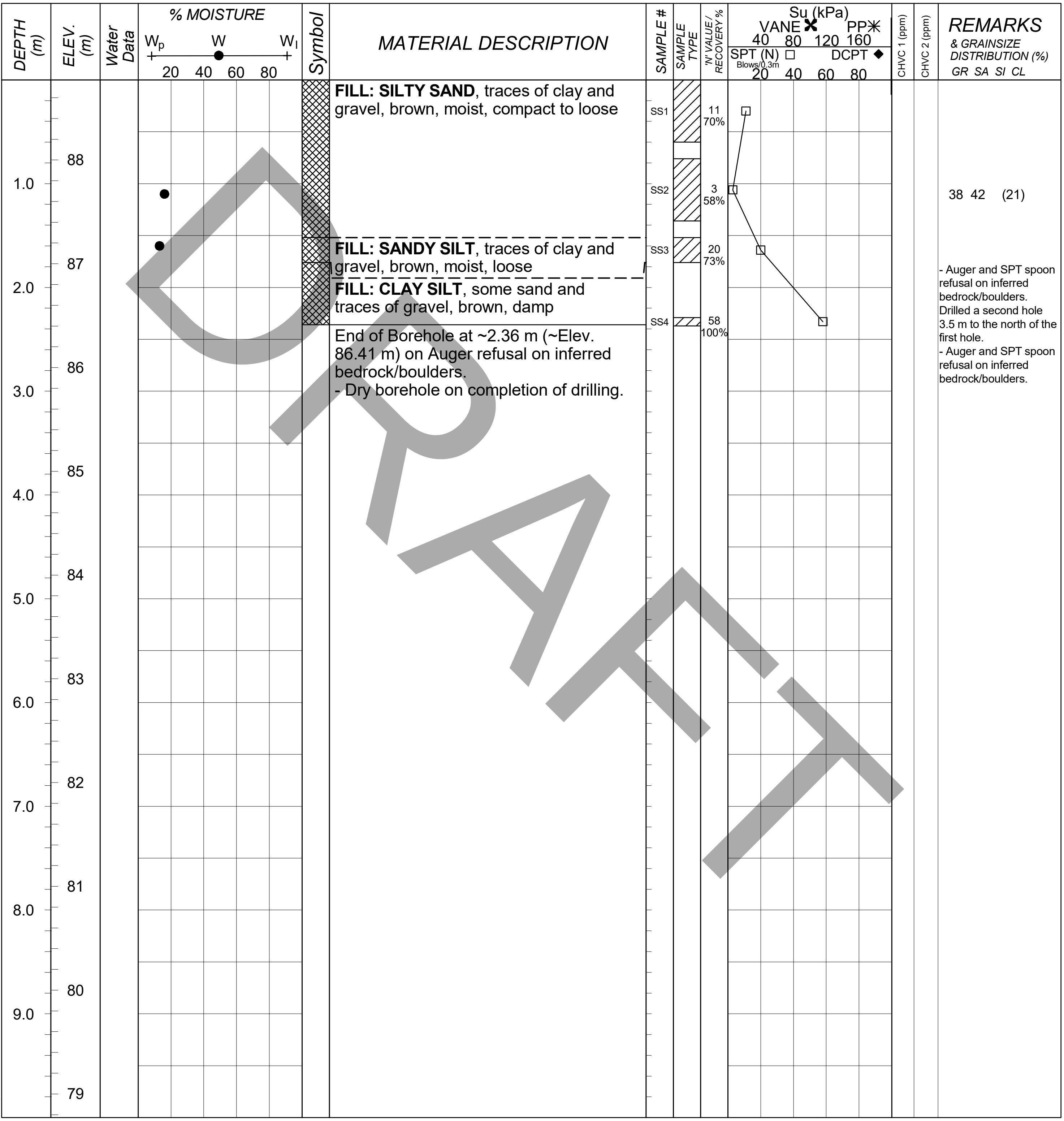
Descriptive Term	Proportion of Soil (%)
Trace	1 – 10
Some	10 – 20
(ey) or (y)	20 – 35
And	35 – 50

Eg.: A soil comprising 65% Silt, 21% Sand and 14% Clay would be described as a: Sandy SILT, Some Clay

LOG OF BOREHOLE BH22-1

DST REF. No.: **02203079.000**
CLIENT: **WestUrban Developments Ltd.**
PROJECT: **Two 6-Storey Apartment Buildings-1050 Tawadina Rd, Ottawa**
LOCATION: **1050 Tawadian Rd, Ottawa**
SURFACE ELEV.: **88.77 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **05/04/2022**
COMPLETION DATE: **5/4/2022**
COORDINATES: **5033601.535** m N, **450646.901** m E



FILL: SILTY SAND, traces of clay and gravel, brown, moist, compact to loose

FILL: SANDY SILT, traces of clay and gravel, brown, moist, loose

FILL: CLAY SILT, some sand and traces of gravel, brown, damp

End of Borehole at ~2.36 m (~Elev. 86.41 m) on Auger refusal on inferred bedrock/boulders.
- Dry borehole on completion of drilling.

SAMPLE #	'N' VALUE / RECOVERY %	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	11 70%	
SS2	3 58%	38 42 (21)
SS3	20 73%	
SS4	58 100%	

- Auger and SPT spoon refusal on inferred bedrock/boulders. Drilled a second hole 3.5 m to the north of the first hole.
- Auger and SPT spoon refusal on inferred bedrock/boulders.

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SAMPLE TYPE LEGEND

- ☰ Auger Sample
- ▨ Split Spoon Sample
- ▥ Bulk Sample
- ▩ Rock Core
- ▥ Core Sample
- ◫ Shelby Tube

WELL LEGEND

- ■ Bentonite
- ▦ Sand
- ▦ Screen

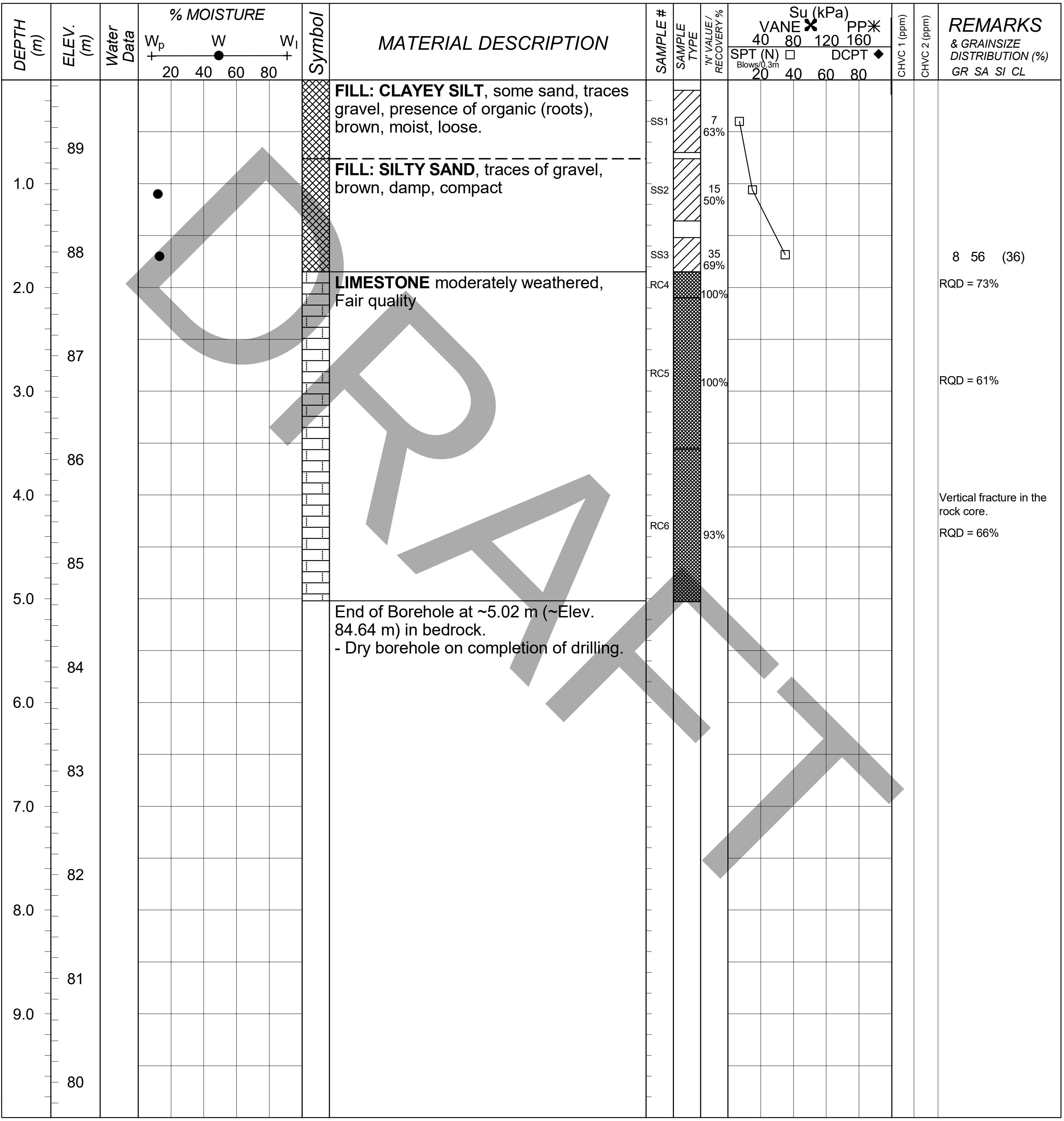
$\times^3$ Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 1

LOG OF BOREHOLE BH22-2

DST REF. No.: **02203079.000**
CLIENT: **WestUrban Developments Ltd.**
PROJECT: **Two 6-Storey Apartment Buildings-1050 Tawadina Rd, Ottawa**
LOCATION: **1050 Tawadian Rd, Ottawa**
SURFACE ELEV.: **89.66 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **05/04/2022**
COMPLETION DATE: **5/4/2022**
COORDINATES: **5033585.527** m N, **450586.063** m E



MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE / RECOVERY %	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
FILL: CLAYEY SILT, some sand, traces gravel, presence of organic (roots), brown, moist, loose.	SS1	▨	7 63%	
FILL: SILTY SAND, traces of gravel, brown, damp, compact	SS2	▨	15 50%	
	SS3	▨	35 69%	8 56 (36)
LIMESTONE moderately weathered, Fair quality	RC4	▩	100%	RQD = 73%
	RC5	▩	100%	RQD = 61%
	RC6	▩	93%	Vertical fracture in the rock core. RQD = 66%
End of Borehole at ~5.02 m (~Elev. 84.64 m) in bedrock. - Dry borehole on completion of drilling.				

