Phoenix Homes

Geotechnical Investigation

Type of Document
Final

Project Name
Proposed Mixed Use Residential/Commercial Building
770 Somerset Street West, Ottawa, Ontario

Project Number
OTT-00210537-A0

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Date Submitted
January 29, 2013

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Attention: Michael Boucher

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Date Submitted:
January 29, 2013

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Executive Summary

A geotechnical investigation was undertaken at the site of the proposed residential and commercial development to be located on the property registered by the street address of 770 Somerset Street West and situated in the southeast quadrant of the intersection of Lebreton and Somerset streets in the City of Ottawa, Ontario. This work was authorized by Mr. Mike Boucher of Phoenix Homes on December 15, 2012.

The proposed development will comprise of a nine-storey residential/commercial building with three levels of underground parking. A two-storey residential dwelling and a retaining wall currently situated on the property will be demolished as part of the proposed development.

The fieldwork for this investigation comprised the drilling of five boreholes at the site to 2.8 m to 13.3 m depth. Standard penetration tests were performed in the overburden to refusal depth and soil samples were obtained. Below refusal depth, three of the five boreholes were cased and advanced further into the bedrock by washboring and core drilling techniques.

The investigation revealed the site to be underlain by silty sand with some gravel, decayed wood pieces and organic fill which extends to depths ranging between 0.6 and 3.0 m, Elevation 72.6 m to 75.3 m. The fill is underlain by compact to dense sand to sand till which extends to the bedrock surface contacted at depths of 1.8 m to 4.6 m at the locations of the boreholes. The bedrock at the site is limestone with shale partings of very good to excellent quality.

Attempt was made to measure the groundwater at the site in standpipes installed in three of the boreholes drilled as part of this investigation; however, this was not possible due to the presence of ice and snow on top of the steel casing installed around the installations. The groundwater table is expected to be at depths ranging between 2.8 m and 3.8 m, based on the information contained in the Phase II Environmental Site Assessment (ESA) prepared by MMM group and provided to **exp** as background material.

The investigation has revealed that the geotechnical conditions at the site are well suited to construction of the proposed building on spread and strip footing foundations. Spread and strip footings founded on the limestone bedrock may be designed for factored geotechnical resistance at Ultimate Limit State (ULS) of 1000 kPa to 3000 kPa depending on the amount of inspection and testing undertaken during construction. The settlements of the footings founded on sound limestone bedrock are expected to be less than 10 mm.

The lowest level floor slab of the proposed building may be constructed as slab-on-grade. Perimeter as well as underfloor drainage system should be provided for the proposed structure.

Based on the subsurface conditions, the site has been classified as Class A in accordance with Table 4.1.8.4.A of the 2006 Ontario Building Code (OBC).

General Use Portland cement may be used in the subsurface concrete at the site.

Excavation at the site will extend to 8 m to 9 m depth close to the property boundaries. This excavation will extend through the fill, sand/sand till and into the bedrock. The founding depth and founding medium of the existing structures located on the east and south sides of the site are not known.

These structures, if not founded on bedrock, would require underpinning. The alternative to underpinning would be to support the sides of the excavation along the east and south face with a rigid reinforced concrete secant wall, which is capable of supporting the lateral earth pressure “at rest” as well as building loads. If the structures located on the east and south side of the site are founded on bedrock, conventional shoring consisting of steel H piles and timber lagging is expected to suffice. Shoring may also be required along the west and north sides of the excavation where city streets are located depending on the proximity of the excavation to these streets and presence of any settlement sensitive structures or services under the roadways.

The reinforced concrete secant wall and/or steel H piles and timber lagging shoring may be designed based on the expressions given in the report.

The overburden at the site may be excavated with conventional mechanical equipment. Excavation of the bedrock would require blasting and should be undertaken by an experienced blaster working under the direction and supervision of a blasting expert. A condition survey of the neighboring structures and services should be undertaken prior to commencement of blasting operations. Vibrations generated during blasting should be monitored and should not exceed the criteria presented in the report. Seepage of surface and subsurface water into the excavation should be anticipated. However, it should be possible to collect the water in perimeter ditches and to remove it by pumping. Permit to take water may be required from the Ontario Ministry of the Environment if the quality of water to be pumped from the site exceeds 50,000 liters per day.

The backfill against the subsurface walls should be free draining granular material properly conforming to Ontario Provincial Standard Specifications (OPSS) for Granular B, Type II. It should be compacted to 95 percent of standard Proctor maximum dry density (SPMDD).

The above and other related considerations are discussed in greater detail in the report.

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1 Introduction

Exp Services Inc. (**exp**) was retained by Phoenix Homes to undertake a geotechnical investigation at the site of the proposed new residential and commercial development to be located at 770 Somerset Street in the City of Ottawa. This work was authorized by Mr. Mike Boucher on December 15, 2012.

The proposed residential and commercial building will comprise of nine storeys with three levels of underground parking.

The investigation was undertaken to:

a.) Establish geotechnical and groundwater profile at the site;

b.) Recommend type of foundations and bearing pressure/factored geotechnical resistance in limit states design of the founding stratum, including anticipated settlements;

c.) Provide lateral resistance of the foundations;

d.) Discuss slab on grade construction of the lowest floor level and permanent drainage requirements;

e.) Comment on lateral earth pressures (static and dynamic) against subsurface walls;

f.) Comment on excavation conditions and the requirement of any support system or underpinning of the existing structures;

g.) Discuss backfilling requirements and suitability of on-site soils for backfilling purposes;

h.) Provide site classification for seismic site response in accordance with the current Ontario Building Code (OBC);

i.) Recommend pavement structure for the access ramps and lowest level of the underground parking; and

j.) Comment on subsurface concrete requirements.

The comments and recommendations given in this report are based on the assumption that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2 Site Description

The subject site is located in the southeast quadrant of the intersection of Somerset and Lebreton and registered by the street address of 770 Somerset Street West in the City of Ottawa, Ontario (Figure 1). It measures approximately 42 m by 37 m in plan and is currently used as a parking lot. A two-storey residential dwelling (13 Lebreton Street) and a retaining wall are currently located on the southern portion of the site and will be demolished as part of the proposed development.

The site is bounded by Somerset Street to the north, Lebreton Street to the west, a six-storey building with basement to the east and a two-storey basementless building to the south. The site slopes down toward the south and west with the ground surface elevations at borehole locations varying between Elevation 75.9 m to Elevation 70.1 m.

3 Available Reports

A Phase I and Phase II Environmental Site Assessment (ESA) prepared by MMM group for the subject site in December 2012 was provided to **exp** as reference material. As part of the fieldwork, a total of eight boreholes were drilled at the site to depths ranging between 2.89 m and 4.11 m. The investigation revealed the bedrock at the site is present at 2.89 m to 3.91 m depth. Groundwater measurements taken as part of the Phase II ESA indicated a groundwater table to be at depths ranging between 2.8 m and 3.8 m below the existing ground surface. This report should be consulted to assess the environmental conditions of the soil and groundwater at the site and their impact on the construction, dewatering, etc.

4 Procedure

The fieldwork for this investigation was completed on January 9 and 10, 2013 and comprised the drilling of five boreholes (Borehole Nos. 1 to 5) to depths ranging between 2.8 m to 13.3 m.

The fieldwork was undertaken with a track-mounted drill rig equipped with continuous flight hollow stem augers and core drilling equipment and was supervised on a full-time basis by a representative of **exp**. The locations of the boreholes are shown on the Proposed Borehole Location Plan, Figure 2.

The boreholes were initially advanced in the overburden by auguring techniques to refusal depth. Standard penetration tests were performed in the overburden in all the boreholes at 0.75 m to 1.5 m depth intervals and soil samples retrieved by split barrel sampler. Below the refusal depth, Borehole Nos. 1, 2 and 5 were cased and advanced further into the bedrock by washboring and core drilling techniques using Nx-size core barrel to termination at depths ranging between 10.4 m and 13.3 m. During core drilling of the bedrock, a careful record of any sudden drops of the drill rods, wash water return and its colour, was kept.

Water levels were measured in the open boreholes on completion of drilling. In addition, long-term groundwater monitoring installations consisting of 13 mm diameter PVC (polyvinyl chloride) pipe were placed in Borehole Nos.1, 2, and 4. The installation configuration is documented on the respective borehole logs. All the boreholes were backfilled upon completion of the fieldwork. The locations and elevations of the boreholes were established by a representative of **exp**. The elevations of the boreholes refer to the geodetic datum.

All the soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified. Similarly, the rock cores retrieved were placed in core boxes visually examined and logged. On completion of the fieldwork, all the soil samples and rock cores were transported to the **exp** laboratory in the City of Ottawa, Ontario

All the soil and rock samples were visually examined in the laboratory by a geotechnical engineer and borehole logs prepared. The engineer also assigned the laboratory testing which consisted of performing natural moisture content, grain size analysis, pH and sulphate content tests on selected soil samples and unconfined compressive strength tests on selected rock cores samples.

5 Subsurface Soil/Bedrock and Groundwater Conditions

A detailed description of the geotechnical conditions encountered in the five boreholes drilled at the site is given on the borehole logs, Figures 3 to 7 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted. Boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

A review of Figures 3 to 7 inclusive indicates that 40 mm to 50 mm of asphaltic concrete was encountered in the vicinity of Borehole Nos. 2 to 4. Beneath the asphaltic concrete in these boreholes, and from the existing ground surface in Borehole Nos. 1 and 5, 600 mm of crusher run limestone sand and gravel was contacted.

The crusher run limestone sand and gravel is underlain by fill in Borehole Nos. 1 to 3 which extends to 2.1 m to 3.0 m depth (Elevation 72.6 m to 73.2 m). The fill comprises of silty sand and gravel with decayed wood pieces, occasional brick pieces, and some organics. It is compact to loose (N values of 2 to 13) and its moisture content varies from 7 to 47 percent.

The fill in Borehole Nos. 1 to 3 and crusher run sand and gravel in Borehole Nos. 4 and 5 are underlain by sand to sand till which extends to refusal depth of 1.8 m to 4.6 m depths in all the boreholes (Elevations 70.1 m to 72.3 m). This stratum comprises of sand to sand till with trace to some gravel and silt. It is loose to compact (N values of 13 to in excess of 50 blows). Its moisture content is 5 to 17 percent. A grain-size analysis performed on the sand stratum yielded a composition of 2 percent silt, 83 percent sand, and 15 percent gravel (Figure 8).

The sand to sand till in Borehole Nos. 1, 2 and 5 is underlain by limestone bedrock which extends to the entire depth investigated, 10.4 m to 13.3 m (Elevation 61.4 m to 63.4 m). The bedrock contains black shale partings and frequent calcite veins. A Total Core Recovery (TCR) and Rock Quality Designation (RQD) of 56 to 100 percent and 86 to 100 percent respectively were determined when core drilling the bedrock. On this basis, the bedrock quality may be classified as very good to excellent.

Unconfined compressive strength tests were performed on selected rock core samples. The test results are presented on Table No. I.

The unit weight of the bedrock was established to vary from 2621 to 2663 kg/m3 whereas its unconfined compressive strength varied from 42.5 to 83.6 MPa indicating that the bedrock is strong to very strong.

Table No. I: Results of Compressive Strength Tests on Rock Cores

Borehole No.	Depth (m)	Unit Weight (kg/m3)	Unconfined Compressive Strength (MPa)
1	11.0 – 11.1	2621	42.5
1	12.8 – 12.9	2663	75.5
5	3.1 – 3.2	2652	83.6
5	6.1 – 6.2	2662	68.8

Water level observations were made in the boreholes during drilling and in standpipes installed in Borehole Nos. 1, 2 and 4 immediately following the completion of drilling. Subsequently, an attempt was made to collect water level measurements from the standpipes installed in the boreholes. However, this was not possible due to prevailing environmental conditions and the presence of ice and snow on top of the well covers. A review of the Phase II Environmental Site Assessment prepared by MMM group for the site was undertaken. It indicates that the groundwater table at the site is at depths of 2.8 m to 3.8 m below the existing ground surface. **Exp** will however collect additional water levels in the standpipes installed in the geotechnical boreholes when the weather improves and access to the wells becomes available.

