• DCR Phoenix Group of Companies

Preliminary Geotechnical Investigation

Type of Document
Final

Project Name
Proposed Residential Subdivision
1154-1208 Old Montreal Road
Ottawa (Formerly Township of Cumberland), Ontario

Project Number
OTT-00234493-A0

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Date Submitted
November 7, 2016

DCR Phoenix Group of Companies

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Attention: Mike Boucher, Manager of Planning

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Executive Summary

A preliminary geotechnical investigation was undertaken at the site of the proposed residential subdivision to be located on the south side of Old Montreal Road at the civic address of 1154-1208, in the City of Ottawa, Ontario. This work was authorized by Phoenix Homes Ltd. A Phase I and II Environmental Site Assessments were also completed by **exp** on this property and reported under separate covers.

The proposed subdivision will comprise of one to two-storey single-family residences with basements. Associated roadways and underground services are also to be constructed as part of the subdivision.

The fieldwork for the geotechnical investigation comprised the drilling of seven boreholes (Borehole Nos. 1 to 7) to depths ranging between 7.2 and 23.3 m. The boreholes revealed that beneath 25 mm to 200 mm of topsoil, silty sand or fill extends to 0.7 m to 2.3 m depth. The silty sand/fill are underlain by clay, which extends to the entire depth investigated of 7.3 m to 8.6 m in Borehole Nos. 2, 4, 5, 6 and 7 and to a depth of 18.9 m and 20.9 m in Borehole Nos. 1 and 3 respectively. The clay is stiff to hard and is over-consolidated by 78.0 kPa to 520 kPa based on the results of consolidation tests undertaken on select undisturbed clay samples. The clay in Borehole Nos. 1 and 3 is underlain by gravelly sand till which extends to the maximum depth investigated of 20.4 in Borehole No. 1 and to refusal to auger depth of 23.3 m in Borehole No. 13.

Water level observations made in the boreholes indicate that the groundwater table is at 1.3 m to 1.5 m depth. However, this is considered to be a perched water table. The groundwater table is expected to be at a depth of 3 m to 4 m based on the natural moisture content of the soil samples.

Based on the results of the investigation and consolidation tests undertaken on the clay sample, a maximum grade raise of 2.5 m is permitted at the site.

The investigation has revealed that the geotechnical conditions at the site are suitable for construction of the 1 to 2 storey residential dwellings on spread and strip footing foundations. It is recommended that the footings should be founded above the groundwater table and designed for Serviceability Limit State (SLS) bearing pressure of 100 kPa and factored geotechnical resistance at Ultimate Limit State (ULS) of 150 kPa.

The lowest level floor slabs of the structures may be constructed as slabs-on-grade. Perimeter as well as underfloor drains should be provided for the structures with basements.

Excavations at the site will be undertaken to a maximum depth of 3 m below the existing ground surface and will be above the groundwater table. The excavations may be undertaken as open cut provided they are cut back at 45 degrees. Seepage of surface and subsurface water into the excavations should be anticipated. However, it should be possible to collect this water in perimeter ditches and remove by pumping from sumps. The backfill against the subsurface walls should be free draining granular materials conforming to OPSS 1090 for Granular B, Type II. It should be compacted to 95 percent of standard Proctor Maximum Dry Density (SPMDD).

The pavement structures of the access roads and driveways are given on Table IV. General Use (GU) Portland cement may be used in the subsurface structures at the site.

The site has been classified as Class D for seismic site response. In addition, the on-site soils are not considered to be liquefiable during a seismic event.

Trees should not be planted in close proximity of the structures to prevent settlements due to shrinkage of the clay as a result of water extraction by tree roots.

A slope stability analysis of the slope to a ravine located along the south boundary of the site was undertaken. It revealed that the slope is stable and that a geotechnical setback is not required. The exception to this is Section A-A where the factor of safety was less than 1.5. A reiterative analysis of this section was undertaken and gave a geotechnical setback of 24 m for factor of safety of 1.5. A slope is also located along the north property boundary to Old Montreal Road. This slope is at an incline of 4.2H:1V or flatter. Based on the results of the stability analysis of the south slope, the north slope is also considered to be stable and its stability was not analysed. Therefore, the limit of hazardous land was computed as 11 m (5 m toe erosion allowance and 6 m access allowance) from the crest of the south slope except in the vicinity of Section A-A where it is 35 m from the crest of the south slope. This setback was determined as 6 m from the crest of the north slope. No development should be undertaken beyond the limit of hazardous land shown on Figure 2.

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

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1 Introduction

A preliminary geotechnical investigation was undertaken at the site of the proposed residential subdivision to be located at 1154-1208 Old Montreal Road in the City of Ottawa, Ontario. The site location is shown on Figure 1. This work was authorized by Mike Boucher of Phoenix Homes Group of Companies.

The proposed development would consist of a residential subdivision with associated roadways and utilities

The investigation was undertaken to:

a) Establish geotechnical and groundwater profile at the site at the locations of the boreholes;

b) Establish the maximum grade raise permissible at the site;

c) Make recommendations regarding the most suitable type of foundations, founding depth and Serviceability Limit State (SLS) and Ultimate Limit State bearing capacities of the founding soil;

d) Determine anticipated settlements;

e) Classify the site for seismic site response in accordance with the requirements of National Building Code (NBC), 2012.

f) Comment on excavation conditions and effect of groundwater on the excavations;

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

h) Recommend pavement structure thickness for access roads and parking areas;

i) Comment on subsurface concrete requirements; and

j) Assess the stability of the slopes of the valley located on the south side of the site and establish the limit of hazardous lands for the proposed subdivision;

The comments and recommendations given in this report are preliminary in nature and 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 Procedure

The fieldwork for the preliminary geotechnical investigation was undertaken between August 15 and 18, 2016 and comprised the drilling of seven boreholes (Borehole Nos. 1 to 7) to depths ranging between 7.2 m and 23.3 m. A dynamic cone penetration test was performed in Borehole No. 5 below 8.5. m depth to refusal at 20.9 m depth. The locations of the boreholes are shown on Site Plan, Figure 2.

The fieldwork was undertaken with a track-mounted drill rig equipped with continuous flight hollow stem augers and was supervised on a full-time basis by a representative of **exp**.

Standard penetration tests were performed in all the boreholes at 0.75 m to 1.5 m depth intervals and soil samples retrieved by split barrel sampler. Relatively undisturbed thin wall tube samples of the silty clay were obtained from Borehole Nos. 2 and 3. The undrained shear strength of the clay was established by in-situ field-vane shear tests.

Water levels were measured in the open boreholes on completion of drilling. In addition, long-term groundwater monitoring installations consisting of 19 mm diameter PVC (polyvinyl chloride) pipes were placed in Borehole Nos. 1, 3 and 7. The installation configuration is documented on the respective borehole logs. All the boreholes were backfilled upon completion of the fieldwork. The initial locations of the boreholes were established by a representative of **exp** using GPS technology. The final elevations and locations of the boreholes were determined by a survey crew from **exp**.

All the soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified. The thin wall tube samples were also visually examined, logged, the thin wall tubes capped, taped and identified. On completion of the fieldwork, all the soil samples were transported to the **exp** laboratory in the City of Ottawa, Ontario.

All the soil samples were visually examined in the laboratory by a geotechnical engineer and borehole logs prepared. The engineer also assigned the laboratory testing which consisted of performing natural moisture content, unit weight, grain-size analysis, one dimensional oedometer, Atterberg Limit, pH and sulphate content tests on selected soil samples.

3 Site Description

The subject site is located on the south side of Old Montreal Road, at 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, as shown on Figure 1. At the time of the investigation, the site was used for residential and agricultural purposes. The surrounding properties are mostly residential and agricultural. The site is rectangular and covers a total area of 14.6 hectares (36 acres).