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SAMPLE TYPE LEGEND

☰	Auger Sample	▩	Rock Core
▨	Split Spoon Sample	▥	Core Sample
▯	Bulk Sample	◫	Shelby Tube

WELL LEGEND

■	Bentonite
▦	Sand
▦	Screen

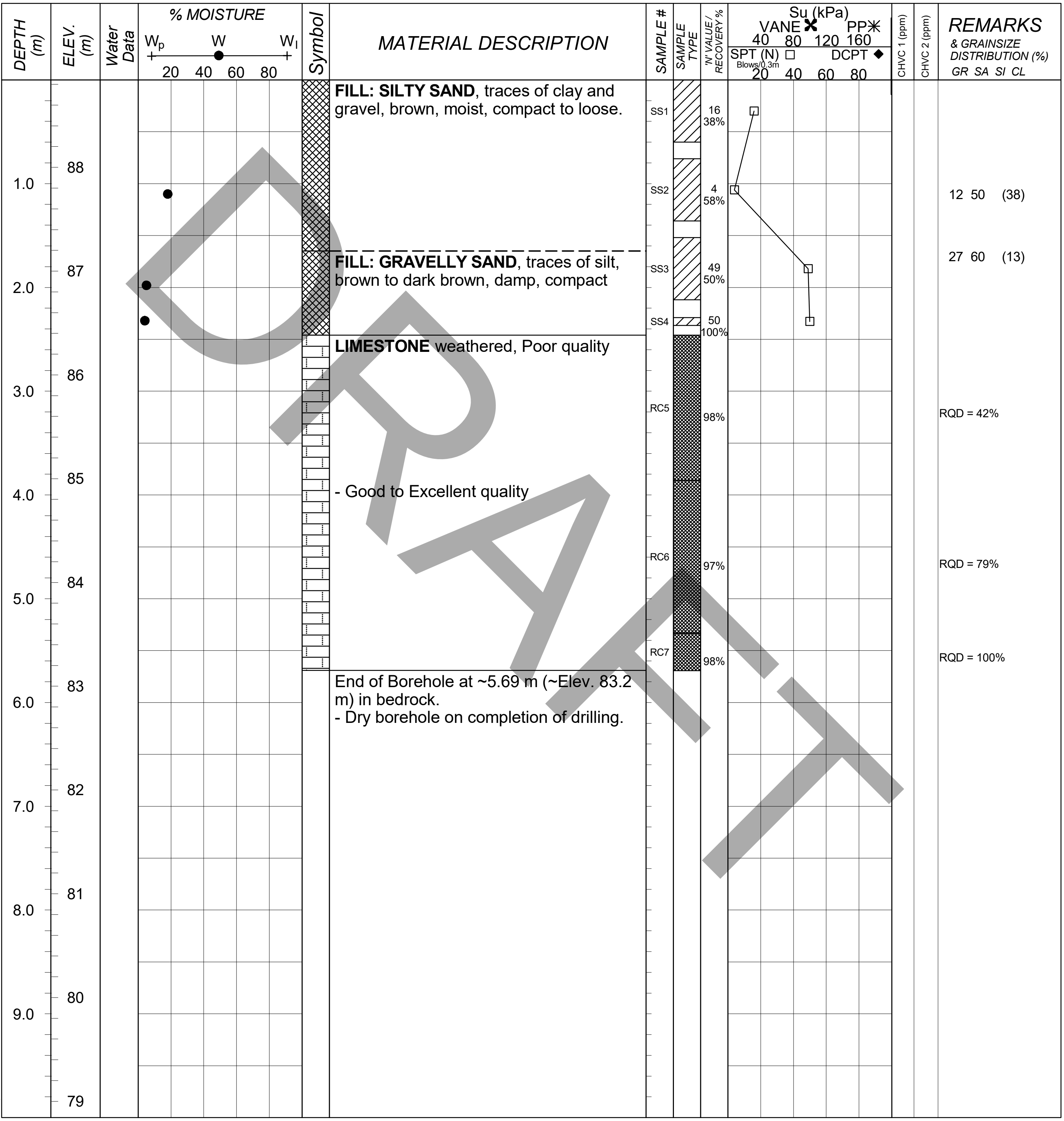
$\times^3$ Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 2

LOG OF BOREHOLE BH22-3

DST REF. No.: **02203079.000**
CLIENT: **WestUrban Developments Ltd.**
PROJECT: **Two 6-Storey Apartment Buildings-1050 Tawadina Rd, Ottawa**
LOCATION: **1050 Tawadian Rd, Ottawa**
SURFACE ELEV.: **88.84 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **05/04/2022**
COMPLETION DATE: **5/4/2022**
COORDINATES: **5033576.327** m N, **450656.556** m E



FILL: SILTY SAND, traces of clay and gravel, brown, moist, compact to loose.

FILL: GRAVELLY SAND, traces of silt, brown to dark brown, damp, compact

LIMESTONE weathered, Poor quality

- Good to Excellent quality

End of Borehole at ~5.69 m (~Elev. 83.2 m) in bedrock.
- Dry borehole on completion of drilling.

SAMPLE #	'N' VALUE / RECOVERY %	REMARKS & GRAINSIZE DISTRIBUTION (%)
SS1	16 38%	
SS2	4 58%	12 50 (38)
SS3	49 50%	27 60 (13)
SS4	50 100%	
RC5	98%	RQD = 42%
RC6	97%	RQD = 79%
RC7	98%	RQD = 100%

SAMPLE TYPE LEGEND

▤ Auger Sample	▩ Rock Core
▨ Split Spoon Sample	▥ Core Sample
▥ Bulk Sample	▯ Shelby Tube

WELL LEGEND

- ■ Bentonite
- ▦ Sand
- ▦ Screen

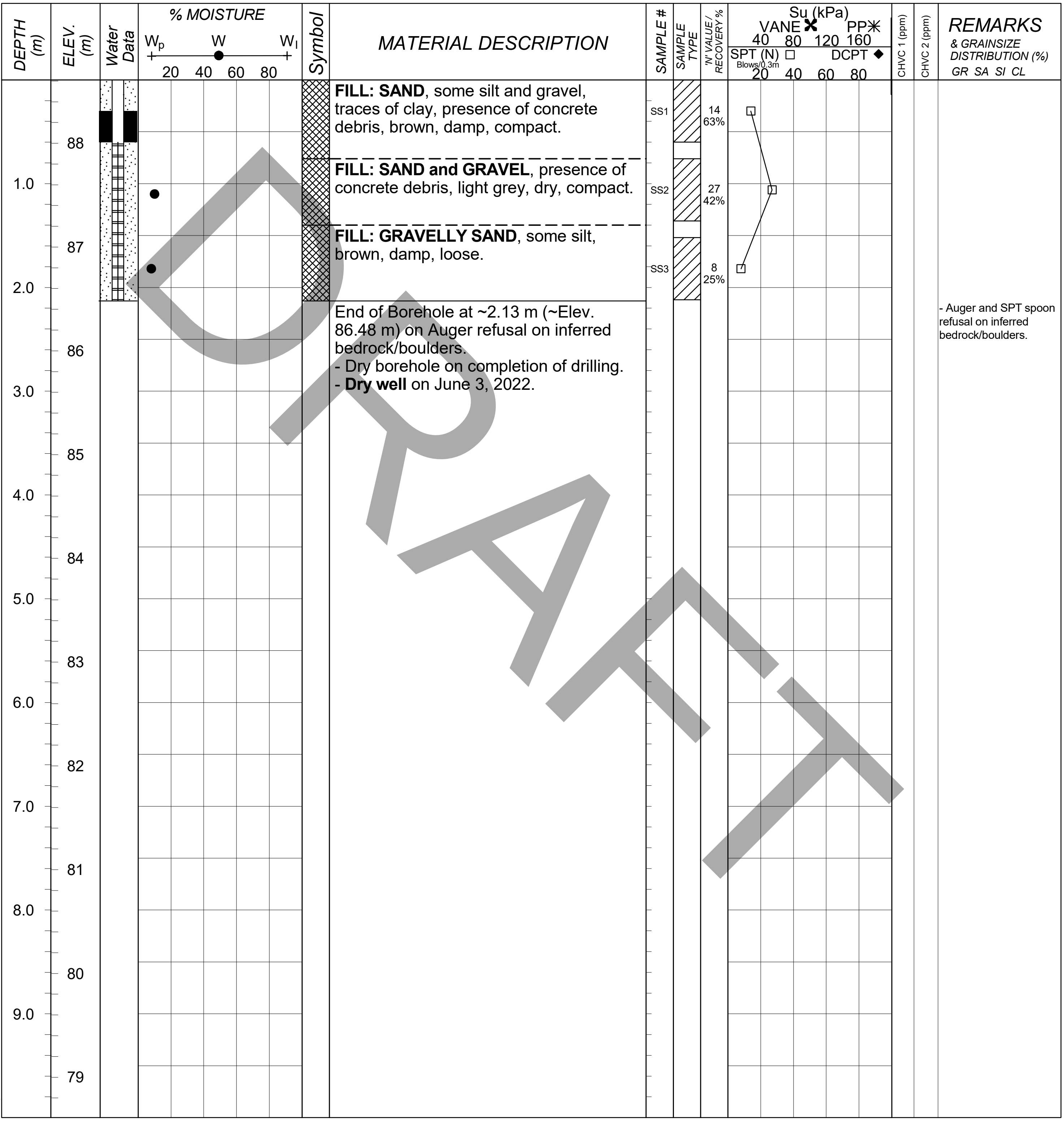
✕^{3} Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 3

LOG OF BOREHOLE MW22-4

DST REF. No.: **02203079.000**
CLIENT: **WestUrban Developments Ltd.**
PROJECT: **Two 6-Storey Apartment Buildings-1050 Tawadina Rd, Ottawa**
LOCATION: **1050 Tawadian Rd, Ottawa**
SURFACE ELEV.: **88.61 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **05/04/2022**
COMPLETION DATE: **5/4/2022**
COORDINATES: **5033555.743** m N, **450613.958** m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE / RECOVERY %	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
FILL: SAND, some silt and gravel, traces of clay, presence of concrete debris, brown, damp, compact.	SS1	14 63%	
FILL: SAND and GRAVEL, presence of concrete debris, light grey, dry, compact.	SS2	27 42%	
FILL: GRAVELLY SAND, some silt, brown, damp, loose.	SS3	8 25%	
End of Borehole at ~2.13 m (~Elev. 86.48 m) on Auger refusal on inferred bedrock/boulders. - Dry borehole on completion of drilling. - **Dry well** on June 3, 2022.			- Auger and SPT spoon refusal on inferred bedrock/boulders.

