SRM ARCHITECTS INC.

UPDATED GEOTECHNICAL STUDY

PROJECTED NEW BUILDING AT 774 BRONSON AVE., OTTAWA, ON

NOVEMBER 2015

UPDATED GEOTECHNICAL STUDY

PROJECTED NEW BUILDING AT 774 BRONSON AVE., OTTAWA, ON

SRM Architects Inc.

Draft report

Project no.: 151-12490-00
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WSP Canada Inc.
500 Greber Blvd., 3rd Floor
Gatineau, Quebec J8T 7W3

Telephone: +1-819-243-2827
Fax: +1-819-243-2019
www.wspgroup.com

November 13th, 2015

Mr. Ryan Hicks
SRM Architects Inc.
200-279 King Street West
Kitchener ON N2G 1B1

Object: Updated Geotechnical Study – Projected New Building at 774 Bronson Ave., Ottawa, ON.

Sir,

We are pleased to submit our updated geotechnical investigation report for the above-noted project. A field investigation and laboratory testing program was conducted to assess soil, bedrock and groundwater conditions at the site as part of a previous geotechnical report and will be used to emit preliminary recommendations pending the additional testing needed for final recommendations.

Included in this report are recommendations for foundation design for the projected building, a site plan with boreholes and test pits layout and results from our field and laboratory investigation.

We trust that the report is straightforward and meets your current requirements.

Please contact us if you have any questions.

Yours truly,

David Feghali, P. Eng.
Project Consultant

WSP Canada Inc.
500 Greber Blvd., 3rd Floor
Gatineau, Quebec J8 7W3

Tel.: +1 819-243-2827
Fax: +1 819-243-2019
www.wspgroup.com

REVISION HISTORY

VERSION	DATE	DESCRIPTION
0	**2015-11-13**	**Updated Geotechnical Study**

SIGNATURES

PREPARED BY

David Feghali, P. Eng.
Project Consultant

REVIEWED BY

Pierre Jean, P. Eng. M. Sc.
Team Leader – Geotechnical Engineering

PROJECT TEAM

SRM ARCHITECTS INC.

Project Manager	Ryan Hicks, B.E.D.S, LEED BD+C

WSP CANADA INC. (WSP)

Project Manager	David Feghali, P. Eng.
Report Review	Pierre Jean, P. Eng. M. Sc.

Reference to mention:

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1 INTRODUCTION

1.1 CONTEXT AND OBJECTIVES

As input to design of a proposed 12 storey apartment building, SRM Architects Inc. (SRM) retained WSP CANADA Inc. (WSP) in November 2015 to update a previous geotechnical investigation achieved on the same site back in 2012. The site is located at 774 Bronson Ave., in Ottawa, Ontario.

The mandate's objectives are to update the previously collected data (in 2011) along with the additional test pits (carried out recently) to provide geotechnical recommendations for the foundation design.

This report includes a brief description of the site and the project, of the investigation method used, of the nature, physical and mechanical properties of the bedrock, of the groundwater conditions, as well as our recommendations. Recommendations are addressed with regards to the existing bedrock geotechnical resistance at the ultimate limit state (ULS) and of the geotechnical reaction at the serviceability limit state (SLS). Excavation and backfilling procedures for and around the foundations will also be discussed. These recommendations will enable SRM's structural engineers to design the new building's foundations.

At the end of this document, the reader can consult the figure showing boreholes location.

Included in the appendices are boreholes logs (Appendix A) and laboratory testing results (Appendix B).

It is important to note that this report will be first delivered as a draft version, prior to achieving the additional test pits, as per SRM's request. Therefore, some sections will be updated when the final report is delivered.

1.2 LIMITATIONS

WSP assumes that the building location and ground elevations, both of which were provided by SRM, are accurate. Information in this report is only valid for the borehole and test pit locations as described. Should the subject tower location or elevation be moved, WSP should be contacted to review our findings and the possible need for additional investigative work.

Furthermore, the following report was produced for SRM for the construction of a 12-storey apartment building. Use of the report by third parties is prohibited without the consent of SRM.

2 DESCRIPTION OF THE SITE AND MANDATE

2.1 SITE LOCATION AND CONDITIONS

The site is located on a vast, relatively flat plot of land located between Cambridge St. and Bronson Ave, Ottawa. Existing buildings during the 2012 campaign are now demolished.

Current site condition will be further described in the final report, since the site conditions have changed likely since the 2012 campaign.

2.2 PROPOSED BUILDING

A preliminary CAD drawing showing the proposed 12-storey apartment building was received from SRM. The footprint of the proposed building is shown in Figure 1. One basement level is now expected. The basement datum is expected to be at the geodetic elevation of 70.4 m. Final structural design might slightly alter the footprint and the basement datum.

2.3 HISTORICAL GEOTECHNICAL DATA (PRIOR TO THE 2012 REPORT)

According to data contained in a report[1] from 1999 by the engineering firm of John D. Patterson and Associates of Nepean, four shallow boreholes were used to identify a bituminous concrete paving over a dark brown silty sand fill (relatively heterogeneous and containing organic matter, asphalt and wood debris in some instances), whose thickness varies between 0.7 and 2.3 m. This fill of poor quality (in geotechnical terms) lies on a thin glacial till deposit in three of the four boreholes; which is apparently of dense compactness and has a thickness of between 15 cm in BH3 and 0.46 m in BH1 (no till found in BH2).

In all of the boreholes the split-spoon sampler encountered refusal on probable bedrock, which consists of highly fractured fossiliferous limestone. The bedrock was cored in BH3 only between 2.13 and 4.47 m in depth; the rock’s RQD values vary between 47 and 82%, which means that this sedimentary rock is of poor to good quality (improving with depth).

A single observation well was installed to the right of borehole BH3. According to the 1999 report, groundwater was also found where the till and bedrock meet (at a depth of 2.13 m).

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[1] 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, May 18, 1999, Ref. E1738-1

3 METHODOLOGY

3.1 SITE INVESTIGATION

It should be noted that this section presents the methodology of the 2011 site investigation. The new proposed test pit investigation will be integrated to the final version of this report.

