KATASA GROUPE + DÉVELOPPVEMENT

GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL MIXED-USE BUILIDNG, 770-774 BRONSON AVENUE, OTTAWA, ONTARIO

JUNE 02, 2021

CONFIDENTIAL

GEOTECHNICAL INVESTIGATION

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KATASA GROUPE + DÉVELOPPVEMENT

GEOTECHNICAL REPORT
CONFIDENTIAL

PROJECT NO.: 211-05706-00
DATE: JUNE 02, 2021

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June 02, 2021

CONFIDENTIAL

KATASA GROUPE + DÉVELOPPVEMENT
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Gatineau, Québec, J8Z 1W2

Attention: Mrs. Tanya Chowieri, Partner

Dear Madam:

Subject: Geotechnical Investigation - Proposed Residential Mixed-Use Building, 770-774 Bronson Avenue, Ottawa, Ontario

We are pleased to submit this geotechnical report for the above-noted project. This report presents the summary of a desktop review of the results from the previous investigations and provides our geotechnical recommendations relevant to the design and construction of the proposed residential mixed-use building.

We trust that the report is straightforward and meets your current requirements. Please do not hesitate to contact the undersigned if you have any questions.

Yours sincerely,

Date: 2021.06.02
21:01:37-04'00'

Robert Edde, M.A.Sc., P.Eng.
Senior Technical Director

WSP ref.: 211-05706-00

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REVISION HISTORY

FINAL REPORT

June 2, 2021	Report Issue	
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Date

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1 INTRODUCTION

1.1 CONTENT

WSP Canada Inc. (WSP) was retained by the Katasa Groupe + Développement (KGD) to carry out a geotechnical desktop review of the previous geotechnical investigations completed at 770-774 Bronson Avenue in Ottawa, Ontario (hereinafter referred as the site) and provide a geotechnical report combining all pertinent technical information.

This geotechnical report summarizes the factual results presented in the previous geotechnical investigations and associated laboratory testing, presents an interpretation of the available factual information, and provides geotechnical recommendations related to geotechnical design aspect of the project and construction considerations for the proposed development.

Our understanding of this project is based upon information provided to WSP and the detailed scope of work outlined in our proposal (Reference No. 2156791), dated April 16, 2021.

1.2 OBJECTIVE AND LIMITATIONS

This report was prepared by WSP for KGD in accordance with the agreed upon scope of work as detailed in WSP's proposal, dated April 16, 2021. This report was prepared at the request of, and for the sole use of KGD, according to the specific terms of the mandate given to WSP. The use of this report by any third party, as well as any decision based upon this report, is under that party's sole discretion and responsibility. WSP may not be held accountable for any possible damages resulting from third party's decisions based on this report or its associated information.

Furthermore, any opinions regarding conformity with laws and regulations expressed in this report are technical in nature; the report is not and shall not, in any case, be considered as a legal opinion.

Reference should be made to the Limitations of this Report, attached in **Appendix A**, which follows the text, but forms an integral part of this document.

2 SITE AND PROJECT DESCRIPTION

2.1 SITE DESCRIPTION

The site is located on 770-774 Bronson Avenue, at the southwest corner of the intersection of Carling Avenue and Bronson Avenue in Ottawa, Ontario, as shown in **Drawing 1**. The site boundary layout is approximately L-shaped, measuring approximately 100 m long and 75 m wide in plan area.

The site is currently occupied by an automotive garage and at grade parking on the northeastern portion of the site. The southern portion of the site previously consists of several low-rise buildings, which have since been demolished.

The site is bordered by two arterial roads, Carling Avenue to the north and Bronson Avenue to the east, and bounded by various low-rise structures to the south, and by a two-storey commercial building with surface parking as well as Cambridge Street South to the west.

2.2 PROJECT DESCRIPTION

It is understood that the KGD plans to construct a mixed-use residential building in two phases. The following is known about the proposed development:

- Phase 1 of the development will be located on the eastern half of the site and will consist of a student rental building that varies from 9 to 26 storeys in height.
- Phase 2 of the development will be located on the western half of the site and will consist of a 9-sotrey residential building
- The structure will include two levels of underground parking levels across its entire building footprint.
- The ground floor for the proposed building will be at Elevation 75.38 m and the finished floor for the lower parking garage will be at approximately Elevation 68.9 m.
- The development will also consist of outdoor surface parking and landscaping areas.

Two geotechnical investigations and a Phase II Environmental Site Assessment (ESA) were carried out in the past for different parts of the site. Since then, there have been a number of design changes that yielded the proposed development, including the building height and footprint as well as number of basement levels. Therefore, an updated geotechnical report is produced in support of the revised design requirements and for a construction permit for the new proposed building.

A hydrogeological study will also be required to support the design of foundations and drainage system, as well as to evaluate the requirements and impacts of construction dewatering and the long-term groundwater management.

2.3 PUBLISHED GEOLOGY MAPPING

Based on a review of the published geological mapping, the subsurface conditions on this site should consist of thin deposits of glacial till underlain by shallow bedrock. The bedrock at the site is expected to consist of limestone of the Shallow Lake Formation.

3 METHODOLOGY

3.1 DESKTOP REVIEW

WSP carried out a desktop review of the previous geotechnical and environmental investigations completed within the project area. The results of those previous investigations are summarized in the following reports:

- "Phase I and Limited Phase II Environmental Site Assessment, Existing Office/Commercial/Residential Buildings 551, 553, 555, 557 Cambridge Street South, 774, 780, 782, 784 Bronson Avenue, Ottawa Ontario", dated May 18, 1999 (Ref. E1738-1) by John D. Patterson and Associates. (Patterson, 1999)
- "Geotechnical Investigation, Proposed Residential Development, 770 Bronson Avenue, Ottawa, Ontario" dated August 2015 (Project No. 1525987-02) by Golder Associates Ltd. (Golder, 2015)
- "Updated Geotechnical Study, Projected New Building at 774 Bronson Ave., Ottawa, ON" Draft Report dated November 2015 (Project No. 151-12490-00) by WSP Canada Inc. (WSP, 2015)
- "Updated Geotechnical Study, Projected New Building at 774 Bronson Ave., Ottawa, ON" Final Report dated February 2016 (Project No. 151-12490-00) by WSP Canada Inc. (WSP GEO, 2016)
- "Phase Two Environmental Site Assessment, 774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario", dated March 29, 2016 (Project No. 151-13503-00) by WSP Canada Inc. (WSP ENVIRO, 2016)

Based on a review of the above-noted previous investigations, the proposed excavation for the underground parking levels will extend to a water bearing zone in the upper bedrock. Analysis of the reported bedrock quality shows the upper bedrock zone to be more fractured beneath the 774 Bronson Avenue property and more intact beneath the 770 Bronson Avenue property. Based on the available information from the previous reports, it is WSP's opinion that a desktop study is sufficient, at this stage, to submit an updated geotechnical report in view of the new proposed construction. The details of the previous geotechnical investigations completed at this site are summarized in the sections below.

It should be noted that the 2015 Golder report provided preliminary estimates of expected short and long-term water infiltration into the future foundation excavation (which was based on up to 11 m below ground surface) using assumed hydraulic parameters; however, no in-situ hydraulic conductivity was conducted nor was there any water quality analysis performed on the raw water contained in the water bearing zone for comparative analysis to the City of Ottawa Sewer Use Bylaw (Bylaw No. 2003-514). A hydrogeological study will therefore be required to address the gaps in the groundwater quantity and quality data. Such work will be necessary to assist the design of foundations and drainage system, as well as to evaluate the requirements and impacts of construction dewatering and the long-term groundwater management.

3.2 PREVIOUS INVESTIGATIONS

3.2.1 *GEOTECHNICAL DRILLING*

As noted in Section 3.1, previous subsurface investigations were carried out across the site. The location of the previous boreholes is shown in **Drawing 2**. The borehole logs from those previous investigations are presented in **Appendix B**.

In 2011, WSP carried out a geotechnical investigation on the southern portion of the site, which included the drilling of five (5) boreholes (FE-1-2011, FE-2-2011 and FG-1-2011 thru FG-3-2011) undertaken on December 8 and 9, 2011. The boreholes were advanced using a truck-mounted CME-55 drill-rig, equipped with hollow stem auger and split spoon sampling equipment, supplied and operated by Forage André Roy Inc. of Saint-Isidore, Québec.

- FE-1-2011 and FE-2-2011 were advanced to depths of approximately 2.2 and 0.9 m below the ground surface, respectively.
- FG-1-2011 thru FG-3-2011 were advanced to auger refusal, which ranged from depths of 0.8 to 1.1 m below ground surface. Upon encountered auger refusal, the boreholes were extended into the bedrock using NQ sized coring equipment to final depths ranging from 4.1 m to 4.7 m below the ground surface.
- Monitoring wells were installed in FG-1-2011 thru FG-3-2011to permit subsequent groundwater level measurement.

In the 2015 Golder geotechnical investigation, five (5) boreholes (1525987 15-1 to 1525987 15-5) were advanced at the northeastern portion of the site on March 24, March 25, and June 19, 2015. The boreholes were advanced using a truck-mounted drill rig, equipped with hollow stem augers, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario.

- Boreholes 1525987 15-1 through 1525987 15-5 were advanced to practical refusal at depths ranging from 2.4 to 3.1 m below the ground surface. Upon encountering auger refusal, the boreholes were extended into the bedrock to final depths ranging from about 5.6 to 15.3 m below the ground surface using rotary diamond drilling equipment while retrieving NQ or HQ sized bedrock cores.
- Monitoring wells were installed in all of the boreholes to permit subsequent groundwater level measurement.

In the 2016 WSP environmental investigation, a total of seven (7) boreholes were advanced in the southern portion of the site on January 11 to 13, 2016. The additional boreholes were identified as BH15-1 to BH15-6, BH15-3A and BH15-3B. The boreholes were advanced using a track-mounted CME-55 drill-rig supplied and operated by Downing Estate Drilling Ltd. of Grenville-sur-la Rouge, Quebec.

- BH15-1 to BH15-6, BH15-3A and BH15-3B were advanced to depths of 0.8 to 2.6 m below the ground surface.
- Upon encountering auger refusal, four of the boreholes (BH15-2, BH15-3B, BH15-4 and BH15-6) were into the underlying bedrock using HQ sized coring equipment to final depths ranging from 7.4 m to 8.0 m below the ground surface.
- Eight monitoring wells were installed at four (4) borehole locations, with the shallow wells identified as BH15-2A, BH15-3A, BH15-4A and BH15-6A and the deeper wells identified as BH15-2B, BH15-3B, BH15-4B, and BH15-6B.

The ground surface elevation and location of each borehole was surveyed and referenced to geodetic datum, except for FE-1-2011 and FE-2-2011 where the elevations were approximated. The ground surface elevation and depth of the boreholes advanced during the previous investigations are summarized in **Table 3-1** below.

Table 3-1 Ground Surface Elevation and Depth of Boreholes from Previous Investigations

BOREHOLE NUMBER	LOCATION	GROUND SURFACE ELEVATION (M)	BOREHOLE DEPTH (M)
FE-1-2011	Southern Portion of Site	75.0(1)	2.2
FE-2-2011	South (Outside) of Site Limit	75.0(1)	0.9
FG-1-2011	Southern Portion of Site	75.1	4.2
FG-2-2011	Southern Portion of Site	75.5	4.7
FG-3-2011	Southern Portion of Site	74.6	4.1
1525987 15-1	Northeastern Portion of Site	75.9	5.6
1525987 15-2	Northeastern Portion of Site	75.7	5.9
1525987 15-3	Northeastern Portion of Site	75.8	5.9
1525987 15-4	Northeastern Portion of Site	75.6	6.0
1525987 15-5	Northeastern Portion of Site	75.5	15.3
BH15-1	Southern Portion of Site	75.0	2.1
BH15-2	Southern Portion of Site	75.6	7.8
BH15-3A	Southern Portion of Site	75.5	2.6
BH15-3B	Southern Portion of Site	75.5	7.9
BH15-4	Southern Portion of Site	74.5	7.4
BH15-5	Southern Portion of Site	74.7	1.4
BH15-6	Southern Portion of Site	73.7	8.0

Note: [1] Ground surface elevation was approximated.

3.2.2 *LABORATORY INVESTIGATION*

Unconfined Compressive Strength (UCS) testing was carried out on three (3) selected bedrock samples recovered from the 2011 WSP investigation. The results of UCS testing are presented in **Appendix C**.

One sample of groundwater from Golder’s borehole 152598 15-3 was submitted to Paracel Laboratories (Paracel) for basic chemical analysis to determine potential for sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of the chemical analysis are included in **Appendix D**.

Table 3-2 summarizes the geotechnical laboratory testing completed during the previous investigations

Table 3-2 Geotechnical Laboratory Testing from Previous Investigations

BOREHOLE	SAMPLE NUMBER	DEPTH (m)	TEST EXECUTED
FG-1-2011	DC-5	2.5 – 2.7	Unconfined Compressive Strength
FG-2-2011	DC-6	3.9 – 4.2	Unconfined Compressive Strength
FG-3-2011	DC-6	3.3 – 3.7	Unconfined Compressive Strength
152598 15-3	GW	4.4 – 5.9	Chemical Analysis (chloride concentration, sulphate concentration, pH, electrical conductivity, resistivity)

3.2.3 GEOPHYISCAL TESTING

During the Golder 2015 investigation, a 50 mm inside diameter PVC pipe was installed in borehole 152598 15-5, with the outside of the pipe above the well screen backfilled with a bentonite-cement grout, to allow for subsequent geophysical testing.

The geophysical testing was carried out on June 24, 2015 and consisted of Vertical Seismic Profiling (VSP) through the overburden soils and the underlying bedrock. A detailed description of the procedure used for the VSP testing is provided in **Appendix E**.

4 SUBSURFACE CONDITIONS

The following provides a general description of the major soil types and bedrock encountered during the geotechnical investigations. It should be noted that the following discussion includes some simplifications for the purposes of discussing broadly similar soil strata. It should also be noted that the differences in soil types and changes between various soil and bedrock strata are often gradational, as opposed to precise boundaries of geological change.

A detailed description of the soil and bedrock stratigraphy encountered at each borehole location is shown on the borehole logs provided in **Appendix B**. The soil and bedrock stratigraphy are shown on the profiles on **Drawings 3A** and **3B**. Please note that the factual descriptions shown in each borehole log take precedence over the generalized (and simplified) descriptions presented below. Also, it is merit to consider the fact that boreholes findings represent the very location of these holes and not necessarily mean it represents the soil formation in the surrounding area.

4.1 TOPSOIL

A surficial topsoil layer was encountered in three of the boreholes (BH15-3A, BH15-5B and BH15-5) advanced during the WSP's 2016 environmental investigation. At the borehole locations, the topsoil was approximately 120 to 150 mm thick.

4.2 PAVEMENT STRUCTURE AND FILL

Pavement structure was encountered at boreholes FE-1-2011, FE-2-2011, FG-1-2011, FG-3-2011 (WSP's 2011 investigation), boreholes 1525987 15-1 thru 1525987 15-5 (Golder's 2015 investigation), and boreholes BH15-1 and BH15-2 (WSP's 2015 investigation).

Golder's 2015 boreholes were drilled within the existing parking lot at the northeastern portion of the site. At the borehole locations, the pavement structure consisted of 100 mm of asphaltic concrete, which overlies 150 to 210 mm of gravelly sand granular base (at boreholes 1525987 15-4 and 15-5) while the granular base was not identified in the remaining boreholes. The pavement structure was in turn underlain by a layer of sand and gravel fill, containing cobbles and organic matter. The fill extends to depths of about 2.4 to 3.1 m below the ground surface.

WSP's 2011 and 2016 investigations were advanced at the southern portion of the site. At the borehole locations (FE-1-2011, FE-1-2011, FG-1-2011, FG-3-2011, BH15-1 and BH15-2), the pavement structure, where encountered, consisted of 20 to 50 mm of asphaltic concrete, overlying 100 to 350 mm of sand and gravel granular base (except at FG-1-2011 where no granular base was identified). At FE-1-2011, the granular base was underlain by 820 mm of sand granular subbase, which was not encountered at the remaining boreholes.

Fill was encountered at all of WSP's boreholes either beneath topsoil, pavement structure or at the ground surface. The fill consisted of a heterogenous mixture ranging from sandy silt, silt and sand, silty sand, sand, to sand and gravel, with varying amount of gravel, organic matter and construction debris (e.g. pieces of brick, asphalt, wood, black carbon ashes). The fill extends to depths of about 0.5 to 2.2 m below the ground surface.

Standard penetration tests (SPTs) carried out within the pavement structure and fill measured 'N' values ranging widely from 3 to greater than 50 blows per 0.3 m of penetration, indicating a very loose to very dense state of packing. Some of the higher blow counts towards the lower portion of the overburden likely reflect the bedrock surface rather than the state of packing of the soil matrix.

It should be noted that the thickness of pavement structure and fill was based on the results of the previous investigations and may have altered as a result of site activities since the investigations were completed.

4.3 NATIVE SANDY AND GRAVELLY SOILS

A thin deposit of sandy and gravelly silt was encountered beneath the fill at borehole FE-1-2011. The deposit is approximately 0.4 m thick and extends to a depth of 2.2 m below the ground surface prior to encountering sampler refusal. The deposit was described as a probable compact glacial till.

In Golder's borehole 1525987 15-5, a deposit of silty sand was encountered below the fill. The silty sand deposit is approximately 0.3 m thick and contains a trace of gravel as well as organic matter, extending to a depth of 2.6 m below the ground surface. One SPT 'N' value of greater than 50 blows per 0.3 m of penetration was measured within the silty sand. However, this high blow count likely reflects the presence of the bedrock surface rather than the state of packing of the soil matrix.