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SAMPLE TYPE LEGEND

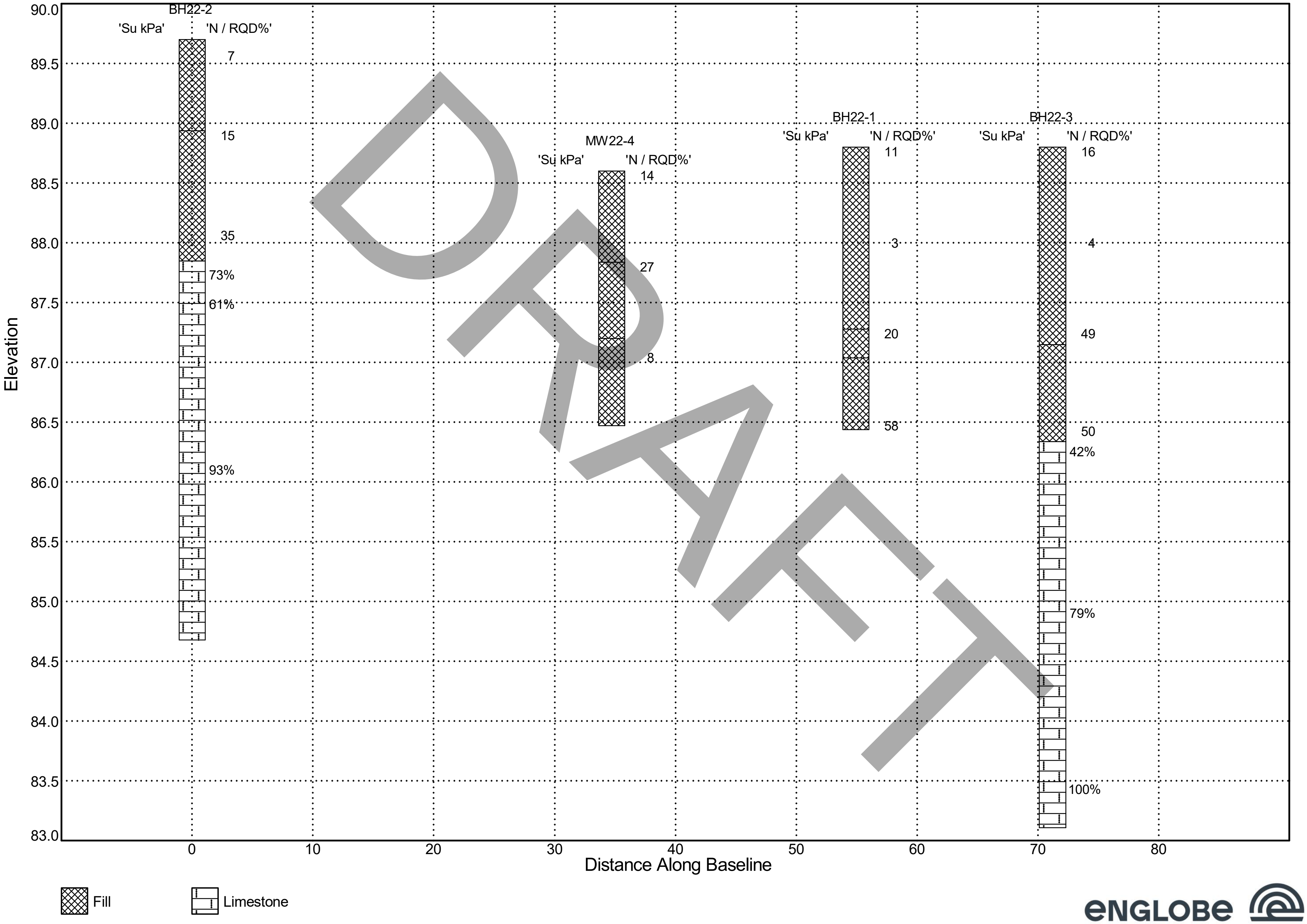
☰ Auger Sample	▩ Rock Core
▨ Split Spoon Sample	▥ Core Sample
▯ Bulk Sample	◫ Shelby Tube

WELL LEGEND

- ■ Bentonite
- ▤ Sand
- ▦ Screen

$\times^3$ Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 4


BH22-2
'Su kPa'
'N / RQD%'
7
15
35
73%
61%
93%
MW22-4
'Su kPa'
'N / RQD%'
14
27
8
BH22-1
'Su kPa'
'N / RQD%'
11
3
20
58
BH22-3
'Su kPa'
'N / RQD%'
16
4
49
50
42%
79%
100%
Elevation
90.0
89.5
89.0
88.5
88.0
87.5
87.0
86.5
86.0
85.5
85.0
84.5
84.0
83.5
83.0
Distance Along Baseline
0
10
20
30
40
50
60
70
80
▨ Fill
Limestone

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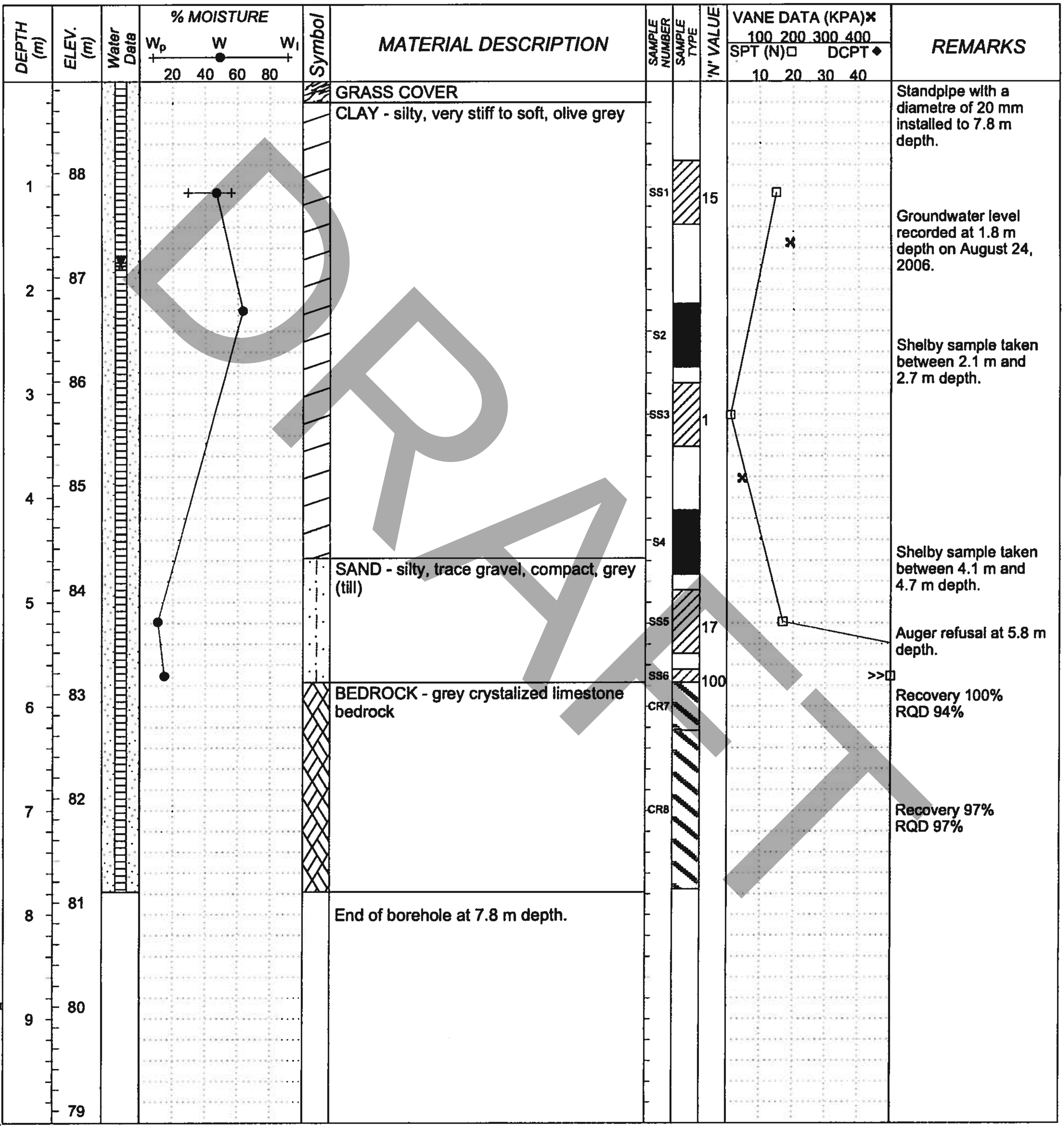
LOG OF BOREHOLE BH7

DST REF. No.: **OG06562**
CLIENT: **Canada Lands Company**
PROJECT: **Preliminary Geotechnical Investigation**
LOCATION: **CFB Rockcliffe, Ottawa, Ontario**
SURFACE ELEV.: **88.88 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **August 14 2006**



GRASS COVER

CLAY - silty, very stiff to soft, olive grey

SAND - silty, trace gravel, compact, grey (till)

BEDROCK - grey crystalized limestone bedrock

End of borehole at 7.8 m depth.

Standpipe with a diametre of 20 mm installed to 7.8 m depth.

Groundwater level recorded at 1.8 m depth on August 24, 2006.

Shelby sample taken between 2.1 m and 2.7 m depth.

Shelby sample taken between 4.1 m and 4.7 m depth.

Auger refusal at 5.8 m depth.