3.1.1 BOREHOLES

The geotechnical site investigation included the drilling of five (5) boreholes, three of which for geotechnical purposes and two for environmental characterisation. The borehole layout is provided in Figure 1. Borehole logs are provided in Appendix B.

Field work was undertaken on the 8th and 9th of December 2011 under the supervision of David Feghali, P.Eng. from WSP. Forage André Roy Inc., based in Saint-Isidore, Québec, was the drilling company that was contracted for the boreholes.

Boreholes were advanced using a truck-mounted CME-55 drill-rig, using rotating hollow auger sampler in the fill and natural soil deposit, while bedrock was cored using NQ size (72 mm exterior diameter) core barrels. Soil samples were collected in B caliber split-spoons, and the number of blows (N-index values) was recorded for each 15 cm (6 in) interval in accordance with standard ASTM D1586 procedures. Rock core samples (48 mm diameter) were placed in labelled core boxes for review and analysis.

Boreholes coordinates, elevation and depth are presented in Table 3-1 below.

Table 3-1 Boreholes Coordinates and Depth

BOREHOLE	UTM COORDINATES(1) (M)	GEODETIC ELEVATION (M)	DEPTH REACHED (M)	MONITORING WELL
FE-1-2011	445239, 5027674	75(2)	2.21	No
FE-2-2011	445236, 5027625	75(2)	0.88	No
FG-1-2011	445187, 5027657	75.12	4.20	Yes
FG-2-2011	445219, 5027667	75.45	4.70	Yes
FG-3-2011	445240, 5027666	74.59	4.14	Yes

(1) UTM NAD 83 zone 18
(2) Approximate elevation

3.1.2 MONITORING WELLS

Monitoring wells were installed at the bottom of the cavity created by three boreholes, FG-1-2011 to FG-3-2011. Each monitoring well consisted of a 51 mm-diameter slotted pipe with 0.51 mm openings, screwed onto a PVC tube of the same diameter, whose top extremity extends to the surface to the right of these three boreholes. The slotted pipe was installed inside a silica sand matrix of at least 3 m long, and then a bentonite plug was put between the upper limit of the sand pocket and the ground surface. A 200 mm-diameter (8 in) protective tube close to the ground was set at the ground surface.

Groundwater levels were measured using a water interface meter after the investigation work, on December 12, 2011, when it is assumed that the water levels had reached their static equilibrium.

3.1.3 SURVEYING

Boreholes were surveyed by WSP (previously known as GENIVAR), by measuring the distance between the boreholes location relative to the corners of the previously existing buildings.

The geodetic surface elevation was only measured at three of the boreholes, where a monitoring well was installed. The other two boreholes could not be found by the surveying team at the time of the survey.

3.1.4 LABORATORY TESTING

Three compression resistance tests were conducted according to standard ASTM D2938 on rock core from the following boreholes:

- FG-1-2011:sample DC-5, between 2.5 and 2.7 m of depth;
- FG-2-2011: sample DC-6, between 3.9 and 4.2 m of depth;
- FG-3-2011: sample DC-6, between 3.3 and 3.7 m of depth.

It should be noted that no more samples coming from these boreholes are still stored for additional testing.

4 SUBSURFACE CONDITIONS

The subsurface profile at the site consists of surficial fill, with an occasional layer of glacial till, overlying bedrock. Three boreholes penetrated and were terminated in bedrock. Specific descriptions of individual geological units are presented below. More detailed description will be added to this section when the test pit data will be obtained.

4.1 BITUMINOUS CONCRETE

A bituminous concrete layer is encountered in four boreholes (FE-1-2011, FE-2-2011, FG-1-2011 and FG-3-2011). This layer showed the same consistent thickness of 50 mm.

4.2 FILL

Considering the varying nature of this unit, description will be done separately for each borehole, as follow:

BOREHOLE FE-1-2011

Below the bituminous concrete, a 0.35 m thick granular base (brown gravel and sand with trace to some silt) is encountered over a 0.82 m thick sandy sub-base (fine brown sand with some silt and trace gravel.

Including the bituminous concrete pavement, total thickness of the engineered fill structure is 1.22 m. According to two standard penetration N-index values, 8 and 3 blows per 300 mm of penetration, compactness of this unit is judged to be Loose to very loose.

Below the engineered structure, a probable fill consisting of dark brown sandy silt with trace organic matter and gravel is encountered to 1.83 m depth. Compactness of this layer is judged very loose (SPT N index: 4 blows per 300 mm of penetration).

BOREHOLE FE-2-2011

Below the bituminous concrete, an engineered fill consisting of 0.15 m thick granular base (brown sand and gravel, trace to some silt) was encountered. No sandy sub-base was encountered. However, a rather heterogeneous fill including a thin layer of 10 cm-thick red brick debris (between 0.20 and 0.30 m depth), covering a brown layer of sand and gravel with trace red brick debris, down to a depth of 0.71 m. This unit is considered compact, according to a standard penetration N-index value, equal to 14 blows per 300 mm of penetration.

This heterogeneous fill covers a thin fill of dark brown sandy silt with trace organic matter and gravel, between 0.71 and 0.88 m depth (thickness: 17 cm), where refusal was encountered on probable bedrock (end of the borehole).

BOREHOLE FG-1-2011

Below the bituminous concrete, fill consisting of dark brown silt and sand with trace organic matter and gravel was encountered to a depth of 0.51 m. This fill covers a layer of weathered bedrock (equivalent to a very dense gravelly soil layer).

BOREHOLE FG-2-2011

Surficial fill consisting of thin layer of brown sand and gravel with trace silt was encountered to 0.15 m depth and is overlaying fill of dark brown sandy silt with trace organic matter and gravel which is present to 1.01 m depth. This loose layer (SPT N-index value: 9 blows per 300 mm of penetration) covers directly the bedrock.

BOREHOLE FG-3-2011

Below the bituminous concrete layer, a 25 cm-thick granular base (brown sand and gravel with trace silt) was encountered to a depth of 0.30 m. As in FE-2-2011, there is no sandy sub-base, but rather a layer of dark brown sandy silt with trace organic matter and trace gravel, at depths between 0.30 and 0.76 m (thickness: 46 cm). The compactness of this fill is loose according to a standard penetration N-index value of 6 blows per 300 mm of penetration where the refusal of the split spoon sampler was met on probable bedrock (end of the borehole).

