• Julian Jacobs Architects

Geotechnical Investigation

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Project Name
Proposed Iraq Embassy
215 McLeod Street
Ottawa, Ontario

Project Number
OTT-00204847-A0

Prepared By:

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Date Submitted
August 3, 2012

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August 3, 2012

Executive Summary

Exp Services Inc. (**exp**) is pleased to present the results of the geotechnical investigation recently completed for the proposed Iraq Embassy to be located at 215 McLeod Street, Ottawa, Ontario. Written authorization to proceed with this work was provided by Julian Jacobs Architects via facsimile dated February 8, 2012.

The subject site is currently occupied by the existing embassy building which is in the west half of the site with outdoor paved parking lot in the east half. It is understood plans call for the existing building to be demolished and replaced by a new embassy building. The proposed building will occupy a plan area of 27 by 30 m and consist of four (4) above grade storeys with a sloping ramp on the east side of the building to the single level underground parking garage.

The fieldwork for this investigation was carried out from February 17 to 22, 2012 and consisted of three (3) boreholes and two dynamic cone penetration tests. A dynamic cone penetration test was performed in Borehole 1 from 23.8m depth to refusal at 35.7 m depth. Another dynamic cone penetration test was performed adjacent to Borehole 3 to cone refusal at 34.1 m depth. The fieldwork was supervised on a full time basis by geotechnical personnel from **exp**.

The subsurface conditions consist of a surficial pavement structure underlain by fill to depths ranging from 1.5 to 2.2 m, elevation 68.9 to 69.3 m. The clay to silty clay is grey in colour and has a firm to very stiff consistency. Dynamic cone penetration tests undertaken at the site indicate that the bedrock at the site is likely present at 35.7 and 34.1 m depths, elevation 35.4 and 36.7 m. Review of local geology maps indicates the bedrock is limestone of the Ottawa or Eastview formation.

Upon completion of drilling, all boreholes remained dry. Groundwater levels taken in the standpipes approximately two (2) months following completion of drilling ranged from 3.8 to 4.6 m depths, elevation 66.5 to 67.1 m.

It is understood the grades at the site will not be raised as part of the proposed development. The site is underlain by a silty clay to clay, which is susceptible to being overstressed due to loads imposed on it by the building foundations, fill placement and post development groundwater lowering at the site. Therefore, the serviceability limit state (SLS) bearing pressure and factored geotechnical resistances in limit states design recommended for mat foundation in subsequent sections of this report is based on the assumption that the grade at the site will not be raised from the existing level and that the post development groundwater table may be lower by 1 m. Due to the sensitive nature of the silty clay, if plans are revised to include a raise in site grades, **exp** must be contacted to conduct an additional settlement analysis to determine the feasibility of the site grade raise.

Strip and spread footings are not feasible for the proposed structure since settlements will be more than the normally tolerated limits. The proposed structure may be founded on a raft foundation founded on the silty clay and designed for a bearing pressure at serviceability limit state (SLS) of 100 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 150 kPa. It is recommended that a 50 mm thick layer of concrete mud slab should be placed on surface of the silty clay to protect it from disturbance. 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 following excavation and approval to protect the subgrade from the detrimental effects of weather and construction traffic (such as foot traffic).

Alternatively, the building may be supported by steel H or pipe piles, which may be designed according to the recommendations given in the report.

The retaining walls for the ramp may be founded on spread and strip footings in accordance with recommendations made in the report.

Based on the subsurface soil conditions, the site has been classified as Class D in accordance with Table 4.1.8.4.A of the 2006 OBC. The onsite soils are not considered to be liquefiable during a seismic event.

General Use (GU) Portland cement may be used in the subsurface concrete at this site.

Since the basement floor is anticipated to be below the groundwater level, a permanent perimeter as well as an underfloor drainage system will be required. The finished exterior grade should be sloped away from the building to prevent surface ponding close to the exterior walls.

The subsurface basement walls should be backfilled with free draining granular material.

Excavation of the fill 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.

The excavations at the site should comply with the latest requirements of the Occupational Health and Safety Act (OHSA). The subsurface soils above the groundwater table are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation. Excavations that will extend below the groundwater table are expected to slough and may eventually stabilize at a slope of approximately 2H:1V to 3H:1V.

If the above slopes cannot be achieved due to space restrictions, the excavations would have to be shored. The support system should be designed and installed in accordance with OHSA requirements. In this regard, **exp** can provide additional comments once information is available with respect to the number of basement levels of adjacent buildings, location and invert levels of adjacent underground services and other nearby settlement sensitive features.

Excavations for the building foundations are anticipated to extend approximately 1.5 m below the groundwater level. 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 remove it by pumping from sumps. High capacity pumps may be required within permeable zones of the fill and silty clay to clay.

The soils to be excavated will comprise of limited amount of crushed limestone and a mixture of clayey silty to silty sand fill and native brown and grey silty clay. These materials are not suitable for backfilling purposes and therefore should be discarded. It is anticipated that the majority of the material required for backfilling purposes would need to be imported and should preferably conform to OPSS Granular B Type II and select subgrade material.

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

Legal Notification

This report was prepared by **exp** Services Inc. for the account of **Julian Jacobs Architects.**

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

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1 Introduction

Exp Services Inc. (**exp**) is pleased to present the results of the geotechnical investigation recently completed for the proposed Iraq Embassy to be located at 215 McLeod Street, Ottawa, Ontario. Written authorization to proceed with this work was provided by Julian Jacobs Architects via facsimile dated February 8, 2012.

The subject site is currently occupied by the existing embassy building which is a two (2) storey building located in the west half of the site with outdoor paved parking lot in the east half. It is understood plans call for the existing building to be demolished and replaced by a new embassy building. The proposed building design and site development will include the following:

- The site grades will not be raised;
- The building will occupy a plan area of 27 by 30 m and consist of four (4) above grade storeys with a sloping ramp on the east side of the building to the single level underground parking garage;
- Ground floor elevation will be at 71.6 m and basement (garage) floor elevation at 68.3 m;
- The parking garage will occupy the majority of the building footprint. The southern part of the basement will house the majority of the mechanical and electrical equipment as well as two (20 offices for staff;
- Type of construction includes mixed reinforced concrete with cast-in-place slabs and shear walls and structural steel frame. The basement and floor levels will be reinforced concrete and the roof structure will be structural steel frame with steel deck;
- The preferred foundation type is strip and spread footings; and
- Approximate column load will be 4000 kN at ultimate limit state (ULS) and 3000 kN at serviceability limit state (SLS) with column spacing at 7.5 m.

This geotechnical investigation was undertaken to:

(a) Establish the subsurface soil and groundwater conditions at the borehole locations;

(b) Comment on grade raise restrictions;

(c) Recommend foundation types and bearing pressure/factored geotechnical resistance in limit states design of the founding soils and bedrock as well as comment on anticipated settlements of the proposed foundations;

(d) Provide lateral resistance of the foundations;

(e) Estimate the modulus of subgrade reaction of the founding soil;

(f) Provide site classification for seismic site response in accordance with the current Ontario Building Code (OBC) and comment on the liquefaction potential of onsite soils during a seismic event;

(g) Discuss slab on grade construction and permanent drainage requirements;

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

(i) Discuss excavation conditions and de-watering requirements for the construction of the new building;

(j) Assess backfilling requirements and suitability of on-site soils for backfilling purposes;

(k) Comment on the corrosion potential of the subsurface soils to buried steel and on subsurface concrete requirements; and,

(l) Comment on tree planting.

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 in the Centretown area of the City of Ottawa. Development in the surrounding area is mixed commercial and residential. The site topography is relatively flat with ground surface elevation in the order of 71.0 m. The site is occupied by a two (2) storey embassy building in the west half of the site and outdoor paved parking area in the east half. The existing building has one (1) level basement. The adjacent existing properties are occupied by a high-rise building to the east, two (2) storey building with garage to the west, outdoor paved parking lot to the north with buildings beyond and McLeod Street to the south.

Details regarding the foundation type and founding level of the adjacent building were not available at the time this report was prepared.

3 Procedure

The fieldwork for this investigation was carried out from February 17 to 22, 2012 and consisted of three (3) boreholes. The boreholes (1 to 3) were advanced to termination depths of 8.5 to 23.8 m. A dynamic cone penetration test was performed in Borehole 1 below the 23.8 m termination depth to cone refusal depth at 35.7 m. In addition, a dynamic cone penetration test (Borehole 3A) was conducted adjacent to Borehole 3 from 2.0 m depth to cone refusal at 34.1 m depth. The fieldwork was supervised on a full time basis by geotechnical personnel from **exp**.

