Geotechnical Investigation

Proposed High Rise Development

99 Parkdale Avenue

Ottawa, ON

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Table of Contents

1.0 INTRODUCTION 4

2.0 DESCRIPTION OF PROJECT AND SITE 4

3.0 PROCEDURE 5

4.0 SUBSURFACE CONDITIONS 6

4.1 General 6

4.2 Fill and Topsoil 6

4.3 Glacial Till 6

4.4 Bedrock 6

4.5 Groundwater and Hydraulic Conductivity 8

5.0 PROPOSED HIGH RISE DEVELOPMENT 9

5.1 General 9

5.2 Excavation 9

5.3 Groundwater Management 10

5.4 Foundations 11

5.5 Seismic Design 12

5.6 Basement Floor Slab 12

5.7 Frost Protection 12

5.8 Basement Walls 13

5.8.1 Open Cut Excavations 13

5.8.2 Excavations in Bedrock 13

5.8.3 Lateral Earth Pressures 13

5.9 Impacts on Adjacent Development 15

5.10 Environmental Considerations 16

5.11 Corrosion and Cement Type 16

6.0 ADDITIONAL CONSIDERATIONS 17

Important Information and Limitations of This Report

FIGURES

Figure 1 – Key Plan
Figure 2 – Site Plan

APPENDICES

APPENDIX A
Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets
Current Investigation

Current Investigation APPENDIX B
Borehole and Test Pit Logs
Previous Investigations

APPENDIX C
Unconfined Compressive Strength of Rock Core Testing Results
Bedrock Core Photos

APPENDIX D
Geophysical VSP Test Results

APPENDIX E
Results of Basic Chemical Analysis
Exova Laboratories Report Number 1217166

1.0 INTRODUCTION

This updated report presents the results of a geotechnical investigation carried out for a proposed high rise development to be located at 99 Parkdale Avenue in Ottawa, Ontario.

The purpose of this subsurface investigation was to determine the general soil, bedrock, and groundwater conditions across the site by means of three boreholes and, based on an interpretation of the factual information obtained, along with the existing subsurface information available for the site, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The geotechnical investigation was carried out concurrently with a hydrogeological assessment. The results of the hydrogeological assessment were provided to Urbandale Construction in an updated letter titled "Hydrogeological Assessment, Predicted Groundwater Inflow and Radius of Influence, 99 Parkdale Avenue, Ottawa, Ontario" dated July 23, 2019.

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

2.0 DESCRIPTION OF PROJECT AND SITE

Consideration is being given to the design and construction of a high rise development to be located at 99 to 107 Parkdale Avenue (known as the 99 Parkdale Avenue site) in Ottawa, Ontario (see Key Plan, Figure 1).

The following is known about the existing property:

- The site measures about 30 metres by 45 metres in plan area.
- The site was previously occupied by 4 residential houses and several out-buildings, all of which were removed prior to this investigation being carried out.
- The site is bordered to the north by an 11 storey building, to the west by Parkdale Avenue, to the south by a 2 storey apartment building, and to the east by a lane way and a concrete parking garage.

Although preliminary in nature, current plans indicate:

- The new development will occupy essentially the entire site.
- The proposed structure will vary from 2 storeys in height (western and southern portions of the building) to about 30 storeys in height (within the "core").
- The structure will have 6 basement levels, which will be located beneath the entire superstructure.

Golder Associates carried out a preliminary geotechnical investigation for this development in February 2012. The results of that investigation were provided in a report to Urbandale Construction titled "Preliminary Geotechnical Investigation, Proposed High Rise Development, 99 Parkdale Avenue, Ottawa, Ontario" dated February 2012 (Report No. 11-1121-0275-1000).

In addition, Golder Associates or McRostie Genest St-Louis & Associates carried out the following investigations near by.

1) Report to P.W.G.S.C. by McRostie Genest St-Louis & Associates Ltd. titled "Subsurface Investigation, For Proposed Watermain Upgrading at Tunney's Pasture, Ottawa, Ontario" dated March 1997 (project number SF-4437).
2) Report to Winbro Homes Inc. by Golder Associates Ltd. titled "Subsurface Investigation, Proposed Condominium & Parking Garage, Forward Avenue, Ottawa, Ontario" dated February 1985 (project number 851-2019).
3) Report to Varriano Holdings Ltd. by Golder Associates Ltd. titled "Subsurface Investigation, Proposed Condominium Building, Parkdale Avenue, Ottawa, Ontario" dated March 1984 (project number 841-2101).

Golder Associates also carried geophysical testing on an adjacent Tunney's Pasture site for Public Works and Government Services Canada in 2011.

Based on the results of the previous investigations, the subsurface conditions on this site consist of topsoil and fill overlying glacial till, with the bedrock surface at depths varying from about 0.5 to 1.5 metres below the existing ground surface.

Published bedrock geology mapping indicates that the site is underlain by dolomite and limestone of the Bobcaygeon Formation.

3.0 PROCEDURE

The field work for this investigation was carried out on July 16 and 17, 2012. At that time, 3 boreholes (numbered 12-101, 12-102, and 12-103) were put down at the approximate locations shown on the Site Plan, Figure 2.

The boreholes were advanced using a track mounted hollow stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced through the overburden to the bedrock surface. Boreholes 12-101 and 12-102 were then extended into the underlying bedrock to a depth of about 25.4 metres below the existing ground surface using NQ sized rotary coring equipment. In situ hydraulic conductivity packer testing was carried out at regular overlapping intervals of depth within the boreholes.

Monitoring wells were sealed into boreholes 12-101 and 12-102 to permit subsequent groundwater level measurement and in situ rising and falling head hydraulic conductivity testing.

The field work was supervised by an experienced technician from our staff who located the boreholes, directed the drilling and packer testing operations, logged the boreholes and samples, directed the in situ testing, and took custody of the soil and bedrock samples retrieved.

On completion of the drilling operations, samples of the soils and bedrock encountered in the boreholes were transported to our laboratory for examination by the project engineer and for laboratory testing. The laboratory testing included unconfined compressive strength testing on two samples of the bedrock.

A groundwater sample from borehole 12-101 was submitted to EXOVA Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The groundwater levels in the monitoring wells in boreholes 12-101 and 12-102 were measured on July 27, 2012. In situ hydraulic conductivity testing was also carried out at that time.

The borehole locations were selected, located in the field, and surveyed by Golder Associates Ltd. The ground surface elevation at each borehole location was referenced to a job benchmark, which was indicated to be the top-of-spindle of a fire hydrant located just west of the site (see Figure 2). The elevation of this benchmark was indicated to be 61.07 metres on a plan provided by Annis, O'Sullivan, Vollebekk Ltd.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes put down for the current investigation are shown on the Record of Borehole and Drillhole Sheets in Appendix A. The subsurface conditions encountered in the boreholes put down for previous investigations are shown on the borehole logs in Appendix B. The subsurface conditions encountered in the test pits excavated for the preliminary investigation are shown on the Record of Test Pits in Appendix B. The results of the laboratory unconfined compressive strength testing carried out on two samples of the bedrock as well as core photos are provided in Appendix C. A summary of the geophysical testing carried out by Golder Associates on an adjacent site is provided in Appendix D. The results of the basic chemical analyses carried out on a sample of groundwater from borehole 12-101 are provided in Appendix E.

A detailed description of the subsurface conditions encountered at each borehole and test pit are provided in Appendices A and B.

In general, the subsurface conditions on this site consist of topsoil and fill overlying glacial till, with the bedrock surface at depths varying from about 0.5 to 1.3 metres below the existing ground surface.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes and test pits advanced on the site.

4.2 Fill and Topsoil

A surficial layer of fill exists at all of the borehole and test pit locations, with the exception of TP 12-5. The fill varies from approximately 0.1 to 0.9 metres in thickness and consists of asphaltic concrete, crushed stone, silty sand, bricks, and sand and gravel.

Topsoil existed at the ground surface at TP 12-5 and buried beneath the fill in TP 12-3 at the time of the previous investigation. The topsoil was approximately 0.5 and 0.2 metres in thickness, respectively, at the test pit locations

4.3 Glacial Till

A deposit of glacial till underlies the topsoil and fill. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt and silty sand with a trace to some clay. The deposit varies from about 0.2 to 0.9 metres in thickness.

4.4 Bedrock

The fill and glacial till are underlain by bedrock.

The depths and elevations of the bedrock surface, as well as the ground surface elevations at the borehole and test pit locations, are shown in the following table.

Borehole/ Test Pit Number	Ground Surface Elevation at Borehole or Test Pit (m)	Bedrock Depth at Borehole or Test Pit (m)	Bedrock Surface Elevation at Borehole or Test Pit (m)
12-1	59.99	0.90	59.09
12-2	59.89	1.20	58.69
12-3	60.31	1.30	59.01
12-4	60.18	0.45	59.73
12-5	60.10	0.85	59.25
12-6	60.25	0.70	59.55
12-101	59.90	0.91	58.99
12-102	59.75	0.91	59.84
12-103	60.11	0.86	59.25

The above data is also shown on Figure 2.

Boreholes 12-101 and 12-102 were extended into the bedrock while retrieving NQ sized bedrock core. Based on the core retrieved, the bedrock on this site consists of thinly to medium bedded limestone with dark grey dolomitic limestone layers and partings. Published geological mapping indicates that this bedrock is of the Bobcaygeon Formation.

The Rock Quality Designation (RQD) values range from about 80 to 100 percent indicating good to excellent quality rock. Photos of the bedrock core are provided in Appendix C.

Details on the fracture frequency, Total Core Recovery (TCR), and Solid Core Recovery (SCR) are shown on the Record of Drillhole Sheets provided in Appendix A.

Laboratory unconfined compressive strength testing was carried out on two selected samples of the bedrock core. The results of the compressive strength testing are provided in Appendix C, and are summarized in the following table.

Borehole Number	Elevation of Bedrock Sample (m)	Depth of Bedrock Sample (m)	Unconfined Compressive Strength (MPa)
12-101	38.0	21.9	199
12-102	35.4	24.4	96

The results indicate compressive strengths of about 199 and 96 megapascals, indicating a medium strong to very strong rock.

4.5 Groundwater and Hydraulic Conductivity

Monitoring wells were installed in boreholes 12-101 and 12-102. The groundwater levels were measured on July 27, 2012. The results are provided in the following table.

Borehole Number	Date of Measurement	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)
12-101	July 27, 2012	59.90	2.1	57.8
12-102	July 27, 2012	59.75	6.2	53.5

No groundwater seepage was observed within the test pits.