4.3 NATURAL DEPOSIT: PROBABLE GLACIAL TILL

In borehole FE-1-2011 only and at a depth of 1.83 m (under the sandy silt fill with trace organic matter), a thin deposit of brown sandy and gravelly silt was found to a depth of 2.21 m (thickness: 38 cm). This deposit is judged to be a glacial till of dense compactness covering directly probable bedrock.

4.4 WEATHERED BEDROCK

In borehole FG-1-2011 only, under the silt and sand fill with trace organic matter, a weathered limestone layer was encountered at a depth between 0.51 and 1.07 m (thickness: 56 cm). This material reacts like a very dense granular soil. Indeed, the weathered bedrock was collected by a split-spoon sampler. The compactness of this layer is very dense, according to a standard penetration N index value of 59 blows per 300 mm of penetration.

4.5 LIMESTONE BEDROCK

At the location of boreholes FG-1-2011 to FG-3-2011, the rock mass was cored revealing a grey limestone, belonging to the Trenton Geological Group. Trenton group limestone is a carbonate sedimentary and fossiliferous rock dating from the Middle Ordovician Era (some 471 to 460 million years ago).

Table 4-1 presents the bedrock depths detected by either split spoon sampler refusal (boreholes FE-1-2011 and FE-2-2011) or by coring procedures for the other boreholes.

Table 4-1 Encountered Bedrock depth

BOREHOLE	DEPTH TO BEDROCK (M)	BEDROCK SURFACE GEODETIC ELEVATION (M)	BEDROCK CORE LENGTH (M)
FE-1-2011	2.21	72.8 (1)	---
FE-2-2011	0.88	74.1 (1)	---
FG-1-2011	1.07	74.05	3.13
FG-2-2011	1.01	74.44	3.69
FG-3-2011	0.76	73.83	3.38

(1) Approximate elevation

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 pieces by the total length drilled. The RQD classification of the rock according to this value is indicated in the following Table 4-2:

Table 4-2 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 quality of the cored limestone is generally poor to very poor at the upper end in the three boreholes, but becomes fair to good or excellent with depth as shown in table 4-3 below

Table 4-3 Limestone quality in the boreholes as a function of depth

BOREHOLE	VERY POOR TO POOR QUALITY LIMESTONE ZONE (M) (RQD)	FAIR QUALITY ZONE (M) (RQD)	GOOD QUALITY ZONE (M) (RQD)	EXCELLENT QUALITY ZONE (M) (RQD)
FG-1-2011	1.07 to 1.35 (0%)	1.35 to 2.83 (56%)	--	2.83 to 4.20 (90%)
FG-2-2011	1.01 to 3.10 (0%. 49% and 22%)	--	3.10 to 4.70 (80%)	--
FG-3-2011	0.76 to 1.23 (0%)	1.23 to 2.75 (70%)	2.75 to 4.14 (75%)	--

The results of the three laboratory-conducted uniaxial compression strength tests indicated values of 109 MPa in FG-1-2011 (2.5 to 2.7 m of depth), 74 MPa in FG-2-2011 (3.9 to 4.2 m of depth) and 128 MPa in FG-3-2011 (3.3 to 3.7 m of depth).

4.6 GROUNDWATER

Groundwater levels Measurement was taken on December 12, 2011, in the monitoring wells in boreholes FG-1-2011 to FG-3-2011. Table 4-4 below presents the groundwater depths measured.

Table 4-4 Groundwater depth and elevation in the boreholes

BOREHOLE	GROUNDWATER DEPTH (M)	GROUNDWATER GEODETIC EVATION (M)
FG-1-2011	2.19	72.96
FG-2-2011	2.33	73.12
FG-3-2011	1.93	72.66

It should be noted that the groundwater depths are only representative of the period during which the readings were taken. Indeed, the depth of the groundwater varies by many decimetres depending on the season and precipitation.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

The geotechnical investigation results indicate the following stratigraphy with increasing depth:

- Four of the five boreholes have a 5-cm thick bituminous concrete pavement, covering a granular base of variable quality and with generally loose to very loose compactness (sand and gravel or gravel and sand with trace silt). No granular base found in FE-2-2011 or FG-1-2011;
- In all five boreholes, fill of dark brown sandy silt to silt and sand, with trace organic matter and gravel;
- In FE-1-2011, thin layer of probable glacial till, at depth of between 1.83 and 2.21 m;
- In FG-1-2001, layer of weathered rock acting as a soil layer (very dense compactness), between 0,51 and 1,07 m depth;
- Sound Limestone bedrock located at depths varying between 0.76 m (FG-3-2011) and 2.21 m (FE-1-2011). Limestone quality is very poor to poor surficially, improving from fair to good (excellent in FG-1-2011) with depth. The uniaxial compression strength of three rock core samples varies from 74 to 128 MPa.

On December 12, 2011, the groundwater was measured at elevations of between 72.66 and 73.12 m in the observation wells within the geotechnical boreholes.

The recommendations formulated in the following sections are based on the hypothesis that the results from the boreholes are representative of the conditions throughout the investigated site.

Additional test pits will be achieved on site in order to cover all of the building footprint area and confirm bedrock depth. The reader should therefore accept that the recommendations below might be altered depending on the findings of the additional investigation.

As mentioned in section 2, basement level is expected to be at an elevation of around 70.4 m depth.

5.2 ALLOWABLE BEARING CAPACITY

We have assessed the allowable bearing capacity (q_{all} of the bedrock) at 7.4 MPa, below the depth of the projected surficial footings and with respect to the recommendations set forth in section 5.4 of the report. Given that the joints found have very small gaps (around 0.5 mm on average), the differential and total settlement will be negligible. We recommend that the footings be casted directly onto the intact and hereinbefore cleaned bedrock.

By allowable bearing capacity, we mean any additional load that the foundation rock can support, in addition to the actual weight of the earth.

5.3 BEDROCK EXCAVATION

In the event that blasting is used for excavation in the bedrock, these operations should be conducted carefully and according to Ontario Provincial Standard Specification (OPSS) 120.

Moreover, in the event that our recommendations regarding blasting are too restrictive for the designer, other methods for breaking up rock can be used, such as a hydraulic hammer (Tramac). Recommendations in this section apply to all rock-breaking methods.

