REPORT

Geotechnical Investigation

LeBreton Library Parcel
665 Albert Street
Ottawa, Ontario

Submitted to:

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Submitted by:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Dream Impact Master LP (Dream) to conduct a geotechnical investigation at the property located at 665 Albert Street. The site is located north of Albert Street, east of Booth Street, south of the Fleet Street Aqueduct (open aqueduct), and west of the site of the new Ottawa Public Library (currently under construction). A Site Location Plan is attached as Figure 1.

The purpose of this investigation was to assess the general subsurface and groundwater conditions within the study area by means of a limited number of boreholes and associated laboratory testing. Based on an interpretation of the factual information obtained during the current investigation, along with the existing subsurface information available for the site from previous investigations, a general description of the soil and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions. A Phase Two Environmental Site Assessment was completed concurrently with the geotechnical investigation, the results of which are presented under separate cover.

The reader is referred to the 'Important Information and Limitations of This Report' which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

It is understood that the proposed new development is an irregularly shaped structure which will consist of a four-storey podium filling the entire site, above which will be two 30 to 35 storey residential towers, covering a portion of the site. The development will include 2 levels of underground parking (below the entire footprint of the podium). The lowest level of parking is indicated to have a finished floor elevation of 53.6 m. The main ground floor of the development is indicated to be at an elevation of 62.0 m.

The site is currently vacant and forms part of the larger LeBreton Flats area which included a variety of historical industrial uses (past uses of the property are discussed in detail in the Phase One and Two Environmental Site Assessments. The site is unsurfaced and is relatively flat with existing ground elevations ranging from 60.5 m to 62.9 m (based on spot elevations at borehole locations).

Based on the results of previous investigations and the published geology maps available from the Geologic Survey of Canada (GSC) for this area, the subsurface conditions at this site are expected to consist of a surficial layer of fill, overlying a thick deposit of glacial till. The glacial till is underlain by interbedded limestone and shale bedrock of the Verulam formation.

3.0 PROCEDURE

3.1 Desktop Study

A previous geotechnical investigation was completed at the site by Golder Associates in 2011. This investigation included six boreholes located within the subject site. The boreholes (BH11-33, BH11-35 and BH11-37 to BH11-40) have been used to supplement the current investigation. The locations of these previous boreholes are shown on Figure 1. Copies of the previous borehole logs are included in Appendix A.

Based on the results of previous investigations and the published geology maps available from the Geologic Survey of Canada (GSC) for this area, the subsurface conditions at this site are expected to consist of a surficial layer of fill, overlying a thick deposit of glacial till. The glacial till is underlain by limestone and shale bedrock of the Verulam formation.

3.2 Field Investigation

The fieldwork for this current investigation was carried out between February 14th and 24th, 2022. During that time, a total of five boreholes (BH22-01 to BH22-05) were advanced at the approximate locations shown on Figure 1.

The boreholes were advanced using a track-mounted CME-55 hollow-stem auger drill rig with diamond coring capabilities supplied and operated by Downing Drilling of Hawkesbury, Ontario. The boreholes were advanced to depths ranging from 12.2 m to 16.5 m below the existing ground surface using a combination of auger drilling and diamond coring using NQ sized core barrels. Standard Penetration Tests (SPTs) were carried out within the overburden at regular intervals of depth. Samples of the soils encountered were recovered using 35 mm diameter split-spoon sampling equipment.

The fieldwork was supervised by technicians from our staff who located the boreholes, directed the drilling and in-situ testing operations, logged the boreholes and samples, and took custody of the soil and bedrock samples retrieved. On completion of the drilling operations, the soil samples were transported to our laboratory for further examination and laboratory testing. Laboratory testing was carried out on selected soil samples, including natural water content and grain size distribution tests. Basic chemical analysis related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements was also completed on selected soil samples.

The borehole locations were selected in consultation with the City of Ottawa, marked in the field, and subsequently surveyed by City of Ottawa personnel. The geodetic reference system used for the survey is the North American Datum of 1983 (NAD83). The borehole coordinates are based on the Modified Transverse Mercator (MTM Zone 9) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is presented as follows:

- Borehole records are provided in Appendix A.
- Results of water content testing are shown on the relevant borehole logs; results of grain size distribution tests are presented in Figures 2 and 3.
- Results of basic chemical analysis related to corrosivity are included in Appendix B.
- The results of the MASW testing are included in Appendix C.

The Record of Borehole sheets describe the subsurface conditions at the borehole locations only.
The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling in some cases, observations of drilling progress as well as results of SPTs and, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface soil, bedrock and groundwater conditions will vary between and beyond the borehole locations.

Unless otherwise noted, the following sections present an overview of the subsurface conditions encountered in the boreholes advanced during the current investigation. It should be noted that the shallow subsurface conditions noted on the borehole logs from the previous investigations may have changed since the boreholes were drilled, as such only auger refusal/bedrock depths and hydraulic response tests from previous drilling are discussed herein.

4.2 Overview of Subsurface Conditions

In general, the subsurface stratigraphy within the area of the investigation consists of surficial fill materials overlying glacial till, which in turn overlies limestone and shale bedrock.

4.3 Fill Material

Fill material was encountered in each of the boreholes from ground surface. The fill is heterogeneous in nature predominantly ranging from silty sand to sand. The fill also contains gravel, brick fragments, concrete and mortar fragments, glass, wood and layers of organic material and clay. Cobbles and boulders were also encountered during drilling. Fill material is, by its nature a heterogeneous material and other debris or obstructions could also be encountered with the fill.

SPT “N” values measured within the fill ranged from 6 to greater than 50 blows per 0.3 m of penetration during the two investigations (in 2011 and the current 2022 investigation). The SPT “N” values suggest that the fill has a highly variable very loose to very dense state of packing.

The fill material was fully penetrated in all of the boreholes at depths of between 2.1 and 3.7 m below the existing ground surface.

4.4 Glacial Till

A deposit of glacial till was encountered beneath the fill material at all of the boreholes. The glacial till typically consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sand and silt with a trace to some clay. Cobbles and boulders were encountered throughout the till during drilling and should be expected during construction.

The 2011 boreholes were terminated at auger and/or sampler refusal within the glacial till, and therefore did not fully penetrate the till layer. The five boreholes drilled during the 2022 investigation were all extended through the till and into the underlying bedrock, confirming the till extended to depth of 11.2 to 14.7 m.

SPT “N” values within the glacial till layer gave ‘N’ values ranging from 8 blows to greater than 100 blows per 0.3 m of penetration and are typically greater than 50 blows per 0.3 m of penetration suggesting the majority of the till has a dense to very dense state of packing. Very high blow counts, however, could be indicative of boulders and cobbles in the till rather than the state of packing.

Borehole 22-04 encountered a layer of “till-like” silty sand and sandy silt which was less dense and not as coarse as the till at lower depths (and at similar depths in the surrounding boreholes) between approximately 2.1 and 6.1 m depth.

4.5 Bedrock

The 2011 boreholes were terminated at refusal in the glacial till layer at depths of 4.2 m to 10.0 m below the existing ground surface. Based on the current 2022 boreholes, it is unlikely that the majority of these refusals were the result of encountering the bedrock surface and were more likely due to cobbles and boulders within the till.

The current 2022 boreholes were extended through the glacial till deposit into the underlying bedrock using rotary diamond drilling techniques, while retrieving NQ core. The depths and elevations to bedrock surface in the current investigation are summarized below:

Borehole No.	Ground Surface Elevation (masl)	Depth to Bedrock (m)	Elevation of Bedrock (masl)
22-01	62.9	14.7	48.2
22-02	62.5	14.2	48.3
22-03	61.7	11.2	50.5
22-04	60.5	11.2	48.3
22-05	62.3	11.2	48.3

The bedrock consists of limestone with shale interbeds of the Verulam formation. Additional description of the bedrock is provided on the Borehole records provided in Appendix A.

4.6 Groundwater Conditions

Monitoring wells were installed in boreholes 22-01 to 22-05 during the current investigation. The groundwater levels observed in the monitoring wells have been summarized in the following table:

Well ID	Geologic Unit of Screened Interval	Groundwater Level		Date of Measurement	Date Well Installed
		Depth (mbgs)	Elevation (masl)		
22-01	Glacial Till	7.8	55.1	February 25, 2022	February 15, 2022
22-02	Glacial Till	7.9	54.6	February 25, 2022	February 16, 2022
22-03	Bedrock	13.0	48.7	February 25, 2022	February 22, 2022
22-04	Glacial Till	10.7	49.8	February 25, 2022	February 22, 2022
22-05	Glacial Till	8.2	54.1	February 25, 2022	February 24, 2022

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.7 Corrosion Testing

Five samples of soil, one each from boreholes 19-101 and 19-102 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix C and are summarized below.

Borehole / Sample Number	Chloride (%)	Sulphate (%)	pH	Resistivity (Ohm-cm)
22-01 SA 2 (Fill)	0.003	0.02	9.4	3450
22-02 SA 3 (Fill)	0.011	0.02	8.3	3570
22-02 SA 9 (Till)	0.005	0.03	8.9	5000
22-03 SA 3 (Fill)	0.009	0.07	8.2	1520
22-04 SA 8 (Till)	0.005	0.03	9.1	5000

5.0 DISCUSSION AND GEOTECHNICAL RECOMMENDATIONS

This section of the report provides engineering information related to the geotechnical design aspects of the project based on our interpretation of the available subsurface information and on our understanding of the project requirements. The discussion below focuses on the development of the proposed structure.

The information in this portion of the report is provided for detailed design purposes in support of the design by the engineers and architects. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the site 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, schedule, equipment capabilities, costs, sequencing and the like.

This report addresses only the geotechnical aspects of the subsurface conditions at this site.
The geo-environmental (chemical) aspects, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, are outside the terms of reference for this report. The results of concurrent Environmental Site Assessment(s) for this project are provided under separate cover(s).

5.1 Site Grading

It is understood that a grade raise of up to 2.4 m is proposed at the site to match the proposed grade raise of Albert Street. The proposed grade raise is within acceptable limits for the soils at this site. A proposed grading plan was not available for review at the time of writing this report. The currently proposed ground floor level (Level 0) is indicated to have a finished floor elevation of 62.0 m. Based on elevations of the existing boreholes the current site grades are slightly above this (between 62 and 63 m elevation), with localized areas being lower (for example, BH22-02 and BH22-04 at 61.7 m and 60.5 m). The majority of the developed site will be excavated to accommodate the two floors of underground parking.

Based on the underlying soil conditions, there are no significant concerns with settlement due to the relatively minor grade raises required to develop the site.

5.2 Foundation Design

Based on the preliminary drawings provided, the entire footprint of the proposed development includes two floors of underground parking. The finished floor elevation of the lower (P2) parking level is indicated to be 53.6 m elevation. This compares with existing grades of approximately 62 to 63 m elevation based on the borehole elevations, and a bedrock surface at approximately 48 m elevation at the majority of borehole locations.

There are a number of options, from a foundation perspective:

- Assuming the lowest level of parking remains at 53.6 m elevation as shown on the drawings it will be within the glacial till. It is unlikely that the large 30 to 35 storey towers can be founded on conventional spread footings on the glacial till. Deep foundations (piles or caissons) would be appropriate for the high-rise towers. Deep foundations are discussed in Section 5.2.1 below.
- It may be feasible to found the lower podium structure (which is only 4 storeys) on shallow foundations (spread footings). Shallow foundations are discussed in Section 5.2.2 below.
- It would also likely be feasible to found the podium structure on a raft or mat foundation within the glacial till. Raft/mat foundations are discussed in Section 5.2.3 below.
- If the foundations (for the high-rise towers) can be lowered to bedrock (approximately 5 m lower than the current P2 level) it would be feasible to found the large towers on shallow foundations on bedrock).
- It may be feasible to found the entire development (podium and towers) on a single large raft within the till. A raft foundation suitable for the high-rise towers, however, would likely have a significant thickness (potentially several metres) to provide the rigidity required. Given, however, that there is only approximately 5 m of soil between the bedrock surface and the P2 level it is likely that it would be more cost-effective to simply found the building on rock than to construct a very thick continuous raft below the entire development.