Water levels were measured in the exploratory boreholes at the times and under the conditions stated in the scope of services. These data were reviewed and **exp**'s interpretation of them discussed in the text of the report. Note that fluctuations in the level of the groundwater may occur due to seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

6 Foundation Considerations

The investigation has revealed that geotechnical conditions at the site are suitable for construction of the proposed nine-storey commercial and residential building on spread and strip footings. Footings of the proposed structure are expected to be founded in the limestone bedrock.

The factored geotechnical resistance at Ultimate Limit State (ULS) of the sound limestone bedrock, competent and free of soil filled seams is expected to be 1000 kPa to 3000 kPa depending on the amount of inspection and testing undertaken during construction. Factored geotechnical resistance at ULS of 1000 kPa would require only visual inspection of the footing beds. The use of factored geotechnical resistance at ULS of 2000 kPa would require star drilling and probing of all the spread and strip footings (minimum 50 mm diameter hole may be used, with its depth equal to at least twice the footing width). The strip footings should be star drilled and probed at 3 m intervals. The use of factored geotechnical resistance at ULS of 3000 kPa would require a full scale load test in addition to star drilling and probing of the footings. The bearing pressure at Serviceability Limit State (SLS) required to produce detrimental settlements of the structure will be much larger than the factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design.

The resistance to sliding of the building footings will be provided by friction between the footing concrete and the limestone bedrock. The resistance to sliding of the foundations may be computed using unfactored ULS friction angle of 30 degrees between the footing and the underlying limestone bedrock. The resulting coefficient of friction is 0.50.

Settlements of footing designed for the above recommended factored geotechnical resistance at ULS are expected to be less than 10 mm.

7 Floor Slab and Drainage Requirements

The lowest level floor of the proposed building may be constructed as slab-on-grade provided it is set on bed of well compacted 19 mm clear stone at least 300 mm thick placed on the natural soil or on well compacted fill. The clear stone would prevent the capillary rise of moisture from the sub-soil to the floor slab. Adequate saw cuts should be provided in the floor slab to control cracking. Any underfloor fill required should conform to OPSS Granular B Type II and should be placed in 300 mm lift thickness and each lift compacted to at least 100 percent of the standard Proctor maximum dry density (SPMDD).

It is recommended that perimeter as well as underfloor drains should be provided for the structure. The underfloor drainage system may consist of 100 mm diameter perforated pipe or equivalent placed in parallel rows at 5 m to 6 m centres and at least 300 mm below the underside of the floor slab. The drains should be set on 100 mm of pea-gravel and covered on top and sides with 150 mm of pea-gravel and 300 mm of CSA Fine Concrete Aggregate (Figure 9). The perimeter drains may also consist of 100 mm diameter of perforated pipe set on the footings and surrounded with 150 mm of pea-gravel and 300 mm of CSA Concrete Aggregate. The perimeter and underfloor drains should preferably be connected to separate sumps so that at least one system would be operations should the other fail.

The finished exterior grade should be sloped away from the building to prevent surface ponding close to the exterior walls.

8 Earth Pressures

The subsurface walls of the proposed structure will be subjected to lateral static earth pressure as well as lateral dynamic earth pressure during a seismic event. The lateral static earth pressure that the subsurface walls would be subjected to may be computed from the following equation; this equation assumes that the groundwater behind the walls will be maintained at the base of the footing by the installation of subdrains and by the use of free draining granular material (OPSS Granular B Type II as backfill material):

$$P_A = ½ K_a \gamma H^2$$

where P_A = lateral active force kN
K_O = at rest earth pressure coefficient = 0.5
γ = unit weight of backfill = 22 kN/m^3
H = height of wall, (m)

The lateral force due to seismic loading may be computed from the equation given below:

$$\Delta P_E = 0.07 \gamma H^2$$

where ΔP_E = resultant force due to seismic activity; acts at 0.6 H from the footing base
γ = unit weight of backfill = 22 kN/m^3
H = height of wall, (m)

9 Excavation

Excavation at the site for three levels of underground parking is expected to extend to a depth of 8 m to 9 m below the existing ground surface. The excavation will extend through the fill and sand stratum into the underlying bedrock. The excavation will be 5 to 6 m below the groundwater table.

The excavation will extend to the property boundary on the east and south side. It will extend within 3.5 m of the Somerset Street curb on the north side and 5 m of the Lebreton Street curb on the west side. The founding levels of the structures situated on the east and south sides of the subject site are not known. Irrespective, with the proposed three basement levels, it is anticipated that these structures will require underpinning if they are not founded on bedrock. The alternative to underpinning would be to support the excavation along the east and south sides with a rigid continuous reinforced concrete secant wall which is designed to support the lateral earth pressure as well as the footing loads. This wall would have to be socketed into bedrock and tied back using rock anchors. It should be designed to support "at rest" earth pressure as well as full hydrostatic pressure. If the adjacent buildings are founded on bedrock, conventional steel H soldier piles and timber lagging shoring system is expected to suffice.

The lateral earth pressure and hydrostatic pressures that the reinforced concrete secant wall should be designed to support may be obtained from expression (i) given below:

$$P = k\,(\gamma h+q)+ \gamma_w h_w \qquad (i)$$

The conventional steel H soldier piles and timber lagging shoring system must be designed to support the lateral earth pressures given by the expression (ii). In this case, the hydrostatic pressure may be eliminated since drainage can be permitted between the lagging system:

$$p = k(\gamma h + q) \qquad (ii)$$

where p = the pressure, at any depth, h, below the ground surface

k = earth pressure coefficient applicable

γ = unit weight of soil to be retained, estimated at 21 kN/m3

h = the depth, in metres, at which pressure, p, is being computed

q = the equivalent surcharge acting on the ground surface adjacent to the shoring

γ_w = unit weight of water

h_w = height of groundwater table above excavation base

The earth pressure coefficient, k, may vary between the following limits:

0.25 where adjacent building footings or settlement-sensitive services lie below a 45 degree line drawn up from the toe of the excavation.

0.35	where adjacent building footings or settlement-sensitive services lie below a 60 degree line to the horizontal drawn up from the toe of the excavation.
0.5	where adjacent building footings or settlement-sensitive services lie above a 60 degree line to the horizontal drawn up from the base of the toe of the excavation.

It may be possible to undertake excavation on the north and west sides as open cut provided that any structures or services are not located under the street adjacent to the site and if spaces allow this. Alternatively, the excavation on the north and west sides may have to be shored. The shoring may consist of steel H solider piles and timber lagging and designed based on earth pressure given by expression (ii).

The shoring system should be designed using appropriate K values depending on the location of any settlement-sensitive services under the streets. The traffic loads on the streets should be considered as surcharge. If rock excavation extends to the property boundary, it would be necessary to toe the soldier piles into the sound rock below the excavation base. If there is room to permit at least 0.7 m of rock ledge around the perimeter, the soldier piles could be toed into the upper levels of the rock provided that a rock bolts and plate arrangement is installed on the rock face to support the toe. The rock bolts should be designed to take the full toe pressure.

The rigid reinforced concrete secant wall or the soldier piles should be tied back by anchors grouted into the sound bedrock. Based on the excavation depth proposed and the overburden depth encountered, it is anticipated that one set of anchors may suffice. A maximum allowable grout to rock bond of 700 kPa (100 psi) may be used for design of the anchors. A minimum free anchor length of 3 m is recommended. It is anticipated that the bedrock in the lower levels is relatively tight and therefore the grout loss would be minor when grouting anchors in the bedrock. The grout loss may be higher in fractured bedrock which may be present in the upper levels of the bedrock at the site..

Selected anchors should be load tested to two times the design capacity. All anchors should be proof tested to 1.33 times the working load. The anchors should be locked off at working load plus an allowance for relaxation (usually 10%). When installing the tiebacks, casing would be required to advance through the fill, silty sand and any fractured rock. The deflection of the shoring should be carefully monitored during construction on a weekly basis.

Depending on the condition of the bedrock, stabilization of the excavated rock face may be required by rock bolts and wire mesh. The spacing of rock bolts, if required, will depend on the block sizes in the rock mass. Based on the borehole information, a rock bolt spacing of 1 ½ to 2 m center to center, in the vertical direction and 1 ½ m in the horizontal direction is suggested subject to modification base on field inspections. Rock bolts must extend into the rock at least 2 m. In some weathered areas, closer spacing, longer bolts, and surface treatment may be required to stabilize the rock mass.

Excavation of the overburden soils may be undertaken by conventional mechanical equipment. The majority of the soil to be excavated from the site is fill including decayed wood pieces, cobbles and boulders. Excavation of the limestone bedrock would require the use of blasting. In order to prevent any damage to the surrounding structures and services, the blasting operations would have to be carefully planned and closely monitored. It is recommended that the blasting contractor should retain the services of a blast specialist to provide him with a blasting plan. The contractor should have a licensed blaster on site at all times during the blasting and a vibrations engineer on retainer. Prior to commencement of the blasting operations, the contractor's proposed plan should be reviewed by this office. A condition survey of all the structures and services in the vicinity of the site should be undertaken prior to commencement of the blasting work.

Vibration monitoring should be carried out in the adjacent structures during blasting operations. Vibrations should be monitored at property boundaries and should be limited so that there will be no damage to the existing structures. As a guideline for blasting, Table No. II presents the recommended maximum peak particle velocities generated during blasting for various types of structures.

Table No. II: Recommended Limits of Peak Particle Velocities Generated during Blasting

Building Category	Particle Velocity (mm/sec)
New Construction	50
Residential, Poor to Very Poor Condition	25 - 12.5
Historic Building	2 - 3

Water inflow into the excavation should be expected. However, it should be possible to adequately handle this inflow by collecting the water in perimeter ditches and pumping from properly filtered sumps. It is possible that additional localized sumps may be required in areas where the seepage is more extensive. It is noted that permit to take water may be required from Ontario Ministry of the Environment if the quantity of water to be pumped exceeds 50,000 litres per day.

10 Backfilling and Drainage Requirements for Subsurface Foundation Walls

The soils to be excavated from the site will comprise of existing fill, silty sand and silty sand till. These materials are not considered suitable for backfilling purposes at the site as they are not free draining and should be discarded as per recommendation of the Phase II ESA prepared for the site by MMM Group. Therefore, all the fill required to backfill footing trenches, service trenches, construction of ramps, etc. and against the subsurface walls would have to be imported and should preferably conform to the Ontario Provincial Standard Specifications (OPSS) requirements for Granular B, Type II. It should be placed in 300 mm lifts and compacted to 95 percent of the SPMDD against the subsurface walls and to 98 percent SPMDD under bearing surfaces.

In areas where the foundation walls are formed on both sides, the walls should be damp-proofed and backfilled with conventional free draining granular material preferably conforming to the OPSS for Granular B, Type II. The granular backfill should be placed in layers not exceeding 300 mm in thickness and compacted to 95 percent of SPMDD.

Where basement walls will be poured against the bedrock or temporary shoring, vertical drainage board must be installed on the face of the excavation wall or lagging to provide the necessary drainage. Vertical drains such as Alidrain, Geodrain or Miradrain may be used for this purpose (Figure 10). At least one vertical drain should be installed every 2.5 m. As an added protection, full coverage using the drainage boards can be considered in order to minimize the risk of water penetration through the walls.

Depending on the groundwater inflow after excavation, additional drains may be required locally. Where the upper portion of the wall is backfilled with granular material, the vertical drains should extend into this backfill to provide drainage of the backfill. The vertical drains could be connected to a solid drain placed inside the building close to the foundation wall and leading to a storm sump in the interior of the building. The lateral earth pressures on the vertical drains may be calculated using the formula presented in the previous section and using a K value at rest of 0.5.

11 Site Classification for Seismic Site Response

Shear wave velocity measurements were undertaken at the site by Geophysics GPR International Inc. and the results are presented in Appendix A.