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

Hydrogeological tests were completed in borehole 12-101 and 12-102 to estimate the hydraulic conductivity (K) of the bedrock below the site. Constant head packer testing was carried out in both open boreholes within seven, approximately four metre length, overlapping intervals per borehole. Results from the packer testing were analyzed using the Houlsby (1976) method. In both boreholes, only the results obtained in the uppermost interval could be interpreted. The hydraulic conductivity in the lower six intervals were too low to measure using the available testing equipment. Hydraulic conductivity estimates for the uppermost interval in the two boreholes are presented in the following table:

	BH 12-101	BH 12-102
Top of Interval (mbgs)	1.7	2.7
Bottom of Interval (mbgs)	5.4	6.4
Estimated Hydraulic Conductivity (metres per second)	4×10^{-6}	3×10^{-6}

Note: mbgs – metres below ground surface

Following packer testing, a monitoring well was installed within each borehole, and slug tests were carried within the monitoring wells. Both falling and rising head tests were carried out in BH 12-101 whereas, due to the very slow recovery of the water level in the well, only a falling head test was carried out in BH 12-102. The results of the slug testing were analyzed using the Bouwer and Rice (1976) method. Hydraulic conductivity estimates for the slug tests in the two boreholes are presented in the following table:

	BH 12-101	BH 12-102
Top of Interval (mbgs)	1.8	16.2
Bottom of Interval (mbgs)	5.5	19.8
Estimated Hydraulic Conductivity – Falling Head Test (metres per second)	4×10^{-6}	3×10^{-9}
Estimated Hydraulic Conductivity – Rising Head Test (metres per second)	9×10^{-6}	--

Note: mbgs – metres below ground surface

The estimated K values for the upper portion of the bedrock ranged from about $3x10^{-6}$ to $9x10^{-6}$ metres per second (m/s) which is relatively consistent. The two methods for estimating K (packer testing and rising/falling head tests) demonstrate that the deeper bedrock formations were significantly less permeable than the upper portion.

5.0 PROPOSED HIGH RISE DEVELOPMENT

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the proposed development based on our interpretation of the test hole information and project requirements.

The results and guidelines presented herein are 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.

5.2 Excavation

It is understood that the lowest basement floor level will be at about 18 metres depth. It is expected that the excavation will extend about 1 to 1.5 metres below that level, to accommodate the footing construction, such that the founding levels are expected to be between about 19.5 and 20.0 metres depth. The founding levels will therefore be within limestone bedrock.

No unusual problems are anticipated in excavating the existing fill and glacial till using conventional hydraulic excavating equipment, recognizing that some larger material may be encountered.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the soils above the water table at this site (which is the case) would generally be classified as Type 3 soils. Unsupported side slopes in the overburden *above the water table* may therefore be sloped no steeper than 1 horizontal to 1 vertical. Below the water table (if encountered, which could be the case during extremely wet periods of the year), the excavation side slopes will slough to a shallower inclination and could possibly remain stable at about 2 horizontal to 1 vertical. However, in accordance with the OHSA of Ontario, the soils below the water table would generally be classified as Type 4 soils, and excavation side slopes must be sloped no steeper than 3 horizontal to 1 vertical, or be shored.

Excavation into the bedrock will require drill and blast procedures or mechanical removal (e.g., hoe ramming). It is considered that mechanical removal will be too slow to be efficient for the bulk bedrock removal; however it might be appropriate for the localized deeper excavations for the footings and for final trimming of the excavation side after bulk blasting.

Blast induced damage to the bedrock must be avoided, otherwise rock reinforcement could be required. It should therefore be planned to either line drill the bedrock along the perimeter of the excavation at a close spacing in advance of blasting so that a clean bedrock face is formed, or to carry out perimeter drilling and pre-shearing of the excavation limits using controlled blasting.

Significant caution should be exercised in carrying out blasting due to the near proximity of existing buildings. The blasting should therefore be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field.

A pre-blast survey should be carried out of all of the surrounding structures and utilities. Selected existing interior and exterior cracks in the structures identified during the pre-blast survey should be monitored for lateral or shear movements by means of pins, glass plate telltales and/or movement telltales.

The contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This plan would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

The contractor should be limited to only small controlled shots. The following frequency dependent peak vibration limits at the nearest structures and services are suggested:

Frequency Range (Hz)	**Vibration Limits (millimetres/second)**
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

These limits should be practical and achievable on this project. Blasting will probably generate vibrations in excess of 40 Hz at the closest structures.

If practical, blasting should commence at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics and to confirm the anticipated ground vibration levels based on the contractor's blasting proposal.

The available information indicates that near vertical excavation walls in the bedrock should stand unsupported for the construction period. However, the exposed bedrock should be inspected by qualified geotechnical personnel at the time of excavation to confirm that assessment.

5.3 Groundwater Management

The geotechnical investigation was carried out concurrently with a hydrogeological assessment. The results of the hydrogeological assessment were provided to Urbandale Construction in a letter titled "Hydrogeological Assessment, Predicted Groundwater Inflow and Radius of Influence, 99 Parkdale Avenue, Ottawa, Ontario" and dated September 12, 2012. The hydrogeological letter provides a detailed discussion on the modeling and assessment results. The following provides a very brief summary regarding groundwater management.

Groundwater inflow to the excavation should be expected. The results of in situ rising head hydraulic conductivity testing carried out in the monitoring wells in boreholes 12-101 and 12-102 indicate hydraulic conductivity values for the bedrock of 10^{-9} to 10^{-6} metres per second.

The results of the hydrogeological modelling indicate that groundwater inflow into the excavation will decrease over time as the bedrock dewaters within the zone of influence. The initial groundwater inflows are estimated to be approximately 230,000 L/day, and are predicted to decrease to approximately 3,000 L/day as the construction dewatering progresses towards steady-state. The vast majority of the flow into the excavation will be from the bedrock near the surface of the site. During the progression to steady-state and once steady-state is reached, short-term increases in groundwater inflows would be expected following precipitation events where the weathered zone is recharged and subsequently drains into the excavation.

During the construction period, precipitation accumulation for the proposed excavation area would be approximately 126,000 L/day during a 70 mm precipitation event (return rate of 10 years, as observed at the Ottawa International Airport).

The steady-state groundwater inflow is predicted to be approximately 3,000 L/day (assuming the upper weathered zone remains dewatered). Increases in post-construction flows would be expected following precipitation events where the weathered zone is recharged and subsequently drains into the post-construction sump.

The ability of the existing sewer system to accept the volume of pumped groundwater will need to be evaluated.

According to O.Reg. 63/16 and O.Reg 387/04, if the volume of water to be pumped from an excavation 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”. Alternatively, a Permit to Take Water (PTTW) is required from the Ministry of the Environment, Conservation and Parks (MECP) if a volume of water greater than 400,000 litres per day is to be pumped from the excavations. Based on the groundwater conditions observed at the site, water taking exceeding 50,000, but less than 400,000 litres per day may be required to dewater groundwater and incident precipitation from the excavation. As a result, EASR registration may be necessary for the water taking associated with the proposed work.

The design of the dewatering system, if required, should be the responsibility of the excavation contractor. An outlet (or outlets) should be identified which the contractor can use to dispose of the pumped groundwater and incident precipitation. The contractor will be responsible for ensuring that discharge of the water does not result in erosion, flooding or siltation. Based on the site location, it is expected that the contractor will propose to discharge the water to land, storm or sanitary sewers near the site (if applicable). In order for pumped groundwater to be discharged to a City sewer, it needs to meet the City of Ottawa Sewer Use By-law criteria, and a separate sewer discharge permit must be obtained. If the water is discharged to land or a storm sewer that is within 30 metres of a water body, O.Reg. 63/16 imposes certain conditions. Discharge of water taken under an EASR must be in accordance with a Discharge Plan (to be developed by the contractor).

Based on the results of the steady-state numerical modelling, it is not anticipated that a PTTW would be required for dewatering of post-construction (i.e., after construction of the development has been completed) groundwater inflows.

5.4 Foundations

In general, the subsurface conditions on this site consist of about 0.5 to 1.3 metres of fill and glacial till overlying limestone bedrock.

It is understood that the lowest basement floor level will be at about 18 metres depth. It is expected that the excavation will extend about 1 to 1.5 metres below the basement floor level to accommodate footing construction. As such, the founding levels are expected to be between about 19.5 and 20.0 metres depth. The founding levels will therefore be within limestone bedrock.

Footings on or within competent bedrock can be sized using an Ultimate Limit States (ULS) factored bearing resistance of 5 Megapascals. Provided the bedrock surface is acceptably cleaned of loose bedrock, the settlement of footings at the corresponding service (unfactored) load levels will be less than 25 millimetres and therefore Serviceability Limit States (SLS) need not be considered in the foundation design.

The ultimate resistance of the footings to lateral loading may be calculated using an ULS friction value of 0.7 (unfactored) across the interface between the footing and the bedrock. If greater resistance is required, the footings could be provided with shear keys or prestressed rock anchors could be used to increase the normal stress level across the interface. Further guidance on this issue can be provided, if required.

5.5 Seismic Design

The seismic design provisions of the 2012 Ontario Building Code (2012 OBC) depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level.

Site specific shear wave velocity profiling, using the Vertical Seismic Profiling (VSP) method (down-hole geophysical method), was carried out in a borehole on an adjacent Tunney's Pasture site for Public Works and Government Services Canada in 2011. The results of that testing are provided in Appendix D.

A review of the borehole information indicates that both sites are underlain by similar overburden conditions (i.e., less than about 1 metre of fill material) and similar bedrock conditions (i.e., limestone of the Bobcaygeon Formation). The results of the nearby VSP testing would therefore also be applicable to this site, as permitted by the 2012 OBC. The results of the VSP testing indicate an average shear-wave velocity for the bedrock of 2,200 metres per second. As such, this site can be assigned a Site Class of A.

5.6 Basement Floor Slab

In preparation for the construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab.

Provision should be made for at least 300 millimetres of 16 millimetre clear crushed stone to form the base of the floor slab. To prevent hydrostatic pressure build up beneath the floor slab, it is suggested that the granular base for the floor slab be drained. This should be achieved by installing rigid 100 millimetre diameter perforated pipes in the floor slab bedding at 6 metre centres. The perforated pipes should discharge to a positive outlet such as a sump from which the water is pumped.

If or where an asphalt surface will be provided for the basement level, at least 150 millimetres of OPSS Granular A base should be provided above the clear stone, compacted to at least 100 percent of the material's standard Proctor maximum dry density.

If it is determined that water-tight construction is required for this structure, then the basement floor slab will have to be of concrete slab construction, and designed to resists hydro-static uplift pressures. Further discussion will be provided, if required.

5.7 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements in unheated areas should be provided with a minimum of 1.5 metres 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 metres of earth cover.