Recovery 100%
RQD 94%

Recovery 97%
RQD 97%

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SAMPLE TYPE LEGEND

☰	Auger Sample	■	Rock Core	▥	Ponar Sample
▨	Split Spoon Sample	▩	Side Sampler		
☐	Thin Wall Tube	☐	Grab Sample		

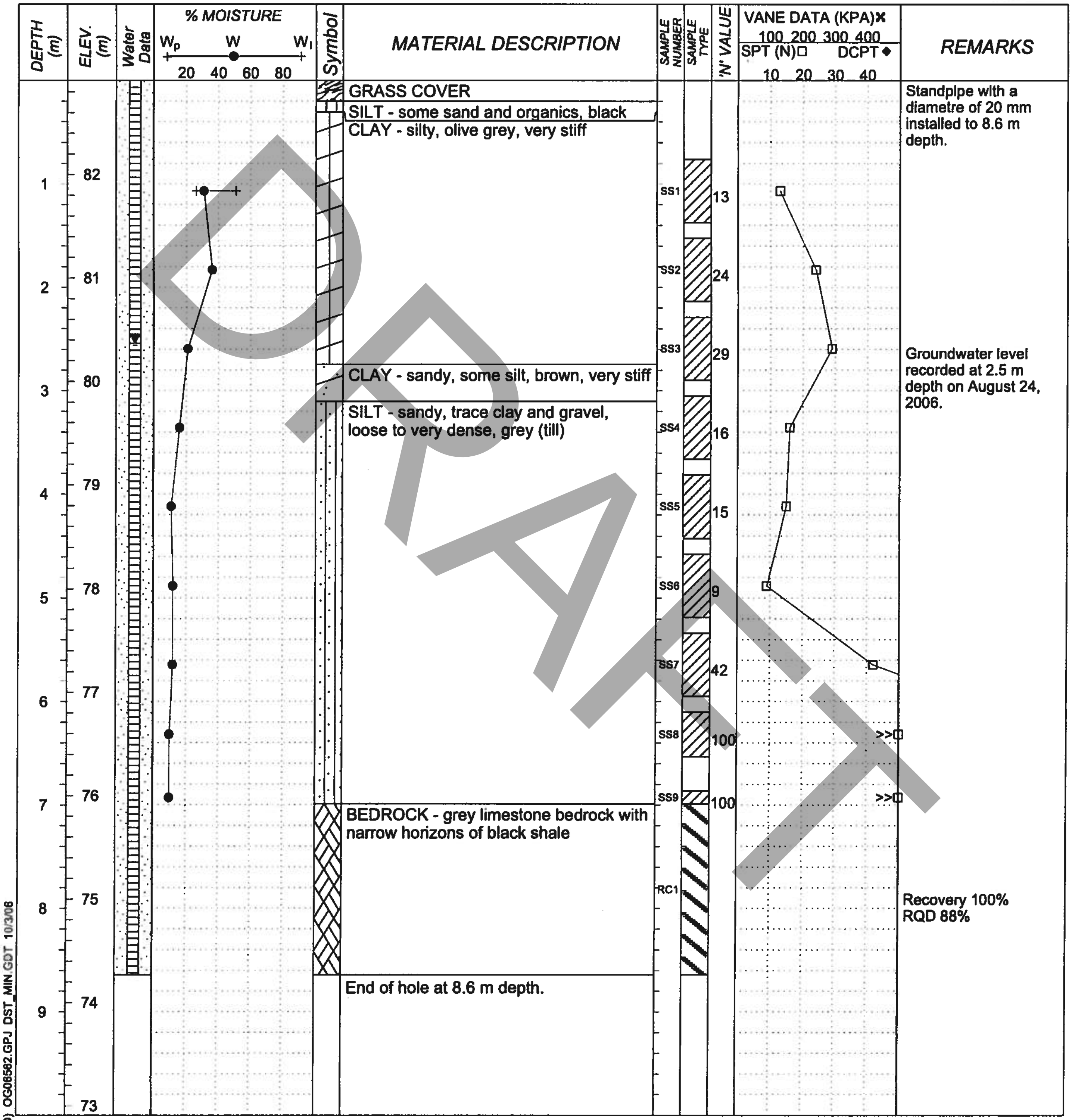
APPENDIX D

LOG OF BOREHOLE BH8

DST REF. No.: **OG06562**
CLIENT: **Canada Lands Company**
PROJECT: **Preliminary Geotechnical Investigation**
LOCATION: **CFB Rockcliffe, Ottawa, Ontario**
SURFACE ELEV.: **82.91 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **August 11 2006**



MATERIAL DESCRIPTION

GRASS COVER

SILT - some sand and organics, black

CLAY - silty, olive grey, very stiff

CLAY - sandy, some silt, brown, very stiff

SILT - sandy, trace clay and gravel, loose to very dense, grey (till)

BEDROCK - grey limestone bedrock with narrow horizons of black shale

End of hole at 8.6 m depth.

REMARKS

Standpipe with a diameter of 20 mm installed to 8.6 m depth.

Groundwater level recorded at 2.5 m depth on August 24, 2006.

Recovery 100%
RQD 88%

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Web: www.dstgroup.com

SAMPLE TYPE LEGEND

- ▤ Auger Sample
- ▨ Split Spoon Sample
- ☐ Thin Wall Tube
- ■ Rock Core
- ▧ Side Sampler
- ☐ Grab Sample
- ▥ Ponar Sample

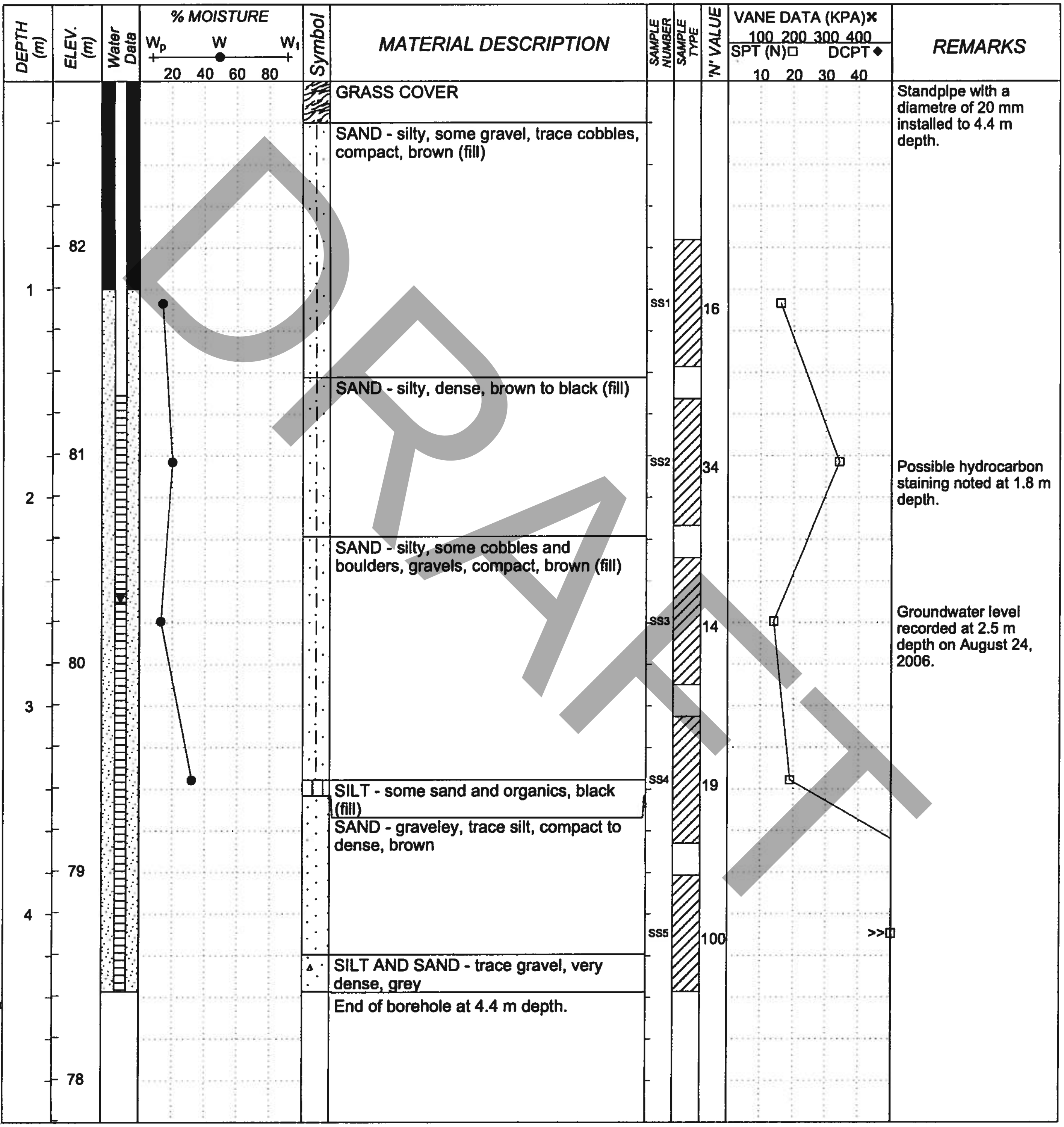
APPENDIX D

LOG OF BOREHOLE / MONITORING WELL BHMW10 OB

DST REF. No.: **OG06562**
CLIENT: **Canada Lands Company**
PROJECT: **Preliminary Geotechnical Investigation**
LOCATION: **CFB Rockcliffe, Ottawa, Ontario**
SURFACE ELEV.: **82.79 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **August 10 2006**



Material description	Sample number	'N' value
GRASS COVER		
SAND - silty, some gravel, trace cobbles, compact, brown (fill)	SS1	16
SAND - silty, dense, brown to black (fill)	SS2	34
SAND - silty, some cobbles and boulders, gravels, compact, brown (fill)	SS3	14
SILT - some sand and organics, black (fill)	SS4	19
SAND - graveley, trace silt, compact to dense, brown	SS5	100
SILT AND SAND - trace gravel, very dense, grey		
End of borehole at 4.4 m depth.		

REMARKS

Standpipe with a diametre of 20 mm installed to 4.4 m depth.

Possible hydrocarbon staining noted at 1.8 m depth.

Groundwater level recorded at 2.5 m depth on August 24, 2006.