A review of the report indicates that the shear wave velocity to 30 m depth is 960 m/s and therefore the site Class would be classified as Class B in accordance with Table 4.1.8.1A of the Ontario Building Code, 2006. However, since the proposed building would be set below 8 m to 9 m depth, the average shear wave from this level to 30 m depth was computed to be in excess of 1500 m/s. Therefore, in this case the site has been classified as Class A for the proposed structure with three levels of underground parking. It is noted that this classification may change if the founding level of the structure is different than assumed. If this is the case, this office should be consulted to review the site classification.

12 Subsurface Concrete Requirements

Chemical tests limited to pH and sulphate tests were performed on three selected soil/rock samples and the results are given on Table No. III.

Table No. III: Chemical Test Results

Borehole No.	Depth	PH	Sulphate (%)
3 – Soil	3.0 – 3.6	7.9	<0.01
1 – Rock	11.0 – 11.1	8.6	0.02
5 - Rock	6.1 – 6.2	8.7	0.01

The test results indicate the founding rock contains a sulphate content of less than 0.1 percent. This concentration of sulphates in the rock would have a negligible potential of sulphate attack on subsurface concrete. The concrete for the site should be designed in accordance with the requirements of CSA A23.1-09.

13 Pavement Structure Thickness

It is currently not known whether the ramp and the lowest level floor of the garage would be paved or will be constructed as a concrete pavement. If it is paved, the pavement structure may consist of 40 mm surface coarse (HL3), 40 mm base coarse (HL8), 150 mm of Granular A base and 300 mm of Granular B Type II sub-base. The asphalt cement recommended for this site is PG 58-34. If concrete is used, this office should be contacted for additional recommendation.

The upper 300 mm of the subgrade fill should be compacted to 98 percent of standard Proctor maximum dry density.

The granular materials used for pavement construction should conform to OPSS for Granular A and Granular B Type II and should be compacted to 100 percent of standard Proctor maximum dry density. The asphaltic concrete used and its placement should meet OPSS 1150/1151 requirements. It should be placed and compacted to OPSS 311 and 313.

14 Additional Investigation

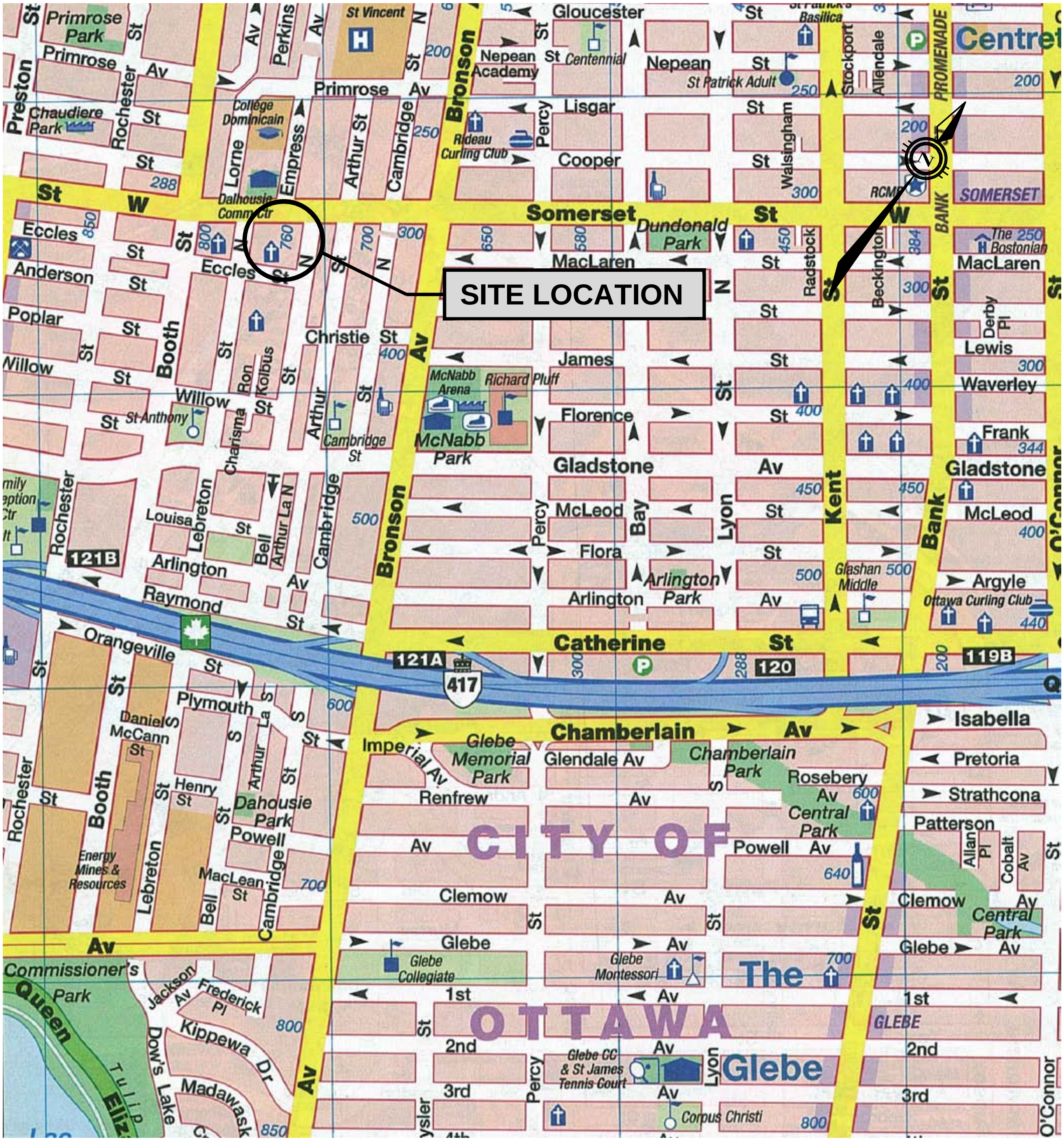
It is recommended that test pits should be excavated adjacent to the exterior walls of the structures located on the east and south sides in order to determine the founding stratum and depth since this information will influence the support system required for the excavation at the site.

15 General Closure

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions, between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well, as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

We trust that the information contained in this report will be satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Figures



0 100m 200m 400m

HORIZONTAL 1:10,000

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scale 1:10,000	CLIENT: PHOENIX HOMES	project no. OTT-00210537-A0
date JAN. 2013	TITLE: SITE LOCATION PLAN 770 SOMERSET ST., OTTAWA, ON	FIG 1
drawn by M. N.		

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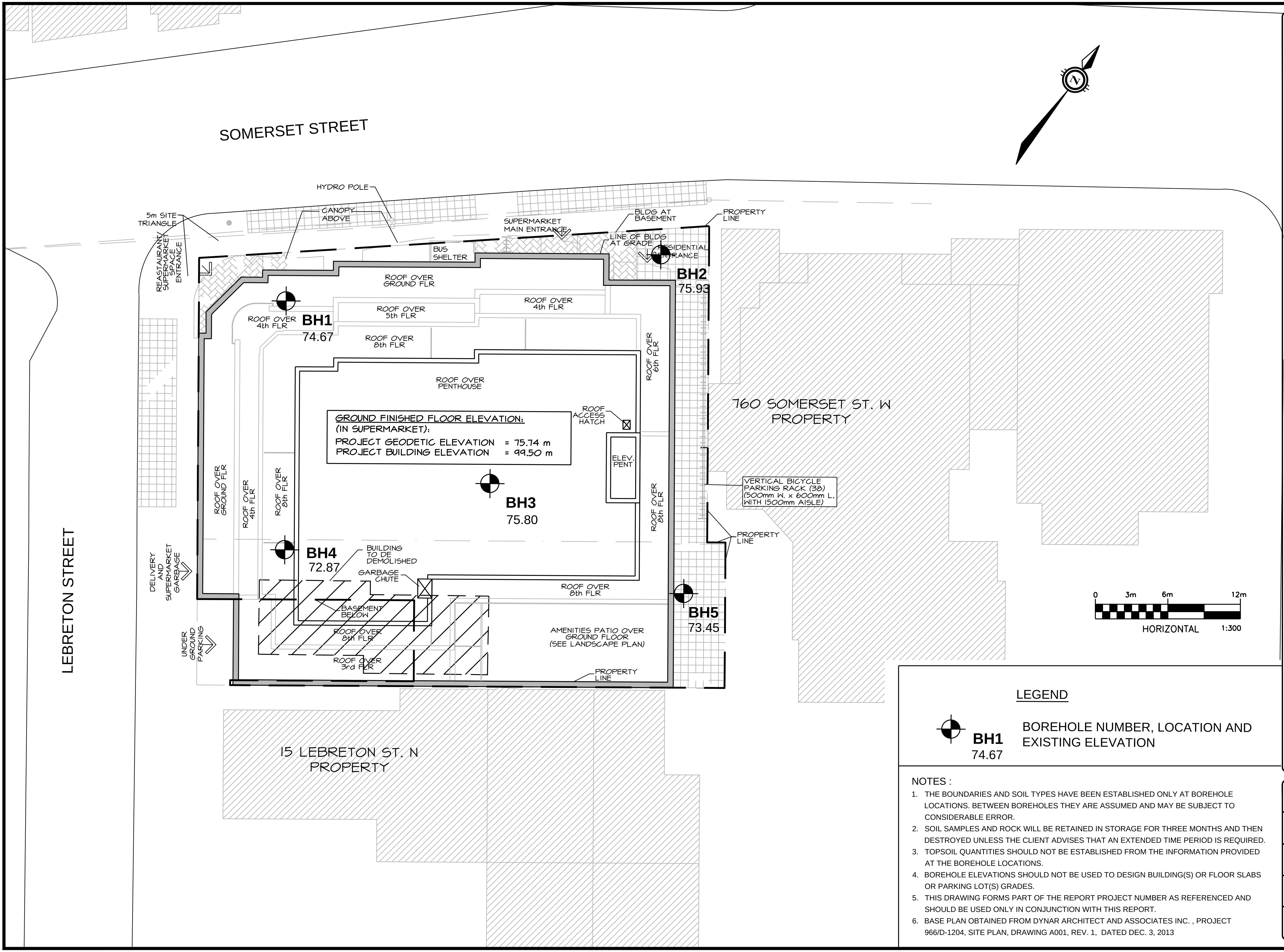

LEGEND

BH1 74.67 — BOREHOLE NUMBER, LOCATION AND EXISTING ELEVATION

NOTES :

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE PLAN OBTAINED FROM DYNAR ARCHITECT AND ASSOCIATES INC. , PROJECT 966/D-1204, SITE PLAN, DRAWING A001, REV. 1, DATED DEC. 3, 2013

exp Services Inc.
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www.exp.com

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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

CLIENT: PHOENIX HOMES

TITLE: BOREHOLE LOCATION PLAN
770 SOMERSET ST., OTTAWA, ON

scale 1:300
date JAN. 2013
drawn by M. NUGENT
project no. OTT-00210537-A0

Figure 2

Filename: r:\210000\210000\210537-a0 mixed use bldg lebreton and somerset\210537-a0 fig 1-2 rev.dwg
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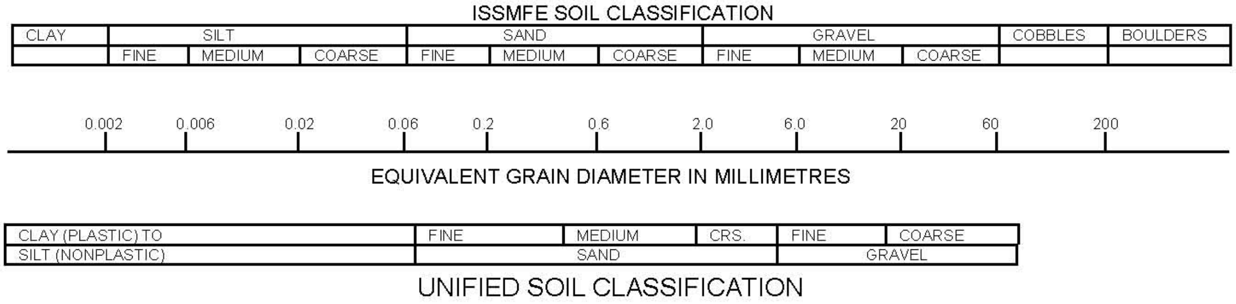
Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Societv for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by **exp** Services Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Log of Borehole BH 1