At boreholes BH15-1 and BH15-5 (WSP's 2016 investigation), a gravel layer was encountered beneath the fill. The gravel layer was approximately 0.2 m thick, containing sand and shale fragments, and extends to depths of 1.4 m and 0.9 m below the ground surface, respectively.

4.4 BEDROCK

4.4.1 *WEATHERED BEDROCK*

Weathered limestone was encountered below the fill or gravel layer at approximately 0.5 to 2.2 m below the ground surface (Elevation 73.3 to 74.8 m) in WSP's boreholes FG-1-2011, BH15-1, BH15-2, BH15-3A, BH15-4 and BH15-5. Hollow stem augers were able to penetrate past this upper portion of bedrock. The weathered zone is estimated to be approximately 0.3 to 0.7 m in thickness prior to encountering refusal to augering.

No weathered bedrock was encountered within any of Golder's 2015 boreholes advanced at the northeastern portion of the site.

The depths and elevations of the weathered bedrock surface are summarized in **Table 4-1**.

Table 4-1 Weathered Bedrock Surface Depths and Elevations

BOREHOLE NUMBER	GROUND SURFACE ELEVATION (M)	WEATHERED BEDROCK SURFACE DEPTH (M)	WEATHERED BEDROCK SURFACE ELEVATION (M)	WEATHERED BEDROCK THICKNESS (M)
FG-1-2011	75.1	0.5	74.6	0.6
BH15-1	75.0	1.4	73.6	0.7
BH 15-2	75.6	0.8	74.8	0.3
BH15-3A	75.5	2.2	73.3	0.4
BH15-4	74.5	1.0	73.5	0.3
BH15-5	74.7	0.9	73.8	0.4

4.4.2 SOUND LIMESTONE BEDROCK

Sound limestone bedrock was encountered at boreholes FG-1-2011 to FG-3-2011 (WSP's 2011 investigation), boreholes 1525987 15-1 thru 1525987 15-5 (Golder's 2015 investigation), and boreholes BH15-2, BH15-3B, BH15-4 and BH15-6 (WSP's 2016 investigation). The bedrock was confirmed by diamond drilling techniques while retrieving NQ or HQ sized bedrock cores.

Table 4-2 summarizes the depths and elevations of the sound bedrock surface.

Table 4-2 Sound Bedrock Surface Depths and Elevations

BOREHOLE NUMBER	LOCATION	GROUND SURFACE ELEVATION (M)	SOUND BEDROCK SURFACE DEPTH (M)	SOUND BEDROCK SURFACE ELEVATION (M)
1525987 15-1	Northeastern Portion of Site	75.9	2.4	73.4
1525987 15-2	Northeastern Portion of Site	75.7	2.7	73.0
1525987 15-3	Northeastern Portion of Site	75.8	2.8	73.0
1525987 15-4	Northeastern Portion of Site	75.6	3.1	72.6
1525987 15-5	Northeastern Portion of Site	75.5	2.6	72.9
FG-1-2011	Southern Portion of site	75.1	1.1	74.0
FG-2-2011	Southern Portion of site	75.4	1.0	74.4
FG-3-2011	Southern Portion of site	74.6	0.8	73.8
BH15-2	Southern Portion of site	75.6	1.1	74.5
BH15-3B	Southern Portion of site	75.5	2.2	73.3
BH15-4	Southern Portion of site	74.5	1.3	73.2
BH15-6	Southern Portion of site	73.7	1.5	72.2

The bedrock was described as fresh, thinly to medium bedded, grey, fine grained, non-porous limestone, with black shale partings (Golder, 2015). It was believed that the limestone belongs to the Trenton Geological Group, which is composed of a carbonate sedimentary and fossiliferous rock dating from the Middle Ordovician Era (some 471 to 460 million years ago) (WSP, 2015).

The Rock Quality Designation (RQD) constitutes an indirect measure of the number of fractures and degree of alteration of the rock mass. This is obtained using the length of rock coring, adding the lengths of intact pieces, which are at least 100 mm long. The RQD value, indicated as a percentage, is the ratio of the sum of all minimum 100 mm-long cores by the total length drilled. The RQD classification of the rock according to this value is indicated in **Table 4-3** below.

Table 4-3 Rock Classification according to the Rock Quality Designation (RQD)

CLASSIFICATION	RQD VALUES INTERVAL (%)
Very poor quality	< 25
Poor quality	25 – 50
Fair quality	50 – 75
Good quality	75 – 90
Excellent quality	90 – 100

The RQD values measured in Golder's 2015 boreholes (advanced at the northeastern portion of the site) range from 81% to 100%, indicating that the rock quality of the limestone bedrock is good to excellent throughout the entire core lengths.

Based on the RQD values in WSP's boreholes (advanced at the southern portion of the site), in general, the rock quality of the upper 1.2 m of the limestone bedrock is poor to very poor, becomes fair between depths of about 1.2 m and 2.8 m, and below which the rock quality is good to excellent. The measured RQD values from the WSP's 2011 and 2016 investigations are presented in **Table 4-4** below.

Table 4-4 Limestone Rock Quality as a Function of Depth

BOREHOLE NUMBER	VERY POOR TO POOR QUALITY ZONE (M) (RQD)	FAIR QUALITY ZONE (M) (RQD)	GOOD QUALITY ZONE (M) (RQD)	EXCELLENT QUALITY ZONE (M) (RQD)
FG-1-2011	1.1 – 1.4 (0%)	1.4 – 2.8 (56%)	-	2.8 – 4.2 (90%)
FG-2-2011	1.0 – 3.1 (0% – 49%)	-	3.1 – 4.7 (80%)	-
FG-3-2011	0.8 – 1.2 (0%)	1.2 – 2.8 (70%)	2.8 – 4.1 (75%)	-
BH15-2	0.9 – 1.2 and 2.7 – 4.2 (36% and 49%)	1.2 – 2.7 and 7.2 – 7.8 (67% and 54%)	4.2 – 7.2 (87%)	-
BH15-3B	-	1.8 – 2.7 (55%)	-	2.7 – 7.9 (92% – 98%)
BH15-4	-	-	4.4 – 5.9 (78%)	1.3 – 4.4 and 5.9 – 7.4 (92% – 96%)
BH15-6	-	1.5 – 2.8 (63%)	4.3 – 8.0 (75% – 90%)	2.8 – 4.3 (100%)

The RQD values measured from the recovered bedrock are plotted against the elevation of each sample, as shown on **Drawing 4**.

UCS testing was carried out on three selected bedrock core samples from WSP's 2011 investigation. The laboratory test results are provided in **Appendix C** and are summarized in the **Table 4-5** below. Based on the results of the UCS testing, the limestone bedrock at this site is strong to very strong.

Table 4-5 Results of Unconfined Compressive Strength

BOREHOLE NO.	SAMPLE NUMBER	CORE DEPTH (M)	UNCONFINED COMPRESSIVE STRENGTH (MPA)
FG-1-2011	DC-5	2.5 to 2.7	109
FG-2-2011	DC-6	3.9 to 4.2	74
FG-3-2011	DC-6	3.3 to 3.7	128

4.5 GROUNDWATER CONDITIONS

Groundwater levels in the monitoring wells were measured on December 12, 2011 (at boreholes FG-1-2011 to FG-3-2011), on March 27, 2015 (at boreholes 1525987 BH15-1 thru 1525987 BH15-4), and on January 19, 2016 (at boreholes BH15-2, BH15-3A, BH15-3B, BH15-4 and BH15-6). **Table 4-6** presents the results of the groundwater level measurements.

Table 4-6 Groundwater Depth and Elevations from Previous Investigations

BOREHOLE NUMBER	GEOLOGICAL UNIT	GROUND SURFACE ELEVATION (M)	GROUNDWATER DEPTH (M)	GROUNDWATER ELEVATION (M)	DATE OF MEASUREMENT
FG-1-2011	Weathered/Sound Bedrock	75.1	2.1	73.0	Dec 12, 2011
FG-2-2011	Fill/Bedrock	75.4	2.3	73.1	Dec 12, 2011
FG-3-2011	Fill/Bedrock	74.6	1.9	72.7	Dec 12, 2011
1525987 15-1	Bedrock	75.9	2.5	73.4	Mar 27, 2015
1525987 15-2	Bedrock	75.7	2.9	72.8	Mar 27, 2015
1525987 15-3	Bedrock	75.8	3.4	72.4	Mar 27, 2015
1525987 15-4	Bedrock	75.6	2.7	72.9	Mar 27, 2015
BH15-2	Bedrock	75.6	1.5(1) 2.0(2)	74.1(1) 73.6(2)	Jan 19, 2016
BH15-3A	Fill/Bedrock	75.5	1.3	74.2	Jan 19, 2016
BH15-3B	Bedrock	75.5	5.1	70.4	Jan 19, 2016
BH15-4	Bedrock	74.5	2.4(1) 5.8(2)	72.1(1) 68.7(2)	Jan 19, 2016
BH15-6	Bedrock	73.7	2.0(1) 6.6(2)	71.7(1) 67.1(2)	Jan 19, 2016

Notes: (1) Shallow monitoring well screen (2) Deeper monitoring well screen

It should be noted that the groundwater levels are only representative of the period during which the readings were taken. Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as springs, or following heavy rainfall events.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

This section of the report provides engineering guidance related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and the project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities. Reference should be made to the Limitations of this Report, attached in **Appendix A**, which follows the text but forms an integral part of this document.

5.2 OVERVIEW

In general, the subsurface conditions on this site consist of about 0.5 to 3.1 m of pavement structure and fill, overlying thin deposits of native sandy and gravelly soils, above limestone bedrock. The surface of the limestone bedrock varies from about 0.8 to 3.1 m below ground surface (Elevation 72.2 to 74.8 m). The upper portion of the limestone bedrock is generally weathered.

The proposed development will consist of a mixed-use of residential building, which will be 9- to 26- storeys in heights and contains 2 levels of underground parking.

The following list summarizes some key geotechnical issues associated with this project:

- Excavation for the construction of the basement and building foundations and basement levels will extend to through the surficial fill, sandy and gravelly soils, and into the underlying limestone bedrock. Excavation into the sound bedrock needs to be carried out using techniques with minimum disturbance to the adjacent structures and services. Vibration monitoring will be required during excavation activities.
- Given the constraints imposed by adjacent properties and roadways, it is expected that temporary shoring systems will be necessary to support the overburden. Design of a shoring system is beyond the scope of this report. However, along the perimeter where no adjacent structures exists (north, east, as well as a portion of the south wall), it is anticipated typical system may consist of steel soldier piles and timber lagging. Along the perimeter where adjacent structure exists (west wall and remaining portion of south wall), a shoring system consisting of interlocking steel sheet piles or diaphragm walls that controls movement to within tolerable limits is required. The use of ground anchors may also be required.
- Underpinning of the adjacent structures located adjacent to the western and southern portions of the property may be necessary.
- Foundations (such as spread footings and raft foundations) founded on or within sound limestone bedrock can be designed using an Ultimate Limit States (ULS) factored bearing resistance of 7.4 MPa in accordance with the Canadian Foundation Manual (CFEM). For seismic design, this site can be assigned a Site Class of A in accordance with the Ontario Building Code (OBC) regulations.
- The groundwater levels on this site were measured at depths of about 1.3 to 6.6 m below the ground surface (Elevation 67.1 to 74.2 m). A hydrogeological study will be required to evaluate the requirements and impacts of construction dewatering and long-term groundwater management.

5.3 EXCAVATIONS

The proposed residential building will consist of two levels of underground parking, with the finished floor of the lowest basement level at approximately Elevation 68.9 m, which is approximately 5 to 7 m below the existing ground surface. Considering that the excavation will likely extend a further 1.0 to 1.5 m below the lowest basement floor level to accommodate the foundations and possible elevator pits, it is expected that the excavation will extend to about 6.5 to 8.5 m below the existing ground surface (Elevation 67.4 to 67.9 m).

Based on the above, the excavation for basement and foundation construction will extend through the existing fill, native sandy and gravelly soils, and into the underlying limestone bedrock.

The structures that are at risk of being impacted by ground movements around the excavation are the low-rise buildings located immediately west and southeast of site. These structures may have been founded on the bedrock surface and, if that is the case, the excavation will likely have little impact on the structure. Otherwise, if the foundations of the adjacent structures are founded on overburden and are within the close proximity of the excavation, then underpinning may be required.

As general guideline for excavation, a minimum distance of 1 m should be maintained between adjacent footings and the boundaries of excavation. To avoid undermining of the rock and/or disturbance of the rock (which could jeopardize the support for the structure), careful line drilling of the excavation limits in this area must be undertaken.

Geotechnical recommendations on excavations of overburden and bedrock are discussed further below.

5.3.1 *OVERBURDEN EXCAVATIONS*

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). Temporary excavation details and requirements are given in Part III of Ontario Regulation 213/91.

The soils at the sites include granular fill and native sandy to gravelly soils, which above the groundwater level can be classified as Type 3 Soils. Excavation side slopes for Type 3 soils could be sloped at a minimum of 1H:1V. However, if the excavation is not first dewatered, the overburden below the groundwater level would be classified as a Type 4 soil; side slopes of at least 3H:1V would be required in accordance with OSHA.

It is understood that the proposed building will encompass essentially the full limits of the property. Given the constraints imposed by the adjacent properties and roadways, it is expected that temporary support (shoring) systems will be required to support the excavation faces within the overburden along the property boundaries. Additional guidance on temporary shoring are provided in Section 5.3.4 below.

All excavated surfaces should be kept free of frost, water, etc. during construction operations, and all excavated surfaces should be inspected by qualified geotechnical personnel, to confirm the consistency of the findings presented in this investigation report and the design and construction of similar structures.

Stockpiling of soil beside the excavations should be avoided; the weight of the stockpiled soil could lead to basal instability of braced excavations, or slope instability of unsupported excavations.

It is recommended heavy vehicles not be parked close to excavation edges, or within the projected 1H:1V distance from the bottom of the excavation. If excavation material is to be temporarily stored on site, it must be placed at a minimum distance from the crest of the slope, equivalent to the depth of the excavation. This must be respected at all times unless specific geotechnical analyses to confirm otherwise.

The excavated soils should be disposed of in a proper manner, depending on its environmental quality and following the recommendations from the Phase II ESA report completed for this site.

5.3.2 BEDROCK EXCAVATIONS

The bedrock surface varies from Elevation 72.2 to 74.8 m. Bedrock removal will be required for excavation that extends below these elevations.

Excavations of the upper weathered bedrock could be accomplished using mechanical methods (such as by hydraulic shovel and hoe ramming). Deeper excavations into the sound bedrock could be carried out using drill and blast procedures. If blasting is required, these operations should be conducted carefully and in accordance to Ontario Provincial Standard Specification (OPSS) 120.

Near vertical walls in the limestone bedrock is considered feasible during the construction period.

No major open fractures were detected in the bedrock during the previous geotechnical investigations. However, it is possible that such fractures be encountered during the bedrock excavation. If present, such fractures may have an effect on the behaviour of the rock mass during excavation due to undesirable rock movements and the foundations integrity of the existing adjacent buildings may be compromised. In such case, as would be pointed out by the geotechnical inspection during excavation, rock bolting may be required to support the excavated rock faces. Inspection during excavation is recommended at the early stages to evaluate the potential need for rock reinforcement.

Blast induced damage to the bedrock must be avoided; otherwise rock reinforcement would be required. It should 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 and services. The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that the blast induced damage will be avoided. A blast design specialist in this field will be required.

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

5.3.3 VIBRATION MONITORING

The contractor should be required to submit a detailed blasting design and vibration monitoring plan proposal prepared by a blasting/vibration specialist prior to commencing work. This plan would have to be reviewed and approved 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 and should be verified by the specialist:

FREQUENCY RANGE (HZ)	PEAK VIBRATION LIMITS (MM/S)
<10	5
10 to 40	5 to 50 (sliding scale)
>40	20

If practical, blasting should be carried out at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics, to confirm the anticipated ground vibration levels, and to adjust the contractor’s blasting methods as needed.

5.3.4 TEMPORARY SUPPORT (SHORING)

5.3.4.1 SHORING OPTIONS

The excavation will essentially encompass the full limits of the property and therefore near vertical excavation walls will be required.

Design of the shoring system is beyond the scope of this report. Detailed design and performance of the temporary shoring systems needs to be established prior to start of the excavation. The shoring system design should take into considerations the impact of movement of the installed shoring system on the adjacent structures (both buildings and infrastructure), and feasibility of construction. A detailed investigation of the adjacent structures is required to incorporate in the shoring design. Below are typical shoring systems that are suitable for the soil conditions at the site.

- It is envisioned that steel soldier piles and timber lagging shoring would be feasible along the northern (Carling Avenue), eastern (Bronson Avenue), and southwestern (undeveloped) limits of the site, where the excavation will be adjacent to the existing roadways or undeveloped land. Soldier piles and lagging systems are suitable where the objective is to maintain an essentially vertical excavation wall and where the movements above and behind the wall need only be sufficiently limited that relatively flexible features (such as roadways) will not be adversely affected.
- For excavations where existing buildings are present adjacent to the excavation (such as at the western and southeastern limits of the site), interlocking driven steel sheet pile system with pre-stressed tiebacks will likely be needed. The sheet piling systems with pre-stressed tie could greatly limit the shoring deflections. However, its feasibility will depend on whether those existing structures are founded on overburden or on the bedrock surface.
- Continuous concrete shoring (such as a secant piles or diaphragm walls) could also be a feasible alternative for the sides of the excavation adjacent to the existing structures. Diaphragm walls are appropriate where difficulties may be encountered installing sheet piles, where heavily loaded foundations exist adjacent to the shoring, or where groundwater inflow needs to be controlled. Such systems could greatly mitigate the potential for foundation movements but would also be much more expensive.
- At locations where structures sensitive to movement exists, such as adjacent to western and southeastern sides of the site, underpinning may be required to control displacement. That is, if the resulting displacement due to the movements of the applied shoring system is unacceptable and/or if the loads on the adjacent foundations are large. Further details on the foundations of the existing structures will be required for a full assessment of the required shoring to be implemented.

