7151896 Canada Corporation

Geotechnical Investigation

Type of Document
FINAL

Project Name
Proposed Condominium Building
280 Herzberg Road
Ottawa, Ontario

Project Number
OTT-00203421-A0

Prepared By:

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Ottawa, ON   K2B 8H6
Canada

Date Submitted
June 8, 2012

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Executive Summary

Exp Services Inc. (**exp**) is pleased to present the results of the geotechnical investigation completed for the proposed condominium building to be located at 280 Herzberg Road, Ottawa, Ontario. This work was authorized by 7151896 Canada Corporation.

It is understood two (2) building concepts are being considered; a four- to five-storey building with one-level underground parking garage and a seven-storey building with a two-level underground parking garage. The footprint for both building concepts will measure 25 by 30 m in size. The proposed site grade raise is 3.0 m. In addition, a parking lot and access road will be constructed at the front of the building.

The fieldwork for this investigation was carried out on November 16 and 17, 2011 and consisted of three (3) boreholes advanced to auger refusal depths of 11.3 and 14.0 m. The fieldwork was supervised on a full-time basis by a geotechnical technician from **exp** Services Inc.

The subsurface conditions consist of surficial topsoil and fill layer underlain by localized silty sand layer, extensive silty clay, a till-like silty clay and sandy silt to silty sand mantling inferred bedrock. Groundwater was at 3.5 and 3.6 m depths.

The raise in site grades should be restricted to 1.0 m, based on the soil fill having a unit weight of 22.0 kN/m^3. The proposed 3.0 m grade raise may be achieved by using light weight fill or a combination of light weight fill and soil fill.

The proposed four- to five-storey building with one-level underground parking garage may be supported by strip and spread footings founded on the native silty clay at 3.5 m depth and designed for a bearing pressure at serviceability limit state (SLS) of 120 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 180 kPa, provided the bearing pressure at SLS is sufficient to support a four- to five-storey building. Alternatively, it may be supported by a raft foundation designed for a bearing pressure at SLS of 120 kPa and factored geotechnical resistance at ULS of 180 kPa. The proposed seven-storey building with two-level underground parking garage may also be supported by a raft foundation designed for the same values in limit states design as noted above. Both building concepts may be supported by piles designed to bear within the bedrock.

Based on shear wave velocity measurements and comparison with Section 4.1.8.4 of the current Ontario Building Code (OBC), the site is categorized as Class C.

The lowest floor of the parking garage may be designed as a slab-on-grade or paved surface. The slab-on-grade for both building concepts may be set on 200 m thick bed of 19 mm clear stone constructed on the natural undisturbed silty clay for the one-level underground parking garage and on a 300 mm thick engineered fill pad consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type II for the two-level underground parking garage. Alternatively, the lowest floor may be constructed as a paved surface consisting of 90 mm asphaltic concrete, 150 mm Granular A and 450 mm Granular B Type II.

Perimeter and under floor drainage systems should be provided for the building with one- or two-level underground parking garage.

The subsurface parking garage walls should be designed to resist lateral static and seismic earth pressures. The walls should be backfilled using OPSS Granular B Type II compacted to 95 percent SPMDD.

Excavations for the one-level underground parking garage may be undertaken as open cut in accordance with the current Occupational Health and Safety Act and Regulations for Construction Projects (OHSA). It is likely the excavation for the two-level underground parking garage would have to be shored with an engineered support system designed in accordance with OHSA. The excavation for the one and two-level underground parking garage is anticipated to extend below the groundwater level. Seepage of the surface and subsurface water into the excavation for one or two basement levels is anticipated and may be handled by collecting it in perimeter ditches and removing it by pumping from sumps. High capacity pumps may be required within permeable zones of the fill, silty sand and silty clay.

Select portions of the onsite soils may be used as backfill material, subject to further evaluation and testing during construction. It is anticipated the majority of material required for backfilling purposes will have to be imported to site.

Based on the anticipated subgrade conditions, the pavement structure for heavy duty traffic should consist of 90 mm asphaltic concrete, 150 mm Granular A and 450 mm Granular B Type II. For light duty traffic, the pavement should consist of 75 mm asphaltic concrete, 150 mm Granular A and 300 mm Granular B Type II.

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

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1 Introduction

Exp Services Inc. (**exp**) is pleased to present the results of the geotechnical investigation completed for the proposed condominium building to be located at 280 Herzberg Road, Ottawa, Ontario. This work was authorized by 7151896 Canada Corporation.

It is understood two (2) building concepts are being considered; a four- to five-storey building with one-level underground parking garage and a seven-storey building with two-level underground parking garage. The footprint for both building concepts will measure 25 by 30 m in size. It is proposed to raise the grade at the site by 3.0 m. In addition, a parking lot and access road will be constructed at the front of the building.

This geotechnical investigation was undertaken to:

(a) Assess the subsurface soil and groundwater conditions at three (3) borehole locations;

(b) Discuss site grade raise restrictions;

(c) Recommend bearing pressures and factored geotechnical resistances in limit states design of the founding soils for the proposed building foundations and comment on anticipated total and differential settlements of the proposed foundations;

(d) Provide classification for seismic site response in accordance with the 2006 Ontario Building Code and discuss liquefaction potential of subsurface soils;

(e) Comment on slab-on-grade construction and requirement for under floor and perimeter drainage systems;

(f) Provide lateral earth pressures against subsurface walls;

(g) Discuss subsurface concrete requirements;

(h) Discuss excavation conditions and de-watering requirements;

(i) Comment on backfilling requirements and the suitability of the onsite soils for backfilling purposes; and

(j) Recommend a pavement structure for the parking lot and access road areas.

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 site is located at 280 Herzberg Road in the Kanata area of Ottawa, Ontario. It is currently occupied by a single residential building and separate garage and shed buildings. The location of the site is shown on the Site Location Plan, Figure 1. The topography of the site is relatively flat with elevations in the order of 77.5 to 77.7 m.

3 Procedure

The fieldwork for this investigation was carried out on November 16 and 17, 2011 and consisted of the placement of three (3) boreholes advanced to auger refusal depths of 11.3 and 14.0 m. The fieldwork was supervised on a full-time basis by a geotechnical technician from **exp**.

The borehole locations were established in the field by **exp**. The borehole elevations were estimated from spot elevations (geodetic) indicated on the drawing titled, "Surveyor's Real Property Report – Part 1 and Topographical Plan of Survey of Part of Lot 7 Concession 4 Geographic Township of March, City of Ottawa (Being Parts 1 and 3, Plan 5R-1127) dated September 25, 2003 and prepared by Farley, Smith and Denis Surveying Ltd. Therefore, the borehole elevations should be considered approximate. The borehole locations are shown on the Borehole Location Plan, Figure 2.

The boreholes were undertaken with a truck-mounted drill rig. Standard penetration tests were performed in all the boreholes at 0.75 and 1.5 m depth intervals with soil samples retrieved by the split barrel sampler. Three (3) relatively undisturbed thin walled tube samples of the cohesive soil were obtained in one (1) borehole from select depths. The undrained shear strength of the cohesive soil was established by performing in-situ field vane and pocket penetrometer tests. Standpipes were installed in the three (3) boreholes to monitor the groundwater level over time. All the boreholes were backfilled on completion of the fieldwork.

All soil samples were visually examined in the field, logged, preserved in plastic bags and identified. On completion of the fieldwork, all the soil samples were transported to the **exp** laboratory in the City of Ottawa where they were examined by a senior geotechnical engineer and laboratory testing assigned. Laboratory testing consisted of performing moisture content tests on all the soil samples. Unit weight, Atterberg limit determination, grain-size analysis, one dimensional consolidation test and pH/sulphate content analyses were conducted on select soil samples.

4 Subsurface Soil and Groundwater Conditions

A detailed description of the subsurface soil and groundwater conditions determined from the boreholes are given on the attached Log of Borehole sheets, Figures 3 to 5. 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.

It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The “Notes on Sample Descriptions” preceding the borehole logs forms an integral part of this report and should be read in conjunction with this report.

A review of the borehole logs indicates the following subsurface soil and groundwater conditions with depth.

4.1 Topsoil

A surficial 125 to 600 mm topsoil layer was contacted in Boreholes 1 and 3.

4.2 Fill

A surficial fill was encountered in Borehole 2 and beneath the topsoil of Borehole 1. The fill extends to 0.8 m depth and consists of silty sand with a 100 mm thick sand and gravel surface layer in Borehole 2. The fill is in a loose state with natural moisture content of 10 to 24 percent.

4.3 Silty Sand

The fill in Borehole 2 is underlain by silty sand to 1.8 m depth. The silty sand is in a loose state and a moisture content of 32 percent.

4.4 Silty Clay

The underlying native soil contacted in all three (3) boreholes consists of silty clay having an upper brown weathered, desiccated crust becoming unweathered and grey below 3.8 to 4.6 m depths. The consistency of the brown silty clay, based on undrained shear strength values of 100 to greater than 120 kPa, is very stiff. The natural moisture content and unit weight are 28 to 44 percent and 17.9 to 19.7 kN/m^3 respectively. Grain-size analysis of one (1) sample of the brown silty clay indicates a soil composition of 56 percent clay, 41 percent silt and 3 percent sand. The grain size distribution curve is shown on Figure 6. The soil may be described as silty clay with trace sand.

The underlying grey silty clay was encountered below 3.8 to 4.6 m depths and extends to 8.7 to 10.7 m depths. Based on shear strength values ranging from 43 to 100 kPa, the consistency o the silty clay is firm to stiff and it is considered to be moderately to highly sensitive. Its natural moisture content is 44 to 64 percent. Atterberg limit tests conducted on two (2) samples of the grey silty clay give liquid and plastic limits of 39 and 19 to 24 percent, respectively. The limit test results indicate the grey silty clay may be classified as an inorganic clay of medium plasticity. Grain-size analysis indicates a soil composition of 54 percent clay, 42 percent silt and 4 percent sand and may be described as silty clay with trace of sand. The grain size distribution curve is shown on Figure 7.

The results of the laboratory one dimensional consolidation test carried out on the thin walled tube sample of the grey silty clay recovered in Borehole 3 from 8.2 to 8.8 m depths are shown on Figure 8 and Table No. I.