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SAMPLE TYPE LEGEND

- ☰ Auger Sample
- ▨ Split Spoon Sample
- ☐ Thin Wall Tube
- ■ Rock Core
- ▩ Side Sampler
- ▽ Grab Sample
- ▥ Ponar Sample

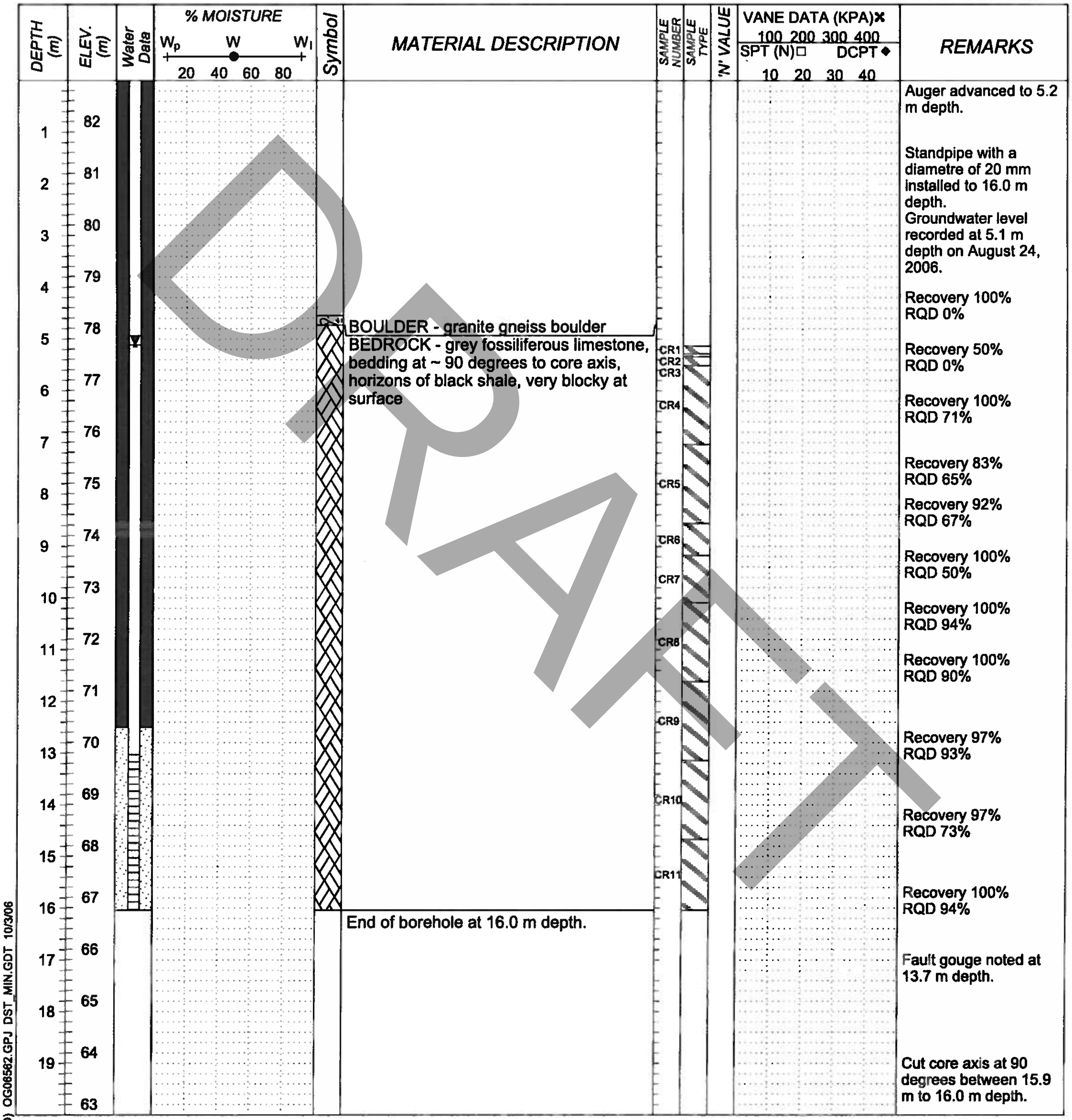
APPENDIX D

LOG OF BOREHOLE / MONITORING WELL BHMW10 BR

DST REF. No.: **OG06562**
CLIENT: **Canada Lands Company**
PROJECT: **Preliminary Geotechnical Investigation**
LOCATION: **CFB Rockcliffe, Ottawa, Ontario**
SURFACE ELEV.: **82.79 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **August 10 2006**



MATERIAL DESCRIPTION

BOULDER - granite gneiss boulder

BEDROCK - grey fossiliferous limestone, bedding at ~ 90 degrees to core axis, horizons of black shale, very blocky at surface

End of borehole at 16.0 m depth.

REMARKS

Auger advanced to 5.2 m depth.

Standpipe with a diametre of 20 mm installed to 16.0 m depth.
Groundwater level recorded at 5.1 m depth on August 24, 2006.

Recovery 100%
RQD 0%

Recovery 50%
RQD 0%

Recovery 100%
RQD 71%

Recovery 83%
RQD 65%

Recovery 92%
RQD 67%

Recovery 100%
RQD 50%

Recovery 100%
RQD 94%

Recovery 100%
RQD 90%

Recovery 97%
RQD 93%

Recovery 97%
RQD 73%

Recovery 100%
RQD 94%

Fault gouge noted at 13.7 m depth.

Cut core axis at 90 degrees between 15.9 m to 16.0 m depth.

BOREHOLE (STANDARD) OG06562.GPJ DST_MIN.GDT 10/3/06

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SAMPLE TYPE LEGEND

☐ Auger Sample	■ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▩ Side Sampler	
☐ Thin Wall Tube	☐ Grab Sample	

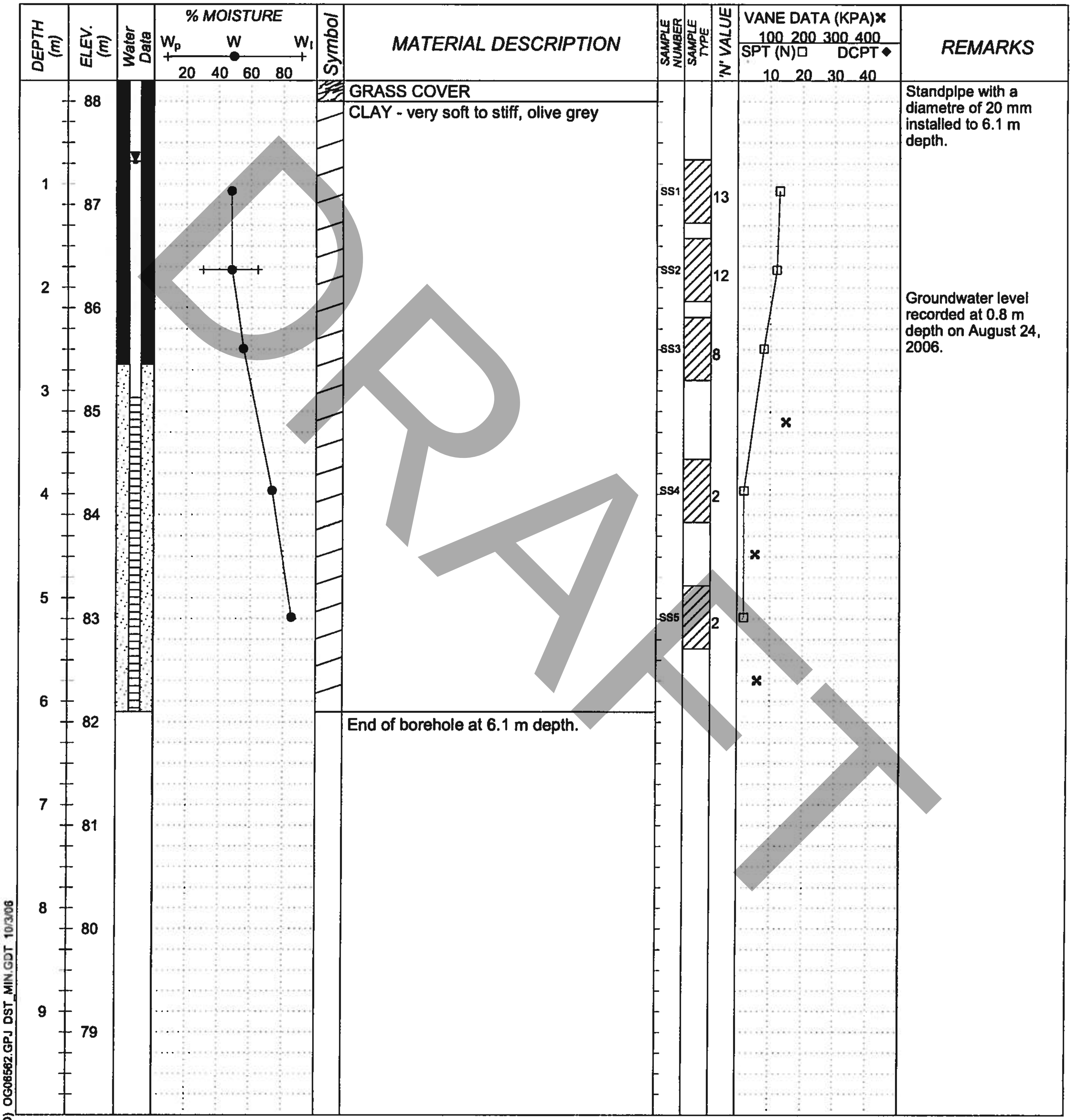
APPENDIX D

LOG OF BOREHOLE / MONITORING WELL BHMW11

DST REF. No.: **OG06562**
CLIENT: **Canada Lands Company**
PROJECT: **Preliminary Geotechnical Investigation**
LOCATION: **CFB Rockcliffe, Ottawa, Ontario**
SURFACE ELEV.: **88.20 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **August 14 2006**



GRASS COVER

CLAY - very soft to stiff, olive grey

End of borehole at 6.1 m depth.

Sample Number	'N' Value
SS1	13
SS2	12
SS3	8
SS4	2
SS5	2

Standpipe with a diametre of 20 mm installed to 6.1 m depth.

Groundwater level recorded at 0.8 m depth on August 24, 2006.