The borehole locations were established in the field by **exp.** The borehole elevations were estimated from spot elevations shown on the survey plan, titled, "Surveyor's Real Property Report Part 1 – Plan Showing Lots 3 and 4 and Part of Lot 5 Registered Plan No. 30 City of Ottawa", dated November 8, 2011 and prepared by J.D. Barnes Limited. Therefore, the borehole elevations should be considered approximate. The borehole location plan is shown on Figure 1.

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. Two (2) relatively undisturbed thin walled tube samples of the cohesive soil were obtained at select depths from one (1) borehole. The undrained shear strength of the cohesive soils was established by performing in-situ field vane and penetrometer tests. Dynamic cone tests were conducted in Borehole 1 below the sampled depth of 23.8 m to dynamic cone refusal depth at 35.7 m and adjacent to Borehole 3 from 2.0 m to dynamic cone refusal at 34 1 m depth. 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, 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 determination as well as soil classification tests including grain size analysis and Atterberg limit determination and corrosion analysis (pH/sulphate, electrical conductivity, resistivity) were conducted on select soil samples. One dimensional consolidation test was conducted on one (1) undisturbed thin wall tube sample.

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 2 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 Pavement Structure

The boreholes were located within the paved area of the site. The pavement structure consists of 50 mm asphaltic concrete and 200 to 300 mm thick granular base.

4.2 Fill

The pavement structure is underlain by fill which extends to depths ranging from 1.5 to 2.2 m, elevation 68.9 to 69.3 m. The fill consists of a mixture of clayey silt, silty sand and sand and gravel with fragments of concrete, brick and plaster. The fill is in a compact state and has a natural moisture content of 4 to 18 percent.

4.3 Clay to Silty Clay

Clay to silty clay was encountered in all the boreholes beneath the fill. The upper 0.8 to 1.3 m of the clay to silty clay is a weathered brown crust. The consistency of the crust (based on shear strength measurements from penetrometer tests ranging from 72 to 192 kPa), is stiff to very stiff. The natural moisture content ranges from 40 to 64 percent and the natural unit weight is 17.9 kN/m^3.

Below the weathered zone, the clay to silty clay is grey in colour and contains some wet silt pockets. Shear strength measurements from in situ vane tests range from 43 to 101 kPa indicating a firm to stiff consistency. Below 20.0 m depth (elevation 51.0 m) in Borehole 1, shear strength measurements were greater than 120 kPa indicating a very stiff consistency. The clay to silty clay is considered to have a sensitivity ranging from medium to high. The Atterberg limit tests conducted on two (2) samples of the grey silty clay gave liquid and plastic limits of 67 and 68 and 27 and 33 percent, respectively. The limit test results indicate the grey silty clay may be classified as clay having a high plasticity. Grain size analysis of two (2) samples of the silty clay indicate a soil composition of 76 and 83 percent clay, 15 and 23 percent silt, 1 and 2 percent sand. The grain size distribution curves are shown on Figures 6 and 7. Based on the results of the grain size analysis, the soil may be classified as a clay with some silt, trace of sand to silty clay with trace of sand.

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 2 from 7.6 to 8.2 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,	Estimated Pre-consolidation Pressure, kPa	Over-consolidation Ratio	Initial Void Ratio	Compression Index	Re-compression Index
2	TW 9	7.6 – 8.2 (63.4 – 62.8)	72.7	87	192	2.0	2.053	1.2	0.061

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

4.4 Inferred Bedrock

Dynamic cone penetration tests undertaken below the 23.8 m sampled depth in Borehole 1 and below the 2.0 m power augered depth in Borehole 3A, indicate cone refusal was met on inferred bedrock at 35.7 and 34.1 m, elevation 35.4 and 36.7 m, respectively. Review of local geology maps indicates the bedrock is limestone of the Ottawa or Eastview formation.

4.5 Groundwater Conditions

Upon completion of drilling, all boreholes remained dry. Groundwater level measurements taken approximately two (2) months following completion of drilling in the standpipes installed in Boreholes 1 to 3 ranged from 3.8 to 4.6 m depths, elevation 66.5 to 67.1 m.

Water levels were made in the exploratory boreholes 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 Site Grade Raise Restrictions

It is understood the grades at the site will not be raised as part of the proposed development.

The proposed development will result in loading the clay due to building foundations, grade raise and post development groundwater lowering at the site. It is imperative that the clay is not loaded to in excess of 80 percent of its over consolidation pressure to ensure that settlements of the structure will be within tolerable limits. Therefore, the SLS bearing pressure and factored geotechnical resistances in limit states design estimated for raft foundation in subsequent sections of this report are based on the assumption that the grade at the site will not be raised from the existing level. It has been assumed that the post development groundwater table may be lower by up to 1.0 m.

Due to the sensitive nature of the silty clay, if design plans are revised to include grade raise, **exp** must be contacted to conduct a settlement analysis to determine if the settlements will be within tolerable limits due to the increased loads.

6 Foundation Considerations

It is considered that spread and strip footings are not feasible for the proposed structure since the resulting settlements will exceed the normally tolerated limits of 25 mm total and 19 mm differential movements. It is therefore recommended that the proposed structure should be founded on a raft foundation. Steel H or closed end pipe piles driven to practical refusal on bedrock anticipated at depth of 34.1 m to 35.7 m are also feasible but may be less economical compared to raft foundations.

6.1 Raft Foundation

The proposed structure may be supported by a raft foundation founded at elevation 67.7 m on 300 mm of Granular B Type II pad compacted to 100 percent standard Proctor maximum dry density set on the native silty clay. The raft may be designed for serviceability limit state (SLS) bearing pressure of 100 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 150 kPa.

It is understood the total building unfactored dead load is 90,000 kN and the unfactored live load is 25,000 kN and includes the weight of an assumed 600 mm thick raft foundation under the 27 by 30 m building footprint. This results in total unfactored dead and live loads of 115,000 kN. The weight of the soil that will be permanently excavated and removed from within the building footprint is estimated at 40,400 kN. Therefore, the net increase in load on the silty clay from the building is 74,600 kN which translates to a net load stress increase of 92 kPa over the building footprint area. Based on the net load stress increase, the settlement of the raft foundation is calculated to be approximately 80 mm with differential settlement of 40 mm (50 percent of the total settlement).

However, it is noted that the differential settlement will not result in any distress to the structure. Since the raft is rigid, this may result in very slight tilting of the structure.

6.2 General Comments

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

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

6.2.1 Modulus of Subgrade Reaction

The modulus of subgrade reaction of the silty clay that may be used for design is estimated at 15 MPa/m.

6.3 Pile Foundations

6.3.1 Steel H and Pipe Piles

The building may be supported by piles driven to practical refusal on the limestone bedrock inferred at 35.7 and 34.1 m, elevation 35.4 and 36.7 m.

Closed end concrete filled steel pipe or H piles driven to practical refusal within the limestone bedrock, anticipated at 0.5 to 1.0 m depths below the bedrock surface, may be designed for the factored geotechnical resistances at ULS shown in Table No. II.

Table No. II: Factored Geotechnical Resistance at USL 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.

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

The limit state design will be governed by the ULS value since the bedrock is considered to be a non-yielding material and the bearing pressure at serviceability limit state (SLS) to cause 25 mm of deformation will be much greater than the factored geotechnical resistance at ULS.

The depth to bedrock will have to be confirmed by conducting an additional geotechnical investigation consisting of one (1) borehole advanced to bedrock and coring a 3.0 m length of the bedrock. The borehole should be sampled over its entire depth to the bedrock surface to determine the presence of cobbles and boulders and the need to equip the piles with driving shoes. The additional investigation should also include probeholes to the bedrock surface to provide accurate estimates of pile lengths.

In order to achieve the capacities given above, the pile driving hammer must seat the pile into the inferred 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 inferred 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.

A number of test piles (3 percent of the total number of piles) should be monitored with the Pile Driving Analyzer during the initial driving and re-striking at the beginning of the project and 3 percent of the piles tested should be subjected to CAPWAP analysis. This monitoring will allow 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 inferred 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. For guidance, the piles should be spaced at twice the average pile diameter or 1.75 times the diagonal dimension of the pile cross-section but not less than 600 mm.

The exterior pile caps should be protected from frost action. This may be accomplished by providing the pile caps with 1.5 m of earth cover.