Table No. I: Summary of Consolidation Test Results

Borehole No.	Sample No.	Sample Depth (Elevation) (m)	Natural Moisture Content (%)	Calculated Effective Over-burden Pressure (kPa)	Estimated Pre-consolidation Pressure (kPa)	Over-consolidation Ratio	Initial Void Ratio	Estimated Compression Index	Estimated Re-compression Index
3	TW 10	8.2 – 8.8	45	115	228	2.0	1.235	0.686	0.022

Based on the results of the consolidation test, the silty clay is considered to be over-consolidated.

4.5 Silty Clay (Till-Like)

Silty clay with a faint till-like structure exists below the grey silty clay in Boreholes 1 and 2 at 8.7 m depth. The till-like silty clay contains some sand and gravel and has a very soft to stiff consistency with a natural moisture content of 10 to 32 percent.

4.6 Sandy Silt to Silty Sand

The grey silty clay in Borehole 3 is underlain by sandy silt to silty sand, which is in a loose state. The natural moisture content is 14 to 22 percent.

4.7 Auger Refusal

Auger refusal as met in all three (3) boreholes at 11.3 and 14.0 m depths on inferred bedrock. Based on a review of local geology maps, the bedrock in the vicinity of the site is dolostone or limestone of the Oxford formation.

4.8 Groundwater Conditions

Upon completion of the fieldwork, all boreholes remained dry. Approximately two (2) weeks following completion of drilling, the groundwater level was at 3.5 and 3.6 m depths below existing grade.

It is recommended that once the standpipes are no longer required, they be decommissioned in accordance with Ontario Regulation 903.

Water levels were made in the exploratory borehole at the times and under the conditions stated in the scope of services. 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.

5 Grade Raise Restriction

As part of the development, it is understood the proposed site grade raise will be 3.0 m.

The site is underlain by native silty clay that is prone to being overstressed from loads imposed on it by site grade raise, foundations, and groundwater level lowering following construction. Overstressing of the silty clay may result in consolidation of the silty clay and subsequent settlement of foundations, which may exceed the tolerable limits of the structure.

Based on the laboratory consolidation test results and settlement calculations, the grade raise at the site should be restricted to 1.0 m, for a soil having a unit weight of 22 kN/m^3. This restriction is based on the bearing pressures recommended for design of the footings discussed in Section 7 of this report. The proposed 3.0 m raise in grade at the site may be achieved by using light weight fill or a combination of light weight fill and soil fill. It is noted the use of light weight fill may be cost prohibitive. **Exp** can provide additional comments in this regard, as more details regarding the proposed development become available.

The final foundation plan and grading plan should be reviewed by **exp** to ensure that the foundation design and final grades comply with the recommendations of this report.

6 Site Grading

Prior to placement of any fill around the building, all topsoil and unsuitable portions of the fill should be removed to the natural undisturbed soil. The exposed subgrade should be proof rolled with 5 to 6 passes of a heavy roller in the presence of a geotechnical engineer. Any soft areas identified should be sub-excavated and replaced with imported Ontario Provincial Standard Specification (OPSS) Granular B or select subgrade material (SSM) within 2 percent of the optimum moisture content and compacted to 95 percent standard Proctor maximum dry density (SPMDD). Following approval of the subgrade, the site grades may be raised to the design subgrade level by the placement of approved onsite excavated material, imported OPSS Granular B or SSM compacted to 95 percent SPMDD. The degree of compaction should be checked by conducting in place density tests to ensure the compaction is undertaken in conformance with the project specifications.

7 Foundations

Based on a review of the borehole information, the proposed four- to five-storey condominium building with one-level underground parking garage may be supported by shallow foundations such as strip and spread footings set on the silty clay, provided the bearing capacity of the silty clay is sufficient to support the building. Alternatively, the building may be supported by a raft foundation founded on the brown and grey silty clay or by a deep foundation system such as piles bearing on bedrock. The seven-storey building with a two-level underground parking garage may be supported by a raft foundation or deep foundation system such as piles.

Each foundation alternative is discussed in the following section of this report. The serviceability limit state (SLS) bearing pressure and the factored geotechnical resistance at ultimate limit state (ULS) recommended are based on a 1.0 m grade raise as indicated in Section 5 of this report.

7.1 Shallow Foundations

7.1.1 Strip and Spread Footings

It is considered that the proposed four- to five-storey building with one-level underground parking garage may be supported by footings set on the native brown or grey silty clay at approximately 3.5 m depth and designed for a bearing pressure at serviceability limit state (SLS) of 120 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 180 kPa.

Settlements of the footings set on the silty clay and designed for the bearing pressure at SLS recommended above and properly constructed are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential.

All the footing beds should be examined by a geotechnical engineer to ensure the founding soil is capable of supporting the bearing resistance at SLS and that the footings beds have been properly prepared.

A minimum of 1.5 m of earth cover should be provided to the exterior footings of the heated structure to protect them from damage due to frost penetration.

Based on the anticipated subsurface soil conditions, it is recommended a 50 mm thick skim slab of concrete be placed on the silty clay subgrade immediately following excavation and approval, to protect the subgrade from disturbance due to inclement weather and construction traffic (such as foot traffic).

Within the footprint of the buildings to be demolished, the depth to the native silty clay may vary from that indicated on the borehole logs. Therefore, near existing and former features, the footings may have to be stepped down to reach the silty clay and **exp** should be contacted to verify that the silty clay will not be overstressed by the deepened footings.

Footings at different elevations should be located such that higher footings are set below a line drawn up at 10H:7V from the near edge of the lower footing. This concept should also be applied to excavations for new foundations in relation to existing footings. Where foundations for existing structures do not satisfy the criteria, underpinning or positive shoring may be required. Also, the lower footings should be constructed prior to constructing the higher footings in order to prevent the latter from being undermined due to subsequent construction.

7.1.2 Raft Foundation

Both building concepts may be supported by a raft foundation designed for a net bearing pressure at SLS of 120 kPa and factored geotechnical resistance at ULS of 180 kPa. Total settlement of the raft foundation at 3.5 m depth is anticipated to be 15 mm and 10 mm for differential settlement. A raft founded at 6.5 m depth will act as a 'floating foundation' and the total and differential settlements are expected to be less than 10 mm.

The subgrade of the raft foundation should be examined by a geotechnical engineer to ensure the founding soil is capable of supporting the bearing resistance at SLS and that the subgrade has been properly prepared.

Based on the anticipated subsurface soil conditions, it is recommended a 50 mm thick skim slab of concrete be placed on the silty clay subgrade immediately following excavation and approval, to protect the subgrade from disturbance due to inclement weather and construction traffic (such as foot traffic).

7.1.3 General Comments

The recommended bearing pressure at SLS and factored geotechnical resistance at ULS have been calculated by **exp** from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes, when foundation construction is underway. The interpretation between boreholes, and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage.

7.2 Deep Foundations

7.2.1 Pile Foundations

Alternatively, both building concepts may be supported by piles driven to practical refusal within the bedrock inferred at 11.3 and 14.0 m depths.

Closed-end concrete-filled steel pipe or H piles driven to practical refusal within the dolostone/limestone bedrock may be designed for the factored geotechnical resistances at ULS shown in Table No. II.

Table No. II: Factored Geotechnical Resistance at ULS for Piles

Pile Section	Pile Size	Factored Geotechnical Resistance at ULS (kN)
Steel Pipe	245 mm OD by 12 mm wall	1400
	324 mm OD by 12 mm wall	2030
Steel H	HP 310 x 79	1260
	HP 310 x 110	1780
	HP 310 x 125	2000
	HP360 x 132	2115

The above factored geotechnical resistances are based on steel piles with a yield strength of 350 MPa and structural concrete having a compressive strength of 30 MPa.

Placement of soil fill on the site will cause consolidation of the silty clay resulting in the generation of negative skin friction along the pile shaft which will reduce the factored geotechnical resistance at ULS of the pile. Therefore, if piles are used to support the building**, exp** should be contacted to provide the reduced factored geotechnical resistances at ULS for the various pile sections, once details regarding site grade raise are available.

If pile foundations are to be used, it is recommended an additional geotechnical investigation be undertaken to confirm by coring techniques, the presence of the bedrock at the auger refusal depths of 11.3 and 14.0 m below existing grade and assess the engineering properties of the bedrock (such as quality).

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

In order to achieve the capacity given above, the pile driving hammer must seat the pile into the bedrock without overstressing the pile material. For guidance purposes, it is estimated that a hammer with rated energy of 54 kJ to 70 kJ (40,000 to 52,000 ft lbs) per blow would be required to drive the piles to practical refusal in the bedrock. Practical refusal is considered to have been achieved at a set of 5 blows for 6 mm or less of pile penetration. However, the driving criteria for a particular hammer-pile system must be established at the beginning of the project. This may be undertaken with the Pile Driving Analyzer.

The till-like structure below the silty clay is expected to contain cobbles and boulders. It is therefore recommended that the pile tips should be reinforced with a 25 mm thick steel plate and equipped with a driving shoe in accordance with Ontario Provincial Standard Drawing (OPSD) 3001.100, Type II.

It is recommended that three (3) to five (5) piles should be monitored with the Pile Driving Analyzer during the initial driving and re-striking at the beginning of the project for the evaluation of transferred energy into the pile from the hammer, determination of driving criteria and an evaluation of the geotechnical resistance at ULS of the piles. Depending on the results of the pile driving analysis, the pile capacity may have to be proven by at least one pile load test for each pile type before production piling begins. If necessary, the pile load test should be performed in accordance with ASTM D 1143.

Closed-end pipe piles tend to displace a relatively large volume of soil. When driven in a cluster or group, they may tend to jack up the adjacent piles in the group. Consequently, the elevation of the top of each pile in a group should be monitored immediately after driving and after all the piles in the group have been driven. This is to ensure that the piles are not heaving. Any piles found to heave more than 3 mm should be re-tapped.

Piles driven at the site may be subject to relaxation i.e. loss of set with time. It is therefore recommended that all the piles should be re-tapped a minimum of 24 hours after initial driving and at 24 hour intervals thereafter until it can be proven that relaxation is no longer a problem.