BOREHOLE (STANDARD) OG06562.GPJ DST_MIN.GDT 10/3/06

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Web: www.dstgroup.com

SAMPLE TYPE LEGEND

☰ Auger Sample	■ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▩ Side Sampler	
☐ Thin Wall Tube	▽ Grab Sample	

APPENDIX D

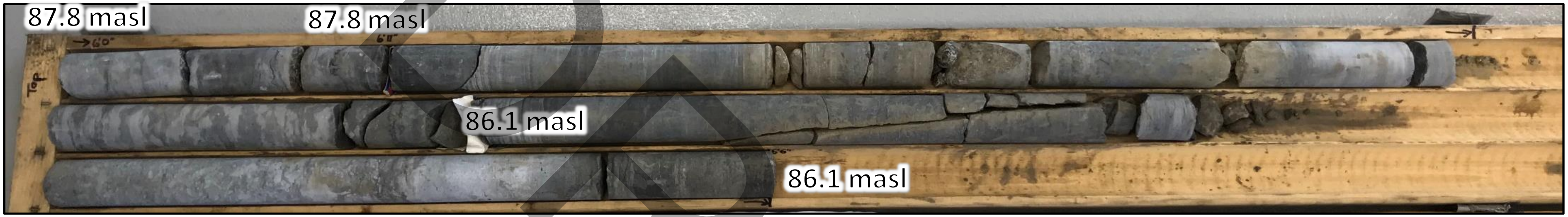
Rock Core Photographs
1050 Tawadina Road, Ottawa, ON

Project No.: 02203079.000

RC 4 – 87.8 to 87.7 masl
(RQD = 73%)

RC 5 – 87.7 to 86.1 masl
(RQD = 61%)

RC 6 – 86.1 to 84.6 masl
(RQD = 66%)



Rock Core No.: RC4 to RC6

Borehole: BH22-02

Date: May 2022

RC 5 – 86.4 to 85.0 masl
(RQD = 42%)

RC 6 – 85.0 to 83.5 masl
(RQD = 79%)

RC 7 – 83.5 to 83.2 masl
(RQD = 100%)

Rock Core No.: RC5 to RC7

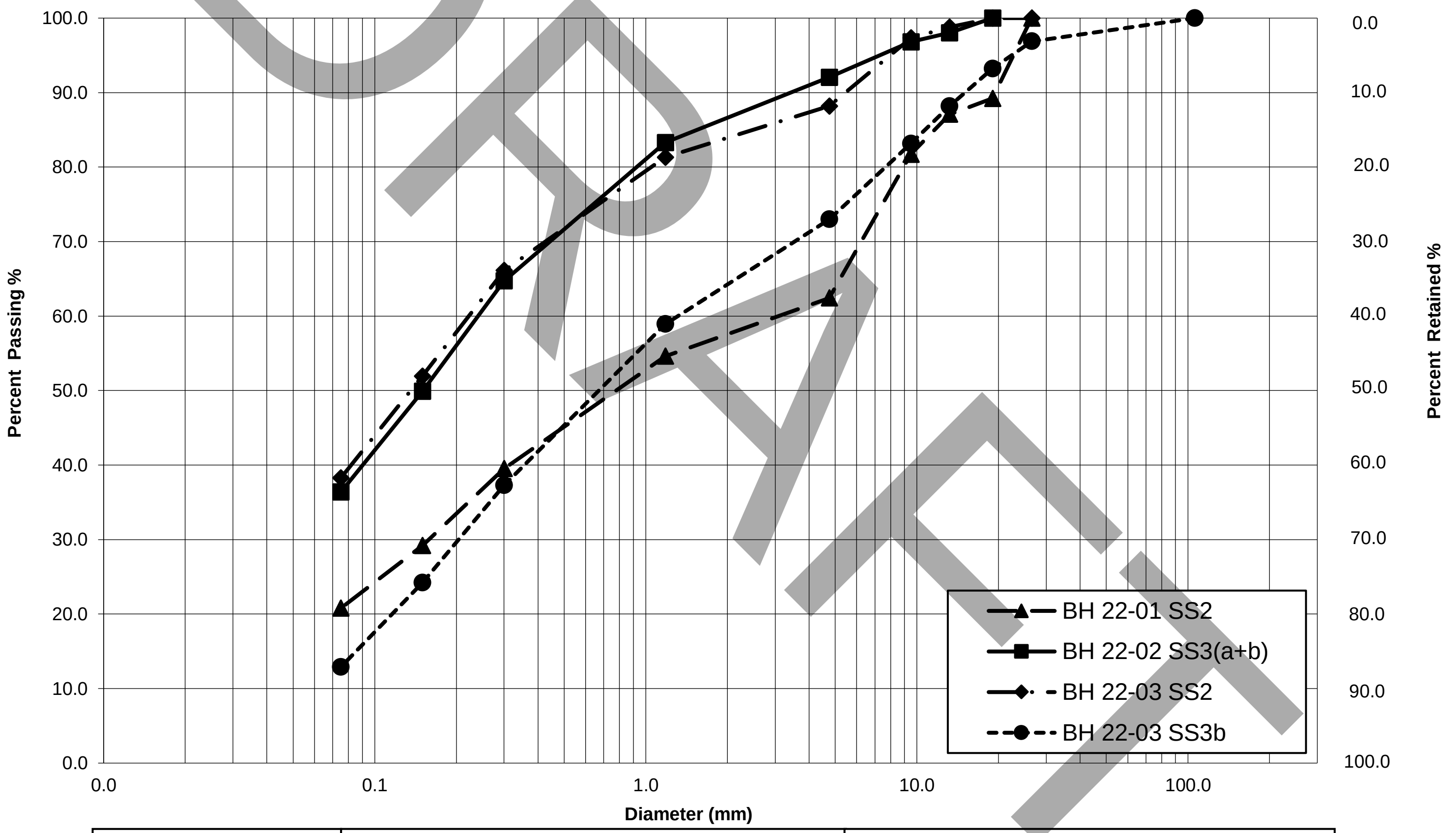
Borehole: BH22-03

Date: May 2022

Englobe Corp.
2713 Lancaster Road, Unit 101
Ottawa, ON
K1B 5R6
T: 1.877.300.4800
Fax: 613 748 1356
www.englobecorp.com
ottawalab@englobecorp.com

GRAIN SIZE ANALYSIS

Englobe Ref. No.:	2203079.000	**Date Sampled:**	5-May-22
Project:	Perliminary Geotchnical Investigation	**Sampled By:**	AA
Client:	Westurban Devolpmant Ltd.	**Material Source:**	Geotechnical Investigation
Project Location:	1050 Tawadina Rd, Ottawa,ON	**Sampling Location:**	1050 Tawadina Rd, Ottawa,ON
Sample #:	See table below	**Material Description:**	Sand/Sand and Gravel



Clay & Silt	**Sand**			**Gravel**	
	Fine	**Medium**	**Coarse**	**Fine**	**Coarse**
Particle-Size Limits as per USCS (ASTM D-2487)					

SUMMARY

Sample #	Depth (m)	Gravel (%)	Sand (%)	Clay and Silt (%)
BH 22-01 SS2	1.1	38	42	21
BH 22-02 SS3(a+b)	1.7	8	56	36
BH 22-03 SS2	1.1	12	50	38
BH 22-03 SS3b	1.7	27	60	13

Appendix D
- Chemical Testing Results

ENGLOBE

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Certificate of Analysis

Englobe Corp. (Ottawa)

2713 Lancaster Road, Unit 101
Ottawa, ON K1B 5R6
Attn: Alexandre Aramouni

Client PO:
Project: 02203079
Custody: 137091

Report Date: 16-May-2022
Order Date: 10-May-2022

Order #: 2220226

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2220226-01	02203079 BH22-02 SS2
2220226-02	02203079 BH22-03 SS4

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis
Client: **Englobe Corp. (Ottawa)**
Client PO:

Report Date: 16-May-2022
Order Date: 10-May-2022
Project Description: 02203079

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	12-May-22	12-May-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	11-May-22	11-May-22
Resistivity	EPA 120.1 - probe, water extraction	16-May-22	16-May-22
Solids, %	Gravimetric, calculation	11-May-22	11-May-22

Certificate of Analysis
Client: Englobe Corp. (Ottawa)
Client PO:

Report Date: 16-May-2022
Order Date: 10-May-2022
Project Description: 02203079

	Client ID:	02203079 BH22-02 SS2	02203079 BH22-03 SS4	-	-
	Sample Date:	04-May-22 13:00	04-May-22 14:00	-	-
	Sample ID:	2220226-01	2220226-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	86.8	92.4	-	-
General Inorganics					
pH	0.05 pH Units	6.97	7.33	-	-
Resistivity	0.10 Ohm.m	22.0	46.0	-	-
Anions					
Chloride	5 ug/g dry	7	11	-	-
Sulphate	5 ug/g dry	291	48	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g	ND			NC	20	
Sulphate	ND	5	ug/g	ND			NC	20	
General Inorganics									
pH	6.90	0.05	pH Units	6.97			1.0	2.3	
Resistivity	13.8	0.10	Ohm.m	13.6			2.0	20	
Physical Characteristics									
% Solids	71.3	0.1	% by Wt.	69.5			2.5	25	

Certificate of Analysis
Client: **Englobe Corp. (Ottawa)**
Client PO:

Report Date: 16-May-2022
Order Date: 10-May-2022
Project Description: 02203079

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	110	5	ug/g	ND	110	82-118			
Sulphate	105	5	ug/g	ND	105	80-120			

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Redox.

Applies to samples: 02203079 BH22-02 SS2

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

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Paracel ID: 2220226

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com

Paracel Order Number
(Lab Use Only)
2220226

Chain Of Custody
(Lab Use Only)
№ 137091

Client Name: Englobe Corp.
Contact Name: Alexandre Aramouni
Address: 2713 Lancaster Rd, Unit 101
Ottawa, ON
Telephone: 613-697-7450

Project Ref: 02203079
Quote #:
PO #:
E-mail: alexandre.aramouni@englobecorp.com

Page 1 of 1

Turnaround Time
☐ 1 day ☐ 3 day
☐ 2 day ☒ Regular
Date Required:

☐ REG 153/04 ☐ REG 406/19
☐ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table ___
For RSC: ☐ Yes ☐ No

Other Regulation
☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: **S** (Soil/Sed.) **GW** (Ground Water)
SW (Surface Water) **SS** (Storm/Sanitary Sewer)
P (Paint) **A** (Air) **O** (Other)

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	pH	CrVI	B (HWS)	Redox Potential	Sulphide	Chloride	Sulphate	Resistivity
1	02203079 BH 22-02 SS2	S		1	2022-05-04	13:00					X			X	X	X	X	X
2	02203079 BH 22-03 SS4	S		1	2022-05-04	14:00					X			X	X	X	X	X
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Comments:

Method of Delivery: Walk in

Relinquished By (Sign): Alexandre Aramouni
Relinquished By (Print):
Date/Time: 2022-05-10 /8:25

Received By Driver/Depot:
Date/Time:
Temperature: °C

Received at Lab:
Date/Time: May 10/22 8:23a
Temperature: 18.8 °C

Verified By:
Date/Time: May 10, 22 16:14
pH Verified: ☐ By:

Chain of Custody (Env) xlsx

Revision 4.0

Subcontracted Analysis

Englobe Corp. (Ottawa)
2713 Lancaster Road, Unit 101
Ottawa, ON K1B 5R6
Attn: Alexandre Aramouni

Paracel Report No.	**2220226**	Order Date:	10-May-22
Client Project(s):	**02203079**	Report Date:	24-May-22
Client PO:			
Reference:	**Standing Offer**		
CoC Number:	**137091**		