No major open fractures were detected in the rock found in the geotechnical boreholes. However, it is possible that such fractures be found during the excavation of the rock mass.

If this were to happen, such fractures could have a major effect on the behaviour of the rock mass during excavation by generating undesirable rock movements under the foundations. In that case, these fractures could require bolting to support the excavated rock faces.

Due to the network of fractures in the bedrock, falling blocks through gravitational pull from the rock faces exposed by excavation can seriously compromise the structural integrity of existing buildings located over or near the rock faces. The borehole data cannot be used to determine the direction of the main fracture sets, but, to make up for this gap, we recommend that a rock fracture survey be conducted by a geological engineer shortly after the beginning of excavation, to gather information regarding the newly-exposed rock surfaces. This survey could be conducted during work stoppages by employees conducting excavation operations so that the safety of the personnel in charge of this survey is not compromised nor is the work unduly delayed. This would thus provide clear information to be used when deciding whether to systematically or selectively bolt the rock faces. However, minimal bolting of the rock face (a few well-placed bolts) should be designed, and the bolting procedure included in the plans and specifications. The depth of the anchoring shall be determined according to the observations made during excavation. The bolting design shall consider split-anchor bolts (i.e., sealed) to prevent bolts from rusting and ensure their durability.

Moreover, given the size of the existing buildings neighbouring the projected residential building, we recommend that the top of the rock faces not be less than 3 m from the base of existing foundation walls. This is so that the effect of any accidental rock fall which could lead to the collapse of one of these walls is minimized. If this distance of 3 m was not respected, systematic bolting of the rock face under the existing buildings should be considered.

5.4 EXCAVATION, FOUNDATION PLACING AND BACKFILLING

Recommendations for the placing of the sub-base and embankments are as follow:

- Excavate completely the soil in place (surface embankment and natural deposit, if the case arises). These soils should not be used as backfill for the new building foundations due to great grain-size variability and occasional organic material content;
- Excavate the weathered rock (encountered in FG-1-2011) using a powerful hydraulic shovel (if the case arises), and a Tramac type rock-hammer in fractured rock or blasting the sound rock up to the projected footing depth. The bottom of the excavation on the building periphery should be wide enough to install a peripheral drain and liner (clean stone of 20 mm in diameter);
- Excavation slopes shall be cut at 1H:1V for the existing fill and in the thin surficial natural deposit (in non-saturated conditions) and in the weathered rock layer (if the case arises). The bedrock excavation slopes should have a slope of 1H:10V and have their surface prepared by removing all loose pieces;

- Since the groundwater level will probably be at a higher elevation than the base of the excavation, 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;
- We believe that the blasted bedrock surface (bottom of the excavation) can include irregular features that depend on the borehole and blasting technique used (see previous section 6.3). The foundation and slab surface must be free of any loose rock pieces and not part of the rock mass. In addition, cavities and cracks must be filled with lean concrete (cavities) or with grout (cracks) with a compression strength of at least 14 MPa after curing for 28 days. The purpose is to have a uniform and horizontal surface under the slab and footing. In all cases, the prepared bedrock surface must be inspected by a geotechnical engineer;
- Pour the footing and foundation walls;
- The top level of the peripheral drain must be placed at the base of the peripheral footing, to adequately collect the storm water surrounding the structure. The drain should be covered using granular material of 20 mm in diameter (150 mm in thickness). A nonwoven geotextile (Texel 909 type or equivalent) should be used to enclose the drain/clean stone and this way, stop the soil in-place from seeping through the clean stone;
- After curing the foundation walls for 48 hours, backfill around the walls using a Type 2-B non-frost susceptible granular fill (which complies with OPSS 1010). This backfill must be installed in successive layers of 300 mm maximum thickness, each compacted to 95% of the maximum dry density obtained from the modified Proctor test (ASTM D1557).

5.5 LATERAL EARTH PRESSURES

The basement of the building will be constructed considering foundation walls located in an excavation performed mainly in the limestone rock mass. The space between the exterior side of the concrete walls and the rock walls will be backfilled with a granular B-Type 2 non-frost susceptible fill (OPSS 1010). Table 5-1 below presents the geotechnical parameters to be used for the structural design of basement walls:

Table 5-1 Geotechnical parameters for granular fill

PARAMETER	MATERIAL : TYPE 2-B OPSS 1010
Wet unit weight (γ_h)	20kN/m^3
Internal friction angle (φ’)	33°
Earth pressure coefficient at rest (K_0)	0.46
Active Earth pressure coefficient (K_a)	0.29
Passive Earth pressure coefficient (K_p)	3.36

Assuming that the basement would include unyielding walls, we recommend using the coefficient of earth pressures at rest, K_0.

The total earth pressure P_{ae} includes the static earth pressure component (P_0) and the seismic component (ΔP_{ae}), determined with the following equations:

Static horizontal earth Pressures, P_0 (triangular distribution):

$$P_0 = K_0\ \gamma_{\boxed{?}}\ H$$

where:

K_0 = Earth pressure coefficient at rest of the applicable retained soil (0.46);
γ_h = unit weight of the granular fill of the applicable retained soil (kN/m³);
H = height of the wall (m).

Seismic earth pressures, ΔP_{ae}:

$$\Delta P_{ae} = 0{,}375\ a_c\ \gamma_h\ H^2/g)\ a_{max}$$

where:

a_c = $(1{,}45 – a_{max}/g)\ a_{max}$;
γ_h = unit weight of the granular fill of the applicable retained soil (kN/m³);
H = height of the wall (m);
g = gravity (9.81 m/s²).

The peak ground acceleration (a_{max}) for the Ottawa area is 0.42g according to OBC 2006. The vertical seismic coefficient is assumed to be zero.

The total earth pressure (P_{ae}) is considered to act at a height, h (m), from the base of the wall, as per the following equation:

$$H = \{P_0\ (H/3) + \Delta P_{ae}\ (0.6H)\}/P_{ae}$$

For the ULS case, the earth pressure loads should be factored as live loads, as per OBC 2006.