6.3.2 Lateral Capacity of Piles

Since the piles will extend to the bedrock at inferred depths of 34.1 and 35.7 m, they are classified as long piles. Based on the assumption that the piles will be designed for fixed (restrained) head condition, the ultimate lateral resistance for long piles installed in cohesive soil may be computed from the following equations using Broms' method (reference: "U.S. Army Corps of Engineers, Engineering and Design, Bearing Capacity of Soils, Engineer Manual 1110-1-1905", dated October 1992):

Long Piles:

$$T_u = 9C_uB_s \left[\left(2.25B_s^2 + \frac{4}{9} \times M_y\right)^{1/2} - 1.5B_s \right]$$

$$f = \frac{T_u}{9C_uB_s}$$

T_u	=	lateral resistance at ULS; kN
C_u	=	undrained shear strength of soil, kPa (refer to Table No. III)
B_s	=	diameter of pile; m
M_y	=	ultimate resisting bending moment of entire cross-section; kN-m
γ'	=	effective unit weight of soil; kN/m^3 ; refer to Table No. III
Z	=	section modulus; m^3
$(1.5B_s + f)$	=	distance below ground surface to point of maximum bending moment in cohesive soil; m

The calculated ultimate lateral resistance should be multiplied by 0.5 to determine the factored lateral resistance at ULS.

The lateral resistance and deflection of the caisson may also be computed by proprietary software computer programs such as LPILE using estimated soil parameters shown in Table No. III.

If higher lateral capacity is required for piles, the installation of batter piles should be considered.

Table No. III: Estimated Soil Parameters

Soil Type	Natural Unit Weight, Y(kN/m^3)	Undrained Shear Strength (kPa)	Cohesion c' (kPa)	Angle of Internal Friction, Ø'	Effective Unit Weight, Y' (kN/m^3)
Brown and Grey Silty Clay	17	85	11	28	7.2

6.3.3 General Comments

The recommended bearing pressure at SLS and factored geotechnical resistances 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 Access Ramp

7.1 Retaining Wall Footings

An access ramp to the underground parking garage is to be constructed on the east side of the site. The retaining walls for the access ramp may be founded on strip footings set on the silty clay. A SLS bearing pressure of 100 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 150 kPa may be used for designing the footings.

It is currently not known whether the apartment building located to the east of the site has one or two basement levels. Depending on the number of basements and the method of construction, it is possible that deep fill may be present in the vicinity of this structure thereby necessitating the use of pile foundations for the retaining wall(s).

A minimum of 2.4 m of cover should be provided to the footings to protect them from damage due to frost penetration since the ramp would be exposed to the environment.

The footings of the retaining walls would step down from the south property boundary to the north. These footings should be stepped at a slope of 1.5H:1V to prevent transference of stresses from the higher footing to the lower one. Also, the lower portion of the footing should be constructed first to prevent undermining of the higher footing during subsequent construction.

7.2 Lateral Earth Pressure on Retaining Walls

The retaining wall will be subjected to lateral active static earth as well as lateral dynamic earth forces during a seismic event as indicated in the following equation:

$$P_{ae} = 0.22\gamma H^2 + k_a q H$$

where
- P_{ae} = total lateral active earth thrust (static and dynamic) on wall, kN/m
- k_a = static active earth pressure coefficient for Granular B Type II level backfill = 0.33
- γ = unit weight of Granular B Type II backfill = 22 kN/m^3
- H = height of wall, metre
- q = surcharge that may be acting close to the subsurface walls, kPa

The dynamic active earth pressure is an inverted triangle. The total lateral active earth thrust should be assumed to act 0.35H from the bottom of the retaining wall.

The passive resistance to sliding of the wall will be provided by soil in front of the wall and friction between the footing and the founding soil. The passive earth thrust acting against the walls and the dynamic earth thrust induced during a seismic event may be computed from the expression given below. As for the active case, the dynamic passive earth thrust is an inverted triangle.

$$P_{pe} = 1.1\,\gamma\,h^2$$

where Ppe = total lateral passive earth thrust (static and dynamic) on wall, kN/m

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

h = height of backfill in front of wall, metre

It is noted that the dynamic component acts in the opposite direction to the static component thus reducing the available passive resistance.

The above expressions assume that a permanent drainage system together with free draining granular backfill material (Granular B Type II) adjacent to the walls will prevent the build up of hydrostatic pressure behind the wall. The backfill should be compacted to 95 percent SPMDD.

The resistance to sliding of the footings will be provided by friction between the footing concrete and the sandy silt to silty sand till. The unfactored geotechnical resistance to sliding of the footing at ULS may be computed using a friction angle of 28 degrees between the footing and the underlying silty clay. Therefore, the resulting unfactored coefficient of friction at ULS is 0.53.

7.3 Access Ramp Fill

All of the existing fill from the ramp area should be sub-excavated to the underlying silty clay. The exposed subgrade should be proof rolled. Any soft areas identified should be sub-excavated and backfilled with engineered fill. Any fill required to raise the grade to subgrade level should conform to OPSS for Granular B, Type II. It should be placed in maximum of 300 mm lift thickness and each lift compacted to at least 98 percent of standard Proctor maximum dry density.

8 Seismic Site Classification and Liquefaction Potential of On Site Soils

The subsoil and groundwater information at the site has been examined in relation to Section 4.1.8.4.A of the 2006 Ontario Building Code (OBC). The subsoils at the site consist of fill to 2.1 m depth underlain by an extensive firm to very stiff native silty clay. Inferred bedrock was contacted at 34.1 to 35.7 m depths.

The average shear strength of the silty clay was calculated to be 85 kPa and the site may be classified as Class D in accordance with Table 4.1.8.4.A of the 2006 OBC.

The onsite soils are not considered to be liquefiable during a seismic event.

9 Floor Slab and Drainage Requirements

It is recommended that perimeter as well as underfloor drains should be provided for the building. 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 No. 10). The perimeter drains may also consist of 100 mm diameter 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 operational should the other fail.

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

If the lowest level of the underground parking garage will be paved, the pavement structure given in Table 2 may be used for design purposes.

10 Lateral Earth Pressure Against Subsurface Walls

The subsurface walls should be backfilled with free draining material, such as Granular B Type II and equipped with a permanent drainage system to prevent the hydrostatic 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 'at rest' condition 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 should be assumed to act 0.6 H from the bottom of the wall.

The 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. The drainage system should be suitably outletted as previously discussed.

The subsurface walls should also be waterproofed.

The perimeter drainage system should be connected to a separate sump from the underfloor drainage system and also equipped with back-up pumps and generators.

11 Excavations and De-Watering Requirements

11.1 Excavations

Excavations for the building foundations are anticipated to extend to 3.5 m depth and are expected to be about 1 m to 1.5 m below the groundwater level.

Excavation of the fill and silty clay may be undertaken using a large mechanical shovel capable of removing 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 depth of 3.5 m, the silty clay is not considered susceptible to heave. Should excavations extend below the 3.5 m depth, **exp** should be contacted to determine if the base of the deeper excavation within the silty clay is susceptible to heave.

The excavations at the site should comply with the current Occupational Health and Safety Act (OHSA). The subsurface soils are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation. Excavations that will extend below the groundwater table are expected to slough and may eventually stabilize at a slope of approximately 2H:1V to 3H:1V.

If the above slopes cannot be achieved due to space restrictions, the excavations would have to be constructed within the confines of an engineered support system (such as soldier pile and timber lagging) designed and installed in accordance with OHSA requirements.

The shoring 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; kPa

k = earth pressure coefficient applicable

γ = unit weight of soil to be retained, estimated at 20 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; 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 appropriate k values depending on the location of any settlement-sensitive structures and services under the streets. The traffic loads on the streets should be considered as surcharge. The shoring system may consist of soldier pile and timber lagging tied back by soil anchors. It is noted that a continuous caisson wall tied back with soil anchors may be required in areas where the excavation comes close to and extends below the foundations of the existing buildings. Alternatively, the footings of such buildings may be underpinned.

Details of the adjacent buildings such as number of basement levels, depth of lowest floor level, type and founding depth of foundation as well as invert levels of adjacent underground services and the location and depth of any other adjacent settlement sensitive features are required to determine the most appropriate shoring system (s). Once these details are available, **exp** can provide additional geotechnical engineering comments and recommendations with respect to shoring design.

It is recommended a conditions 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.

11.2 De-Watering Requirements

Based on the above elevations, excavations for the building foundations are anticipated to be up to 1.5 m below the groundwater level. 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 remove it by pumping from sumps. High capacity pumps may be required within permeable zones of the fill and silty clay to clay.

Although this investigation has estimated the groundwater levels at the time of the field work, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring and excavating techniques and which may affect the type and nature of dewatering 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 dewatering systems.

12 Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes

The on-site materials are not suitable for backfilling purposes and should be discarded. Therefore, it is anticipated that the majority of the material required for backfilling purposes would need to be imported and should preferably conform to the following specifications:

(a) Under floor fill including engineered fill and backfilling in footing and service trenches inside the building - OPSS 1010 Granular B Type II placed in 300 mm thick lifts with each lift compacted to 98 percent SPMDD.

(b) Backfill against exterior subsurface walls - OPSS 1010 Granular B Type II placed in 300 mm thick lifts with each lift compacted to 95 percent SPMDD.