It is expected that these requirements will be satisfied for all of the structure footings due to the deep founding levels required to accommodate the below-grade parking.

5.8 Basement Walls

The backfill and drainage requirements for basement walls, as well as the lateral earth pressures will depend on the type of excavation that is made to construct the basement levels.

The following sections assume that water-tight construction will not be required. If it is determined that water-tight construction is needed, additional design guidelines will be required.

5.8.1 Open Cut Excavations

The soils at this site are frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration (1.5 metres) to avoid problems with frost adhesion and heaving. Free draining backfill materials are also required if hydrostatic water pressure against the basement walls (and potential leakage) is to be avoided. The foundation and basement walls therefore should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in 0.3 metre thick lifts, compacted to at least 95 percent of the material's standard Proctor maximum dry density.

The basement wall backfill should be drained by means of a perforated pipe subdrain in a surround of 19 millimetres clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

5.8.2 Excavations in Bedrock

Where basement walls will be poured against bedrock, vertical drainage such as Miradrain must be installed on the face of the bedrock to provide the necessary drainage. The top edge of the Miradrain should be sealed or covered with a geotextile to prevent the loss of soil into the void between the sheet and geotextile of the Miradrain.

Where the basement walls will be constructed using formwork, it will be necessary to backfill a narrow gallery between the shoring or bedrock face and the outside of the walls. The backfill should consist of 6 millimetre clear stone 'chip', placed by a stone slinger or chute.

In no case should the clear stone chip be placed in direct contact with other soils. For example, surface landscaping or backfill soils placed near the top of the clear stone back fill should be separated from the clear stone with a geotextile.

Both the drain pipe for the wall backfill and/or the Miradrain should be connected to a perimeter drain at the base of the excavation which is connected to a sump pump.

5.8.3 Lateral Earth Pressures

It is considered that three design conditions exist with regards to the lateral earth pressures that will be exerted on the basements walls:

1) Walls cast directly against the bedrock face.

2) Walls cast against formwork with a narrow backfilled gallery provided between the basement wall and the adjacent excavation bedrock face.

3) Walls cast against formwork with a wide backfilled gallery provided between the basement wall and the adjacent excavation face.

For the first case (wall cast against the bedrock), there will no effective lateral earth pressures on the basement wall.

For the second case, the magnitude of the lateral earth pressure depends on the magnitude of the arching which can develop in the backfill and therefore depends on the width of the backfill, its angle of internal friction, as well as the interface friction angles between the backfill and both the rock face and the basement wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_h(z) = \frac{\gamma B}{2\tan\delta}\left(1 - e^{-2K\frac{z}{B}\tan\delta}\right) + K\,q$$

Where:

$\sigma_h(z)$ = Lateral earth pressure on the basement wall at depth z, kilopascals

K = earth pressure coefficient, use 0.6

γ = unit weight of retained soil, use 20 kilonewtons per cubic metre for clear stone chip

B = width of backfill (between basement wall and bedrock face), metres

δ = average interface friction angle at backfill-basement wall and backfill-rock face interfaces, use 15 degrees

z = depth below top of formwork, metres

q = surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 15 kilopascals)

For the third case, the basement walls should be designed to resist lateral earth pressures calculated as:

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

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, use 22 kilonewtons per cubic metre

z = depth below top of wall, metres

q = uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (use 15 kilopascals)

For all cases, hydrostatic groundwater pressures would also need to be considered if the structure is designed to be water-tight.

Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against shoring or bedrock, damp proofing using a crystalline barrier such as Crystal Lok or Xypex could be used. The use of a concrete additive that provides reduced permeability could also be considered.

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}, K_A = The seismic and active earth pressure coefficient, use 0.8 and 0.33 respectively

H = The total depth to the bottom of the foundation wall (m)

Hydrodynamic groundwater pressures would also need to be considered if the structure is designed to be water-tight. However, more sophisticated analyses may need to be carried out at the detailed design stage.

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

It has been assumed that the underground parking levels will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidelines for the design of the basement walls and foundations will be required.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible backfill placed beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 millimetre 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.

5.9 Impacts on Adjacent Development

Possible impacts on adjacent developments could result from:

- Ground movement around the perimeter of the excavation.
- Ground settlements due to the planned temporary and permanent groundwater level lowering, if sensitive and compressible clay soils exist within the expected zone of influence of the groundwater level lowering (which, as discussed below, it not the case for this development).

A preconstruction survey of all structures located within close proximity to this site should be carried out prior to commencement of the excavation.

The structures that are mostly at risk of being impacted by ground movements around the excavation are the 11 storey building located north of the site and the 2 storey building located south of the site. For the 11 storey building, this structure has one basement level and is likely supported on spread footings on bedrock. The portion of this structure which is closest to the excavation is a one-storey parking garage, with expected fairly light foundation loads and, as such, no rock reinforcement requirements are anticipated for this side of the excavation. However, the exposed bedrock should be inspected by qualified geotechnical personnel at the time of excavation to confirm that assessment.

For the 2 storey building located south of the site, it is also expected that this structure is founded on the bedrock surface. Given the expected light foundation loads (likely less than 100 kilopascals) and the distance from the structure to the site limits, no rock reinforcement requirements are anticipated for this side of the excavation. However, the exposed bedrock should be inspected by qualified geotechnical personnel at the time of excavation to confirm that assessment.

As a general guideline for excavation planning, the excavation for the new structure should not come within 0.5 metres of the edge of the footings of the existing buildings. To avoid undermining of the rock and/or disturbance of the rock, careful line drilling of the excavation limits in this area must be undertaken.

The planned temporary and permanent groundwater level lowering would be an issue with regards to surrounding ground settlements if sensitive and compressible clay soils exist within the expected zone of influence of the groundwater level lowering (both during construction and in the long term due to the foundation drainage system). The predicted steady-state radius of groundwater drawdown is approximately 150 metres. The results of this investigation as well as published geologic mapping do not indicate compressible soils being present within this zone. Therefore, the planned groundwater level lowering will not be an issue with regards to ground settlements due to overstressing sensitive and compressible clay soils.

5.10 Environmental Considerations

If it is decided to design the basement of the building to be fully drained, that drainage system will result in groundwater flow from the surrounding properties towards this site. Therefore, groundwater contamination beneath adjacent properties, if present, could be drawn towards this site. The inflow of contaminated groundwater during construction could result in increased groundwater disposal costs. If the inflow of contaminated groundwater would be anticipated in the longer term, it should be planned to construct a water-tight foundation.

5.11 Corrosion and Cement Type

One sample of groundwater from borehole 12-101 was submitted to Exova Laboratories Ltd. for chemical analysis related to potential corrosion of buried ferrous elements and sulphate attack on buried concrete elements. The results of the testing are provided in Appendix E.

The results indicate a very high potential for corrosion of exposed ferrous metal

The results also indicate that Type MS cement should be used for substructures

6.0 ADDITIONAL CONSIDERATIONS

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that bedrock having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction viewpoint. Also, the proposed blasting design and monitoring proposed by the contractor should be reviewed.

Pumping from the excavation will result in groundwater flow from the surrounding properties towards this site. Therefore groundwater contamination beneath adjacent properties, if present, could be drawn towards this site. Additional chemical testing should be carried out at the time of construction to determine the groundwater quality so that disposal requirements can be confirmed. The inflow of contaminated groundwater during construction could result in increased groundwater disposal costs.

Ontario Regulation 903 would ultimately require abandonment of the monitoring wells installed for this investigation. However these devices may be useful during construction. The monitoring wells may be useful in evaluating the effectiveness of the dewatering program. It is therefore proposed that decommissioning of these devices be made part of the construction contract.

At the time of the writing of this report, only preliminary details for the proposed development were available. Golder Associates should be retained to review the detailed drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
M.S. SNOW
July 31, 2019
PROVINCE OF ONTARIO

Michael Snow

Michael Snow, P.Eng., ing., M.A.Sc.
Principal, Sr. Geotech Engineer, Infrastructure & P3/DB Lead

MSS/hdw

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https://golderassociates.sharepoint.com/sites/112765/project files/6 deliverables/geotech/19127365-001 99 parkdale 0108_19.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, **Brigil**. 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.

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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.

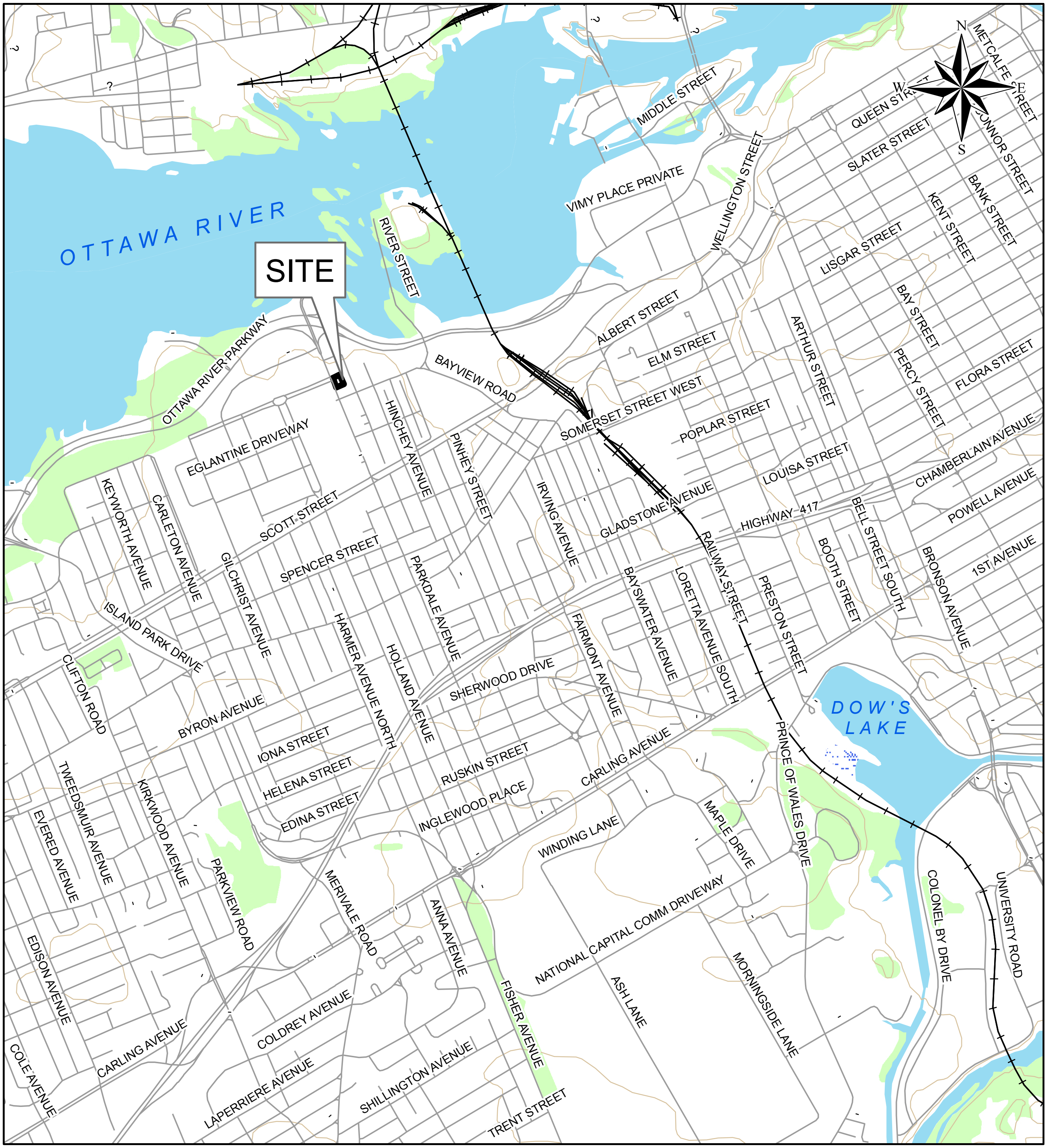
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater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