5.6 INSTALLATION OF FLOOR SLAB AND DRAINAGE

To prevent groundwater infiltration through the floor slab, we recommend to slightly tilt the bottom of the excavation under the slab with a slight slope of 1% towards a low point, which would allow the drainage of the whole area occupied by the new building. This slab will lie on a granular-based sub-base, made of a 300 mm thick and permeable sand cushion, spread on the excavated rock mass. The grain-size distribution properties of this sand must meet a uniformity coefficient, CU, lower than 3 and the grain-size distribution requirements are listed in Table 5-2.

Table 5-2 Grain Size Distribution Requirements for Permeable Sand

SIEVE (MM)	PASSING PERCENTAGE (%)
80	100
5	50-100
0.315	0-50
0.160	0-10
0.080	0-5

Within the periphery and at every 10 m distance under the slab, perforated plastic drains (150 mm in diameter) shall be provided, covered and lined on both sides with clean granular material of 20 mm in diameter and at least 150 mm thick. The clean granular material – perforated drain set will have to be protected against fine material infiltration by covering them with a model 900 Texel type geotextile (or equivalent).

The upper part of all drains must be located at about 300 mm below the bottom of the slab and have a drainage slope greater than 1% and a surface height variation of about 10 mm must be planned at every pipe elbow (90°). The water collected by these drains must runoff towards the appropriate outlet.

5.7 FROST PROTECTION

We recommend that all foundations shall be installed directly on sound limestone rock mass (after blasting and cleaning), which is considered non-susceptible to frost detrimental action. The floor slab for the basement shall be installed while accounting for section 5.6 of this report.

5.8 SEISMIC CLASSIFICATION

The National Building Code of Canada (NBC) 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’. Based on the explored soil properties and in accordance with Table 4.1.8.4.A of the National Building Code (2005), it is recommended that Site Class ‘B’ (soft rock) be applied for structural design at this site.

5.9 OUTDOOR PARKING LOTS AND ACCESS ROADS

5.9.1 EXCAVATION AND BACKFILLING

When writing this report, elevations of the projected pavement structure of the outside parking lots and access lanes were not known around the new building. Therefore, the following recommendations assume that the present levels are maintained.

Thus, we recommend completely removing existing fill and the thin till deposit and replacing them with an OPSS 1010 granular B, Type 2 fill. This new embankment will be built up to the infrastructure line and compacted in successive, horizontal layers of 300 mm thickness, to 90% of the maximum dry density obtained from the Modified Proctor test (ASTM D1557).

With regards to the pipework (storm sewer or sanitary sewer) excavation slopes are the same as mentioned in section 5.4.

The trench features (thickness of the bedding and bottom width) have to meet Standard OPSS 401 – Construction Specification for Trenching, Backfilling and Compacting and Drawings for Roads, Barriers, Drainage, Sanitary Sewers, Watermains and Structures, Volume 3, edited by the Ontario Provincial Standards [OPS] for Roads & Public Works. The pipework bedding shall be built with coarse sand containing 15% to 30% of gravel (particles with a diameter ranging from 5 to 100 mm), 10% maximum of bottom particles with a 0.08 mm in diameter (silt) and presenting a uniformity coefficient (C_U) of at least 5. The bedding shall have to be compacted to at least 90% of the maximum dry density obtained from the Modified Proctor test.

For traffic lanes, the pavement structure remains the same, except for the granular sub-base, which thickness must be increased to 250 mm.

The materials features and the placing of the pavement structure must comply with the standards established in the Drawings for Roads, Barriers, Drainage, Sanitary Sewers, Watermains and Structures, Volume 3, edited by the OPS for Roads & Public Works.

5.9.2 PAVEMENT STRUCTURE OF PARKING LOTS AND TRAFFIC LANES

The pavement structure of parking lots shall be built in compliance with the specifications of the Ontario Provincial Standards, as indicated in Table 5.3.

Table 5-3 Grain Size Distribution Requirements for Permeable Sand

LAYER	MATERIAL	THICKNESS (MM)	COMPACTION (MODIFIED PROCTOR TEST) (%)
Bituminous concrete (surface course)	Superpave 9.5	35	--
Bituminous concrete (base course)	Superpave 19.0[(1)]	65	--
Base	Granular A	200	98%
Sub-base	Granular B Type 2	300	95% for the 150 mm of subgrade 90% for the 450 mm of the sub-base
Total Thickness of the Pavement		**600 mm**	

(1) Asphalt mixture according to OPSS 1101 specifications (PG 58 34 binder recommended)

The material features and the placing of the pavement structure must comply with the standards established in the Ontario Provincial Standards.