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Location: 770 Somerset Street West, City of Ottawa, Ontario

Figure No. 3

Page. 1 of 2

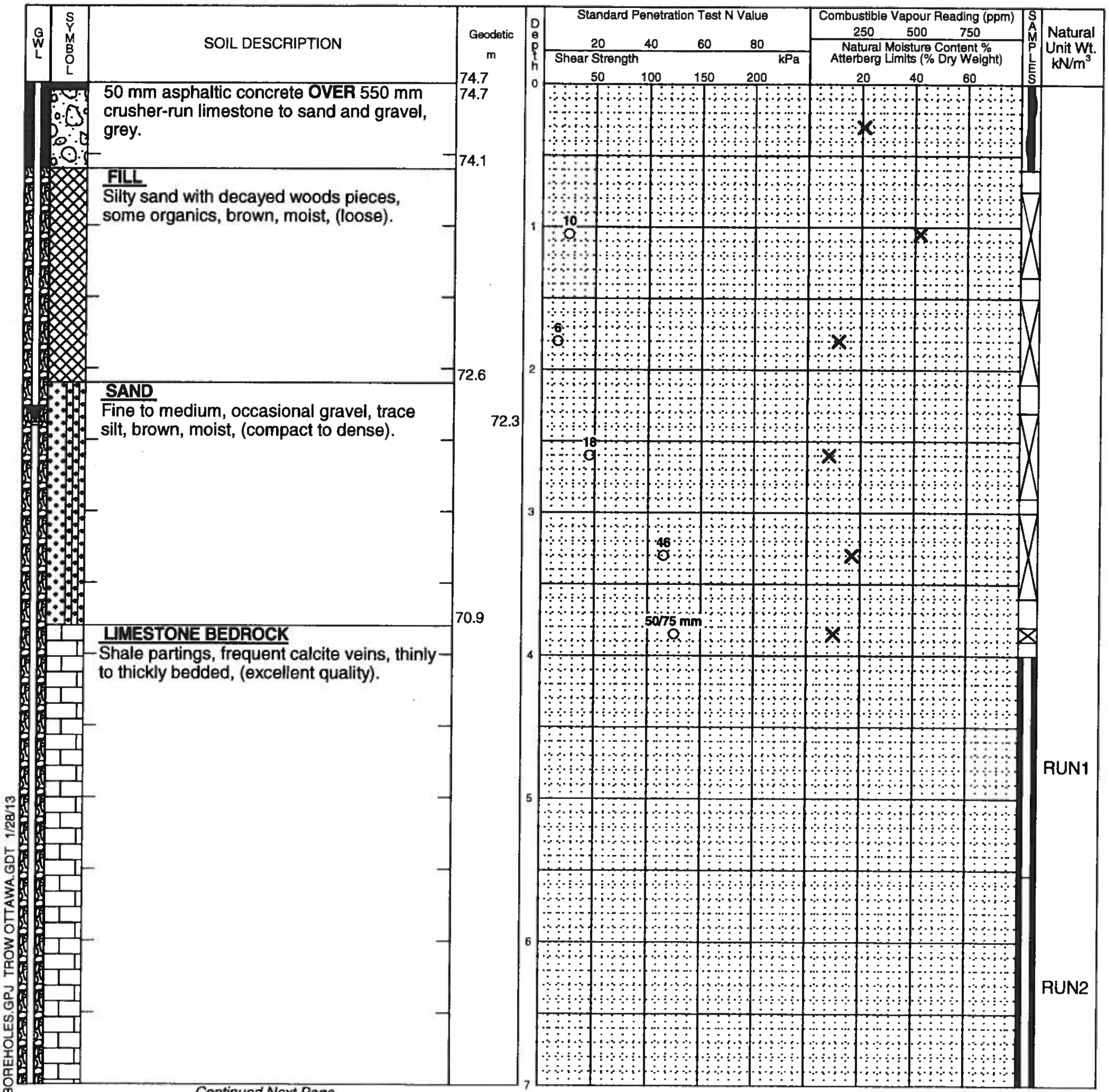
Date Drilled: 'January 9, 2013

Drill Type: CME-55 Truck Mount

Datum: Geodetic

Logged by: A. Neguss   Checked by: S.A.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	▥	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



50 mm asphaltic concrete **OVER** 550 mm crusher-run limestone to sand and gravel, grey.

FILL
Silty sand with decayed woods pieces, some organics, brown, moist, (loose).

SAND
Fine to medium, occasional gravel, trace silt, brown, moist, (compact to dense).

LIMESTONE BEDROCK
Shale partings, frequent calcite veins, thinly to thickly bedded, (excellent quality).

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13mm slotted standpipe was installed in the Borehole Upon Completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	2.4	13.3

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	4 - 5.5	100	100
2	5.5 - 7.1	100	100
3	7.1 - 8.7	97	97
4	8.7 - 10.2	99	99
5	10.2 - 11.6	100	100
6	11.6 - 13.3	99	99

Log of Borehole BH 1

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Figure No. 3

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 / Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m³
		LIMESTONE BEDROCK Shale partings, frequent calcite veins, thinly to thickly bedded, (excellent quality). *(continued)*	67.7	7				
				8				RUN3
				9				RUN4
				10				
				11				RUN5
				12				RUN6
				13				
		Borehole Terminated at 13.3 m Depth	61.4					

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13mm slotted standpipe was installed in the Borehole Upon Completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	2.4	13.3

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	4 - 5.5	100	100
2	5.5 - 7.1	100	100
3	7.1 - 8.7	97	97
4	8.7 - 10.2	99	99
5	10.2 - 11.6	100	100
6	11.6 - 13.3	99	99

Log of Borehole BH 2

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Location: 770 Somerset Street West, City of Ottawa, Ontario

Figure No. 4

Page. 1 of 2

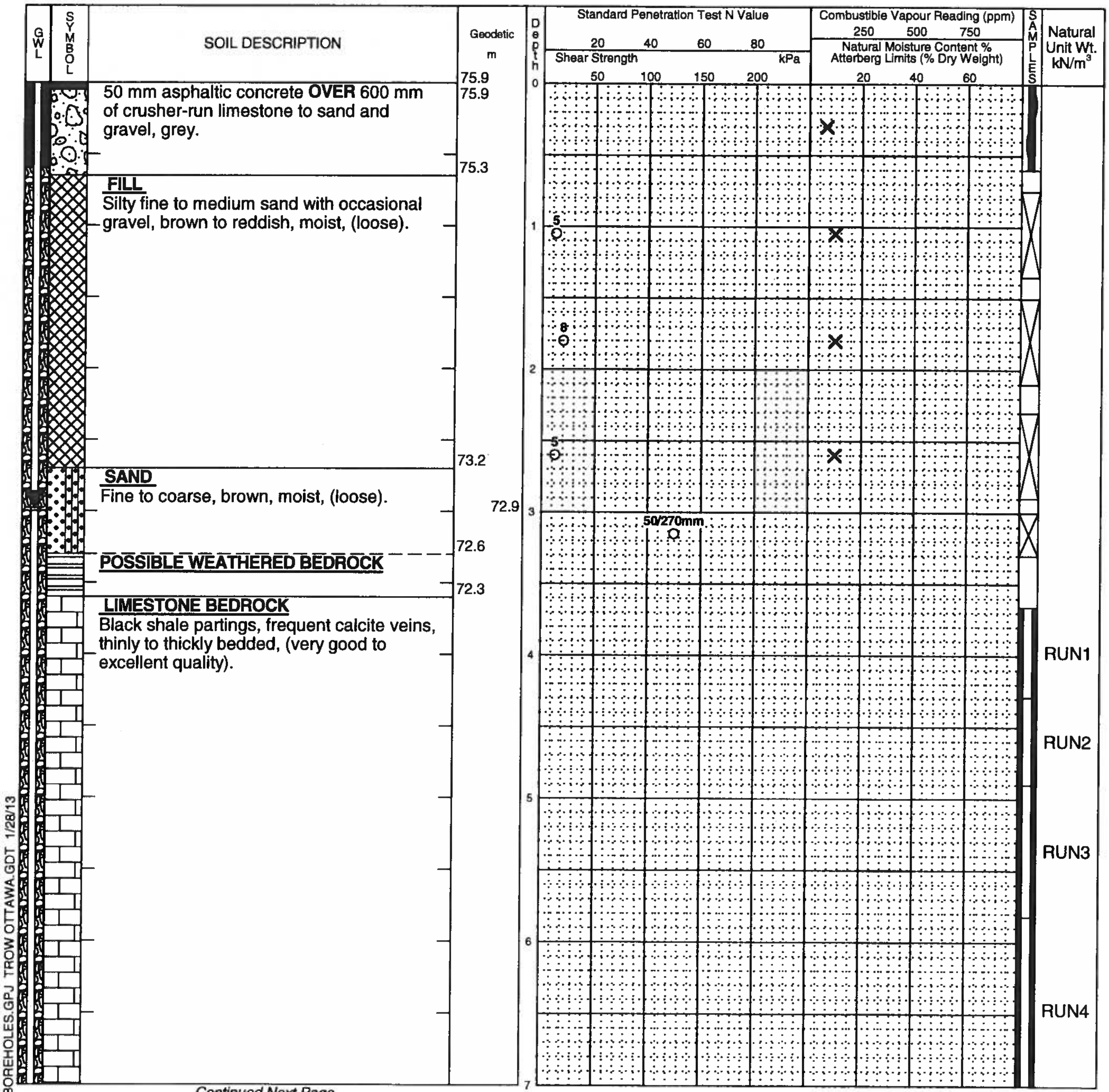
Date Drilled: 'January 9, 2013

Drill Type: CME-55 Truck Mount

Datum: Geodetic

Logged by: A. Neguss Checked by: S.A.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



50 mm asphaltic concrete **OVER** 600 mm of crusher-run limestone to sand and gravel, grey.

FILL
Silty fine to medium sand with occasional gravel, brown to reddish, moist, (loose).

SAND
Fine to coarse, brown, moist, (loose).

POSSIBLE WEATHERED BEDROCK

LIMESTONE BEDROCK
Black shale partings, frequent calcite veins, thinly to thickly bedded, (very good to excellent quality).