5.2.1 Deep Foundations

Assuming foundation level cannot be lowered to bedrock, it is likely that at least the large high-rise towers would need to be founded on deep foundations. Typically, driven steel piles or cast-in-place concrete piles (with rock sockets) would be used.

Driven steel piles are typically more cost-effective for moderate vertical loads, but because of the short length (less than 5 m) they will provide almost no uplift or lateral resistance. Driven steel piles typically require larger groups of piles, with associated pile caps to resist larger loads. Cast-in-place concrete piles tend to be more expensive for resisting purely vertical loading but can provide very large lateral and uplift resistances. Cast-in-place concrete piles can also generate very high compressive resistances and therefore a single pile (or small group) can be used in place of a larger group of driven piles.

5.2.1.1 Driven Steel Piles

The proposed hospital structure may be supported on driven steel piles. Steel H-piles and closed-ended steel pipe piles are both commonly used in the area.

In general, the subsurface conditions in the vicinity of the proposed hospital building consist of variable deposits of fill with some localized areas of silty clay overlying a deposit of glacial till, overlying localized deposits of interlayered sands which in turn overlies shaley limestone bedrock. A piled foundation system could be used to transfer the foundation loads through the overburden soils to the underlying bedrock.

Axial Resistance

Piles driven to sound rock generate high ultimate geotechnical capacities, generally equal to or in excess of the structural capacity of the steel section (i.e., with increased loading or driving stresses, the steel section will become damaged and fail before the bedrock yields). For the purposes of design, the ultimate geotechnical resistance of the rock may be assumed to be equal to the ultimate resistance of the steel section.

A resistance factor of 0.4 should be applied to this value to obtain the factored resistance of a pile driven to sound rock. The resistance factor may be increased to 0.5 if a program of dynamic (PDA) testing is implemented, or 0.6 if static load testing is performed.

As an example, an HP310x79 has an ultimate resistance of 3,493 kN (based on the cross-sectional area, assuming 350 MPa yield stress and ignoring buckling, bending, lateral loads, sacrificial thicknesses or other more complex conditions which may reduce the structural capacity). The factored geotechnical resistance of an HP310x79 driven to sound rock may therefore be assumed to be 1,397 kN (3,493 kN x 0.4). A similar methodology may be used to estimate the geotechnical resistance of other pile sections.

Settlements for piles driven to sound rock are generally negligible, and the geotechnical resistance mobilized at 25 mm of settlement (a typical SLS condition) would be expected to exceed the factored axial resistance at ULS. Geotechnical SLS considerations therefore do not generally govern the design of pile driven to sound rock.

Piles spacings should not be less than three pile diameters (centre-to-centre) to prevent group effects. If closer pile spacings are required they can be accommodated, but the individual pile capacity may need to be reduced to account for the closer spacing. This can be reviewed in detailed design if required.

Uplift Resistance

The uplift resistance of a driven pile is a result of skin friction acting along the surface area of the embedded pile. The unfactored shaft resistance may be assumed to be equal to:

$$q_s = \beta\sigma_v'$$

Where:

q_s = the unfactored shaft resistance (in kPa);

β = a shaft resistance factor based on soil type and strength (use 0.8);

σ_v' = the vertical effective stress at the adjacent to the pile at depth z, equal to $z\gamma'$;

γ'= the effective unit weight of the soil which may be assumed to be 9 kN/m^3

A resistance factor of 0.3 should be applied to this value, to obtain the factored geotechnical uplift resistance. The dead weight of the pile itself, with an appropriate resistance factor for dead weight, may also be added to the geotechnical resistance in calculating the total uplift resistance.

The total uplift resistance of a pile group is the lesser of the sum of the individual pile resistances as described above, or the resistance of a single “block” of soil with a perimeter equal to the perimeter of the pile group (the mass of the soil inside the “block” may be included in the calculation; use a soil weight of 9 kN/m^3).

It should be noted that the uplift resistance of piles is highly dependent upon the installation of the piles as well as the layout of the pile groups. If the piles are relied upon to resist significant uplift loads, and uplift governs the design, consideration may be given to carrying out a tension test to confirm the uplift capacity.

Negative Friction

The raising of the grade or lowering of the groundwater table at or around the site may cause settlement of the existing soils. Localized settlement could also potentially be caused during a seismic event. In any of these cases, the potential will exist to develop negative skin friction (or downdrag) along the piles, and this should be considered in the design.

The magnitude of negative friction depends on the pile loading, pile dimensions and the final configuration of the site as well as the details of the below-grade portions of the building. The location of negative friction forces is also dependent on the location of the neutral axis of the pile which can only be determined once all of the pile details are known. For preliminary design, however, the negative friction can be assumed to be equal to the shaft friction calculated as described above for uplift resistance (the resistance factor of 0.3 should not be applied).

Negative friction is typically only considered in conjunction with dead and sustained live loads (not transient loads such as wind, earthquake and transient live loads) in evaluating the structural capacity of the pile. Negative friction does not impact the geotechnical resistance of the piles.

Lateral Resistance

The lateral resistance of a slender pile is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter most commonly used to determine lateral deflection of piles is the coefficient of horizontal subgrade reaction (k_h). For this site, k_h may be assumed to be:

$$k_h = \eta_h z$$

Where:

k_h = the modulus of subgrade reaction (kN/m^3);

η_h = a coefficient based on soil type (use 4.4 MPa/m); and,

z = the depth under consideration

The value above is for a single pile group. Group interaction must be considered when piles are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal reaction (k_h) by an appropriate factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
6d	1.0
3d	0.25

Values for other spacings may be interpolated from the values above. No reduction is required for the first row of piles (i.e., the row which bears against undisturbed soil with no piles in front).

It should be noted that the method of applying a linear “spring” to represent the soil reaction to loading is a significant simplification of the soil/pile behaviour. If lateral load resistance governs the pile design, more rigorous, non-linear methods of analysing resistance exist, one common one being the method of p-y curves. These methods, however, require knowledge of the pile size, location, loading, pile cap construction, etc. and are therefore typically more suited to the detailed design phase when these items are known. Golder can provide additional assistance during detailed design, if required.

Construction Considerations

The piles will be driven to bedrock through a layer of glacial till which is known to contain cobbles and boulders. Piles can deflect or become damaged if they encounter boulders in the glacial till. Piles (both H-piles and pipe piles) should be equipped with pile points (e.g., Titus Standard H Point, or similar) to provide additional protection to the pile tips against damage from boulders during driving. Even with this measure, it should be expected that damage may occur to some piles and replacement piles will be required. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then gradually increase the energy over a series of blows to seat the pile.

Provision should be made for restriking all piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed a minimum of 48 hours after the previous set.

Pile driving criteria depend not only on the details of the pile (size, length, load, etc.) but also on the equipment used for installation. Preliminary pile driving criteria should be established prior to construction using wave equation analysis (WEAP or similar) or other approved means and confirmed through a program of dynamic (PDA) testing carried out at an early stage in the piling program. Additional PDA testing should be used to confirm the pile capacities at regular intervals as the project progresses. As a preliminary guideline, the specification should require that at least 10% of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing. As well, CAPWAP analyses should be carried out for at least one half of the piles tested, with the results provided no later than three days following testing. The final report should be stamped by an engineer licensed in the province of Ontario. The PDA testing program will justify an increase in the geotechnical resistance factor to 0.5 as discussed above.

It should be noted that the driving energies required confirm the full ultimate resistance of the pile (typically the testing is intended to prove a load of twice the design load) may be higher than the energy required to install the pile. Insufficient energy is a common problem in demonstrating the true ultimate capacity of piles during PDA testing, and larger pile driving hammers may be required for the testing where piles are driven to rock in order to generate high axial capacities.

The piling specifications should be reviewed by Golder prior to tender, as should the contractor’s submission (shop drawings, equipment, procedures, preliminary set criteria, etc.) prior to construction. Piling operations should be inspected on a full-time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

5.2.1.2 Drilled Cast-in-Place Piles

If drilled piles are used, they should be socketed into the limestone bedrock. The use of a casing will be required to advance the caisson through the glacial till material into the underlying bedrock. The casing should be extended so that it is “seated” a minimum of 500 mm into the bedrock.

5.2.1.3 Axial Geotechnical Resistance

Due to the difficulty in socketing liners into the limestone bedrock to completely cut off the water infiltrations, it may not be feasible to dewater and clean the base of the piles, or to inspect the base prior to concreting. As such, end-bearing support may not be fully developed and should be neglected in the design. The axial geotechnical resistance for rock-socketed caissons is therefore recommended to be based on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

Rock-socketed cast-in-place piles should be designed based on the sidewall (shaft) resistance of the rock socket and a factored geotechnical resistance at ULS of 1.1 MPa, provided that the caisson socket is within competent bedrock (i.e., RQD greater than 50 percent). For preliminary design this condition can be assumed to be 1 m below the bedrock surface. This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

Settlements for rock-socket piles are typically small, and the factored ULS axial resistance will be reached before the pile has experienced 25 mm of settlement (a typical SLS condition). Geotechnical SLS considerations therefore do not generally govern the design of rock-socketed cast-in-place piles.

SLS resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial

Pile spacings should not be less than three pile diameters (centre-to-centre) to prevent group effects. If closer pile spacings are required they can be accommodated, but the individual pile capacity may need to be reduced to account for the closer spacing. This can be reviewed in detailed design if required.

5.2.1.4 Lateral Geotechnical Resistance

To provide full fixity, the drilled cast-in-place piles should be provided with a minimum socket length equal to the greater of 2 times the caisson diameter below the depth of any broken or highly weathered surficial bedrock (which may be assumed to be 1 m). The structural engineer should confirm that the shear strength of the concrete is adequate to support these loads. In this condition, the rock sockets may be assumed to be “fixed” at the rock socket for preliminary design. This assumption should be confirmed during detailed design based and the actual pile dimensions, and depths.

The SLS geotechnical response of the soil in front of the caissons under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition). It may be assumed that this resistance (from the soil in front of the piles) will be nearly the same for vertical and inclined piles.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction, Use 4.4 MN/m^3;

z is the depth (m); and,

B is the pile diameter/width (m)?

The discussion provided in Section 5.2.1.1.4 regarding the use of a “spring constant” to represent the relatively complex behaviour of the soil/rock/pile applies to drilled piles as well. Golder can undertake additional analysis during detailed design if lateral loading is a significant issue.

5.2.2 Shallow Spread Footings

Although not likely suitable for the high-rise towers, it may be feasible to support more lightly loaded parts of the structure on shallow spread footings on the dense glacial till. If lowering the foundations is a feasible option, then shallow foundations on bedrock are also suitable (both for the podium and the towers).

5.2.2.1 Footings on Glacial Till

Spread footings founded on the dense glacial till below the currently proposed P2 level may be a feasible option for lighter parts of the structure. An SLS net bearing resistance of 250 kPa and a factored ULS bearing resistance of 400 kPa can be used for design of pad footings up to 5.0 m in width and for strip footings up to 2.0 m in width placed on native and undisturbed glacial till below this elevation. The SLS values provided correspond to calculated total and differential settlement values of 25 and 19 mm, respectively.

It should be noted that because the expected settlements of any piled foundations are very small, differential settlements of up to about 25 mm may occur between the spread footings placed on glacial till and any parts of the development supported on piles. The design of the new structure will have to consider these differential settlements. Structural separation may be required between the foundations supported on piles, and those supported on glacial till.

For ULS sliding resistance of a cast-in-place footing placed on glacial till, an unfactored friction coefficient of 0.45 can be used. In accordance with OBC 2012 requirements, a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying glacial till.

5.2.2.2 Footings on Bedrock

For spread footings placed on sound bedrock, a factored Ultimate Limit States (ULS) bearing resistance of 4,000 kPa may be used for preliminary design. Serviceability Limit States (SLS) net bearing resistances do not generally apply to the design of foundations on the bedrock, provided the bedrock surface is properly cleaned of soil and loose highly weathered/fractured bedrock at the time of construction. As discussed above, differential settlements of up to 25 mm should be anticipated between areas which are founded on rock (which would be expected to experience negligible settlement) and areas which are founded on the glacial till.