0.5 0 0.5

SCALE 1:25,000 KILOMETRES

NOTE

THIS FIGURE IS TO BE READ IN CONJUCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LIMITED REPORT NO. 11-1121-0275-2000

REFERENCE

BASE DATA - CANVEC PROVIDED BY HER MAJESTY THE QUEEN IN RIGHT OF CANADA, DEPARTMENT OF NATURAL RESOURCES
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: UTM ZONE 18

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
99 PARKDALE AVENUE, OTTAWA, ONTARIO

TITLE

KEY PLAN

Golder Associates
Ottawa, Ontario

PROJECT NO. 11-1121-0275			SCALE AS SHOWN	REV. 0.0
DESIGN	TMS	13 Aug. 2012	**FIGURE 1**	
GIS	BT/BR	17 Aug. 2012		
CHECK	TMS	17 Aug. 2012		
REVIEW	TJN	17 Aug. 2012		

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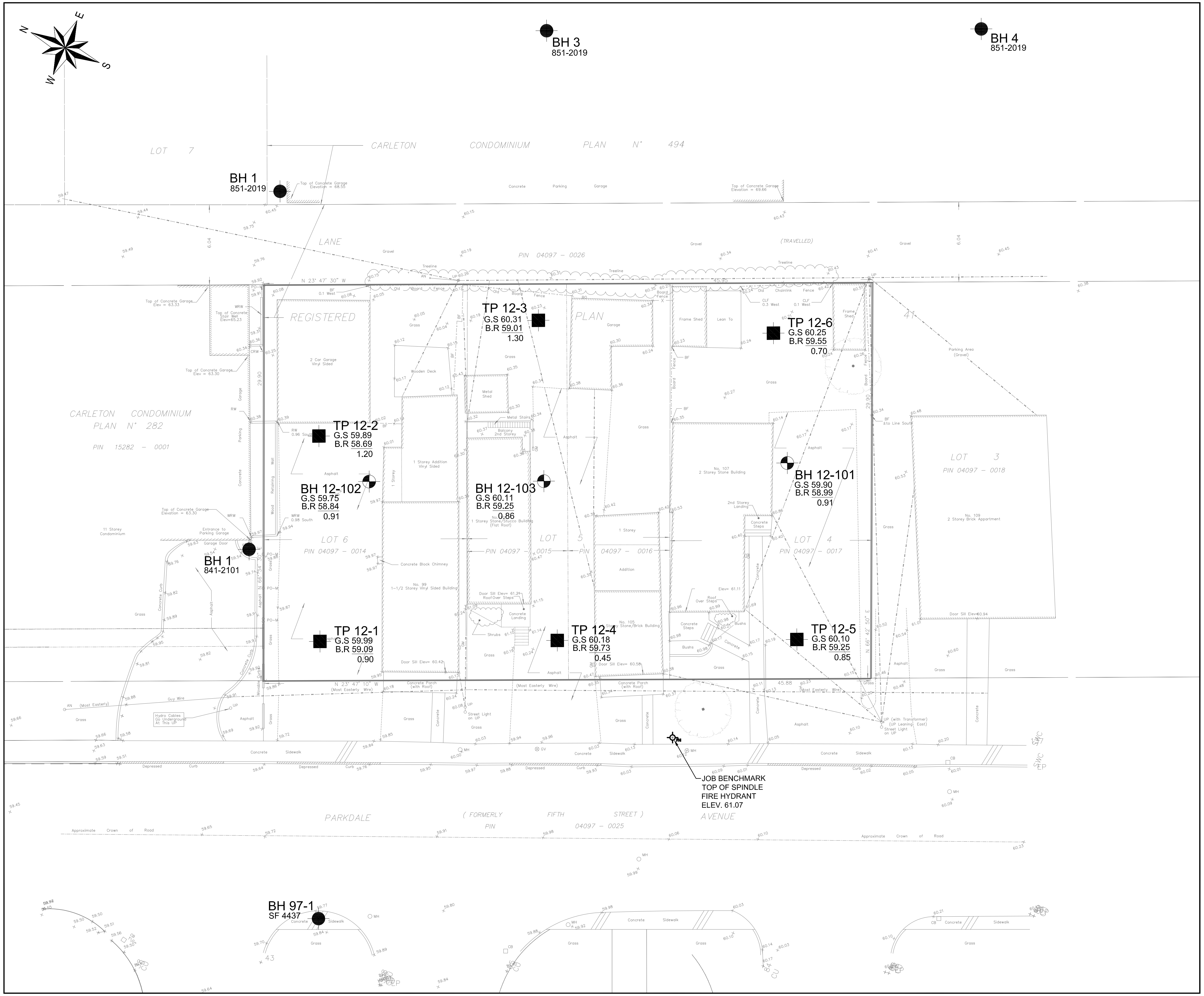


JOB BENCHMARK
TOP OF SPINDLE
FIRE HYDRANT
ELEV. 61.07

LEGEND

◈	APPROXIMATE BOREHOLE LOCATION IN PLAN, CURRENT INVESTIGATION BY GOLDER ASSOCIATES LTD
■	APPROXIMATE TEST PIT LOCATION IN PLAN, PRELIMINARY INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 111-1121-0275-1000
●	APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATIONS
—	APPROXIMATE SITE BOUNDARY
G.S 59.75	GROUND SURFACE ELEVATION, METRES
B.R 58.84	REFUSAL ON ROCK - ELEVATION, METRES
0.91	REFUSAL ON ROCK - DEPTH, METRES

6 0 6

SCALE 1:300 METRES

REFERENCE

BASE PLAN PROVIDED IN ELECTRONIC FORMAT BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 11-1121-0275-2000

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH RISE DEVELOPMENT
99 PARKDALE AVENUE, OTTAWA, ONTARIO

TITLE

SITE PLAN

Golder Associates
Ottawa, Ontario

PROJECT No.		11-1121-0275	FILE No.	1111210275-2000-02.dwg
DESIGN	T.M.S.	Jan. 2012	SCALE 1:300	REV.
CADD	P.L.G.	17 Aug. 2012	FIGURE 2	
CHECK	T.M.S.	17 Aug. 2012		
REVIEW	T.J.N.	17 Aug. 2012		

FILENAME: N:\Active\2011\1121 – Geotechnical\11-1121-0275 Urbandale 99 Parkdale\Spatial IM\ACAD\Phase 2000\1111210275-2000-02.dwg

APPENDIX A

Abbreviations and Symbols Lithological and Geotechnical Rock Description Terminology Record of Borehole Sheets Current Investigation

PROJECT: 11-1121-0275-2000

RECORD OF BOREHOLE: 12-101

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: July 16, 2012

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/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		59.90							
0	Power Auger 200 mm (HS)	Intermixed dark brown silty sand, brick, and gravel (FILL)		0.00							Bentonite Seal
		Compact to dense SILTY SAND, trace gravel (GLACIAL TILL)		59.14 0.76	1	SS	40				
1	Rotary Drill NQ Core	Slightly weathered to fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone		0.91	C1	NQ RC	DD				Pentlandite Seal
2											Silica Sand ▽
3		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone		57.41 2.49	C2	NQ RC	DD				
4											32 mm Diam. PVC #10 Slot Screen
5					C3	NQ RC	DD				
6					C4	NQ RC	DD				
7											
8					C5	NQ RC	DD				
9		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone and very small white calcite inclusions		51.16 8.74	C6	NQ RC	DD				
10											Bentonite Seal
11					C7	NQ RC	DD				
12		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone		48.09 11.81	C8	NQ RC	DD				
13											
14					C9	NQ RC	DD				
15					C10		DD				

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: H.C.
CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF BOREHOLE: 12-101

SHEET 2 OF 2

LOCATION: See Site Plan

BORING DATE: July 16, 2012

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	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
15 16	Rotary Drill NQ Core	Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone			C10	NQ RC	DD				
17		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone -Slickensided sheer zone between 17.40 and 17.47 m depth		43.49 16.41	C11	NQ RC	DD				
18 19					C12	NQ RC	DD				
20					C13	NQ RC	DD				Bentonite Seal
21 22					C14	NQ RC	DD			UCS = 199 MPa	
23					C15	NQ RC	DD				
24 25					C16	NQ RC	DD				
26 27 28 29 30		End of Borehole		34.55 25.35							W.L. in Screen at Elev. 57.76 m on July 27, 2012

MIS-BHS 001 1111210275.GPJ GAL-MIS.GDT 08/17/12 P.L.G.