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13mm slotted standpipe was installed in the Borehole Upon Completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	3.0	12.6

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.7 - 4.3	98	98
2	4.3 - 4.9	96	96
3	4.9 - 5.8	86	86
4	5.8 - 7.1	100	100
5	7.1 - 8.7	95	95
6	8.7 - 10.3	95	95
7	10.3 - 11.9	99	99

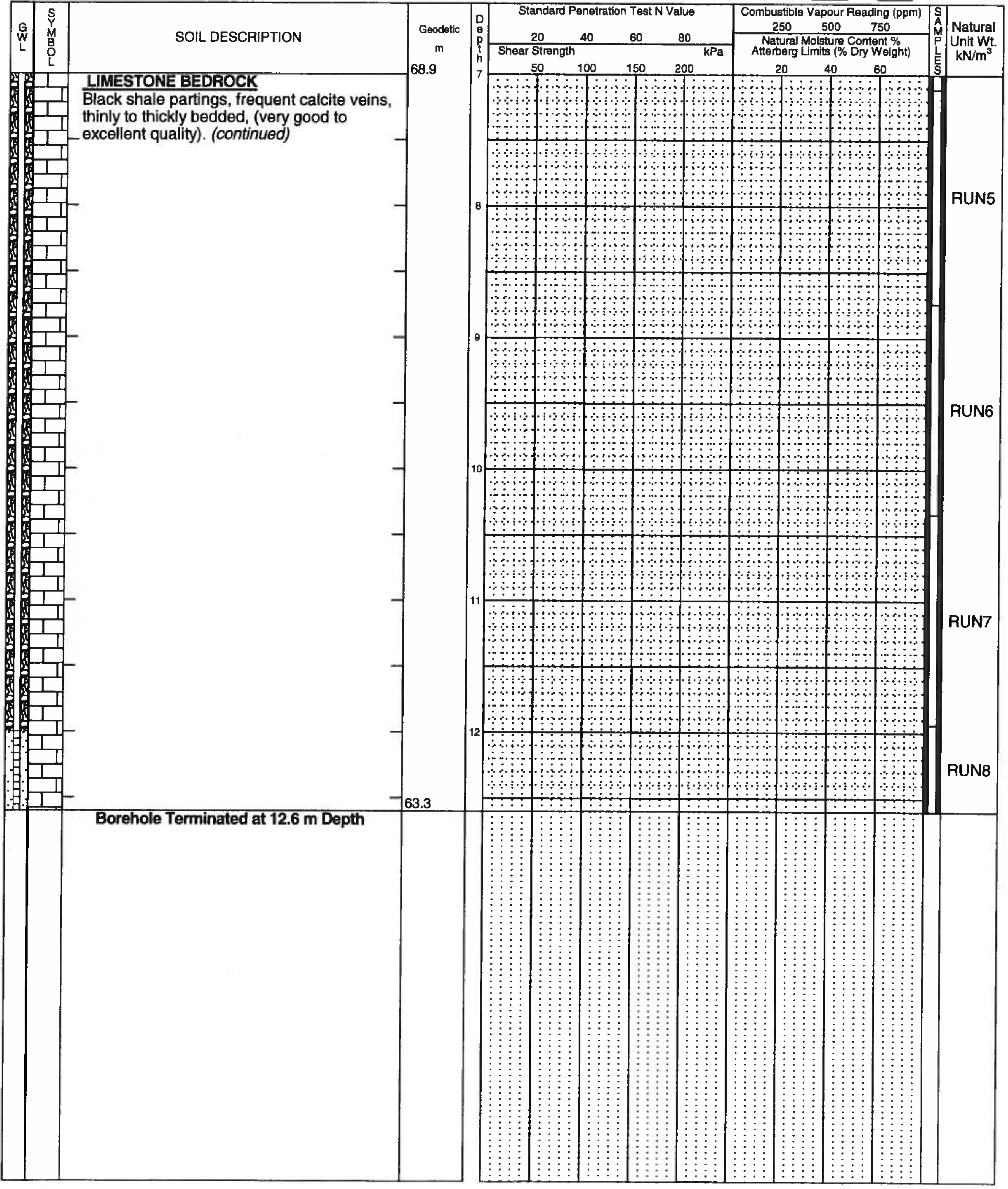
Log of Borehole BH 2

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Figure No. 4

Page. 2 of 2



LIMESTONE BEDROCK

Black shale partings, frequent calcite veins, thinly to thickly bedded, (very good to excellent quality). *(continued)*

Borehole Terminated at 12.6 m Depth

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13mm slotted standpipe was installed in the Borehole Upon Completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	3.0	12.6

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.7 - 4.3	98	98
2	4.3 - 4.9	96	96
3	4.9 - 5.8	86	86
4	5.8 - 7.1	100	100
5	7.1 - 8.7	95	95
6	8.7 - 10.3	95	95
7	10.3 - 11.9	99	99

Log of Borehole BH 3

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Location: 770 Somerset Street West, City of Ottawa, Ontario

Figure No. 5

Page. 1 of 1

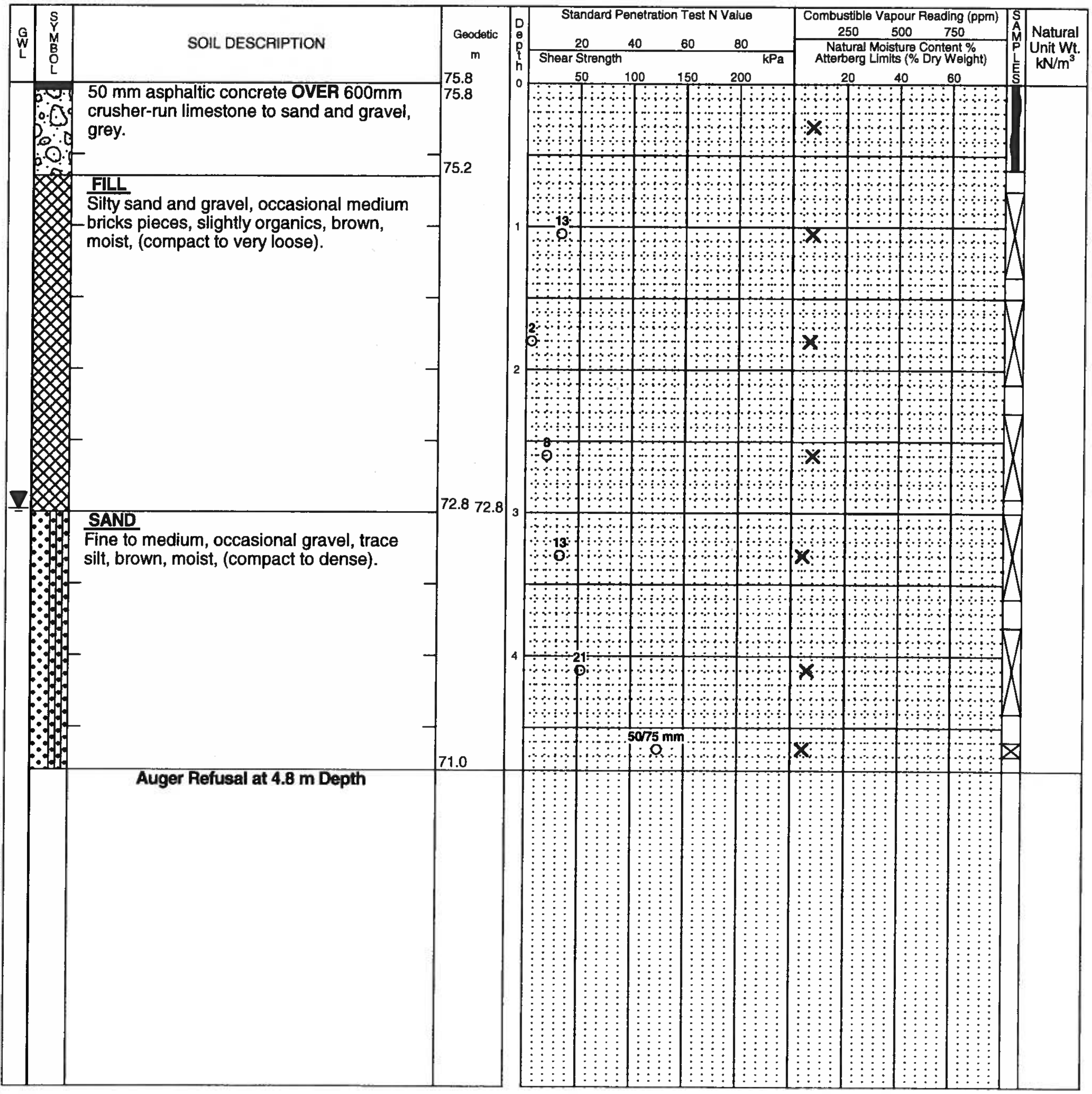
Date Drilled: January 9, 2013

Drill Type: CME-55 Truck Mount

Datum: Geodetic

Logged by: A. Neguss Checked by: S.A.

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



50 mm asphaltic concrete **OVER** 600mm crusher-run limestone to sand and gravel, grey.

FILL
Silty sand and gravel, occasional medium bricks pieces, slightly organics, brown, moist, (compact to very loose).

SAND
Fine to medium, occasional gravel, trace silt, brown, moist, (compact to dense).

Auger Refusal at 4.8 m Depth

NOTES:

1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled Upon Completion of drilling
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	3.0	4.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 4

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Location: 770 Somerset Street West, City of Ottawa, Ontario

Figure No. 6

Page. 1 of 1

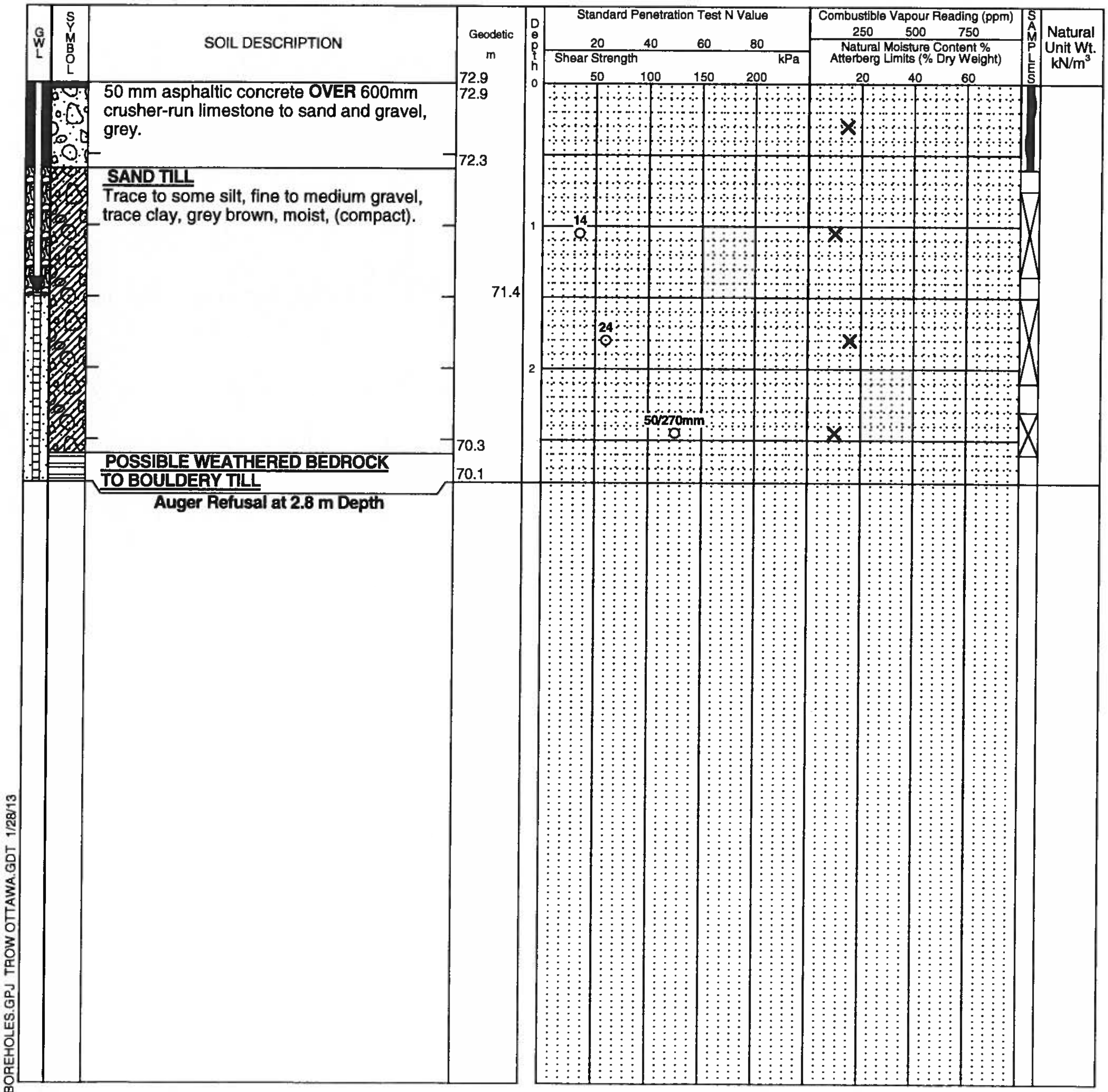
Date Drilled: 'January 10, 2013

Drill Type: CME-55 Truck Mount

Datum: Geodetic

Logged by: A. Neguss Checked by: S.A.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	▥	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



50 mm asphaltic concrete **OVER** 600mm crusher-run limestone to sand and gravel, grey.

SAND TILL

Trace to some silt, fine to medium gravel, trace clay, grey brown, moist, (compact).

