REPORT

Geotechnical Investigation

Proposed Residential Development

1055 Cedar Creek Drive (BLOCK 232), Ottawa, Ontario

Submitted to:

DCR/Phoenix Group

18A Bentley Ave, Ottawa, ON K2E 6T8

Submitted by:

Golder Associates Ltd.

1931 Robertson Road, Ottawa, Ontario, K2H 5B7, Canada

+1 613 592 9600

19131587

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1.0 INTRODUCTION

This report provides geotechnical design guidance for the proposed residential development to be located on 1055 Cedar Creek Drive (Block 232) in Ottawa, Ontario. The previous investigation report titled: "*Geotechnical Investigation, Proposed Residential Development, Remer and Idone Lands, Ottawa, Ontario, report number 13-1121-0083 (1046), dated January 2017*", which was issued for the entire residential subdivision on the Remer and Idone Lands, was used to prepare this geotechnical report for the proposed development at Block 232.

The purpose of the subsurface investigations was to determine the general soil, bedrock, and groundwater conditions across the site by means of five boreholes and one test pit of previous investigations. Based on an interpretation of the factual information, obtained from the existing subsurface information available for the site from previous investigations, engineering recommendations are provided on the geotechnical design aspects of the proposed development, including construction considerations that could affect design decisions.

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

Plans are being prepared to develop residential buildings on 1055 Cedar Creek Drive in Ottawa, Ontario (see Key Plan, Figure 1).

The following information is known about the site and the proposed development:

- The site is Block 232 on Subdivision Plan 4M-1617 and measures approximately 165 m by 65 m in plan area.
- The site is bordered by Cedar Creek Drive to the east, by Salamander Way to the north and by Pingwi Place to the south. Some future developments will be constructed on the west side of the proposed development site.
- The site is proposed to be developed into seven multi-storey residential buildings (numbered building 1 to 7, inclusive), one accessory building and associated parking areas.
- Five buildings (including buildings 1, 2, 5, 6, 7) will be 3-storeys in height and will have one level of basement to be located on the northern, eastern and western portions of the site. Two buildings (including buildings 3 and 4) will be located on the southern portion of the site and will be 3-storeys in height and will have one level of underground parking garage.
- Based on the provided architectural drawings, the finished grades rage from about elevations 100.2 to 101.5 m and the underside of the footing elevations for the proposed buildings range between about 98.4 m and 100.6 m.
- The proposed accessory building will be a one storey slab on grade structure and will be located on the northern portion of the site. The top of the slab on grade is understood to be at elevation of 100.5 m.

The approximate locations of the relevant boreholes and test pits from the previous investigations are shown on the Site Plan, Figure 1.

Based on the results of those previous investigations, as well as a review of the published geological mapping, the subsurface conditions across this site are expected to predominantly consist of variable deposits of sands and silts, overlying bouldery glacial till, above bedrock. The bedrock surface is expected to vary at depths of about 3 to 6 m below the existing ground surface. Geological mapping indicates that the bedrock in the area consists of interbedded sandstone and dolomite of the March Formation.

3.0 SUBSURFACE CONDITIONS

3.1 General

Information on the subsurface conditions is provided as follows:

- Record of test pit, borehole, and drillhole sheets of the previous investigations by Golder (2013 and 2016) are provided in Appendix A.
- Record of borehole sheets of the previous investigation by others (2005 and 1990) are provided in Appendix B.

In general, the subsurface conditions at this site consist of topsoil, over variable thickness of sand and silt deposits underlain by glacial till, over dolostone bedrock.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes and test pit advanced during the previous investigations. It is assumed in the sections below that the site has not been altered since those investigations were completed (i.e., no stripping, excavation or filling has been carried out on the site).

3.2 Topsoil

Topsoil consisting of silty sand to sandy silt existed at the ground surface at all borehole and test pit locations. The thickness of the topsoil ranged from about 0.2 to 0.3 m.

3.3 Sand and Silt

The topsoil was generally underlain by variable deposits of sand and silt. These deposits predominantly consist of sand, silty sand to sandy silt and silt, with varying amounts of gravel, cobbles and boulders. These deposits extend to depths ranging from about 0.8 to 4.2 m below the ground surface.