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: H.C.
CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF DRILLHOLE: 12-101

SHEET 1 OF 2

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 16, 2012
DRILL RIG: CME 850
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

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

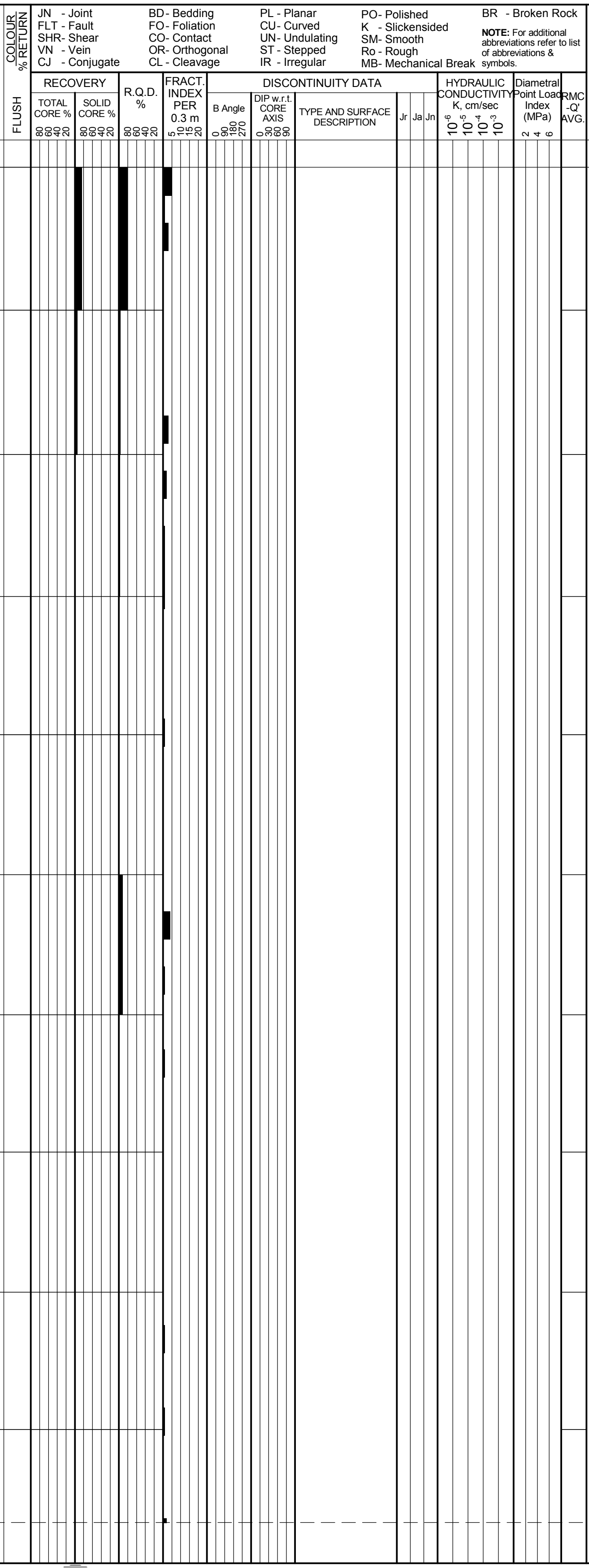
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

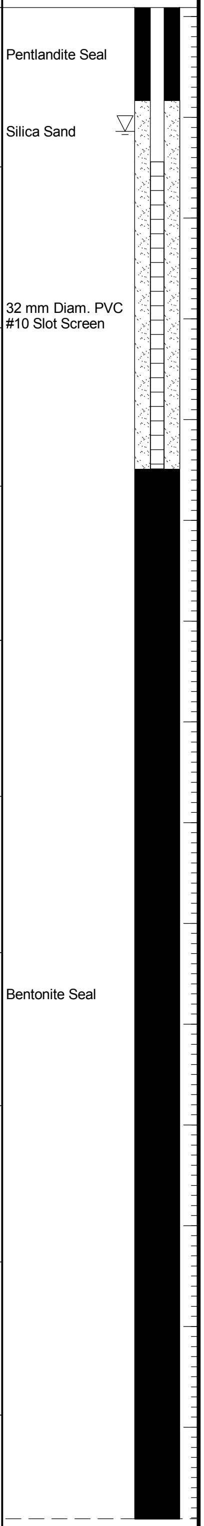
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		BEDROCK SURFACE	58.99	
1–2	Rotary Drill NQ Core	Slightly weathered to fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone	0.91	1
3–8		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone	57.41 2.49	2 3 4 5
9–11		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone and very small white calcite inclusions	51.16 8.74	6 7
12–15		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone	48.09 11.81	8 9 10
		CONTINUED NEXT PAGE		





DEPTH SCALE
1 : 75

Golder Associates

LOGGED: H.C.
CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF DRILLHOLE: 12-101

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: July 16, 2012

DATUM: Geodetic

DRILL RIG: CME 850

INCLINATION: -90°   AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

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 % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		--- CONTINUED FROM PREVIOUS PAGE ---																		
16	Rotary Drill NQ Core			43.49 16.41	10															
17		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone -Slickensided sheer zone between 17.40 and 17.47 m depth			11															
18					12															
19																				
20					13															Bentonite Seal
21																				
22					14															
23					15															
24					16															
25				34.55 25.35																
		End of Drillhole																		W.L. in Screen at Elev. 57.76 m on July 27, 2012
26																				
27																				
28																				
29																				
30																				

MIS-RCK 004 1111210275.GPJ GAL-MISS.GDT 08/17/12 P.L.G.

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: H.C.

CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF BOREHOLE: 12-102

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: July 17, 2012

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		59.75					
0	Power Auger 200 mm (HS)	Intermixed dark brown silty sand, gravel, brick, and crushed stone (FILL)	▨	0.00					
				58.84 0.91	1	SS	>50		
1	Rotary Drill NQ Core	Slightly weathered to fresh thinly bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone							Bentonite Seal
2					C1	NQ RC	DD		
				57.24 2.51					
3		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone			C2	NQ RC	DD		
4									
5					C3	NQ RC	DD		
6					C4	NQ RC	DD		▽
7									
8					C5	NQ RC	DD		Pentlandite Seal
9					C6	NQ RC	DD		
10									
11					C7	NQ RC	DD		
12					C8	NQ RC	DD		
13									
14					C9	NQ RC	DD		
15					C10	NQ RC	DD		

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 75

LOGGED: H.C.

CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF BOREHOLE: 12-102

SHEET 2 OF 2

LOCATION: See Site Plan

BORING DATE: July 17, 2012

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---						
15	Rotary Drill NQ Core	Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone		C10	NQ RC	DD		Pentlandite Seal
16 17				C11	NQ RC	DD		Silica Sand
18				C12	NQ RC	DD		32 mm Diam. PVC #10 Slot Screen
19 20				C13	NQ RC	DD		
21				C14	NQ RC	DD		
22 23				C15	NQ RC	DD		Pentlandite Seal
24 25			34.35	C16	NQ RC	DD	UCS = 96 MPa	
		End of Borehole	25.40					
26								W.L. in Screen at Elev. 53.51 m on July 27, 2012
27								
28								
29								
30								

MIS-BHS 001 1111210275.GPJ GAL-MIS.GDT 08/17/12 P.L.G.

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: H.C.

CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF DRILLHOLE: 12-102

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 17, 2012

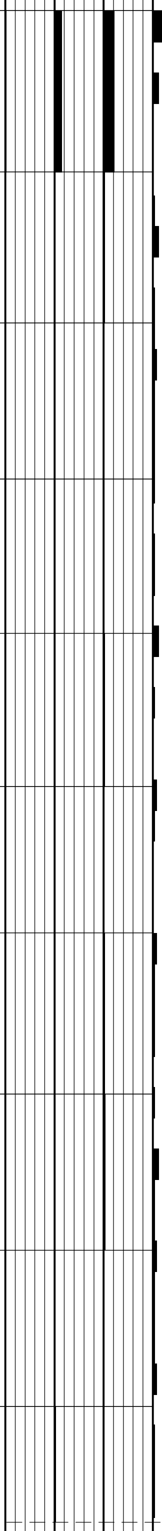
DRILL RIG: CME 850

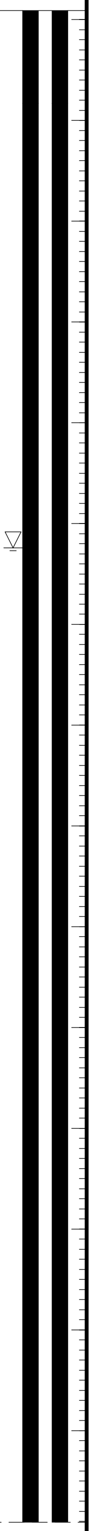
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	
		BEDROCK SURFACE		58.84																
1	Rotary Drill NQ Core	Slightly weathered to fresh thinly bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone		0.91	1															Bentonite Seal
2				57.24 2.51																
3		Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone			2															
4					3															
5																				
6					4															∇
7					5															
8																				
9					6															Pentlandite Seal
10																				
11					7															
12					8															
13																				
14					9															
15					10															





CONTINUED NEXT PAGE

DEPTH SCALE

1 : 75

LOGGED: H.C.

CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF DRILLHOLE: 12-102

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: July 17, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

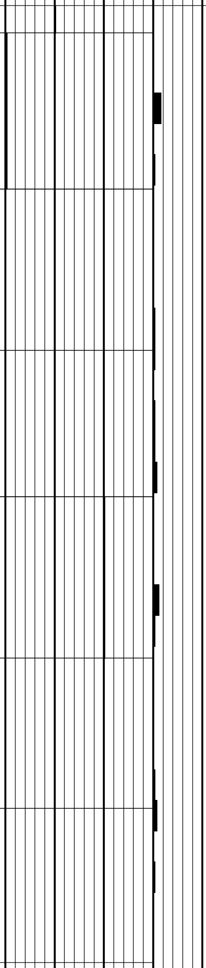
DRILLING CONTRACTOR: Marathon Drilling

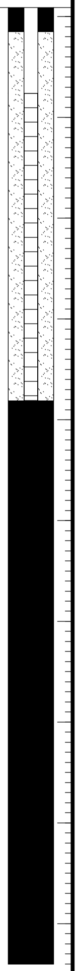
JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

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

Column headings: DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.3 m (5 10 15 20) | DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn | 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 METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	
		--- CONTINUED FROM PREVIOUS PAGE ---			
16	Rotary Drill NQ Core	Fresh thinly to medium bedded grey fine grained LIMESTONE BEDROCK, with very thin beds of dark grey dolomitic limestone		10	Pentlandite Seal
				11	Silica Sand
17					
18					32 mm Diam. PVC #10 Slot Screen
				12	
19					
20				13	
21					
				14	
22					
					Pentlandite Seal
23				15	
24					
				16	
25			34.35		
		End of Drillhole	25.40		
26					W.L. in Screen at Elev. 53.51 m on July 27, 2012
27					
28					
29					
30					





DEPTH SCALE

1 : 75

LOGGED: H.C.

CHECKED: T.M.S.

PROJECT: 11-1121-0275-2000

RECORD OF BOREHOLE: 12-103

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: July 17, 2012

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/0.3m				
		GROUND SURFACE		60.11							
0	Power Auger (HS)	Intermixed brick, sand, and gravel (FILL)	▨	0.00							
1		End of Probehole Auger Refusal		59.25 0.86							
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: H.C.

CHECKED: T.M.S.