POSSIBLE WEATHERED BEDROCK TO BOULDERY TILL

Auger Refusal at 2.8 m Depth

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13mm slotted standpipe was installed in the Borehole Upon Completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	1.5	2.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 5

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Location: 770 Somerset Street West, City of Ottawa, Ontario

Figure No. 7

Page. 1 of 2

Date Drilled: 'January 10, 2013

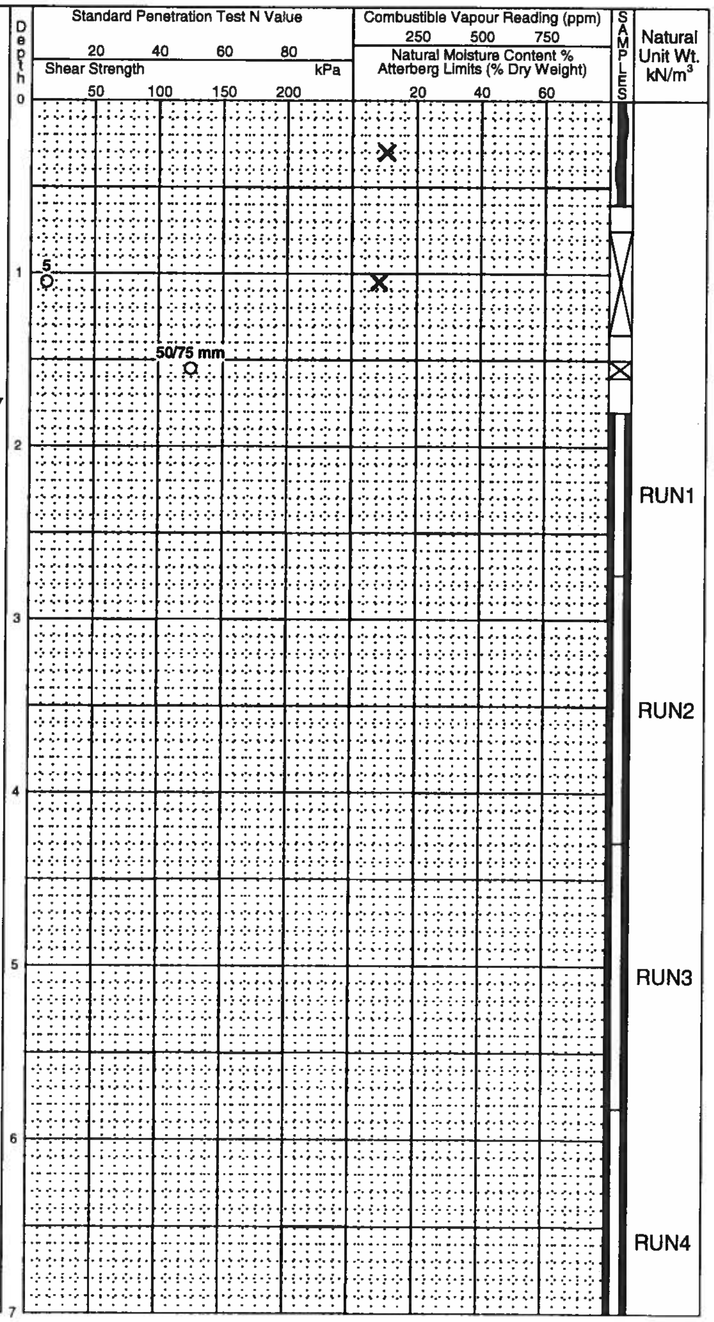
Drill Type: CME-55 Truck Mount

Datum: Geodetic

Logged by: A. Neguss Checked by: S.A.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	▥	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SOIL DESCRIPTION	Geodetic m
		73.5
	40 mm asphaltic concrete **OVER** 560mm crusher-run limestone to sand and gravel, grey.	73.5
	SAND TILL Fine to coarse with some fine to medium gravel, brown, moist, (loose to compact).	72.9
▼	**POSSIBLE WEATHERED BEDROCK TO BOULDERY TILL**	71.9
	LIMESTONE BEDROCK Grey, black shale partings, frequent calcite veins, thinly to thickly bedded, (very good to excellent quality).	71.7 71.7



Continued Next Page

NOTES:

1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled Upon Completion of drilling
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	1.8	10.4

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	1.8 - 2.7	95	87
2	2.7 - 4.3	95	95
3	4.3 - 5.8	100	100
4	5.8 - 7.3	100	100
5	7.3 - 8.9	98	98
6	8.9 - 10.4	96	96

Log of Borehole BH 5

exp.

Project No: OTT-00210537-A0

Project: Geotechnical investigation - Proposed Multi-Use Nine storey Building

Figure No. 7

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m³
		LIMESTONE BEDROCK Grey, black shale partings, frequent calcite veins, thinly to thickly bedded, (very good to excellent quality). *(continued)*	66.5	7 8 9 10				RUN5 RUN6
			63.1					
		Borehole Terminated at 10.4 m Depth						

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled Upon Completion of drilling
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00210537-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Upon completion	1.8	10.4

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	1.8 - 2.7	95	87
2	2.7 - 4.3	95	95
3	4.3 - 5.8	100	100
4	5.8 - 7.3	100	100
5	7.3 - 8.9	98	98
6	8.9 - 10.4	96	96

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Ottawa, ON   K2B 8H6

Method of Test for Particle Size Analysis of Soil

ASTM C-136

Grain Size Distribution Curve

<table>
<tr><td rowspan="2">CLAY</td><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Medium</td><td>Coarse</td><td rowspan="2"></td></tr>
<tr><td colspan="3">SILT</td><td colspan="3">SAND</td><td colspan="3">GRAVEL</td></tr>
<tr><td colspan="11">Modified M.I.T. Classification</td></tr>
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<table>
<tr><td>Trow Project No.:</td><td>OTT-000210537-AO</td><td>Project Name :</td><td colspan="5">Geotechnical Investigation - Proposed Mixed Used Development</td></tr>
<tr><td>Client :</td><td>Phoenix Homes</td><td>Project Location :</td><td colspan="5">770 Somerset Street West, City of Ottawa, Ontario</td></tr>
<tr><td>Date Sampled :</td><td>Jnauary 10, 12</td><td>Borehole No.:</td><td>BH 4</td><td>Sample No.:</td><td>SS3</td><td>Depth (m) :</td><td>1.5 to 2.1</td></tr>
<tr><td>Sample Description :</td><td colspan="5">Sand, Some Gravel, Trace Silt</td><td>Figure :</td><td>8</td></tr>
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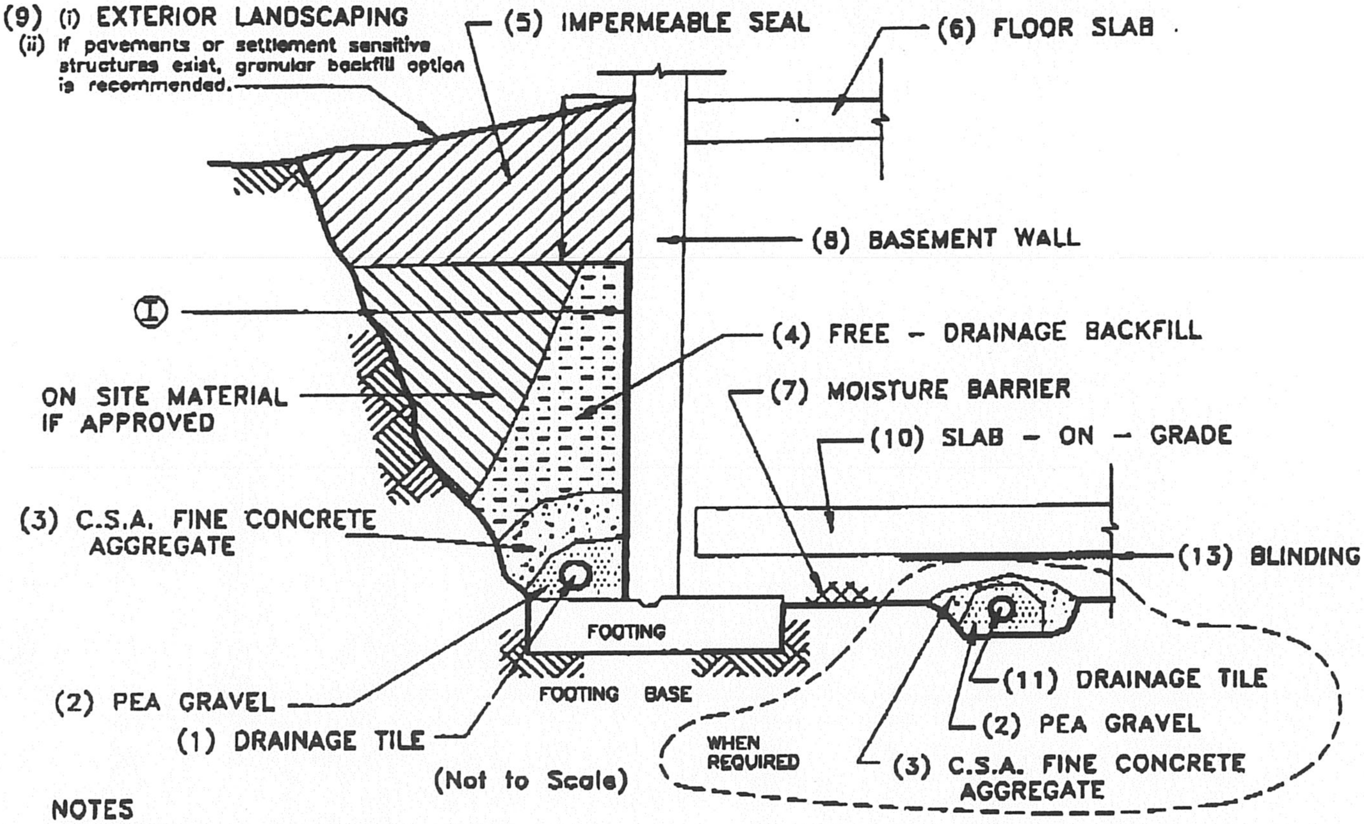
Figure 9

BASEMENT DRAINAGE DRAWING



(Not to Scale)

NOTES

OPTION A – GRANULAR BACKFILL

1. Drainage tile to consist of 100mm (4 in.) diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 150mm (6 in.) below underside of floor slab.
2. Pea gravel 150mm (6 in.) top and sides of drain. If drain is not on footing, place 100mm (4 in.) of pea gravel below drain. 20mm (3/4 in.) clear stone may be used provided it is covered by an approved porous geotextile membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filter material. Miniumum 300mm (12 in.) top and sides of drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).
4. Free-draining backfill – OPSS Granular B or equivalent compacted to 93 to 95 (maximum) percent Standard Proctor density. Do not compact closer than 1.8m (6 ft.) from wall with heavy equipment. Use hand controlled light compaction equipment within 1.8m (6 ft.) of wall.
5. Impermeable backfill seal of compacted clay, clayey silt or equivalent. If original soil is free-draining seal may be omitted.
6. Do not backfill until wall is supported by basement and floor slabs or adequate bracing.
7. Moisture barrier to consist of compacted 20mm (3/4 in.) clear stone or equivalent free-draining material. Layer to be 200mm (8 in.) minimum thickness.
8. Basement walls to be damp-proofed.
9. Exterior grade to slope away from wall.
10. Slab-on-grade should not be structurally connected to wall or footing.
11. Underfloor drain invert to be a least 300mm (12 in.) below underside of floor slab. Drainage tile placed in parallel rows 6 to 8m (20 to 25ft.) centres one way. Place drain on 100mm (4 in.) of pea gravel with 150mm (6 in.) of pea gravel top and sides. CSA fine concrete aggregate to be provided as filter material or an approved geotextile membrane (as in 2 above) may be used.
12. Do not connect the underfloor drains to perimeter drains.
13. If the 20mm (3/4 in.) clear stone requires surface blinding, use 6mm (1/4 in.) clear stone chips.

NOTE: A) Underfloor drainage can be deleted where not required (see report).