The installation of the piles at the site should be monitored on a full time basis by a geotechnician working under the direction and supervision of a qualified geotechnical engineer to verify that the piles are driven in accordance with the project specifications.

It is anticipated that some piles may be driven in a group. Since the piles would be driven to practical refusal in the bedrock, the total factored geotechnical resistance at ULS of a group of piles may be taken as equal to the product of the factored geotechnical resistance at ULS of an individual pile multiplied by the number of piles in the group.

All pile caps should be protected from frost action. This may be accomplished by providing the pile cap with minimum 1.5 m earth cover for heated structures. Alternatively, frost protection may be provided by rigid insulation or a combination of soil cover and rigid insulation.

8 Seismic Site Classification and Liquefaction Potential of Onsite Soils

A shear wave velocity sounding survey was undertaken at the site and the results are shown in Appendix A. The results of the sounding were compared with the range of shear wave velocities provided in Section 4.1.8.4 of the 2006 Ontario Building Code (OBC) and indicate the site may be categorized as Class C.

Once the number of basement levels has been established for the building, the seismic site class should be reviewed by **exp** to see if the site class may be revised.

The soils are not considered to be liquefiable.

9 Floor Slab Construction and Drainage Requirements

The lowest level floor of the one- or two-level underground parking garage may be constructed as a slab-on-grade or as a paved surface.

9.1 Slab-On-Grade Construction

For both building concepts, the slab-on-grade should be constructed on a bed of well compacted 19 mm clear stone at least 200 mm thick. For the one-level underground parking garage, the clear stone should be placed on the undisturbed natural silty clay. For the two-level underground parking garage, the clear stone should be placed on a 300 mm thick OPSS Granular B Type II pad compacted to 98 percent SPMDD. The clear stone will prevent the capillary rise of moisture from the sub-soil to the floor slab. A polyethylene vapour barrier should be placed over the clear stone pad, if a moisture sensitive covering is to be placed on the floor. Adequate saw cuts should be provided in the floor slabs to control cracking.

9.2 Paved Surface

Alternatively, the lowest floor slab level for both building concepts may be a paved surface consisting of 90 mm asphaltic concrete, 150 mm Granular A and 450 mm Granular B Type II. The placement and compaction of the granulars and asphaltic concrete should be in accordance with Section 14 of this report.

9.3 Drainage Requirements

Since the basement floor is anticipated to be at or below the groundwater level, an underfloor drainage system as well as perimeter drainage system will be required for both the one- and two-level underground parking garage buildings.

The invert of the underfloor drains should be at least 300 mm below the underside of the slab. The underfloor drains should be placed in parallel rows at 6 to 8 m centers one way. The drainage system may consist of 100 or 150 mm diameter perforated pipe leading to a positive sump. The drains should be set on a 100 mm thick bed of 19 mm clear stone placed on an approved geotextile such as Terrafix 270R or equivalent. The pipe should be surrounded on the top and sides with 150 mm of clear stone and the clear stone should be completely wrapped with the geotextile. The perimeter and underfloor drainage systems should be connected to separate sumps equipped with back up pumps and generators in case of mechanical failure and/or power outage.

The perimeter drainage system may consist of 100 or 150 mm diameter perforated pipe surrounded by 19 mm clear stone wrapped with an approved geotextile membrane, such as Terrafix 270R.

A schematic illustration of the underfloor and perimeter drainage systems is given on Figure 9.

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

10 Lateral Earth Pressure on Subsurface Walls

The subsurface walls for both building concepts should be backfilled with free draining material, such as Granular B Type II and equipped with a perimeter drainage system to prevent the build up of hydrostatic pressure behind the wall. The walls will be subjected to lateral static and dynamic (seismic) earth pressures.

The lateral static earth pressure against the subsurface wall may be computed from the following equation:

$$P = K_0 (\gamma h + q)$$

where P = lateral earth pressure acting on the subsurface wall; kPa

K_0 = lateral earth pressure coefficient for Granular B Type II backfill material = 0.5

γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3

h = depth of interest below final grade behind wall; m

q = surcharge load; kPa

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

$$\Delta P_E = 0.21 \gamma H^2$$

where ΔP_E = resultant force due to seismic activity; kN/m

γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3

H = height of backfill behind wall; m

The ΔP_E value is assumed to act 0.6H from the bottom of the wall for design purposes.

The subsurface walls should also be dampproofed.

11 Subsurface Concrete Requirements

Chemical tests limited to pH and sulphate content were undertaken on select samples of the silty clay (BH1) and the sandy silt to silty sand (BH 3). The results are summarized on Table No. III.

Table No. III - Results of Chemical Tests on Soil Samples			
Borehole No.	**Depth (m)**	**pH**	**Sulphate (%)**
1	6.1 – 6.8	8.0	0.06
3	10.7 – 11.3	8.0	0.05

Based on a review of the above test results, the subsurface soils have a negligible potential for sulphate attack on subsurface concrete. Therefore, General use (GU) Portland cement may be used in the subsurface concrete. It is recommended the subsurface concrete selected for this project be in accordance with the requirements of the current Canadian Standards Association (CSA- A23).

12 Excavation Conditions and De-Watering Requirements

12.1 Excavations

Excavation for the one-level underground parking garage is anticipated to extend to 3.5 m depth and will be at or slightly below the measured groundwater level. For the two-level underground parking garage, the excavation will extend to 6.5 m depth and will be approximately 3.0 m below the measured groundwater level. The excavation for either building scheme will extend through the fill, silty sand and into the native silty clay.

Excavation of the fill, silty sand and silty clay may be undertaken using a large mechanical shovel capable of removing possible debris within the fill. The silty clay is susceptible to disturbance due to the movement of construction equipment along its surface. It is therefore recommended that the excavation at the site should be undertaken by equipment which does not travel on the excavated surface, such as a mechanical shovel.

At the anticipated excavation depths of 3.5 or 6.5 m, the silty clay is not considered susceptible to base heave.

The excavations for the one-level underground parking garage may be undertaken as open cut in accordance with the current Occupational Health and Safety Act (OHSA). The subsurface soils are considered to be Type 3 and must be cut back at 1H:1V from the bottom of the excavation. Portions of the excavation that will extend below the groundwater level are expected to slough and may eventually stabilize at a slope of approximately 2H:1V to 3H:1V.

The deeper two-level underground parking garage excavation will likely have to be undertaken within the confines of a shoring system consisting of conventional soldier pile and timber lagging.

The shoring system must be designed to support the lateral earth pressure given by the expression below:

$$P = k(\gamma h + q)$$

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

k = earth pressure coefficient applicable (see below)

γ = unit weight of soil to be retained; estimated at 18.7 kN/m^3

h = the depth at which pressure, P, is being computed; m

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

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.
- 0.45 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.

The pressure distribution assumes that drainage is permitted between the lagging boards and that no build-up of hydrostatic pressure may occur.

The shoring should be designed using the appropriate k value depending on the location of any settlement-sensitive structures and underground services.

It is recommended a condition survey of the existing buildings located adjacent to the site as well as nearby underground services be undertaken prior to construction of the new building.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

12.2 De-Watering Requirements

The excavation for the one-level underground parking garage is anticipated to extend to the groundwater level or slightly below. Seepage of the surface and subsurface water into these excavations is anticipated and may be handled by collecting any water entering the excavations in perimeter ditches and removing it by pumping from sumps. High capacity pumps may be required, due to the presence of permeable zones, within the fill, silty sand and silty clay.

Some seepage of groundwater into the shored excavation should be expected. However, it should be possible to control and remove any seepage into the shored excavation using conventional de-watering techniques such as pumping from sumps.

Although this investigation has estimated the groundwater levels at the time of the field work, and commented on de-watering and general construction problems, conditions may be present which are difficult to establish from standard boring techniques and which may affect the type and nature of de-watering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction de-watering systems.

13 Backfilling Requirements and Suitability of Onsite Soils for Backfilling Purposes

The backfill should conform to the following specifications:

- Backfilling in footing and service trenches inside the building – Ontario Provincial Standard Specification (OPSS) Granular B, Type II; placed in 300 mm thick lifts and each lift compacted to 98 percent standard Proctor maximum dry density (SPMDD);

- Backfill against exterior subsurface walls - OPSS Granular B, Type II placed in 300 mm thick lifts and each lift compacted to 95 percent SPMDD; and

- Trench backfill outside building - OPSS 1010 Select Subgrade Material (SSM), free of organics, debris and with a natural moisture content within 2 percent of the optimum moisture content. It should be placed in 300 mm thick lifts compacted to minimum 95 percent SPMDD.

The soils to be excavated from the site will comprise fill and brown and grey silty clay. Portions of the fill, silty sand and brown silty clay may be reused as backfill material outside the building, subject to further evaluation at time of construction. The grey silty clay is considered too wet for reuse as backfill material. The topsoil is not considered suitable for re-use as backfill material.

The quantity of re-usable onsite material for backfilling purposes is expected to be limited. Therefore, it is anticipated that the majority of material required for backfilling purposes would need to be imported and should preferably conform to the specifications listed above.

14 Pavement Structure

It is understood a parking lot and access road will be located at the front of the condominium building. The pavement subgrade is anticipated to comprise of silty sand fill, silty sand and silty clay. Pavement structure thicknesses required for light and heavy duty traffic on the access road and parking lot were computed and are shown on Table No. IV. The thicknesses are based upon an estimate of the subgrade soil properties determined from visual examination, textural classification of the soil samples and functional design life of eight to ten years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table No. IV: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Computed Pavement Structure (mm)	
		Light Duty	Heavy Duty
Asphaltic Concrete (PG 58-34)	92-96% MRD	75 SC	40 SC 50 BC
OPSS Granular A Base (crushed limestone)	100% SPMDD	150	150
OPSS Granular B II Sub-base	100% SPMDD	300	450

Notes:
1. SPMDD denotes standard Proctor maximum dry density, ASTM, D-698
2. The upper 300 mm of subgrade fill must be compacted to 98% SPMDD
3. SC denotes Surface course asphalt and should comprise of Marshall HL3.
4. BC denotes Base course asphalt and should comprise of Marshall HL8.
5. MRD denotes Maximum Relative Density – ASTM, D-2041

The foregoing design assumes that construction is carried out during dry periods and that the subgrade is stable under the load of construction equipment. If construction is carried out during wet weather, and heaving or rolling of the subgrade is experienced, additional thickness of granular material and/or geotextile may be required.