The topography of the site consists of a topographic high at the house and barn locations of the site, with a steep slope downwards to the north to Old Montreal Road. The local groundwater flow direction is anticipated to be north towards the Ottawa River, at a distance of 1.2 km slope is located on the southeast side of the site to a deep ravine.

A slope is located on the southeast side of a site to a deep ravine. The crest of the slope is at an Elevation 82.0 m to Elevation 85.0 m whereas the toe of the slope is at Elevation 61.25 m to Elevation 72.25 m, resulting in a 20.75 m to 12.25 m high slope. The slope inclination varies from 2.63H:1V to 3.37H:1V. The slope is covered with vegetation.

Another slope is located along the northwest part of the site which extends to Old Montreal Road. The crest of this slope is at Elevation 85.0 m whereas its toe is at Elevation 70.0 m to 71.0 m, resulting in a 14.0 m to 15.0 m high slope. The inclination of this slope varies from 7.8H:1V to 1.9H:1V. This slope is also covered with vegetation.

The elevation of the relatively level part of the site varies from Elevation 82 m approximately to Elevation 86.4 m approximately in the east west direction, resulting in a relief of 4.4 m approximately in the westerly direction. The ground surface at the site varies in the north south direction from Elevation 82 m to Elevation 85 m in the south to Elevation 82.1 to Elevation 84.7 m in the north, indicating that it is relatively flat lying in the north-south direction. The site is currently occupied by a number of residences, which will be demolished for the proposed development.

4 Soil Description

A detailed description of the subsurface soil and groundwater conditions determined from the boreholes are given on the attached borehole logs, Figure Nos. 3 to 9. 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. Environmental assessment of the on-site soils and groundwater was completed as part of **exp**'s terms of reference and the results were reported under a separate cover.

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 form an integral part of this report and should be read in conjunction with this report.

A review of the borehole logs indicates the following soil stratigraphy in descending order. The soil properties have been summarized on Table I.

4.1 Topsoil

A 25 mm to 200 thick topsoil layer was contacted at the location of all the boreholes except Borehole No. 7.

4.2 Sand Fill

The surficial soil in the vicinity of Borehole No. 7 is sand fill which also underlies the topsoil in Borehole Nos. 3 and 6. It extends to 0.6 m to 2.3 m depth (Elev. 82.0 to 84.2 m). The natural moisture content of the fil varies from 21 percent to 30 percent.

4.3 Silty Clay Crust

The topsoil in Borehole No. 5, the fill in Borehole Nos. 3, 6 and 7, and the silty sand in Borehole Nos. 1, 2 and 4 are underlain by desiccated silty clay crust, which extends to 3 m to 5 m depth (Elev. 77.5 m to 83.6 m). The natural moisture content and unit weight of the crust vary from 30 to 48 percent and 17.8 to 19.1 kN/m^3 respectively.

The crust is very stiff to hard as indicated by its undrained shear strength, which varies from 180 kPa to greater than 250 kPa.

A grain size analysis performed on a sample of the crust yielded a composition of 72 percent clay, 26 percent silt and 2 percent sand (Figure 10).

The liquid and plastic limits of the clay were established as 64.5 percent and 26.1 percent respectively, indicating that the crust is inorganic clay of high plasticity.

4.4 Grey Silty Clay

The silty clay crust is underlain by grey silty clay which extends to the entire depth investigated in Borehole Nos. 2 and 4 to 7, i.e. 7.2 m to 8.5 m depth (Elevation 72.4 to 78.1 m) and to a depth of 18.9 m and 20.9 m in Borehole Nos. 1 and 3 respectively (Elevation 66.9 m and 63.4 m).

The natural moisture content and unit weight of the silty clay varies from 54 to 74 percent and 15.5 to 17.3 kN/m^3 respectively. The silty clay is stiff to hard as indicated by its undrained shear strength, which varies from 50 kPa to 220 kPa.

Two grain size analyses performed on the silty clay yielded a soil composition of 61 to 69 percent clay, 30 to 37 percent silt and 1 to 2 percent sand (Figure Nos. 11 and 12). The liquid and plastic limits of the silty clay vary from 52.7 to 62.7 percent and 24.6 to 28.3 percent respectively. On the basis of these test results, the grey silty clay may be described as highly plastic inorganic clay.

Results of two consolidation tests performed on the silty clay are shown on Figure Nos. 14 and 15 and have been summarized as Table I. A review of Figure 14 indicates that the desiccated silty clay crust is over-consolidated by 519.6 kPa approximately. Its recompression (c_{cr}) and compression index (c_c) are 0.153 and 1.07 respectively. The grey silty clay is over-consolidated by 78 kPa approximately. Its recompression and compression index are 0.11 and 1.36 respectively (Figure 14).

Table I: Results of Consolidation Tests

Borehole No.	Sample Depth	Effective Overburden Pressure p_o' (kPa)	Effective Consolidation Pressure p_c'(kPa)	Compression Index (C_c)	Re-Compression Index (C_r)	Over-consolidation Pressure (kPa)
2	4.0-4.6	75.4	595.0	1.07	0.153	519.6
3	7.6-8.2	114.0	192.0	1.36	0.110	78.0

4.5 Silty Sand Till

The grey silty clay in Borehole Nos. 1 and 3 is underlain by silty sand till which extends to the termination depth of 20.4 m (Elevation 65.4 m) in Borehole No. 1 and to the maximum auger depth of 23.3 m (Elevation 61.0 m) in Borehole No. 3. A dynamic cone penetration test performed in Borehole No. 5 below 8.5 m depth met refusal at 20.9 m depth (Elevation 60.1 m). The refusal in Borehole Nos. 3 and 5 were likely met on bedrock but not confirmed by core drilling techniques. The silty sand till is compact as indicated by its standard penetration resistance values (N values) which vary from 11 to 20. The natural moisture content of the till is 8 to 10 percent. A grain size analysis performed on a sample of the till from Borehole 1 yielded a soil composition of 3 percent clay, 12 percent silt, 46 percent sand and 39 percent gravel (Figure 13).

4.6 Bedrock

As indicated above, refusal to dynamic cone penetration test or to augering was met in two of the boreholes at a depth of 20.9 m and 23.3 m. This refusal is likely to have met on bedrock. Available information indicates that the bedrock in the area is likely to be shale of the Rockcliffe Formation.

4.7 Groundwater

Water level observations were made in open boreholes during drilling in the stand pipes installed in Boreholes 1, 3 and 7 subsequent to completion of the drilling. The observations made have been tabulated on Table II.

Table II: Groundwater Observations in Boreholes

Borehole No.	Date Drilled	Observation Date	Groundwater Depth (m)	Elevation to Groundwater Table (m)
1	August 15, 2016	September 10, 2016	1.5	84.3
3	August 17, 2016	September 10, 2016	2.5	81.8
7	August 16, 2016	September 10, 2016	1.3	83.8

A review of Table II indicates that the perched water table in Boreholes 1, 3 and 7 is at a depth of 1.3 m to 2.5 m below the existing ground surface, i.e. Elev. 84.3 m to 81.8 m. The natural groundwater table had not stabilized during the time interval near which observations were made. Based on a review of the natural moisture content of the soil samples, the groundwater table is estimated to be at a depth of 3 m to 4 m below the existing ground surface, i.e. Elev. 83.6 m to 77.5 m.

Water levels were determined in the boreholes at the times and under the conditions stated in the scope of services. Note that fluctuations in the level of groundwater may occur due to a 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.