OPTION B – CORE DRAIN

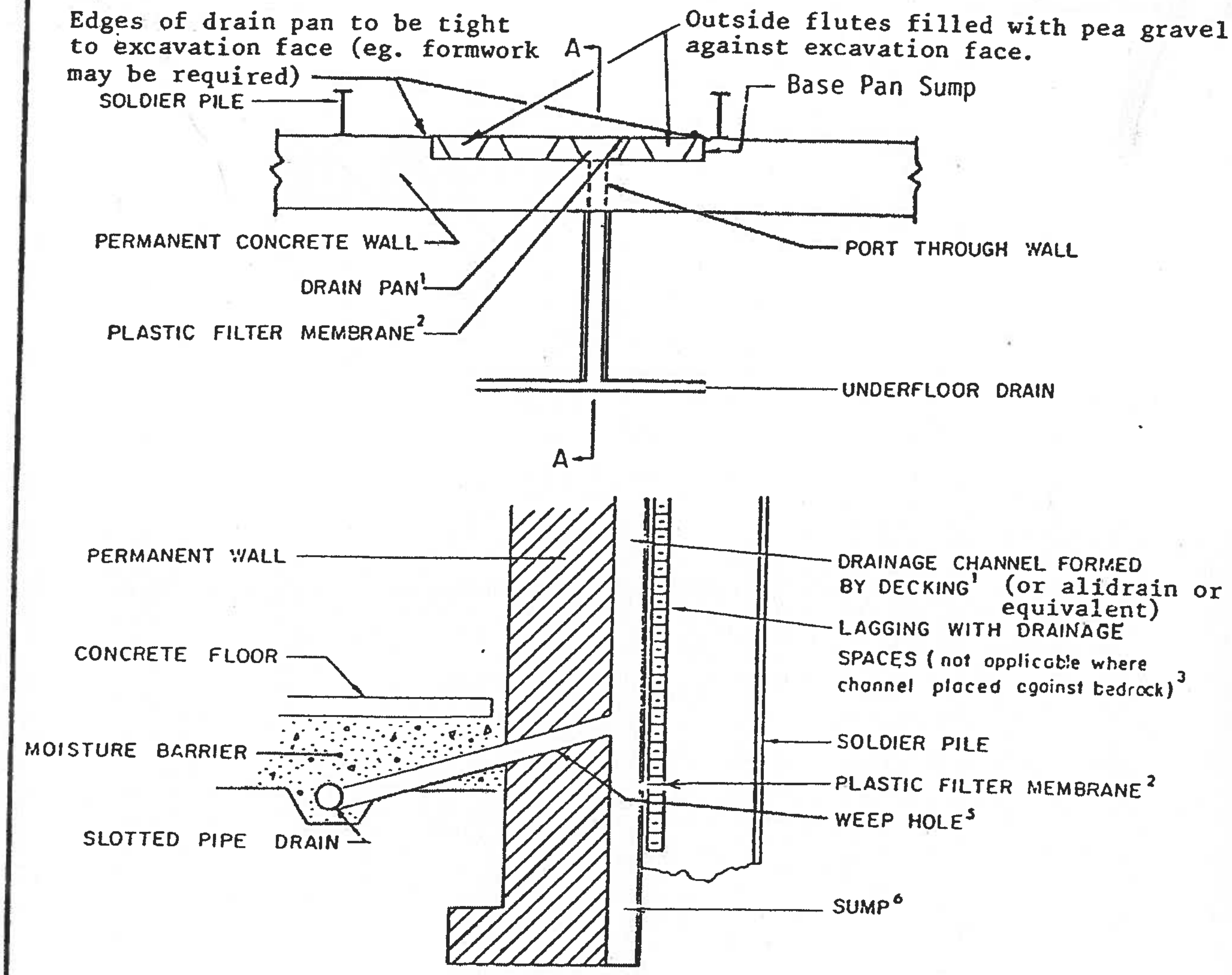
Prefabricated continuous wall drains Ⓘ may be installed and Zone 4 backfilled with on site material compacted to 93 – 95% proctor. Further cost savings may result by placing the wall drains at equal distance strips no greater then 2.5m spacing but the risks of water leakage must by assessed and then assumed by the client.

1. Wall drain option Ⓘ may increase the lateral pressures above those of the conventional detail.
2. The use of waterproofing details at construction and expansion joints may also be required.
3. For Block walls or unreinforced cast in place concrete, the granular backfill option is recommended

Note: If water table exists above the floor slab, then options of granular in combinations with the wall drain should be reviewed

Figure 10

PROJECT:-



SECTION A–A

NOTES:

1. The drain pan should consist of 610 mm (24") wide 20 gauge corrugated galvanized roof deck fastened to lagging. Continuous from bottom of wall to 1.2 m (4') from surface unless otherwise stated. Top and bottom to be capped to exclude soil. Base cap to form sump indicated in note 6 below.
2. Filter membrane to be placed against lagging to exclude fines. Use Terrafix 270R or equivalent.
3. Lagging boards must allow drainage. Spaces should be packed with straw to prevent loss of ground.
4. Drainage pans to be spaced at maximum 4.9 m (16') centres and at obvious wet spots. Crack control joints should be placed at the drain pan locations.
5. Weep hole through concrete wall. Under floor drain (100 mm diameter slotted PVC pipe or equivalent) required to pick up flow from all channels and lead to positive frost-free sump or outlet.
6. Sump provided to collect silt and calcite deposits. Use galvanized metal base pan at bottom of roof deck drain.
7. Cleanouts required for weep hole and slotted pipe drain.

SUGGESTED EXTERIOR DRAINAGE
CONSTRUCTION AGAINST PROPERTY LINE

Appendix A:
Shear-Wave Velocity Sounding by Geophysics GPR International Inc.

GEOPHYSICS GPR INTERNATIONAL INC.

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January 16th, 2013

Transmitted by email: ismail.taki@exp.com
Our Ref.: M-12531C

Mr. Ismail Taki, M.Eng., P.Eng.
exp inc.
100-2650 Queensview Drive
Ottawa, ON
K2B 8H6

Subject: **Shear-wave Velocity Sounding, Somerset St. W & Lebreton St. N, Ottawa**

Dear Mr. Taki,

Geophysics GPR International Inc. has been requested by **exp** inc. to carry out a seismic shear wave velocity sounding on a parking lot, located at the intersection of Somerset Street W and Lebreton Street N, in Ottawa. The geophysical investigations utilized the Multi-channel Analysis of Surface Waves (MASW) and the Extended SPatial AutoCorrelation (ESPAC) methods. Due to the spatial limitation, the surveys were complemented by some basic seismic refraction measurements. From these results, seismic shear wave velocities were calculated for the overburden and the rock, and the V_{S30} value was calculated to identify the Site Class.

The surveys were carried out on December 21st, 2012, by Mr. Nicolas Beaulieu, Jr. Eng. and Mr. Charles Trottier, M.Sc., phys. Figure 1 shows the regional location of the site, and Figure 2 illustrates with more details the location of the seismic spread in surface. Both figures are presented in the appendix.

The following paragraphs briefly describe the survey design, the principles of the test method, and the results in graphic and table format.

MASW Survey

Method Principle

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended Spatial Autocorrelation* (ESPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves ("ground roll"). The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones spread axis. Conversely, the ESPAC is considered a "passive" method, using the low frequency "noises" produced far away. The dispersion properties are measured as a change in phase velocity with frequency. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding). Figure 3 outlines the basic operating procedure for the MASW method.

Figure 4 is an example image of one of the MASW records and resulting 1D V_S model. The ESPAC method allows deeper Vs soundings, but usually with a poor resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to permit a more complete inversion of the data. A detailed description of the MASW method can be found in the paper *Multi-channel Analysis of Surface Waves*, Park, C.B., Miller, R.D. and Xia, J. Geophysics, Vol. 64, No. 3 (May-June 1999); p. 800–808. For the ESPAC method, one can refer to the paper *Shear Velocity Profiles Obtained from Microtremor Array Data with an Example from Direct Fitting of SPAC Curves*, Asten, M.W., 2007, Proceedings of the 20th SAGEEP Conference, Denver, Environmental and Engineering Geophysical Society, and for more details: *The Microtremor Survey Method*, Okada, H., S.E.G., Geophysical Monograph Series No. 12.

Interpretation Method for the MASW surveys

The main processing sequence involved plotting, picking and 1D inversion of the MASW and ESPAC records using the SeisImagerSW™ software. In theory, all the shots records for a given spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, localized surface variations, and/or dipping of overburden layers or rock. In general the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

Survey Design

For the main spread, the spacing between geophones was 2.5 meters, which means that the total length of the 20 geophone spread was 47.5 meters, and 1 meter (total of 23 meters long) for the one dedicated to the near surface details. The seismic records were realized with a seismograph Terraloc MK6 (from ABEM Instrument AB), and the geophones were 4.5 Hz. An 80 pounds weight-drop was used as the primary energy source with impacts being recorded off both ends of the seismic spreads. The shear wave depth sounding involves the average of the bulk area within the geophone spread, especially for its central half-length.

The seismic refraction measurements used the main MASW spread.

Results

The calculated seismic shear wave velocity sounding is presented in Figure 5.

The V_{S30} value is based on the harmonic mean of the shear wave velocities, from the surface to 30 meters deep. It is calculated by dividing the total depth of interest (e.g. 30 meters) by the sum of the time spent in each velocity layer from the surface up to that depth. This harmonic mean value reflects an equivalent single layer response.

Some basic seismic refraction measurements allowed calculations of rock depths below the seismic spread, and seismic rock velocities between 1950 and 2125 m/s.

The calculated V_{S30} value, for the actual site, is 959.7 m/s, indicating a Site Class “B“. Details of this value calculation are presented in Table 1. It must be noted that this value is significantly modulated by the overburden seismic low velocities. As for the actual project, an excavation of approximately 9 meters below the ground surface is planned, and the rock for such minimal depth presents seismic velocities higher than 1930 m/s, a Site Class “A“ could be considered.

CONCLUSION

A seismic shear wave velocity sounding was realized with the MASW/ESPAC method, on a parking lot located to the east of the intersection of Somerset St. W and Lebreton St. N, Ottawa. The V_{S30} calculations, for the actual site, are presented in Table 1.

The calculated V_{S30} value, for the actual site, is 960 m/s. Based on this value (as determined through the MASW/ESPAC method), Table 4.1.8.4.A of the NBC and the Building Code, O. Reg. 350/06, this site would be classified as “B” ($760 < V_{S30} \leq 1500$ m/s). This value is significantly modulated by the overburden seismic low velocities, as the rock seismic velocities are generally of the order of 1950-2200 m/s.

As for the actual building project, an excavation of approximately 9 meters deep is planned, and the rock seismic velocity is higher than 1500 m/s (V_{S30}* value of the order of 2165 m/s), the Site Class “A“ could be considered in this case.

Considering the actual site, it must be noted that other geotechnical information gleaned onsite; including the presence of liquefiable soils, soft clays, high moisture content etc. can supersede the site classification provided in this report based on V_{s30} value.

The V_S values calculated are representative of the in situ materials, and were not corrected for the total and effective stress.

This report has been written by Jean-Luc Arsenault, M.A.Sc., P.Eng.