For ULS sliding resistance of a cast-in-place footing placed on bedrock, an unfactored sliding friction coefficient of 0.70 can be used. In accordance with OBC 2012 requirements, a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying bedrock.

5.2.3 Raft or Mat Foundations

It may be feasible to support the structures (or portions of the structures) on a raft or mat foundation on the dense to very dense. A raft or mat foundation would need to be sufficiently rigid to ensure that the loading is uniformly distributed over the entire footprint of the raft, and to minimise the potential for differential settlement between heavily and lightly loaded areas.

Supporting the four-storey podium, plus two levels of parking on a raft foundation would be relatively straight-forward. Supporting the entire structure on a large raft would be more complex and because of the thickness of the heavily reinforced raft which would be required and the relatively thin layer of soil below the building it may be simpler to just lower the foundation level to the bedrock.

The design of a large, rigid raft foundation is not typically governed by an overall bearing capacity of the soil, but rather by the need to limit the differential settlement between different parts of the raft to some acceptable value. A raft foundation in soil typically experiences relatively large total settlement, but due to its stiffness limits differential settlement.

The geotechnical parameter most commonly used in this assessment is the vertical modulus of subgrade reaction (k_{v1}). For the dense glacial till, the vertical modulus of subgrade reaction may be assumed to be 65 MPa/m. This value is for a 300 mm by 300 mm loaded area which has been adopted as a standard for comparison.

The modulus of subgrade reaction is not a fundamental soil property, and its value depends, in part, on the size and shape of the loaded area. The design modulus should be adjusted based on the loaded area as outlined in Section 7.7.1 of the CFEM (4th Edition, 2006). For a rectangular loaded area of width b and length mb:

$$k_{vb} = \left(\frac{kv1}{3.28b}\right) * \frac{m+0.5}{1.5m}$$

where

k_{vb} = the modulus for the actual loaded area; and

b = the width of the loaded area

The modulus of subgrade reaction is a significant simplification of actual soil behaviour. The presence of rock at relatively shallow depth as well as the likely variety of differently loaded areas also complicate the analysis. For detailed design a more rigorous design method such as a three-dimensional settlement analysis or finite element model would be more appropriate for a project of this scale. These analyses, however, cannot be undertaken without knowledge of the proposed foundation loading.

For the analysis of the contact stress distribution beneath a slab on grade foundation, the modulus of subgrade reaction value obviously depends on the size of the areas over which increased/concentrated contact stresses are anticipated and the stiffness of the raft itself (analogous to equivalent footings beneath the columns); the size of these areas is in turn related to the value of the modulus of subgrade reaction, i.e., they are inter-related. The design of a raft foundation is therefore typically an iterative process requiring both geotechnical and structural analysis of the settlement, load distribution and stiffness of the structure.

If the preliminary values provided above suggest that a raft foundation may be possible, Golder can assist with additional analysis during detailed design using this iterative approach.

5.3 Rock Anchors

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

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

i) Failure of the steel tendon or top anchorage

ii) Failure of the grout/tendon bond

iii) Failure of the rock/grout bond, and

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

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

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

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

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

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

φ = Geotechnical resistance factor (use 0.4);

$\gamma^{/}$ = Effective unit weight of rock and soil (use 10 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 10 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).

A geotechnical engineer should 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.

5.4 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements (i.e., footings, pile caps, grade beams, etc.) in unheated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior foundation elements adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 m of earth cover.

As an alternative to earth cover, consideration could be provided to the use of an insulation detail. Additional guidance on insulation details can be provided if required.

5.5 Seismic Design Considerations

5.5.1 Seismic Liquefaction

There is no significant risk of liquefaction at the site during a seismic event.

5.5.2 Seismic Site Class

The OBC 2012 contains seismic analysis and design methodology. The seismic Site Class value, as defined in Section 4.1.8.4 of the OBC 2012, depends on the average shear wave velocity of the upper 30 m of soil and/or rock below founding level.

Based on the in-situ testing data, this site can be assigned a Site Class of C for seismic design purposes according to the 2012 OBC.

A higher site Class can be assigned for “rock” sites (where the foundations are on, or very close to rock). The lowest level of the currently proposed development is indicated to be at an elevation of 53.6 m. This compares with a rock elevation of approximately 48 m over the majority of the site. If the final design is such that the underside of the foundations is within 3 m of the bedrock (i.e., at or below approximately 51 m elevation) a Site Class A would apply.

5.6 Excavations and Shoring

Based on the preliminary site plan provided, the lowest finished floor elevation is at 52.6 m. The main excavation will be lower than this by at least the thickness of the lower-level slab-on-grade, granular base, drainage, etc. Localized excavations would also be required for pile caps, footings, etc. as well as services. Based on borehole elevations this will require excavations on the order of 9 to 10 m deep over the entire site, with deeper localized excavations for foundations and services.

Excavations for the construction of the foundations and basement levels will be through the existing fill, and into the underlying glacial till. No unusual problems are anticipated with excavating the overburden using conventional hydraulic excavating equipment. Cobbles and boulders should be expected in the fill, glacial till and sand and gravel deposits. Debris (e.g., organics, brick, metal, wood, stone, concrete, etc.) should also be expected in the fill.

It is likely that significant portions of the site will require shoring (due to insufficient space to complete open cut excavations; for example, along Albert St., Booth St., adjacent to the OLRT ROW, etc.). There may be other areas where sufficient space exists for open excavation. Both are discussed below.

5.6.1 Open Cut Excavations

Above the groundwater level and within the fill, silty sand, native silty clay and glacial till side slopes should be stable in the short term at 1 horizontal to 1 vertical; these soils would be classified as Type 3 soils in accordance with the Occupational Health and Safety Act of Ontario (OHSA). This would also apply to areas where the groundwater table was drawn down and maintained below the final excavation depth in advance of excavation (in which case the soils are effectively above the water table at the time of excavation).

Excavations within the silty and sandy soils (both fill and till) *below the water table* would be classified as a Type 4 soil; these excavations would therefore require side slopes at a minimum slope of 3H:1V (i.e., flatter than 3H:1V).

It is expected that open-cut methods will generally be feasible (from a technical perspective) provided sufficient space exists to accommodate the excavations, though given the height they may require benching, access ramps, etc. to be incorporated into the design. It should be noted that the height of the excavations (10 m) exceeds the height for prescriptive design under the OHSA. Deeper portions of the excavation (even if open cut) will require an engineered design to comply with the relevant regulations.

Temporary excavations for foundations or site services (if required) will be through similar soils as discussed above. These excavations can also likely be made with sloping excavations where space permits. Where space does not exist, localized excavations for foundations or temporary services could be carried out with vertical sides and fully braced, steel trench boxes or shoring systems.

5.6.2 Shored Excavations

Where sufficient space does not exist (or if it is preferable to limit the size and impact of the excavation as well as associated excavation and backfilling) the temporary excavations could be carried out using a shoring system to ensure support for the soil and provide for worker safety. This section of the report provides some general guidelines on possible concepts for the shoring to be used by the designers for assessing the possible impacts of the shoring design and site works as well as to evaluate, at the design stage, the potential for impacts of this shoring on the adjacent properties and infrastructure. Temporary shoring can be used in combination with open cuts above the top of shoring, however, the earth pressure distribution must take into account the effects of the soil pressures from the upper sloped section.

This type of shoring system is typically designed and constructed by a specialist contractor who is fully responsible for the detailed design and performance of the temporary shoring systems. In addition to supporting the soils surrounding the excavation, the design of temporary support systems will need to consider the support requirements of adjacent structures, roads, utilities, etc.

The shoring method(s) chosen (and in particular the selection of the appropriate design earth pressures; higher design earth pressures are required if it is necessary to limit the deflection of the shoring) to support the excavation sides must take into account the soil and bedrock stratigraphy, the permissible movement of the shoring, the groundwater conditions, the methods adopted to manage the groundwater and construct the shoring systems, the potential ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities.

The City of Ottawa rights-of-way for Albert Street and Booth Street, which contain below grade services (as well as bridge structures in the case of Booth St.) are located adjacent to the south and west sides, respectively, of the proposed excavation for the building. As such, any services located in close proximity to and/or within the zone of influence of the shoring system could be affected by ground movements behind the shoring. Details on the utilities in these areas should be confirmed during the detailed design studies to better tailor the shoring guidelines provided herein. Additionally, the right-of-way for the OLRT, as well as Pimisi Station is located adjacent to the north side of the proposed development and, if in close proximity to and/or within the zone of influence of the shoring system, could be affected by ground movements behind the shoring.

Shoring for this type of project would typically include tied back sheet pile walls or soldier pile and lagging systems (if a soldier pile and lagging system is employed the potential for flowing sands below the water table must be considered and addressed as part of the shoring/dewatering design). Due to the presence of very dense till with boulders at shallow depth on the site, soldier piles may require predrilling to provide sufficient embedment for toe fixity. Depending on the final design it may also be possible/necessary to socket the toe of the piles into rock. The shoring system must be provided with appropriate lateral support. Steel sheet piles cannot be pre-drilled and may have difficulty penetrating cobbles and boulders within the till (and certainly cannot be extended into rock for additional toe support).

Where foundations or settlement sensitive infrastructure, such as buried utilities, are present within the zone of influence of the shoring system and deflections need to be greatly limited a secant pile wall with pre-stressed tie backs may also be considered. 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 that relatively flexible features (such as roadways or sidewalks) will not be adversely affected.

Some form of lateral support to the wall is typically required for excavation depths greater than about 3 to 4 m. Lateral restraint could be provided by means of tie-backs consisting of grouted rock anchors. The use of rock anchor tie-backs would require the permission of the adjacent property owners since the anchors would be installed beneath their properties. The presence of utilities beneath the adjacent streets, which could interfere with the tie-backs, should also be considered, though this is typically manageable provided the first row of anchors is below the typical burial depth of municipal services. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or to raker piles and/or footings within the excavation.

5.6.3 Ground Movements

During the excavation for the underground levels of the proposed buildings, lateral deformation and vertical settlement of the adjacent ground will occur as a result of installation and deflection of the retaining/shoring system and dewatering activities. The ground movements induced could affect the stability or performance of buildings or underground utilities adjacent to the excavation. Therefore, the magnitude and extent of ground movement and potential impacts on surrounding infrastructure should be assessed prior to construction to confirm movements will be in tolerable limits and monitored during construction.

Based on previous experience with nearby projects, the OLRT right-of-way may require additional analysis and review of the shoring design than is normally the case.

5.7 Groundwater Control

During the current investigation groundwater was encountered within the glacial till as high as 55.1 m. Lower groundwater levels were encountered in some boreholes, but these measurements were taken relatively quickly after drilling and may not represent fully stabilized groundwater levels.

It should also be noted that these represent the groundwater level on a single date (February 2022). These levels may not represent typical groundwater levels (because they were measured in winter) and certainly do not represent the maximum levels which could be encountered. As a comparison, Golder has experience with an adjacent site which encountered groundwater in the large excavation at 57 m elevation.

Based on this it is evident that the proposed development will extend below the groundwater level at the site and temporary and permanent groundwater control will be required.

5.7.1 Temporary Groundwater Control

Given the anticipated size and depth of the excavation, as well as the likely groundwater conditions at the site dewatering of the site will be required during construction to maintain a safe, dry working area and to prevent disturbance of the soil subgrade.

According to O.Reg 63/16 and O.Reg 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 litres per day and less than 400,000 litres per day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a “Water Taking Plan” and a “Discharge Plan”. Alternatively, a Permit to Take Water (PTTW) is required from the Ministry of the Environment, Conservation (MECP) if a volume of water greater than 400,000 litres per day is to be pumped from the excavations.

Calculation of anticipated groundwater flows have not been completed as part of this current phase, however, based on previous experience it is recommended that it be assumed a PTTW will be required. Once the final excavation footprint and depth are confirmed a hydrogeological study will be required to support the permit application.

