FINAL REPORT

Geotechnical Investigation

Massey Library Retrofit – RMC Kingston, ON

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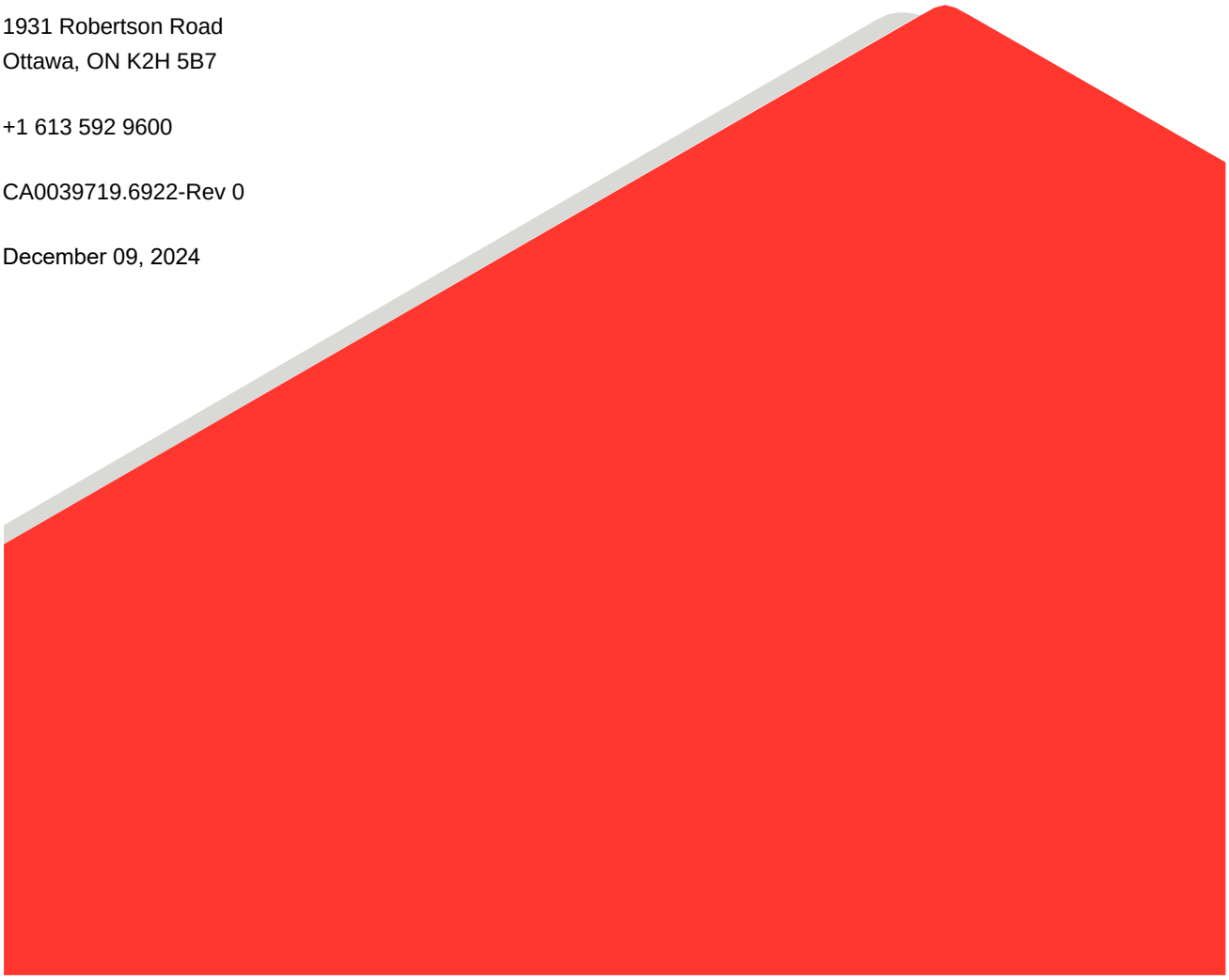
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1.0 INTRODUCTION

1.1 Context

This report presents the results of a geotechnical investigation carried out in support of the Massey Library retrofit project located in the Royal Military College complex at 7 Valour Drive, Kingston, ON. It is understood that the project includes the replacement of the existing interior elevator and shaft, believed to supported on a “deep” foundation extending to bedrock, although the founding depth and bearing condition are not clearly documented in historical records. NORR (herein the “Client”) retained WSP to assess the bedrock depth at the elevator location, and also provide a Seismic Site Classification (per NBCC 2020) to inform the retrofit design.

Based on WSP’s interpretation of the information obtained during the records and drawing review and our field investigation, a general description of the subsurface soil, bedrock and groundwater conditions is presented in this report. Site conditions and available project details were used to prepare geotechnical considerations and recommendations for the retrofit project. In preparing our fieldwork, testing plans and report outline for this assignment, we consulted historical reports and data, and government published mapping and information for the site area and region.

1.2 Limitations and Disclaimer

This report has been prepared at the request and for the sole use of NORR. With respect to limitations and disclaimers, reference should be made to the contract agreement with NORR dated July 29, 2024.

1.3 Scope of Work

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

- A desktop study and review of existing building and construction drawings, limited geotechnical information for the site and surrounding areas, discussions with the architect and DCC, and a site reconnaissance visit;
- Laying out borehole locations and obtaining utility clearances at the site, including public and private (DCC-owned) services and infrastructure;
- Drilling of two exploratory boreholes, one inside the building as close as possible to the new elevator shaft, and one outside the building within a clear approved area;
- Obtaining soil and bedrock core samples for inspection and possible testing;
- Installing casing at the outdoor location for Vertical Seismic Profiling (VSP);
- Geotechnical and chemical laboratory testing of soil samples; and
- Preparation of this report which presents the factual results of the investigation and provides geotechnical considerations and recommendations related to the design and construction of the proposed retrofit project.

2.0 SITE INVESTIGATION

2.1 Desktop Study

WSP reviewed available geological maps and databases, as well as all information made available during the initial project planning phase. This information included some historic building information and foundation design details.

Surficial geology maps indicate the soils in the project area consist of fine textured glaciomarine deposits, including silt and clay and minor sand. Soil mapping indicates that the overburden in the project area also consists of stone-poor, sandy silt to silty sand-textured glacial till.

Bedrock geology maps indicate the bedrock in the project area consists of limestone, dolostone, shale, arkose, sandstone of the Ottawa group, Simcoe group and Shadow Lake formation.

The Ontario Geotechnical Boreholes database does not include any records of historic boreholes drilled within 500 m of the site. However, the Ministry of the Environment, Conservation and Parks (MECP) well record database indicates that there were three historic wells installed within 300 m of the site. The well records encountered fill and clay materials overlying limestone at depths ranging between 0.6 and 3.4 meters below ground surface (mbgs).

Based on the architectural and structural as-built drawings of the Massey Library, provided by the Client and attached in Appendix E, the building is supported on driven (and some battered) piles to bedrock with a design of one to four piles per pile cap. Based on the 18 historic borehole records shown on the drawings, the ground elevation at the building location ranged between about 77.9 and 76.0 meters above sea level (masl) and the pile refusal elevation ranged between about 74.8 and 72.0 masl. The overburden consisted of stones and clayey fill. Compression tests on two rock core samples indicated UCS values of 74 and 86 MPa. It was noted that on the drawings that foundations shall bear on rock capable of supporting 10 tons per sq. foot (1,072 KPa) and that piles shall be uncased and capable of supporting 15 tons (747 kN) per pile.

2.2 Geotechnical Fieldwork

The geotechnical borehole investigation was carried out between August 19 and 22, 2024, during which two boreholes were advanced at the locations shown on Figure 1.

The exterior borehole was advanced on the patch of lawn next to the southern building entrance using a truck-mounted CME 75 rig. The interior borehole was advanced in the lecture hall next to the existing elevator location using portable drilling equipment (Hilti DD200). The drilling equipment and machinery was supplied and operated by OGS Inc. based in Almonte, ON.

Standard Penetration Tests (SPTs) were carried out in the exterior borehole at regular depth intervals, in general conformance with ASTM D 1586. In the interior borehole, the SPTs were advanced using a 21 kg (47 lb) hammer. To account for that, the N values shown on the BH24-02 log were determined by dividing the counted number of blows in the field by three. Soil samples were recovered using 50 mm outside diameter (OD) split-spoon samplers, driven using the SPT technique. Sampling continued into the bedrock using diamond coring techniques after auger refusal was encountered.

One 62 mm (2.5'') diameter PVC pipe was installed in the exterior borehole BH24-01 to allow for subsequent vertical seismic profiling (VSP), which was conducted on August 28, 2024. The theory, methodology, and results of the VSP are presented in Appendix D.

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

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

All borehole locations were selected, staked in the field, and subsequently surveyed by WSP personnel. The geodetic reference system used for the survey is the North American Datum of 1983 (NAD83). The borehole coordinates are based on the Universal Transverse Mercator (UTM 18N) coordinate system. The elevations are referenced to the Geodetic datum (CGVD28). The borehole general information is summarized in Table 1 below.

Table 1: Borehole General Information

Borehole No.	Coordinates: UTM 18N		Refusal Depth (m)	Comments
	Northing (m)	Easting (m)		
BH24-01	4898559	382707	4.3	Rock coring to 9.8 mbgs. VSP casing installed.
BH24-02	4898568	382720	3.1	Rock coring to 5.3 mbgs.

3.0 SUBSURFACE CONDITIONS

3.1 General

The following section provides a general description of the major soil and bedrock layers encountered during the geotechnical investigation. It should be noted that the following discussion includes several simplifications for the purposes of discussing broadly similar soil strata and bedrock types. Boundaries between geological materials may be gradational, and variable across lateral distances.

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

3.2 Topsoil and Organics

A layer of topsoil and/or fill with organics was encountered at the ground surface of exterior borehole BH24-01. The measured layer thickness was approximately 300 mm.

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

3.3 Existing Slab Structure

A 70 mm terrazzo floor coating was encountered in the interior borehole BH24-02 location. The terrazzo overlaid a 300 mm thick concrete slab followed by a 150 mm thick crushed stone gravel layer, for a total thickness of 520 mm.