A summary of the subsurface conditions established is presented in Table III:

Table III: Summary of Subsurface Conditions

Soil Type	MC (%)	γ (kN/m^3)	SPT N Values	Cu (kPa)	Atterberg Limit Test Results			Hydrometer Test Results (%)			
					MC (%)	PL (%)	LL (%)	Clay	Silt	Sand	Gravel.
Fill	21-30		5 - 14	-	-	-	-	-	-	-	-
Silty Sand	21		6 - 7	-	21-30	-	-	-	-	-	-
Clay Crust	30-48	17.8 - 19.1	5 - 23	180 - >250	38	26.1	64.5	72	26	2	-
Clay	54-74	15.5 - 17.3	HW - 4	50 - 220	54-70	24.6-28.3	52.7-62.7	61-69	30-37	1-2	-
Silty Sand Till	8-10	-	11 - 20	-	-	-	-	3	12	46	39

Mc = Natural Moisture Content, Cu= Undrained Shear Strength, HW = Hammer Weight, PL= Plastic Limit, LL= Liquid Limit

5 Site Re-grading

The investigation has reveled the site to be underlain by a deep deposit of clay (in the order of 19 m to 21 m). This clay deposit is prone to consolidation settlements if fill is placed on the site beyond the permissible amount which will result in settlements and cracking of any structures founded in the clay due to overstressing of the clay.

In order to evaluate if the grade at the site can be raised or the maximum allowable grade raise, two one-dimensional odometer tests were undertaken on the clay samples from Borehole Nos. 2 and 3 and the results summarized in Table I. A review of this table indicates that the clay is over-consolidated by 78 kPa to 520 kPa. It is therefore considered that the additional load that can be applied on the clay underlying the desiccated clay crust is 62 kPa below Elev. 76.4 m for the settlements to be within normally tolerated limits of 25 mm total and 19 mm differential (assumed 80 percent of over-consolidation pressure).

The groundwater table at the site is located 3 m to 4 m below the existing ground surface. As such, lowering of the groundwater table at the site will not result due to proposed development. Therefore, an allowance for groundwater lowering is not required. Allowing for the Serviceability Limit State (SLS) bearing pressure recommended in Section 6, it is considered that the grades at the site may be raised by up to 2.5 m.

The site-grading plan must be reviewed by this office when available to ensure that these requirements have been complied with.

6 Foundation Considerations

The investigation has revealed that the geotechnical conditions at the site are suitable for construction of the proposed one to two-storey structures with one level of basement on spread and strip footing foundations. As required by the City of Ottawa, it is recommended that the footings of the proposed structures should be set above the groundwater table, i.e. at a maximum depth of 2.5 m below the existing ground surface. Footings founded on the clay below any fill or silty sand at a maximum depth of 2.5 m below the existing ground surface 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.

The recommended bearing capacities have been calculated by **exp** from the borehole information for the preliminary 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.

A minimum of 1.5 m of earth cover should be provided to all the exterior footings of heated structures to protect them from damage due to frost penetration. Where earth cover is less than 1.5 m, an equivalent combination of earth fill and rigid polystyrene insulation (i.e. Styrofoam HI-40) should be provided. Footings of unheated structure should be provided with a cover of 2.1 m if snow would not be cleared from their vicinity. If the snow would be cleared from the vicinity of the footings, they should be provided with 2.4 m of earth cover.

All the footing beds should be examined by a geotechnical engineer/geotechnician to ensure that the founding soil is capable of supporting the design bearing pressure and that the footings beds have been prepared satisfactorily.

Settlements of the residences founded on strip and spread footings design according to the above recommendations and properly constructed are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements.

7 Floor Slab and Drainage Requirements

The lowest level floors of the proposed structures may be constructed as slabs-on-grade provided they are set on beds of well-compacted 19 mm clear stone at least 200 mm thick placed on the natural soil or on well-compacted fill. The clear stone would prevent the capillary rise of moisture from the sub-soil to the floor slab. Adequate saw cuts should be provided in the floor slabs to control cracking. Any underfloor fill required should conform to OPSS 1010 for Granular B, Type II and should be placed in 300 mm lift thickness and each lift compacted to at least 98 percent of the standard Proctor maximum dry density (SPMDD).

Perimeter as well as underfloor drains should be provided for structures with basements (Figure 16). The drainage system should be outletted to roadside ditches. All subsurface walls should be properly damp-proofed. The exterior grade should be sloped away from the structures at an inclination of 1 to 2 percent to prevent the ingress of surface runoff.

8 Lateral Earth Pressure Against Subsurface Walls

The subsurface walls should be backfilled with free draining material, such as OPSS 1010 for Granular B, Type II and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces.

For design purposes, the lateral static earth thrust against the subsurface walls may be computed from the following equation:

$$P = K_0 H (q + ½ \gamma H)$$

where P = lateral earth thrust acting on the subsurface wall; kN/m

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 = 22 kN/m^3

H = Height of backfill adjacent to foundation wall, m

q = surcharge load, kPa

The lateral seismic thrust may be computed from the equation given below:

$$\Delta P_E = 0.32 \gamma H^2$$

where ΔP_E = resultant thrust 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 ΔPE value does not take into account the surcharge load. The resultant load should be assumed to act at 0.6 H from the bottom of the wall.

9 Excavations

Excavations for construction of spread and strip footings are expected to extend to a maximum depth of 2.5 m below the existing grades. These excavations are expected to terminate in the clay. They are expected to be above the groundwater table.

Excavations above the groundwater table in the silty clay are expected to be stable when cut back at 45 degrees. Excavations in the clay below the groundwater table would not experience a 'base-heave' type of failure of the excavation and to slough and eventually stabilize at a slope of 2H:1V to 3H: 1V.

Seepage of surface and subsurface water into the excavations is anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps. 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 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 engineer construction dewatering systems adequately.

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.

The clay at the site is susceptible to disturbance due to the movement of construction equipment, and personnel on 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 e.g. a gradall, or mechanical shovel. It is anticipated that temporary granular roads may be required to gain access to the site.

Whether a Permit to Take Water will be required or not be will depend on the depth of the excavation. This office should be contacted once the site grades and invert of the underground services are known so comments can be provided regarding whether a Permit to take Water will be required for this site.

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

The backfill in footing and service trenches inside the buildings and against subsurface walls should consist of free draining material preferably conforming to OPSS 1010 for Granular B, Type II. It should be compacted to 95 percent of the SPMDD.

The backfill in service trenches outside of the building areas should be compactible i.e. free of organics and debris and with natural moisture content, which is within 2 percent of the optimum moisture content. It should also be compacted to 95 percent of the SPMDD.

The material to be excavated during construction of the footings and installation of services is fill, silty sand and silty clay. The upper desiccated silty clay is expected to be compactible and may be used to backfill service trenches outside of the buildings and for regrading of driveways, etc. The silty sand and existing on-site fill may be used for general site grading. Any fill that has to be imported to backfill footing trenches, service trenches, and against subsurface walls should conform to OPSS 1010 for Granular B, Type II. It should be placed in 300 mm lift thickness and compacted to 95 percent of the SPMDD.