SPT “N” values in the sand and silt deposits ranged widely, from 4 to >50 blows per 0.3 m of penetration, indicating a loose to very dense state of packing.

The measured water contents of samples from these deposits vary from 8 to 20 percent.

3.4 Glacial Till

Glacial till exists beneath the sand and silt deposits at all the borehole locations, except at test pit 16-6. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt. The glacial till was not penetrated at the test holes, with the exception of borehole 16-103, but was proven extend to depths ranging between about 2.3 and 6.1 m beneath the existing ground surface prior to encountering refusal to augering or excavating or being terminated. At borehole 16-103, the glacial till extends to a depth of 5.1 m.

SPT “N” values obtained in this deposit were reported to be greater than 50 blows per 0.3 m of penetration, indicating a very dense state of packing. However, some but not all of the “N” values likely reflect the presence of cobbles and boulders within the deposit or the bedrock surface, rather than the actual state of packing of the soil matrix deposit.

The measured water contents of samples of the glacial till ranged from 9 to 16 percent.

3.5 Refusal or Bedrock

Practical refusal to augering or excavating was encountered in all boreholes and test pit (except borehole 16-105) at depths varying between about 2.3 to 6.1 m below the existing ground surface. Refusal may indicate the bedrock surface; however, it could also represent boulders within the glacial till.

Borehole 16-103 was extended into the bedrock to a depth of about 7.0 m using rotary diamond drilling techniques while retrieving NQ sized core.

The following table provides a summary of the ground surface elevation, depth to the bedrock surface, and the elevation of the bedrock surface; elevations are provided in m above sea level (masl).

Borehole / Test Pit Number	Ground Surface Elevation (masl)	Depth to Refusal (m)	Refusal Elevation (masl)
16-6	98.9	4.2	94.7
16-103	98.1	5.1	93.0
16-105	101.5	N/A	N/A
13-5	99.7	6.1	93.6
8	99.0	3.6	95.5
90.6	99.7	2.3	97.4

The bedrock encountered in borehole 16-103 consists of dolostone with shale interbeds. The bedrock is generally slightly weathered to fresh, thinly to medium bedded, and grey in colour.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples from borehole 16-103 ranged between 80 and 90 percent, indicating a very good to excellent rock quality.

3.6 Groundwater

The measured groundwater level in the standpipe piezometer installed in borehole 8 was about 3.2 m below the existing ground surface (or about elevation 95.8 m). Test pit 16-6 and borehole 16-105 were noted to be dry upon completion at depths of 4.20 and 5.0 m, respectively.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.0 DISCUSSION

4.1 General

This section of the report provides engineering recommendations on the geotechnical design aspects of this project based on our interpretation of the test hole information as well as the project requirements, and is subject to the limitations in the "Important Information and Limitations of This Report" attachment which follows the text of this report, but forms an integral part of this document.

4.2 Site Grading

In general, the subsurface conditions at this site consist of topsoil, overlying variable thicknesses of silts and sands, followed by glacial till, which is in turn underlain by bedrock. The depth to refusal varied, ranging from about 2.3 to 6.1 m below the existing ground surface.

From a foundation design perspective, no practical restrictions apply to the thickness of grade raise fill that may be placed within the proposed development. However, grade raises in excess of 4 m should be reviewed and approved.

For predictable performance of the structures, roadways, and site services, preparation for filling of the site should include stripping the existing topsoil. The topsoil is not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only. In areas with no structures, roadways or services, the existing topsoil may be left in place provided some long term settlement of the ground surface following filling can be tolerated.

4.3 Material Reuse

The native soils encountered at this site are not considered to be generally suitable for reuse as structural or engineered fill. Within foundation areas, imported engineered fill should be used.

The silt and sand deposits and glacial till may be suitable for use as controlled fill beneath pavement areas, provided they are not too wet to place and compact. Glacial till encountered below the groundwater may be too wet to feasibly be used as controlled fill. These materials could however be reused in non-structural areas (i.e., landscaping).

4.4 Foundations

The native undisturbed, inorganic overburden soils encountered at the site are considered suitable for supporting the proposed residential buildings. Topsoil and fill (if encountered) would not be considered suitable to support the building foundations and therefore must be removed from underneath the building footings and slabs.