5.9.3 DRAINAGE

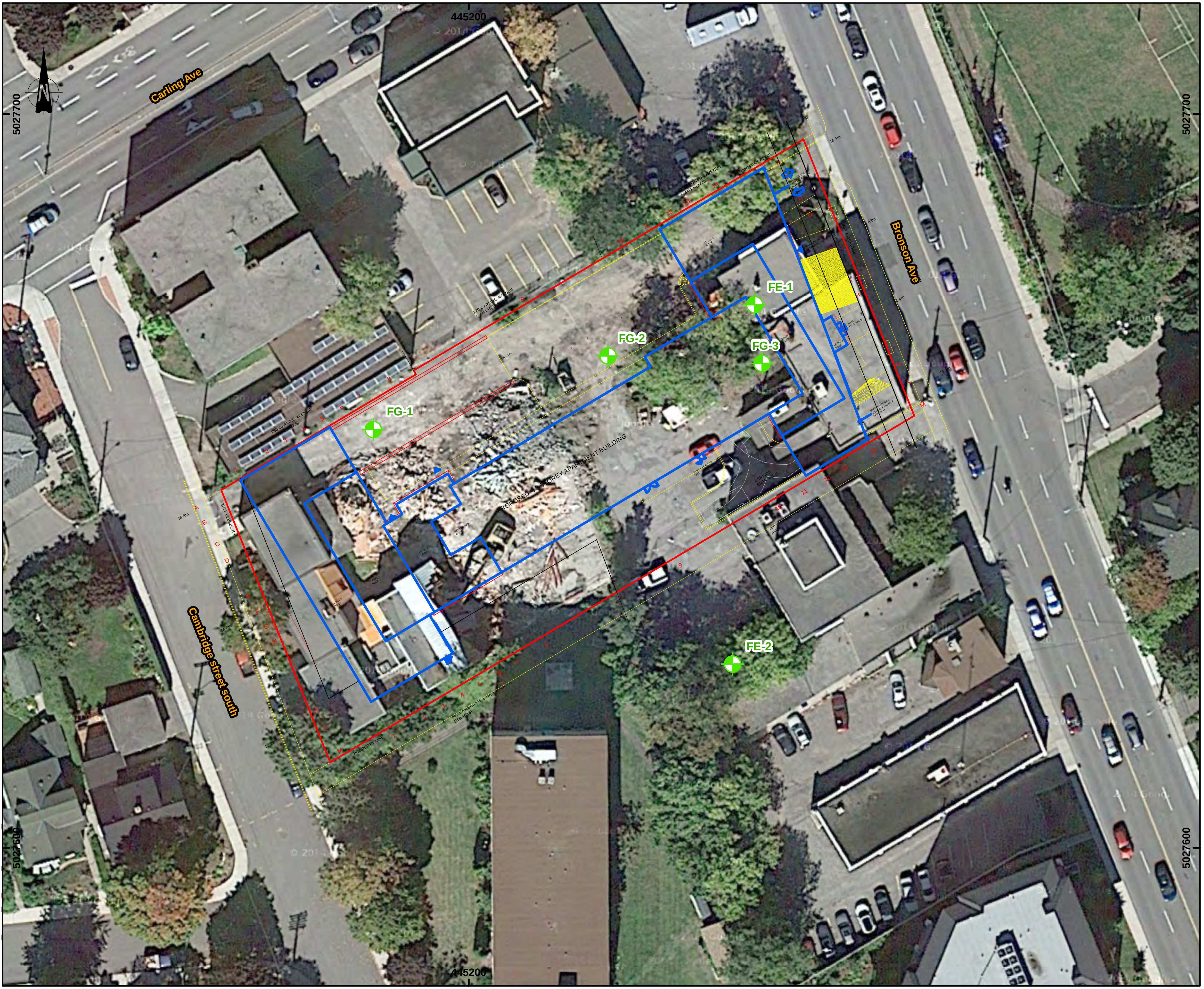
The drainage of surface runoff in parking lots and of low sections under the infrastructure line should be performed using the appropriate piping and catch basins, which shall be linked to a proper outlet.

5.10 QUALITY CONTROL

We recommend that the quality and placement of materials (soil, concrete cement and, eventually, asphalt concrete) be monitored by the experienced quality control technical staff of a specialized material engineering firm, under the supervision of an engineer who is skilled in that field

.

Figure



● Borehole

🟥 Site limits

🟦 Projected Building Footprint

1 : 500

Projection : NAD83, UTM fuseau 18N

SRM ARCHITECTS INC.

UPDATED GEOTECHNICAL STUDY

Projected New Building at 774 Bronson Ave.
Ottawa, On

Figure 1

Boreholes location

Sources :
Orthophoto : Google Map
Maps : - ESRI World topographic Map
Municipal Limits : SDA20K, 2010-01

Preparation : D. Feghali
Drawing : I. Douce
Approval : P. Jean

12 novembre 2015

Fichier : 151_12490_00_GSF1_boreholes_151112.mxd

Appendix A

BOREHOLE LOGS

WSP

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

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	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.							

>>

WSP

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

Page 1 of 1

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

Project Name:	**Geotechnical Investigation**	Project Number:	**111-26060-00**
Site:	**Projected building between Bronson Ave and Cambridge S. St.**	Geographic Coordinates:	X = 445236 W Y = 5027625 N
Sector:	**Southwest of Site**	Surface Elevation:	75 m *(Approximatif)*
Client:	**Samcon Inc.**	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

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)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	GEOTECHNICAL	DIAGRAM
		Ground surface.									
0.05 *74.95*	■	Bituminous concrete.				MA-1 SS-2	☒	70	5 7 7 12 (14)	▲	
0.20 0.30 *74.70*		Fill: brown sand and gravel, trace to some silt. Compact compactness.									
0.5		Red brick debris.									0.5
0.71 *74.29*		Fill: brown sand and gravel, trace red brick debris. Compact compactness.				SS-3	☒	81	9 50/12 cm		
0.88 *74.12*		Probable fill: dark brown sandy silt, trace organic material. Compact compactness. Split spoon refusal at 0.88 m.									
1.0											1.0
1.5		End of borehole at 0.88 m.									1.5
2.0											2.0
2.5											2.5
3.0											3.0
3.5											3.5
4.0											4.0
4.5											4.5
5.0											5.0

BOREHOLE DRILLING RECORD : FG-1-2011

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

Date (Start): **2011-12-08**
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