11 Access Roads

The subgrade at the site will be silty clay. Pavement structure thicknesses required for the access roads and parking areas set on sandy lean clay subgrade were computed and are shown on Table 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 18 to 20 years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table IV: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Driveways	Access Roads	
			Sand Subgrade	Clay Subgrade
Asphaltic Concrete (PG 58-34)	92 to 97 % MRD	65 mm HL3	40 mm – SP12.5 50 mm – SP19	40 mm – SP12.5 50 mm – SP19
Granular A Base (crushed limestone)	100% SPMDD*	150 mm	150 mm	150 mm
Granular B Sub-base, Type II	100% SPMDD*	300 mm	450 mm	600 mm

SPMDD* Standard Proctor Maximum Dry Density, ASTM-D698
MRD denotes Maximum Relative Density, ASTM D2041
Asphaltic Concrete in accordance with OPSS 1150 and 1151

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 parking area are as follows:

1. As part of the subgrade preparation for the areas to be paved, the subdivision roadways should be stripped of topsoil and other obviously unsuitable material. Fill required to raise the grades to design elevations should conform to OPSS 1010 Select Subgrade Material (SSM) and should be placed in 300 mm lifts and each lift compacted to 95 percent of the SPMDD. The subgrade should be properly shaped, crowned, then proofrolled with a heavy vibratory roller in the full-time presence of a representative of this office. Any soft or spongy subgrade areas detected should be sub excavated and properly replaced with suitable OPSS 1010 Granular B Type II compacted to 95% SPMDD (ASTM D698) as indicated previously and in order to prevent overstressing the clay subgrade.

2. The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. The need for adequate drainage

cannot be over-emphasized. Subdrains should be installed on both sides of the access road(s). Subdrains must be installed in the proposed parking area at low points and should be continuous between catchbasins to intercept excess surface and subsurface moisture and to prevent subgrade softening. This will ensure no water collects in the granular course, which could result in pavement failure during the spring thaw. The location and extent of subdrainage required within the paved areas should be reviewed by this office in conjunction with the proposed site grading.

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

4. The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, temporary construction roadways, etc., may be required, especially if construction is carried out during unfavorable weather.

5. 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 catchbasins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.

6. Relatively weaker subgrade may develop over service trenches at subgrade level. These areas may require the use of thicker/coarser sub-base material and the use of a geotextile at the subgrade level. if this is the case, it is recommended that additional 150 mm of granular sub-base Granular B should be provided in these areas in addition to the use of a geotextile at the subgrade level. On-site excavated wet soils should not be used as backfill of the service trenches.

7. The granular materials used for pavement construction should conform to OPSS 1010 for Granular A and Granular B, Type II and should be compacted to 100 percent of the SPMDD (ASTM D698). The asphaltic concrete used and its placement should meet OPSS 1151 and 310/313 requirements. It should be compacted to 92 to 97 percent of the maximum relative density in accordance with ASTM D2041.

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.

12 Subsurface Concrete Requirements

Subsurface concrete will be used to construct basements of the residences. Chemical tests limited to pH and sulphate tests were performed on three selected soil samples. The results are given on Table V.

Table V: Chemical Test Results

Borehole No.	Depth	PH	Sulphate (%)
2	1.5 – 2.1	6.83	0.003
4	1.4 – 1.6	6.40	0.0009
7	3.0 – 3.6	6.81	0.0005

The test results indicate the clay contains a sulphate content of less than 0.1 percent. This concentration of sulphates in the soil would have a negligible potential of sulphate attack on subsurface concrete. It is therefore considered that General Use Portland cement may be used in the basement walls of the residences. The concrete for the site should be designed in accordance with the requirements of CSA A23.1-14.

13 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 of the Ontario Building Code (OBC) 2012. The subsoils at the site comprise of stiff to hard silty clay deposit, to 18.9 m to 20.9 m depth overlying compact gravelly sand till to 20.4 m to 23.3 m and limestone bedrock. The undrained shear strength of the silty clay varies between 50 kPa and greater than 250 kPa.

The average shear-wave velocity value of the overburden and bedrock to 30 m at the site was estimated. For this purpose, the shear-wave velocity value of bedrock was assumed as 760 m/s. The shear-wave velocity (Vs) values of the silty clay deposit layer are correlated to the undrained shear strength (S_u) values using Dickenson (1994) formula:

$$Vs(m/s) = 23.Su^{0.475}$$

The shear-wave velocity (V_s) of the compact gravelly sand till can be correlated to the standard penetration values (SPT) using Imai & Tonouchi[1] (1982) formula:

$$Vs(m/s) = 91.7\,N^{0.26}$$

An average shear-wave velocity value to 30 m depth was estimated as 213 m/s. On this basis, the site has been classified as Class D for seismic site response in accordance with Table 4.1.8.4A of the Ontario Building Code, 2012.

The liquefaction potential of the clay on the site was assessed by plotting the results of Atterberg Limit Tests on Bray et al plot. A review of this figure (Figure 17) indicates that the clay is not susceptible to liquefaction during a seismic event.

---

1 Imai, T, and K Tonouchi (1982). Correlation of N value with S-wave velocity and shear modulus, Proc., 2nd European Symp. on Penetration Testing, Amsterdam, pp. 67–72.

14 Tree Planting

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

City of Ottawa guidelines indicate that fast-growing, high-water demand trees must not be planted closer to a building than a distance equal to their height at maturity. Only one of the small-sized trees listed below can be placed a minimum distance of 7.5 m away from any buildings, including when planting along road allowances (see Table VI). In addition, newly planted trees must be a minimum of 2.5 m from the curb and have a small-sized canopy at maturity to allow sufficient space for snow and ice control.

Table VI: List of Trees Suitable for Planting On Site (City of Ottawa Guidelines)

Species	**Water Demand**
Amur Maple (Acer ginnala)	Moderate
Serviceberry (Amelanchier canadensis)	Low
Crabapple (Malus spp.)	Moderate
Japanese Lilac (Syringa reticulate)	Moderate
Green Colorado Spruce or any conifer species (Picea pungens)	Low

For further information, an arborist should be consulted.

15 Slope Stability

15.1 Slope Stability Analysis

The stability of the existing slopes was analyzed by using Morgenstern-Price Method, GeoStudio /Geo-slope office, Version 8.13 computerized system. The purpose of the analysis was to assess the stability of the existing slopes and to determine the required set back of the proposed structures from the crest of the slopes. A total of four cross-sections were analyzed. These cross-sections have been shown as Sections A-A, B-B, C-C and D-D on Figure 2.

These cross-sections were obtained from the 2015 Lidar Survey available for the site.

The natural slope inclinations at the cross-sections analyzed were determined, and the results have been presented on Table VII.

Table VII: Natural Slope Inclination of Cross-sections Analyzed

Section	Crest of Slope (m)	Toe of Slope (m)	Height of Slope (m)	Overall Slope Inclination
A-A	81.75	62.0	19.75	3.83H:1V
B-B	84.0	68.0	16.0	3.12H:1V
C-C	85.0	72.25	12.75	3.69H:1V

The slopes were analyzed for the following conditions:

1.) Effective stress analysis.
2.) Total stress analysis; and
3.) Total stress analysis with seismic loading;

The following assumptions were made:

1.) The crest of the existing slopes varies from Elevation 82.0 m to 85.0 m whereas the toe of the slopes is at Elevation 62.0 m to 72.25 m (Table VII).
2.) The soil stratigraphy for the various cross-sections is shown on Figure Nos. 18 to 26 inclusive. The soil stratigraphy was established from the boreholes drilled at the site.
3.) The unit weight of the various soils was established from laboratory tests. The undrained strength of the clay was established by performing in-situ field vane tests. The effective shear strength parameters were selected based on literature search. Previous work undertaken by various

researchers was reviewed. The review indicated that the effective cohesion (c') and effective angle of internal friction (φ') values for the silty clay crust and grey silty clay are as follows:

Weathered Silty Clay Crust	Effective cohesion = 0 – 12 kPa Effective angle of internal friction = 25° – 38°
Grey Silty Clay	Effective cohesion = 0 – 12 kPa Effective angle of internal friction = 25° – 38°

Based on the review of the literature and site conditions, and using somewhat conservative approach an effective cohesion of 9.8 kPa and effective angle of internal friction of 36 degrees was used in the analysis for the desiccated crust and the underlying grey clay.