3.4 Fill

Fill was encountered in both boreholes and extended to a depth of 1.7 mbgs at borehole BH24-01 and to top of bedrock (3.1 m) at borehole BH24-02. The fill was a heterogenous mixture of clay, silt and sand (silty sand, silty clay).

Standard Penetration Tests (SPTs) were carried out within the fill and returned 'N' values ranging from 1 to 13 blows per 0.3 m of penetration.

A grain size distribution test was conducted on one sample of the fill layer; results are presented in Appendix C and the following table. Grain size distribution boundaries in this report are based on the Unified Soil Classification System (see laboratory test results in Appendix C).

Table 2: Results of Grain Size Analysis – Fill

Borehole No.	Sample No.	Sample Depth (m)	Grain Size Distribution			
			% Gravel	% Sand	% Silt	% Clay
BH24-02	SS-2	0.53 – 1.14	9	53	24	14

Atterberg limits were conducted on one sample of the cohesive fill and the results are presented in Appendix C. A summary of the results is also presented in the table below.

Table 3: Results of Atterberg Limits Test – Fill

Borehole No.	Sample	Depth (m)	Water content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Liquidity Index	USCS Symbol
BH24-02	SS-5	2.44 – 3.05	37	39	20	19	0.9	Cl

Natural moisture content determination carried out on six samples of the fill material yielded moisture contents ranging from approximately 20% to 37%.

3.5 Cohesive (Clayey Silt, Silty Clay) Deposit

A natural or potentially reworked cohesive deposit consisting of a brown, fissured, mottled and varved silty clay to clayey silt with trace sand was encountered at borehole BH24-01. The deposit was encountered at a depth of 1.7 mbgs and extended the bedrock surface at a depth of 4.3 mbgs.

Standard Penetration Tests (SPTs) returned 'N' values ranging from 2 to 8 blows per 0.3 m of penetration. The material was not soft enough to conduct vane shear tests.

Atterberg limits were conducted on two samples; results are presented in Appendix C and summarized as follows.

Table 4: Results of Atterberg Limits Test – Cohesive Deposit

Borehole No.	Sample	Depth (m)	Water content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Liquidity Index	USCS Symbol
BH24-01	SS-3	1.52 – 2.13	36	58	27	31	0.3	CH
BH24-01	SS-5	3.05 – 3.66	42	43	22	21	0.9	Cl

The moisture water content for four samples varied between 31% and 42%, based on laboratory tests

3.6 Refusal and Bedrock

Auger and/or split-spoon sampler was noted in both boreholes at depths ranging between approximately 3.1 mbgs to 4.3 mbgs.

Bedrock was confirmed by coring in both boreholes using rotary diamond drilling techniques and retrieving HQ-sized rock cores. Total cored lengths in the boreholes varied between 2.2 m to 5.4 m.

The cored rock was generally described as fresh, bedded, grey to light grey, fine to medium grained, non- to slightly porous, medium strong to strong, limestone with thin shale beds. A greenish dolostone was encountered at borehole BH24-01 only at a depth of 8.4 mbgs.

Rock Quality Designation (RQD) values for the recovered rock core samples ranged from 73 % (fair) to 100 % (excellent); being generally good to excellent.

The results of the UCS testing carried out on two samples of the bedrock indicate values of 193 and 250 MPa which indicates very strong bedrock under ISRM criteria (CFEM, 2023).

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

3.7 Groundwater

No monitoring wells were installed as part of the current investigation, although water levels were measured in the open boreholes after the completion of drilling at a depth of 0.4 mbgs in borehole BH24-02 and approximately 4 mbgs in borehole BH24-01.

Groundwater levels are expected to fluctuate seasonally and over shorter periods of time. Higher groundwater levels should be expected during wet periods of the year, such as spring after the snowmelt or during periods of heavy rain. Groundwater at the site is generally controlled by the level of Lake Ontario, near elevation 74 masl.

3.8 Corrosion Testing

Soil samples from were submitted to Eurofins Environmental Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. Test results are provided in Appendix C and are also summarized in the following table.

Table 5: Chemical Test Results (Corrosion Parameters)

Borehole No.	Sample Number	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
BH24-01	SA-3	1.52 – 2.13	0.007	< 0.01	0.21	8.06	4762
BH24-02	SA-4	1.83 – 2.44	0.002	0.01	0.21	7.99	4762

4.0 DISCUSSION

4.1 General

This section of the report provides engineering guidance related to the geotechnical design aspects of the RMC Massey Library retrofit project, based on our interpretation of the available information described herein and the project requirements. It should be noted that considerations and recommendations are intended for Architects and Designers. Contractors bidding on or undertaking construction works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedules, safety, equipment capabilities, etc. With respect to limitations and disclaimers, reference should be made to the contract agreement dated July 29, 2024.

4.2 Frost Protection

All perimeter and exterior foundation elements in heated areas should be provided with a minimum of 1.2 m of earth cover for frost protection purposes. Isolated, unheated exterior foundation elements are adjacent to surfaces that are cleared of snow cover during winter months should be provided with a minimum of 1.5 metres of earth cover.

As an alternative to earth cover, consideration could be provided to the use of rigid insulation. It is understood that an unheated exterior slab-on-grade with a ramp may be built at the front entrance of the library. To provide adequate frost protection, a 75 mm thick layer of rigid insulation should be placed underneath the slab and extending horizontally at least 1.5 m beyond the slab edges. The rigid insulation extending outside the slab footprint should have a minimum soil cover of 300 mm. The rigid insulation should be placed on a minimum of 150 mm thick, clean well graded granular engineered fill (such as OPSS Granular A). Refer to manufacturer design requirements for specific products.

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

4.3 Liquefaction Potential

The shallow soils at the site are not considered susceptible to liquefaction. No further assessments or analyses are recommended.

4.4 Seismic Site Classification

As outlined in the 2020 National Building Code of Canada (NBCC 2020), building foundations must be designed to resist a minimum earthquake force. Based on the results of the geophysical testing, which included VSP testing at borehole BH24-01, the measured shear wave velocity in the limestone bedrock varied between 824 and 1267 m/s. In accordance with Tables 4.1.8.4.A and B of the NBCC 2020, Site Class B can be considered for the design of foundations resting on bedrock.

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

The VSP testing results are presented in Appendix D.

4.5 Foundation Design

Bedrock was encountered at a depth of 3.1 mbgs in the interior borehole location (BH24-02) and 4.3 mbgs in the exterior location (BH24-01). Shallow footings founded on existing soils are not recommended due to the presence of significant amounts of cohesive fill, and to avoid creating a surcharge on any buried structure. The elevator structure could be supported by spread footings on bedrock or bored pipe piles extending to bedrock.

4.5.1 Spread Footings

Concrete columns founded on spread footings could be built from the bedrock surface after excavating and building a supported shaft down to the bedrock. Recommendations on excavation shoring are provided in Section 4.7.

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

The bedrock bearing surface should be checked, evaluated, and approved at the time of construction by a Geotechnical Engineer who is familiar with the findings of this investigation and the design and construction of similar projects prior to placement of any concrete, back-fill, etc. Inspections shall confirm that zones of loose, weathered rock are removed. If any fractures, joints, or voids are found within the exposed bedrock surface at the footing locations, they should be cleaned out (Hydrovac) and backfilled with U-Fill or acceptable grout. Consideration could also be given to placing a concrete mud slab depending on the bedrock surface conditions.

For the ULS sliding resistance of a cast-in-place footing placed on bedrock, an unfactored sliding friction coefficient of 0.7 may be considered. In accordance with the 2023 Canadian Foundation Engineering Manual (CFEM 2023), a resistance factor of 0.8 should be applied. The resistance to lateral loads could be increased by constructing a shear-key at the bottom of the footings if needed, or through the use of rock anchors. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments.

After the footing and column construction, the open shored excavation can be backfilled using acceptable earth borrow (e.g., per OPSS.MUNI 206/212), Select Subgrade Material (per OPSS.MUNI 1010), and/or existing site materials approved by the Geotechnical Engineer and/or Qualified Professional (QP) for reuse during construction.

4.5.1.1 Rock Anchors

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

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

i) Failure of the steel tendon or top anchorage;

ii) Failure of the grout/tendon bond;

iii) Failure of the rock/grout bond, and

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

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

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

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

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

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

φ = Geotechnical resistance factor (use 0.4);

γ' = Effective unit weight of rock and soil (use 16 kN/m^3 below the groundwater level);

D = Anchor length in metres; and,

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

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

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

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

D = Depth of anchor group (m);

a = Width of anchor group (m);

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

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

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

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

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

φ = Geotechnical resistance factor, use 0.4;

γ' = Effective unit weight of rock and soil, use 16 kN/m^3 below the water table; and,

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

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

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

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

4.5.2 Bored Pipe Piles

Bored/drilled shaft foundations extending to the limestone bedrock could be considered. Bored piles can generally be constructed using portable equipment, though the high-water level indoors could be problematic during construction.

The following unfactored geotechnical resistances may be used to evaluate the pile's end bearing resistance.

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

Where: q_a = allowable bearing pressure

σ_c = use the smallest between the average unconfined compressive strength of rock core (150 MPa) and the maximum compressive strength of concrete (assume 35 MPa at 28 days)

K_{sp} = empirical factor, use 0.2 for medium spacing, includes a safety factor of 3

d = depth factor, use 1.0, assuming no socket

The ULS factored geotechnical axial compression resistance for piles tipped on sound bedrock, assuming the values presented above, is calculated as 10.5 MPa (after dividing by 3 and applying a safety factor of 2). This factored ULS value may be greater than the structural capacity of the pile, which will likely govern the design and should be checked by the Designer. For end-bearing piles on bedrock, SLS conditions generally do not govern design. The post-construction settlement of the structural elements, which derive their support from the piles driven to bedrock, should be negligible.

Rock socketed piles may be considered if higher axial compression and lateral resistances are required, or if uplift resistance is required. It should be noted that pile base bonding generally does not contribute to uplift resistance. Further geotechnical support can be provided by WSP if rock sockets are to be considered in the design.

Concrete, if required to fill piles, should be placed using the tremie method assuming that groundwater will be encountered above the bedrock. Installation should follow the requirements mentioned in OPSS 903 (Construction Specification for Deep Foundation) or similar applicable standards.