For all of the above systems, some form of lateral support to the shoring system is required for excavation depths greater than about 3 or 4 m. Lateral restraint could be provided by means of tiebacks consisting of grouted bedrock anchors. However, the use of rock anchor tiebacks would require the permission of the adjacent property owners (including the City of Ottawa, who owns the adjacent roadways), since the anchors would be installed beneath their properties. The presence of utilities beneath the adjacent streets, which could interfere with the tiebacks, should also be considered. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or design of raker piles and/or footings within the excavation. However, internal struts could interfere with the construction of the foundations and superstructure.

The shoring should also be designed to account for lateral earth pressures resulting from the weight of the retained earth and other dead and surcharge loads. The earth pressure distribution used for shoring design is dependent upon the specific wall design and on the nature of the lateral support provided. The potential for the loads from the adjacent foundations to apply additional lateral pressure to the shoring system should be considered. The selection of that design lateral earth pressure should therefore be the responsibility of the contractor who will be responsible for the shoring design.

5.3.4.2 GROUND MOVEMENTS

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground will occur as a result of excavation, installation of shoring system, deflection of the ground support system (including bending of the walls, compression of the struts and/or extension of the tiebacks), as well as deformation of the soil/rock in which the toes of the shoring walls are embedded. The ground movements could affect the performance of buildings, surface structures, and underground utilities adjacent to the excavation.

As a preliminary guideline, typical settlements behind soldier pile and lagging shoring systems are less than about 0.2% of the excavation depth, provided good construction practices are used, voids are not left behind the lagging, and also provided that large foundation loads from existing buildings are not applied behind the shoring. This guideline would suggest that about 10 to 15 mm of ground settlement would occur for shoring systems installed through the overburden and bedrock to about 5 m depth.

Movements behind a properly constructed steel sheet pile or continuous caisson wall would be less than what would be expected for a soldier pile and lagging wall. However, this is only a preliminary guideline and is provided only to assist the owner's designers in carrying out an initial assessment of the expected settlements and the potential impacts of these settlements. A more detailed assessment of the expected settlements should be undertaken by the contractor and must consider the effects of adjacent foundation loads.

Should the preliminary assessment carried out using this estimated settlement indicate unacceptably large settlements to adjacent structures, roadways, or utilities, then a more detailed assessment should be carried out during future design stages (but prior to tender) to better assess the shoring requirements, or a more rigid form of shoring should be selected.

A pre-construction survey of all adjacent structures should be carried out prior to commencement of the excavation.

Underground utilities should be considered during the shoring design in terms of possible conflicts with tieback installation and/or possible restrictions on the acceptable ground/shoring movements. Therefore, an inventory of these utilities should be made at an early stage in the design.

5.3.5 *GROUNDWATER MANAGEMENT*

The groundwater levels at this site were measured at elevations varying from 67.1 m to 74.2 m, which is above the anticipated base of the excavation. Groundwater inflows into the rock excavation is therefore expected.

The actual rate of groundwater inflow will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where volumes of precipitation, surface runoff and/or groundwater collects in an open excavation must be pumped out. The contractor shall provide a pumping system to remove all water to the bottom of the excavation. Excavation shall be kept dry at all times.

A Permit-to-Take Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water pumped from the excavations under normal operation will be greater than 400,000 L/day. However, if the volume of water to be pumped will be less than 400,000 L/day but more than 50,000 L/day, the water taking will not require a PTTW, but will instead need to be registered in the Environmental Activity Sector Registry (EASR) as a prescribed activity.

A hydrogeological study will be required to determine the groundwater pumping requirements for this site and support an application for a PTTW or registration of EASR.

The planned temporary (during construction) and permanent dewatering (long-term due to the foundation drainage system, if one is provided) would directly impact ground settlements. Consequently, adjacent structures founded on sensitive and compressible clay would be affected if within the zone influenced by the lowering of ground water table. The results of this investigation as well as the published geologic mapping do not reveal the presence of such soils, at least within the immediate vicinity of the site. Regardless, a hydrogeological study will be required to confirm/evaluate the potential impacts of the proposed development on the adjacent structures.

Design of a dewatering system is beyond the scope of this report. Typical design may be provided by WSP under separate mandate and/or can be reviewed if proposed by a specialty contractor. An outlet (or outlets) should be identified which the contractor can use to dispose of the pumped groundwater and incident precipitation. In order for pumped groundwater to be discharged to a City of Ottawa's sewer, the groundwater quality needs to meet the City of Ottawa's Sewer Use By-law criteria and a separate sewer discharge permit must be obtained. Additional ongoing chemical testing should be carried out at the time of construction to monitor the groundwater quality so that disposal requirements can be confirmed throughout the duration of construction.

5.4 FOUNDATIONS

5.4.1 *GEOTECHNICAL BEARING RESISTANCE*

As previously noted, the proposed building will consist of two levels of underground parking, with the finished basement floor at approximately Elevation 68.9 m. Based on the assumed depth of excavations, it is anticipated that the underside of foundation will be founded within the sound limestone bedrock.

The results of UCS testing from the WSP's 2011 investigation indicated that the limestone bedrock at this site is strong to very strong. Based on the results of the UCS testing, for spread footings or raft foundations constructed within the sound limestone bedrock surface may be designed using an Ultimate Limit States (ULS) factored geotechnical resistance of 7.4 MPa in accordance with CFEM. The upper portion of the bedrock was noted to be weathered in some of the boreholes. However, based on the founding depths of the interior foundations, the underside of the foundations is expected to be below the depth of weathering.

Provided the bearing surface is cleaned of loose bedrock, Serviceability Limit States (SLS) net bearing resistances will not apply to the design of foundations on the bedrock, since the settlement of foundations at the corresponding (unfactored) service will be less than 25 mm.

5.4.2 *SLIDING RESISTANCE*

The ultimate resistance of the foundation to lateral loading may be calculated using a factored ULS coefficient of friction value of 0.56 across the interface between the footing and the bedrock.

5.5 ROCK ANCHORS

Rock anchors may be required to resist overturning and/or uplift forces. The anchors could consist of either grouted or mechanical anchors. In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) Failure of the steel tendon or top anchorage.

ii) Failure of the grout/tendon bond.

iii) Failure of the rock/grout bond.

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

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 1,000 kPa for ULS design purposes. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

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

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where:

Q_r = Factored uplift resistance of the anchor, kN

ϕ = Resistance factor, 0.4

γ' = Effective unit weight of rock, use 27 kN/m^3 above groundwater level, 17 kN/m^3 below the groundwater level

D = Anchor length, m

θ = Half of the apex angle of the rock failure cone, use 30 degrees

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity (as required by OBC for the use of a resistance factor of 0.6). For preliminary evaluation purposes, the testing procedures should be in accordance with the Post-Tensioning Institute's Recommendations for Prestressed Rock and Soil Anchors and testing procedures outlined in the OPSS. A more detailed testing program should be developed once further details on the rock anchors (e.g., required loads, total number of anchors, anchor spacing etc.) are known.

Rock anchors intended as permanent structural elements should be provided with double corrosion protection and tested in accordance with OPSS 942.

The installation and testing of the anchors should be supervised by qualified geotechnical personnel. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the anchor holes to be grouted to ensure an adequate bond between the grout and the rock.

Prestressing of the anchors prior to loading would minimize anchor movement due to service loads.

Further guidance can be provided for assessing the anchor resistance once the final anchor layout and loads have been established, if requested.

5.6 SEISMIC DESIGN

The OBC specifies that the structure should be designed to withstand forces due to earthquakes. For the purpose of earthquake design, the information relevant to the geotechnical conditions at this site is the 'Site Class'. The seismic design provisions of the OBC depend, in part, on the shear wave velocity of the upper 30 m of soil and/or rock below founding level.

Site specific shear wave velocity profiling using the VSP testing (a down-hole geophysical method) was carried out within Golder's borehole 15-5. The results of that testing are provided in **Appendix E**.

The results of the VSP testing indicate that the bedrock below about 8 m depth has an average shear wave velocity of greater than 1,750 m/s. In accordance with OBC, the proposed building can be designed using a Site Class A designation.

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 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.

With the anticipated founding level to accommodate the below-grade parking, and assuming that the parking garage will not be allowed to freeze, there will be sufficient earth cover to protect against frost. In addition, the foundations are expected to be placed directly on sound limestone bedrock (following blasting and cleaning), which is considered as non-frost susceptible in the absence of soil filled joints and/or seams.

However, the buildings immediately west and southeast of the site are likely supported on shallow foundations, which may be founded on frost susceptible soils. The excavation shoring will be constructed within very close proximity to the foundations of the buildings and, if construction is carried out during the winter months, the existing foundations may be adversely affected due to frost movement. Therefore, if construction is anticipated during sub-zero temperatures, provision should be made to protect the soils behind the shoring from frost movement.

5.8 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.

It is not known if the basement levels will be designed to be of drained or water-tight construction. If a “drained” structure will be considered, provision should be made for at least 300 mm of free draining granular material, such as 19-mm diameter clear crushed stone, to form the base of the floor slab. To prevent hydrostatic pressure build up beneath the floor slab, the granular base for the floor slab should be drained. This should be achieved by installing rigid 100 mm diameter perforated pipes in the floor slab bedding at 6 m 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 mm of Granular A (City of Ottawa SP F-3147) base should be provided above the clear stone and compacted to at least 100 % of the material’s standard Proctor maximum dry density.

If water-tight construction is required for this structure, then the basement floor slab will have to be of concrete slab construction, rather than asphalt, and would have to be designed to be integral with the foundation walls (i.e., to form a raft slab). The basement floor and foundation walls will have to be designed to resist hydro-static uplift pressures. Rock anchors may be required to resist the hydro-static uplift pressures (buoyant forces). Recommendations for rock anchors are provided in Section 5.6 above.

5.9 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 and the forming methods. The following sections assume that water-tight construction will not be required.

If water-tight construction is needed, additional design guidance will need to be provided.

5.9.1 BASEMENT WALLS AGAINST SOILS

The soils at this site are potentially frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration 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 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 m thick lifts and compacted to at least 95% of the material's standard Proctor maximum dry density.

The basement wall backfill (for the full height of the wall) should be drained by means of a perforated pipe subdrain in a surround of 19-mm diameter 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.

5.9.2 BASEMENT WALLS AGAINST 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 mm 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 backfill 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.

5.9.3 LATERAL EARTH PRESSURES

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

1) Walls cast directly against the bedrock face.
2) Walls cast against formwork with a narrowly 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 (including the upper portions of the walls, above the bedrock surface).

For the first case (walls cast against the bedrock with Miradrain), there will be 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:

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

Where:

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

K = Earth pressure coefficient, use 0.6

γ = Unit weight of retained soil, use 22 kN/m^3

B = Width of backfill between basement wall and bedrock face, m

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

z = Depth below top of shoring, m

q = Uniform surcharge at ground surface to account for traffic, equipment, or stockpiled materials, use 15 kPa. Additional/higher surcharge loads associated with existing building foundations should also be accounted for where existing buildings are located adjacent to the basement walls.

It should be noted that the resulting pressure distribution for the first case is not triangular and that the lateral earth pressures above the gallery (i.e., bedrock surface) should be calculated as noted below for the second case.

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, kPa

K_o = At-rest earth pressure coefficient, use 0.5

γ = Unit weight of retained soil, use 22 kN/m^3

z = Depth below top of wall, m

q = Uniform surcharge at ground surface to account for traffic, equipment, or stockpiled materials, use 15 kPa. Additional/higher surcharge loads associated with existing building foundations should also be accounted for where existing buildings are located adjacent to the basement walls.

For all cases, hydrostatic groundwater and different lateral earth pressures (e.g., effective unit weights of the soils would apply to the above equations) would also need to be considered if the structure is designed to be water-tight. Additional guidance will therefore need to be provided if water-tight construction is considered.

Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against shoring or bedrock (i.e., without a drainage layer), 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 should 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:

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

Where:

K_{AE} = The seismic earth pressure coefficient, use 0.6

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, if this option is selected, more sophisticated analyses would need to be carried before guidance could be provided.

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Limit States Design (LSD) 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 guidance for the design of the basement walls and foundations will need to be provided.

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 may have to be placed to form a frost taper, depending on the composition of the existing fill. The frost taper should be brought up to pavement subgrade level from 1.5 m below the finished exterior grade at a slope of 3H:1V, or flatter, away from the wall. The granular fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.10 SITE SERVICING

Recommendations on excavations for site servicing (e.g., storm, sanitary sewers etc.) are provided in Section 5.3.

At least 150 mm of Granular A should be used as pipe bedding. Depending on the condition of the subgrade, it may be necessary to place a sub-bedding layer consisting of 300 mm of Granular B Type II beneath the Granular A, or the Granular A layer could be thickened. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95% of the material's 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 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 Granular A or Granular B Type I with a maximum particle size of 26.5 mm. The cover material should be compacted to at least 95% of the material's standard Proctor maximum dry density.

All trench backfill should conform to City of Ottawa specification SP F-2120. The trench backfill should consist of Granular A, Granular B Type I or II (City of Ottawa SP F-3147). The granular fill should be placed in maximum 200 mm thick lifts and should be compacted to at least 95% of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

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% of the material's standard Proctor maximum dry density using suitable compaction equipment.

Clay dykes are not required for this site.

5.11 PAVEMENT DESIGN

In preparation for pavement construction, all loose and deleterious should be subexcavated from all pavement areas (which is expected to be completed as part of the foundation excavations).

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or Select Subgrade Material (SSM) meeting the requirements of OPSS.MUNI 212 and SP F-3147, respectively. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% 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 catchbasins for a distance of at least 3 m in four orthogonal directions, or longitudinally where parallel to a curb.

The pavement structure for parking lots and heavy traffic lanes are provided in **Table 5-1** and **Table 5-2** below.

Table 5-1 Pavement Design for Parking Lots

LAYER	MATERIAL	THICKNESS (MM)	COMPACTION (STANDARD PROCTOR MAXIMUM DRY DENSITY) (%)
Asphaltic concrete (Surface course)	Superpave 12.5	40	See Note 1
Asphaltic concrete (Base course)	Superpave 19.0	50	See Note 1
Granular Base	Granular A	200	100%
Granular Subbase	Granular B Type II	300	100%

Note: [1]Asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

Table 5-2 Pavement Design for Heavy Truck Traffic Lanes

LAYER	MATERIAL	THICKNESS (MM)	COMPACTION (STANDARD PROCTOR MAXIMUM DRY DENSITY) (%)
Asphaltic concrete (Surface course)	Superpave 12.5	40	See Note 1
Asphaltic concrete (Base course)	Superpave 19.0	50	See Note 1
Granular Base	Granular A	200	100%
Granular Subbase	Granular B Type II	400	100%

Note: [1]Asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The granular base and subbase should consist of Granular A and Granular B Type II (City of Ottawa SP F-3147), respectively, and should be uniformly compacted to at least 100% of the materials' standard Proctor maximum dry density using suitable vibratory compaction equipment.

The asphaltic concrete should meet the requirements of City of Ottawa specification F-3106. The Performance Graded Asphalt Cement (PGAC) should consist of PG 58-34. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The above pavement designs are 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.

5.12 CORROSION AND CEMENT TYPE

One sample of groundwater from Golder's previous borehole was submitted to Paracel for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of the testing are provided in **Appendix D**.

Table 5-3 Results of Chemical Analysis

BOREHOLE NUMBER	WELL SCREEN DEPTH (M)	CHLORIDE (MG/L)	SULPHATE (MG/L)	PH	CONDUCTIVITY (µS/CM)
1525987 15-3	4.4 – 5.9	1,800	141	7.2	5,730

The results indicate an elevated potential for corrosion of exposed ferrous metal, which should be considered in the design of exposed steel (such as rock anchors). The results also indicate that concrete made with Type GU Portland cement should be acceptable for substructures.

6 CONSTRUCTION CONSIDERATIONS

6.1 QUALITY CONTROL/ASSURANCE

The successful execution of the project will depend upon excellent workmanship and quality control/assurance during construction.

All foundation and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that competent bedrock has been reached and that the bearing surfaces have been properly prepared.

The installation and testing of the rock anchors, if required, should be supervised by qualified geotechnical personnel. The placement and compaction of any engineered fill should be inspected to ensure that the materials used conform to the project specifications.

WSP can provide these services upon request. WSP should also be retained to review the detailed drawings and specifications for this project prior to tendering to ensure that the recommendations provided in this report have been adequately interpreted.

In addition, the proposed blasting design and monitoring plans proposed by the contractor should be reviewed prior to commencement of the work.

The monitoring wells installed during the previous investigations at this site will need to be abandoned in accordance with Ontario Regulation 903. However, these devices may be useful and more economically removed during construction. It is therefore proposed that decommissioning of these devices be made part of the construction contract.