13 Corrosion Potential of Subsurface Soils to Buried Concrete and Steel Structures

Soil samples were submitted for laboratory analysis to assess the corrosion potential of the subsurface soils to buried steel. A summary of the laboratory test results are summarized in Table No. IV.

Table No. IV: Laboratory Test Results

Borehole No.	Depth (m)	Soil Type	pH	Sulphate (%)	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)
1	4.0 – 4.6	Clay	8.2	0.01	0.68	1470
2	2.3 – 2.9	Silty Clay	7.4	0.05	2.73	366
3	6.1 – 6.6	Silty Clay	8.3	0.03	0.48	2080

The test results indicate the soil has a negligible potential for sulphate attack on concrete. Therefore, it is recommended the subsurface concrete selected for this project be in accordance with the current Canadian Standards Association (CSA - A23).

The tests results also indicate the soil has a severe corrosion potential to steel. It is recommended that subsurface steel structures/members be protected from corrosion. A corrosion specialist should he consulted to determine the most appropriate corrosion protection method for this project.

14 Pavement Structure Thickness

It is currently not known whether the ramp to the garage would be paved or will be constructed as a concrete slab-on-grade. 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 recorded for this site is PG 58-34.

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 Ontario Provincial Standard Specifications (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.

15 Tree Planting

The clay in the Ottawa area is prone to shrinkage on drying. During the dry seasons, the tree roots draw moisture from the clay, thereby resulting in drying and subsequent shrinking of the clay. This process is largely not reversible. Therefore settlement and cracking of structures can result if trees are planted too close to them.

Published literature indicates that a good working rule is to preferably plant a tree no nearer to a building on shrinkable clay than the eventual height to which the tree may be expected to grow. Obviously, evergreens are better as they have a lower water demand than deciduous trees.

For additional guidance, an arborist should be consulted.

16 General Closure

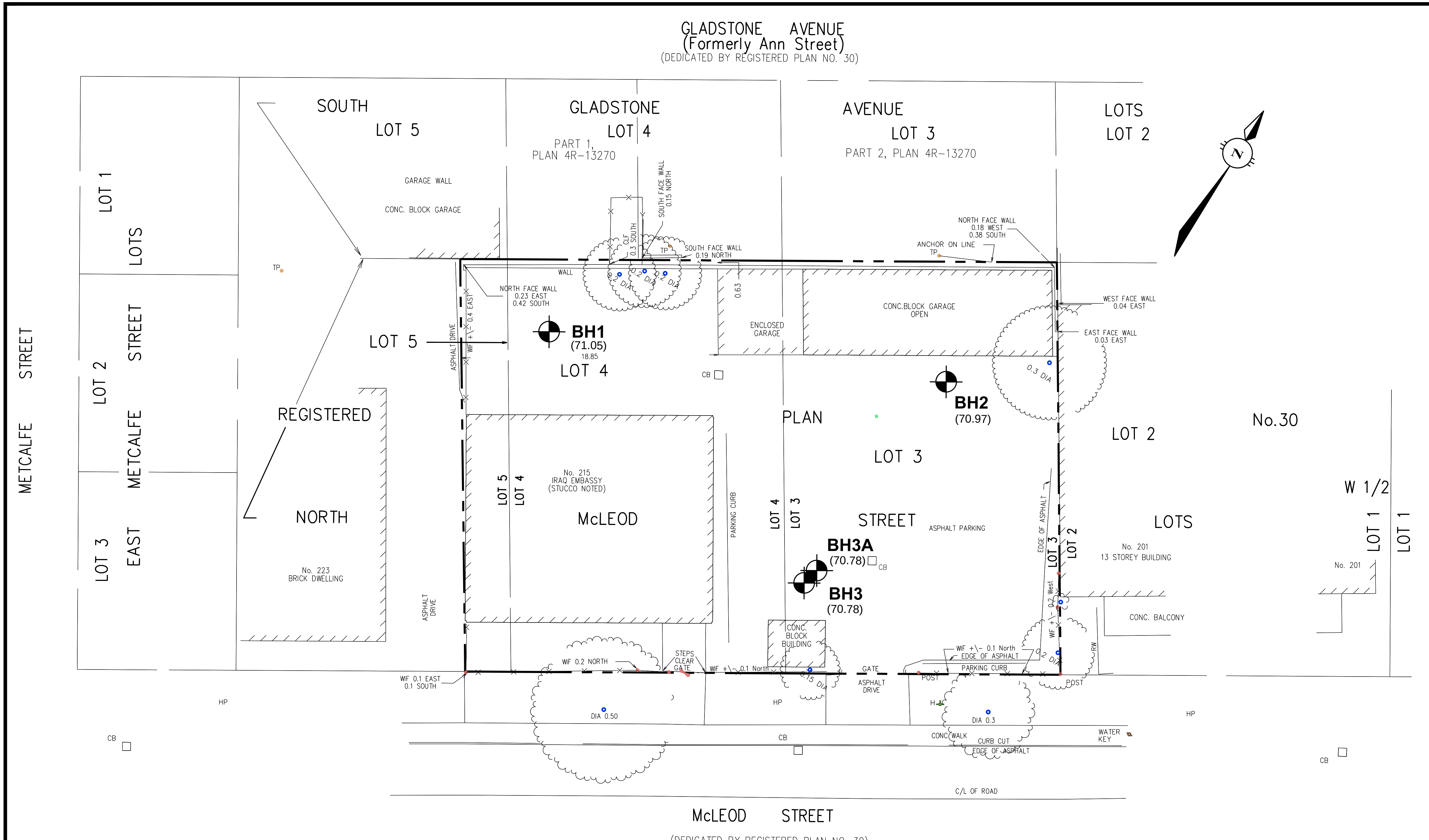
The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than 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.

Exp also recommends that once the standpipes are no longer required, they be decommissioned in accordance with Ontario Regulation 903.

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

Figures



LEGEND

BH2 (70.97) — BOREHOLE NUMBER AND LOCATION; APPROXIMATE GROUND SURFACE ELEVATION (m)

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. ASPHALT 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 J.D. BARNES LTD., REFERENCE NO. 11-10-437-01, DATED NOVEMBER 9, 2011.

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• BUILDINGS • EARTH & ENVIRONMENT • ENERGY •
• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

CLIENT: JULIAN JACOBS ARCHITECTS

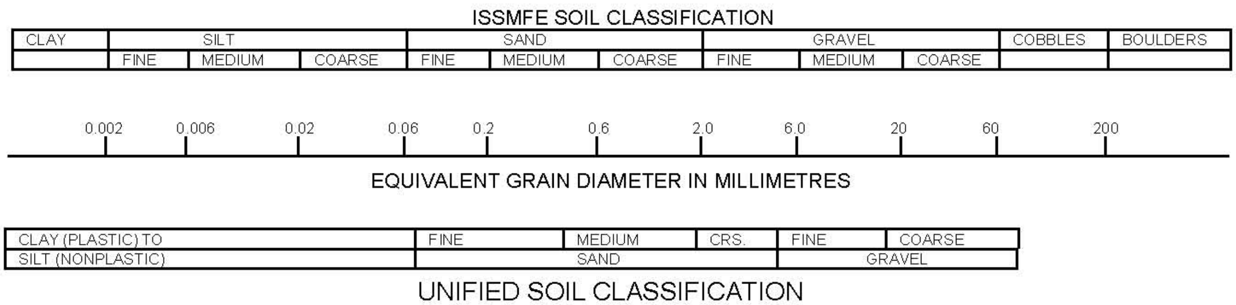
TITLE: BOREHOLE LOCATION PLAN
IRAQ EMBASSY, 215 McLEOD ST., OTTAWA,ON