The rate of groundwater inflow to the excavation will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the material, incident precipitation, and the time of year at which the excavation is made (e.g., fluctuation in seasonal groundwater elevation). Moderate flows into the main excavation could potentially be managed using properly filtered sumps in closely space trenches or pits. Groundwater inflow for service trenches or smaller localized excavations for foundations, elevator pits, etc., should also be possible to control by pumping from within the excavations.

If higher flows are encountered, then a more active dewatering system (wells or well points) could also be considered to maintain the groundwater level below the base of the excavation. This requirement is particularly critical if shallow foundations (either footings or a raft/mat foundation) are considered as the uncontrolled seepage into the floor of the excavation (even if collected and pumped out in sumps) is likely to cause disturbance and piping of the subgrade resulting in a need to over-excavated and replace soils to maintain a suitable bearing surface.

The contractor should be fully responsible for design of the groundwater control system.

The glacial till soils that will form the floor of the foundation excavations are expected to be sensitive to disturbance. Consideration should therefore be given to protecting the subgrade in foundation areas with a mud slab of lean concrete or a layer of compacted granular fill materials (particularly if the areas will remain open for extended periods of time such as if a raft is used). The thickness of the mud slab and/or compacted granular fill working mat will depend on the size and weight of the equipment to be used at the bottom of the excavation. Any disturbed soil will need to be removed prior to placing the protective layer. That mud slab/granular fill materials should be placed immediately following inspection and approval of the subgrade. The period of time between exposure of the subgrade and covering with the protective layer should be limited to as brief as possible and, in the interim, no construction traffic should be permitted on the subgrade.

5.7.2 Permanent Groundwater Control

The measured groundwater depth at the site is variable, but it is above the lowest level of the proposed underground parking. To manage the long-term groundwater levels a drainage system diverting collected groundwater inflow to the sewer system is recommended. It is recommended that a hydrogeological assessment be completed to provide input toward the volumes of water anticipated to be diverted to the municipal sewer system (this can be done in conjunction with the study for the PTTW discussed above).

The subfloor drainage system (i.e., below the lowest garage level) should consist of a network of robust sub-drain pipes conveying collected groundwater to a sump or sumps from which the groundwater can be pumped to a municipal sewer. The drainage system would consist of interconnected perforated drain pipes (bedded and backfilled with free draining granular soils) installed around the perimeter and within the building footprint.
The capacity of the subfloor drainage system should be initially based on the hydrogeology assessment and then modified during construction if required.

Drainage, such as a composite synthetic drainage system or equivalent, should be provided to the exterior walls. The composite drain must withstand the design horizontal earth pressures used for basement wall design and should be connected to the basement level underslab drainage system. The drainage system collector pipes should drain to a sump for collection and discharge to a sewer.

5.8 Garage Floor Slab

In preparation for the construction of the lowest floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab down to the undisturbed native soil. Provision should be made for at least 250 mm of OPSS Granular A to form the base of the floor slab. Any bulk fill required to raise the grade up to the underside of the Granular A (as well as any areas where over-excavation and replacement are required) should consist of OPSS Granular B Type II. The under-slab fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) using suitable vibratory compaction equipment.++1111111111

Provision should be made for drainage underneath the floor slab consisting of perforated pipe subdrains in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit from which the water is pumped. For preliminary design purposes, these drains should be placed at approximately 6 m centres.

5.9 Foundation Wall Backfill

Foundation/basement walls should be backfilled with free draining non-frost susceptible granular fill meeting the requirements of OPSS Granular B Type I or II materials. Basement walls should be covered with drainage board such as Miridrain (or equivalent).

Backfill should be compacted to 95% of the material's SPMDD using suitable compaction equipment. To reduce compaction induced stresses, only light compaction rollers or plate tampers should be used within 1 m of the wall.

Beneath hard surfacing (e.g., pavements or sidewalks/walkways), the granular backfill for the foundation wall should be placed to form a frost taper at 3 horizontal to 1 vertical from a depth of 1.8 m (i.e., the frost depth) to the underside of the granular base for the hard surfacing. The purpose of this frost taper is to limit the severity of differential heaving that could occur between areas backfilled with non-frost susceptible engineered fill and the adjacent areas underlain by the existing frost susceptible soils.

5.10 Lateral Earth Pressures for Design

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

Where the wall support and structure allow lateral yielding, (e.g., for unrestrained retaining walls), active earth pressures may be used in the design of the wall. Where the support does not allow lateral yielding, (i.e., for typical basement walls) at-rest earth pressures should be assumed. The following parameters (unfactored) may be used for design where there is limited granular material between the basement and the native soil (for example where the site is shored):

Soil	Unit Weight (kN/m^3)	Coefficients of static lateral earth pressure	
		Active, Ka	At rest, Ko
Granular Backfill or Glacial Till	21	0.33	0.50
Glacial Till	22	0.31	0.47

If the garage/foundation wall is backfilled with granular free draining fill either in a zone with width equal to at least 50 percent of the height of the wall or within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing/pile cap/grade beam, the following parameters (unfactored) may be used:

Material	Unit Weight (kN/m^3)	Coefficients of static lateral earth pressure	
		Active, Ka	At rest, Ko
Granular A or Granular B Type II	22	0.27	0.43
Granular B Type I	22	0.31	0.47

For the purposes of shoring design, the designer (who is entirely responsible for the design including selection of design parameters) should carefully review the subsurface information and determine appropriate earth pressure parameters for use in their design. In particular, higher values than indicated in the tables above may need to be assumed in order to limit deflection of the shoring near existing structures.

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

The horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the design PGA (i.e., k_h = 0.32). For structures which allow lateral yielding, k_h is taken as 0.5 times the design PGA (i.e., k_h = 0.16).

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

Wall Type	Site PGA (2475-year Earthquake)	K_{AE}	
		Granular A/Granular B Type II	Granular B Type I
Yielding Wall	0.32g	0.39	0.43
Non-Yielding Wall		0.53	0.59

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution).

A minimum surcharge pressure of 12 kPa due to traffic and compaction induced pressure should be included in the total lateral earth pressures for the structural design of the wall.

The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d) + q$$

Where: $\sigma_h(d)$ = Lateral earth pressure at depth, d, (kPa)

K_o = Coefficient of static earth pressure

γ = Unit weight of the backfill soil (kN/m^3); as given previously

d = Depth below the top of the wall (m)

K_{AE} = Seismic active earth pressure coefficient

q = Surcharge to account for traffic and compaction pressure, where applicable

H = Total height of the wall (m)

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Ultimate Limit States design purposes.

5.11 Site Servicing

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

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

It should generally be possible to re-use the existing inorganic fill and glacial till as trench backfill provided it is properly moisture conditioned. Where trenches will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 mm depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material’s SPMDD using suitable vibratory compaction equipment.

Seepage barriers should be constructed at periodic intervals along the trench and at the connection points to off-site infrastructure to reduce the potential for groundwater level lowering in the surrounding area due to the “french drain” effect on the granular bedding and surround. Groundwater level lowering could lead to long-term settlement of nearby structures that are supported on the sensitive silty clay soil underlying the site.

It is important that these barriers extend from trench wall to trench wall and that they fully penetrate the granular surround materials to the trench bottom. The seepage barriers should be at least 1.5 metres long. In addition to providing a drainage cut-off, these cut-offs also serve as impenetrable cut-offs to stop the potential migration of contaminants along the relatively permeable backfill in the trenches.

Construction of the seepage barriers should also be in accordance with the City of Ottawa's Standard Drawing No. S8 of the Department of Public Works and Services, Infrastructure Services branch.

5.12 Pavement Design

In preparation for pavement construction, all topsoil, unsuitable fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the pavement areas. Some of the existing fill could remain provided that it is free of organic matter, and that the subgrade be subjected to a proof roll with a loaded tandem truck to reveal weak or soft areas prior to the construction of the new pavement structure. Soft or weak areas should be removed and repaired with acceptable earth borrow or OPSS Select Subgrade Material (SSM) or Granular B.

Pavement areas requiring grade raising to proposed subgrade level should be brought to grade using acceptable (compactable and inorganic) earth borrow, OPSS SSM or Granular B. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable compaction equipment.

The surface of the pavement subgrade should be crowned or sloped to promote drainage of the pavement granular structure towards perimeter swales or subdrains placed at the subgrade level

No traffic or paving details are available at the current stage. The following pavement designs are recommended for preliminary purposes based on experience with similar projects and developments. These designs should be confirmed during detailed design based on actual traffic requirements.

Material		Light Duty Pavement Thickness of Pavement Elements (mm)	Heavy Duty Pavement Thickness of Pavement Elements (mm)	Loading Dock Thickness of Pavement Elements (mm)
Bituminous Concrete OPSS 1150	Superpave 12.5 mm	60	40	-
	Superpave 19.0 mm	-	50	-
Portland Cement Concrete	Portland Cement Concrete	-	-	200
Granular Material OPSS 1010	Granular A Base	150	150	150
	Granular B, Type II Subbase	300	450	450
	Prepared and Approved Subgrade			

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B.

The Portland cement concrete should meet the requirements of CSA A 23.1 Class C2 exposure. Concrete joint specifications and spacing should be in accordance with OPSD 552.020 and 551.010.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., grade raise fill has been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

Where the new pavements will connect to existing pavements, the new pavement structures should be continued at least to the limits of construction, with any longitudinal transitions and/or tapers occurring thereafter. At these locations, the longitudinal transitions should be constructed by cutting the existing pavement structure vertically to the bottom of the existing subbase. The new granular layers should then be tapered up or down, as required, at a slope of 5 horizontal to 1 vertical to match the existing pavement structure. The asphaltic concrete does not need to be tapered between the new construction and the existing pavement. However, the asphaltic concrete of the existing pavement should be milled back an additional 300 mm to a depth of about 60 mm or 40 mm in areas where its thickness is greater than 100 mm, matching the proposed surface course of the new asphaltic concrete. A tack coat should be provided and the new surface course asphaltic concrete placed over the milled surface to form the new pavement joint. Where the existing pavement is less than 100 mm, then a butt joint on a vertical saw cut surface is acceptable. A tack coat should be placed on the vertical saw cut surface. The tack coat should be in accordance with the City SP F-3107.

5.13 Corrosion and Cement Type

Five samples of soil for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix C. The results indicate that concrete made with Type GU Portland cement should be acceptable for concrete substructures.

The results also indicate a potential for corrosion of buried ferrous elements, which should be considered in the design of substructures and pile foundations.

6.0 ADDITIONAL CONSIDERATIONS

At the time of writing this report, only conceptual details related to the proposed building as well as adjacent significant structures such as existing sewers and the OLRT were available. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted.

The construction activities could impact the existing adjacent structures and buildings. Appropriate damage assessments (pre and post condition surveys for example) should be carried out as necessary.

During construction, sufficient foundation inspections, subgrade inspections, in-situ density tests, materials testing, pile and rock anchor installation monitoring should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications. Concrete testing should be carried out in a CCIL certified laboratory.

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost. All bearing surfaces must be inspected prior to filling or concreting to ensure that strata having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared.