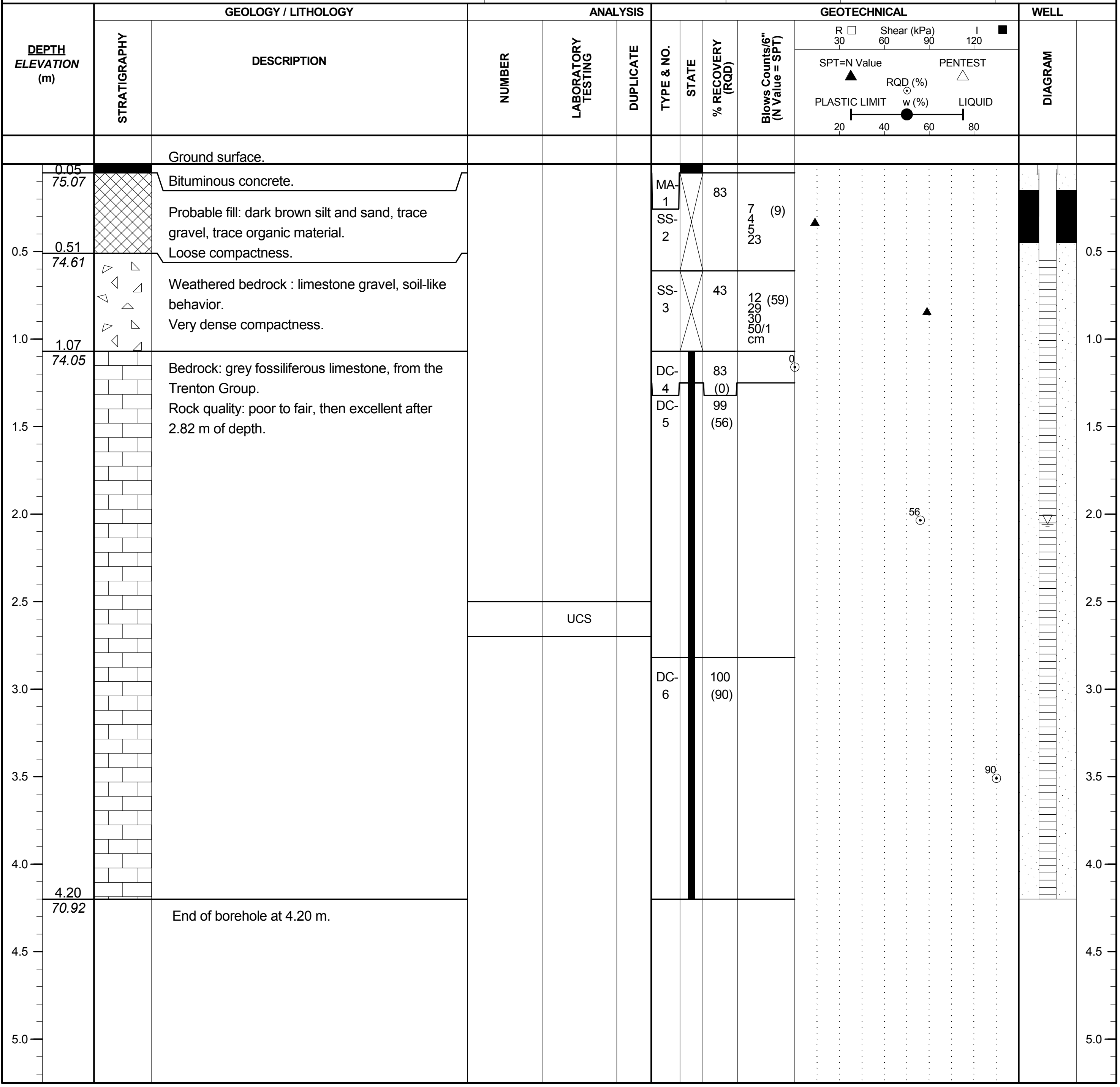
COPING	Elevation :	75.08 m
SCREEN	Bottom Depth :	4.2 m
	Length :	3.66 m
	Opening :	0.51 mm
WATER Elevation:		72.96 m
WATER Date:		2011-12-12

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

TYPE & NO.	% 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)	

LABORATORY TESTING
UCS

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

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

Page 1 of 1

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

Date (Start): **2011-12-08**
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

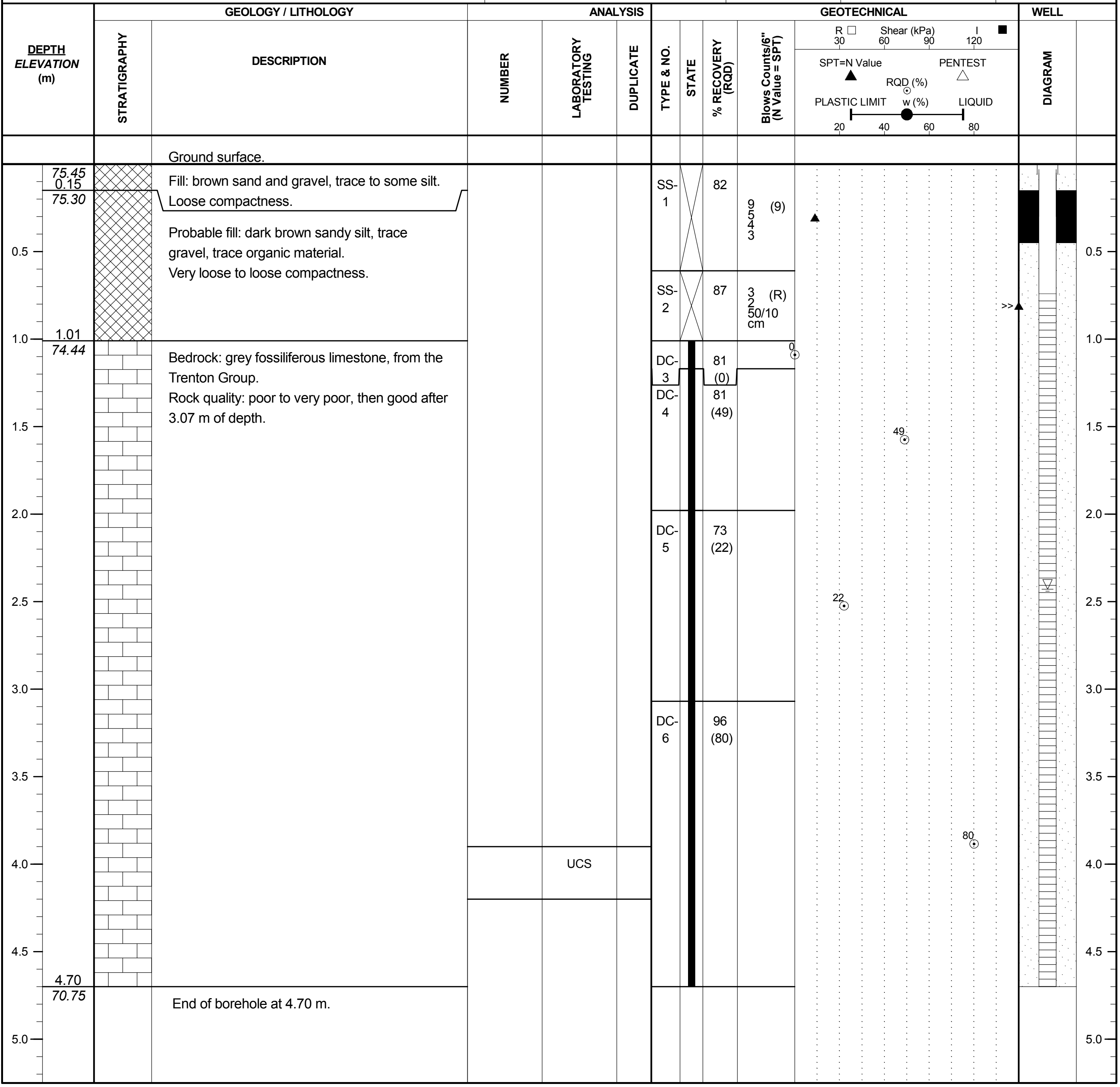
Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Water