For frost protection purposes, exterior footings for buildings should be founded at least 1.5 m below finished exterior grade. Isolated footings in unheated areas should be provided with at least 1.8 m of soil for frost protection (see Section 5.6 below). Any subexcavation below underside of the footing elevations should be removed and replaced with engineered fill. The engineered fill should consist of Ontario Provincial Standard Specification (OPSS) Granular B Type II, placed in maximum 300 mm thick lifts, and compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The engineered fill material must be placed within the full zone of influence of the building foundations. The zone of influence is considered to extend out and down from the edge of the perimeter footings at a slope of 1 horizontal to 1 vertical (1H:1V).

Strip or pad footings, up to 3 m in width, placed on the surface of the native soils or on engineered fill may be designed using a maximum allowable net bearing pressure of 150 kPa at serviceability limit states (SLS) and a factored bearing resistance at ultimate limit states (ULS) of 250 kPa.

The post-construction total and differential settlements of footings sized using the above maximum allowable net bearing pressure should be less than about 25 and 15 mm, respectively, provided that the subgrade at or below founding level is not disturbed by groundwater inflow or construction traffic.

The overburden materials on this site, in particular the glacial till deposit, contain cobbles and boulders. Any cobbles or boulders in footing areas which are loosened by the excavation process should be removed (and not pushed back into place) and the cavity filled with lean concrete. Otherwise, recompression of the disturbed soils could lead to larger than expected post-construction settlements.

There may be portions of the site where the shallow sand and silt deposits will be exposed at footing/subgrade level. Prior to construction of footings or the placement of engineered fill within these areas, the surface of the native sandy and silty materials should be proof rolled to provide surficial densification of any loose or disturbed material.

4.5 Seismic Design

The seismic design provisions of the 2012 Ontario Building Code (OBC) depend, in part, on the shear wave velocity of the upper 30 m of soil and/or bedrock below founding level. Based on the 2012 OBC methodology, this site can be assigned a Site Class of D.

More favourable Site Class values could potentially be assigned for portions of the site if shear wave velocity testing were carried out.

The soils at this site are not considered liquefiable under earthquake loadings.

4.6 Frost Protection

The native subgrade soils on this site are considered to be highly frost susceptible. Therefore, all exterior perimeter foundation elements or foundation elements in unheated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior footings 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. Insulation could be provided as an alternative to earth cover for frost protection. Golder can provide further details if required.

4.7 Excavations for Basements and Underground Parking Garages

Based on the provided preliminary drawings, the underside of the footing elevations for the proposed buildings will range between about 98.4 m and 100.6 m. Therefore, the excavations for the underground levels (i.e., basements or underground parking garages) will be through the topsoil and into the underlying sand and silt deposits and/or glacial till. Bedrock excavation is not anticipated for the basements or underground parking excavations.

No unusual problems are anticipated in excavating the overburden materials using conventional hydraulic excavating equipment, recognizing that large boulders (which may be nested) will likely be encountered in the glacial till. Boulders larger than 0.3 m in size should be removed from the excavation side slopes, for worker safety.

Based on the measured groundwater levels in the previous investigations, excavations to the underside of footing depth are not expected to extend below groundwater level. However, perched water or seasonal fluctuations in groundwater level should be anticipated. It is considered that it should generally be possible to handle the groundwater inflow into the excavations by pumping from well filtered sumps in the floor of the excavations. Where the subgrade is found to be wet and sensitive to disturbance, consideration should be given to placing a mud slab of lean concrete over the subgrade (following inspection and approval by geotechnical personnel), or a 150 mm thick layer of OPSS Granular A underlain by a non-woven geotextile, to protect the subgrade from construction traffic.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the overburden materials above the groundwater table (or where the groundwater level is lowered below the floor of the excavation) would generally be classified as a Type 3 soil and therefore, the side slopes should be stable in the short term at 1 horizontal to 1 vertical. Boulders larger than 0.3 m in diameter should be removed from the excavation side slopes for worker safety.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavations. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity.

It is understood that a Category 3 Permit-To-Take-Water (PTTW) has been obtained from the Ministry of the Environment and Climate Change (MOECC) for this development and no further registration would be required.