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	**Client ID**	**Analysis**
2220226-01	02203079 BH22-02 SS2	Redox potential, soil Sulphide, solid
2220226-03	02203079 BH22-03 SS4+SS3a	Redox potential, soil Sulphide, solid

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

20-May-2022

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 17 May 2022
LR Report: **CA15259-MAY22**
Reference: Project#: 2220226

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		20-May-22
2: Analysis Start Time		07:57
3: Analysis Completed Date		20-May-22
4: Analysis Completed Time		09:49
5: QC - Blank		< 0.04
6: QC - STD % Recovery		107%
7: QC - DUP % RPD		7%
8: RL		0.02
9: 02203079 BH22-02 SS2	04-May-22	< 0.04
10: 02203079 BH22-03 SS4+SS3a	13-May-22	< 0.04

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Client: Dale Robertson
Company: Paracel Laboratories Ltd. - Ottawa
Address: 300-2319 St. Laurent Blvd.
Ottawa, ON, K1G 4J8
Phone/Fax: (613) 731-9577 / (613) 731-9064
Email: drobertson@paracellabs.com

Work Order Number: 462882
PO #:
Regulation: None
Project #: 2220226
DWS #:
Sampled By:

Date Order Received: 5/17/2022
Arrival Temperature: 15 °C

Analysis Started: 5/24/2022
Analysis Completed: 5/24/2022

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
02203079 BH22-02 SS2	1754141	Soil	None		5/4/2022	1:00 PM
02203079 BH22-03 SS4+SS3a	1754142	Soil	None		5/13/2022	

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B

REPORT COMMENTS

Non Testmark containters received 05/17/2022 - K.G
Proceed regardless of hold time as requested by client -K.G
Lot# 2220226-01D & 2220226-03A- K.G

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 462882

This report has been approved by:

Marc Creighton
Laboratory Director

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 462882

WORK ORDER RESULTS

Sample Description	02203079 BH22 - 02 SS2		02203079 BH22 - 03 SS4+SS3a		
Sample Date	5/4/2022 1:00 PM		5/13/2022 12:00 AM		
Lab ID	1754141		1754142		
General Chemistry	Result	MDL	Result	MDL	Units
RedOx (vs. S.H.E.)	375	N/A	370	N/A	mV

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

MDL: Method detection limit or minimum reporting limit.

Quality Control: All associated Quality Control data is available on request.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

Reproduction of Report: Report shall not be reproduced, except in full, without the approval of Testmark Laboratories Ltd.

ICPMS Dustfall Insoluble: The ICPMS Dustfall Insoluble Portion method analyzes only the particulate matter from the Dustfall Sampler which is retained on the analysis filter during the Dustfall method.

Appendix E
- 2010 National Building Code Seismic Hazard Calculations
- Shear Wave Velocity Report (Geophysics GPR International Inc.)

ENGLOBE

2010 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.454N 75.632W

2023-08-23 17:58 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.2)	0.633	0.386	0.249	0.090
Sa (0.5)	0.309	0.187	0.123	0.043
Sa (1.0)	0.138	0.088	0.056	0.017
Sa (2.0)	0.046	0.028	0.018	0.006
PGA (g)	0.321	0.201	0.123	0.039

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada
Ressources naturelles Canada

Canada

GEOPHYSICS GPR INTERNATIONAL INC.
100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7
Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

August 8th, 2023

Transmitted by email: Gary.Cui@englobecorp.com
Our Ref.: GPR23-04748

Mr. Gary Cui
Engineering Intern, Geotechnical
Englobe Corp.
101 – 2713 Lancaster Road
Ottawa ON K1B 5R6

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
1050 Tawadina Road, Ottawa (ON)

[Project Nº: 02203079]

Dear Sir,

Geophysics GPR International inc. has been mandated by Englobe Corp. to carry out seismic surveys at 1050 Tawadina Road, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Class.

The surveys were conducted on July 19th, 2023, by Mrs. Anne-Catherine Cyr, trainee and Mr. Noé De Wergifosse. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the testing methods, and the results presented in table and graph.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *SPatial AutoCorrelation* (SPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface wave. The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the SPAC is considered a "passive" method, using the low frequency "signals" produced far away. The method can also be used with "active" seismic source records. The SPAC method generally allows deeper Vs soundings. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion. The dispersion properties are expressed as a change of velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding).

Figure 3 schematically outlines the basic operating procedure for the MASW method. Figure 4 illustrates an example of one of the MASW/SPAC records, the corresponding spectrogram analysis and resulting 1D V_S model.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is around 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

SURVEY DESIGN

The seismic spreads were laid out on a vacant lot (Figure 2). The geophone spacing was 3.0 metres for the main spread, using 24 geophones. Two shorter seismic spreads, with geophone spacings of 0.5 and 1.0 metre, were dedicated to the near surface materials. The seismic records were produced with a seismograph Terraloc Pro (from ABEM Instrument), and the geophones were 4.5 Hz.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and at 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. An 8 kg sledgehammer was used as the energy source, with impacts being recorded off both ends of the seismic spreads. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

RESULTS

The MASW calculated V_S results are illustrated at Figure 5.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\bar{V}_{S30} = \frac{\sum_{i=1}^{N} \mathrm{H}_i}{\sum_{i=1}^{N} H_i / V_i} \quad | \quad \sum_{i=1}^{N} \mathrm{H}_i = 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : $\mathrm{V_S}$ of layer "i")

Thus, the $\overline{V}_{S30}$ value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.

The calculated $\overline{V}_{S30}$ value of the actual site is 1414.3 m/s (Table 1), corresponding to the Site Class "B". However, the Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation. In the case the bottom of the foundation would be 1.9 metres or less from the rock, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" (Table 2).

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 1050 Tawadina Road, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and the seismic refraction to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 1414 m/s, corresponding to the Site Class "B" ($760 < \overline{V}_{S30} \leq 1500$ m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.-A of the NBC (2015), and the Building Code, O. Reg. 332/12. It must be noted that the Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation.

In the case the bottom of the foundation would be 1.9 metres or less from the rock surface, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" ($\overline{V}_{S30} > 1500$ m/s).

It must also be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Table 4.1.8.4.-A of the NBC 2015) can supersede the Site classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly,

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Senior Project Manager

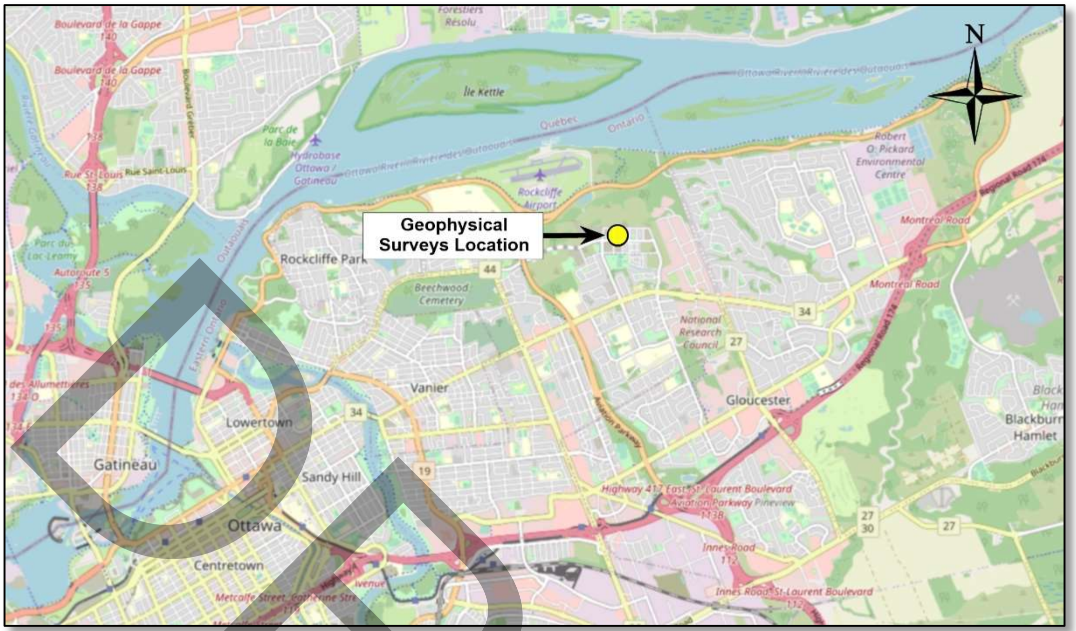


Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

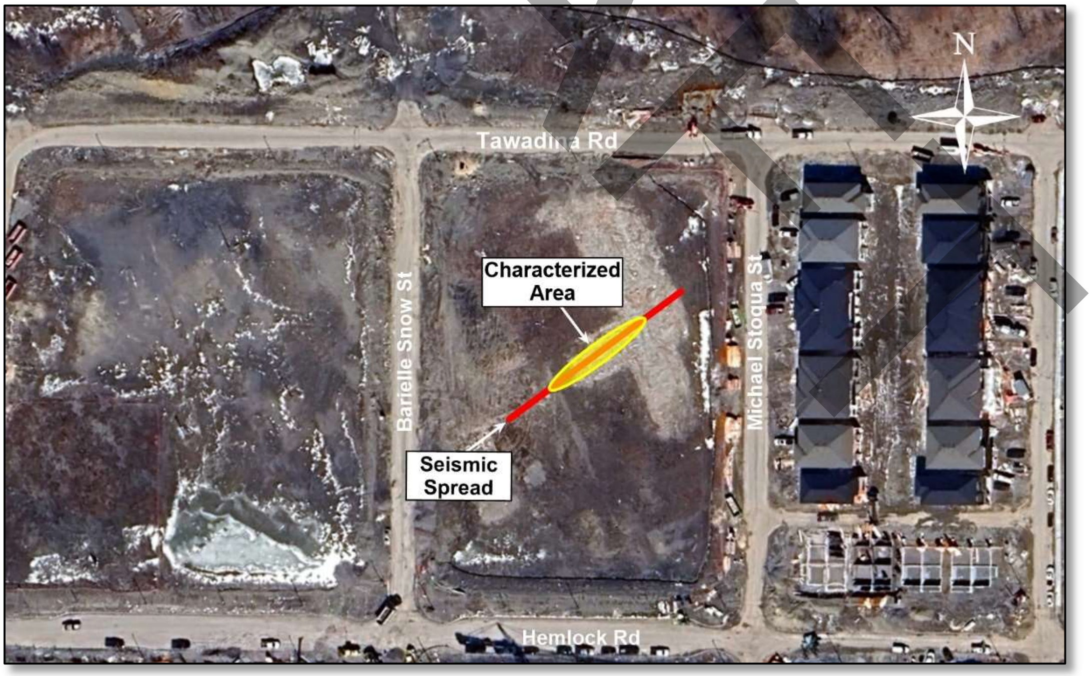


Figure 2: Location of the seismic spreads
(source: *Google Earth™*)