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	Laboratory testing	Type & No.	% Recovery (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.				
75.45 0.15 75.30	Fill: brown sand and gravel, trace to some silt. Loose compactness.		SS-1	82	9 5 4 3 (9)
	Probable fill: dark brown sandy silt, trace gravel, trace organic material. Very loose to loose compactness.		SS-2	87	3 2 50/10 cm (R)
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.		DC-3	81 (0)	
			DC-4	81 (49)	
			DC-5	73 (22)	
			DC-6	96 (80)	
		UCS			
4.70 70.75	End of borehole at 4.70 m.				

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

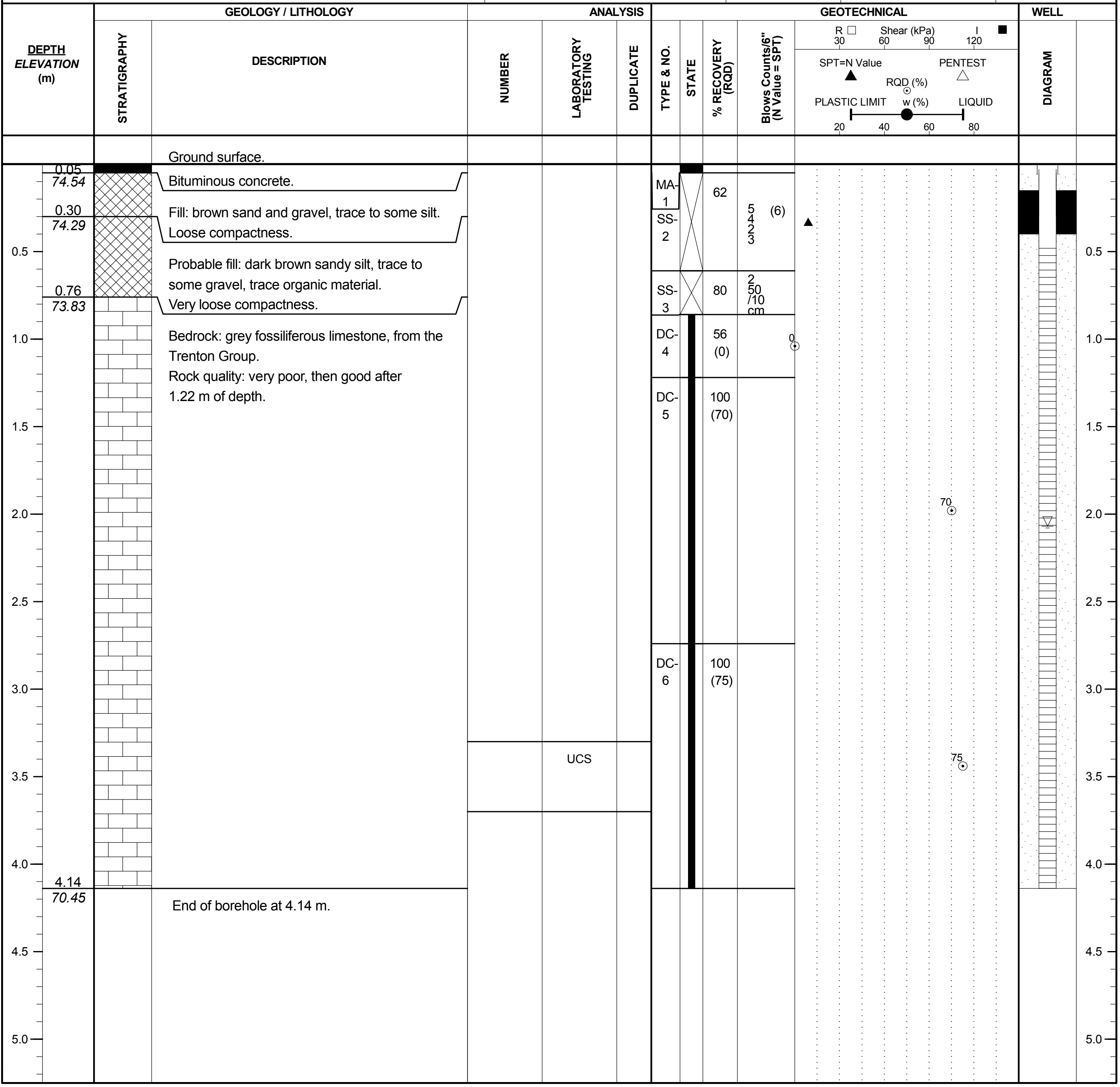
▽ Water Level ▼ Free Phase

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



DEPTH *ELEVATION* (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	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)
0.76 *73.83*	Probable fill: dark brown sandy silt, trace to some gravel, trace organic material. Very loose compactness.				SS-3	80	2 50 /10 cm
	Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: very poor, then good after 1.22 m of depth.				DC-4	56 (0)	
					DC-5	100 (70)	
					DC-6	100 (75)	
			UCS				
4.14 *70.45*	End of borehole at 4.14 m.						

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

Appendix B

LABORATORY TESTING RESULTS

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
☐ 1471, boul. Lionel-Boulet, Varennes (Qc) J3X 1P7 Tél: (450) 652-6151 Fax: (450) 652-6451
☐ 75, rue Queen, bureau 5200, Montréal (Qc) H3C 2N6 Tél: (514) 982-6001 Fax: (514) 982-6106
☐ 4540, rue Laval, Lac-Mégantic (Qc) G6B 1C5 Tél: (819) 583-4255 Fax: (819) 583-1997
☐ 2111, boul. Fernand-Lafontaine, Longueuil (Qc) J4G 2J4 Tél: (450) 651-0981 Fax: (450) 651-9542

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'égo
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.