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

4.5.2.1 Lateral Resistance

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

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

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

d = pile diameter or width (m)

n_h = constant of horizontal subgrade reaction (MPa / m), use 1,300 kN/m^3 for general fill

z = depth below ground surface (m)

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

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

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

d = pile diameter or width (m)

Su = undrained shear strength, use 10 kPa for general fill

Considering the heterogeneous nature of the existing general fill, the coefficient of horizontal subgrade reaction is to be determined for both cohesionless and cohesive soils conditions, and the lowest value of the two is to be used.

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

Table 6: Horizontal Group Reduction Design Factors

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

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

A more detailed assessment of the lateral resistance of the foundations to lateral loading (e.g., p-y method) may need to be undertaken once the number of piles and the layout of the pile group is known.

It is understood that the piles supporting the shaft could be braced either between the piles or to the building to provide additional lateral resistance. In this case, the group effect would not be applicable.

4.5.2.2 Downdrag

When piles have been installed in or through a clay deposit that is subject to consolidation, from surcharge loadings for example, the resulting downward movement of the clay around the piles, as well as in any soils above the clay layers, induces downdrag forces in the piles through negative skin friction.

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

4.6 Excavations & Groundwater Control

No unusual problems are anticipated in excavating the overburden using conventional hydraulic excavating equipment. Excavations within the bedrock may require heavy hydraulic breakers.

Based on the groundwater level observed in the interior borehole BH24-02 during drilling (0.4 mbgs), the excavations will likely encounter considerable groundwater seepage, and dewatering control will likely be required during construction and excavation activities. It should be possible to manage groundwater seepage using properly filtered sumps, ditches, pumps, etc.

4.7 Temporary Shoring

If the spread footing resting on bedrock option is preferred by the designer, a 3 to 4 m excavation down to the bedrock will be required, and thus a temporary shoring system should be considered.

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

For design purposes, a soldier pile and timber lagging system may be a feasible shoring method. Due to the presence of shallow bedrock beneath the overlying deposits, the soldier piles might need to be braced, socketed or anchored into the competent bedrock to provide sufficient toe fixity. Soldier pile and lagging walls are considered suitable for the sides of the excavations (provided that settlement-sensitive structures or utilities are not present in the zone of influence of the walls) where the objective is to maintain an essentially vertical excavation wall, and the movements above and behind the wall need only be sufficiently limited so existing structures and utilities will not be adversely affected.

Where foundations or settlement-sensitive infrastructure are present within the zone of influence of the shoring, the deflections may need to be greatly limited; therefore, a more rigid shoring structure could be required. WSP can provide further recommendations and guidance in the design stages when more information is available.

4.7.1 Lateral Earth Pressures

The lateral earth pressures acting on the shoring system will depend on the existing soil conditions, on the magnitude of surcharge including construction loadings, the freedom of lateral movement of the shoring structure, and drainage conditions behind it.

Based on the results of the drilling program, the excavations will be carried out within the existing fill and the cohesive deposit. The following parameters (unfactored) may be used for the design of the shoring system.

Table 7: Lateral Earth Pressures – Soil Parameters

Material	Unit Weight (kN/m^3)	Coefficients of static lateral earth pressure	
		Active, Ka	At rest, Ko
Fill	17	0.36	0.53
Cohesive Deposit	17	1.0 (Short Term) 0.33 (Long Term)	1.0 (Short Term) 0.50 (Long Term)

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

The following equation can be used to calculate the lateral earth pressures for static loading conditions. It is assumed in this equation that the ground above the shoring structure is flat, not sloping. Surcharge forces (weight above the shoring structure, if any) will need to be considered and included in the design.

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

Where: P=Lateral earth pressure(kPa)

K=Earth pressure coefficient.

γ=The unit weight of soil used for backfilling behind the wall

h=The depth to the point of interest

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

4.8 Corrosion and Cement Type

Soil samples from both boreholes were submitted to Eurofins Environmental Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. The results of this testing are provided in Appendix C and are summarized in the following table.

Table 8: Results of Basic Chemical Testing

Borehole No.	Sample Number	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
BH24-01	SA-3	1.52 – 2.13	0.007	< 0.01	0.21	8.06	4762
BH24-02	SA-4	1.83 – 2.44	0.002	0.01	0.21	7.99	4762

The pH, resistivity and chloride concentration give an indication of the degree of corrosiveness of the sub-surface environment. Generally, based on ASTM STP 741, the test results indicate a moderate potential for corrosion of exposed ferrous metal at the site which should be considered in the design of substructures.

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

5.0 CLOSURE

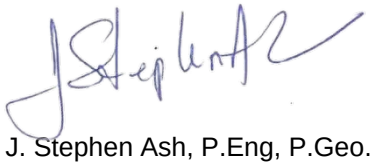
We trust that this geotechnical report provided sufficient information to support the design and construction of the proposed retrofit project. We remain available for any questions or concerns about the report.

Signature Page

WSP Canada Inc.



Othmane Benkirane, MASc., P.Eng
Geotechnical Engineer

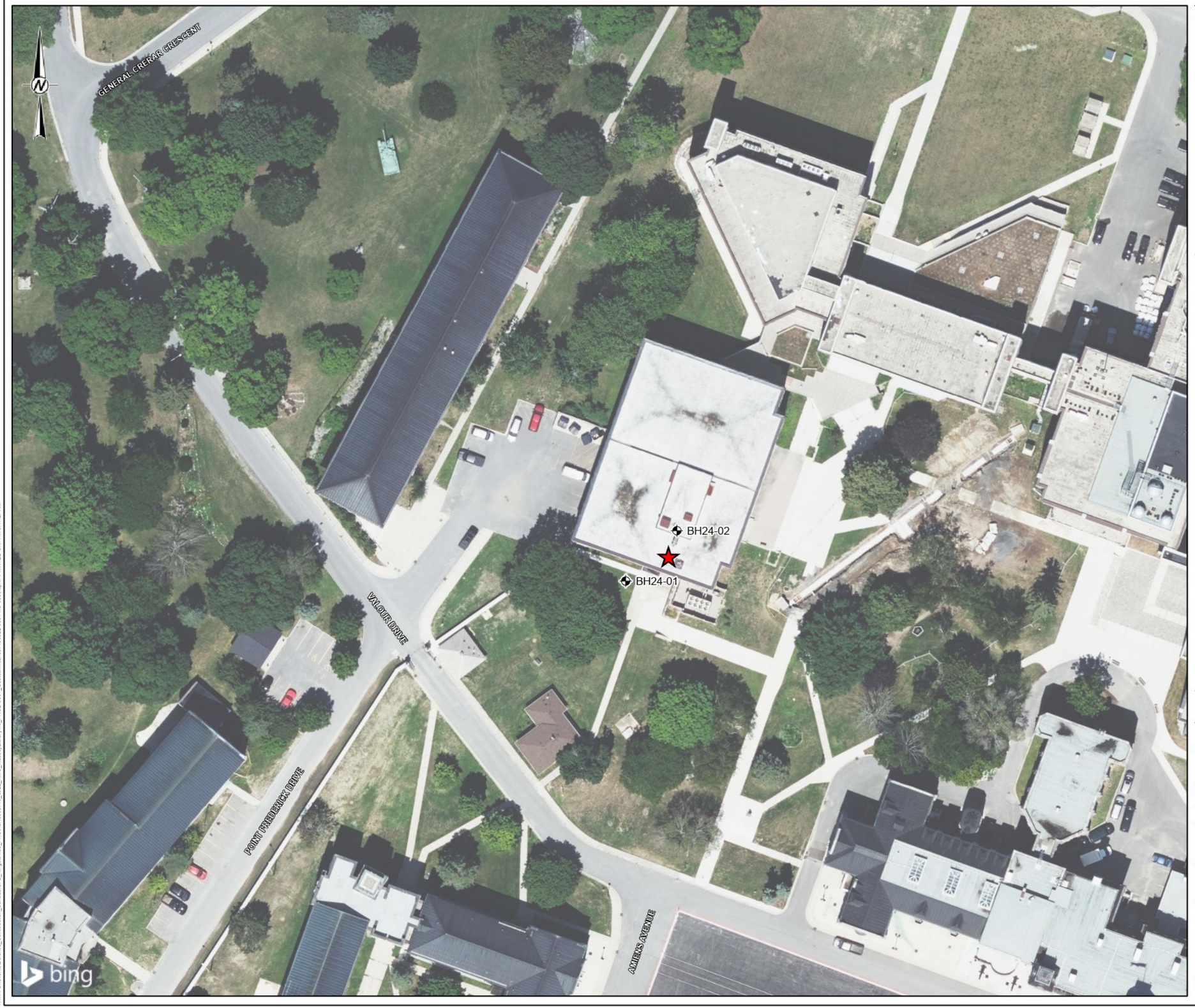


J. Stephen Ash, P.Eng, P.Geo.
Senior Principal Geotechnical Engineer

OB/JSA/yj

FIGURE 1

Site Plan



KEY MAP



Scale: 1:150,000

LEGEND

- PROJECT LOCATION
- BOREHOLE LOCATION

DRAFT

0 50

1:800 METRES

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. CONTAINS INFORMATION LICENSED UNDER THE OPEN GOVERNMENT LICENCE - ONTARIO.
2. BASE MAP: PROVINCE OF ONTARIO, ESRI CANADA, ESRI, TOMTOM, GARMIN, SAFEGRAPH, METI/NASA, USGS, EPA, NPS, USDA, USFWS, NRCAN, PARKS CANADA, © 2024 MICROSOFT CORPORATION © 2024 MAXAR ©CNES (2024) DISTRIBUTION AIRBUS DS
3. PROJECTION: NAD 1983 UTM ZONE 18N, TRANSVERSE MERCATOR

CLIENT
NORR ARCHITECTS & ENGINEERS LTD.