4.8 Basement and Underground Parking Floor Slabs

In preparation for the construction of the basement and underground parking floor slabs, all loose, wet, and disturbed material should be removed from beneath the floor slabs. Provision should be made for at least 200 mm of 19 mm crushed clear stone to form the base of the floor slabs. The underslab fill should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

To prevent hydrostatic pressure build up beneath the basement and underground parking floor slabs, it is suggested that the granular base for the floor slabs be positively drained. This could be achieved by providing a hydraulic link between the underfloor fill material and the exterior drainage system.

A geotextile should be provided between the clear stone underslab fill and the subgrade soil (which will likely consist of the finer sand and silt deposits), to avoid loss of fine soil particles from the subgrade soil into the voids in the clear stone and ultimately into the drainage system. The geotextile should consist of a Class II non-woven geotextile with a Filtration Opening Size (FOS) not exceeding 100 microns, in accordance with OPSS 1860.

4.9 Slab on Grade (For Accessory Building)

Conventional slab on grade construction can be used for the proposed accessory building.

For predicable performance of the slab, all topsoil beneath the proposed floor slab should be removed. Once removed, proof-rolling of the existing subgrade materials is recommended to provide surficial densification of the existing subgrade materials and to identify any isolated areas of soft or loose soils which would need replacing.

Provision should be made for at least 150 mm of OPSS Granular A to form the base for the floor slab. Any bulk fill required to raise the grade to the underside of the Granular A should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

4.10 Basement Walls and Foundation Wall Backfill

The soils at this site are frost susceptible and should not be used as backfill directly against exterior, unheated, or well insulated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should either be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I or, alternatively, a bond break such as the Platon system sheeting could be placed against the foundation walls.

Drainage of the wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to a positive outlet, such as an adjacent storm sewer or sump pit. Conventional damp proofing of the basement walls is appropriate with the above design approach.

Basement walls made within open cut excavations, backfilled with granular material, and effectively drained as described above should be designed to resist lateral earth pressures calculated using a triangular distribution of the stress with a magnitude of:

$$\sigma_h(z) = K_o(\gamma z + q)$$

Where:

	$\sigma_h(z)$	=	Lateral earth pressure on the wall at depth z, kilopascals
	K_o	=	At-rest earth pressure coefficient, use 0.5
	γ	=	Unit weight of retained soil, 21.5 kilonewtons per cubic metre
	z	=	Depth below top of wall, m
	q	=	Uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (use 12 kilopascals as a minimum)

The lateral earth pressure equation given above is in an unfactored format and will need to be factored for Limit States Design purposes.

These lateral earth pressures would increase under seismic loading conditions. 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). The combined pressure distribution (static plus seismic) may be determined as follows:

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

Where:

K_{AE}	=	The seismic earth pressure coefficient, use 0.8 for a non-yielding wall
Ka	=	Active earth pressure coefficient, use 0.34
H	=	The total depth to the bottom of the foundation wall, m

4.11 Site Servicing

Excavations for the installation of site services will be made through the topsoil, silt and sand deposits, glacial till and bedrock. Based on the anticipated groundwater levels at this site, the excavations for site servicing may extend near or just below the groundwater level.

No unusual problems are anticipated in excavating in the overburden using conventional hydraulic excavating equipment, recognizing that large boulders may be encountered in the glacial till. Boulders larger than 0.3 m in size should be removed from the excavation side slopes, for worker safety.

Excavation side slopes above the water table should be stable in short term at 1H:1V (i.e., for Type 3 soils per OSHA of Ontario). Excavation side slopes below groundwater level will need to be at 3H:1V (i.e., Type 4 soils).

The stand up time for exposed side slopes will be extremely short and the subgrade will be disturbed if left exposed for any length of time. Construction of site services should be planned to be carried out in short sections, which can be fully completed in a minimal amount of time. The rate of groundwater inflow from the overburden could be significant. Based on past experience on the adjacent sites and particularly where the excavations are deeper and/or where the overburden is coarser, some pre-drainage of the overburden will be required. For example, several sumps could be constructed, and pre-pumping of the overburden carried out.

Alternatively, excavations within the overburden soils could also be carried out within a fully braced steel trench box, which would minimize the width of the excavation. The use of a trench box will not, however, eliminate the potential for disturbance outside the trench box limits.

Some groundwater inflow through the overburden into the service excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps in the excavations, provided that multiple suitably sized pumps are used.