The undrained shear strength used in the analysis was computed from the field-vane test results. Undrained shear strength of 100 kPa and 60 kPa respectively for the desiccated crust and grey clay was used in the analysis.

4.) The slopes were assumed to be fully submerged i.e. the groundwater table in the slope coincides with the existing ground surface.

5.) Building loads were not taken into consideration in the analyses since the structures would be located away from the slopes.

The results of the analyses are given on Figures 18 to 26 inclusive and have also been tabulated on Table VIII.

Table VIII: Results of Slope Stability Analysis

Section	Condition Analysed	Factor of Safety	Required Geotechnical Set Back	Figure No.
A-A	Effective stress analysis and set back determination	1.50	24 m	18
	Total stress analysis	1.94	-	19
	Total stress analysis with seismic loading	1.20	-	20
B-B	Effective stress analysis and set back determination	1.75	0	21
	Total stress analysis	2.55	-	22
	Total stress analysis with seismic loading	1.63	-	23
C-C	Effective stress analysis and set back determination	1.59	0	24
	Total stress analysis	2.21	-	25
	Total stress analysis with seismic loading	1.49	-	26

Current practice of the City of Ottawa requires a minimum acceptable factor of safety of 1.5 for static loading conditions. The minimum acceptable factor of safety for seismic loading conditions is 1.1 (Mitchell 1983). The computed factors of safety of all the cross-sections analyzed for effective stress analysis were 1.5 or greater except for Section A-A. A reiterative slope stability analysis was undertaken for this section to determine the set back required for a factor of safety of 1.5 m. A geotechnical set back of 24 m was computed for this section.

It is noted that a slope is also located along the north property boundary. The inclination of this slope to Montreal Road was determined to range between 4.2H:1V and 6.2H:1V. Based on the results of the slope stability analysis undertaken for the south slope (which indicates that a 3.1H:1V slope has a factor of safety of 1.5), it is considered that the north slope is stable. Therefore, this slope was not analyzed.

The slopes located at the north and south property boundaries are covered with vegetation and trees. The vegetation and trees provide stability to the slopes and should not be disturbed in anyway.

During construction, the following precautions should be taken so that the stability of the slopes is not adversely affected:

1.) Care should be exercised during construction to ensure that the existing slopes are not steepened by placement of fill close to the crest of the slope since this would reduce the stability of the slope.
2.) Excavations should not be undertaken at the toe of the slopes since this would adversely affect the stability of the slopes.
3.) Natural drainage paths should not be blocked by placement of fill on the slope. If fill must be placed on the slope, adequate drainage should be provided to prevent buildup of pore pressures in the soil.
4.) Vegetation should not be removed from the faces of the slopes to prevent erosion. Additional vegetation should be planted on the slopes wherever necessary.

15.2 Behaviour of Slopes During Earthquakes

Lafebvre, G. (1981)[2] has stated that if the clay is not liquefiable, liquefaction during dynamic loading of earthquake will not be a concern. It has been previously demonstrated that the clay at the site is not susceptible to liquefaction. Therefore, it is concluded that the stability of the slopes at the site will not be adversely affected during a seismic event.

15.3 Flow Slides

Mitchell and Markall (1974)[3] have developed a method based on undrained shear strength to estimate the likelihood of flow slides. They measured the undrained shear strength of the clay using field shear vane.

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[2] Lefebvre, G. (1981), "*Fourth Canadian Geotechnical Colloquium: Strength and Slope Stability in Canadian Soft Clay Deposits"*, Can. Geot. J., Vol 18, pgs. 8420-422.6

[3] Mitchell, R..J. and Markell, A.R. 1974, *"Flow Slides in Sensitive Soils", Can Geot. J11, pgs. 423-454.*

Based on analyzing of the data for more than 40 sites, they established that flow slides will only occur in soils with total overburden pressure more than six times the undrained shear strength of the soil, i.e.:

$$\frac{\gamma H}{Su} > 6$$

Where ɣ is the bulk density of soil,

H is the height of the slope, and

Su is the undrained shear strength of the soil.

The maximum ratio of total overburden pressure to undrained shear strength of the on-site clay was computed as 3.6. Therefore, it is concluded that the clay at the site is not prone to flow slides.

16 Limit of Hazardous Lands

It is noted that to establish the limit of hazardous lands, in addition to the geotechnical set back, two other factors have to be taken into consideration. These are toe erosion allowance and erosion access allowance. The magnitude of the toe erosion allowance depends on the soil types, the state of erosion along the creek/river bank and upon the width of the channel. The Ministry of Natural Resources procedures permit either the installation of erosion protection or alternatively to consider a toe erosion allowance.

The north slopes of the creek located south of the site were examined by geotechnical engineers from **exp** to determine if creek banks are eroding. The examination of the slopes revealed that the slopes were heavily vegetated. The field observations also indicated that the ravine carries very little water except possibly during spring run-off. The locations along the creek where the photographs were taken have been plotted on Figure 2. However, localized minor erosion was observed at some locations as shown on photographs in Appendix A. Boreholes drilled at the site have indicated that the natural soil in the vicinity of the creek bottom is stiff clay. Based on this information, it is considered that a toe erosion allowance of 5 m should be provided. In addition to the toe erosion allowance, an erosion access allowance of 6 m is normally required. Therefore, the required setback for the south slopes (i.e. limit of hazardous lands) is 11 m from the crest of the slope except in the vicinity of Section A-A where it is 35 m.

The required limit of hazardous lands for the slope located on the south side of the site has been tabulated on Table IX at cross-section locations and has been plotted on Site Plan, Figure 2. The crest of the slope was assumed at the location where the ground surface flattens to inclination of 10H:1V. The limit of hazardous lands should be staked out in the field by a registered Ontario Land Surveyor as shown in Figure 2. No development should take place within the hazardous lands limits.

Table IX: Limit of Hazardous Lands at Cross-Section Locations

Section	Geotechnical Setback from the Toe (m)	Erosion Set Back (m)	Available Erosion Allowance in Valley Floor (m)	Erosion Access Allowance (m)	Total Setback from creek or toe of the slope (Limit of Hazardous Lands) (m)
A-A	24	5	0	6	35
B-B	0	5	0	6	11
C-C	0	5	0	6	11

A ravine or creek is not located along the north slope. Therefore, an erosion allowance is not required for the slope. The required setback along the north side of the site is 6 m (erosion access allowance) from the crest of the slope.

17 General Comments

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

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

Figures

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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

scale 1:20,000	CLIENT:	DCR/PHOENIX GROUP OF COMPANIES	project no. OTT-00234493-A0
date NOV. 2, 2016	TITLE:	SITE LOCATION PLAN 1154 - 1208 OLD MONTREAL ROAD, OTTAWA, ON	FIG 1
drawn by M.N.			

Filename: r:\230000\234000\234493 - 1154-1208 old montreal road\drawings\1154-1208 old montreal.dwg
Last Saved: 11/2/2016 11:38:48 AM
Last Plotted: 11/2/2016 11:41:18 AM Plotted by: nugentm Pen Table:: exp-64.ctb

NOTES

THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

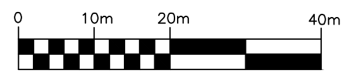
NOTES :

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

NO.	REVISION DESCRIPTION	DATE	BY	APPD
1		DD/MM/YY		

SCALE

HORZ 1:1000



HORIZONTAL 1:1000

LEGEND

⊕ BH1 ELEVATION= 85.82	BOREHOLE NUMBER, LOCATION AND ELEVATION
① ↘	PHOTO NUMBER, LOCATION AND DIRECTION

CLIENT

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BASEPLAN

DESIGN: S. AGGARWAL

CHECKED: S. AGGARWAL

CAD: J.R./ M.N.