6.2 EXCESS SOIL MANAGEMENT

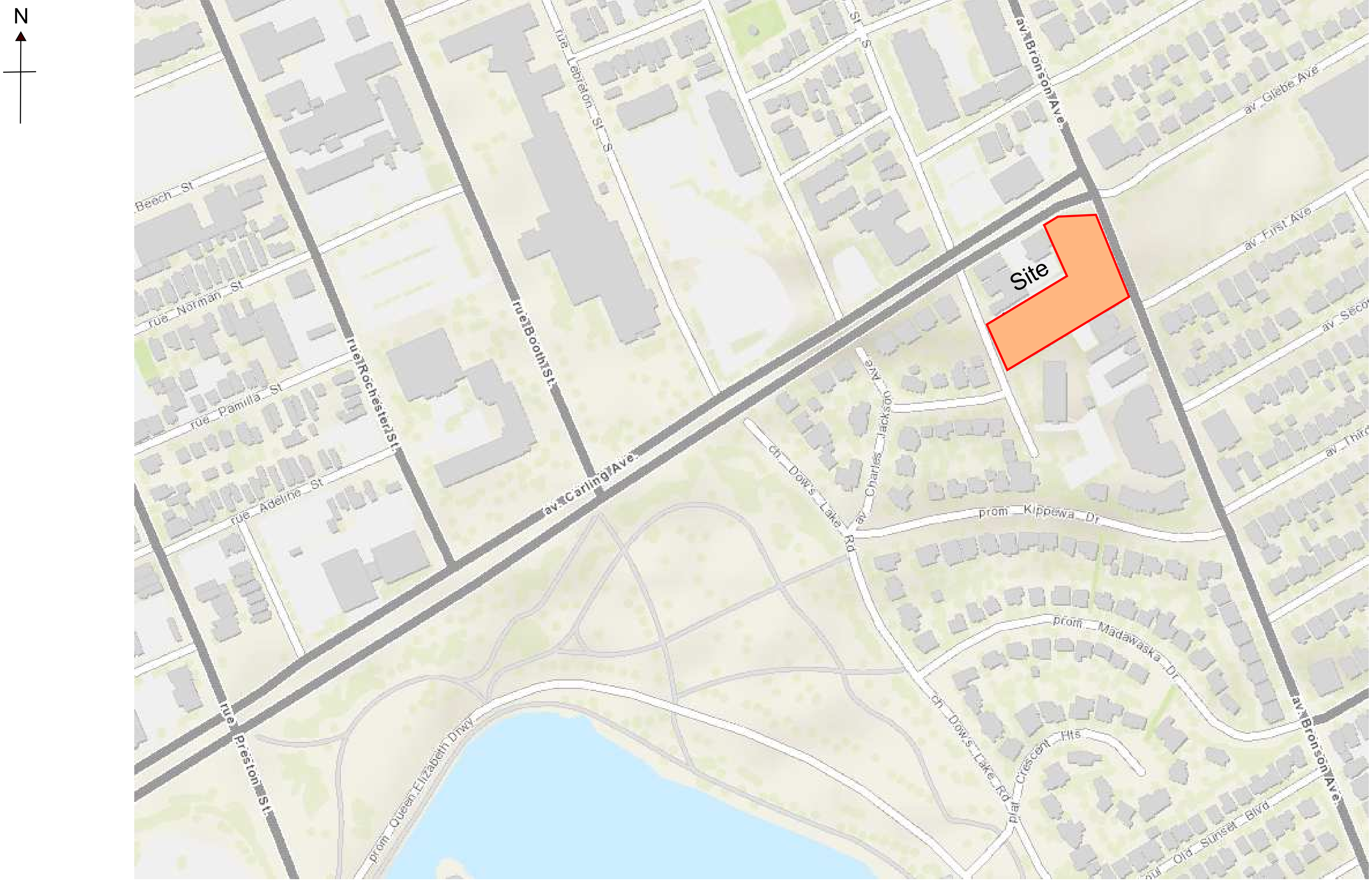
For Projects that will generate excess soil, management of such soil both on-site and off-site will need to be carried out in compliance with the Excess Soil Regulation (Ontario Regulation 406/19). The Excess Soil Regulation is being phased in over a five-year period commencing on January 1, 2021, and the Excess Soil Reuse Planning Requirements (as noted below) will come into effect on January 2022.

i) Assessment of Past Uses (similar to a Phase One Environmental Site Assessment)
ii) Sampling and Analysis Plan
iii) Soil Characterization Report
iv) Excess Soil Destination Assessment Report
v) Filing of Notice on Registry
vi) Soil Tracking

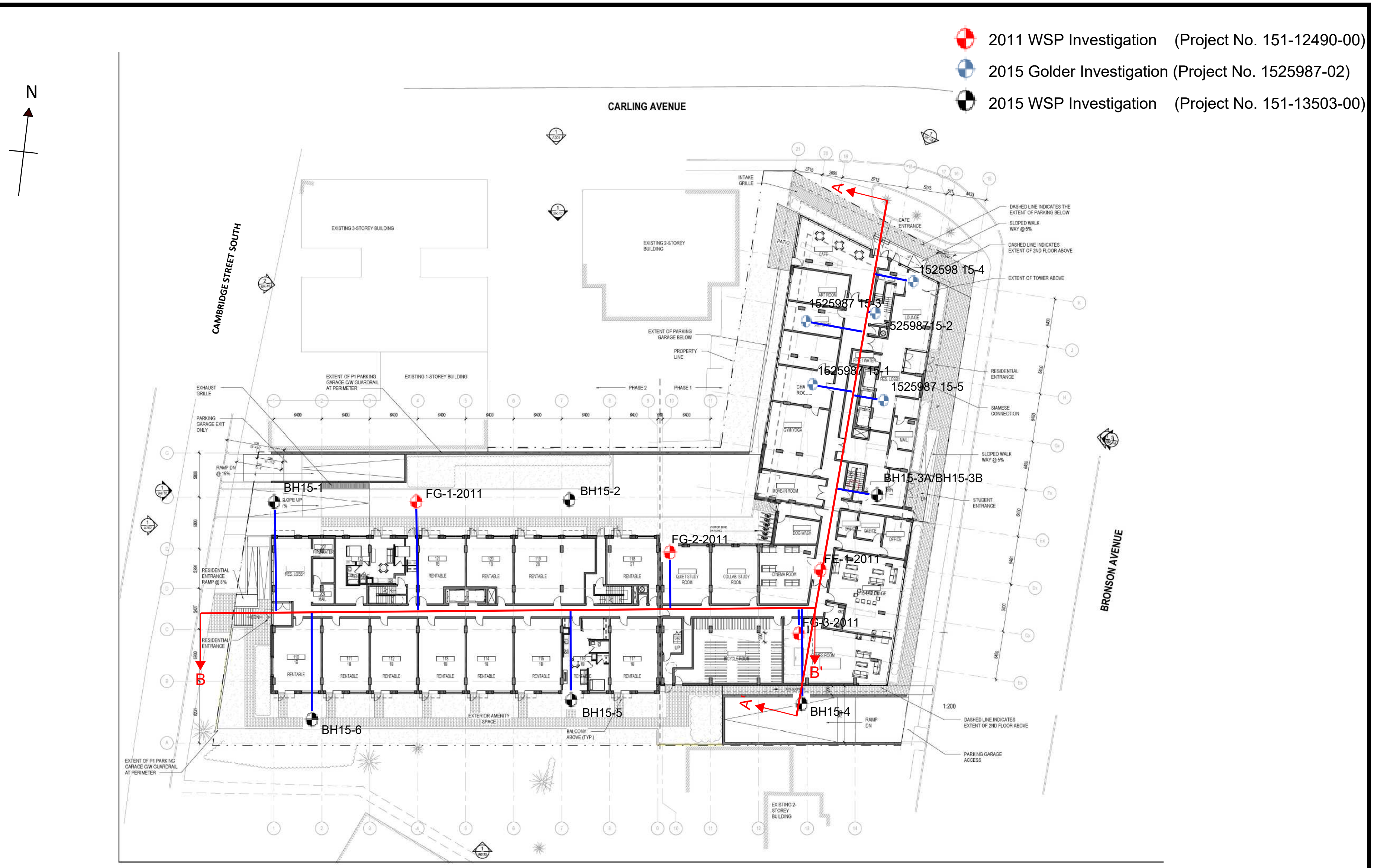
Although there are some exemptions, which would be determined at the Project onset including construction contracts signed before January 1, 2022 (Grandfathering Provision), for most earthworks projects, these planning requirements would be required for sites that generate greater than 2,000 m^3 of excess soil within a settlement area (e.g. built-up areas such as towns and cities) and/or sites that have potentially contaminating activities (PCAs) associated with them.

WSP can provide more guidance on the Excess Soil Regulation to ensure full compliance with the regulation, if requested.

FIGURES



Client:	Katasa Groupe + Développement			Title:	Site Location Plan
Project#:	211-05706-00	DWG #:	1	Project:	Geotechnical Investigation Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario
Drawn:	DW	Approved:	DW		
Date:	May 2021	Scale:	N. T. S.		
Size:	Letter	Rev:	0		



🔴 2011 WSP Investigation (Project No. 151-12490-00)

🔵 2015 Golder Investigation (Project No. 1525987-02)

⚫ 2015 WSP Investigation (Project No. 151-13503-00)

Client: Katasa Groupe + Développement

Project#: 211-05706-00

DWG #: 2

Drawn: DW

Approved: DW

Date: May 2021

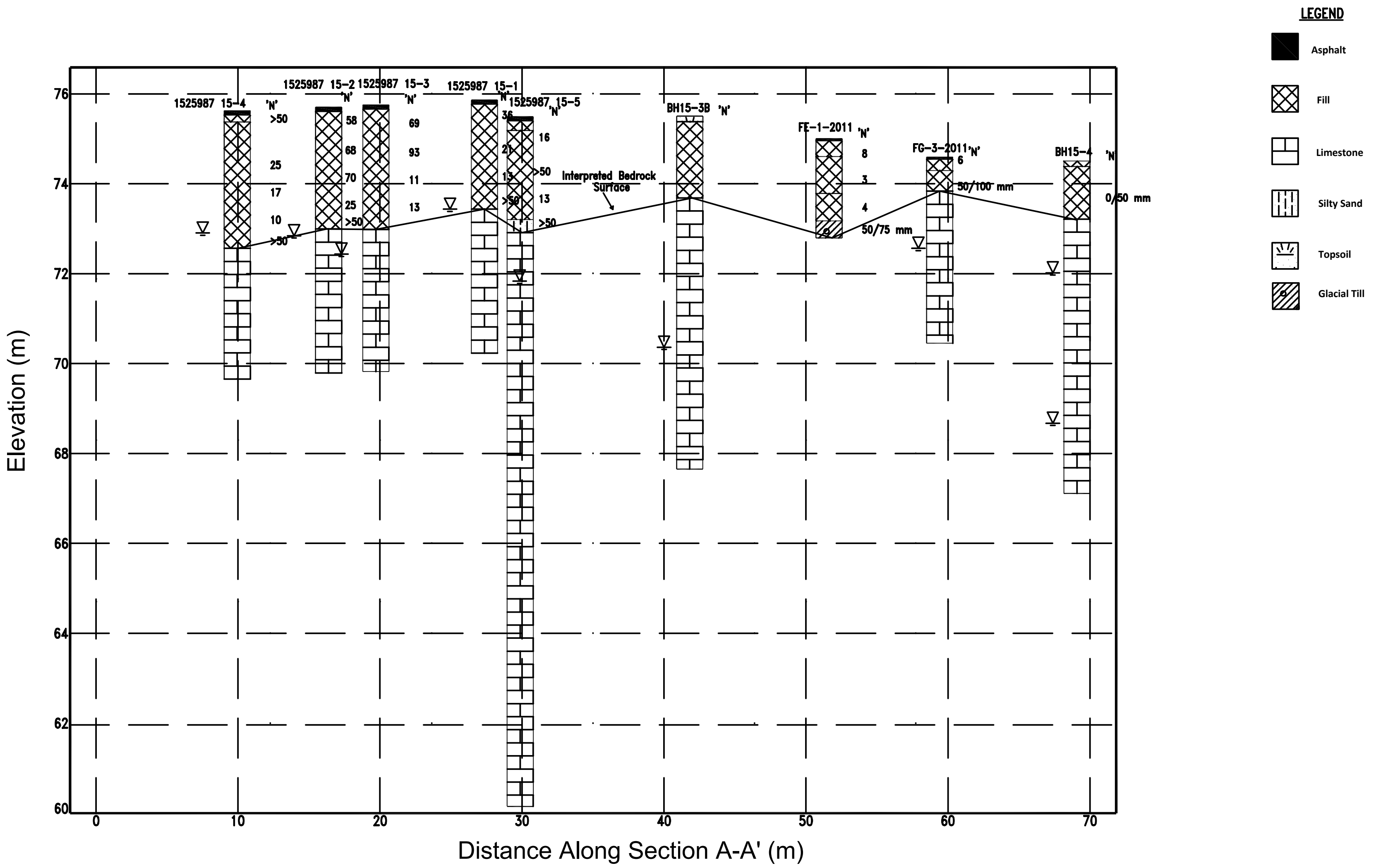
Scale: N. T. S.

Size: Letter

Rev: 0

Title: Borehole Location Plan

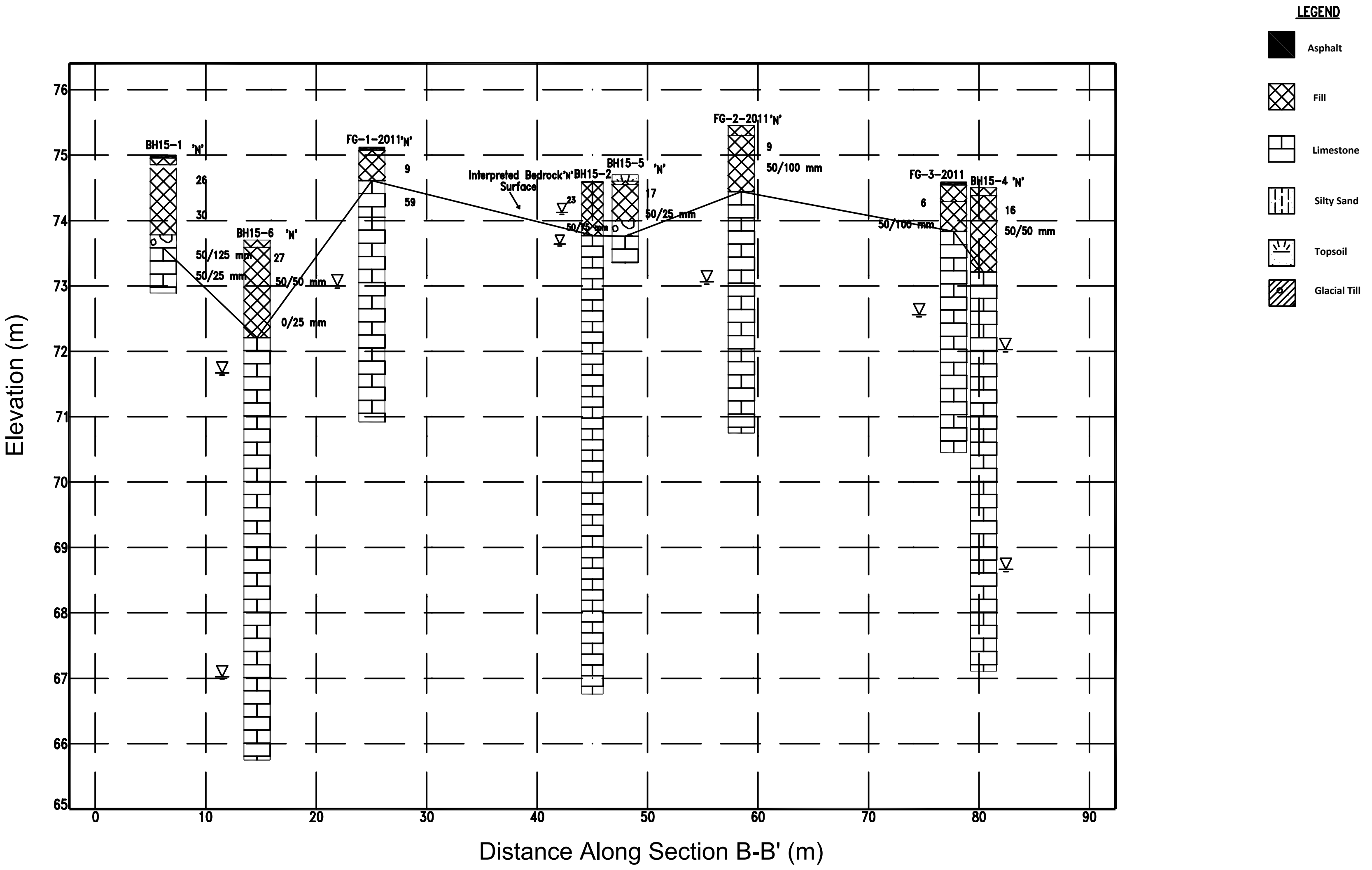
Project: Geotechnical Investigation
Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario



Client:	Katasa Groupe + Développement		
Project#:	211-05706-00	DWG #:	3A
Drawn:	DW	Approved:	DW
Date:	May 2021	Scale:	N. T. S.
Size:	Letter	Rev:	0