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 does occur, it may be necessary to place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density. 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 or sandy soils on the trench walls could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from 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 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the overburden soils as trench backfill. Material from below the water table may be re-used provided that it can be adequately placed and compacted.

Some of the overburden materials below the water table may be too wet to compact. Where that is the case, these materials should be wasted (and drier materials imported) or these materials should be placed only in the lower portions of the trench, recognizing that some future ground settlement over the trenches will likely occur.

In that case, it would also be prudent to delay final paving for as long as practical and significant padding of the roadways may be required in these areas prior to final paving.

Boulders larger than 300 mm in diameter will also interfere with the backfill compaction and should be removed from the excavated material prior to re-use as backfill.

Where the trench will be covered with hard surfaced areas, the type of native material placed in the frost zone (between subgrade level and 1.8 m 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 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

4.12 Pavement Design

In preparation for pavement construction, all topsoil should be removed from all pavement areas.

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material (SSM). These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 percent of the materials' standard Proctor maximum dry density using suitable compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 m in four orthogonal directions or longitudinally where parallel to a curb.

The pavement structure for local roads or parking lots, which will not experience bus or truck traffic (other than school bus and garbage collection), should consist of:

Pavement Component	Thickness (millim)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	375

The pavement structure for collector roadways which will experience bus and/or truck traffic should consist of:

Pavement Component	Thickness (millim)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

If any of the roadways would be categorized as 'collector' roadways and/or will experience bus or truck traffic (other than school buses, garbage trucks, moving trucks, etc.), then additional pavement design recommendations will need to be provided.

The granular base and subbase materials should be uniformly compacted to at least 100 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310. The composition of the asphaltic concrete pavement should be as follows:

- Superpave 12.5 mm Surface Course – 40 mm
- Superpave 19 mm Base Course – 50 mm

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

The above pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have 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.

4.13 Tree Planting Restrictions

Silty clay soils in the Ottawa area are highly sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the silty clay, the silty clay undergoes shrinkage which can result in settlement of adjacent structures.

Based on the results of the previous investigations, no silty clay encountered within the referenced boreholes and test pit locations. No restrictions on the types or sizes of trees that may be planted or tree to foundation setback distances need to be considered for this development.

4.14 Corrosion and Cement Type

No chemical analyses on soil samples were performed as part of the current investigation. It is recommended to carry out chemical analysis related to potential corrosion of exposed buried ferrous elements and potential sulphate attack on buried concrete elements on a few selected soil samples retrieved from the founding level of the proposed buildings foundations, during the construction phase. The results can be obtained quickly without significantly interrupting the construction schedule and additional instructions on the cement type and corrosion potential can be provided accordingly.

5.0 IMPACTS TO ADJACENT PROPERTIES OR INFRASTRUCTURE

Based on the information available to Golder at the time of this report, excavations for foundations or services at this site will not be within the zone of influence (defined within a line drawn from the existing underside of foundation or utility invert at an angle of 1 horizontal to 1 vertical) of existing structures or utilities. The planned excavations should therefore not have an impact on adjacent properties or utilities.

6.0 ADDITIONAL CONSIDERATIONS

The soils on this site are sensitive to disturbance from ponded water, construction traffic, and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soils having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

The test pits excavated and backfilled during the previous investigations constitute zones of disturbance to the native soils. The presence of the backfill materials could affect the performance of surface structures or other settlement-sensitive facilities should they be constructed above the zone of influence of those locations. In such cases, the excavated soil should be removed and replaced with engineered fill.

Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

7.0 CLOSURE

We trust that this report meets your current requirements. If you have any questions, or if we may be of further assistance, please contact the undersigned.

Signature Page

Golder Associates Ltd.

Ali Ghirian, P.Eng.
Geotechnical Engineer

Bill Cavers, P.Eng.
Associate, Senior Geotechnical Engineer

AG/WC/hdw

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IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT

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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.