Additional comments on the construction of the access road and parking lot are as follows:

(1) As part of the subgrade preparation, the proposed access road and parking lot areas should be stripped of topsoil, fill and other obviously unsuitable material. Fill required to raise the grades to design subgrade level (note 1.0 m grade raise restriction – Section 5 of report) should be organic-free and at a moisture content which will permit compaction to the densities indicated. The subgrade should be properly shaped, crowned, then proofrolled with a heavy roller in the full-time presence of a representative of this office. Any soft or spongy subgrade areas detected should be subexcavated and properly replaced with suitable approved backfill compacted to 95 percent SPMDD.

(2) It is stressed that the overall satisfactory performance of the recommended pavement structures is contingent upon the provisions of good drainage. In parking areas, they should be located at low points and should be continuous between catch basins. The drains should be located with their invert approximately 300 mm below the subgrade level and may consist of 100 to 150 mm diameter perforated pipe set on 100 mm bed of 19 mm clear stone and covered top and sides with 150 mm of 19 mm stone. The stone should be surrounded with a suitable filter cloth, such as Terrafix 270 R or equivalent. The remainder of the trench should be backfilled with well compacted, free draining granular material.

(3) To minimize the problems of differential movement between the pavement and catch basins/manholes due to frost action, the backfill around the structures should consist of free-draining granular material preferably conforming to OPSS Granular B Type II. Weep holes should be provided in the catch basins to facilitate drainage of any water which may accumulate in the granular fill around the catch basin.

(4) The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum cross fall of 2 percent) to provide effective surface drainage towards catch basins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.

(5) 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 SPMDD. The asphaltic concrete used and its placement should meet the requirements of OPSS 313 and 1150.

It is recommended that **exp** be retained to review the final pavement structure design and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report.

15 General Closure

The comments given in this report are intended only for the guidance of the 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 to draw their own conclusions as to how the subsurface conditions may affect them.

The information contained in this report is not intended to reflect on environmental aspects of the soils. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Figures

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www.exp.com

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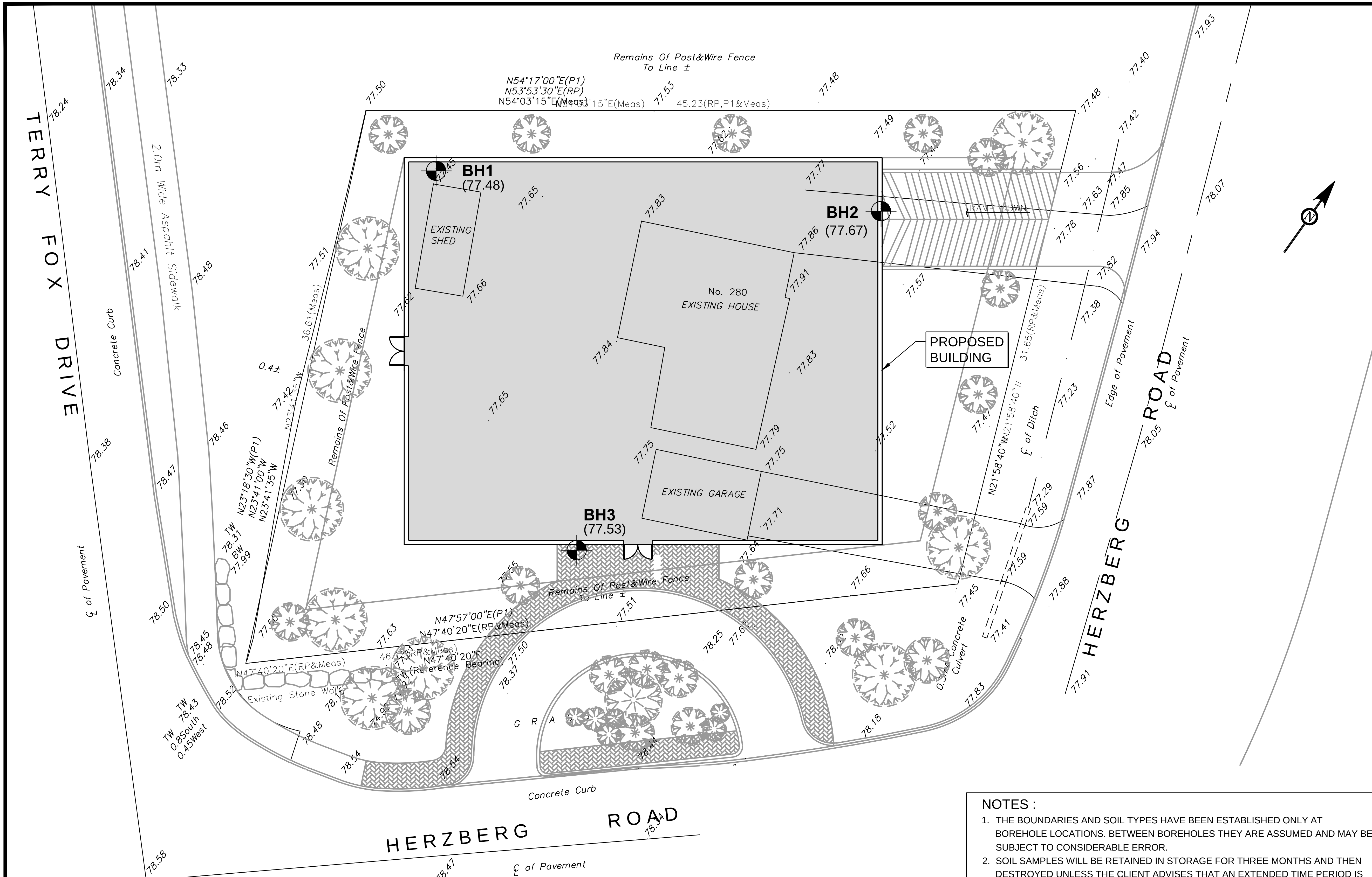
• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

scale 1:20,000	CLIENT:	7151896 CANADA CORPORATION	project no. OTT-00203421-A0
date JUNE 2012	TITLE:	SITE LOCATION PLAN 280 HERZBERG ROAD, OTTAWA, ON	Figure 1
drawn by M.N.			



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 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 SUSAN D. SMITH ARCHITECT, DATED MAY 2011 And FARLEY, SMITH & DENIS SURVEYING LTD., JOB NO. 414-03, DATED SEPTEMBER 25, 2003.

BH1 (77.48) — BOREHOLE NUMBER, LOCATION AND APPROXIMATE GEODETIC ELEVATION

0 2m 4m 10m
HORIZONTAL 1:250

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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

CLIENT: 7151896 CANADA CORPORATION

TITLE: BOREHOLE LOCATION PLAN - PROPOSED CONDOMINIUM BUILDING
280 HERZBERG ROAD, OTTAWA, ON

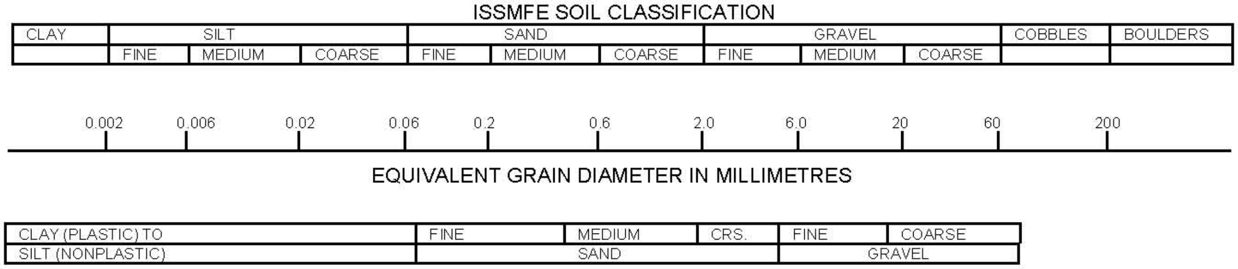
scale	1:250
date	JUNE 2012
drawn by	P.G. / M.N.
project no.	OTT-00203421-A0

Figure 2

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.

ISSMFE SOIL CLASSIFICATION



UNIFIED SOIL CLASSIFICATION

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 1

Project No: OTT-00203421-A0

Project: Proposed Condominium Building

Location: 280 Herzberg Road, Ottawa, Ontario

Figure No. 3

Page. 1 of 1

Date Drilled: 'November 16, 2011

Drill Type: CME-55 (Truck Mount)

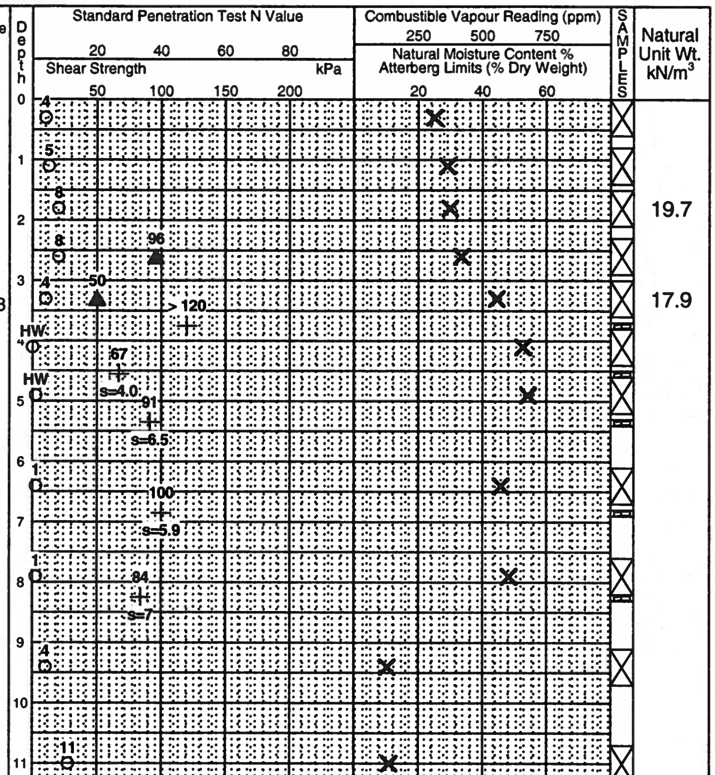
Datum: Approximate Elevation

Logged by: ________ Checked by: ________

Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	X
SPT (N) Value	o	Atterberg Limits	⊢—⊖
Dynamic Cone Test	——	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ s		