scale: 1:300
date: APRIL 2012
drawn by: M. NUGENT
project no.: OTT-00204847-A0

Figure 1

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 1

Project No: OTT-00204847-A0

Project: Geotechnical Investigation - Proposed Iraq Embassy

Location: 215 McLeod Street, Ottawa, Ontario

Date Drilled: 'February 21 and 22, 2012

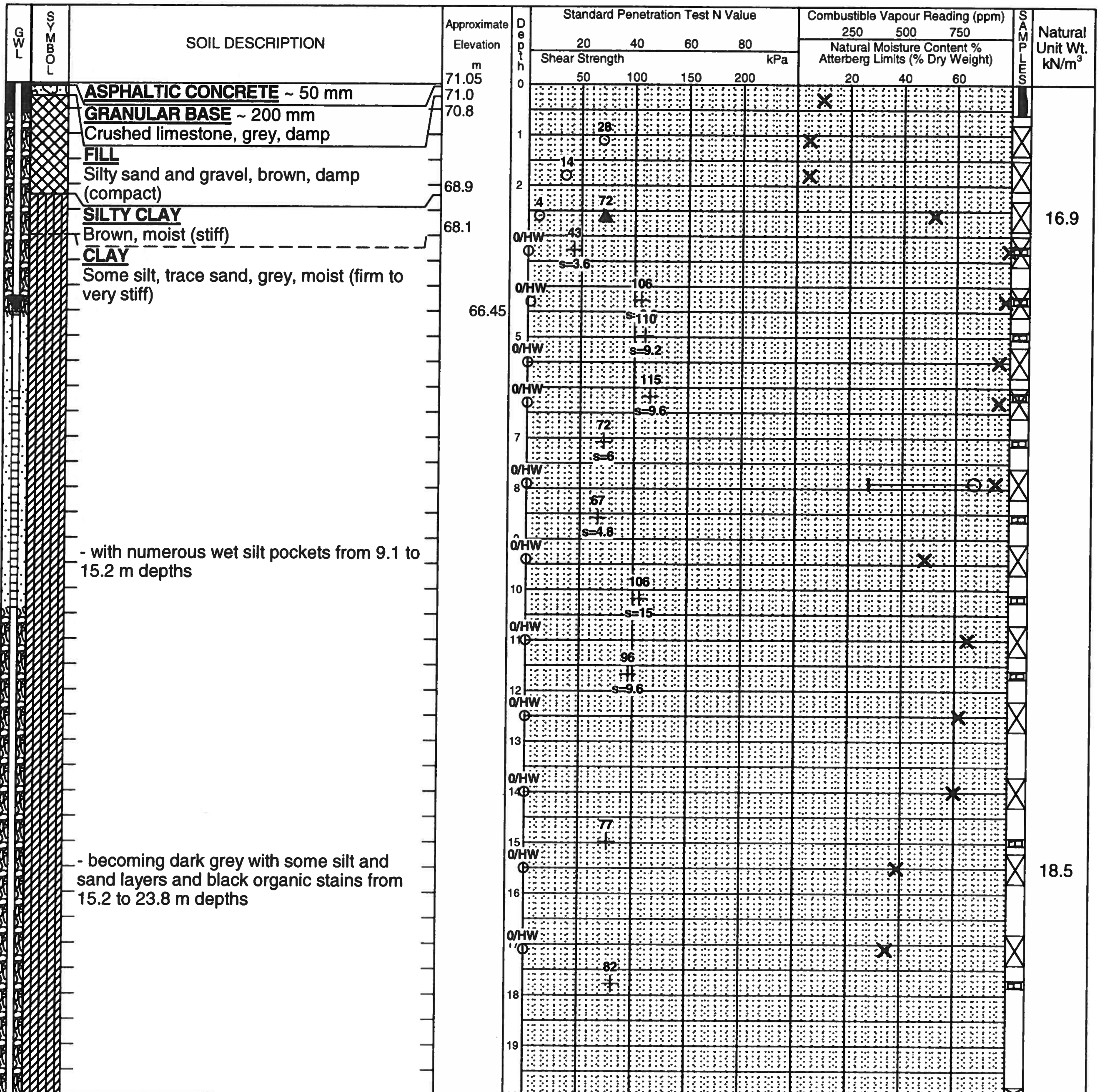
Drill Type: CME-55 Truck Mount

Datum: Approximate Elevation

Logged by: S. Bilan   Checked by: S. Potyondy

Figure No. 2

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



Approximate Elevation m	Depth (m)	Soil Description	Natural Unit Wt. kN/m³
71.05	0		
71.0		**ASPHALTIC CONCRETE** ~ 50 mm	
70.8		**GRANULAR BASE** ~ 200 mm Crushed limestone, grey, damp	
68.9		**FILL** Silty sand and gravel, brown, damp (compact)	
68.1		**SILTY CLAY** Brown, moist (stiff)	16.9
		CLAY Some silt, trace sand, grey, moist (firm to very stiff)	
66.45			
		- with numerous wet silt pockets from 9.1 to 15.2 m depths	
		- becoming dark grey with some silt and sand layers and black organic stains from 15.2 to 23.8 m depths	18.5

Continued Next Page

NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2. A 19 mm diameter standpipe installed to 10.4 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-00204847-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	22.9
Two Months	4.6	

CORE DRILLING RECORD

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

LOG OF BOREHOLE OTT-00204497-A0 BOREHOLE LOGS.GPJ TROW OTTAWA.GDT 8/3/12

Log of Borehole 1

Project No: OTT-00204847-A0

Project: Geotechnical Investigation - Proposed Iraq Embassy

Figure No. 2

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth
			51.05	20
		CLAY Some silt, trace sand, grey, moist (firm to very stiff) *(continued)* - becoming very stiff below 20.0 m depth		21 22 23
		Borehole Terminated at 23.8 m Depth	47.3	24
		Dynamic cone penetration test conducted from 23.8 to 35.7 m depths.		25 26 27 28 29 30 31 32 33 34 35
		Dynamic Cone Refusal at 35.7 m Depth	35.4	

Standard Penetration Test N Value: 20, 40, 60, 80

Shear Strength (kPa): 0/HW, 50, 100, 150, 200

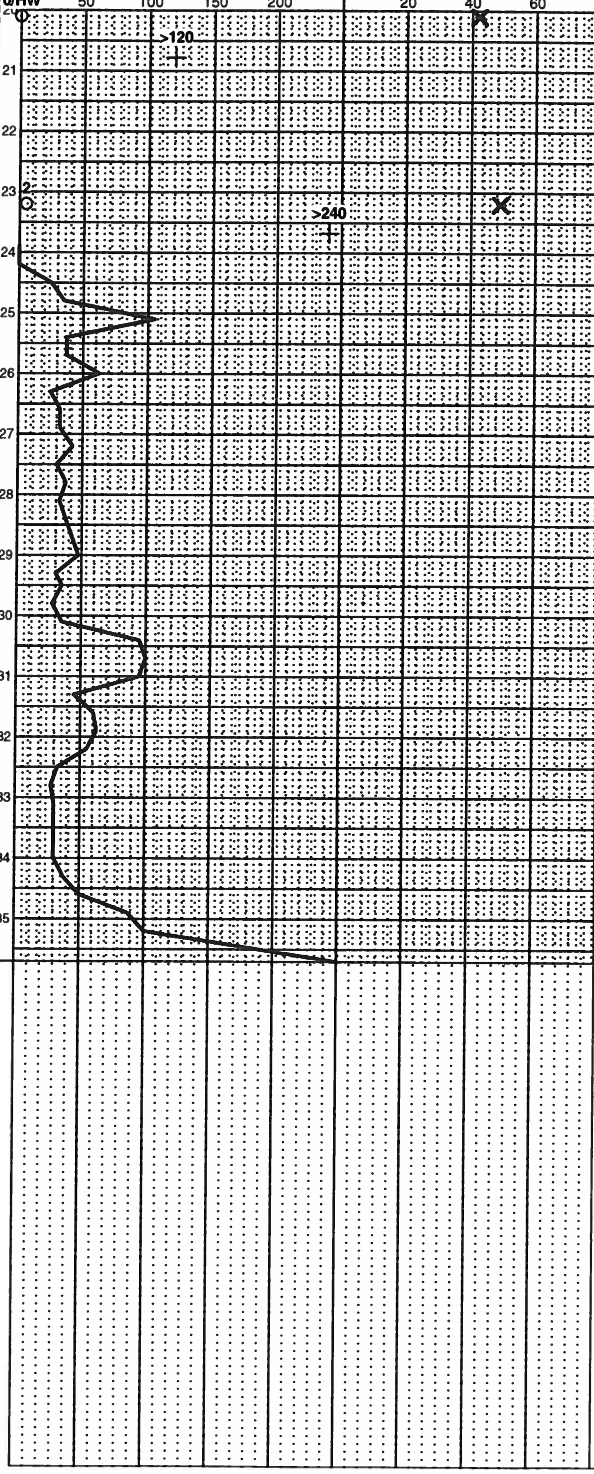
Combustible Vapour Reading (ppm): 250, 500, 750

Natural Moisture Content %

Atterberg Limits (% Dry Weight): 20, 40, 60

SAMPLES

Natural Unit Wt. kN/m³



NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2. A 19 mm diameter standpipe installed to 10.4 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-00204847-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	22.9
Two Months	4.6	