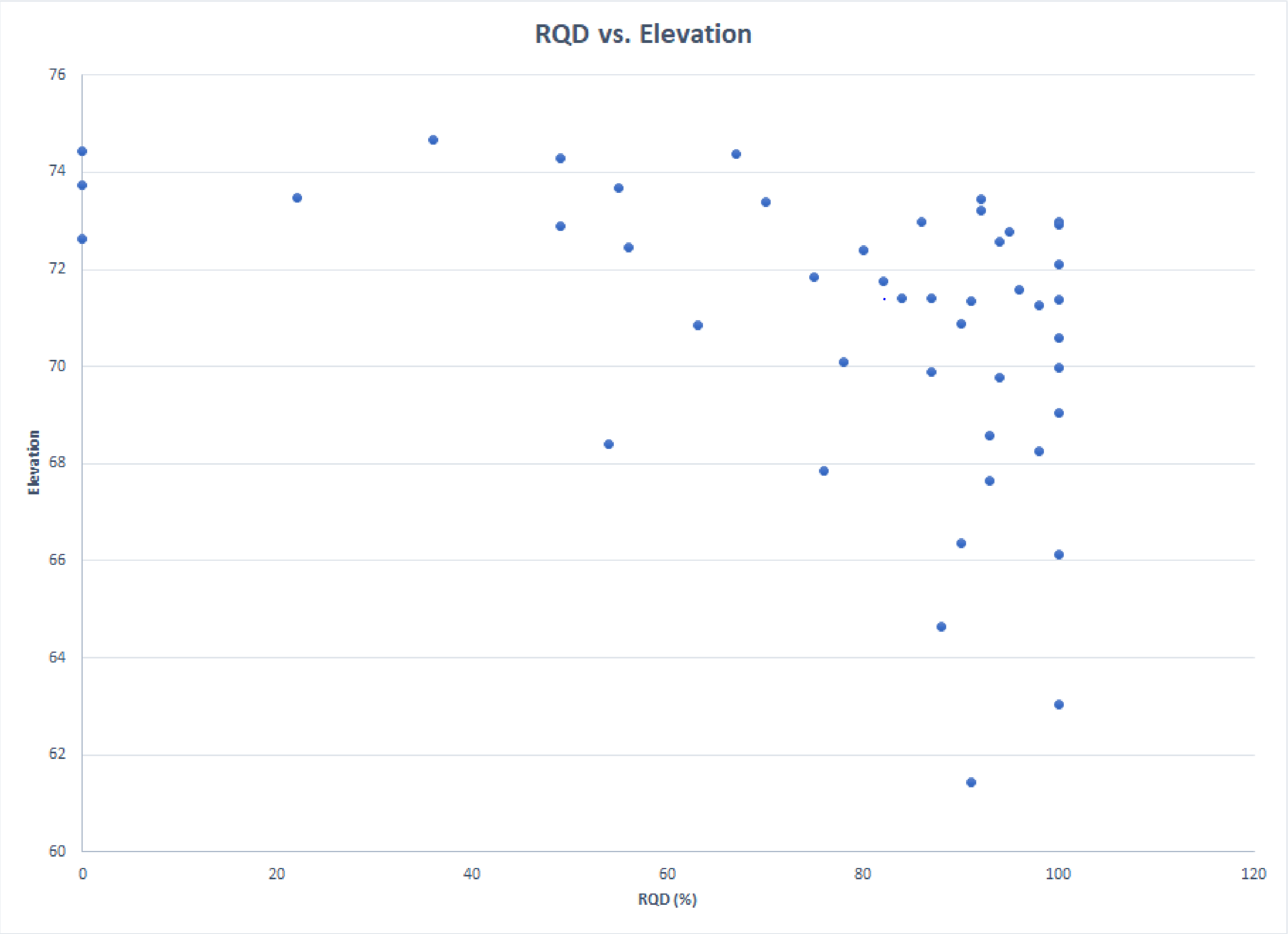
Title: Profile Along Section A-A'

Project: Geotechnical Investigation
Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario



Client:	Katasa Groupe + Développement	Title:	Profile Along Section B-B'
Project#:	211-05706-00	DWG #:	3B
Drawn:	DW	Approved:	DW
Date:	May 2021	Scale:	N. T. S.
Size:	Letter	Rev:	0

Project: Geotechnical Investigation
Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario



Client:	Katasa Groupe + Développement	Title:	RQD vs. Elevation
Project#:	211-05706-00	DWG #:	4
Drawn:	DW	Approved:	DW
Date:	May 2021	Scale:	N. T. S.
Size:	Letter	Rev:	0

Project: Geotechnical Investigation
Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario

APPENDIX

A

LIMITATIONS OF THIS REPORT

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Incorporated (WSP) at the time of preparation. Unless otherwise agreed in writing by WSP, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

APPENDIX

B

BOREHOLE LOGS – PREVIOUS INVESTIGATIONS

Updated Project Number: 151-12490-00

BOREHOLE DRILLING RECORD : **FE-1-2011**

Page 1 of 1

Prepared by: **David Feghali, ing.**
Reviewed by: **Pierre Jean**

Date (Start): **2011-12-09**
Date (End): **2011-12-09**

Project Name: **Geotechnical Investigation**
Site: **Projected building between Bronson Ave and Cambridge S. St.**
Sector: **Eastern Site Entrance**
Client: **Samcon Inc.**

Project Number: **111-26060-00**
Geographic Coordinates: X = 445239 W
Y = 5027674 N
Surface Elevation: 75 m *(Approximatif)*
Plunge / Azimuth: -90 deg / 0 deg

Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger
Borehole Diameter: 200 mm
Drilling Fluid: None

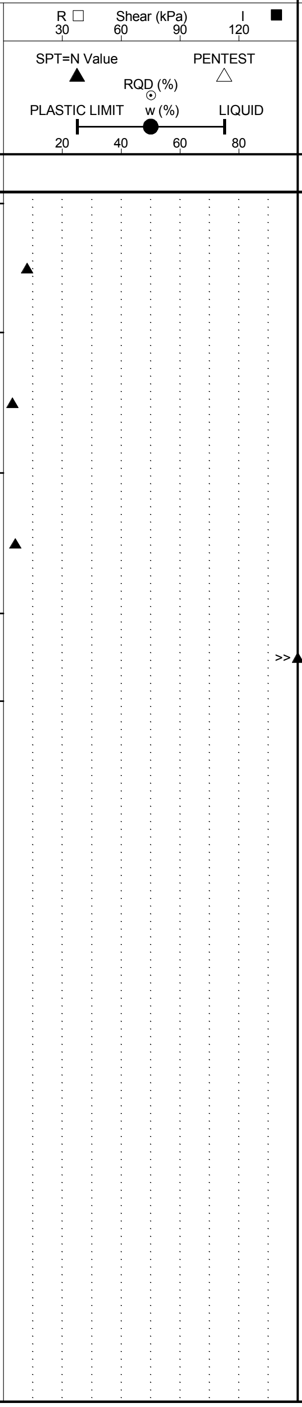
WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
⊻ Water Level ▼ Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
▨ Undisturbed
☒ Remoulded
■ Lost
▭ Cored

DEPTH ELEVATION (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
		Ground surface.							
0.05 *74.95*	■	Bituminous concrete.				MA-1	■	64	
0.40 *74.60*	▨	Fill: brown gravel and sand, trace to some silt. Loose compactness.				SS-2	☒		7 5 3 3 (8)
1.22 *73.78*	▨	Fill : brown fine sand, some silt, trace gravel. Very loose compactness.				SS-3	☒	83	2 2 1 1 (3)
1.83 *73.17*	▨	Glacial till (probable) : dark brown sandy silt, trace organic material, trace gravel. Very loose compactness.				SS-4	☒	33	1 2 2 3 (4)
2.21 *72.79*		Glacial till (probable): brown sandy and gravelly silt. Compact compactness. Split spoon refusal at 2.21 m.				SS-5	☒	92	7 13 50/8 cm (R)
		End of borehole at 2.21 m.							



WELL
DIAGRAM

Projet : ENGLISH LOG FORAGE SAMCON.GPJ Type rapport : WSP_EN_WELL-GEOTECHNICAL ONLY Data Template : WSP_TEMPLATE_GEOTECH.GDT 2015-11-12

Updated Project Number: 151-12490-00

WSP

BOREHOLE DRILLING RECORD : **FE-2-2011**

Page 1 of 1

Prepared by: **David Feghali, ing.**
Reviewed by: **Pierre Jean**

Date (Start): **2011-12-09**
Date (End): **2011-12-09**

Project Name: **Geotechnical Investigation**
Site: **Projected building between Bronson Ave and Cambridge S. St.**
Sector: **Southwest of Site**
Client: **Samcon Inc.**

Project Number: **111-26060-00**
Geographic Coordinates: X = 445236 W
Y = 5027625 N
Surface Elevation: 75 m *(Approximatif)*
Plunge / Azimuth: -90 deg / 0 deg

Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger
Borehole Diameter: 200 mm
Drilling Fluid: None

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
▽ Water Level ▼ Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
▨ Undisturbed
☒ Remoulded
■ Lost
◨ Cored

GEOLOGY / LITHOLOGY — ANALYSIS — GEOTECHNICAL — WELL

GEOTECHNICAL: R ☐ Shear (kPa) I ■ (30, 60, 90, 120); SPT=N Value ▲; PENTEST △; RQD (%) ⊙; PLASTIC LIMIT — w (%) ● — LIQUID (20, 40, 60, 80)

DEPTH *ELEVATION* (m)	DESCRIPTION
	Ground surface.
0.05 *74.95*	Bituminous concrete.
0.20	Fill: brown sand and gravel, trace to some silt. Compact compactness.
0.30 *74.70*	Red brick debris.
0.71 *74.29*	Fill: brown sand and gravel, trace red brick debris. Compact compactness.
0.88 *74.12*	Probable fill: dark brown sandy silt, trace organic material. Compact compactness. Split spoon refusal at 0.88 m.

End of borehole at 0.88 m.

TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	SPT=N Value
MA-1	70		
SS-2		5 7 7 12 (14)	▲
SS-3	81	9 50/12 cm	

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

Updated Project Number: 151-12490-00

WSP

BOREHOLE DRILLING RECORD : **FG-1-2011**

Page 1 of 1

Prepared by:	**David Feghali, ing.**	Date (Start):	**2011-12-08**
Reviewed by:	**Pierre Jean**	Date (End):	**2011-12-09**

Project Name:	**Geotechnical Investigation**
Site:	**Projected building between Bronson Ave and Cambridge S. St.**
Sector:	**West of Site**
Client:	**Samcon Inc.**
Project Number:	**111-26060-00**
Geographic Coordinates:	X = 445187 W Y = 5027657 N
Surface Elevation:	75.12 m *(Geodetic)*
Plunge / Azimuth:	-90 deg / 0 deg

Drilling Company:	Forage André Roy Inc.
Drilling Equipment:	CME 55
Drilling Method:	Auger / HQ Casing
Borehole Diameter:	200 mm / 96 mm
Drilling Fluid:	Water

WELL DETAILS

COPING	Elevation :	75.08 m
SCREEN	Bottom Depth :	4.2 m
	Length :	3.66 m
	Opening :	0.51 mm
WATER Elevation:		72.96 m
WATER Date:		2011-12-12

▽ Water Level ▼ Free Phase

SAMPLE TYPE

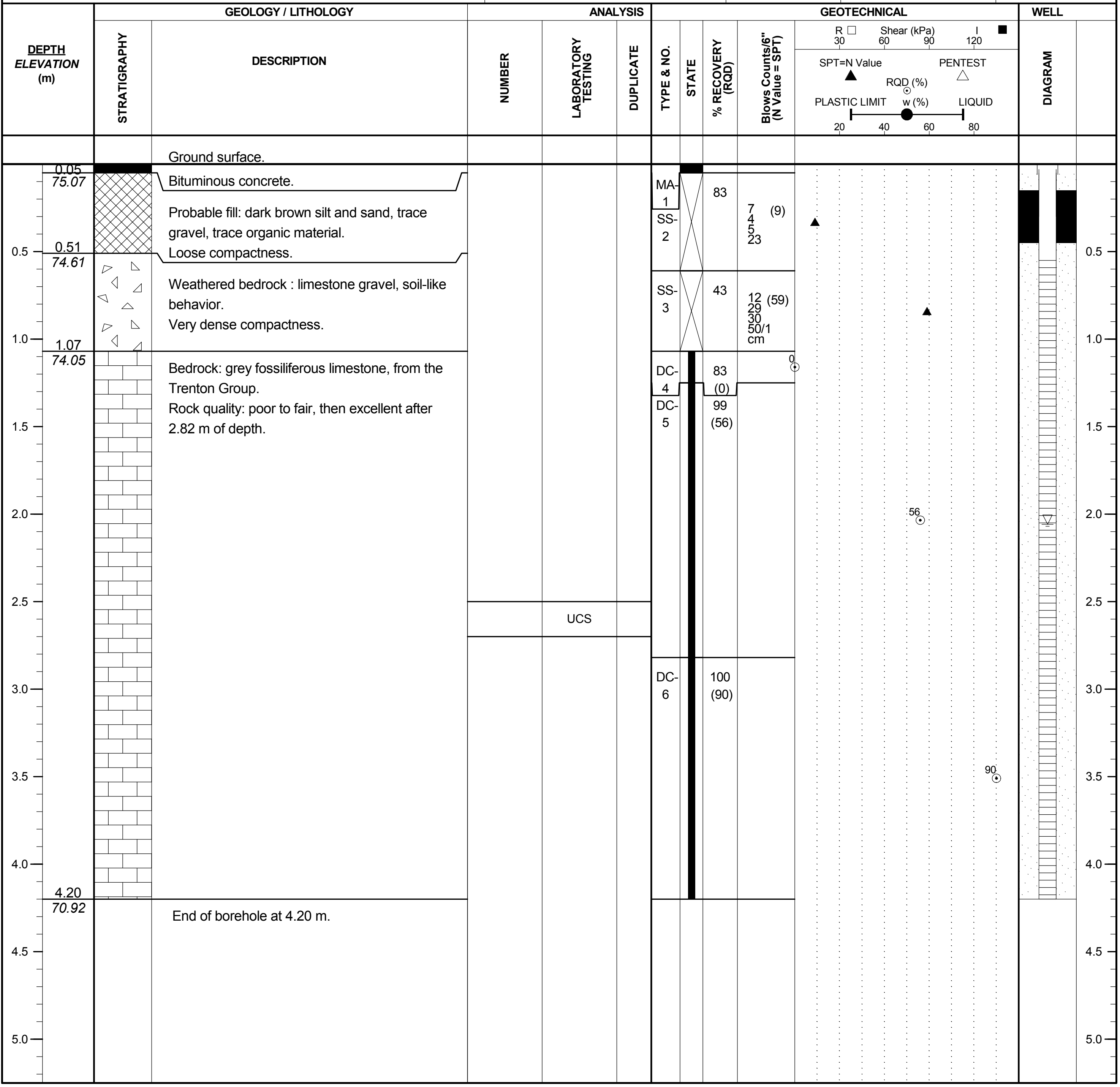
- DC - Diamond Core
- SS - Split Spoon
- PS - Piston Sample
- TC - Hollow Tube
- MA - Manual Auger
- TR - Trowel
- ST - Shelby Tube
- TT - DT-32 Liner

ANALYSIS

AL	- Atterberg Limits
GSA	- Grain Size Analysis
PENTEST	- Blow Counts/300mm
PL	- Point Load Test
Sg	- Specific Gravity
SPT	- N Value (Blow Counts/300mm)
UCS	- Uniaxial Compressive Strength
w	- Moisture Content
wL	- Liquidity Limit
wP	- Plasticity Limit

SAMPLE STATE

▨	Undisturbed
☒	Remoulded
■	Lost
◧	Cored



DEPTH / *ELEVATION* (m)	DESCRIPTION
	Ground surface.
0.05 / *75.07*	Bituminous concrete.
	Probable fill: dark brown silt and sand, trace gravel, trace organic material. Loose compactness.
0.51 / *74.61*	Weathered bedrock : limestone gravel, soil-like behavior. Very dense compactness.
1.07 / *74.05*	Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: poor to fair, then excellent after 2.82 m of depth.
4.20 / *70.92*	End of borehole at 4.20 m.

NUMBER	LABORATORY TESTING	DUPLICATE
	UCS	

TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
MA-1		83	
SS-2			7 4 5 23 (9)
SS-3		43	12 29 30 50/1 cm (59)
DC-4		83 (0)	
DC-5		99 (56)	
DC-6		100 (90)	

Updated Project Number: 151-12490-00

WSP

BOREHOLE DRILLING RECORD : **FG-2-2011**

Page 1 of 1

Prepared by: **David Feghali, ing.** Date (Start): **2011-12-08**
Reviewed by: **Pierre Jean** Date (End): **2011-12-08**

Project Name: **Geotechnical Investigation**
Site: **Projected building between Bronson Ave and Cambridge S. St.**
Sector: **Site Centre**
Client: **Samcon Inc.**

Project Number: **111-26060-00**
Geographic Coordinates: X = 445219 W
Y = 5027667 N
Surface Elevation: 75.45 m *(Geodetic)*
Plunge / Azimuth: -90 deg / 0 deg

Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Water

WELL DETAILS
COPING Elevation : 75.51 m
SCREEN Bottom Depth : 4.7 m
Length : 3.96 m
Opening : 0.51 mm
WATER Elevation: 73.12 m
WATER Date: 2011-12-12
▽ Water Level ▼ Free Phase

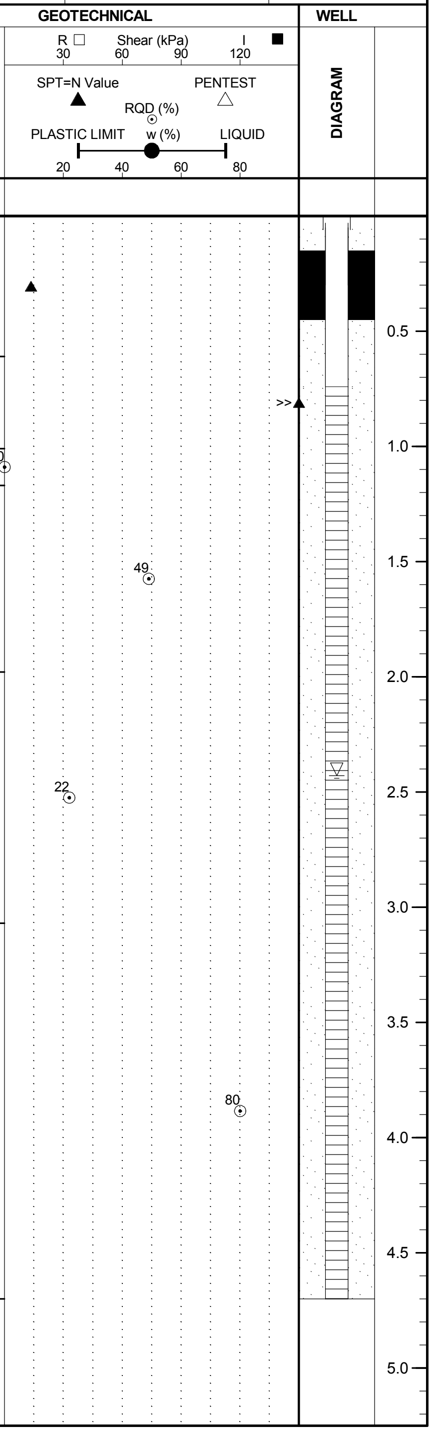
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
▨ Undisturbed
☒ Remoulded
■ Lost
◧ Cored

DEPTH ELEVATION (m)	DESCRIPTION	ANALYSIS: LABORATORY TESTING
	Ground surface.	
75.45 0.15 75.30	Fill: brown sand and gravel, trace to some silt. Loose compactness.	
	Probable fill: dark brown sandy silt, trace gravel, trace organic material. Very loose to loose compactness.	
1.01 74.44	Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: poor to very poor, then good after 3.07 m of depth.	UCS (approx. 3.9–4.2 m)
4.70 70.75	End of borehole at 4.70 m.	

TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
SS-1	82	9 5 4 3 (9)
SS-2	87	3 2 50/10 cm (R)
DC-3	81 (0)	
DC-4	81 (49)	
DC-5	73 (22)	
DC-6	96 (80)	



Projet : ENGLISH LOG FORAGE SAMCON.GPJ Type rapport : WSP_EN_WELL-GEOTECHNICAL ONLY Data Template : WSP_TEMPLATE_GEOTECH.GDT 2015-11-12

WSP

BOREHOLE DRILLING RECORD : **FG-3-2011**

Page 1 of 1

Prepared by: **David Feghali, ing.**
Reviewed by: **Pierre Jean**

Date (Start): **2011-12-09**
Date (End): **2011-12-09**

Project Name: **Geotechnical Investigation**
Site: **Projected building between Bronson Ave and Cambridge S. St.**
Sector: **East of Site**
Client: **Samcon Inc.**

Project Number: **111-26060-00**
Geographic Coordinates: X = 445240 W
Y = 5027666 N
Surface Elevation: 74.59 m *(Geodetic)*
Plunge / Azimuth: -90 deg / 0 deg

Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Water

WELL DETAILS
COPING Elevation : 74.69 m
SCREEN Bottom Depth : 4.14 m
Length : 3.66 m
Opening : 0.51 mm
WATER Elevation: 72.66 m
WATER Date: 2011-12-12
▽ Water Level ▼ Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
▨ Undisturbed
☒ Remoulded
■ Lost
◧ Cored

GEOTECHNICAL: R □ Shear (kPa) 30 60 90 120 I ■ ; SPT=N Value ▲ ; PENTEST △ ; RQD (%) ⊙ ; PLASTIC LIMIT w (%) ● LIQUID 20 40 60 80

DEPTH / *ELEVATION* (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	GEOTECHNICAL
	Ground surface.							
0.05 / *74.54*	Bituminous concrete.			MA-1	■	62		
0.30 / *74.29*	Fill: brown sand and gravel, trace to some silt. Loose compactness.			SS-2	☒		5 4 2 3 (6)	SPT ▲ 6
	Probable fill: dark brown sandy silt, trace to some gravel, trace organic material. Very loose compactness.			SS-3	☒	80	2 50 /10 cm	
0.76 / *73.83*	Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: very poor, then good after 1.22 m of depth.			DC-4	◧	56 (0)		RQD 0
				DC-5	◧	100 (70)		RQD 70
				DC-6	◧	100 (75)		RQD 75
			UCS					
4.14 / *70.45*	End of borehole at 4.14 m.							

WELL DIAGRAM: coping to 0.05 m, seal approx. 0.15–0.40 m, screen approx. 0.48–4.14 m, water level ▽ approx. 2.0 m.

PROJECT: 1525987

RECORD OF BOREHOLE: 15-1

SHEET 1 OF 2

LOCATION: N 5027695.2 ;E 445226.2

BORING DATE: March 25, 2015

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/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		75.86							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00							Flush Mount Casing
		FILL - (GW/SW) SAND and GRAVEL; grey brown; non-cohesive, moist, compact to very dense - Black staining from 0.25 m to 0.46 m	▨	0.10	1	SS	36				
1			▨		2	SS	21				
			▨		3	SS	13				
2			▨		4	SS	>50				Bentonite Seal ▽
		Borehole continued on RECORD OF DRILLHOLE 15-1		73.43 2.43							
3											
4											
5											
6											
7											
8											
9											
10											

MIS-BHS 001 1525987.GPJ GAL-MIS.GDT 08/21/15 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-1

LOCATION: N 5027695.2 ;E 445226.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 25, 2015

DRILL RIG:

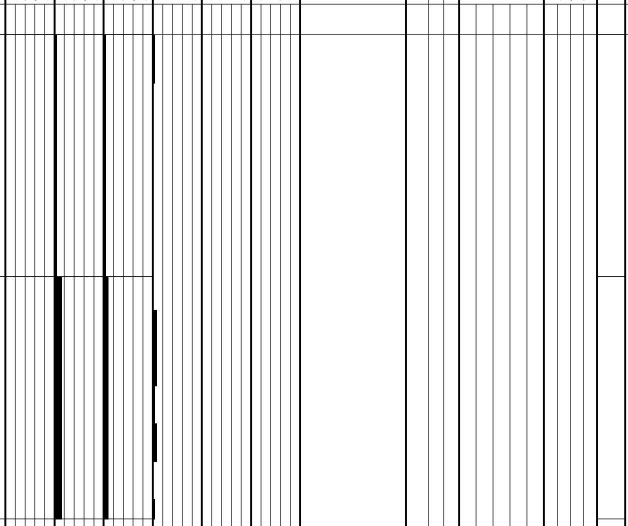
DRILLING CONTRACTOR: Marathon Drilling

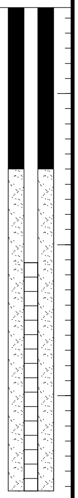
DATUM: Geodetic

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	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN / FLUSH
		BEDROCK SURFACE		73.43		
3 4 5	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale		2.43 70.23	1 2	
6 7 8 9 10 11 12		End of Drillhole		5.63		





Bentonite Seal

Silica Sand

38 mm Diam. PVC
#10 Slot Screen

W.L. in Screen at
Elev. 73.36 m on
March 27, 2015

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: JD
CHECKED: TMS

PROJECT: 1525987

RECORD OF BOREHOLE: 15-2

SHEET 1 OF 2

LOCATION: N 5027707.7 ;E 445228.3

BORING DATE: March 24, 2015

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/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		75.70							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							Flush Mount Casing
		FILL - (GW/SW) SAND and GRAVEL; grey brown; non-cohesive, moist, compact to very dense		0.10	1	SS	58				
1					2	SS	68				
					3	SS	70				
2					4	SS	25				Bentonite Seal
					5	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-2		72.99 2.71							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-2

SHEET 2 OF 2

LOCATION: N 5027707.7 ;E 445228.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 24, 2015

DRILL RIG:

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

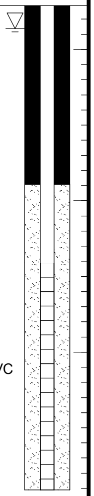
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

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	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	DISCONTINUITY DATA 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		72.99																
3	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale		2.71	1	100														Bentonite Seal
4																				Silica Sand
5					2	85														38 mm Diam. PVC #10 Slot Screen
				69.79																
6		End of Drillhole		5.91																W.L. in Screen at Elev. 72.79 m on March 27, 2015
7																				
8																				
9																				
10																				
11																				
12																				



DEPTH SCALE
1 : 50

LOGGED: JD
CHECKED: TMS

PROJECT: 1525987

RECORD OF BOREHOLE: 15-3

SHEET 1 OF 2

LOCATION: N 5027716.1 ;E 445220.9

BORING DATE: March 24, 2015

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/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		75.75							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							Flush Mount Casing
		FILL - (GW/SW) SAND and GRAVEL; grey brown; non-cohesive, moist, compact to very dense		0.10	1	SS	69				
1					2	SS	93				
					3	SS	11				
2					4	SS	13				Bentonite Seal
					5	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-3		72.98 2.77							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-3

SHEET 2 OF 2

LOCATION: N 5027716.1 ;E 445220.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 24, 2015

DRILL RIG:

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

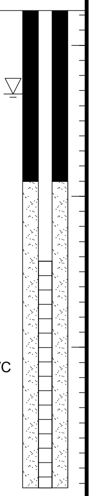
JN - Joint	BD- Bedding	PL - Planar	PO- Polished
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH
		BEDROCK SURFACE	72.98		
3	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale	2.77	1	85
4					
5				2	
			69.82 5.93		
6		End of Drillhole			
7					
8					
9					
10					
11					
12					

Column headings also printed: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.25 m (5 10 15 20); DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jcon; Jr; Ja); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.



Bentonite Seal

Silica Sand

38 mm Diam. PVC #10 Slot Screen

W.L. in Screen at Elev. 72.38 m on March 27, 2015

DEPTH SCALE

1 : 50

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF BOREHOLE: 15-4

SHEET 1 OF 2

LOCATION: N 5027715.5 ;E 445230.4

BORING DATE: March 24, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

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

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		75.62				
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00				Flush Mount Casing
		FILL - (SW) gravelly SAND; grey with dark grey staining; non-cohesive, moist, very dense	▨	0.10	1	SS	>50	
		FILL - (GW/SW) SAND and GRAVEL, trace silt; grey brown; non-cohesive, moist, compact to very dense	▨	0.25				
1					2	SS	25	
					3	SS	17	
2								Bentonite Seal
					4	SS	10	
					5	SS	>50	▽
3				72.56 3.06				
		Borehole continued on RECORD OF DRILLHOLE 15-4						
4								
5								
6								
7								
8								
9								
10								

MIS-BHS 001 1525987.GPJ GAL-MIS.GDT 08/21/15 JM

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: JD
CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-4

SHEET 2 OF 2

LOCATION: N 5027715.5 ;E 445230.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 24, 2015

DRILL RIG:

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

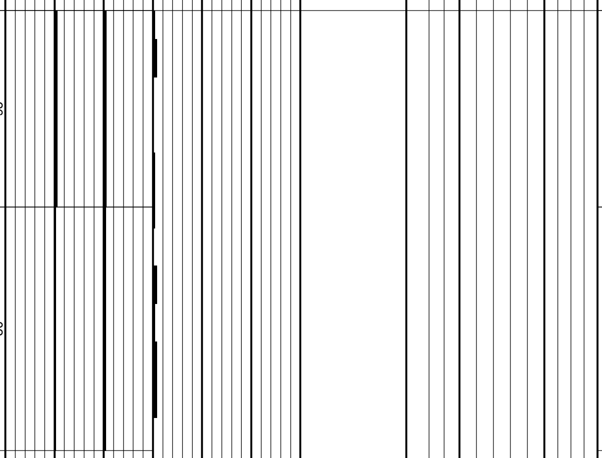
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

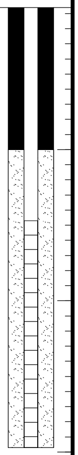
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR	FLUSH % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 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	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		72.56																	
	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings and thin interbeds of black shale		3.06	1		95														Bentonite Seal Silica Sand
4																					
5					2		80														38 mm Diam. PVC #10 Slot Screen
				69.65																	
6		End of Drillhole		5.97																	W.L. in Screen at Elev. 72.87 m on March 27, 2015
7																					
8																					
9																					
10																					
11																					
12																					
13																					





DEPTH SCALE

1 : 50

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF BOREHOLE: 15-5

SHEET 1 OF 3

LOCATION: N 5027698.4 ;E 445234.4

BORING DATE: June 19, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m				
		GROUND SURFACE		75.49							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00							Cement Grout
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)	▨	0.10							
		FILL - (SW) gravelly SAND; brown, contains cobbles; non-cohesive, moist, dense to very dense	▨	75.18 0.31	1	SS	61				
1			▨		2	SS	>50				
			▨								Peltonite and Cement Grout
			▨		3	SS	32				
2			▨								
		(SM) SILTY SAND, trace gravel; brown, contains organic matter; non-cohesive, wet, very dense		73.20 2.29	4	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-5		72.91 2.58							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

LOGGED: HEC

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-5

SHEET 2 OF 3

LOCATION: N 5027698.4 ;E 445234.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 19, 2015

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

BR - Broken Rock

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

Column headings: DEPTH SCALE METRES; DRILLING RECORD; DESCRIPTION; SYMBOLIC LOG; ELEV. DEPTH (m); RUN No.; FLUSH; COLOUR % RETURN; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.25 m (5 10 15 20); DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION, Jcon, Jr, Ja; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

Drilling record: Rotary Drill, NQ Core

DESCRIPTION	ELEV. DEPTH (m)
BEDROCK SURFACE	72.91
Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale	2.58
- Broken core from 10.85 m to 10.90 m	
- Broken core from 11.35 m to 11.38 m	

RUN No.	FLUSH % RETURN
1	100
2	100
3	100
4	100
5	100
6	100
7	100
8	

TYPE AND SURFACE DESCRIPTION (in order of depth):

- ,BD,PL,RO
- ,BD,PL,RO
- ,BD,PL,RO
- ,BD,PL,RO
- ,BD,CU,RO
- ,BD,IR,RO
- ,BD,PL,RO
- ,BD,PL,RO
- ,BD,PL,RO
- ,BD,PL,SM
- ,BD,PL,SM
- ,BD,PL,RO
- ,BD,PL,RO
- ,BD,IR,RO

Depth scale: 3, 4, 5, 6, 7, 8, 9, 10, 11, 12

Peltonite and Cement Grout

∇

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

LOGGED: HEC

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-5

SHEET 3 OF 3

LOCATION: N 5027698.4 ;E 445234.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 19, 2015

DRILL RIG: CME 55

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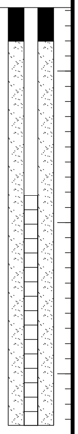
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

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.	COLOUR / % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		--- CONTINUED FROM PREVIOUS PAGE ---																		
13	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale			8	100														Silica Sand
14					9	100						● ●	,BD,PL,RO ,BD,PL,RO							
												● ●	,BD,PL,RO ,BD,PL,RO							50 mm Diam. PVC #10 Slot Screen
15				60.15 15.34								●	,BD,PL,RO							
		End of Drillhole																		W.L. in Screen at Elev. 71.83 m on August 20, 2015
16																				
17																				
18																				
19																				
20																				
21																				
22																				



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: TMS

BOREHOLE DRILLING RECORD : **BH15-1**

Page 1 of 1

WSP

Prepared by:	**Kathryn Maton**	Date (Start):	**1/11/2016**
Reviewed by:	**Phil Romeril**	Date (End):	**1/11/2016**

Project Name:	**Phase Two Environmental Site Assessment**	Project Number:	**151-13503-00**
Site:	**774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**	Geographic Coordinates:	X = 445171 mE Y = 5027646 mN
Sector:			
Client:	**Textbook Student Suites**	Surface Elevation:	75 m *(Approximate)*
		Top of PVC Elevation:	

Drilling Company:	Downing Estate Drilling Ltd.
Drilling Equipment:	CME 55
Drilling Method:	Auger
Borehole Diameter:	200 mm
Drilling Fluid:	None
Sampling Method:	Split Spoon

ODOUR
- F - Light
- M - Medium
- P - Persistent

VISUAL
- D - Disseminated Product
- S - Saturated with Product

SAMPLE TYPE
- DC - Diamond Corer
- SS - Split Spoon
- MA - Manual Auger
- TR - Trowel
- ST - Shelby Tube
- TU - DT32 Liner

▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls	MAH	Monocyclic Aromatic Hydrocarbons
BTEX	Benzene, Toluene, Ethylbenzene, Xylene	PAH	Polycyclic Aromatic Hydrocarbons
Inorg. C.	Inorganic Compounds	PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
Phenol. C.	Phenolic Compounds	PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
VOC	Volatil Organic Compounds (MAH & CAH)	Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
Diox. & Fur.	Dioxins & Furans	HWR	Leacheate Tests (Haz. Waste Reg.)
CAH	Chlorinated Aliphatic Hydrocarbons		

	GEOLOGY / LITHOLOGY		OBSERVATIONS			SAMPLES						MONITORING WELL		
DEPTH *ELEVATION* (m)	LITHOLOGY	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR F M P	VISUAL D S	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	DIAGRAM	DESCRIPTION	REMARKS
		Ground surface.												
0.05 *74.95*		ASPHALT												
0.15 *74.85*		FILL, crushed limestone gravel and sand	1.6						BH15-1 1					
		FILL, sand and crushed limestone gravel, with some asphalt and pieces of brick, compact, dry, grey	1.2			SS	40	12 14 7	BH15-1 2	PHCs F1-F4 BTEX PAH VOC				
0.5														0.5
			1.3			SS	50	5 5 25 30	BH15-1 3					
1.0														1.0
1.22 *73.78*		GRAVEL, shale fragments	2.1			SS	37	39 50 50-5"	BH15-1 4					
1.42 1.44 *73.56*		BEDROCK, shale	6.1						BH15-1 5					
1.5		BEDROCK, limestone with black shale partings												1.5
						SS	0	50-1"						
2.0														2.0
2.11 *72.89*		*Auger Refusal at 2.11 mbgs*												
		End of borehole at 2.11 m.												
2.5														2.5
3.0														3.0

BOREHOLE DRILLING RECORD : **BH15-2**

Page 1 of 1

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/13/2016**

Project Name:	**Phase Two Environmental Site Assessment**
Site:	**774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:	
Client:	**Textbook Student Suites**
Project Number:	**151-13503-00**
Geographic Coordinates:	X = 445204 mE Y = 5027668 mN
Surface Elevation:	75.6 m *(Approximate)*
Top of PVC Elevation:	76.62 m *(Approximate)*

Drilling Company:	Downing Estate Drilling Ltd.
Drilling Equipment:	CME 55
Drilling Method:	Auger / HQ Casing
Borehole Diameter:	200 mm / 96 mm
Drilling Fluid:	Municipal Water
Sampling Method:	Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

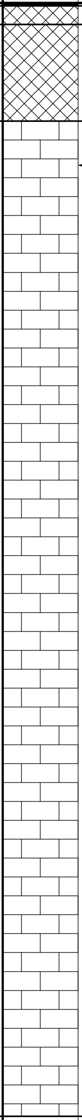
SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

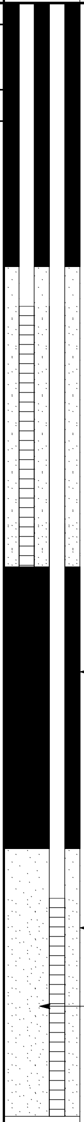
▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls	MAH	Monocyclic Aromatic Hydrocarbons
BTEX	Benzene, Toluene, Ethylbenzene, Xylene	PAH	Polycyclic Aromatic Hydrocarbons
Inorg. C.	Inorganic Compounds	PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
Phenol. C.	Phenolic Compounds	PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
VOC	Volatil Organic Compounds (MAH & CAH)	Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
Diox. & Fur.	Dioxins & Furans	HWR	Leacheate Tests (Haz. Waste Reg.)
CAH	Chlorinated Aliphatic Hydrocarbons		

DEPTH *ELEVATION* (m)	GEOLOGY / LITHOLOGY DESCRIPTION	OBSERVATIONS VAPOR CONC. (ppm OR % LIE)	ODOUR F M P	VISUAL D S	SAMPLES SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	REMARKS
	Ground surface.										
0.02 0.15 *75.45*	ASPHALT										
	FILL, crushed limestone gravel and sand	8.5			SS	18	7 11 12 14	BH15-2 1			
0.83 *74.77*	FILL, sandy silt and crushed limestone gravel with trace clay, compact , saturated, brown, black crystal material observed	13.8			SS	12	6 50-3"	BH15-2 2	PHCs F1-F4 VOC PAHs BTEX		
	BEDROCK, limestone with black shale partings *Auger Refusal at 1.14 mbgs, HQ Coring begins*				DC	100					RQD = 36
					DC	100					RQD = 67
					DC	100					RQD = 49
					DC	100					RQD = 87
					DC	100					RQD = 87
					DC	100					RQD = 54
7.82 *67.78*	End of borehole at 7.82 m.										