Depth (m)	SOIL DESCRIPTION	Approximate Elevation m
0		77.48
0	**TOPSOIL** ~ 125 mm	77.4
0.8	**FILL** Silty sand, brown, moist (loose)	76.7
	SILTY CLAY Trace sand, brown, moist (stiff to very stiff)	
3.5		73.98
3.8	**SILTY CLAY** Grey, wet (stiff)	73.7
8.8	**SILTY CLAY (TILL-LIKE)** Some sand and gravel, grey, wet (firm to stiff)	68.7
11.3		66.2

Auger Refusal at 11.3 m Depth



NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2. A 19 mm diameter slottedf standpipe installed to 11.3 m depth.
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-00203421-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	Dry	11.3
16 days	3.5	

CORE DRILLING RECORD

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

Log of Borehole 2

Project No: OTT-00203421-A0

Project: Proposed Condominium Building

Location: 280 Herzberg Road, Ottawa, Ontario

Date Drilled: 'November 16, 2011

Drill Type: CME-55 (Truck Mount)

Datum: Approximate Elevation

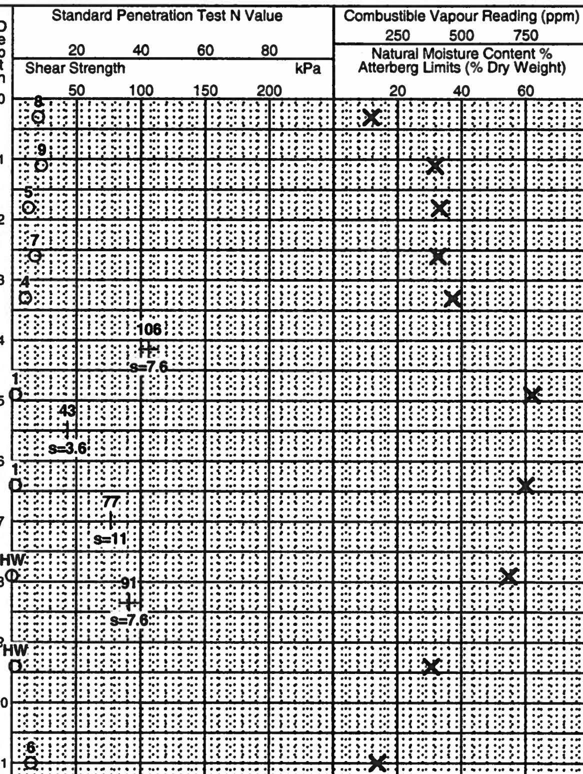
Logged by: ________ Checked by: ________

Figure No. 4

Page. 1 of 1

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

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth	Natural Unit Wt. kN/m³
			77.67	0	
		FILL Sand and gravel, brown, moist ~ 100 mm	77.6		
		FILL Silty sand, some gravel, rootlets, wood pieces, brown, moist (loose)	76.9	1	
		SILTY SAND Some clayey silt partings, brown, moist (loose)	75.9	2	18.8
		SILTY CLAY Brown, moist (very stiff)	74.07	3	18.5
		SILTY CLAY Grey, wet (firm to stiff)	73.1	4–8	
		SILTY CLAY (TILL-LIKE) Some sand layers (100 mm thick), gravel, possible cobbles and boulders, grey, wet (very soft to firm)	69.0	9–11	
			66.4		
		Auger Refusal at 11.3 m Depth			



NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. A 19 mm diameter slotted standpipe installed to 11.3 m depth.
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-00203421-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	Dry	11.3
16 days	3.6	

CORE DRILLING RECORD

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

LOG OF TEST PIT OTT-00202368 - BH LOGS - SEPT 20, 2011.GPJ TROW OTTAWA.GDT 6/8/12

Log of Borehole 3

Project No: OTT-00203421-A0

Project: Proposed Condominium Building

Location: 280 Herzberg Road, Ottawa, Ontario

Figure No. 5

Page. 1 of 1

Date Drilled: 'November 17, 2011

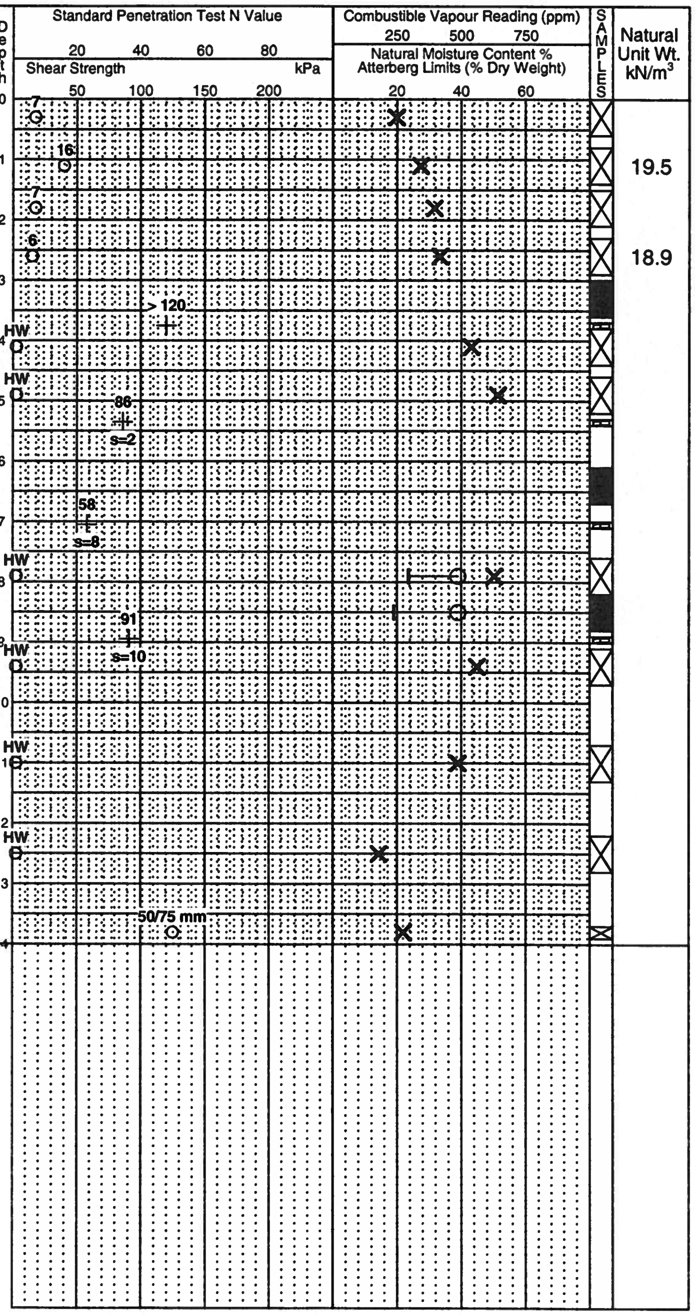
Drill Type: CME-55 (Truck Mount)

Datum: Approximate Elevation

Logged by: ________ Checked by: ________

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		

SOIL DESCRIPTION	Approximate Elevation m	Depth (m)
	77.53	0
TOPSOIL ~ 600 mm	76.9	
SILTY CLAY Some sand, brown, moist (firm to very stiff)	73.93 / 73.5	
SILTY CLAY Grey, moist (stiff)	66.8	
SANDY SILT TO SILTY SAND Clayey, some gravel, grey, wet (very loose) - with some possible cobbles and boulders below 13. 7 m depth	63.5	14
Auger Refusal at 14.0 m Depth		



NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2. A 19 mm diameter slotted standpipe intalled to 14.0 m depth.
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-00203421-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	Dry	14.0
15 days	3.6	