PROJ. MAN: I. TAKI

APPROVED: I. TAKI

PROJECT

PROPOSED RESIDENTIAL DEVELOPMENT
1154 - 1208 OLD MONTREAL ROAD
OTTAWA, ONTARIO

TITLE

BOREHOLE LOCATION
& LIMIT OF HAZARDOUS LANDS

PROJ NO.: OTT-00234493-A0

SURVEY

DATE: NOV. 2, 2016

DRAWING NO.: FIG 2

Filename: r:\230000\234000\234493 - 1154-1208 old montreal road\drawings\1154-1208 old montreal.dwg
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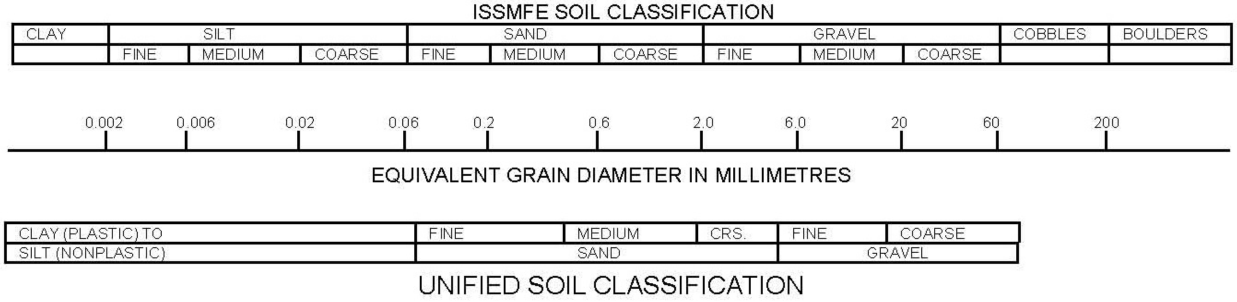
Pen Table:: omm pen setting july 20.ctb
References: 385039A.DWG; 385039B.DWG; 385040D.DWG

Notes On Sample Descriptions

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



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

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

Log of Borehole BH-1

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Figure No. 3

Page. 1 of 2

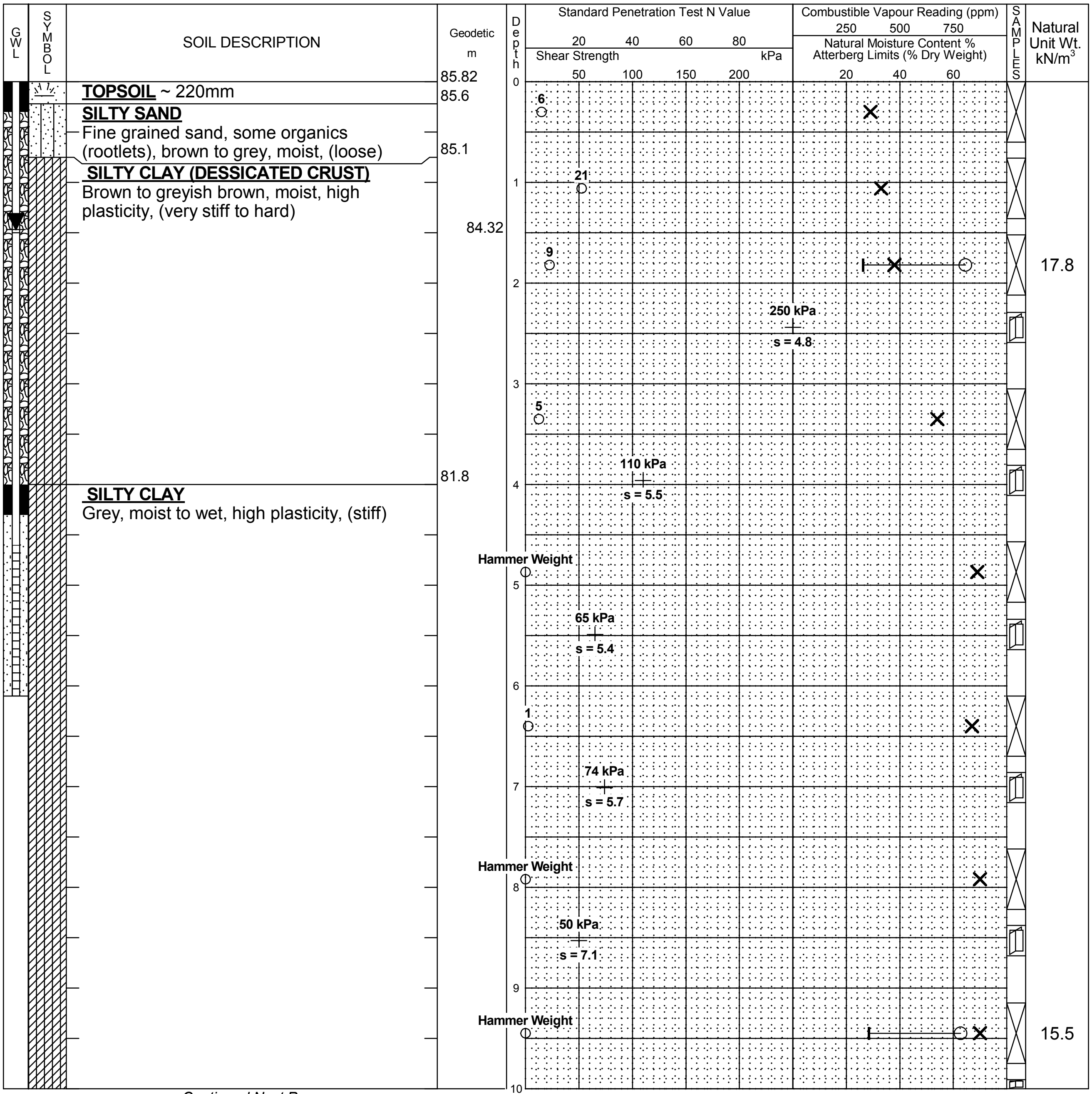
Date Drilled: 'August 15, 2016

Drill Type: 'Trackmount CME 55

Datum: Geodetic

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		



TOPSOIL ~ 220mm

SILTY SAND
Fine grained sand, some organics (rootlets), brown to grey, moist, (loose)

SILTY CLAY (DESSICATED CRUST)
Brown to greyish brown, moist, high plasticity, (very stiff to hard)

SILTY CLAY
Grey, moist to wet, high plasticity, (stiff)

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 19 mm diameter piezometer was installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
'Sept 10, 2016	1.5	