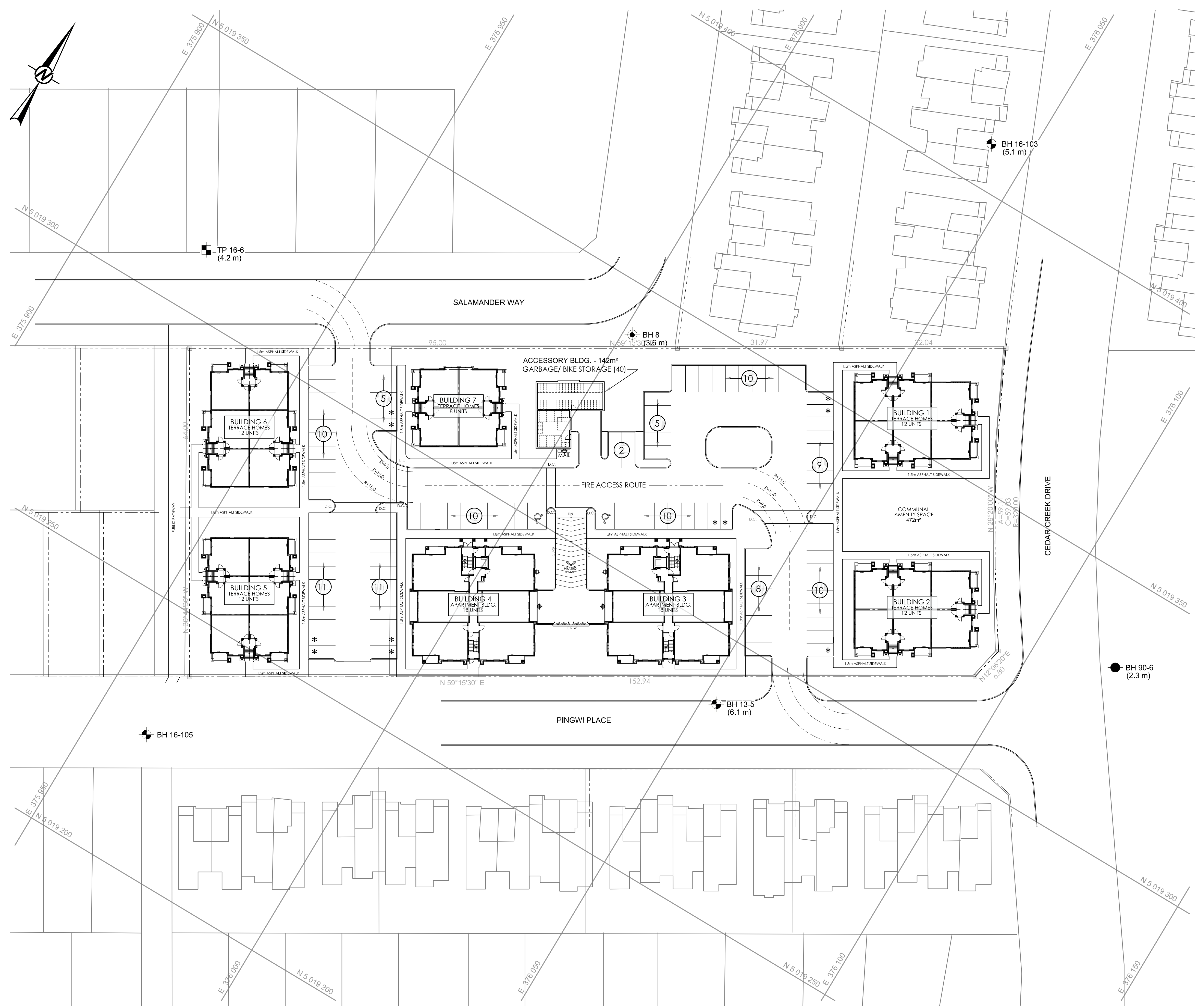
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.



KEY MAP

SCALE: 1:50,000 m

LEGEND

- APPROXIMATE BOREHOLE LOCATION - PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT NO. 13-1121-0083
- APPROXIMATE TEST PIT LOCATION - PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT NO. 13-1121-0083
- APPROXIMATE BOREHOLE LOCATION - PREVIOUS INVESTIGATION BY JACQUES WHITFORD, REPORT NO. 30227-1
- APPROXIMATE BOREHOLE LOCATION - PREVIOUS INVESTIGATION BY JDP, REPORT NO. PG0627
- (2.3 m) DEPTH TO REFUSAL

REFERENCE(S)