CORE DRILLING RECORD

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

Log of Borehole 2

Project No: OTT-00204847-A0

Project: Geotechnical Investigation - Proposed Iraq Embassy

Location: 215 McLeod Street, Ottawa, Ontario

Figure No. 3

Page. 1 of 1

Date Drilled: 'February 17, 2012

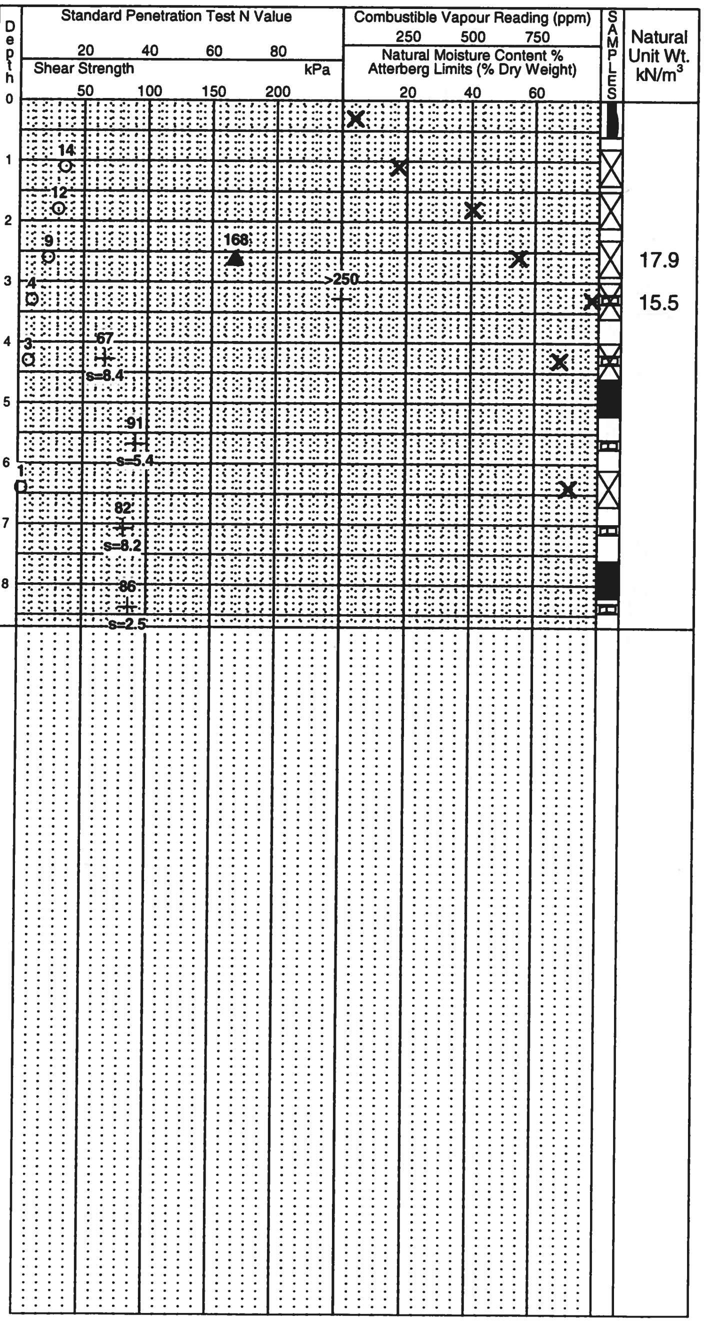
Drill Type: CME-55 Truck Mount

Datum: Approximate Elevation

Logged by: S. Bilan Checked by: S. Potyondy

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	X
Atterberg Limits	├──○
Undrained Triaxial at % Strain at Failure	⊕
Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth
			70.97	0
		ASPHALTIC CONCRETE ~ 50 mm	70.9	
		GRANULAR BASE ~ 300 mm Crushed limestone, grey, damp	70.6	
		FILL Mixture of silty sand and sand and gravel, some oxidized stains, brown, moist (compact)	69.3	1
		SILTY CLAY Brown, moist (very stiff)	68.0	3
		SILTY CLAY Grey, moist (stiff) - with some wet silt pockets from 4.0 to 8.7 m depths	67.07	
			62.3	
		Borehole Terminated at 8.7 m Depth		



NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2. A 19 mm diameter standpipe installed to 8.4 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-00204847-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	8.4
Two Months	3.9	

CORE DRILLING RECORD

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

Log of Borehole 3

Project No: OTT-00204847-A0

Project: Geotechnical Investigation - Proposed Iraq Embassy

Location: 215 McLeod Street, Ottawa, Ontario

Figure No. 4

Page. 1 of 1

Date Drilled: 'February 17, 2012

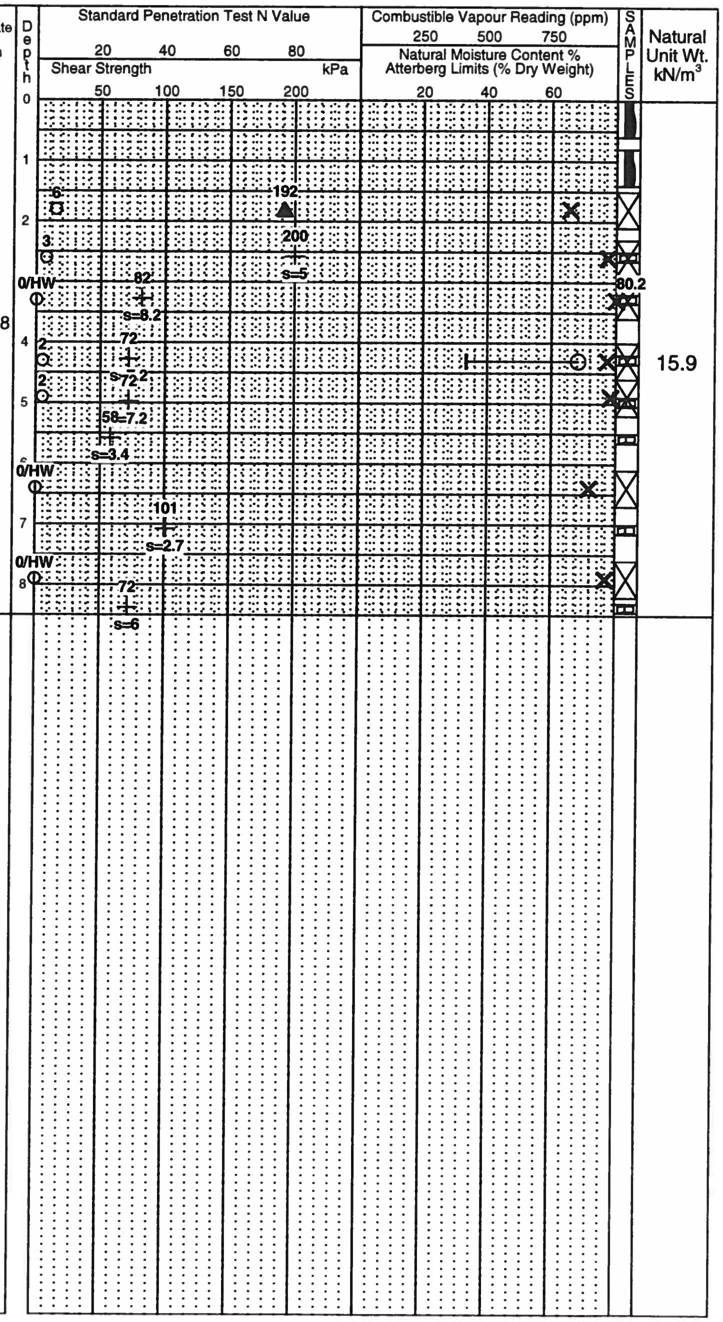
Drill Type: CME-55 Truck Mount

Datum: Approximate Elevation

Logged by: S. Bilan   Checked by: S. Potyondy

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	▲

SOIL DESCRIPTION	Approximate Elevation m
	70.78
ASPHALTIC CONCRETE ~ 50 mm	70.7
GRANULAR BASE ~ 300 mm Crushed limestone, grey, damp	70.4
FILL Mixture of clayey silt and silty sand, numerous concrete fragments, some brick and plaster fragments, dark brown, moist	69.3
SILTY CLAY Brown, moist (very stiff)	68.5
SILTY CLAY Trace sand, grey, moist (stiff to very stiff) - with some wet silt pockets from 3.0 to 8.5 m depth	66.98
	62.3
Borehole Terminated at 8.5 m Depth	



NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2. A 19 mm diameter standpipe installed to 8.4 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-00204847-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	8.4
Two Months	3.8	