CORE DRILLING RECORD

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

Log of Borehole BH-1

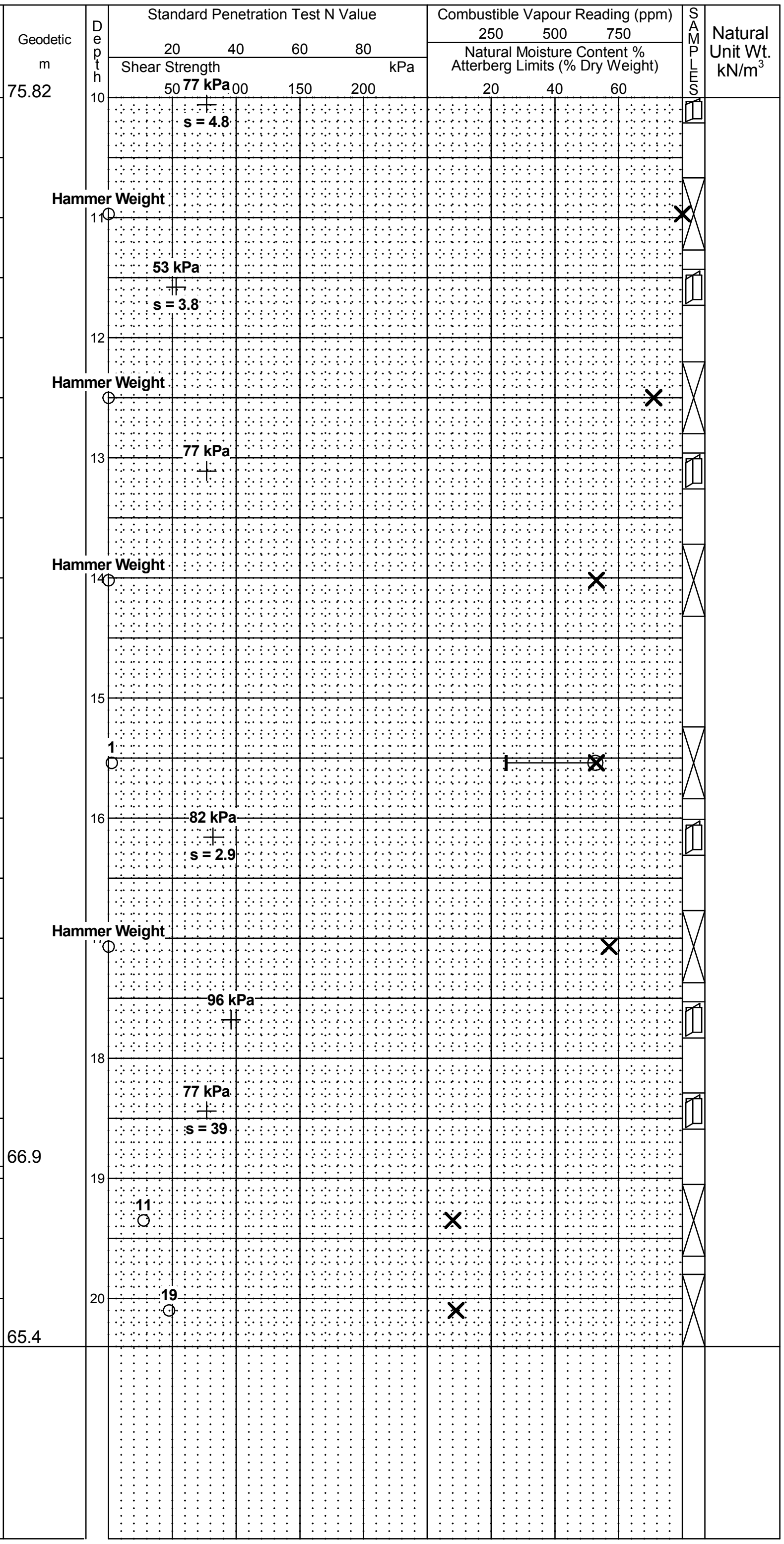
Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 3

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			75.82
		SILTY CLAY Grey, moist to wet, high plasticity, (stiff) *(continued)*	
		GRAVELLY SAND TILL Some silt, trace clay, grey, wet, (compact)	66.9
		Borehole Terminated at 20.4 m Depth	65.4



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 19 mm diameter piezometer was installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
'Sept 10, 2016	1.5	

CORE DRILLING RECORD

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

Log of Borehole **BH-2**

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Figure No. 4

Page. 1 of 1

Date Drilled: 'August 16, 2016

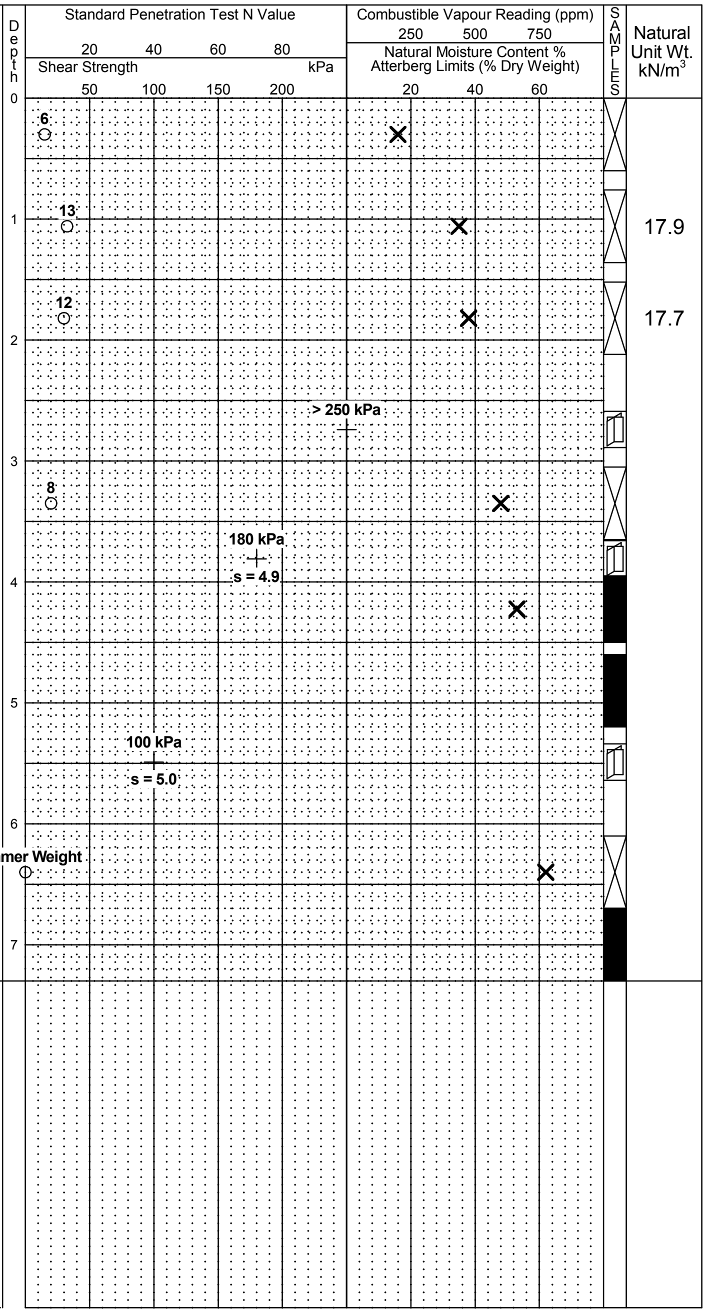
Drill Type: 'Trackmount CME 55

Datum: Geodetic

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			85.4
		TOPSOIL ~ 200mm	85.2
		GRAVELLY SAND Some silt, trace clay, brown, grey, moist, (loose)	84.7
		SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity, (very stiff to hard)	81.4
		SILTY CLAY Grey, moist to wet, high plasticity, (stiff)	78.1
		Borehole Terminated at 7.3 m Depth	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled With Cuttings Upon Completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-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 BH-3