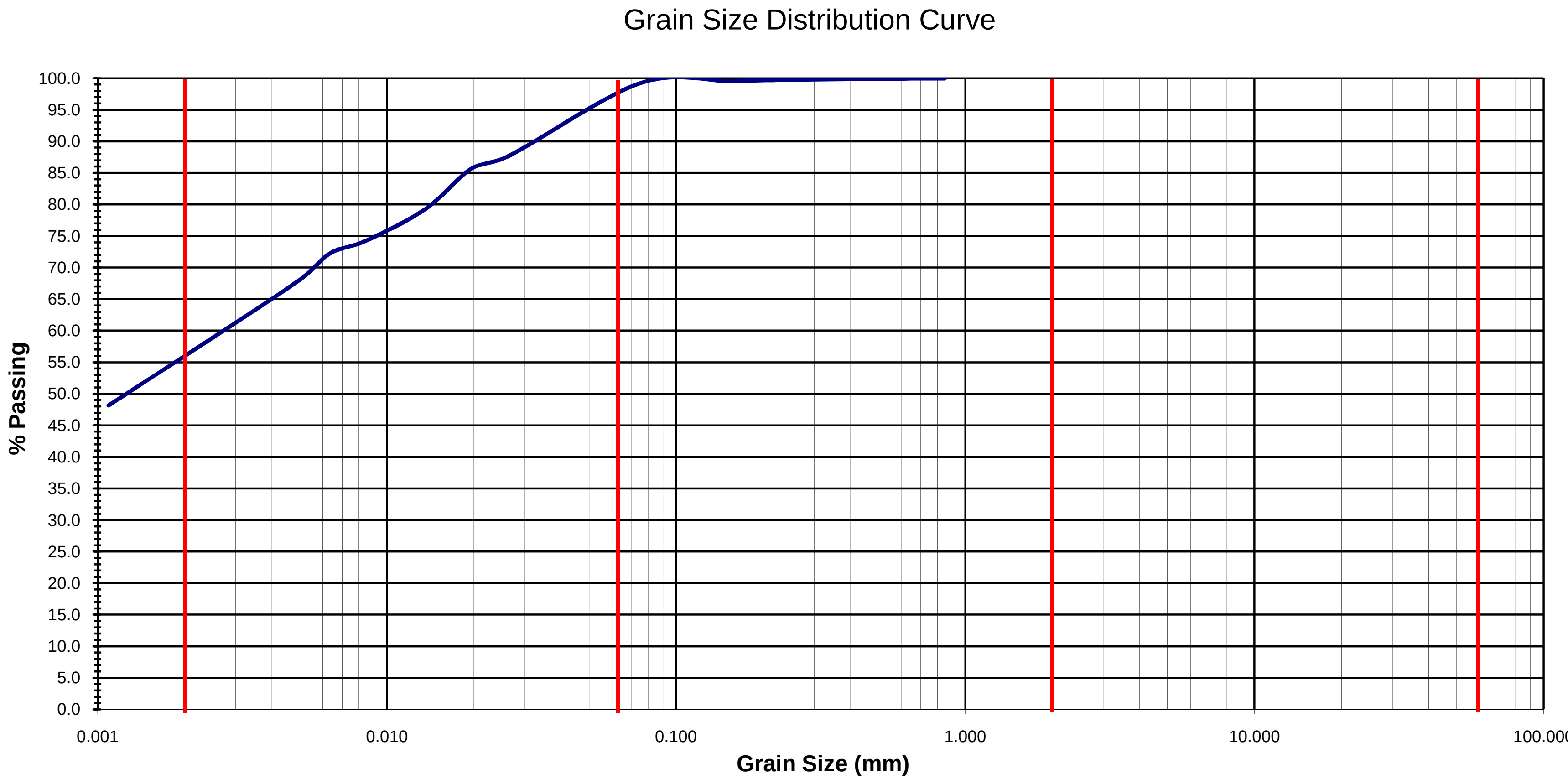
CORE DRILLING RECORD

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

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Method of Test for Particle Size Analysis of Soil

ASTM D-422



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			
Modified M.I.T. Classification										

Exp Project No.:	**OTT-00203421-A0**	**Project Name :**	**Proposed Condominium Building**				
Client :	**7151896 Canada Corporation**	**Project Location :**	**280 Herzberg Road, Ottawa, Ontario**				
Date Sampled :	**November 17, 2012**	**Borehole No:**	**1**	**Sample No.:**	**SS 4**	**Depth (m) :**	**2.3 - 2.9**
Sample Description :	**Silty Clay: Trace Sand**					**Figure :**	**6**

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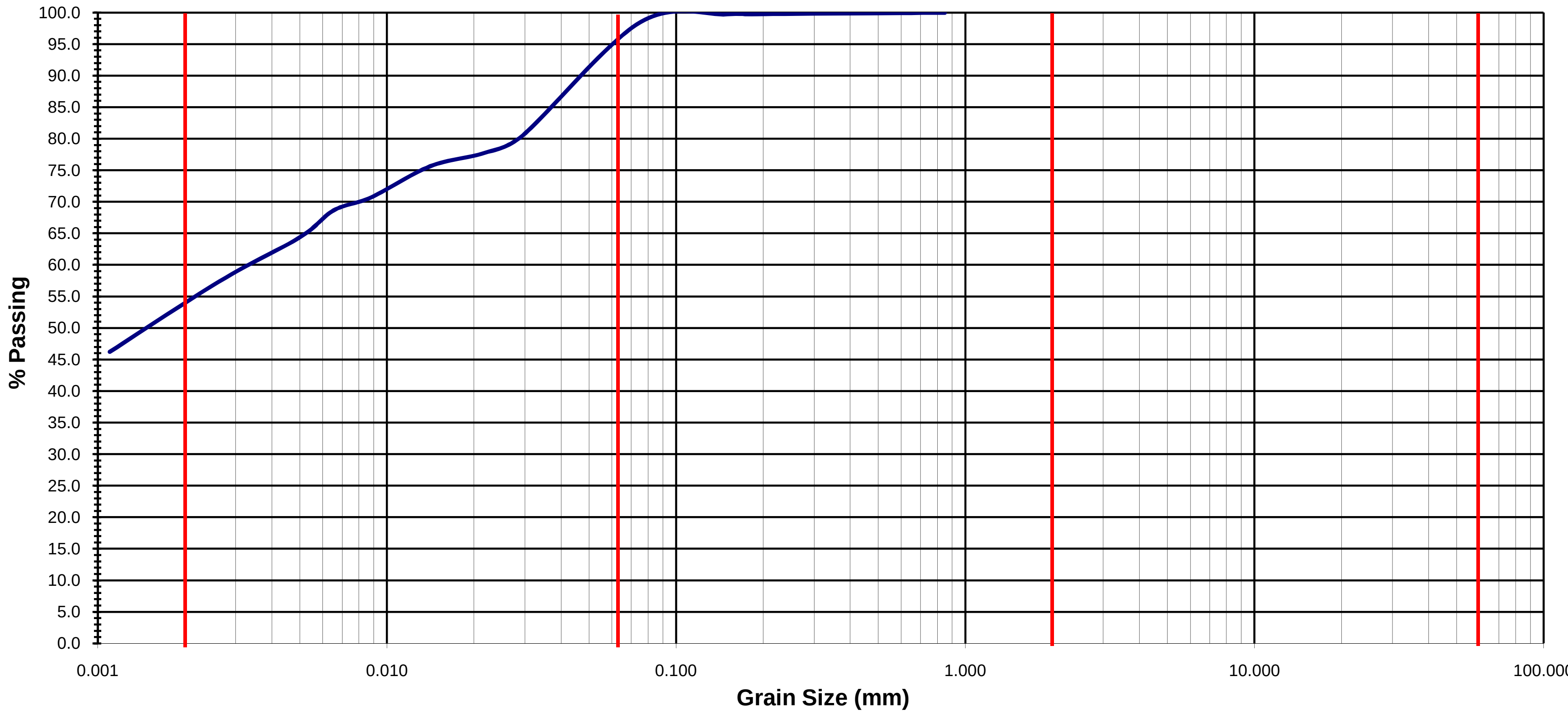
Ottawa, ON K2B 8H6

Method of Test for Particle Size Analysis of Soil

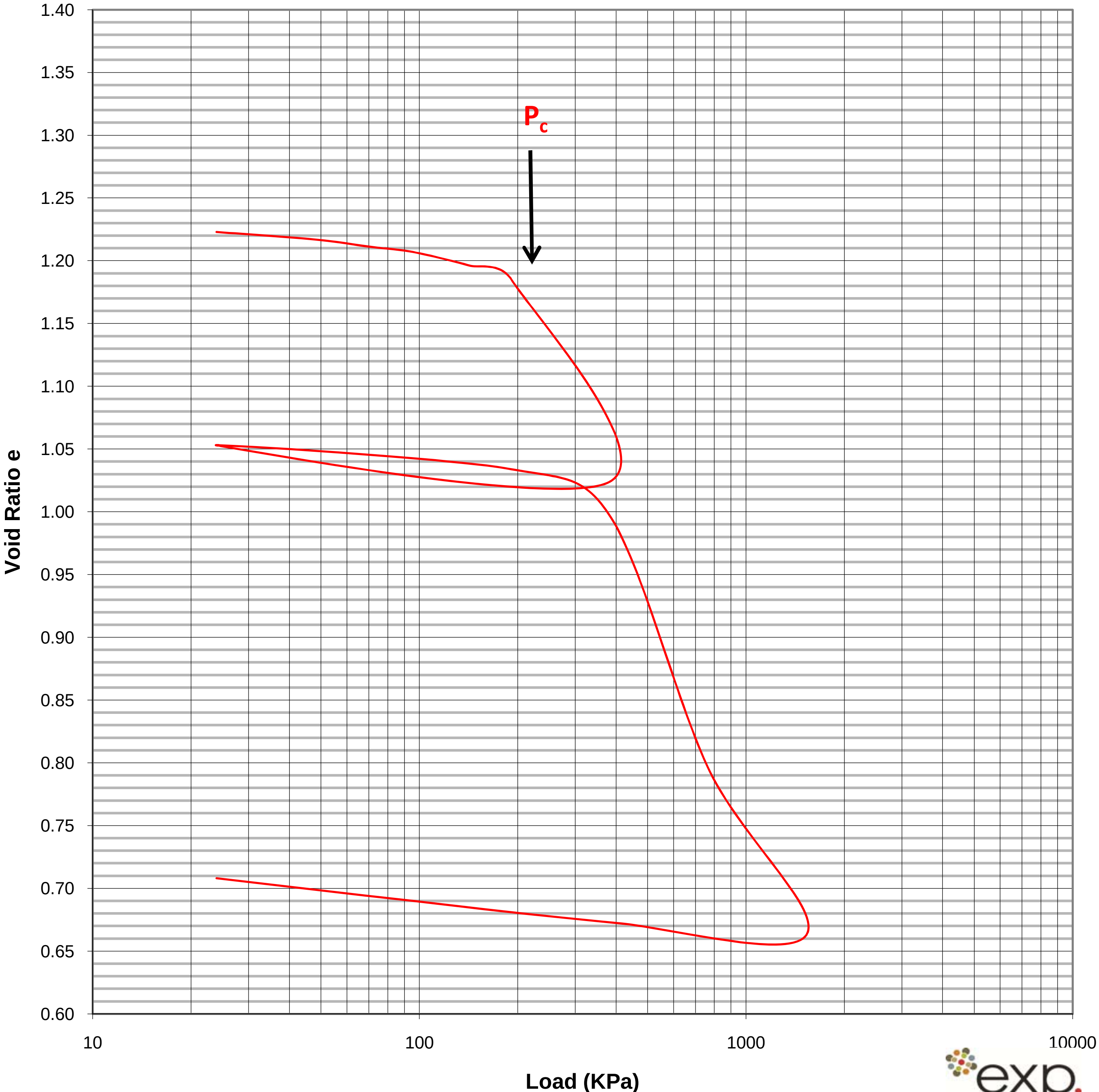
ASTM D-422

Grain Size Distribution Curve



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			
Modified M.I.T. Classification										