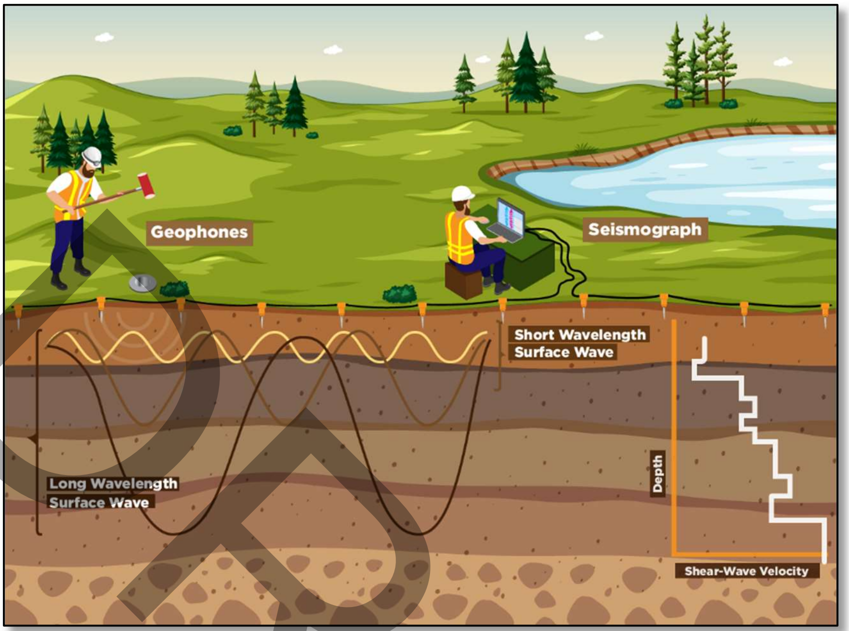
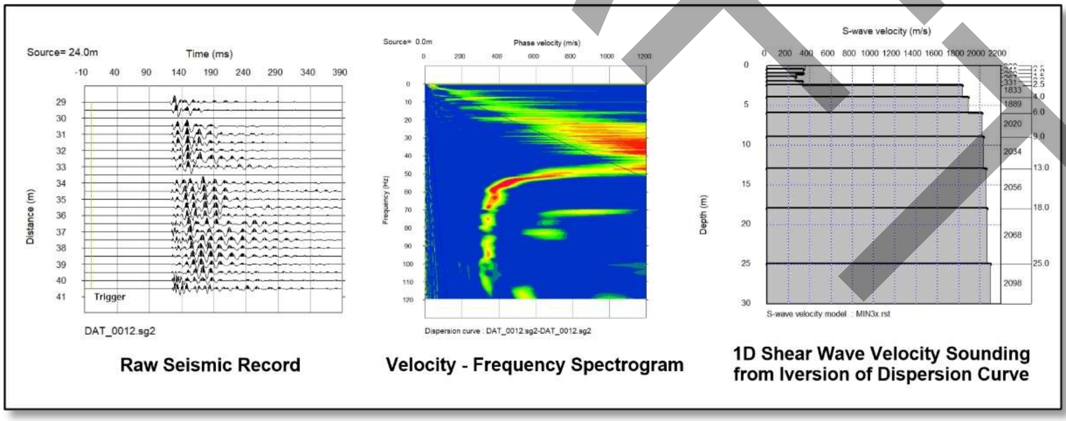


Figure 3: MASW Operating Principle



Raw Seismic Record

Velocity - Frequency Spectrogram

1D Shear Wave Velocity Sounding from Iversion of Dispersion Curve

Figure 4: Example of a MASW/SPAC record, Phase Velocity - Frequency curve of the Rayleigh wave and resulting 1D Shear Wave Velocity Model

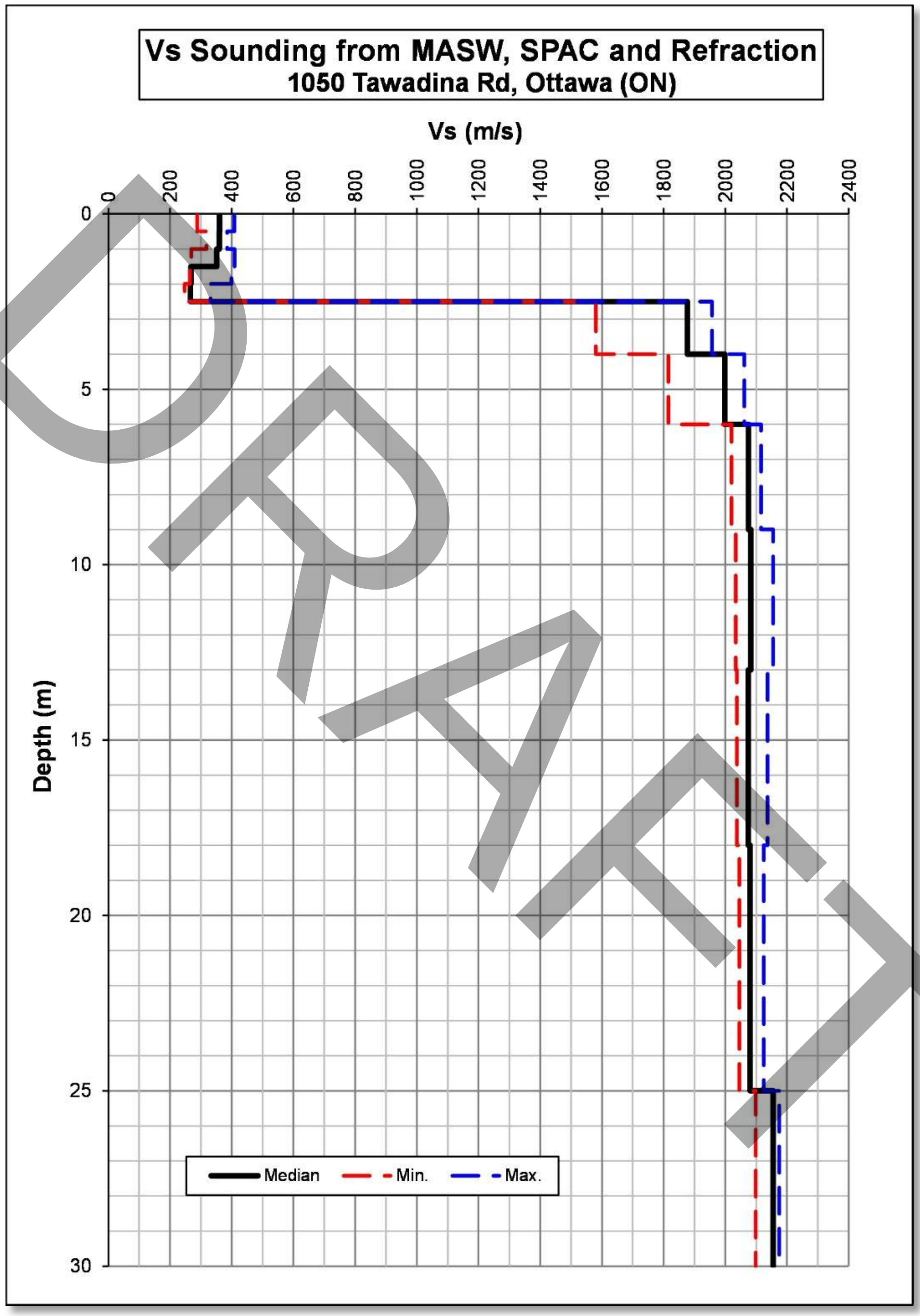


Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1
V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	288.1	360.3	408.3	**Grade Level (July 19th, 2023)**				
0.5	317.6	359.6	385.2	0.50	0.50	0.001388	0.001388	360.3
1.0	269.1	351.0	409.0	0.50	1.00	0.001391	0.002778	359.9
1.5	264.6	267.6	399.2	0.50	1.50	0.001424	0.004203	356.9
2.0	247.6	266.0	331.6	0.50	2.00	0.001869	0.006072	329.4
2.5	1580.5	1877.0	1957.1	0.50	2.50	0.001879	0.007951	314.4
4.0	1815.8	1998.1	2061.2	1.50	4.00	0.000799	0.008750	457.1
6.0	2019.8	2075.4	2116.1	2.00	6.00	0.001001	0.009751	615.3
9.0	2033.8	2082.6	2155.4	3.00	9.00	0.001445	0.011197	803.8
13.0	2037.7	2074.7	2136.8	4.00	13.00	0.001921	0.013117	991.1
18.0	2045.2	2080.1	2124.5	5.00	18.00	0.002410	0.015527	1159.3
25.0	2098.6	2155.4	2174.9	7.00	25.00	0.003365	0.018892	1323.3
30				5.00	30.00	0.002320	0.021212	1414.3

V_{S30} (m/s)	**1414.3**
Class	**B (1)**

(1) The Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation.

TABLE 2
Limit for the Site Class A

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	288.1	360.3	408.3	**Limit for the Site Class A (1.9 metres of soils)**				
0.5	317.6	359.6	385.2					
0.6	317.6	359.6	385.2					
1.0	269.1	351.0	409.0	0.40	0.40	0.001112	0.001112	359.6
1.5	264.6	267.6	399.2	0.50	0.90	0.001424	0.002537	354.8
2.0	247.6	266.0	331.6	0.50	1.40	0.001869	0.004406	317.8
2.5	1580.5	1877.0	1957.1	0.50	1.90	0.001879	0.006285	302.3
4.0	1815.8	1998.1	2061.2	1.50	3.40	0.000799	0.007084	479.9
6.0	2019.8	2075.4	2116.1	2.00	5.40	0.001001	0.008085	667.9
9.0	2033.8	2082.6	2155.4	3.00	8.40	0.001445	0.009531	881.4
13.0	2037.7	2074.7	2136.8	4.00	12.40	0.001921	0.011451	1082.8
18.0	2045.2	2080.1	2124.5	5.00	17.40	0.002410	0.013861	1255.3
25.0	2098.6	2155.4	2174.9	7.00	24.40	0.003365	0.017227	1416.4
30.6				5.60	30.00	0.002598	0.019825	1513.3

V_{S30}* (m/s)	**1513.3**
Class	**A**