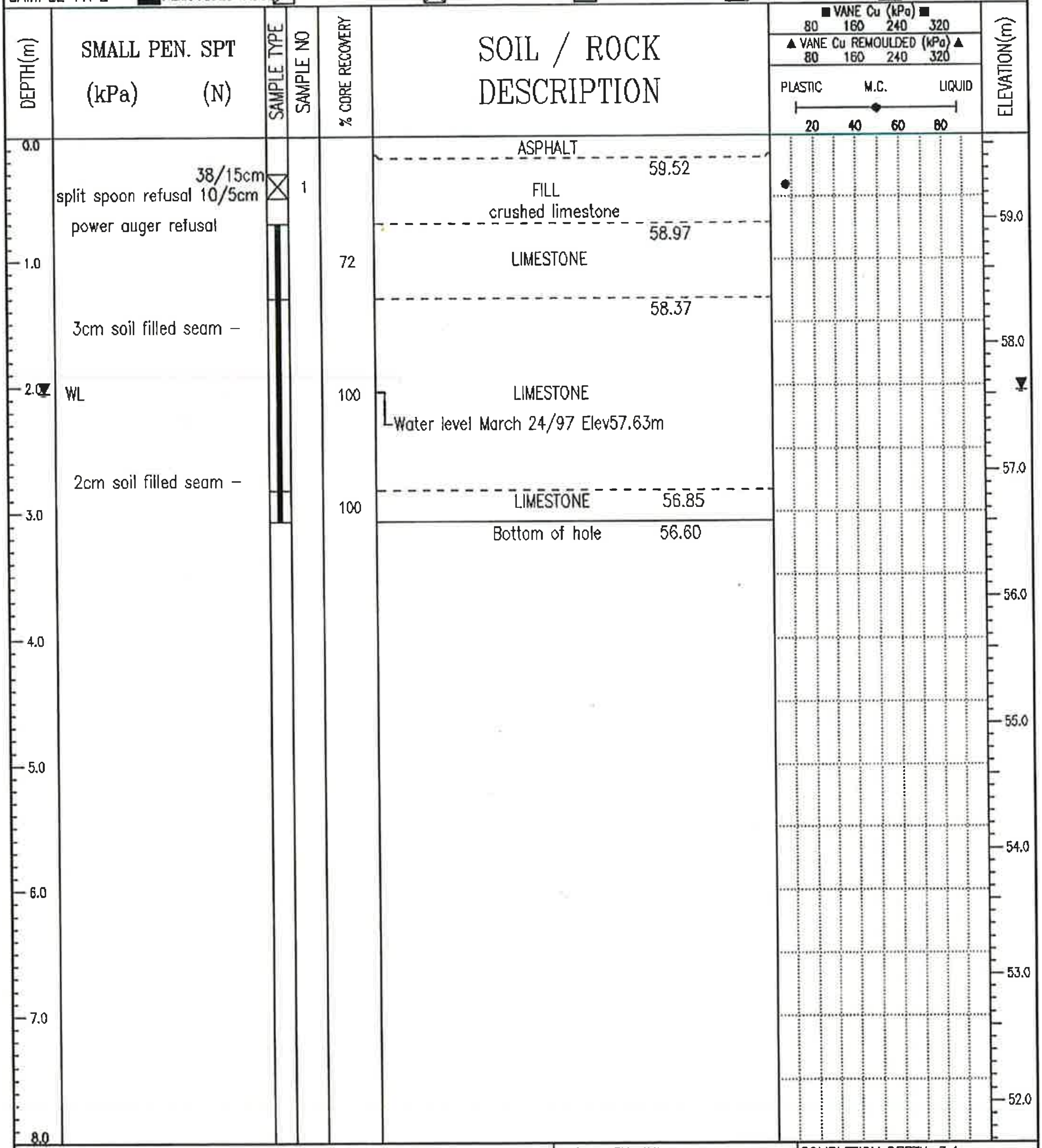
APPENDIX B

Borehole and Test Pit Logs
Previous Investigations

SF 4437

TUNNEY'S PASTURE WATERMAIN UPGRADE	B.M.(ELEV 62.79m)geodetic: Tablet No.	BOREHOLE NO: 97-1
CLIENT: P.W.G.S.C.	56U2791 on west foundation wall of the	PROJECT NO: E - 7538
START DATE: 97/03/18	Archives bldg. supplied by P.W.G.S.C.	ELEVATION: 59.67 (m)

SAMPLE TYPE ■ REMOULDED-AUGER ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ◫ NO RECOVERY ◨ NQ CORE



38/15cm
split spoon refusal 10/5cm
power auger refusal

1

3cm soil filled seam –

WL

2cm soil filled seam –

72

100

100

ASPHALT

59.52

FILL
crushed limestone

58.97

LIMESTONE

58.37

LIMESTONE
Water level March 24/97 Elev57.63m

LIMESTONE 56.85

Bottom of hole 56.60

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 3.1 m
	REVIEWED BY: E.S.	COMPLETE: 97/03/18
	Fig. No: 2.	Page 1 of 1

97/04/01 10:31AM

841 - 2101

RECORD OF BOREHOLES 1 & 2

LOCATION See Figure 2

BORING DATE MAR. 20, 1984

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	ELEV'N. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3 m	CORE RECOVERY (%)	R.Q.D. (%)
		BH. 1						
	59.58	GROUND SURFACE						
BX CASING	0.00	GREY TO BLACK, SAND GRAVEL, SOME SILT AND CINDERS (FILL)	☒	1	50 mm D.O.	6		
50 mm DIAM (UNCASED)	58.91 0.67	HIGHLY WEATHERED BEDROCK		2	RC	—	71	0
	0.85	FAIRLY SOUND TO SOUND GREY THINLY TO MEDIUM BEDDED LIMESTONE BEDROCK (OTTAWA FORMATION)		3		—	100	27
				4		—	58	75
	55.87 3.71	END OF HOLE						
		BH. 2						
	58.86	GROUND SURFACE						
BX CASING	0.00	BROWN TO BLACK SILTY SAND, GRAVEL AND COBBLES (FILL)	☒	1	50 DO	20 (FROZEN)		
				2	RC	—	50	0
				3		—	100	50
				4		—	100	0
50 mm DIAM (UNCASED)	57.96 0.90	SOUND GREY THINLY TO MEDIUM BEDDED LIMESTONE BEDROCK (OTTAWA FORMATION)		5		—	100	58
				6		—	100	88
	54.86 4.00	END OF HOLE						

ELEVATION SCALE

BH. 1: 60, 59, 58, 57, 56, 55

BH. 2: 59, 58, 57, 56, 55, 54

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

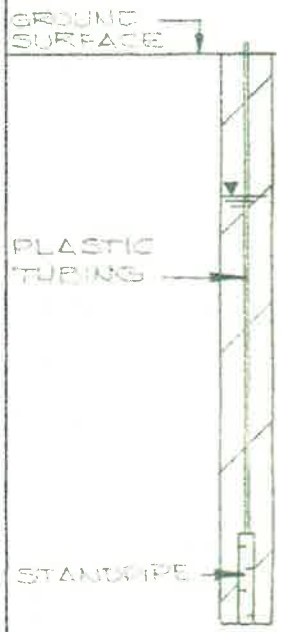
SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○

HYDRAULIC CONDUCTIVITY, k, cm/sec. Ix10 Ix10 Ix10 Ix10

WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. IN STANDPIPE AT ELEV. 58.67 MARCH 23, 1984

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN JC

CHECKED [illegible]

851 - 2019

RECORD OF BOREHOLE 1

LOCATION See Figure 2

BORING DATE JAN 22,23 1985

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD: ROTARY DRILLING; AX CASING; UNCASED 50 MM DIAMETER

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10

SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○

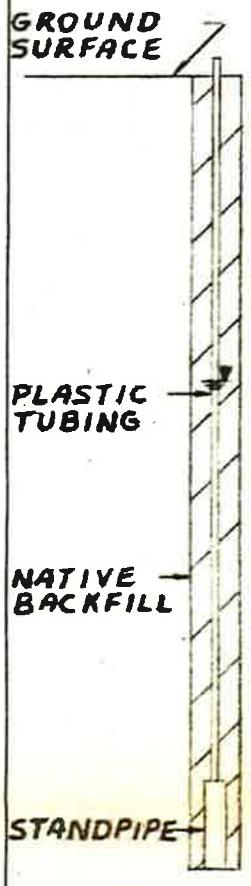
WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3m	CORE RECOVERY (%)	R.Q.D (%)
60.27	GROUND SURFACE					
0.00	COMPACT BLACK SANDY SILT WITH SOME CLAY, TOPSOIL, CINDERS PIECES BRICK (FILL)	1	50 mm DO	12		
59.19 1.08		2	"	29		
58.85 1.42	SLIGHTLY WEATHERED THINLY BEDDED GREY LIMESTONE WITH OCCASSIONAL SOIL FILLED SEAMS (BEDROCK)	3	50 mm		100	0
		4	RC		86	
		5	"		62	42
		6	"		97	97
	FAIRLY SOUND THINLY TO MEDIUM BEDDED GREY LIMESTONE (BEDROCK) WITH OCCASIONAL DARK GREY SHALE BANDS AND SEAMS (OTTAWA FORMATION)	7	"		98	78
		8	"		93	67
		9	"		99	98
54.81 5.46	END OF HOLE					

ELEVATION SCALE: 61, 60, 59, 58, 57, 56, 55, 54

PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN STANDPIPE AT ELEVATION 58.2 METERS JAN 29 1985