7.0 CLOSURE

We trust that this report provides sufficient geotechnical engineering information to facilitate the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

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Senior Geotechnical Engineer

CH/SM/hdw
https://golderassociates.sharepoint.com/sites/158117/project files/6 deliverables/geotechnical/final april 2022/22511882-rev0-dream lebreton library lands geotechnical report-1806_201.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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

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

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

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

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

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

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

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

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

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

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

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

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

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

LEGEND

⊕	APPROXIMATE BOREHOLE/MONITORING WELL LOCATION (GOLDER 2022)
●	APPROXIMATE BOREHOLE LOCATION (GOLDER 2011)
—	ROADWAY
- - -	FORMER ROADWAY (WELLINGSTON STREET)
—+—	FORMER RAILWAY
🟥	SITE BOUNDARY

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020
2. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 25 50 100

1:1,500 METRES

CLIENT

DREAM

PROJECT

GEOTECHNICAL SITE INVESTIGATION
LEBRETON LIBRARY PARCEL, OTTAWA, ONTARIO

TITLE

BOREHOLE AND MONITORING WELL LOCATION PLAN

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2022-03-25
DESIGNED	----
PREPARED	JEM/MG
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
22511882	0003	A	1

Path: N:\Active\Spatial_IM\NCC\LibraryParcel\99_PROJ\22511882_Dream_LibraryParcelLebreton\0003_Geotech\22511882-0003-BG-0001.mxd

25mm 0

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

APPENDIX A

Borehole Logs

PROJECT: 22511882

RECORD OF BOREHOLE: 22-01

SHEET 1 OF 3

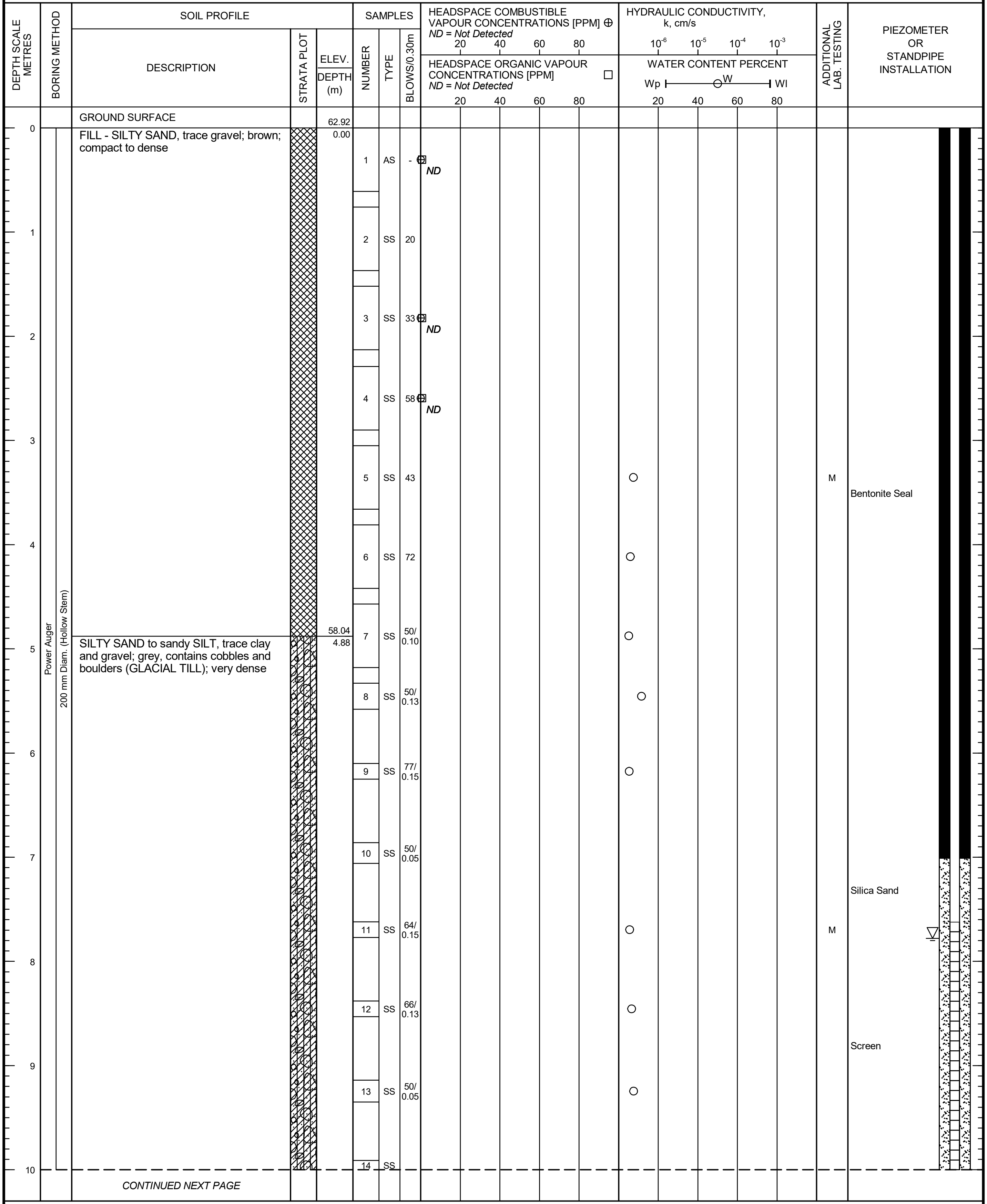
LOCATION: N 5030733.9 ;E 366525.1

BORING DATE: February 14-15, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	HEADSPACE	ADDITIONAL LAB. TESTING
	GROUND SURFACE	62.92					
0	FILL - SILTY SAND, trace gravel; brown; compact to dense	0.00	1	AS	-	ND	
1			2	SS	20		
2			3	SS	33	ND	
			4	SS	58	ND	
3			5	SS	43		M
4			6	SS	72		
5	SILTY SAND to sandy SILT, trace clay and gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense	58.04 4.88	7	SS	50/ 0.10		
			8	SS	50/ 0.13		
6			9	SS	77/ 0.15		
7			10	SS	50/ 0.05		
			11	SS	64/ 0.15		M
8			12	SS	66/ 0.13		
9			13	SS	50/ 0.05		
10			14	SS			

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

LOGGED: ALB
CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-01

SHEET 2 OF 3

LOCATION: N 5030733.9 ;E 366525.1

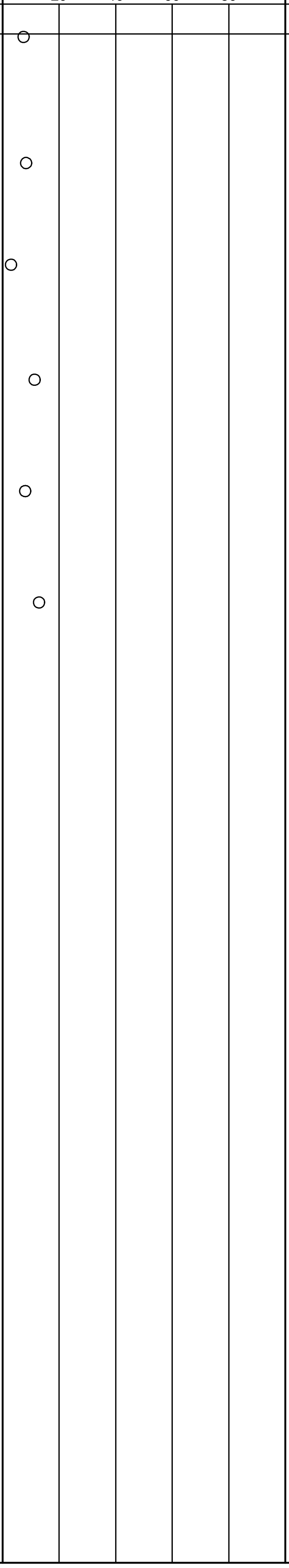
BORING DATE: February 14-15, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20 40 60 80) / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace clay and gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense			14	SS	50/ 0.08			M	Screen
11					15	SS	50/ 0.10				
					16	SS	50/ 0.10				
12					17	SS	50/ 0.13				
13					18	SS	50/ 0.08			M	
14					19	SS	50/ 0.05				
				48.21 14.71							
15		Borehole continued on RECORD OF DRILLHOLE 22-01									
16											
17											
18											
19											
20											



DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF DRILLHOLE: 22-01

SHEET 3 OF 3

LOCATION: N 5030733.9 ;E 366525.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 14-15, 2022

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

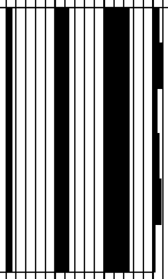
DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		48.21														
15 16	Rotary Drill NQ Core	Weathered, thin to medium bedded, grey black LIMESTONE and SHALE		14.71	1													
				46.46 16.46														
17 18 19 20 21 22 23 24		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 7.78 m (Elev. 55.14 m) on February 25, 2022																



DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-02

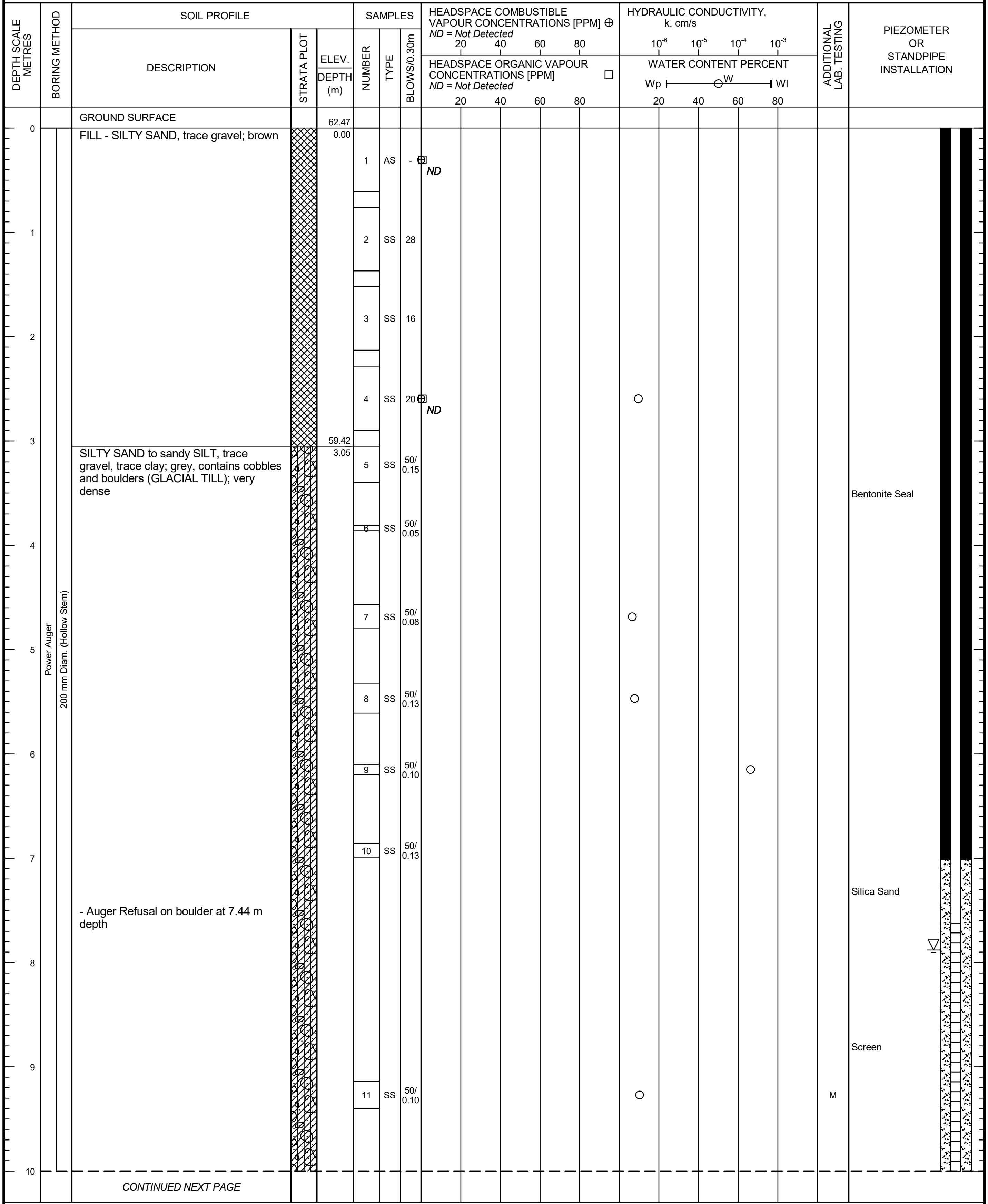
LOCATION: N 5030713.1 ;E 366476.0

BORING DATE: February 16, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	62.47
FILL - SILTY SAND, trace gravel; brown	0.00
SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense	59.42 3.05
- Auger Refusal on boulder at 7.44 m depth	