MONITORING WELL DESCRIPTION

SCREEN
Diam.: 31 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: m
Elev.: 74.12 m
Date: 1/19/2016

SCREEN
Diam.: 31 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: m
Elev.: 73.63 m
Date: 1/19/2016

WSP

BOREHOLE DRILLING RECORD : **BH15-3A**

Page 1 of 1

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/12/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445240 mE
Y = 5027685 mN
Surface Elevation: 75.5 m *(Approximate)*
Top of PVC Elevation: 76.53 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

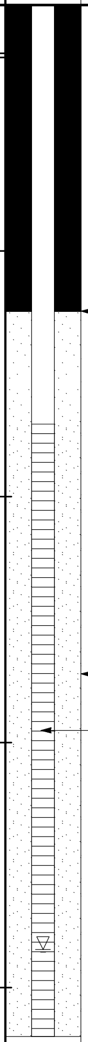
▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls	MAH	Monocyclic Aromatic Hydrocarbons
BTEX	Benzene, Toluene, Ethylbenzene, Xylene	PAH	Polycyclic Aromatic Hydrocarbons
Inorg. C.	Inorganic Compounds	PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
Phenol. C.	Phenolic Compounds	PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
VOC	Volatil Organic Compounds (MAH & CAH)	Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
Diox. & Fur.	Dioxins & Furans	HWR	Leacheate Tests (Haz. Waste Reg.)
CAH	Chlorinated Aliphatic Hydrocarbons		

DEPTH *ELEVATION* (m)	GEOLOGY / LITHOLOGY DESCRIPTION	OBSERVATIONS VAPOR CONC. (ppm OR % LIE)	ODOUR F M P	VISUAL D S	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	REMARKS
	Ground surface.										
75.50	TOPSOIL	15.2			SS	37	11 8 6 5	BH15-3A 1			
0.12 0.13 *75.37*	FILL, Black carbon ashes										
	FILL, silty sand, dry to moist, compact, brown	12						BH15-3A 2			0.5
		12			SS	12	2 14 50-3"	BH15-3A 3			1.0
	becoming saturated	13.1			SS	64	7 14 22 31	BH15-3A 4	PHCs F1-F4 BTEX VOC PAH Duplicate		1.5
1.75 1.82 *73.68*	FILL, crushed limestone gravel and sand										
	FILL, silty sand and crushed limestone gravel, saturated, compact, brown	21.3			SS	29	37 35 50-2"	BH15-3A 5			2.0
2.18 *73.32*	BEDROCK, limestone with black shale partings										2.5
2.56 *72.94*	*Auger Refusal at 2.56 mbgs*										
	End of borehole at 2.56 m.										3.0

MONITORING WELL



SCREEN

Diam.: 51 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: 2.35 m
Elev.: 74.18 m
Date: 1/19/2016

BOREHOLE DRILLING RECORD : BH15-3B

Page 1 of 1

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/12/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445240 mE
Y = 5027685 mN
Surface Elevation: 75.5 m *(Approximate)*
Top of PVC Elevation: 77.468 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Diamond Corer

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

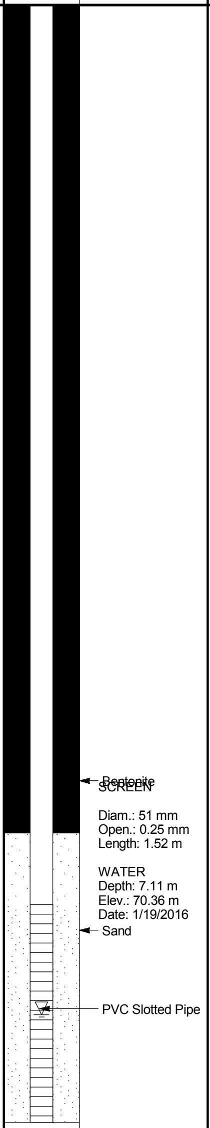
SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls	MAH	Monocyclic Aromatic Hydrocarbons
BTEX	Benzene, Toluene, Ethylbenzene, Xylene	PAH	Polycyclic Aromatic Hydrocarbons
		PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
Inorg. C.	Inorganic Compounds	PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Phenol. C.	Phenolic Compounds	Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
VOC	Volatil Organic Compounds (MAH & CAH)		
Diox. & Fur.	Dioxins & Furans	HWR	Leacheate Tests (Haz. Waste Reg.)
CAH	Chlorinated Aliphatic Hydrocarbons		

DEPTH ELEVATION (m)	GEOLOGY / LITHOLOGY: DESCRIPTION	OBSERVATIONS: VAPOR CONC. (ppm OR % LIE)	ODOUR F M P	VISUAL D S	SAMPLES: SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL: DESCRIPTION	REMARKS
	Ground surface.											
75.50	FILL, see soil description on BH15-3A											
	Casing refusal at 1.8 mbgs, HQ coring begins				DC	100						RQD = 55
2.18 73.32	BEDROCK, limestone with black shale partings											
					DC	100						RQD = 92
					DC	100					Bentonite	RQD = 98
					DC	100					SCREEN Diam.: 51 mm Open.: 0.25 mm Length: 1.52 m WATER Depth: 7.11 m Elev.: 70.36 m Date: 1/19/2016 Sand PVC Slotted Pipe	RQD = 94
					DC	100						RQD = 98
7.85 67.65	End of borehole at 7.85 m.											



BOREHOLE DRILLING RECORD : **BH15-4**

Page 1 of 1

WSP

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/13/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445246 mE
Y = 5027658 mN
Surface Elevation: 74.5 m *(Approximate)*
Top of PVC Elevation: 75.53 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

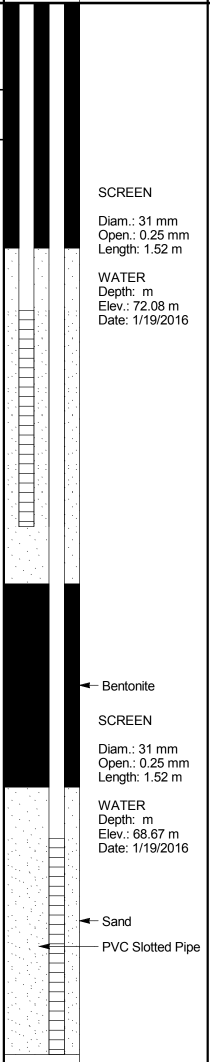
▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls
BTEX	Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C.	Inorganic Compounds
Phenol. C.	Phenolic Compounds
VOC	Volatil Organic Compounds (MAH & CAH)
Diox. & Fur.	Dioxins & Furans
CAH	Chlorinated Aliphatic Hydrocarbons
MAH	Monocyclic Aromatic Hydrocarbons
PAH	Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR	Leacheate Tests (Haz. Waste Reg.)

DEPTH ELEVATION (m)	GEOLOGY / LITHOLOGY DESCRIPTION	OBSERVATIONS VAPOR CONC. (ppm OR % LIE)	ODOUR F M P	VISUAL D S	SAMPLES SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	REMARKS
	Ground surface.										
0.12 *74.38*	FILL, crushed limestone gravel	11.9			SS	37	8 8 8 6	BH15-4 1	Metals and Inorganics		
	FILL, silty sand and crushed limestone gravel	13			SS	46	4 6 50-2"	BH15-4 2			
0.96 *73.54*	BEDROCK, limestone with black shale partings *Auger Refusal at 1.29 mbgs, HQ coring begins*				DC	100					RQD = 92
					DC	100					RQD = 96
					DC	100					RQD = 78
					DC	100					RQD = 93
7.39 *67.09*	End of borehole at 7.41 m.										

MONITORING WELL



SCREEN

Diam.: 31 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: m
Elev.: 72.08 m
Date: 1/19/2016

SCREEN

Diam.: 31 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: m
Elev.: 68.67 m
Date: 1/19/2016

Projet : PHASE II ESA - 774 BRONSON AVE.GPJ Type rapport : WSP_EN_WELL-ENVIRONMENTAL Data Template : WSP_TEMPLATE_GEOTECH.GDT 2/9/2016

BOREHOLE DRILLING RECORD : **BH15-5**

Page 1 of 1

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/11/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445217 mE
Y = 5027643 mN
Surface Elevation: 74.7 m *(Approximate)*
Top of PVC Elevation:

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger
Borehole Diameter: 200 mm
Drilling Fluid: None
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls	MAH	Monocyclic Aromatic Hydrocarbons
BTEX	Benzene, Toluene, Ethylbenzene, Xylene	PAH	Polycyclic Aromatic Hydrocarbons
		PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
Inorg. C.	Inorganic Compounds	PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Phenol. C.	Phenolic Compounds	Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
VOC	Volatil Organic Compounds (MAH & CAH)		
Diox. & Fur.	Dioxins & Furans	HWR	Leacheate Tests (Haz. Waste Reg.)
CAH	Chlorinated Aliphatic Hydrocarbons		

DEPTH ELEVATION (m)	GEOLOGY / LITHOLOGY		OBSERVATIONS						SAMPLES						MONITORING WELL		REMARKS
	LITHOLOGY	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR F	ODOUR M	ODOUR P	VISUAL D	VISUAL S	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	DIAGRAM	DESCRIPTION	
		Ground surface.															
74.70 0.15		TOP SOIL	8						SS	50	2 10 7 4	BH15-5-1 1					
0.17 74.53		FILL, pieces of asphalt	1.3									BH15-5-2 (0.15-0.17)	Metals and Inorganics				
0.5 0.61		FILL, sand and crushed limestone gravel with trace pieces of brick	1.2									BH15-5-3 2					0.5
74.09 0.71		FILL, topsoil with some pieces of wood, compact, moist, dark brown	1.3						SS	41	4 32 50-1"	BH15-5-4 3					
73.99 0.94		GRAVEL and sand	1									BH15-5-5 4					
73.76 1.0 1.35		BEDROCK, limestone with black shale partings															1.0
73.35		*Auger Refusal at 1.35 mbgs* End of borehole at 1.35 m.															
1.5																	1.5
2.0																	2.0
2.5																	2.5
3.0																	3.0

BOREHOLE DRILLING RECORD : **BH15-6**

Page 1 of 1

WSP

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/13/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445189 mE
Y = 5027623 mN
Surface Elevation: 73.7 m *(Approximate)*
Top of PVC Elevation: 74.705 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

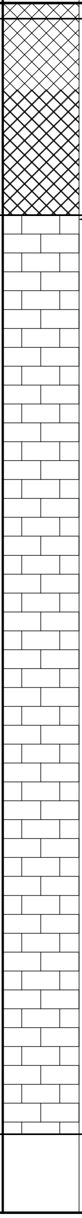
VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

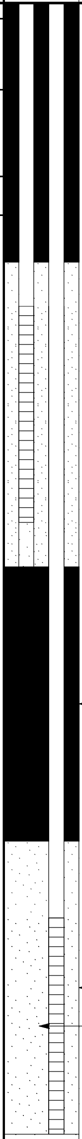
▽ Water Level ▼ Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls
BTEX	Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C.	Inorganic Compounds
Phenol. C.	Phenolic Compounds
VOC	Volatil Organic Compounds (MAH & CAH)
Diox. & Fur.	Dioxins & Furans
CAH	Chlorinated Aliphatic Hydrocarbons
MAH	Monocyclic Aromatic Hydrocarbons
PAH	Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR	Leacheate Tests (Haz. Waste Reg.)



DEPTH ELEVATION (m)	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR F M P	VISUAL D S	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL DESCRIPTION	REMARKS
	Ground surface.											
0.11 73.59	FILL, top soil, with some pieces of brick, dry, compact, dark brown	11.1 13.8			SS	39	3 14 13 8	BH15-6 1 BH15-6 2	Metals and Inorganics Duplicate			
73.09	FILL, crushed limestone gravel and sand, dry, compact, brown-grey becoming silty with trace pieces of brick, saturated, brown	12.1			SS	25	14 50-2"	BH15-6 3				
					DC	27					SCREEN Diam.: 31 mm Open.: 0.25 mm Length: 1.52 m	
1.49 72.21	*Auger Refusal at 1.52 mbgs, HQ Coring begins*	11.9			SS	16	11 50-1"	BH15-6 4				RQD = 63
					DC	92					WATER Depth: m Elev.: 71.67 m Date: 1/19/2016	
	BEDROCK, limestone and black shale partings				DC	104						RQD = 100
					DC	95						RQD = 76
					DC	105					SCREEN Diam.: 31 mm Open.: 0.25 mm Length: 1.52 m WATER Depth: m Elev.: 67.05 m Date: 1/19/2016	RQD = 90
					DC	100						RQD = 75
7.95 65.75	End of borehole at 7.95 m.											



APPENDIX

C

LABORATORY TESTING RESULTS – PREVIOUS INVESTIGATION

Labo S.M. inc.
Une division du Groupe S.M. inc

■ 740, rue Galt Ouest, 2e étage, Sherbrooke (Qc) J1H 1Z3 Tél: (819) 566-8855 Fax: (819) 566-0224
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Rapport n° 1106941

RAPPORT D'ESSAIS
MESURE DE LA RÉSISTANCE EN COMPRESSION SUR CAROTTES DE ROC
ASTM D 7012-07

Numéro de dossier : F115220001
Numéro de laboratoire : 11-10906/11-10908/11-10909
Projet : Étude géotechnique - Reconstruction des conduites d'eau et d'égou
Client : Génivar - Gatineau

Conditionnement : sec
Matériau de coiffe : meule
Température de confinement : 22
Prélevé par : nd ,le
Réalisé par : D. Laroche ,le 11-12-15
Site :
Contrat :

Date rupturée	Forage N°	# échant.	Profondeur d'essais (m)	Diamètre				Longueur		Rapport L/D	Charge	Résistance en compression	Temps de rupture
				1	2	3	moyen	initiale	meulée				
				(mm)				(mm)			(kN)	(MPa)	(sec)
11-12-15	FG-1-2011	11-10906	2,5 à 2,7 m	62,92	62,96	62,85	62,91		142,71	2,27	338,2	**108,8**	370
11-12-15	FG-2-2011	11-10909	3,9 à 4,2 m	62,82	62,76	62,70	62,76		147,02	2,34	229,2	**74,1**	276
11-12-15	FG-3-2011	11-10908	3,3 à 3,7 m	62,96	62,95	62,95	62,95		146,08	2,32	397,2	**127,6**	434

L/D: Rapport Longueur/Diamètre

Remarques:

Préparé par: Sylvie Daigle, tech. Chef Labo Date: 11-12-19 Vérifié par: Éric Ouimet, ing. Date: 11-12-19

lotes : Le résultat s'applique exclusivement à l'échantillon analysé.

Ce rapport ne doit pas être reproduit, sinon en entier, sans l'autorisation écrite de Labo S.M. inc.