Exp Project No.:	**OTT-00203421-A0**	**Project Name :**	**Proposed Condominium Building**				
Client :	**7151896 Canada Corporation**	**Project Location :**	**280 Herzberg Road, Ottawa, Ontario**				
Date Sampled :	**November 17, 2012**	**Borehole No:**	**3**	**Sample No.:**	**SS 11**	**Depth (m) :**	**9.1 - 9.7**
Sample Description :	**Silty Clay: Trace Sand**					**Figure :**	**7**

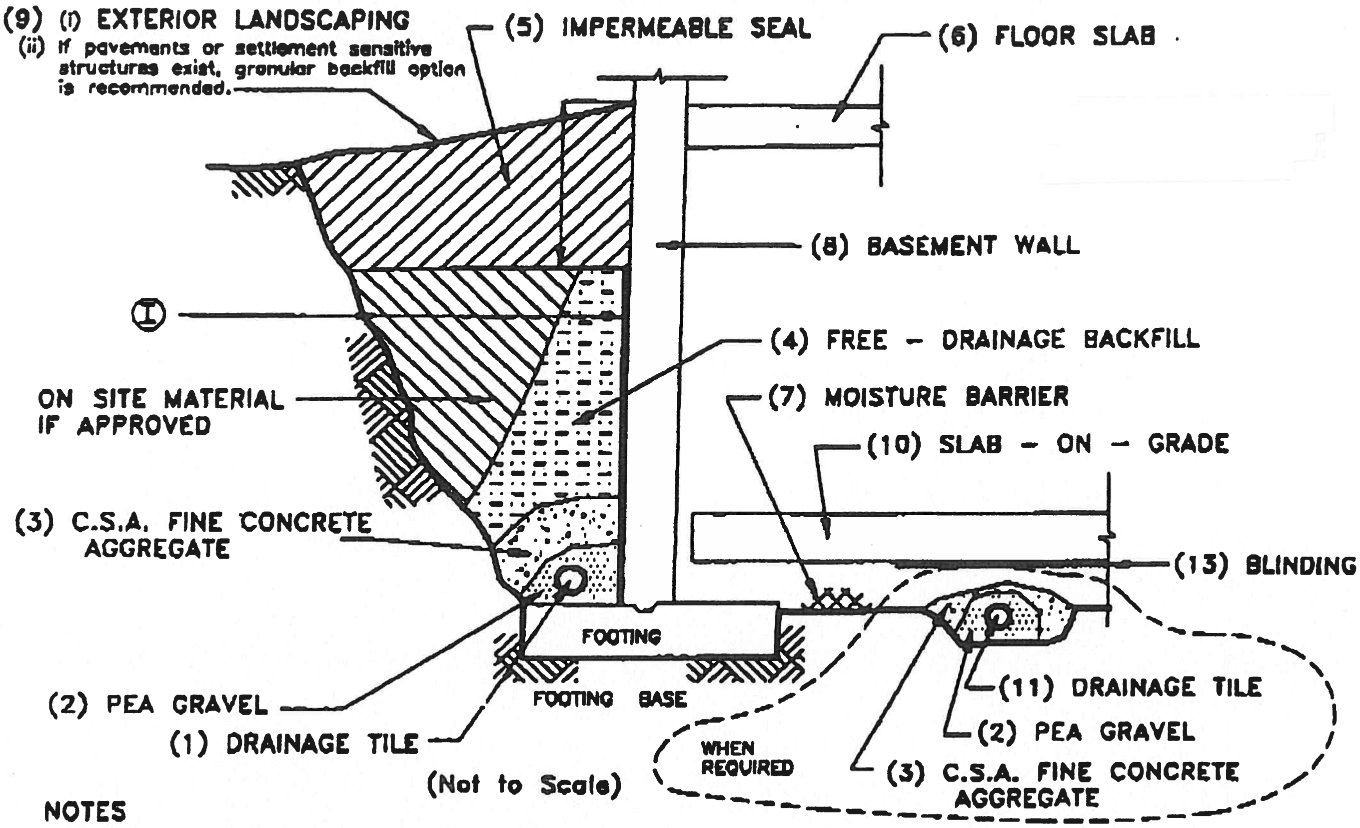


One-Dimensional Consolidation Test Results

CONSOLIDATION TEST DATA SUMMARY					
Borehole	3	P'$_o$	115.0 kPa	Ccr	0.022
Sample No.	TW10	P_c	228 kPa	Cc	0.686
Sample Depth	8.2 to 8.8 m	Oc Ratio	2.0	Wo	45 %
Sample Elev.	69.3 to 68.7 m	Initial Void Ratio	1.235	Unit Wt.	18.7 kN/m^3

Client	**7151896 Canada Corporation**	Project	**OTT-000203421-AO**
Project	**Proposed Condominium Building**	Date	**June-12**
	280 Herzberg Drive, Ottawa, Ontario		

BASEMENT DRAINAGE DRAWING



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 then 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 (I) 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 (I) 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

Appendix A –
Shear Wave Velocity Sounding Survey

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@gprmtl.com
www.geophysicsgpr.com

February 8th, 2012

Transmitted by email: Susan.Potyondy@exp.com
Our Ref.: M-11290-B

Mrs. Susan M. Potyondy, P. Eng.
exp inc.
100-2650 Queensview Drive
Ottawa, ON
K2B 8H6

Subject: **Shear-wave Velocity Sounding, 280 Herzberg Road, Ottawa**

Dear Madam,

Geophysics GPR International Inc. has been requested by **exp** inc. to carry out a shear-wave velocity sounding at a site along Herzberg Road in Ottawa. The site is located just North of the intersection of Terry Fox Drive and Herzberg Road. The geophysical investigations utilized the Multi-channel Analysis of Surface Waves (MASW) and the Extended SPatial AutoCorrelation (ESPAC) methods to generate a shear-wave velocity depth profile, from the surface to 30 meters deep. From these results, the V_{S30} value was calculated to identify the site class.

The surveys were carried out on December 9th, 2011, by Mr. Charles Trottier, M.Sc. Phys., and Mr. Iban Oros. Figure 1 shows the regional location of the site, and Figure 2 illustrates with more details the location of the seismic spread in surface and the characterized area. 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.

ISO 9001

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 (Vs) 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 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.

Survey Design

The spacing between geophones was 3 meters, which means that the total length of the 24 geophone spread was 69 meters for the main spread, and 1 metre (total of 23 meters long) for a one dedicated to the near surface details. Unlike the refraction method, which produces a data point beneath each geophone, the shear-wave depth sounding is the average of the bulk area within the geophone spread, especially for its central half-length. An 80 pounds weight-drop was used as the primary energy source with traces being recorded off both ends of the seismic spread.

Interpretation Method

The main processing sequence involved plotting, picking and 1D inversion of the MASW and ESPAC shot records using the SeisImagerSW™ software. In theory, all the MASW shot 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.

Results

Figure 5 presents the 1-D shear wave velocity values from the inversion models of the merged dispersion curves of the active MASW and ESPAC results.

The V_{S30} value is based on the harmonic mean of the shear-wave velocities, from the surface to 30 meters deep. The V_{S30} value 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 30 meters, single layer response. The calculated V_{S30} value is 437.9 m/s. Details of the V_{S30} calculation are presented in Table 1.

Some very low V_S values were calculated from the surface to 7 meters deep, and some low ones could take place between 7 and approximately 9 meters deep.

CONCLUSION

A seismic shear-wave velocity sounding was realized on a property located North of the intersection of Herzberg Road and Terry Fox Drive, in Ottawa. The location of the seismic spread in surface is presented in Figure 2. This survey aimed to determine the seismic site class according with the V_{S30} value.

The results of the MASW sounding are presented in Figure 5. The V_{S30} value calculated from the sounding results is presented in Table 1.

The calculated V_{S30} value from the 1D MASW sounding is 438 m/s. Based on the V_{S30} value (as determined through the MASW method), Table 4.1.8.4.A of the NBC and the Building Code, O. Reg. 350/06, the investigated site would be categorized as a class "C" ($360 < V_{S30} \leq 760$ m/s).

The seismic velocities calculated for the overburden appear to be very low to low for the upper 7 to 9 meters deep. An appropriate geotechnical evaluation is recommended for these unconsolidated materials.

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.

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

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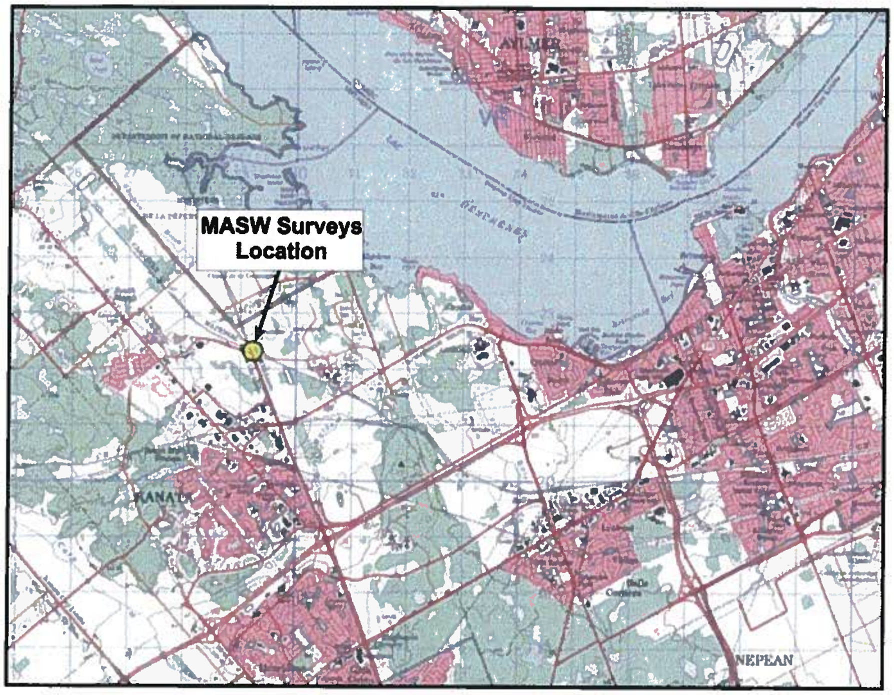

Figure 1: Regional location of the surveyed site
(extracted from the topographic map 031 G/05)