NUMBER	TYPE	BLOWS/0.30m	HEADSPACE	ADDITIONAL LAB. TESTING
1	AS	-	ND	
2	SS	28		
3	SS	16		
4	SS	20	ND	
5	SS	50/ 0.15		
6	SS	50/ 0.05		
7	SS	50/ 0.08		
8	SS	50/ 0.13		
9	SS	50/ 0.10		
10	SS	50/ 0.13		
11	SS	50/ 0.10		M

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

LOGGED: ALB
CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-02

SHEET 2 OF 3

LOCATION: N 5030713.1 ;E 366476.0

BORING DATE: February 16, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

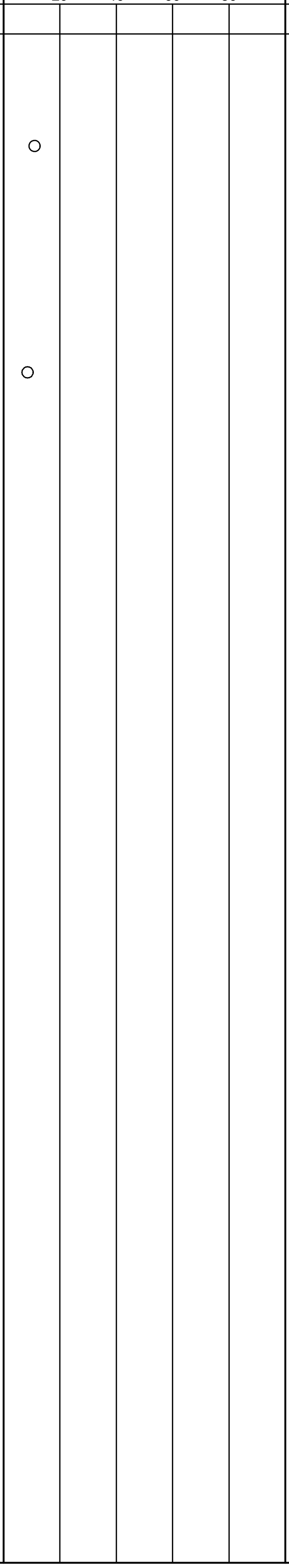
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense						Screen
				12	SS	50/ 0.03		
11								
12				13	SS	50/ 0.03	M	
13								
14			48.27 14.2					
		Borehole continued on RECORD OF DRILLHOLE 22-02						
15								
16								
17								
18								
19								
20								

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕
ND = Not Detected
20 40 60 80

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐
ND = Not Detected
20 40 60 80

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

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



DEPTH SCALE
1 : 50

LOGGED: ALB
CHECKED: CH

PROJECT: 22511882

RECORD OF DRILLHOLE: 22-02

SHEET 3 OF 3

LOCATION: N 5030713.1 ;E 366476.0

DRILLING DATE: February 16, 2022

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		48.27														
15	Rotary Drill NQ Core	Grey, thin to medium bedded LIMESTONE and SHALE		14.20 46.62	1													
16		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 7.88 m (Elev. 54.59 m) on February 25, 2022		15.85														
17																		
18																		
19																		
20																		
21																		
22																		
23																		
24																		

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-03

SHEET 1 OF 3

LOCATION: N 5030756.8 ;E 366500.4

BORING DATE: February 22, 2022

DATUM: Geodetic

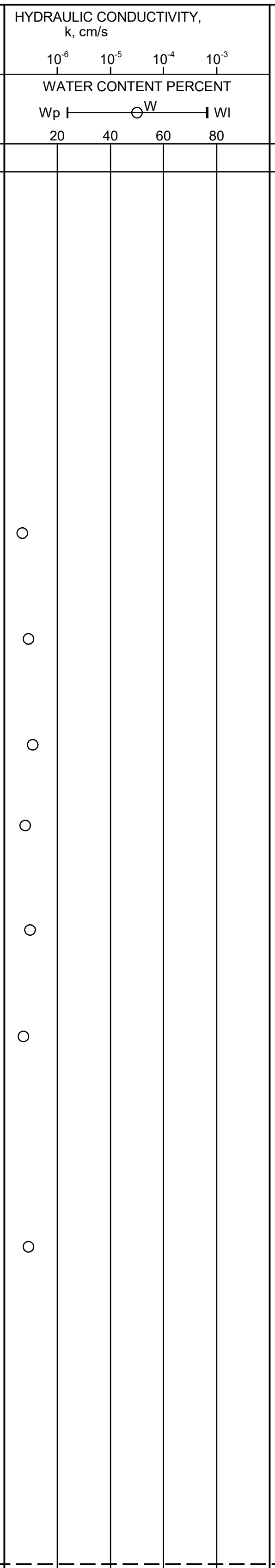
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
		GROUND SURFACE	61.65
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - SILTY SAND, trace gravel and brick; brown	0.00
2.29		SILTY SAND to sandy SILT, trace gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense	59.36 2.29
10		*CONTINUED NEXT PAGE*	

SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected	ADDITIONAL LAB. TESTING
1	AS	-	ND	
2	SS	8		
3	SS	11	ND	
4	SS	43	ND	
5	SS	58		M
6	SS	50/ 0.10	ND	
7	SS	50/ 0.10		
8	SS	50/ 0.08		M
9	SS	50/ 0.08		
10	SS	50/ 0.05		
11	SS	50/ 0.08	ND	

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected



PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite Seal

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-03

SHEET 2 OF 3

LOCATION: N 5030756.8 ;E 366500.4

BORING DATE: February 22, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected 20 40 60 80 HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense	▨		12	SS	61/0.15				Bentonite Seal
11				50.48 11.17							
		Borehole continued on RECORD OF DRILLHOLE 22-03									
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE
1 : 50

wsp GOLDER

LOGGED: ALB
CHECKED: CH

PROJECT: 22511882

RECORD OF DRILLHOLE: 22-03

SHEET 3 OF 3

LOCATION: N 5030756.8 ;E 366500.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 22, 2022

DRILL RIG: CME 55

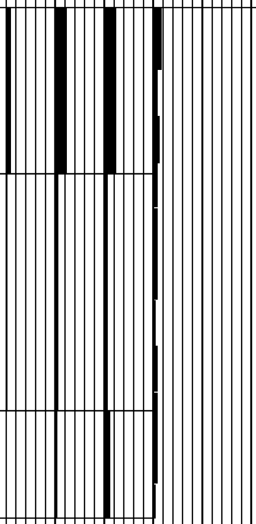
DRILLING CONTRACTOR: Downing Drilling

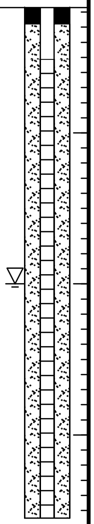
DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	INSTALLATION
		BEDROCK SURFACE	50.48		
	Rotary Drill NQ Core	Slightly weathered to fresh, thin to medium bedded, grey black LIMESTONE and SHALE	11.17	1	Bentonite Seal Silica Sand
12			49.38		
		Fresh, thin to medium bedded, grey to black LIMESTONE and SHALE	12.27	2	
13					Screen ∇
14				3	
			47.10		
		End of Drillhole	14.55		
15		Note(s): 1. Water level in screen measured at a depth of 13.00 m (Elev. 48.65 m) on February 25, 2022			
16					
17					
18					
19					
20					
21					





Other columns printed: SYMBOLIC LOG; FLUSH; COLOUR % RETURN; DIP w.r.t. CORE AXIS (0 30 60 90); DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION; Jcon; Jr; Ja; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

DEPTH SCALE
1 : 50

WSP GOLDER

LOGGED: ALB
CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-04

SHEET 1 OF 3

LOCATION: N 5030713.2 ;E 366411.4

BORING DATE: February 23, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	60.47				
0	FILL - SILTY SAND, trace gravel; brown	0.00	1	AS	-	
1			2	SS	10	
			3	SS	6	
2	SILTY SAND to sandy SILT, trace gravel, trace clay; (possibly till); loose to compact	58.34 2.13	4	SS	8	
3			5	SS	8	Bentonite Seal
4			6	SS	16	
5			7	SS	7	
			8	SS	14	
6	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); dense to very dense	54.37 6.10	9	SS	27	
7						Silica Sand
8			10	SS	78	
9						Screen
			11	SS	47	
10						

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-04

SHEET 2 OF 3

LOCATION: N 5030713.2 ;E 366411.4

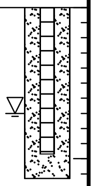
BORING DATE: February 23, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); dense to very dense			12	SS	50/ 0.05				Screen Silica Sand
11		Borehole continued on RECORD OF DRILLHOLE 22-04		49.29 11.18							
12											
13											
14											
15											
16											
17											
18											
19											
20											



DEPTH SCALE

1 : 50

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF DRILLHOLE: 22-04

SHEET 3 OF 3

LOCATION: N 5030713.2 ;E 366411.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 23, 2022

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

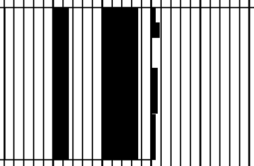
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE		49.29														
12	Rotary Drill NQ Core	Fresh, thin to medium bedded, grey black LIMESTONE and SHALE		11.18 48.28	1													
		End of Drillhole		12.19														



Note(s):

1. Water level in screen measured at a depth of 10.70 m (Elev. 49.77 m) on February 25, 2022

DEPTH SCALE 13 14 15 16 17 18 19 20 21

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-05

SHEET 1 OF 3

LOCATION: N 5030679.9 ;E 366442.7

BORING DATE: February 24, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ *ND = Not Detected* (20, 40, 60, 80)

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ *ND = Not Detected* (20, 40, 60, 80)

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

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	62.34
0	FILL - SILTY SAND, trace gravel; brown	0.00
3	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense	59.29 3.05
10	*CONTINUED NEXT PAGE*	

NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR [PPM]
1	AS	-	⊕ ND
2	SS	33	
3	SS	31	
4	SS	84	
5	SS	94	
6	SS	50	⊕ ND
7	SS	50/ 0.05	
8	SS	50/ 0.03	
9	SS	50/ 0.05	
10	SS	50/ 0.10	⊕ ND
11	SS	50/ 0.13	

PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite Seal

Silica Sand

Screen

▽

DEPTH SCALE

1 : 50

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF BOREHOLE: 22-05

SHEET 2 OF 3

LOCATION: N 5030679.9 ;E 366442.7

BORING DATE: February 24, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected 20 40 60 80 HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m				
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense									
					12	SS	50/ 0.13				
11											
12											
					13	SS	86				
13											
				48.50 13.84	14	SS	50/ 0.13				
14		Borehole continued on RECORD OF DRILLHOLE 22-05									
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

wsp GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 22511882

RECORD OF DRILLHOLE: 22-05

SHEET 3 OF 3

LOCATION: N 5030679.9 ;E 366442.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 24, 2022

DRILL RIG: CME 55

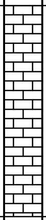
DRILLING CONTRACTOR: Downing Drilling

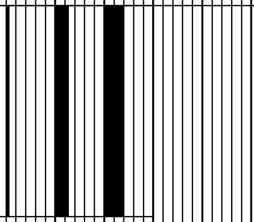
DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE	48.50														
14 15	Rotary Drill NQ Core	Slightly weathered to fresh, thin to medium bedded, grey black LIMESTONE and SHALE	13.84 47.10	1													
		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 8.22 m (Elev. 54.12 m) on February 25, 2022	15.24														





Depth scale: 14, 15, 16, 17, 18, 19, 20, 21, 22, 23

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED: CH

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-33

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 8, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV.	DEPTH (m)
	GROUND SURFACE	62.22	
Power Auger 200 mm Diam. (Hollow Stem)	Dense dark grey crushed stone (Gravel lot BASE)		0.08
	Dense brown fine to medium sand, some coarse sand, some gravel, trace silt (Gravel lot SUBBASE)	61.69	0.53
	Loose to very dense dark brown silty sand, trace to some gravel, brick, wood, organics, concrete, occasional grey silty clay layer (FILL)	59.32	2.90
	Compact to very dense brown to grey brown SILTY SAND to SANDY SILT, trace to some gravel (GLACIAL TILL)	54.60	7.62
	Very dense grey brown SILTY SAND, trace to some gravel, occasional grey silt seam, occasional fine to medium sand seam (GLACIAL TILL)	52.26	9.96
	End of Borehole Split Spoon Refusal		