APPENDIX

D

CEHMCIAL ANALYSIS RESULTS – PREVIOUS INVESTIGATION

Certificate of Analysis

Golder Associates Ltd. (Ottawa)
1931 Robertson Rd.
Ottawa, ON K2H 5B7
Attn: Keith Holmes

Phone: (613) 592-9600
Fax: (613) 592-9601

Client PO:
Project: 1525987
Custody: 105457

Report Date: 10-Jul-2015
Order Date: 8-Jul-2015
Order #: 1528298

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

Paracel ID	Client ID
1528298-01	15-3 (2)

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 10-Jul-2015
Client: **Golder Associates Ltd. (Ottawa)**
Order Date:8-Jul-2015
Client PO:
Project Description: 1525987

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC	8-Jul-15	8-Jul-15
Conductivity	EPA 9050A- probe @25 °C	9-Jul-15	9-Jul-15
pH	EPA 150.1 - pH probe @25 °C	9-Jul-15	9-Jul-15

Certificate of Analysis

Report Date: 10-Jul-2015
Order Date:8-Jul-2015

Client: **Golder Associates Ltd. (Ottawa)**
Client PO:
Project Description: 1525987

	Client ID:	15-3 (2)	-	-	-
	Sample Date:	06-Jul-15	-	-	-
	Sample ID:	1528298-01	-	-	-
	MDL/Units	Water	-	-	-
General Inorganics					
Conductivity	5 uS/cm	5730	-	-	-
pH	0.1 pH Units	7.2	-	-	-
Anions					
Chloride	1 mg/L	1800	-	-	-
Sulphate	1 mg/L	141	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	1	mg/L						
Sulphate	ND	1	mg/L						
General Inorganics									
Conductivity	ND	5	uS/cm						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	1	mg/L	1.02			0.0	10	
Sulphate	22.5	1	mg/L	22.4			0.4	10	
General Inorganics									
Conductivity	687	5	uS/cm	695			1.1	11	
pH	8.4	0.1	pH Units	8.4			0.2	10	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	10.6	1	mg/L	1.02	96.1	78-112			
Sulphate	31.0	1	mg/L	22.4	86.0	75-111			

Qualifier Notes:

Login Qualifiers :

Sample - Not submitted in the correct container - Submitted in amber glass container, instead of plastic.
Applies to samples: 15-3 (2)

Sample not received in Paracel verified container / media
Applies to samples: 15-3 (2)

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

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

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www.paracellabs.com

Chain of Custody
(Lab Use Only)
№ 105457

Page 1 of 1

Client Name: Golder

Contact Name: Keith Holmes

Address:

Telephone: 592 -9600

Project Reference: 1525987

Quote #

PO #

Email Address:

TAT: ☒ Regular ☐ 3 Day
☐ 2 Day ☐ 1 Day

Date Required:

Criteria: ☐ O. Reg. 153/04 (As Amended) Table __ ☐ RSC Filing ☐ O. Reg. 558/00 ☐ PWQO ☐ CCME ☐ SUB (Storm) ☐ SUB (Sanitary) Municipality: ______ ☐ Other: N/A

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

Required Analyses

Paracel Order Number: 1528298

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	Conductivity	Sulphate	Chloride	pH			
1	15-3 (2)	GW		1	07/07/15									X	X	X	X			
2																				
3																				
4																				
5																				
6																				
7																				
8																				
9																				
10																				

Comments: Proceed with non-Paracel bottle

Method of Delivery: walk-in

Relinquished By (Sign):
Relinquished By (Print): Keith Holmes
Date/Time: 8/July/15 11:40

Received by Driver/Depot: Karen Gill
Date/Time: July 8/15 11:41
Temperature: 7.8 °C

Received at Lab: SUNEEPORN DOKMAI
Date/Time: JUL08,2015 01:14
Temperature: 10.3 °C

Verified By: D. Charlebois
Date/Time: July 8 1:39
pH Verified ☒ By: N/A

APPENDIX

E

GEOPHYSICAL VERTICAL SEISMIC PROFILING TEST RESULTS – PREVIOUS INVESTIGATION

Golder Associates

TECHNICAL MEMORANDUM

DATE July 10, 2015 **PROJECT No.** 152987

TO Troy Skinner, P.Eng.
Golder Associates Ltd.

CC Keith Holmes

FROM Patrick Finlay and
Christopher Phillips

EMAIL pfinlay@golder.com;
cphillips@golder.com

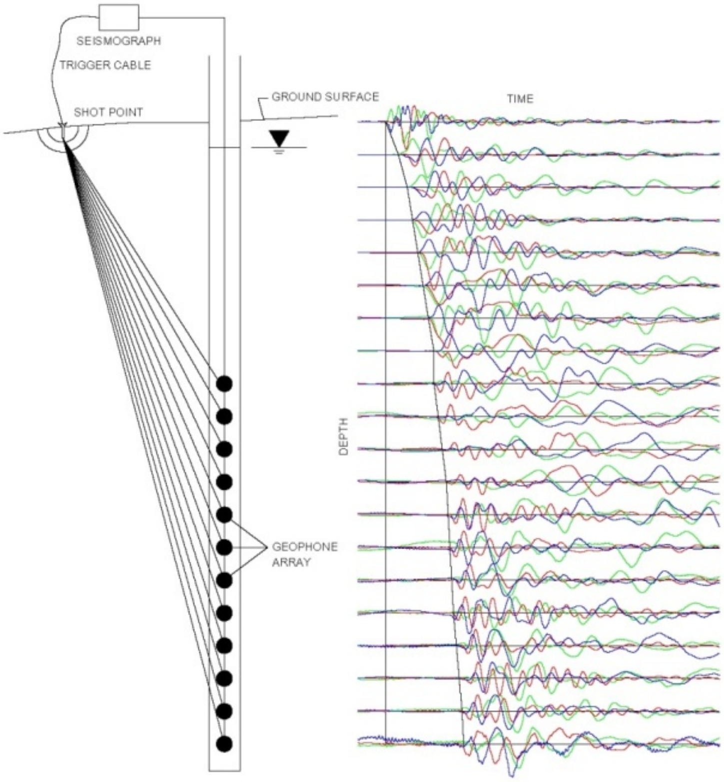
VSP TEST RESULTS
770 BRONSON STREET, OTTAWA, ONTARIO

This memorandum presents the results of the Vertical Seismic Profile (VSP) testing carried out at the proposed residential building located at 770 Bronson Street in Ottawa, Ontario. VSP testing was completed in borehole 15-04, located in the parking lot, on June 24, 2015. Borehole 15-04 was drilled to an approximate depth of 15.3 m below the existing ground and then cased with a PVC pipe grouted in place. The borehole consisted of approximately 2.3 m of fill, followed by 0.3 m of silt over fresh limestone bedrock.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole (Example 1).

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada, 2010.



Example 1: Layout and resulting time traces from a VSP survey.

Field Work

The field work was carried out on June 24, 2015, by personnel from the Golder Associates Ltd. (Golder) Ottawa office.

Both compression and shear-wave seismic sources were used and both were located in close vicinity to the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The plate was located 2 metres from the borehole on the ground surface. The seismic source for the shear-wave test consisted of a 3 metre long, 150 millimetres by 150 millimetres wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was also located 2 metres from the borehole, and coupled to the ground surface by parking a vehicle on top of it. Test measurements started at 1.0-metre below the ground surface. Data were recorded in the borehole with a 3-component receiver spaced at 1.0-metre intervals in the bedrock and 0.5-metre intervals in the overburden below the ground surface to a maximum depth of the casing (14 metres).

The seismic records collected for each source location were stacked a minimum of ten times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
2) Low Pass Filtering of data to remove spurious high frequency noise;
3) First break picking of the compression and shear-wave arrivals; and,
4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following two plots and show the first break picks of the compression-wave and shear-wave arrivals overlaid on the seismic waveform traces recorded at the different geophone depths (Figures 1 and 2). The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

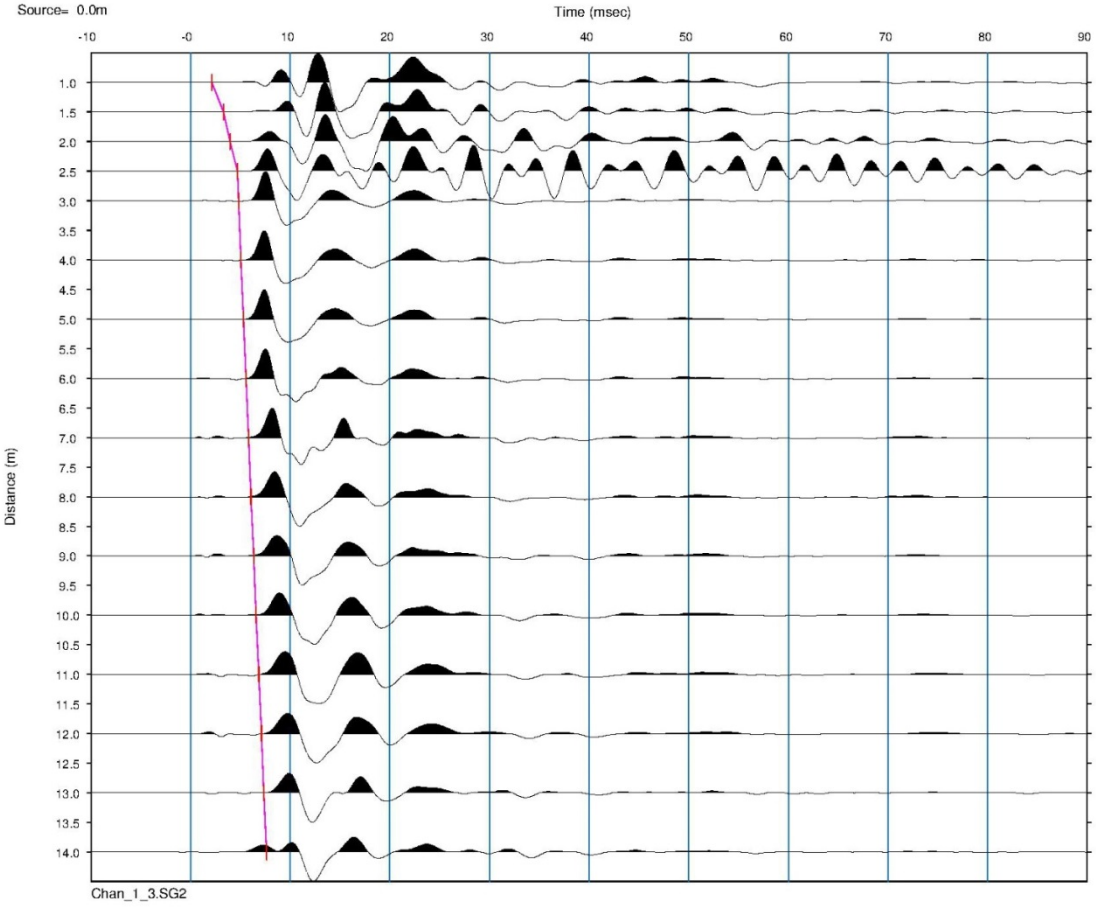


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth.

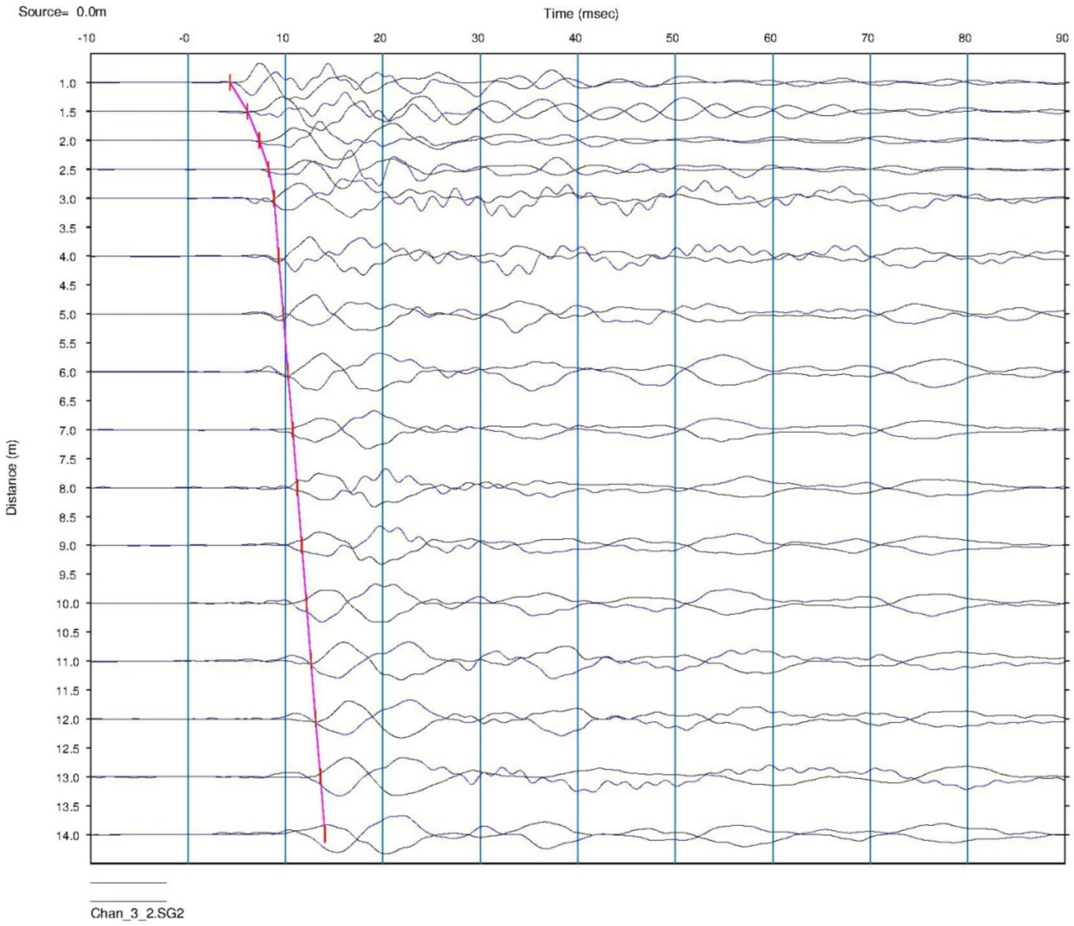


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth.

Results

The VSP results are summarized in Table 1. The shear-wave and compression-wave layer velocities, at the field collected metre intervals, were calculated by best fitting a theoretical travel time model to the field data collected at half metre intervals. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented on Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. A bulk density of 2,200 kg/m^3 was used for fill and silt to a depth of 2.5 m; 2,650 kg/m^3 for used for fresh limestone bedrock from 2.5 m down to 14 m bgs.

The average shear-wave velocity from ground surface to a depth of 30 metres was measured to be 1328 m/s. The average velocity was calculated assuming that the velocity from 14 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 1,650 m/s which is equal to the velocity of the bedrock at the bottom of the borehole.

Assuming the building will be founded on rock at a depth of 2.5 m bgs, the average shear-wave velocity was calculated to be 1,745 m/s, using an assumed velocity of 1,875 m/s from 14 to 32.5 m bgs.

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.

PATRICK I. FINLAY
PRACTISING MEMBER
1860
July 13 2015

Patrick Finlay, P.Geo.
Geophysicist

Christopher Phillips, M.Sc., P.Geo.
Senior Geophysicist, Associate

PF/CRP/sg
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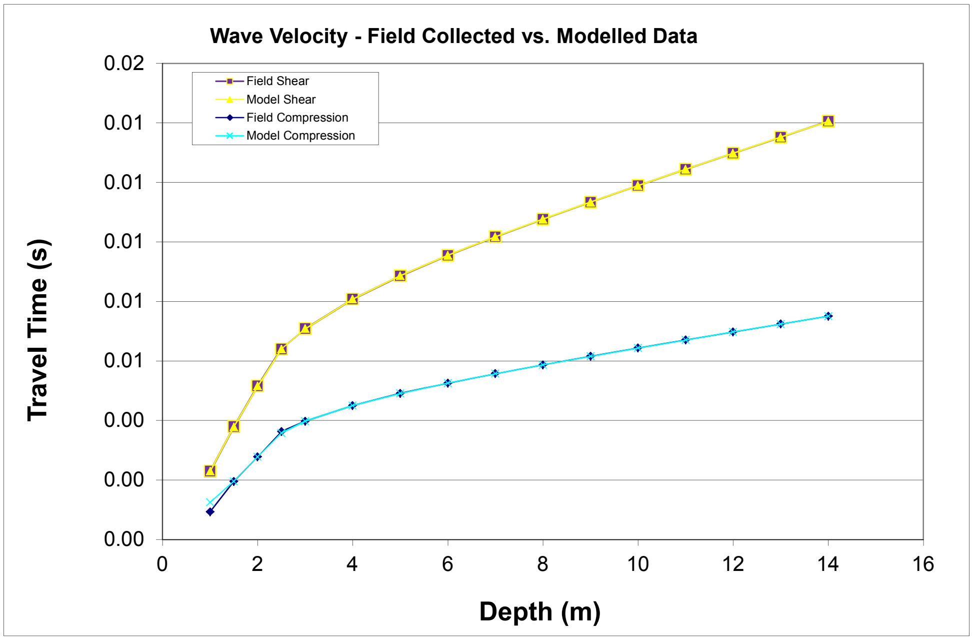
Attachments: Table 1: Shear Wave Velocity Profile at BH 15-04

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BH 15-04

Layer Depth (m)				Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1	800	436	2200	0.29	418	1078	850
1.0	1.5	706	335	2200	0.35	247	669	767
1.5	2	606	365	2200	0.22	293	712	417
2.0	2.5	630	402	2200	0.16	356	822	399
2.5	3	1254	733	2650	0.24	1424	3532	2269
3.0	4	1900	1008	2650	0.30	2693	7023	5976
4.0	5	2460	1268	2650	0.32	4261	11241	10356
5.0	6	2870	1464	2650	0.32	5680	15041	14255
6.0	7	3150	1600	2650	0.33	6784	17993	17249
7.0	8	3380	1690	2650	0.33	7569	20183	20183
8.0	9	3500	1750	2650	0.33	8116	21642	21642
9.0	10	3600	1790	2650	0.34	8491	22684	23023
10.0	11	3690	1830	2650	0.34	8875	23729	24250
11.0	12	3700	1850	2650	0.33	9070	24186	24186
12.0	13	3780	1860	2650	0.34	9168	24575	25640
13.0	14	3780	1875	2650	0.34	9316	24909	25442



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