PROJECT
GEOTECHNICAL INVESTIGATION
MASSEY LIBRARY RETROFIT
ROYAL MILITARY COLLEGE, KINGSTON, ON

TITLE
SITE PLAN
BOREHOLE LOCATIONS

CONSULTANT

wsp

YYYY-MM-DD	2024-09-05
DESIGNED	OB
PREPARED	AP
REVIEWED	OB
APPROVED	SA

PROJECT NO.	CONTROL	REV.	FIGURE
397196922	0001	A	1

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

25mm

PATH: S:\Clients\Defence_Construction_Canada\CFB_Kingston\99_PRO\CA0039719_6922_RMC_Massey_Library\40_PRO\0001_Geotechnical\397196922-0001-H5-0000.aprx, PRINTED ON: AT: 9:47:27 AM

APPENDIX A

Borehole Logs

METHOD OF SOIL CLASSIFICATION

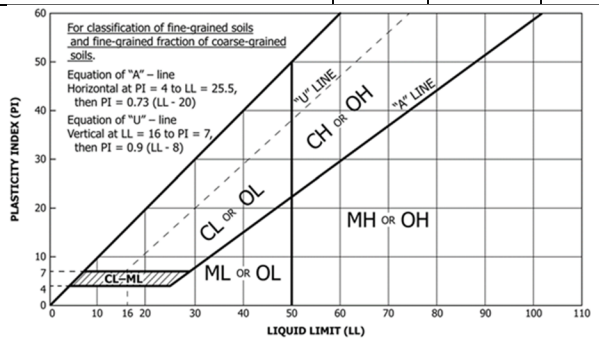
The WSP Canada Soil Classification[1] System is based on the Unified Soil Classification System (USCS) (after ASTM D2487)

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

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

Coarse-Grained Soil Note(s):

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



Fine-Grained Soil Note(s):

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

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

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

GRADATIONAL COMPONENT TERMS

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); Nd:

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

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

SAMPLES

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

SOIL TESTS

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

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

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

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

Field Moisture Condition

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

COHESIVE SOILS

Consistency

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

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

Water Content

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

2 shear strength = (compressive strength)/2

PROJECT: CA0039719.6922

LOCATION: N 4898559.00; E 382707.00

RECORD OF BOREHOLE: 24-01

SHEET 1 OF 1

BORING DATE: August 19, 2024

DATUM:

DRILL RIG: CME-75 Truck

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80)	SHEAR STRENGTH Cu, kPa (nat V. + Q - ● / rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3})	WATER CONTENT PERCENT (Wp W WI) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION / GRAIN SIZE DISTRIBUTION (%)
		GROUND SURFACE											GR SA SI CL
0	Power Auger 200 mm O.D.; 108 mm I.D. HOllow Stem Auger	TOPSOIL/FILL - (SM) SILTY SAND, fine, trace organics; brown; non-cohesive, moist, loose		0.00	1A	SS	6						
		FILL - (CL/CI) SILTY CLAY, some sand, trace to some gravel; brown, fissured, coal present; cohesive, w<PL, stiff		0.30	1B								
1					2	SS	6						
2		(CH/CI) SILTY CLAY, trace sand; brown, fissured, mottled; cohesive, w<PL to w~PL, firm to stiff to (ML/CL) CLAYEY SILT to SILTY CLAY; brown, fissured, mottled; cohesive, w<PL to w>PL, stiff to firm		1.68	3	SS	5						
					4	SS	8						
3					5	SS	3						
4		- w>PL, soft - some sand and gravel			6	SS	2						
		END OF BOREHOLE For rock coring details refer to Record of Drillhole 24-01.		4.32									
5													
6													
7													
8													
9													
10													

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

GTA-BHS 005 S:\CLIENTS\NORR\MASSEY_LIBRARY_RMC_KINGSTON_ON\02_DATA\GINT\MASSEY_LIBRARY_RMC_KINGSTON_ON.GPJ GAL-MIS.GDT 9/16/24

PROJECT: CA0039719.6922

RECORD OF DRILLHOLE: 24-01

SHEET 1 OF 1

LOCATION: N 4898559.00 ;E 382707.00

DRILLING DATE: August 19, 2024

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-75 Truck

DRILLING CONTRACTOR:

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE % (80 60 40 20)	RECOVERY: SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER (5 10 15 20)	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX (W1 W2 W3 W4 W5 W6)	Diametral Point Load Index (MPa) (2 4 6)	FEATURES	PIEZOMETER
		Continued from Record of Borehole 24-01																	
5		Fresh, bedded, grey to light grey, fine grained, slightly porous, medium strong to strong, LIMESTONE with thin shale beds		4.32	1							BD,, BD,,							BC
6					2							BD,, BD,, BD,, BD,,							BC
7					3							BD,, BD,, BD,,							
8												BD,, JN,,							
9					4							BD,, BD,,							BC
10		END OF DRILLHOLE		9.76															
11																			
12																			
13																			
14																			

DEPTH SCALE
1 : 50

LOGGED:
CHECKED:

GTA-RCK 046 S:\CLIENTS\NORR\MASSEY_LIBRARY_RMC_KINGSTON_ON02_DATA\GINT\MASSEY_LIBRARY_RMC_KINGSTON_ON.GPJ GAL-MISS.GDT 9/16/24

PROJECT: CA0039719.6922

LOCATION: N 4898568.00; E 382720.00

RECORD OF BOREHOLE: 24-02

SHEET 1 OF 1

BORING DATE: August 21, 2024

DATUM:

DRILL RIG: Portable Hilti DD200

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ● / rem V. ⊕ U - ○)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp W WI; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION / GRAIN SIZE DISTRIBUTION (%)
		GROUND SURFACE		0.00							GR SA SI CL
0	Portable Hand Auger / Portable Rock Core	Terrazzi floor over concrete sub-floor CONCRETE SLAB		0.07							
		FILL - (GP) GRAVEL, clear crushed stone; grey; non-cohesive, wet		0.37	1	GS	-				
				0.53							
1	Portable BW	FILL - (ML/CI) SILTY SAND to SILTY CLAY, some gravel; brown-grey to grey, fissured, disturbed; cohesive, w>PL, very soft			2	SS	3				
					3	SS	6				
2					4	SS	1				
		- Cobble present - Trace rootlet present			5	SS	40				
3					6	SS	50/0				
		END OF BOREHOLE		3.10							
		For rock coring details refer to Record of Drillhole 24-02.									
		NOTE:									
		1. A hammer weight of 47 lbs was used with a standard drop of 30 in.									
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

GTA-BHS 005 S:\CLIENTS\NORR\MASSEY_LIBRARY_RMC_KINGSTON_ON\02_DATA\GINT\MASSEY_LIBRARY_RMC_KINGSTON_ON.GPJ GAL-MIS.GDT 9/16/24

PROJECT: CA0039719.6922

RECORD OF DRILLHOLE: 24-02

SHEET 1 OF 1

LOCATION: N 4898568.00 ;E 382720.00

DRILLING DATE: August 21, 2024

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Hilti DD200

DRILLING CONTRACTOR:

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE % (80 60 40 20)	RECOVERY: SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER (5 10 15 20)	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATH-ERING INDEX (W1 W2 W3 W4 W5 W6)	Diametral Point Load Index (MPa) (2 4 6)	FEATURES	PIEZOMETER
		Continued from Record of Borehole 24-02																	
4		Fresh, bedded, grey, fine-medium grained, slightly porous to non-porous, medium strong, LIMESTONE, trace shale		3.10	1							BD,, BD,, BD,, BD,, BD,, BD,, BD,, BD,,							BC
5					2							BD,, BD,, BD,, BD,,							
		END OF DRILLHOLE		5.26														BC	
6																			
7																			
8																			
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE

1 : 50

LOGGED:

CHECKED:

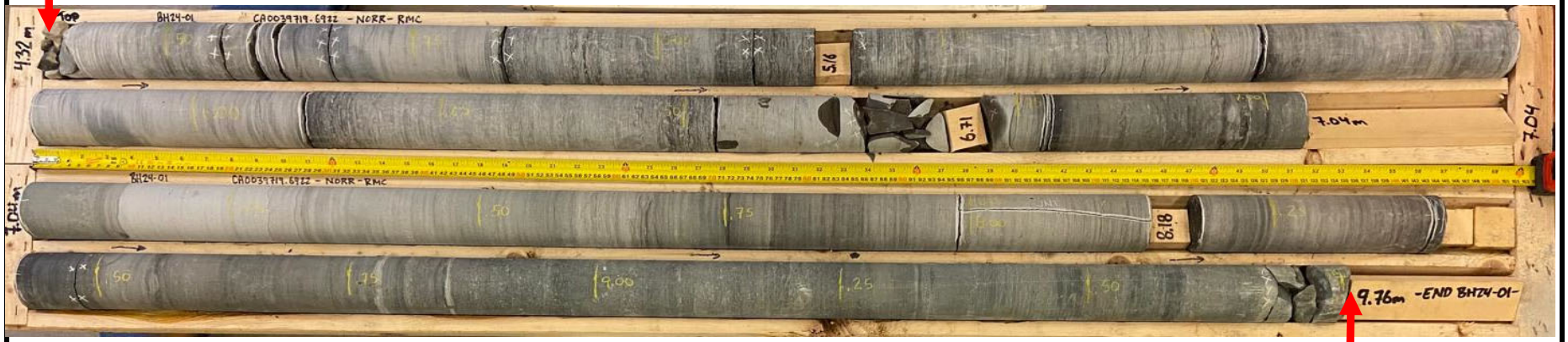
GTA-RCK 046 S:\CLIENTS\NORR\MASSEY_LIBRARY_RMC_KINGSTON_ON\02_DATA\GINT\MASSEY_LIBRARY_RMC_KINGSTON_ON.GPJ GAL-MISS.GDT 9/16/24