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Project Manager

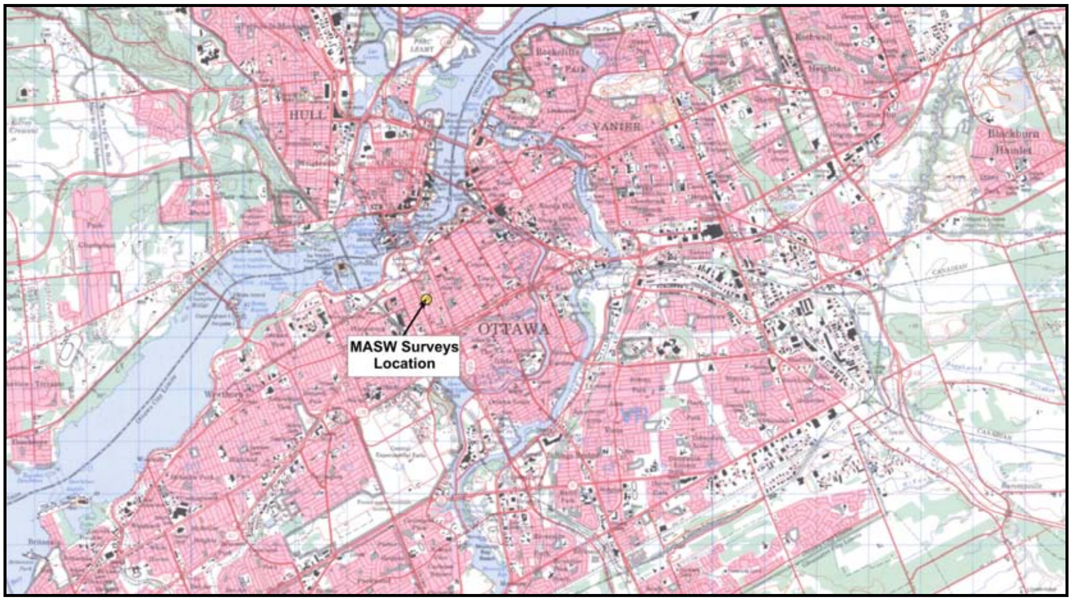


Figure 1: Regional location of the site
(source: topographic map 31 G/05)

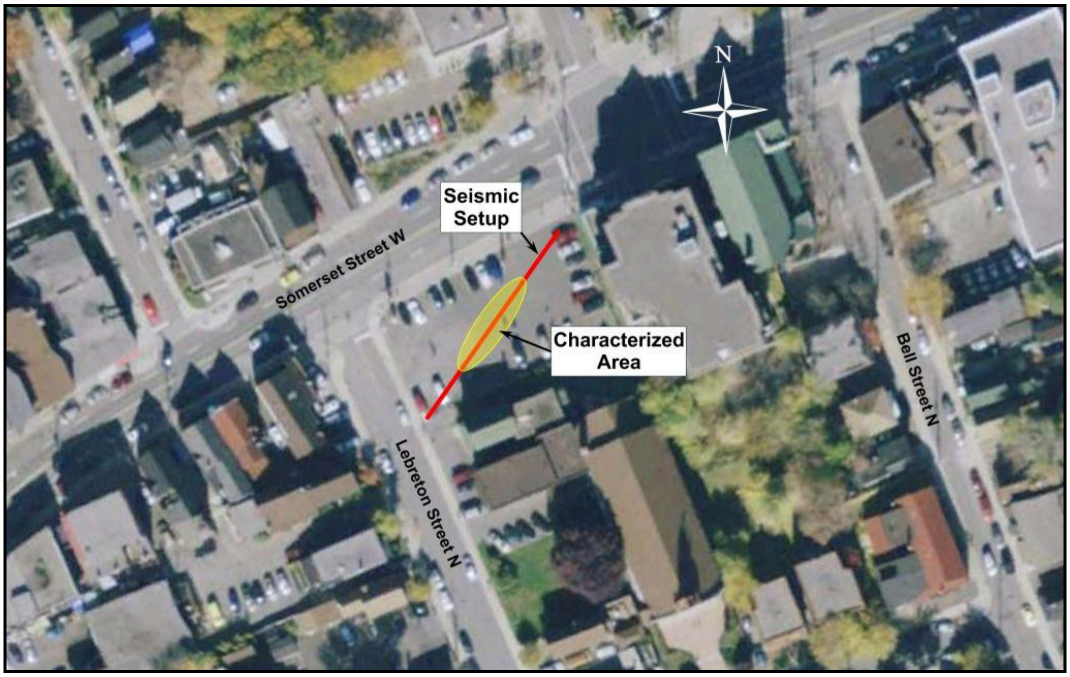


Figure 2: Location of the geophysical spread
(source : *Google Earth™*)

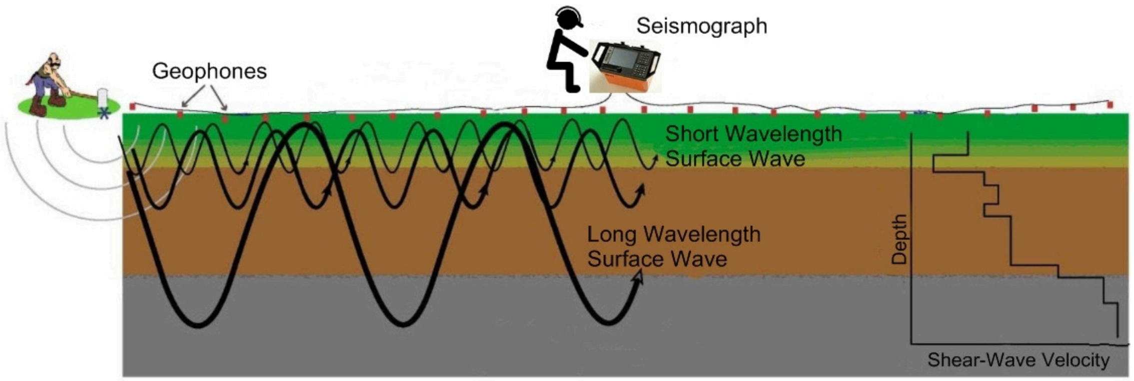
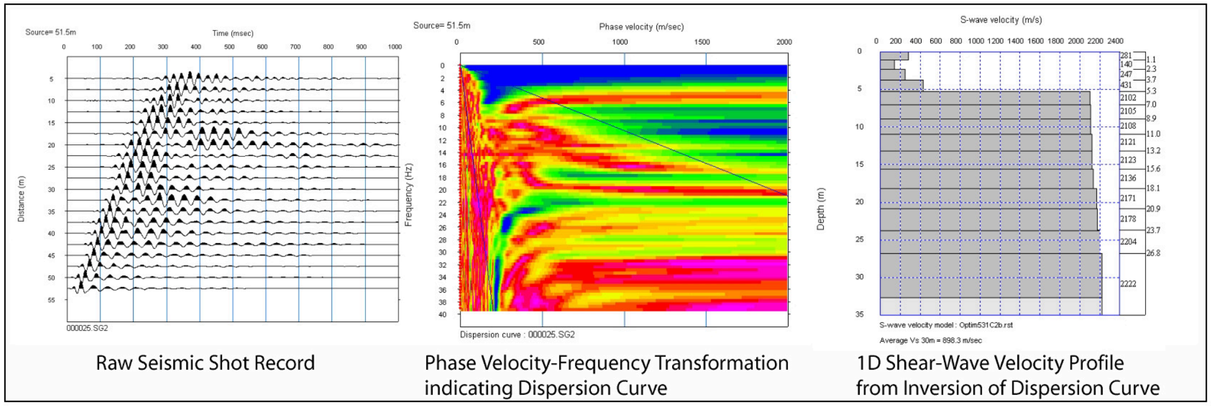


Figure 3: MASW Operating Principle



Raw Seismic Shot Record

Phase Velocity-Frequency Transformation indicating Dispersion Curve

1D Shear-Wave Velocity Profile from Inversion of Dispersion Curve

Figure 4: Example of a typical MASW/ESPAC record, phase velocity/frequency curve and resulting 1D shear wave velocity model

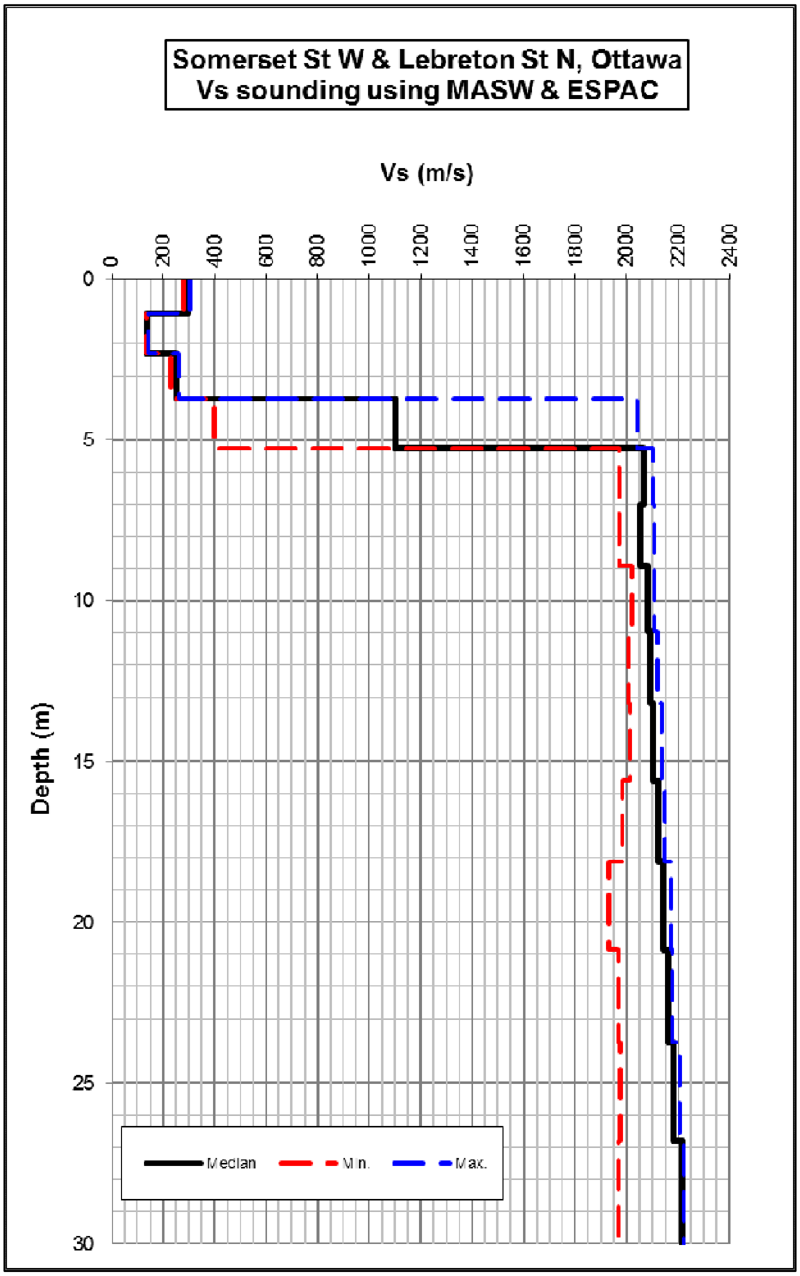


Figure 5: MASW Shear-wave Velocity Sounding

TABLE 1

Actual Site V_{S30} Calculation from MASW & ESPAC Surveys

Depth	Vs			Thickness	Cumulated Thickness	Delay for med. Vs	Cumulated Delay	Avg. Vs for given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0.00	280.7	297.1	303.9					
1.07	136.0	137.2	143.2	1.07	1.07	0.003604	0.003604	297.1
2.31	229.6	249.1	264.7	1.24	2.31	0.009009	0.012613	182.9
3.71	397.5	1101.6	2039.8	1.40	3.71	0.005625	0.018238	203.3
5.27	1970.9	2067.7	2103.0	1.57	5.27	0.001422	0.019660	268.3
7.01	1970.5	2054.9	2105.6	1.73	7.01	0.000837	0.020497	341.8
8.90	2019.4	2081.3	2108.4	1.90	8.90	0.000923	0.021419	415.6
10.96	2008.8	2095.2	2121.1	2.06	10.96	0.000990	0.022409	489.1
13.19	2013.2	2102.0	2134.4	2.23	13.19	0.001062	0.023471	561.8
15.58	1982.2	2123.5	2149.7	2.39	15.58	0.001137	0.024608	633.0
18.13	1932.2	2142.5	2171.6	2.56	18.13	0.001203	0.025811	702.4
20.85	1965.7	2162.0	2178.1	2.72	20.85	0.001270	0.027081	769.9
23.74	1974.2	2182.0	2205.0	2.89	23.74	0.001334	0.028415	835.3
26.79	1968.1	2219.4	2222.6	3.05	26.79	0.001397	0.029813	898.4
30.00	2068.2	2227.7	2242.3	3.21	30.00	0.001449	0.031261	959.7

Vs30 (m/s) =	**959.7**
Site Class :	**B**

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