CORE DRILLING RECORD

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

Log of Borehole 3A

Project No: OTT-00204847-A0

Project: Geotechnical Investigation - Proposed Iraq Embassy

Location: 215 McLeod Street, Ottawa, Ontario

Figure No. 5

Page. 1 of 2

Date Drilled: 'February 22, 2012

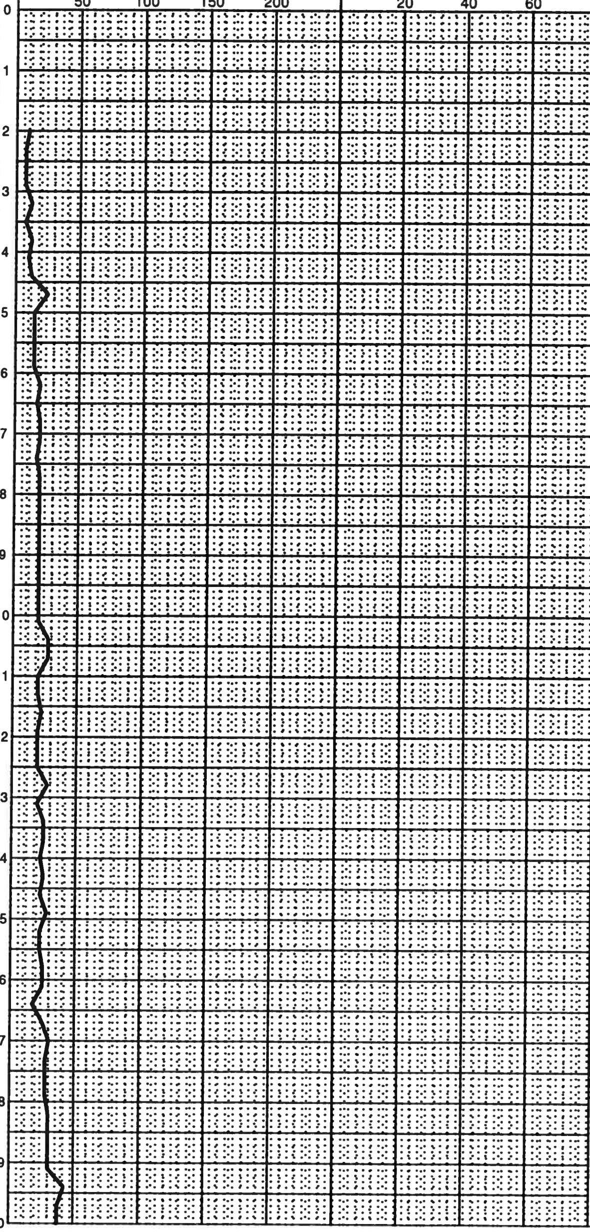
Drill Type: CME-55 Truck Mount

Datum: Approximate Elevation

Logged by: S. Bilan Checked by: S. Potyondy

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	SYMBOL	SOIL DESCRIPTION	Approximate Elevation 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/m^3
		Power augered from 0.0 to 2.0 m depths	70.78	0				
		Dynamic cone penetration test conducted from 2.0 to dynamic cone refusal depth at 34.1 m	68.8	2				



Continued Next Page

NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2.
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-00204847-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

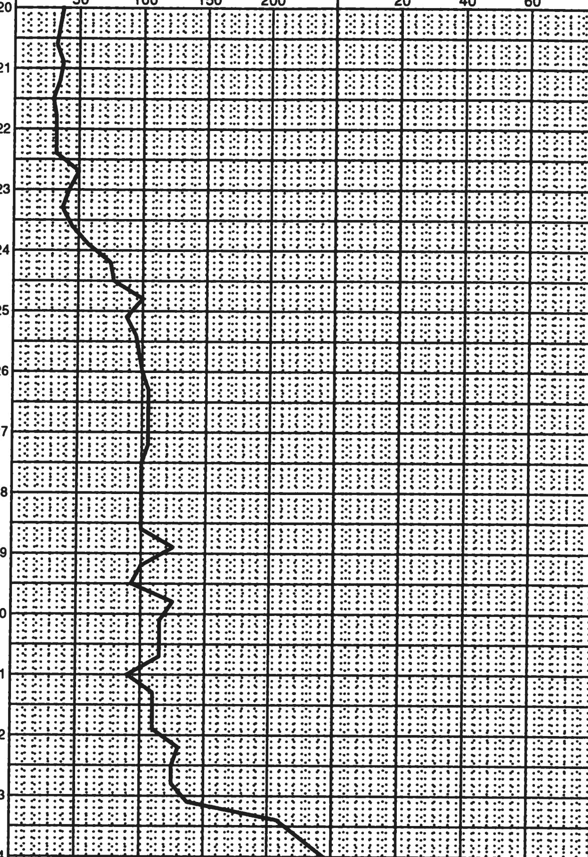
Log of Borehole 3A

Project No: OTT-00204847-A0

Project: Geotechnical Investigation - Proposed Iraq Embassy

Figure No. 5

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation 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
			50.78	20				
			36.7	34				
		Dynamic Cone Refusal at 34.1 m Depth						



NOTES:
1. Borehole/Test Pit data requires interpretation by exp. before use by others
2.
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-00204847-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