APPENDIX B

Rock Core Photos

BH 24-01 (Dry)
Core Box 1 of 1

Top of Bedrock depth 4.3 m



Depth 9.8 m EOH

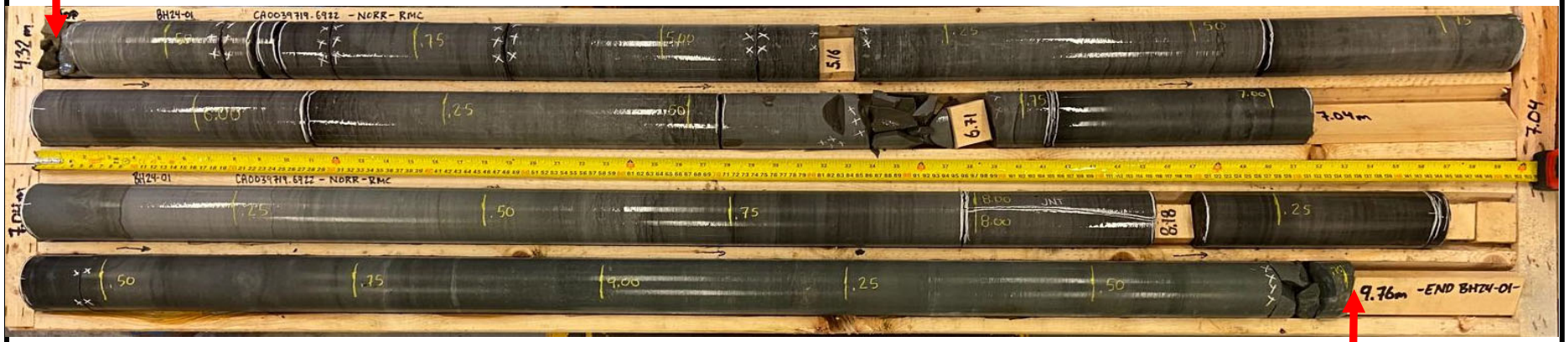
Project No.	CA0039719.6922
Drawn:	KG
Date:	2024-08-27
Checked:	OB
Review:	JSA

Massey Library Retrofit
RMC Kingston

Figure A1

BH 24-01 (Wet)
Core Box 1 of 1

Top of Bedrock depth 4.3 m



Depth 9.8 m EOH

Project No.	CA0039719.6922
Drawn:	KG
Date:	2024-08-27
Checked:	OB
Review:	JSA

Massey Library Retrofit
RMC Kingston

Figure A2

BH 24-02 (Dry)
Core Box 1 of 1



wsp	**Massey Library Retrofit** **RMC Kingston**	Project No. CA0039719.6922 Drawn: KG Date: 2024-08-27 Checked: OB Review: JSA

Figure A3

BH 24-02 (Wet)
Core Box 1 of 1

Top of Bedrock Depth 3.1 m



Depth 5.3 m EOH

wsp	**Massey Library Retrofit** **RMC Kingston**	Project No. CA0039719.6922 Drawn: KG Date: 2024-08-27 Checked: OB Review: JSA	**Figure A4**

APPENDIX C

Laboratory Results

TABLE 1
SUMMARY OF WATER CONTENT DETERMINATIONS (ASTM D2216/LS-701)

PROJECT NUMBER	CA0039719.6922
PROJECT NAME	Massey Library Retrofit - RMC Kingston
DATE TESTED	September 3, 2024

Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)
24-01	1A	0'0"-1'0"	0.00-0.30	18.2%					
24-01	1B	1'0"-2'0"	0.30-0.61	24.1%					
24-01	2	2'6"-4'6"	0.76-1.37	20.5%					
24-01	3	5'0"-7'0"	1.52-2.13	36.5%					
24-01	4	7'6"-9'6"	2.29-2.90	31.7%					
24-01	5	10'0"-12'0"	3.05-3.66	41.8%					
24-01	6	12'6"-14'2"	3.81-4.32	34.0%					
24-02	2	1'9"-3'9"	0.53-1.14	27.3%					
24-02	4	6'0"-8'0"	1.83-2.44	23.0%					
24-02	5	8'0"-10'0"	2.44-3.05	36.9%					

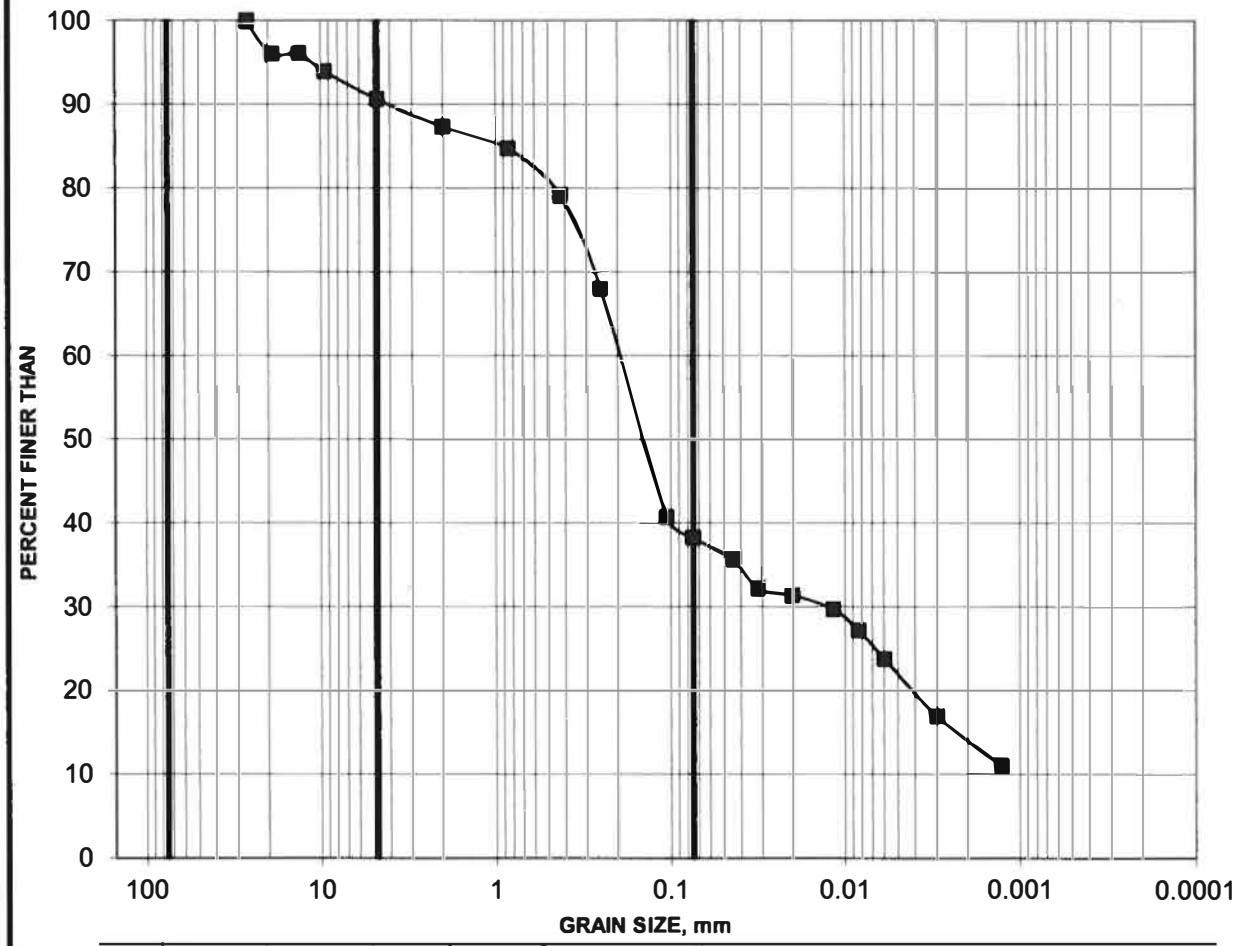
wsp

https://wsponlinecan.sharepoint.com/Sites/Global-OttawaLab/Shared Documents/Active/CA0039719.6922/

Tested By: CW
Checked By: MJ

GRAIN SIZE DISTRIBUTION	FIGURE

FILL



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

	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
—■—	24-02	2	0.53-1.44	9	53	24	14

Project: CA0039719.6922

wsp

Created by: CW

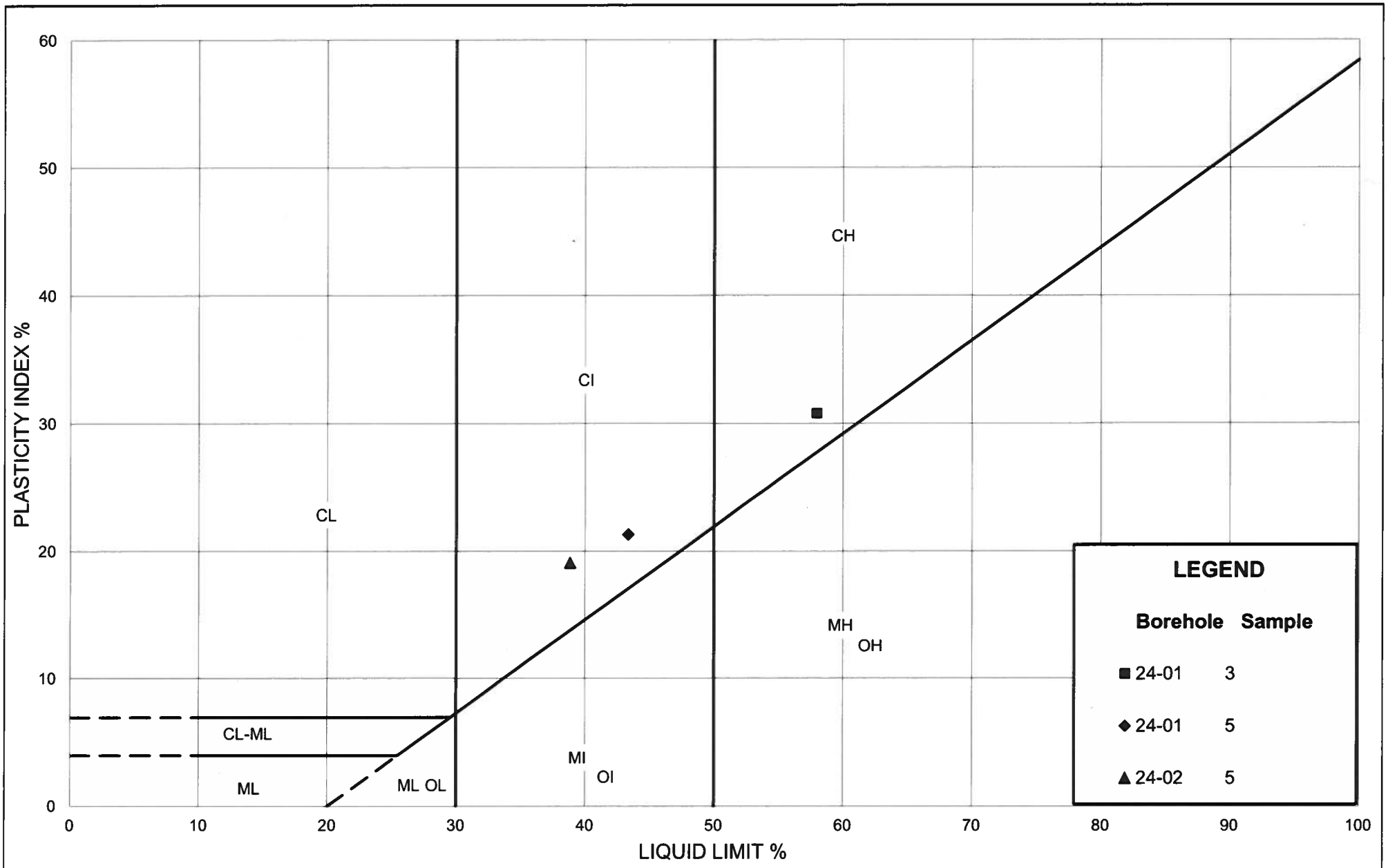
Checked by:

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

PROJECT NUMBER	CA0039719.6922
PROJECT NAME	Massey Library Retrofit - RMC Kingston
DATE TESTED	September 11, 2024

Borehole No.	Sample No.	Depth (m)	Water Content (%)	Atterberg Limits			
				W_L	W_P	LI	PI
24-01	3	1.52-2.13	36.50	58.0	27.2	0.3	30.8
24-01	5	3.05-3.66	41.80	43.4	22.1	0.9	21.3
24-02	5	2.44-3.05	36.90	38.9	19.8	0.9	19.1

Tested By: CW
Checked By: MA



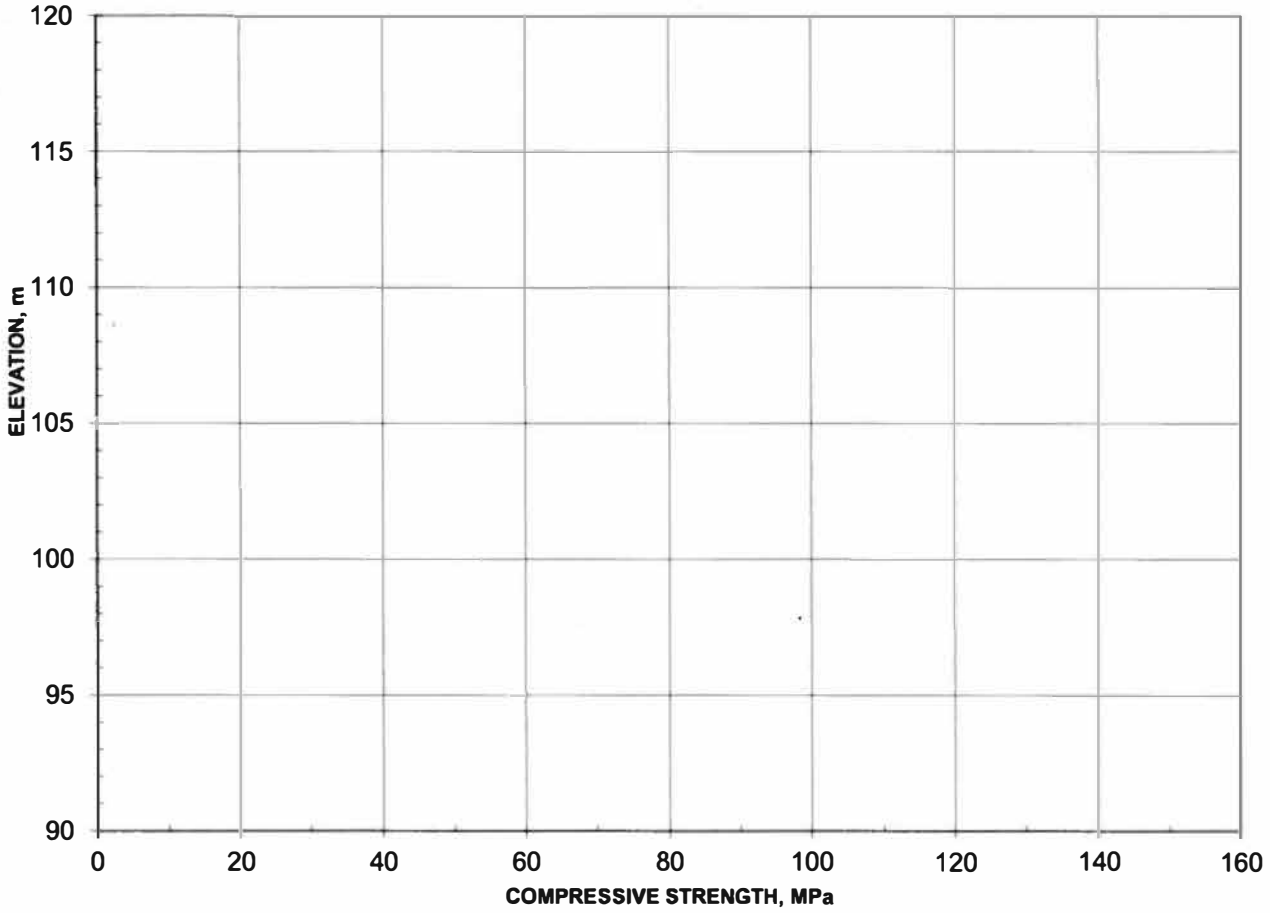
wsp

PLASTICITY CHART

Figure:	
Project:	CA0039719.6922
Created By: CW	Checked By:

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

FIGURE



Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
24-01 RC1	4.5	2	2718	Limestone	193	1
24-02 RC1	3.3	2.3	2698	Limestone	250	1

Notes:

Failure Types

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

Remarks

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

Project: CA0039719.6922

https://wsponlinecan.sharepoint.com/Sites/Global-OttawaLab/Shared Documents/Active/CA0039719.6922/

Created by: CW
Checked by:

Certificate of Analysis

eurofins | Environment Testing

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

Report Number: 3010712
Date Submitted: 2024-09-05
Date Reported: 2024-09-12
Project: CA0039719.6922
COC #: 916549

Dear Othmane Benkirane:

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

Report Comments:

Emma-Dawn Ferguson
2024.09.13 17:45:41 -04'00'

APPROVAL: _____
Emma-Dawn Ferguson, Chemist

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

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

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

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

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

Certificate of Analysis

eurofins | Environment Testing

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

Report Number: 3010712
Date Submitted: 2024-09-05
Date Reported: 2024-09-12
Project: CA0039719.6922
COC #: 916549

					Lab I.D.	1741880	1741881
					Sample Matrix	Soil153	Soil153
					Sample Type		
					Sampling Date	2024-08-19	2024-08-21
					Sample I.D.	24-01 Sa3/5-7'	24-02 Sa4/6'-8'
Group	**Analyte**	**MRL**	**Units**	**Guideline**			
Anions	Cl	0.002	%			0.007	0.002
	SO4	0.01	%			<0.01	0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm			0.21	0.21
	pH	2.00				8.06	7.99
	Resistivity	1	ohm-cm			4762	4762

Guideline = *** = Guideline Exceedence**

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

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

Certificate of Analysis

eurofins | Environment Testing

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

Report Number: 3010712
Date Submitted: 2024-09-05
Date Reported: 2024-09-12
Project: CA0039719.6922
COC #: 916549

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 465477 **Analysis/Extraction Date** 2024-09-09 **Analyst** M B **Method** AG SOIL			
SO4	<0.01 %	102	70-130
Run No 465532 **Analysis/Extraction Date** 2024-09-10 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	92	90-110
pH	6.21	100	90-110
Resistivity			
Run No 465656 **Analysis/Extraction Date** 2024-09-12 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %	92	90-110

Guideline = *** = Guideline Exceedence**

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

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

APPENDIX D

Vertical Seismic Profile Technical Memorandums

TECHNICAL MEMORANDUM

Client:	NORR Architects & Engineers Ltd.		
Project:	Geotechnical Investigation – Massey Library Retrofit – RMC Kingston, ON	**WSP Reference:**	CA0039719.6922
Subject:	Factual Report – Vertical Seismic Profile (VSP) for Determination of Site Specific Vs30	**Date:**	December 09, 2024
Recipient:	Hassan Saffarini, Ph.D., P.Eng., PMP, CAHP (NORR)		
cc:	Othmane Benkirane, P.Eng. (WSP)		

1 Introduction

WSP Canada Inc. was retained by NORR Architects & Engineers Ltd. to perform a seismic investigation to determine the Vs30 value at the Massey Library at the Royal Military College (RMC) in Kingston, ON. The assessment was performed using the Vertical Seismic Profile (VSP) method which is sometimes referred to as the down-hole method. The test was performed in borehole BH24-01 (Figure 1) on August 28, 2024. The methodology, results are discussed in this technical memorandum.

2 Methodology

Seismic investigations conducted on the surface for engineering purposes are generally non-intrusive, relatively fast to perform and are cost-effective when used to produce cross-sectional information at a site. Typical depths of investigation for geotechnical applications range from 5 meters to 30 meters. Several methodologies can be applied to analyse different components of the seismic wave-field and maximize the information to be extracted.

Due to the desired accuracy of the shear-wave velocity measurements, the Vertical Seismic Profile method was selected for the Massey Library.

2.1 Vertical Seismic Profile (VSP)

The objective of a vertical seismic profile (VSP) or downhole seismic survey is to obtain high-resolution seismic wave velocities for a particular site's stratigraphy. VSP testing is conducted by placing a tri-axial geophone in a borehole and using an active source, such as a sledgehammer, to generate a seismic wave.

A VSP determines the compressional (P) and shear (S) wave velocity of subsurface stratigraphy for different depths by measuring the arrival of each wave type. A schematic of a VSP test can be found in Figure 2.

The results are typically used for soil and rock mechanics, foundation and earthquake engineering investigations.

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F: +1-438-843-8111

WSP.COM



Figure 1: Map view of project site with the location of BH24-01 containing the VSP in yellow.

2.1.1 Data Acquisition

Acquisition of the seismic data for this investigation was performed using the Geometrics Geode Seismograph (**Appendix A** for the equipment data sheet) with a Geostuff Wall-Lock BHG-2 Borehole Geophone. The acquisition parameters are summarized in Table 1.

Table 1 : Summary of seismic acquisition parameters.