1. BASE PLANS PROVIDED IN DIGITAL FORMAT BY DCR PHOENIX GROUP, DRAWING NOS. Block 232_2019_06_27 (2) (1).pdf, RECEIVED ON NOV. 15, 2019
2. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83 CSRS, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 12.5 25
1:750 METRES

CLIENT
DCR PHOENIX GROUP

PROJECT
GEOTECHNICAL INVESTIGATION
1055 CEDAR CREEK DRIVE (BLOCK 232),
OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT
GOLDER

YYYY-MM-DD	2019-11-20
DESIGNED	AG
PREPARED	ZS
REVIEWED	AG
APPROVED	WC

PROJECT NO.	CONTROL	REV.	FIGURE
19131587	0001	A	1

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

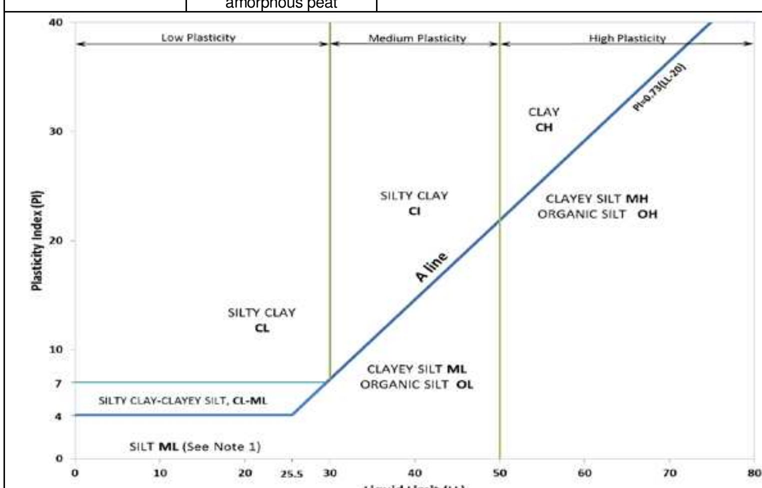
APPENDIX A

Method of Soil Classification and Terms
Lithological and Geotechnical Rock Description Terminology
Record of Boreholes and Test Pits – Previous Investigations by Golder

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

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

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators: Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

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

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

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

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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

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

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

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

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

Field Moisture Condition

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

COHESIVE SOILS

Consistency

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

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

Water Content

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of rock material weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index
A count of the number of naturally occuring discontinuities (physical separations) in the rock core. Mechanically induced breaks caused by drilling are not included.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^{o} angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

TABLE 1 (Continued)

RECORD OF TEST PITS AND HAND AUGERHOLES

Test Pit Number (Elevation m)	Depth (m)	Description
16-6	0.00 – 0.19	TOPSOIL – (ML) sandy SILT; dark brown; moist
(98.93 m)	0.19 – 2.30	(SM) SILTY SAND, some gravel; brown; non-cohesive, moist
	2.30 – 4.20	(SP) SAND, some non-plastic fines and gravel; brown, contains cobbles and boulders up to 1.0 metres in diameter; non-cohesive, moist
	4.20	End of Test Pit – Refusal to excavating on probable bedrock

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.19 – 1.10
2	1.10 – 2.30
3	2.30 – 4.20

RECORD OF BOREHOLE: 16-103

PROJECT: 13-1121-0083-1046

LOCATION: See Site Plan

BORING DATE: October 5, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)
	GROUND SURFACE	98.05
Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL - (ML) sandy SILT; dark brown; moist	0.00
Power Auger, 200 mm Diam. (Hollow Stem)	(SM/ML) SILTY SAND to sandy SILT, some gravel; brown; non-cohesive, dry, loose to very dense	0.17
Power Auger, 200 mm Diam. (Hollow Stem)	(SM) SILTY SAND, some gravel; brown to grey brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, dry to moist, very dense	95.76 / 2.29
Rotary Drill, NQ Core	Fresh, thinly to medium bedded, grey, fine grained DOLOSTONE BEDROCK, with shale interbeds	92.95 / 5.10
	End of Borehole	91.04 / 7.01

SAMPLE NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	4	
2	SS	>50	
3	SS	100	
4	SS	>50	
5	SS	>50	MH
6	SS	59	
7	SS	>50	
C1	RC	DD	
C2	RC	DD	