NUMBER	TYPE	BLOWS/0.30m
1	50 DO	46
2	50 DO	9
3	50 DO	60
4	50 DO	12
5	50 DO	56
6	50 DO	23
7	50 DO	48
8	50 DO	74
9	50 DO	49
10	50 DO	55
11	50 DO	>89
12	50 DO	>100
13	50 DO	>100
14	50 DO	>100
15	50 DO	>111
16	50 DO	>105
17	50 DO	>50
18	50 DO	>100
19	50 DO	>50
20	50 DO	>110

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10 11

DEPTH SCALE
1 : 55

Golder Associates

LOGGED: RI
CHECKED: GDC

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-35

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 12, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m				
		GROUND SURFACE		62.56							
0	Power Auger 200 mm Diam. (Hollow Stem)	Dense grey sand and gravel (Gravel lot BASE)		0.00							
		Compact brown medium to fine sand, trace gravel (Gravel lot SUBBASE)		62.25 0.31	1	50 DO	52				
1		Compact dark brown to black silty sand, trace gravel, ash, wood, brick, mortar (FILL)		61.65 0.91	2	50 DO	17				
					3	50 DO	19				
		Compact brown fine to medium sand, trace gravel (FILL)		60.88 1.68	4	50 DO	24				
2		Dense to very dense light brown to brown SILTY SAND, occasional gravel and medium sand layers, trace gravel (GLACIAL TILL)		60.43 2.13	5	50 DO	45				
					6	50 DO	65				
3					7	50 DO	176				
4					8	50 DO	>50				
		End of Borehole Auger Refusal		58.16 4.40							
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: BM

CHECKED: GDC

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-37

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 12, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES: 0 – 10

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	62.76
Compact sand and gravel (Gravel lot BASE)	0.00
Compact brown medium to fine sand, trace gravel (Gravel lot SUBBASE)	62.46 0.30
Loose dark brown to black silty sand, trace gravel, occasional layers of ash, gravel, sandy mortar, glass, construction debris (FILL)	61.85 0.91
Compact brown medium to fine sand, trace gravel (FILL)	60.63 2.13
Dense to very dense grey brown SILTY SAND, some gravel, trace cobbles (GLACIAL TILL)	60.32 2.44
End of Borehole Auger Refusal	56.23 6.53

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	50 DO	29
2	50 DO	20
3	50 DO	6
4	50 DO	34
5	50 DO	73
6	50 DO	>75
7	50 DO	>65
8	50 DO	>75
9	50 DO	40
10	50 DO	>50

DEPTH SCALE

1 : 50

LOGGED: BM

CHECKED: GDC

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-38

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 19, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
		GROUND SURFACE	62.11			
0	Power Auger 200 mm Diam. (Hollow Stem)	Compact to dense brown sand and gravel (Gravel lot BASE)	0.00			
		Loose to compact brown medium to fine sand, some gravel (Gravel lot SUBBASE)	0.10	1	50 DO	35
1				2	50 DO	8
		Compact to very dense grey brown sand, some gravel, trace silt (FILL)	60.89 1.22	3	50 DO	15
2				4	50 DO	52
		Very dense grey brown SILTY SAND, some gravel, medium brown sand seams (GLACIAL TILL)	59.67 2.44	5	50 DO	61
3				6	50 DO	112
4				7	50 DO	148
		End of Borehole Auger Refusal	57.94 4.17			
5						
6						
7						
8						
9						
10						

DEPTH SCALE
1 : 50

LOGGED: JDR

CHECKED: GDC

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-39

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 15, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
	GROUND SURFACE	62.81			
0	Compact sand and gravel (Gravel lot BASE)	0.00			
	Compact brown to red sandy silt, trace gravel (FILL)	0.15	1	50 DO	15
1	Compact to dense light brown fine to medium sand, trace gravel, silt, and mortar (FILL)	61.90 0.91	2	50 DO	20
			3	50 DO	40
2	Dense sandy gravel to brown fine to medium sand and gravel (FILL)	60.68 2.13	4	50 DO	120
			5	50 DO	67
3			6	50 DO	99
4	Compact to very dense grey SILTY SAND, some gravel (GLACIAL TILL)	59.15 3.66	7	50 DO	34
			8	50 DO	27
5			9	50 DO	33
			10	50 DO	>50
6			11	50 DO	>100
			12	50 DO	>100
	End of Borehole Auger Refusal	56.46 6.35			

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

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

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

WATER CONTENT PERCENT: Wp ├──○W──┤ Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: BM/JD

CHECKED: GDC

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-40

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 16, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger 200 mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	62.77
Compact red to fine brown sand, some gravel (Gravel lot BASE)	0.00
Compact fine to medium brown sand, some gravel, red brick (FILL)	62.39 0.38
Compact light brown fine to medium sand, trace gravel, silt, red brick (FILL)	61.55 1.22
Very dense grey brown SAND, some gravel, trace silt (GLACIAL TILL)	59.78 2.99
Very dense grey brown SILTY SAND, some gravel (GLACIAL TILL)	59.11 3.66
End of Borehole Auger Refusal	56.52 6.25

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	50 DO	13
2	50 DO	19
3	50 DO	15
4	50 DO	25
5	50 DO	51
6	50 DO	59
7	50 DO	100
8	50 DO	>50
9	50 DO	>100
10	50 DO	187
11	50 DO	>50

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

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

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

WATER CONTENT PERCENT Wp ├──○W──┤ Wl: 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

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

MIS-BHS 001 1111220199.GPJ GAL-MIS.GDT 02/24/15 JEM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: GDC

APPENDIX B

Results of Chemical Analysis

eurofins | Environment Testing

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Mr. Chris Hendry
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1974986
Date Submitted: 2022-04-11
Date Reported: 2022-04-19
Project: 22511882
COC #: 889319

				Lab I.D.	1618909	1618910	1618911	1618912
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2022-02-14	2022-02-16	2022-02-16	2022-02-22
				Sample I.D.	22-01 Sa2	22-02 Sa3	22-02 Sa9	22-03 Sa2
Group	**Analyte**	**MRL**	**Units**	**Guideline**				
Anions	SO4	0.01	%		0.02	0.02	0.03	0.07
Cl in Concrete	Cl	0.002	%		0.003	0.011	0.005	0.009
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.28	0.28	0.20	8.22
	pH	2.00			9.39	8.26	8.85	8.22
	Resistivity	1	ohm-cm		3450	3570	5000	1520

				Lab I.D.	1618913
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2022-02-22
				Sample I.D.	SS-03 Sa8
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Anions	SO4	0.01	%		0.03
Cl in Concrete	Cl	0.002	%		0.005
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.20
	pH	2.00			9.08
	Resistivity	1	ohm-cm		5000

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 C

MASW Test Results

TECHNICAL MEMORANDUM

TO: Keith Holmes and Chris Hendry

FROM: Andrew Nicol

SUBJECT: Seismic Site Classification for proposed structure at 665 Albert Street, Ottawa using MASW

PROJECT No.: 221-03921-00

DATE: April 19, 2022

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1 INTRODUCTION

WSP Canada Inc. was retained by Golder & Associates to provide seismic site classification testing at the site of a potential future library at 665 Albert Street in Ottawa, Ontario.

The seismic site classification was achieved using Multichannel Analysis of Surface Waves (MASW).

This technical memorandum will include:

- Description of geophysical method used
- One example record
- One example of a dispersion curve
- One MASW sounding with seismic site class

2 FIELD STUDY SUMMARY

The testing took place on April 8, 2022. The test composed of two (2) seismic soundings located at 665 Albert Street. One sounding with a geophone spacing of one meter and another with a geophone spacing of 4 meters. The data was stacked in the field to improve the signal-to-noise ratio.

The equipment used was the Geode Seismograph made by Geometrics. The geode is controlled via laptop using the Seismodule Controller^{TM} application. An array of twenty-four (24) geophones operating to a frequency of four hertz was used. A 12 lb sledgehammer was used as the active seismic source. An equipment fact sheet can be found in Appendix A.

Figure 1: Location of MASW test line (red).

3 DATA ANALYSIS

MASW testing is the standard surface method for obtaining seismic site class. This is done by measuring the dispersion of Rayleigh surface waves which propagate within the underlying stratigraphy. The seismic phase velocity of these surface waves is directly correlated to the shear-wave velocity of the material(s), and the frequency component of the signal determines the corresponding depths that were sampled.

Therefore, a 1-D MASW sounding can generate an accurate evaluation of the in-situ shear-wave velocities through inversion. More details behind the background of MASW can be found in Appendix B.

The data processing workflow is as follows:

- Remove bad traces.
- Apply a 400 Hz high-cut bandpass filter.
- Plot the phase velocity vs. frequency spectrum.
- Pick the fundamental mode of the dispersion curve.
- Apply an inversion to determine the shear-wave velocity profile.

This analysis is done using the SeisImagerSW™ software package.

3.1 SHOT RECORDS

The MASW records are created from measuring the energy response from an active seismic source. Data was acquired for two different geophone spacings, a one-meter spacing and a four-meter spacing. When combined, the waveforms can typically be used to model the upper 25 meters of soil and rock. The data collected at this site was good.

Figure 2 is an example of a processed shot record from the MASW survey.

Figure 3 shows the dispersion curve associated with the shot record in Figure 2. The interpreted dispersion curve suggests that there are several layers of different materials in the subsurface, with kinks in the curve at 17 Hz and 28 Hz.

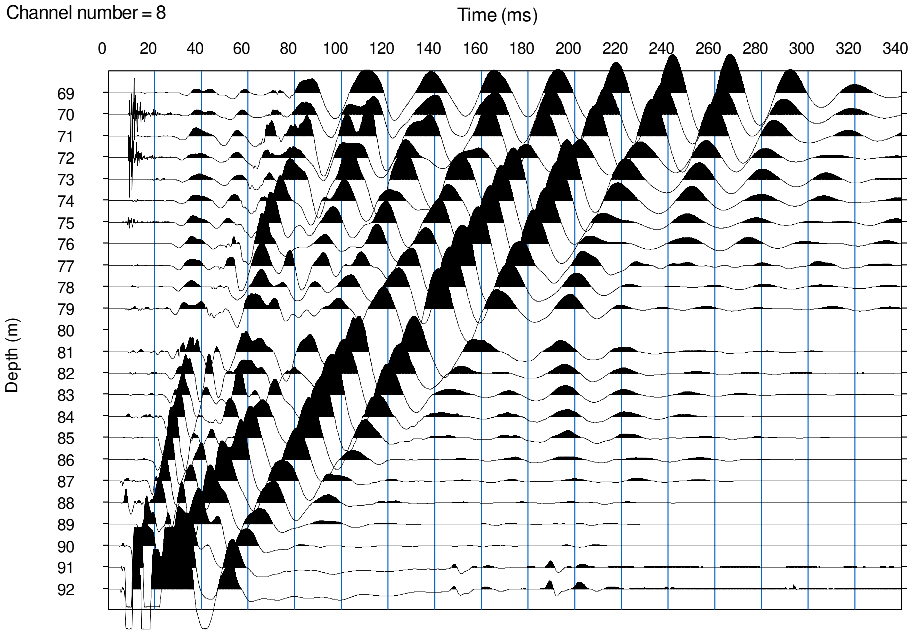

Figure 2: An example shot record.