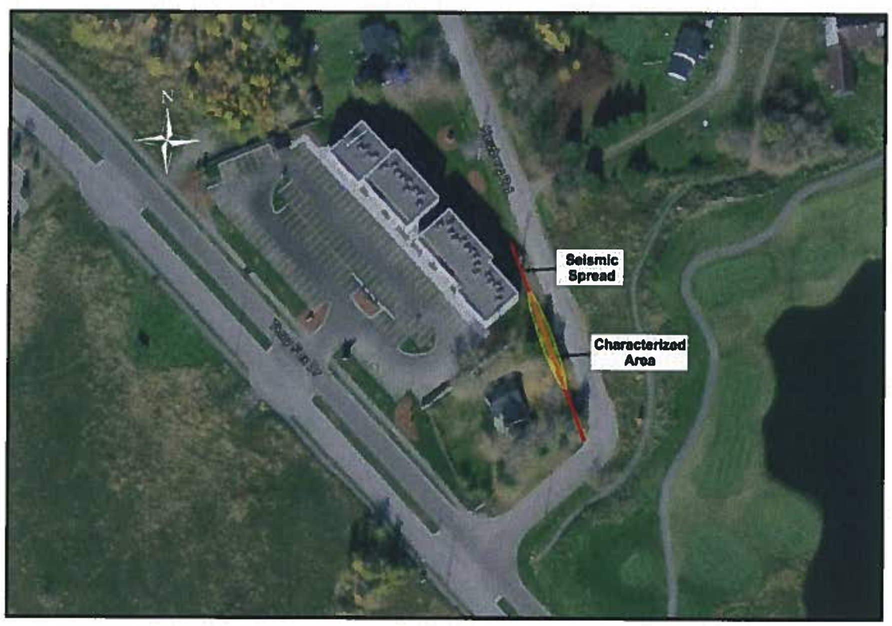

Figure 2: Location of the MASW spread (base picture from Google Earth™)

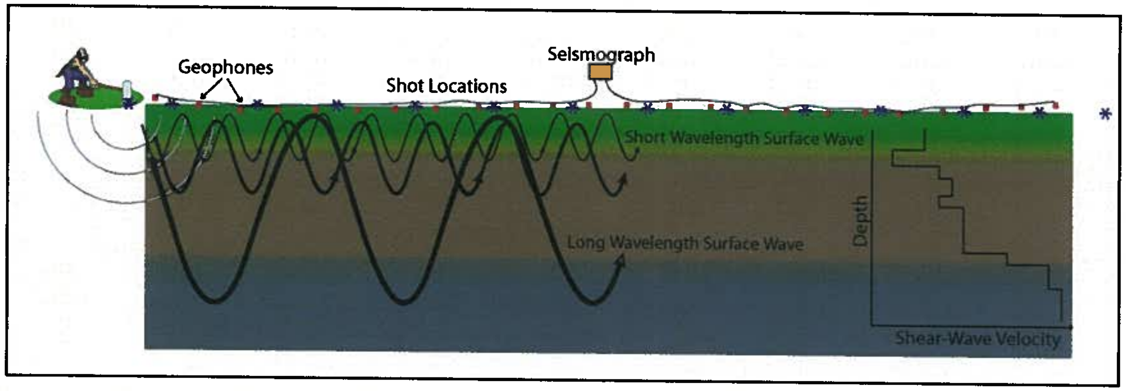

Figure 3: MASW Operating Principle

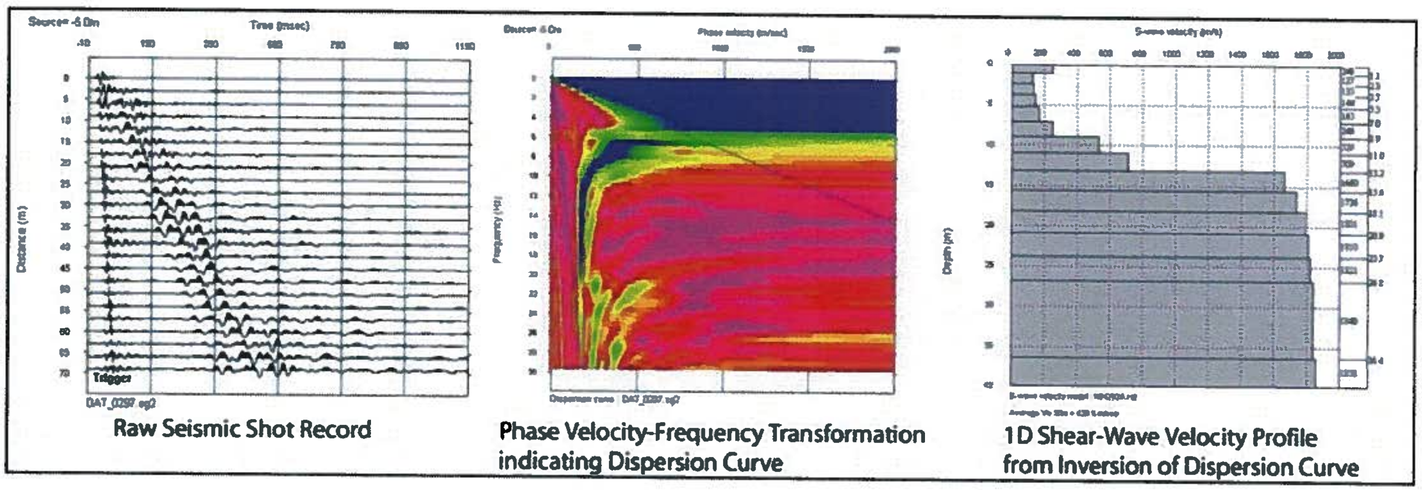

Figure 4: Example of a typical MASW shot record, phase velocity/frequency curve and resulting 1D shear-wave velocity model

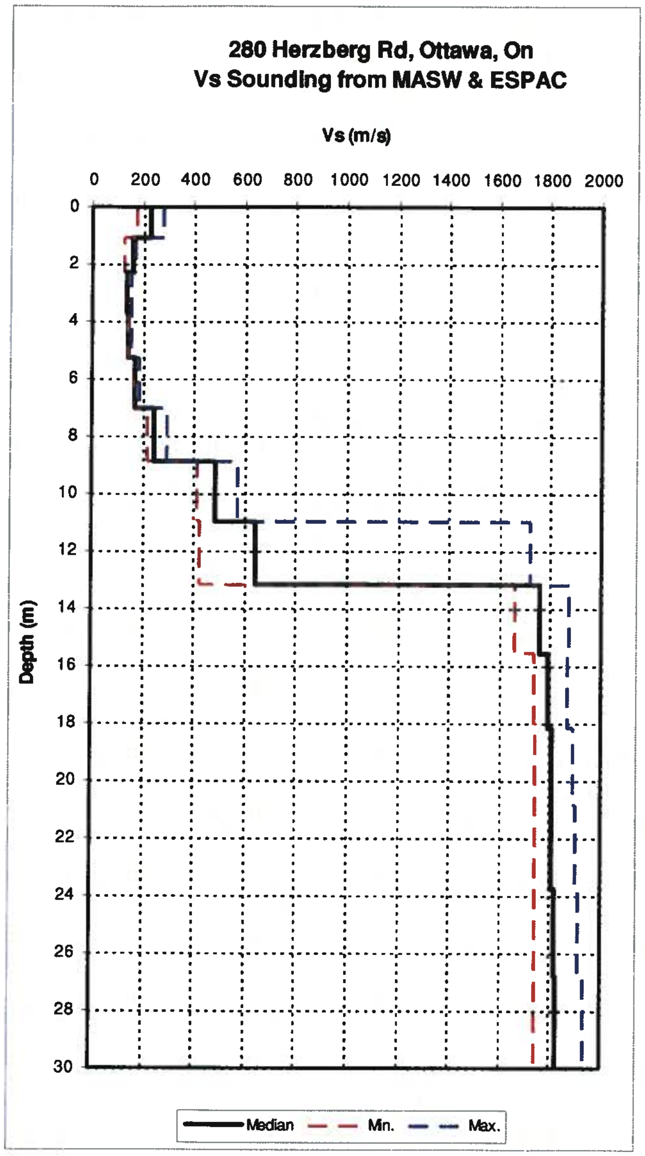

Figure 5: MASW Shear-wave Velocity Sounding

Table 1: MASW Shear-wave Velocity Sounding Results and V_{S30} Calculation

Depth (m)	Vs Min. (m/s)	Vs Median (m/s)	Vs Max. (m/s)	Thickness (m)	Cumulated thickness (m)	Delay for med. Vs (s)	Cumulated delay (s)	Average Vs at given depth (m/s)
0.00	175.0	228.8	280.4					
1.07	125.8	158.8	168.6	1.07	1.07	0.004680	0.004680	228.8
2.31	132.1	135.0	151.4	1.24	2.31	0.007784	0.012464	185.1
3.71	140.2	144.5	153.1	1.40	3.71	0.010377	0.022841	162.3
5.27	162.7	169.8	186.2	1.57	5.27	0.010838	0.033679	156.6
7.01	220.3	247.6	295.8	1.73	7.01	0.010193	0.043872	159.7
8.90	417.3	487.6	575.2	1.90	8.90	0.007658	0.051530	172.7
10.96	427.9	646.5	1719.4	2.06	10.96	0.004225	0.055755	196.6
13.19	1660.5	1758.0	1872.8	2.23	13.19	0.003442	0.059196	222.7
15.58	1736.1	1793.5	1867.8	2.39	15.58	0.001360	0.060556	257.2
18.13	1745.8	1810.2	1889.1	2.56	18.13	0.001425	0.061981	292.5
20.85	1744.2	1811.0	1901.5	2.72	20.85	0.001503	0.063483	328.4
23.74	1740.9	1821.6	1914.3	2.89	23.74	0.001593	0.065076	364.7
26.79	1742.0	1824.6	1936.9	3.05	26.79	0.001674	0.066750	401.3
30.00				3.21	30.00	0.001762	0.068512	437.9

Vs30 (m/s) =	**437.9**
Class :	**C ***

( )* : subject not to encounter a class F condition for the overburden layers.