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^{-8} 10^{-6} 10^{-4} 10^{-2}

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

PIEZOMETER OR STANDPIPE INSTALLATION

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

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: KM

CHECKED: CK

PROJECT: 13-1121-0083-1046

RECORD OF DRILLHOLE: 16-103

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: October 5, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-850

DRILLING CONTRACTOR: CCC

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	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE		92.95															
6	Rotary Drill, NQ Core	Fresh, thinly to medium bedded, grey, fine grained DOLOSTONE BEDROCK, with shale interbeds		5.10	1	30													
					2	40													
7				91.04 7.01															
		End of Drillhole																	

Depth scale: 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: KM

CHECKED: CK

PROJECT: 13-1121-0083-1046

RECORD OF BOREHOLE: 16-105

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: October 5, 2016

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	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^{-8} 10^{-6} 10^{-4} 10^{-2}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		101.46							
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (ML) sandy SILT; dark brown; moist		0.00							
		(SM/ML) SAND and SILT, trace gravel; grey brown; non-cohesive, moist, loose		101.28 0.18	1	SS	6				
1		(SM) SILTY SAND, some gravel; brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, dry to moist, very dense		100.70 0.76	2	SS	76				
					3	SS	>50				
2					4	SS	>50				
3					5	SS	>50				
4					6	SS	>50				
					7	SS	>50				
5		End of Borehole		96.43 5.03							Open borehole dry upon completion of drilling
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: KM

CHECKED: CK

PROJECT: 13-1121-0083

RECORD OF BOREHOLE: 13-5

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 26 & 27, 2013

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	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^{-8} 10^{-6} 10^{-4} 10^{-2}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	99.74							
0	Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL	0.00							
		Brown SILTY SAND	99.46 / 0.28	1	50 DO	8				
1		Dense to very dense brown SILTY SAND, trace gravel and clay, with cobbles and boulders (GLACIAL TILL)	98.98 / 0.76	2	50 DO	49		○		
				3	50 DO	>50				
2	Wash Boring, HQ Core	BOULDER	97.89 / 1.85	4	NQ RC	DD				
		Very dense brown SILTY SAND, trace gravel and clay, with cobbles and boulders (GLACIAL TILL)	97.55 / 2.19	5	50 DO	>50				
3				6	50 DO	>50				
4		Very dense SANDY SILT, trace gravel and clay, with cobbles and boulders (GLACIAL TILL)	95.93 / 3.81	7	50 DO	>50				
		BOULDER	95.17 / 4.57	8	NQ RC	DD				
5		Very dense grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	94.63 / 5.11	9	50 DO	>100				
6		End of Borehole Auger Refusal	93.64 / 6.10							
7										
8										
9										
10										

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: ALB

CHECKED: PAS

MIS-BHS 001 1311210083.GPJ GAL-MIS.GDT 01/20/16 JM/JEM

APPENDIX B

Record of Boreholes – Previous Investigations by others

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Development, Bank Street at Blais Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis O'Sullivan Vollebekk Surveying.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 20 JUL 05

FILE NO. PG0627

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.03
TOPSOIL 0.15							
Compact, brown SILTY fine SAND 0.76		AU	1				
GLACIAL TILL: Very dense, brown silty fine sand with gravel, cobbles and boulders		SS	2	47	79+	1	98.03
		SS	3	67	50+	2	97.03
		SS	4	78	50+	3	96.03
3.56		SS	5	33	77+		
End of Borehole							
Practical refusal to augering @ 3.56m depth							
(GWL @ 3.18m-Sep. 6/05)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

90-6

CLIENT: Ship & Krakow Architects

LOCATION: Leitrim, Ontario

DATES: BORING 90-06-27

WATER LEVEL

PROJECT No. 30067

BOREHOLE No. 90-6

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.70	Ground Surface						
	99.4	Dark brown, TOPSOIL and ROOTMAT						
	98.1	Brown and grey, fine to medium, SAND, trace silt						
	97.4	Very dense, brown, silty sand, some gravel and rock fragments at top of bedrock, TILL			SS	1	250	*
		End of Borehole (Bedrock)						

* Split spoon refusal

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Depth scale (m): 0 1 2 3 4 5 6 7 8 9 10

△ Pocket Penetrometer Test

□ Field Vane Test

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