Parameter	Setting
Sample Rate	20.83µs
Record Length	0.33s
Geophone spacing	0.5m
Trigger Type	Hammer Switch
Source Type	Sledgehammer (3.6kg)

2.1.2 Data Analysis

The workflow to conduct the analysis of seismic data acquired during a VSP is as follows:

1. Inspect seismic shot records;
2. Perform pre-processing on shot records (e.g., Remove bad traces, apply filters and mutes as required);
3. Apply survey geometry;
4. Pick S-wave first breaks from horizontal components of boreholes geophone (Figure 3);

5. Pick seismic layers based on changes in slope of first breaks for the S-wave component.

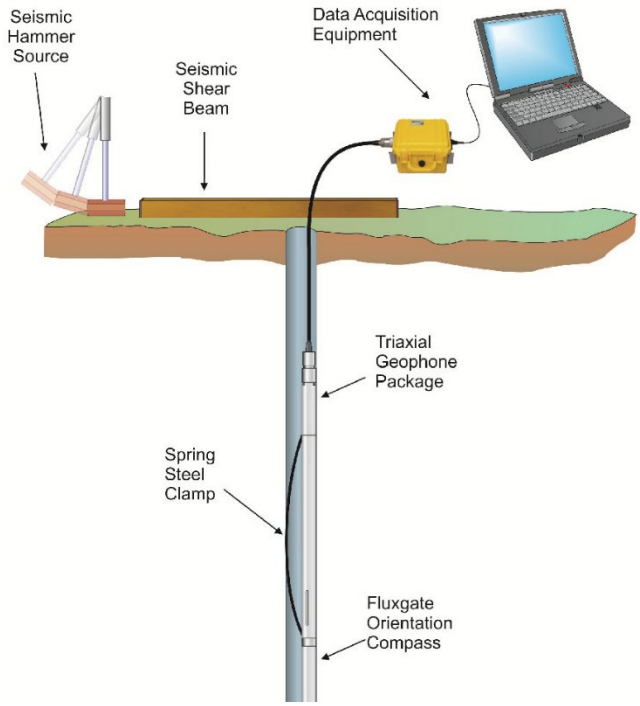


Figure 2: Schematic of VSP acquisition.

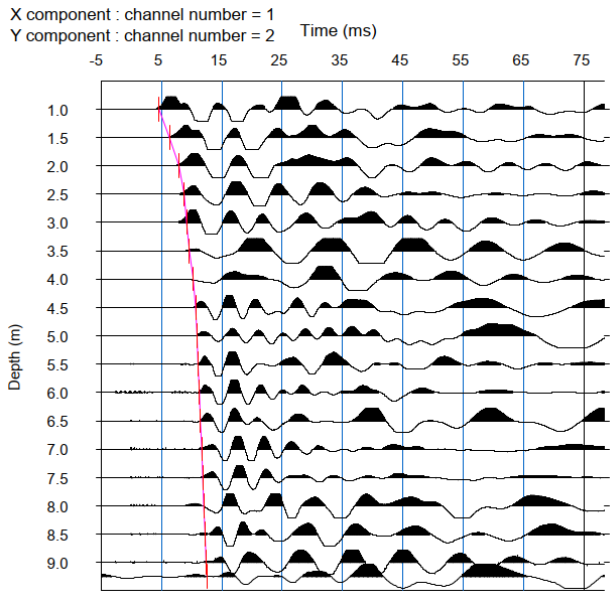


Figure 3: VSP results from combiner horizontal components. The red line shows the picked first breaks of the S-wave.

3 Results

Analysis of the vertical and horizontal components of the borehole geophone produced the results shown in Table 2. At the time the study was performed the Vs30 value for a slab-on-grade foundation was calculated to be 937.7 m/s. Note, to complete the Vs30 calculation, the deepest measured velocity (1266.6 m/s) was extrapolated from the measurement depth of 9.25 metres to a depth of 30 metres.

Table 2 : P- and S-wave velocity results from VSP analysis.

Interval top (m)	Interval bottom (m)	P-wave Velocity (m/s)	S-wave Velocity (m/s)
0.0	2.0	380.3	187.2
2.0	4.3	434.4	243.3
4.3	30	471.0	348.2
2.5	3.0	586.3	374.2
3.0	3.5	1462.0	390.8
3.5	4.0	1598.3	401.6
4.0	4.5	1696.0	824.0
4.5	5.0	1765.0	1120.4
5.0	5.5	4673.3	1159.0
5.5	6.0	4972.2	1187.7
6.0	6.5	5210.9	1209.2
6.5	7.0	5401.4	1225.6
7.0	7.5	5553.8	1238.4
7.5	8.0	5676.5	1248.3
8.0	8.5	5775.8	1256.3
8.5	9.0	5856.7	1262.6
9.0	9.25	5908.1	1266.6
9.25	30	5908.1	1266.6

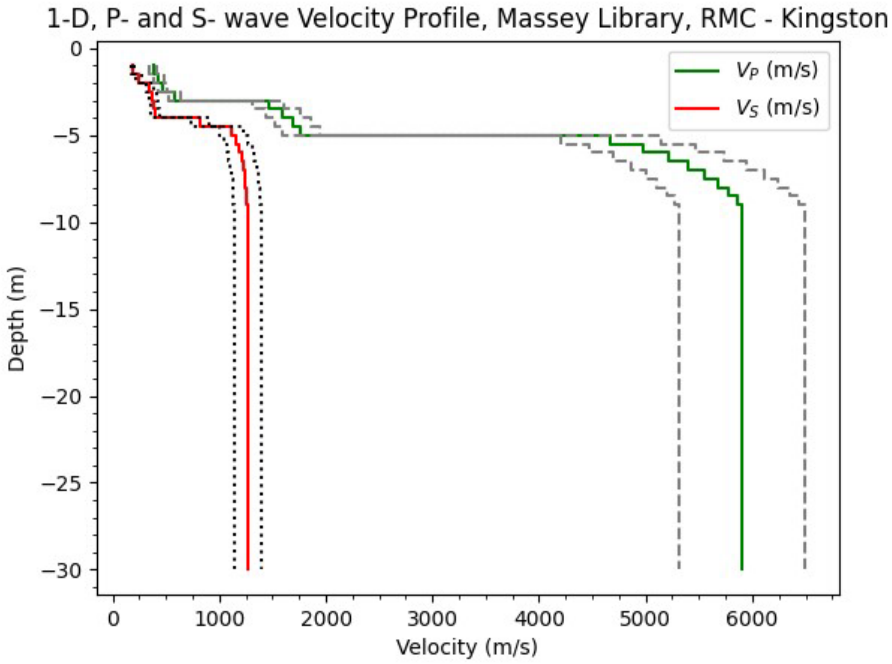


Figure 4: Plot showing measured seismic P- and S-wave velocities from surface to a depth of 30 metres. The deepest velocity measurements were extrapolated to perform the calculation of Vs30. The dashed line shows a 10% error margin inherent in such analyses.

4 Closure

We trust that this technical memorandum fulfills your requirements. Should you wish to discuss the results or any further requirements that you may have, please do not hesitate to contact the undersigned.

PREPARED BY



December 09, 2024

Andrew Nicol, M.Sc.
Professional in Geophysics

Date

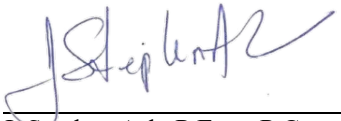
REVIEWED BY



December 09, 2024

Othmane Benkirane, MASc., P.Eng.
Geotechnical Engineer

Date



December 09, 2024

J. Stephen Ash, P.Eng., P.Geo.
Senior Principal Geotechnical Engineer

Date

APPENDIX A

Equipment Datasheets

Geostuff

Wall-Lock Borehole Geophones

- **3-component tri-axial sensors**
- **Motor-driven clamp mechanism**
- **Fits in 2-inch (51-mm) boreholes**
- **Automatic orientation of horizontal geophones to any azimuth (Model BHG-3)**
- **Cable disconnects for convenient surface handling and extending depth**
- **Works in wet or dry holes**



Geostuff's BHG series, 3-component borehole geophones are designed for shallow seismic velocity measurements. Both units include a motor-driven clamp to hold the sensor in position in the borehole.

These geophones are applicable to a wide variety of shallow surveys, including shothole logging, downhole shear wave measurements, static corrections for petroleum shear-wave reflection surveys, cross-hole, tomographic, seam wave, and shallow VSP surveys for coal, minerals, and rock mechanics.

Model BHG-3 includes a fluxgate compass and servo mechanism which automatically orients the horizontal geophones to any magnetic azimuth selected by the operator. Thus, the longitudinal sensor can be aligned with the polarization of the shear wave source.

While downhole shear wave surveys have traditionally been done with random orientation, being able to precisely align one of the horizontal geophones with the plank or energy source provides significant advantages. Anisotrophy appears to be much more common than originally thought, and the velocities of horizontally polarized shear waves vary with azimuth. With an orientable geophone, these velocity variations may be measured or simply avoided. By maintaining orientation of the source-receiver combination all the way down the borehole, the user can maintain better control and recognition of shear wave arrivals.

Download a free copy of our tutorial paper "**Borehole Shear-Wave Surveys for Engineering Site Investigations**" at http://www.geostuff.com

The clamping mechanism is a steel leaf spring, compressed by a motor-driven piston. When compressed, the spring expands, forcing the geophone against the borehole wall.

The tool may be used in soft-wall, uncased holes as well as cased holes. In the unlikely event of a failure to release, the tool may be dragged up the hole against the spring friction. The motor-driven spring is faster, more reliable, and less cumbersome than the common alternative using an inflated bladder.



Control Electronics:

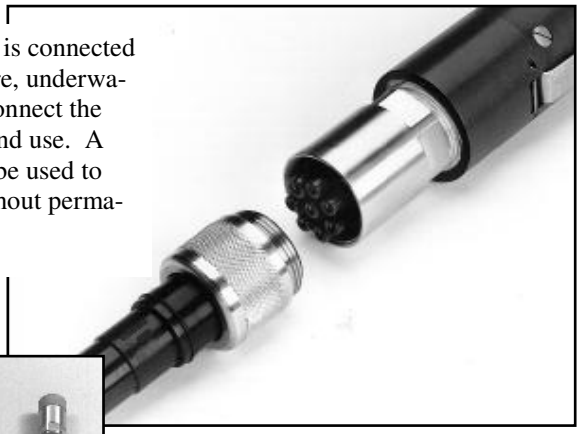
The BHGC-1b controller directs the voltages to control the clamping mechanism and servo mechanism. A meter monitors motor current to indicate the clamping action and force. This unit is usable with either model.