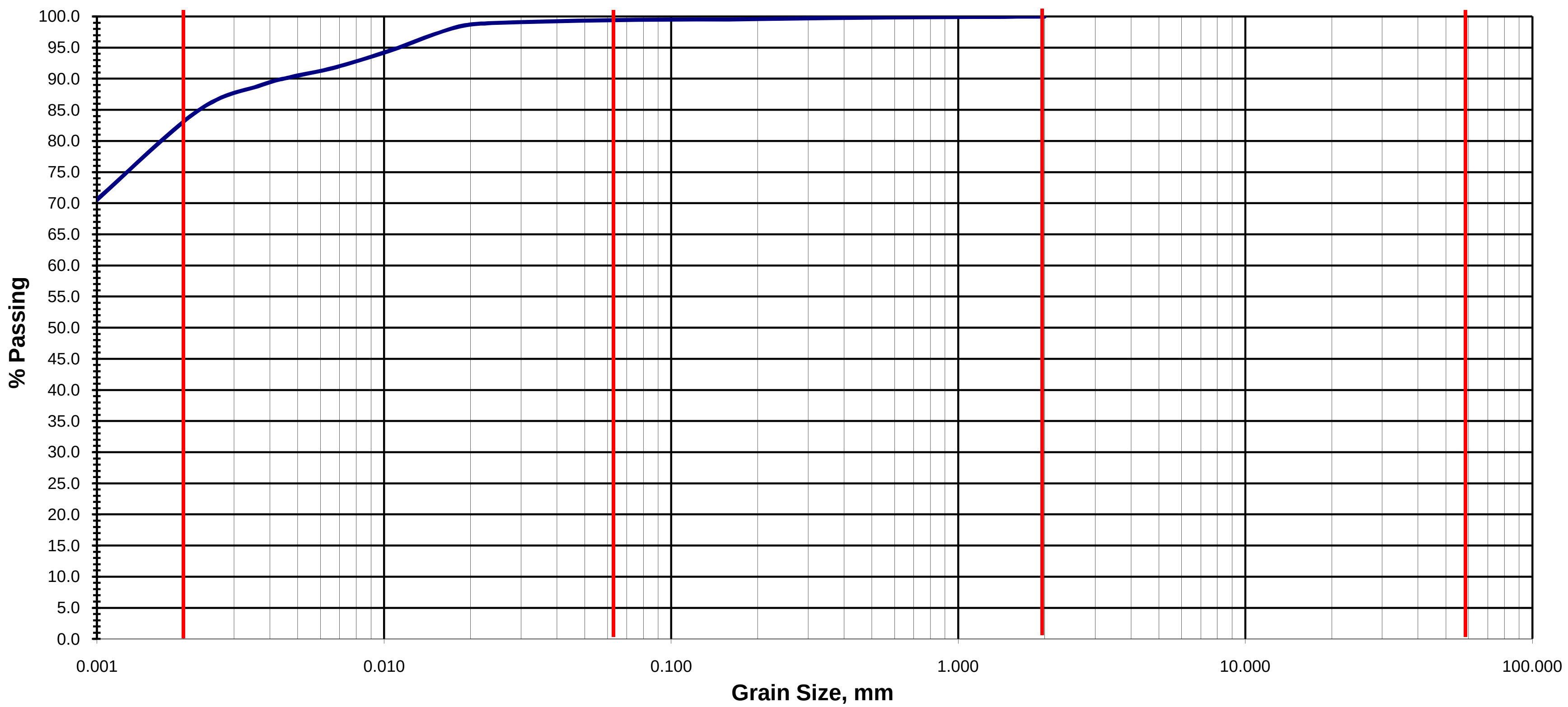
Run No.	Depth (m)	% Rec.	RQD %

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

Method of Test for Particle Size Analysis of Soil

MTO Test Method LS - 702, Rev. No. 19

Grain Size Distribution Curve



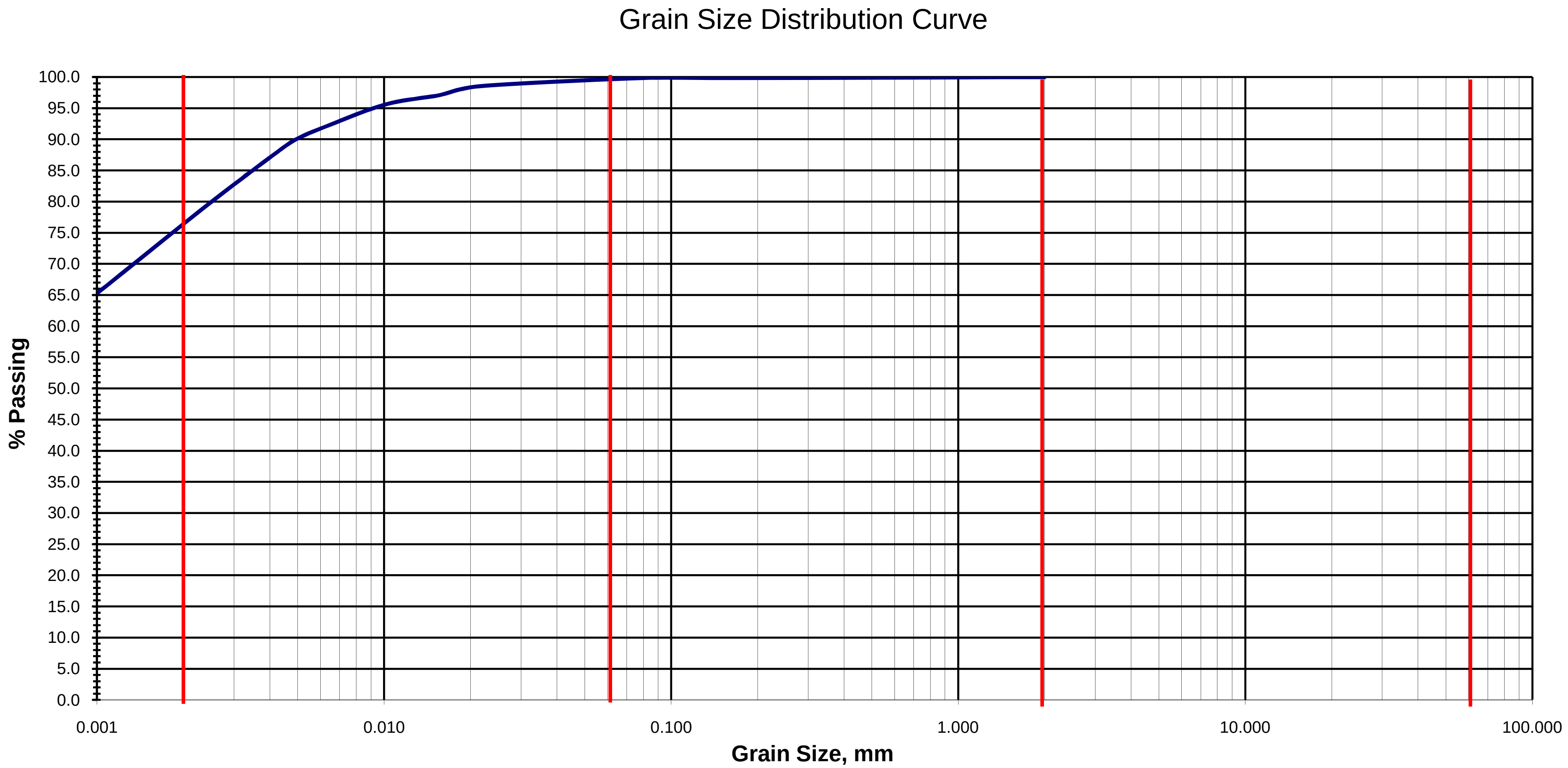
CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			
Modified M.I.T. Classification										

Trow Project No.:	**OTT-000204847-AO**	**Project Name :**	**Proposed Iraqi Embassy**		
Client :	**Julian Jacobs Architects Inc.**	**Project Location :**	**215 McLeod Street, Ottawa, Ontario**		
Date Sampled :	**February 21, 2012**	**Borehole No.:**	**BH 1**	**Sample No.: SS7**	**Depth (m) : 5.2 - 5.8**
Sample Description :	**Clay: Some Silt, Trace Sand**				**Figure : 6**

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

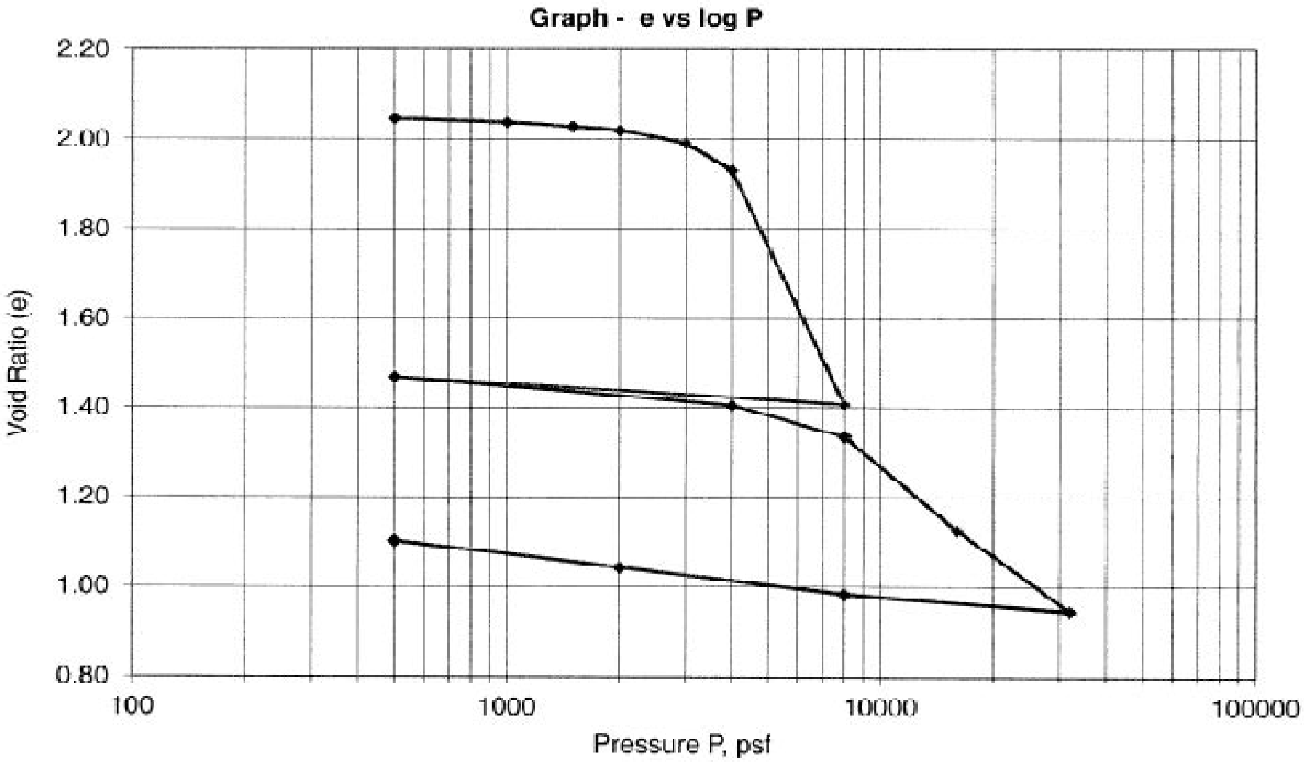
Method of Test for Particle Size Analysis of Soil

MTO Test Method LS - 702, Rev. No. 19



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

Trow Project No.: OTT-000204847-AO	**Project Name :** Proposed Iraqi Embassy				
Client : Julian Jacobs Architects Inc.	**Project Location :** 215 McLeod Street, Ottawa, Ontario				
Date Sampled : February 17, 2012	**Borehole No.:**	**BH 3**	**Sample No.:** SS9	**Depth (m) :**	**7.6 - 8.2**
Sample Description :	**Silty Clay: Trace Sand**			**Figure :**	**7**



One-Dimensional Consolidation Test Results

CONSOLIDATION TEST DATA SUMMARY					
Borehole	2	P'o	87.0 kPa	Cr	0.0610
Sample No.	TW 9	P'c	192.0 kPa	Cc	1.200
Sample Depth	7.6 -8.2 m	OC Ratio	2.0	Wo	73 %
Sample Elev.	63.4-62.8 m	Initial Void Ratio	2.053	Unit Wt.	17.0 kN/m^3

Client	Julian Jacobs Architects	Project	OTT-00204847-A0
Project	Proposed Iraqi Embassy	Date	May-12
	215 McLeod Street, Ottawa, Ontario		

List of Distribution

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