15 —⊕— 5 Percent axial strain at failure
10

VERTICAL SCALE 1:50

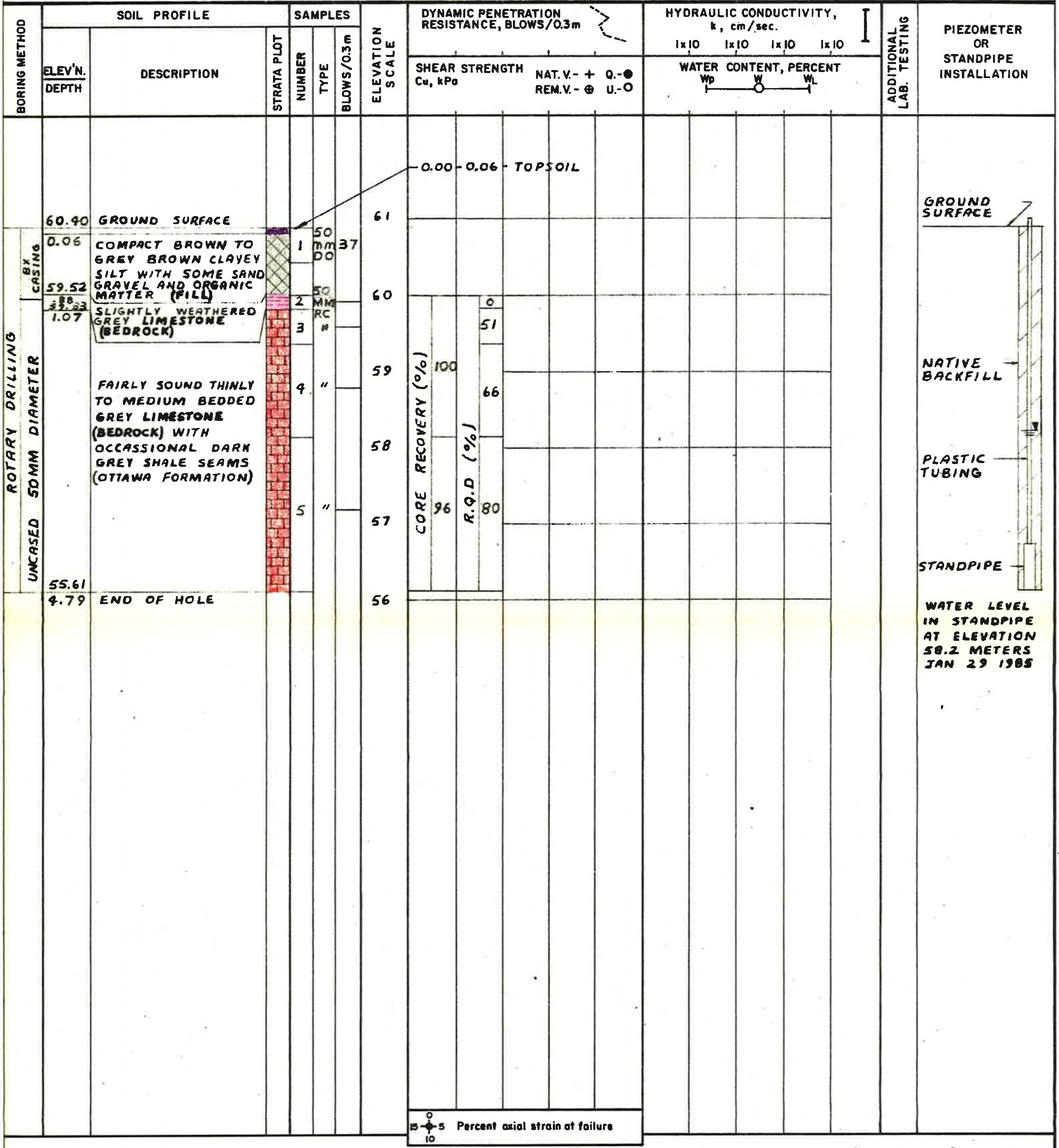
Golder Associates

DRAWN S.L.
CHECKED C.R.M.

RECORD OF BOREHOLE 3

851- 2019

LOCATION See Figure 2
BORING DATE JAN 24 1984
DATUM GEODETIC
SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm
PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm



ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3m
60.40	GROUND SURFACE			
0.06	COMPACT BROWN TO GREY BROWN CLAYEY SILT WITH SOME SAND GRAVEL AND ORGANIC MATTER (FILL)	1	50 mm DO	37
59.52 .88	SLIGHTLY WEATHERED GREY LIMESTONE (BEDROCK)	2	50 MM RC	
59.33 1.07	FAIRLY SOUND THINLY TO MEDIUM BEDDED GREY LIMESTONE (BEDROCK) WITH OCCASSIONAL DARK GREY SHALE SEAMS (OTTAWA FORMATION)	3	"	
		4	"	
		5	"	
55.61 4.79	END OF HOLE			

WATER LEVEL IN STANDPIPE AT ELEVATION 58.2 METERS JAN 29 1985

VERTICAL SCALE 1:50

Golder Associates

DRAWN S.L.
CHECKED CRM

851 - 2019

RECORD OF BOREHOLE 4

LOCATION See Figure 2

BORING DATE JAN 24 1985

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m SHEAR STRENGTH Cu, kPa NAT. V.- + Q.-● REM.V.-⊕ U.-O		HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10 WATER CONTENT, PERCENT Wp W WL	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3 m		CORE RECOVERY(%)	R.Q.D (%)			
	60.88	GROUND SURFACE					61	0.00-0.06 GREY CRUSHED STONE (FILL)				
ROTARY DRILLING BX CASING	0.06 60.00	GREY BROWN CLAYEY SILT WITH SOME SAND AND GRAVEL (FILL)	▨									
UNCASED 50 MM DIA	0.88 59.66	SLIGHTLY WEATHERED GREY LIMESTONE (BEDROCK)	🟪	1 2	50 mm RC		60	86	0			
	1.22	FAIRLY SOUND THINLY TO MEDIUM BEDDED GREY LIMESTONE (BEDROCK) OCCASSIONAL DARK GREY SHALE SEAMS (OTTAWA FORMATION)	🟥	3	"		59	78	36			
			🟥	4	"		58	97	45			
	57.74 3.14	END OF HOLE					57					

15 ✛ 5 (0, 10) Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN S.L

CHECKED CRM

TABLE 1

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (m)	Description
12-1 (59.99 m)	0.00 – 0.05	ASPHALTIC CONCRETE
	0.05 – 0.20	Grey crushed stone (FILL)
	0.20 – 0.25	ASPHALTIC CONCRETE
	0.25 – 0.40	Brown sand and gravel (FILL)
	0.40 – 0.90	Brown SILTY SAND, some gravel, trace clay (GLACIAL TILL)
	0.90	Refusal on BEDROCK

Note: Test pit dry

Sample	Depth (m)
1	0.25 - 0.40
2	0.50 - 0.90

12-2 (59.89 m)	0.00 – 0.05	ASPHALTIC CONCRETE
	0.05 – 0.15	Brown sand and gravel (FILL)
	0.15 – 0.20	ASPHALTIC CONCRETE
	0.20 – 0.30	Brown sand and gravel (FILL)
	0.30 – 0.35	ASPHALTIC CONCRETE
	0.35 – 1.20	Brown SILTY SAND, some gravel, trace clay and roots (GLACIAL TILL)
	1.20	Refusal on BEDROCK

Note: Test pit dry

Sample	Depth (m)
1	0.50 - 1.00

12-3 (60.31 m)	0.00 – 0.30	Brown sand and gravel (FILL)
	0.30 – 0.50	TOPSOIL
	0.50 – 1.30	Brown SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	1.30 – 1.70	Weathered BEDROCK
	1.70	Refusal on BEDROCK

Notes: Test pit dry

Sample	Depth (m)
1	0.10 - 0.30
2	0.60 - 1.20

TABLE 1

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (m)	Description
12-4	0.00 – 0.05	ASPHALTIC CONCRETE
(60.18 m)	0.05 – 0.30	Grey crushed stone (FILL)
	0.30 – 0.45	Brown SILTY SAND, some gravel, trace clay (GLACIAL TILL)
	0.45	Refusal on BEDROCK

Notes: Test pit dry

Sample	Depth (m)
1	0.10 - 0.30
2	0.30 - 0.45

12-5	0.00 – 0.40	TOPSOIL
(60.10 m)	0.40 – 0.60	Brown SILTY SAND, some gravel, trace to some clay (GLACIAL TILL)
	0.60 – 0.85	Grey SILTY SAND, some gravel, trace to some clay (GLACIAL TILL)
	0.85	Refusal on BEDROCK

Note: Test pit dry

Sample	Depth (m)
1	0.40 - 0.60
2	0.60 - 0.85

12-6	0.00 – 0.05	ASPHALTIC CONCRETE
(60.25 m)	0.05 – 0.10	Brown sand, some gravel (FILL)
	0.10 – 0.70	Brown SILTY SAND, some gravel, trace to some clay (GLACIAL TILL)
	0.70	Refusal on BEDROCK

Note: Test pit dry

Sample	Depth (m)
1	0.05 - 0.10
2	0.30 - 0.50

APPENDIX C

Unconfined Compressive Strength of Rock Core Testing Results
Bedrock Core Photos

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario
K2K 2A9

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: 99 Parkdale

Project No.: 11-1121-0275

Client: Urbandale Corporation

Date: August 17, 2012

Location(s):

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)
12-101	21.84-21.97		NQ	47.2	2718	198.5
12-102	24.30-24.46		NQ	47.2	2694	96.2

REMARKS : - Compressive Strength Corrected for L/D Ratio.
- Cores tested in vertical direction.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED: ____________________
C.N.Mangione P.Eng.

Borehole 12-101: 0.9 metres to 3.2 metres

Borehole 12-101: 3.2 metres to 5.5 metres

Golder Associates	DATE August 15, 2012	TITLE **Core Photos** **Urbandale – 99 Parkdale**
	DESIGN BG	
	CHECK TMS	**Ottawa, Ontario**
PROJECT No. 11-1121-0275 Rev. 0	REVIEW TMS	**PLATE 1**

Borehole 12-101: 5.5 metres to 7.8 metres

Borehole 12-101: 7.8 metres to 10.2 metres

Golder Associates

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

PROJECT No. 11-1121-0275 Rev. 0

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 2

Borehole 12-101: 10.2 metres to 12.5 metres

Borehole 12-101: 12.5 metres to 14.9 metres

	DATE August 15, 2012	TITLE
	DESIGN BG	**Core Photos** **Urbandale – 99 Parkdale**
	CHECK TMS	**Ottawa, Ontario**
PROJECT No. 11-1121-0275 Rev. 0	REVIEW TMS	**PLATE 3**