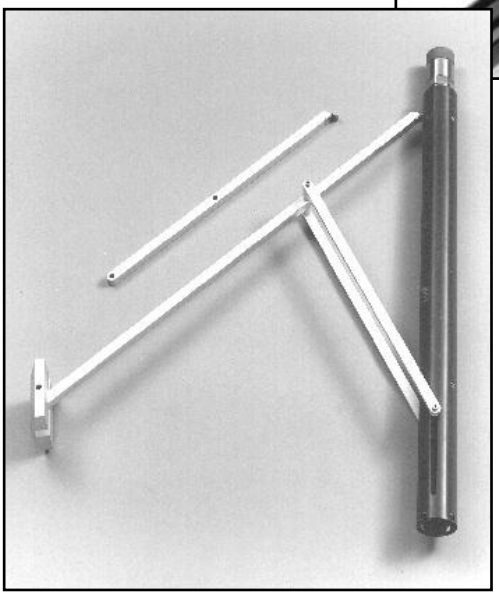
A rechargeable, internal. 24-volt battery is supplied, along with a 110/220 volt charger.

A Model BHGC-4 which can control up to four borehole geophones is also available.



Removable connector: The cable is connected to the geophone by a high-pressure, underwater connector. The ability to disconnect the sensor makes it easier to handle and use. A male-female extension cable can be used to temporarily extend the length without permanently





Big Hole Adaptor: An optional mechanical arm and pressure foot (shown here on the BHG-2) can be substituted for the spring to clamp the tool in large diameter boreholes. Conversion is easily done by the user in minutes. Multiple arms provide for various diameter holes. Largest arm extends diameter up to 480 mm (19 inches). When retracted, will fit inside a 75-mm (3-inch) diameter borehole.

Common Specifications

Number of geophones: 1 vertical and 2 horizontal in an X-Y-Z configuration

Natural Frequency: 15-Hz high-output omnidirectional is standard; 10, 28 and 40 Hz optional

Pressure rating: 300 meters (1000 ft) water depth, consult factory for deeper options

Clamp mechanism: DC motor. Requires 24 volts DC on surface (or more, depending on cable length). Requires ½ amp when moving spring (1 amp peak at clamping)

Expanded diameter: 18 cm (7 in) total diameter including probe body.

BHG-2 Borehole Geophone

Diameter: 48 mm (1.9 in)
Length: 700 mm (27.5 in)
Weight: 2 kg (4 lb)

BHG-3 Borehole Geophone

Diameter: 48 mm (1.9 in)
Length: 1.1 m (44 in)
Weight: 3.4 kg (7 lb)

Compass: fluxgate sensor, powered from same DC voltage as clamp mechanism.

Maximum inclination: +/- 90 degrees from vertical with standard 15-Hz sensors, much less with optional sensors.

Orientation Accuracy: better than 5 degrees

The flux gate compass will not function in steel-cased boreholes.

Cable

7-conductor, with two copper and 5 copperweld conductors, Kevlar-reinforced, polyurethane-jacket cable, with Reed Products SU-8 female connector molded on wet end.

Specifications are subject to change without notice for product improvement or other considerations. For more information, contact:

GEOSTUFF
1579 Lupine Lane
Lincoln, CA 95648
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info@geostuff.com
www.geostuff.com

Geode
Exploration Seismograph

GEOMETRICS
Innovation • Experience • Results



It is no wonder that over 2,700 Geodes have been sold. It is the most versatile and flexible seismograph available. Small and lightweight enough to pack in your suitcase, it expands easily for full-scale 2D and 3D surveys at a cost your bottom line will love. When you are not using the Geode for reflection, refraction, MASW/MAM, or tomography surveys, use it for monitoring earthquakes and other passive sources. The Geode will even do marine profiling or continuous recording. It is the most popular engineering seismograph in the world, and is widely used throughout the academic and research community.

For light-duty applications, you can use your laptop to view, record and even process your data. In harsh conditions, control your Geodes with Geometrics' StrataVisor NZ/C series computers and seismographs. You can connect Geodes together to build systems of over 1,000 channels. Geodes are shock-proof, dust-proof, submersible and able to withstand extreme temperatures.

Fifteen years on, we can say with confidence that the Geode is the most reliable seismograph we have ever produced. Because of this, we can offer a 3-year warranty backed by Geometrics, now in our 48th year of providing prompt, knowledgeable customer support.

FEATURES & BENEFITS

- **Bulletproof** - Not really, but almost. Survives 1.5m drop onto concrete in 14 orientations. The Geode comes standard with a 3-year warranty.
- **Distributed architecture** - Use standard 24-pair geophone cables, no matter how many channels.
- **Ultra-wide bandwidth** - Useful for everything from crosshole surveys to earthquake monitoring.
- **Geophone and line testing** - No need for time-consuming "tap test".
- **Versatile** - Configure systems ranging from 8 to 1000 channels.*
- **Waterproof and dustproof** - No need to pick up the system in a sudden rain or dust storm.
- **High temperature range** - Use in the Sahara, Amazon or at the North Pole.
- **GPS synchronization** - Sub-sample timing accuracy so you know exactly when an event occurs.

* Systems can be expanded temporarily via Geometrics' rental pool or existing loaner networks.

SPECIFICATIONS | Geode Exploration Seismograph

Configurations: 8, 12, 16, or 24 channels in weatherproof field-deployable Geode module. Geode is operated from either Windows XP/7/10-based laptop or by Geometrics' ruggedized StrataVisor NZ field computer/seismograph. Basic operating software controls one Geode. It can also be optionally expanded to control multiple Geodes, as well as do marine surveying, continuous recording, GPS synchronization, and seismic surveillance.

A/D Conversion: 24-bit result using Crystal Semiconductor sigma-delta converters and Geometrics proprietary over sampling.

Dynamic Range: 144 dB (system), 110 dB (instantaneous, measured) at 2 ms, 24 dB.

Distortion: 0.0005% @ 2 ms, 1.75 to 208 Hz.

Bandwidth: 1.75 Hz to 20 kHz. 0.6 and DC low frequency option available.

Common Mode Rejection: > 100 dB at <= 100 Hz, 36 dB.

Crosstalk: -125 dB at 23.5 Hz, 24 dB, 2 ms.

Noise Floor: 0.20 µV, RFI at 2 ms, 36 dB, 1.75 to 208 Hz.

Stacking Trigger Accuracy: 1/32 of sample interval.

Maximum Input Signal: 2.8 V PP, 0 dB.

Input Impedance: 20 kOhm, 0.02 µf.

Preamplifier Gains: Standard factory configuration is 24 and 36 dB. Optional configurations include 12 and 24 dB or 0 dB.

Anti-alias Filters: -3 dB at 83% of Nyquist frequency.

Acquisition and Display Filters:

- **Low Cut:** OUT, 10, 15, 25, 35, 50, 70, 100, 140, 200, 280, 400 Hz, 24 or 48 dB/octave, Butterworth.
- **Notch:** 50, 60, 150, 180 Hz and OUT, with the 50 dB rejection bandwidth 2% of center frequency.
- **High Cut:** OUT, 32, 64, 125, 250, 500 or 1000 Hz, 24 or 48 dB/octave.

Sample Interval: 0.02, 0.03125, 0.0625, 0.125, 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, 16.0 ms.

Correlation: Optional (with SGOS, standard with MGOS) high-speed hardware correlator available in each Geode for fast cycle time with vibrators and pseudo-random sources. Correlates 16K record, unlimited channels, in under 1 second.

Record Length: 16,384 samples standard, 65,536 samples optional.

Pre-trigger Data: Up to full record length.

Delay: Full record length to +100 sec.

Data Transmission: Uses Ethernet transmission standard over CAT-5 copper or multimode fiber-optic cable. Distance between boxes: CAT 5 cable up to 0.25 km; fiber-optic cable up to 1.5 km.

Event Trigger: Based on seismic event; criteria set by user.

Continuous Recording (optional): Record GPS-synchronized, gapless data in SEG-2 format.

Auxiliary Channels: All Geode channels can be programmed as either AUX or DATA.

Roll-along: Built-in, no external roll box required.

Geophone Testing: Pulse test measures resistance, sensitivity, natural frequency, and damping.

Instrument Tests: Optional analog testing available. Measure noise, crosstalk, CMR, dynamic range, gain similarity and trigger accuracy. Additional built-in oscillator required.

Data Formats: SEG-2 standard. SEG-D and SEG-Y available as options.

System Software: Basic operating software includes full compliment of acquisition, display, plotting, filtering and storage features. Numerous optional features available; see SCS data sheet.

Bundled Applications Software: SeisImager/2D Lite refraction analysis software from OYO.

Data Storage: Stores data locally in SEG-2 on laptop/PC media. Drivers available for tape/disk storage in SEG-2/D/Y.

Plotters: Drives any Windows-compatible plotter or printer.

Triggering: Positive/negative TTL or contact closure, software adjustable threshold. STA/LTA-like algorithm for triggering on seismic waveform.

Power: Requires 12V external battery. Uses 0.5 W/channel during acquisition (0.25 ms sample rate). A single 12 Amp-hour battery is sufficient for a typical day of data acquisition; standby mode reduces power consumption by 70%.

Environmental: Operates from -50ºC to +70ºC (-58ºF to +158ºF). Waterproof and submersible. Withstands a 1m drop onto concrete on 6 sides and 8 corners. Passes MIL810E/F vibration.

Physical: L: 25.4 cm; W: 30.5 cm; H: 17.75 cm; Weight: 3.6 kg (10x12x7 in; 8 lb). Uses waterproof Bendix 61-pin connector for geophone input.

Operating System: Windows XP/7/10.

Warranty: Three years standard, extended warranty available.

Optional Built-In Test Functions

Instrument:

- Noise
- DC Offset
- Gain Accuracy
- Gain and Phase Similarity
- Distortion
- Crossfeed
- CMR
- Bandwidth
- Timing Accuracy

Geophone:

- Natural Frequency
- Resistance
- Damping
- Sensitivity

Specifications subject to change without notice. GeodeDS_v1 (0518)

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