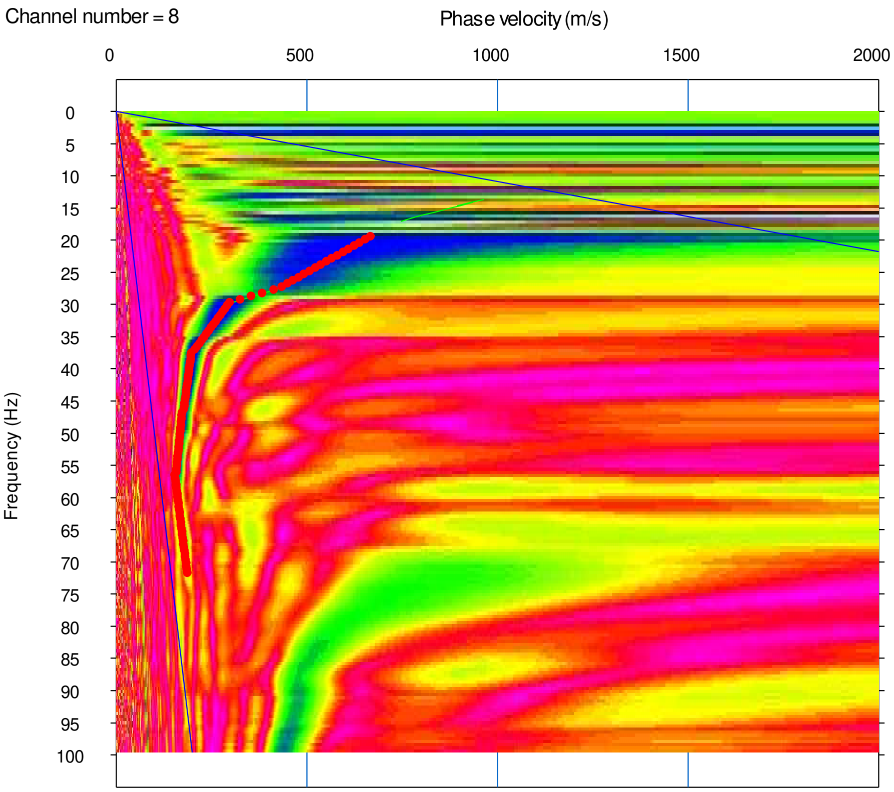

Figure 3: An example dispersion curve.

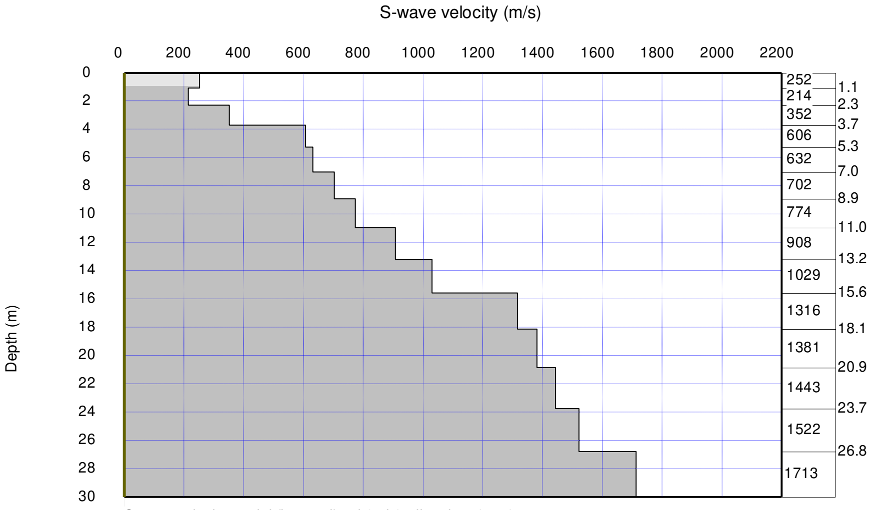

Figure 4: Resulting 1-D V_s profile. Average V_s30 value is approximately 688 m/s.

4 CONCLUSIONS

The MASW shear-wave model is presented in Figure 4. The results have identified the presence of three different layers at this location. The top layer is approximately 3.7 meters thick and has a shear-wave velocity under 360 m/s. Based on borehole data from this site, this layer is made up of engineered fill materials.

The second layer which can be found between the depths of approximately 3.7 meters and 13.2 to 15.6 meters has a shear-wave velocity which increases steadily from 600 m/s to 900 m/s. Based on the borehole information collected, this is a glacial till layer that is roughly 10 meters thick.

The third layer is a bedrock layer with a velocity of approximately 1300 m/s at a depth of 15.6 meters which steadily increases to roughly 1700 m/s at a depth of 27 meters.

The V_s30 values for each of these soundings were calculated from the surface at the time of testing and are therefore appropriate to represent the seismic site class for a slab-on-grade design.

Based on the V_s30 values (as determined through the surface seismic with consideration for the estimated error) and table 4.1.8.4.A of the National Building Code of Canada, 2020 edition (also OBC 2012), **the seismic site classification is within class "C" (360 m/s< $V_s30 \leq 760$ m/s).**

The V_s30 values are based on the harmonic mean of the shear wave velocities over the measurement range of 30 meters. The V_s30 value is calculated (as outlined in Commentary 'J' sentence 4.1.8.4(2) - 101 of the National Building Code) by dividing up the total depth of interest (e.g., 30 m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response.

It must be noted that the site classification provided in this report is based on the V_s30 values as derived from the surface seismic testing method and may be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, the presence of sensitive and/or liquefiable soils, more than 3 m of soft clays, moisture content, etc. The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2020 Edition (also OBC 2012) for more information on the requirements for the site classification.

No further testing is recommended.

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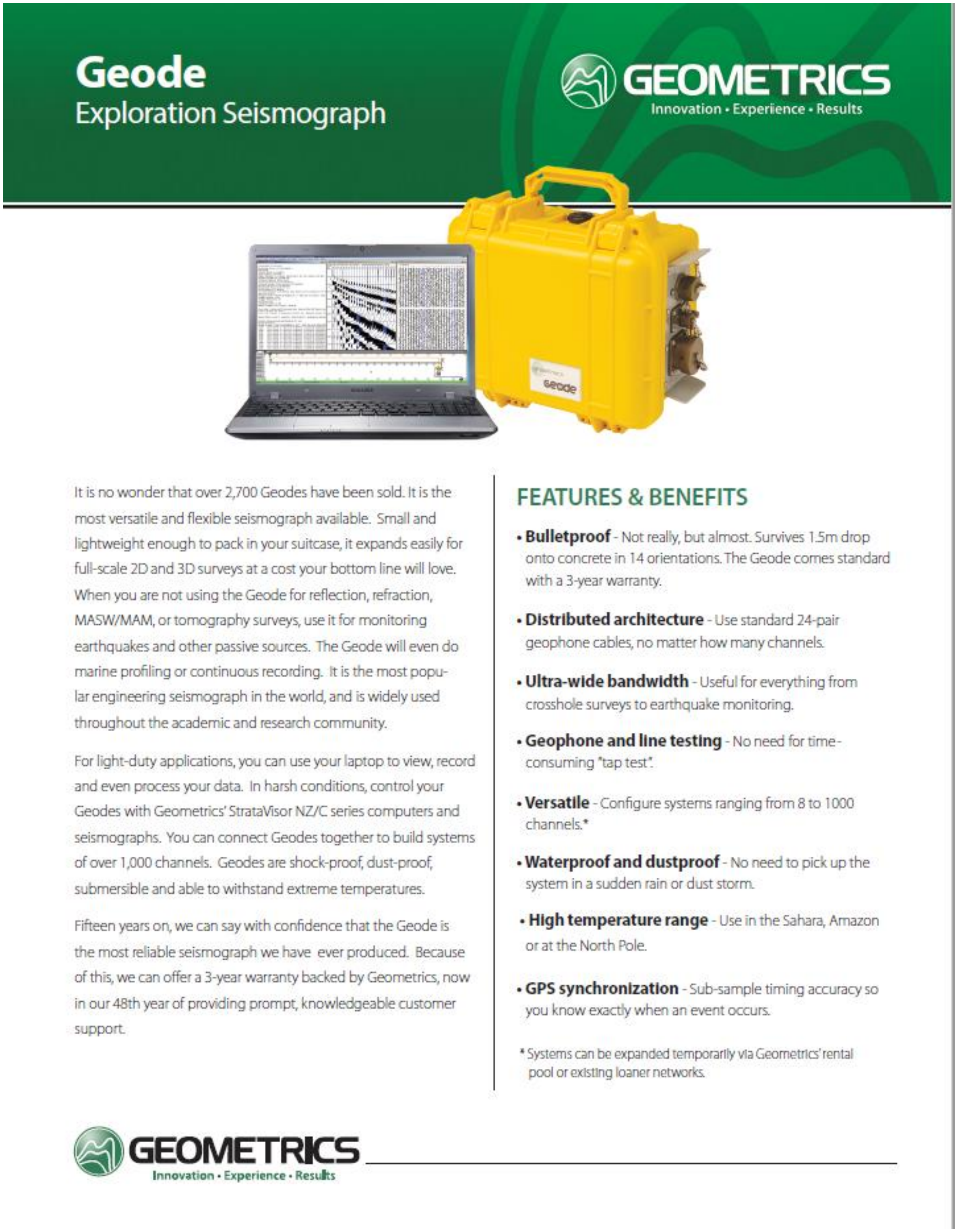
APPENDIX A

EQUIPMENT SHEETS



Geode

Exploration Seismograph

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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

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- **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.*
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- **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.

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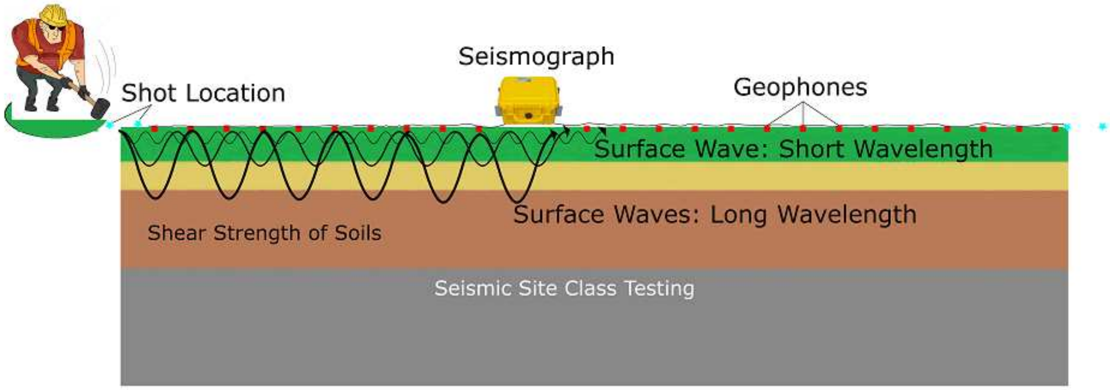
APPENDIX B
GEOPHYSICAL BACKGROUND

Multichannel Analysis of Surface Waves (MASW) and Micro-tremor Array Measurements are geophysical methods that use acoustic waves to measure the shear-wave velocity of the material within the subsurface. To measure this, the dispersion properties of Rayleigh surface waves are measured as a change in phase velocity with frequency. A general property of waves is that their energy decays exponentially with depth, known as the penetration depth, and frequency is inversely proportional to penetration depth. This simply means that lower frequency waves will travel deeper into the underlying strata, and their phase velocities will be influenced by the medium they are travelling in. The range of phase-velocities yields a Rayleigh wave dispersion curve and the inversion of this curve produces a shear-wave velocity (v_s) sounding. The shear wave velocity is useful because it is directly correlated to the stiffness of the underlying strata, known as the shear modulus. The figure below demonstrates the principles of this.

Common Applications of MASW include:

- Seismic Site Classification
- Compaction Evaluation
- Grouting Evaluation
- Anomaly Detection
- Soil-Bedrock Mapping



Site classification involves the principle of wave propagation, as previously described, and amplification of ground motion. The amplification of the ground motion is inversely proportional to the stiffness of the material. The higher the stiffness of the material, the lower the amplification of the ground motions. There are six seismic site classifications: A, B, C, D, E and F. The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2020 Edition (also OBC 2012) for more information on the requirements for the site classification.

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