Borehole 12-101: 14.9 metres to 17.0 metres

Borehole 12-101: 17.0 metres to 19.3 metres

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

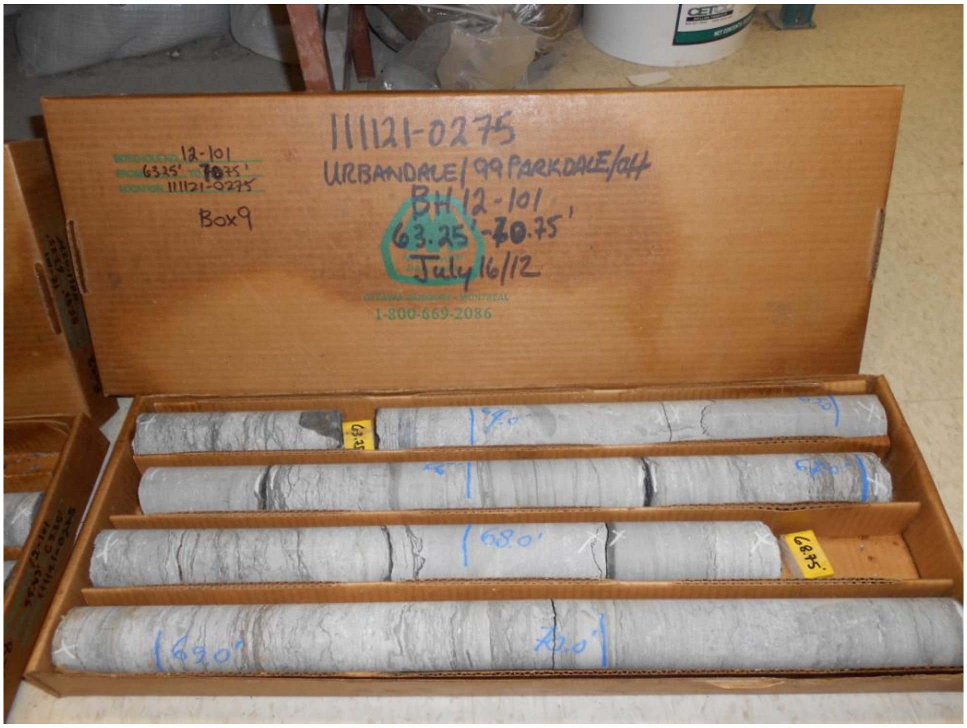
PROJECT No. 11-1121-0275 Rev. 0

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 4



Borehole 12-101: 19.3 metres to 21.6 metres

Borehole 12-101: 21.6 metres to 23.8 metres

Golder Associates		TITLE **Core Photos** **Urbandale – 99 Parkdale**
	DATE August 15, 2012	
	DESIGN BG	
	CHECK TMS	**Ottawa, Ontario**
PROJECT No. 11-1121-0275 Rev. 0	REVIEW TMS	**PLATE 5**

Borehole 12-101: 23.8 metres to 25.3 metres

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

PROJECT No. 11-1121-0275 Rev. 0

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 6

Borehole 12-102: 0.9 metres to 3.2 metres

Borehole 12-102: 3.2 metres to 5.6 metres

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

PROJECT No. 11-1121-0275 Rev. 0

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 7

Borehole 12-102: 5.6 metres to 7.9 metres

Borehole 12-102: 7.9 metres to 10.2 metres

Golder Associates

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

PROJECT No. 11-1121-0275 Rev. 0

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 8

Borehole 12-102: 10.2 metres to 12.6 metres

Borehole 12-102: 12.6 metres to 15.0 metres

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 9

PROJECT No. 11-1121-0275 Rev. 0

Borehole 12-102: 15.0 metres to 17.4 metres

Borehole 12-102: 17.4 metres to 19.8 metres

DATE	August 15, 2012
DESIGN	BG
CHECK	TMS
REVIEW	TMS

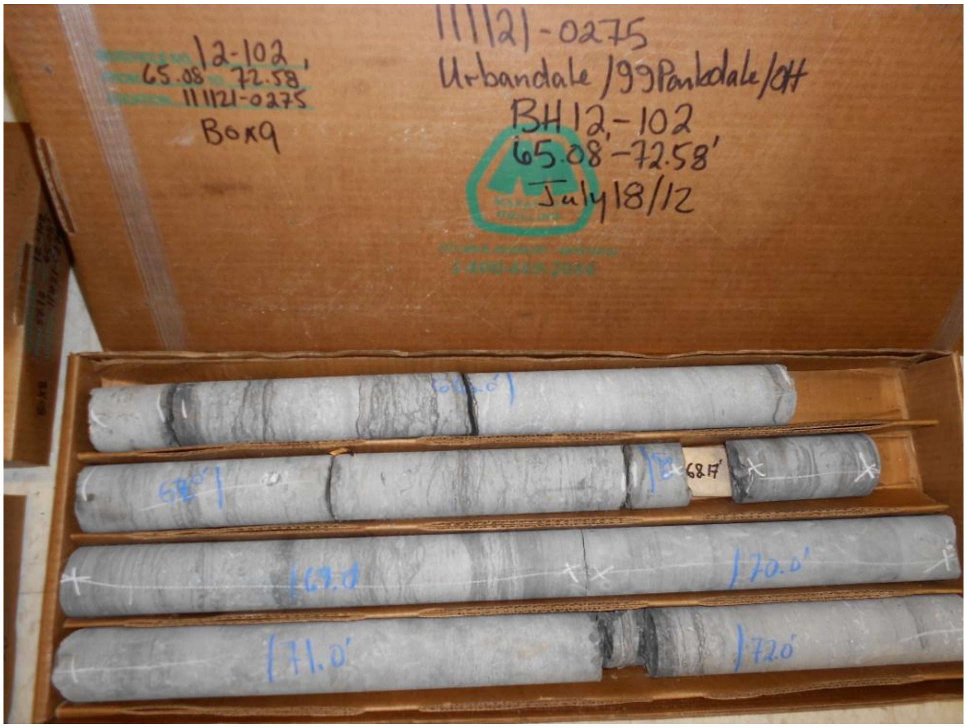
PROJECT No. 11-1121-0275 Rev. 0

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 10



Borehole 12-102: 19.8 metres to 22.1 metres

Borehole 12-102: 22.1 metres to 24.4 metres

Golder Associates

PROJECT No. 11-1121-0275 Rev. 0

DATE August 15, 2012
DESIGN BG
CHECK TMS
REVIEW TMS

TITLE

Core Photos
Urbandale – 99 Parkdale

Ottawa, Ontario

PLATE 11

Borehole 12-102: 24.4 metres to 25.4 metres

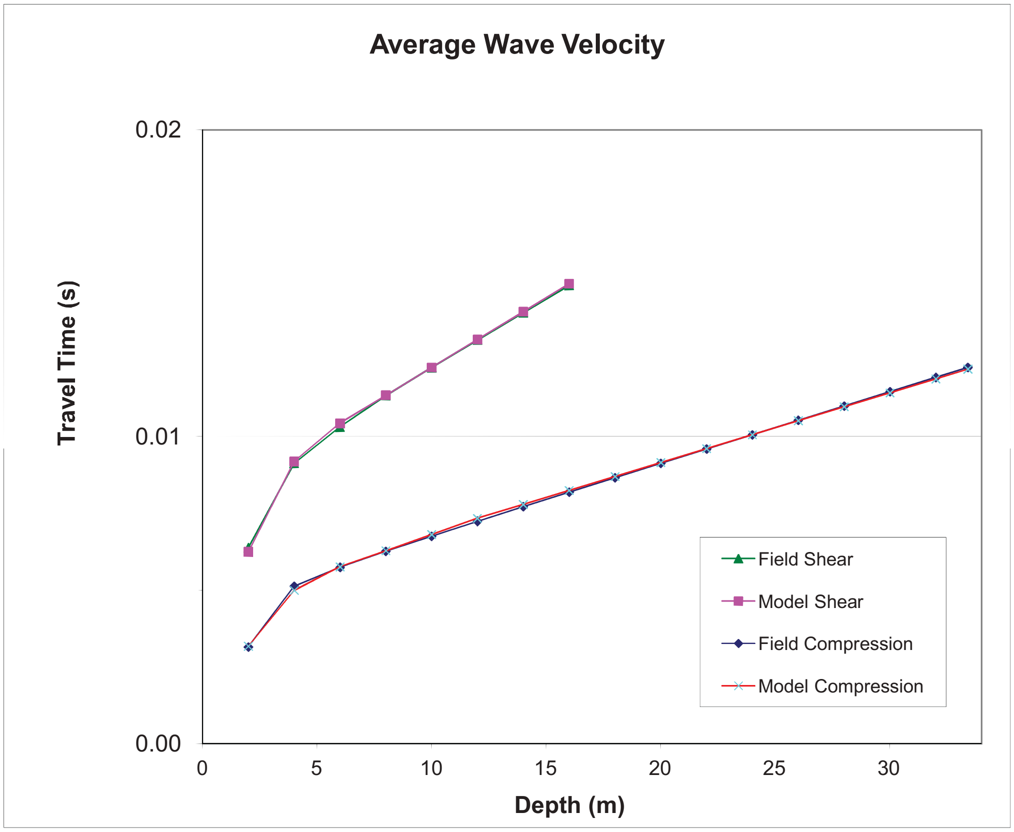
Golder Associates	DATE August 15, 2012	TITLE	
	DESIGN BG	**Core Photos** **Urbandale – 99 Parkdale**	
	CHECK TMS	**Ottawa, Ontario**	**PLATE 12**
PROJECT No. 11-1121-0275 Rev. 0	REVIEW TMS		

APPENDIX D

Geophysical VSP Test Results

TABLE 1
VSP SURVEY RESULTS - BOREHOLE BH11-1
251 SIR FREDERICK BANTING DRIVEWAY
OTTAWA, ONTARIO

Layer Depth (m)		Seismic Velocity		Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compression Wave (m/s)	Shear Wave (m/s)		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0	2	630	320	1750	0.33	179	475	456
2	4	1100	680	2600	0.19	1202	2863	1543
4	6	2600	1600	2600	0.20	6656	15911	8701
6	8	3800	2200	2600	0.25	12584	31408	20765
8	10	3800	2200	2600	0.25	12584	31408	20765
10	12	3800	2200	2600	0.25	12584	31408	20765
12	14	4400	2200	2600	0.33	12584	33557	33557
14	16	4400	2200	2600	0.33	12584	33557	33557
16	18	4400	2200	2600	0.33	12584	33557	33557
18	20	4400	2200	2600	0.33	12584	33557	33557
20	22	4400	2200	2600	0.33	12584	33557	33557
22	24	4400	2200	2600	0.33	12584	33557	33557
24	26	4400	2200	2600	0.33	12584	33557	33557
26	28	4400	2200	2600	0.33	12584	33557	33557
28	30	4400	2200	2600	0.33	12584	33557	33557
30	32	4400	2200	2600	0.33	12584	33557	33557
32	33.4	4400	2200	2600	0.33	12584	33557	33557



Notes
1. Depth presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

APPENDIX E

Results of Basic Chemical Analysis
Exova Laboratories Report Number 1217166

Golder Associates Ltd. (Ottawa)

Troy Skinner

Report Number 1217166
Date Reported: 2012-08-16

Date Submitted 2012-08-10

Project: 11-1121-0275

STATION	DATE	PARAMETER	VALUE	UNIT	MDL
12-101	2012-08-10	pH	7.09		1.00
12-101	2012-08-10	Conductivity	2800	uS/cm	5
12-101	2012-08-10	SO4	196	mg/L	3
12-101	2012-08-10	Cl	545	mg/L	1

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