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Figure No. 5

Page. 1 of 3

Date Drilled: 'August 17, 2016

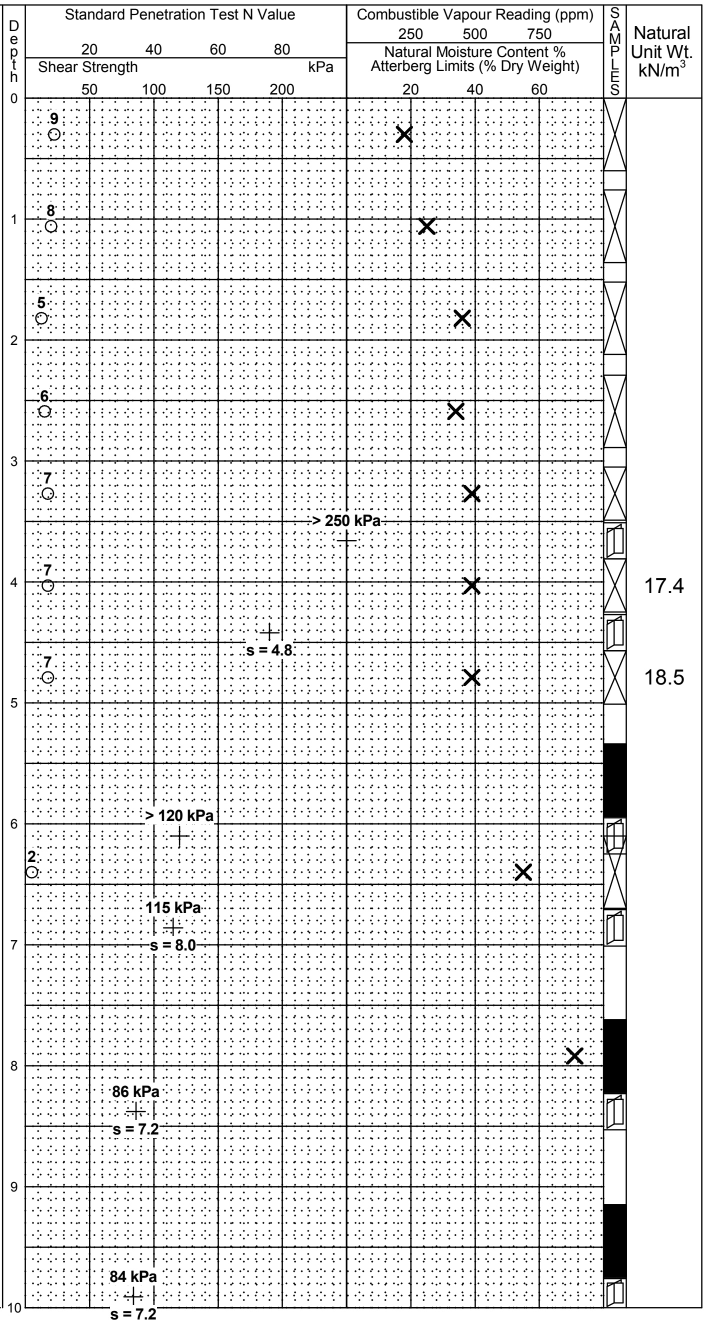
Drill Type: 'Trackmount CME 55

Datum: Geodetic

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		

Geodetic m	SOIL DESCRIPTION
84.3 84.2	**TOPSOIL** ~ 100mm
	FILL Sand and gravel to silty clay with gravel, some organics, dark brown to grey, moist (loose)
82.0	**SILTY CLAY (DESSICATED CRUST)** Brown to greyish brown, moist, high plasticity, (hard to very stiff)
81.8	
79.3	**SILTY CLAY** Grey, moist to wet, high plasticity, (very stiff to stiff)



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NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 19 mm diameter piezometer was installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
'Sept 10, 2016	2.5	

CORE DRILLING RECORD

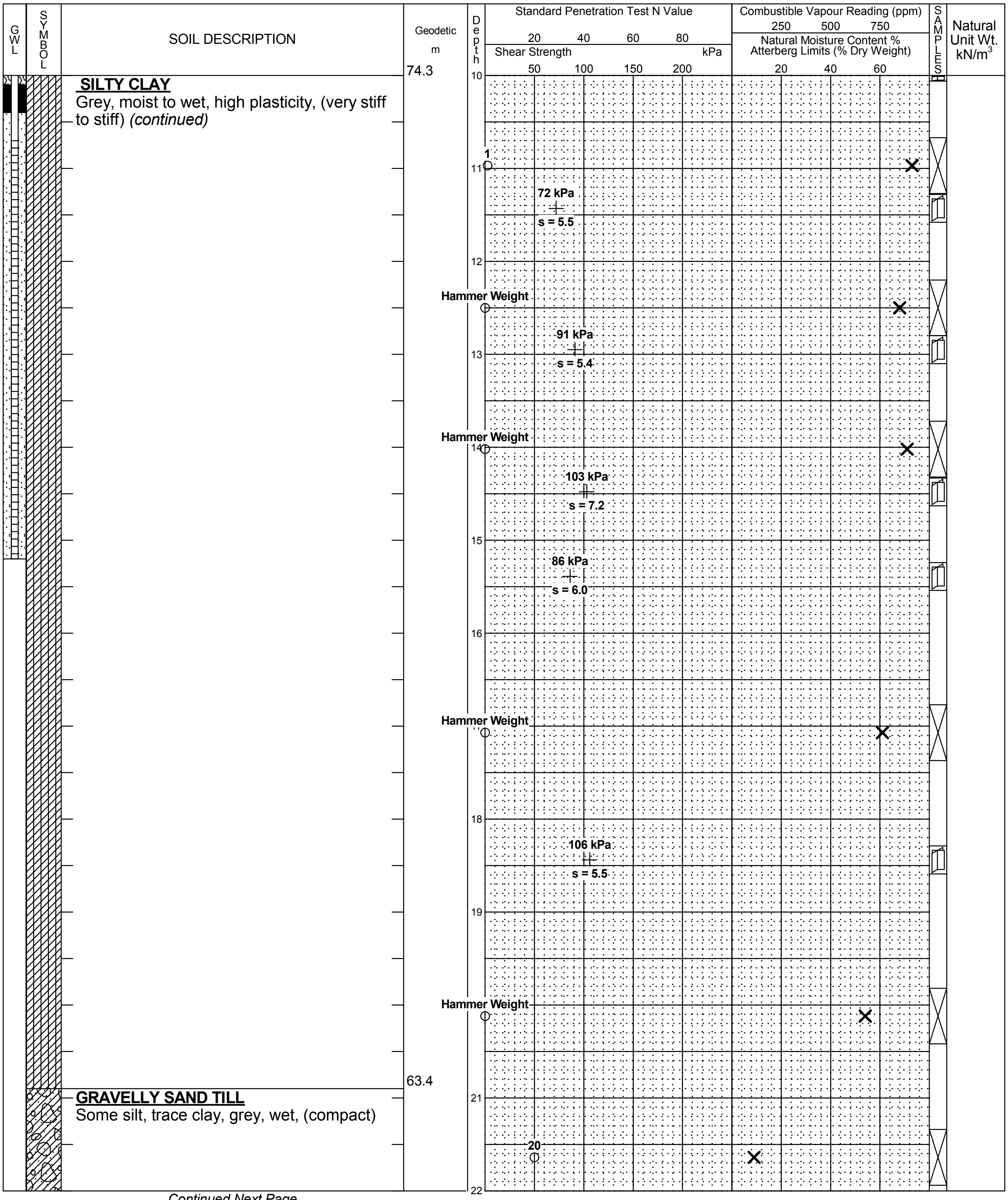
Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-3

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 5



SILTY CLAY
Grey, moist to wet, high plasticity, (very stiff to stiff) *(continued)*

GRAVELLY SAND TILL
Some silt, trace clay, grey, wet, (compact)

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 19 mm diameter piezometer was installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
'Sept 10, 2016	2.5	

CORE DRILLING RECORD

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

Log of Borehole BH-3

Project No: OTT-00234493-A0

Figure No. 5

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Page. 3 of 3

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
	▨	**GRAVELLY SAND TILL** Some silt, trace clay, grey, wet, (compact) *(continued)*	62.3	22				
				23				
			61.0					
		Borehole Terminated at 23.3 m Depth Upon Auger Refusal						

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 19 mm diameter piezometer was installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
'Sept 10, 2016	2.5	

CORE DRILLING RECORD

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

Log of Borehole BH-4

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Figure No. 6

Page. 1 of 1

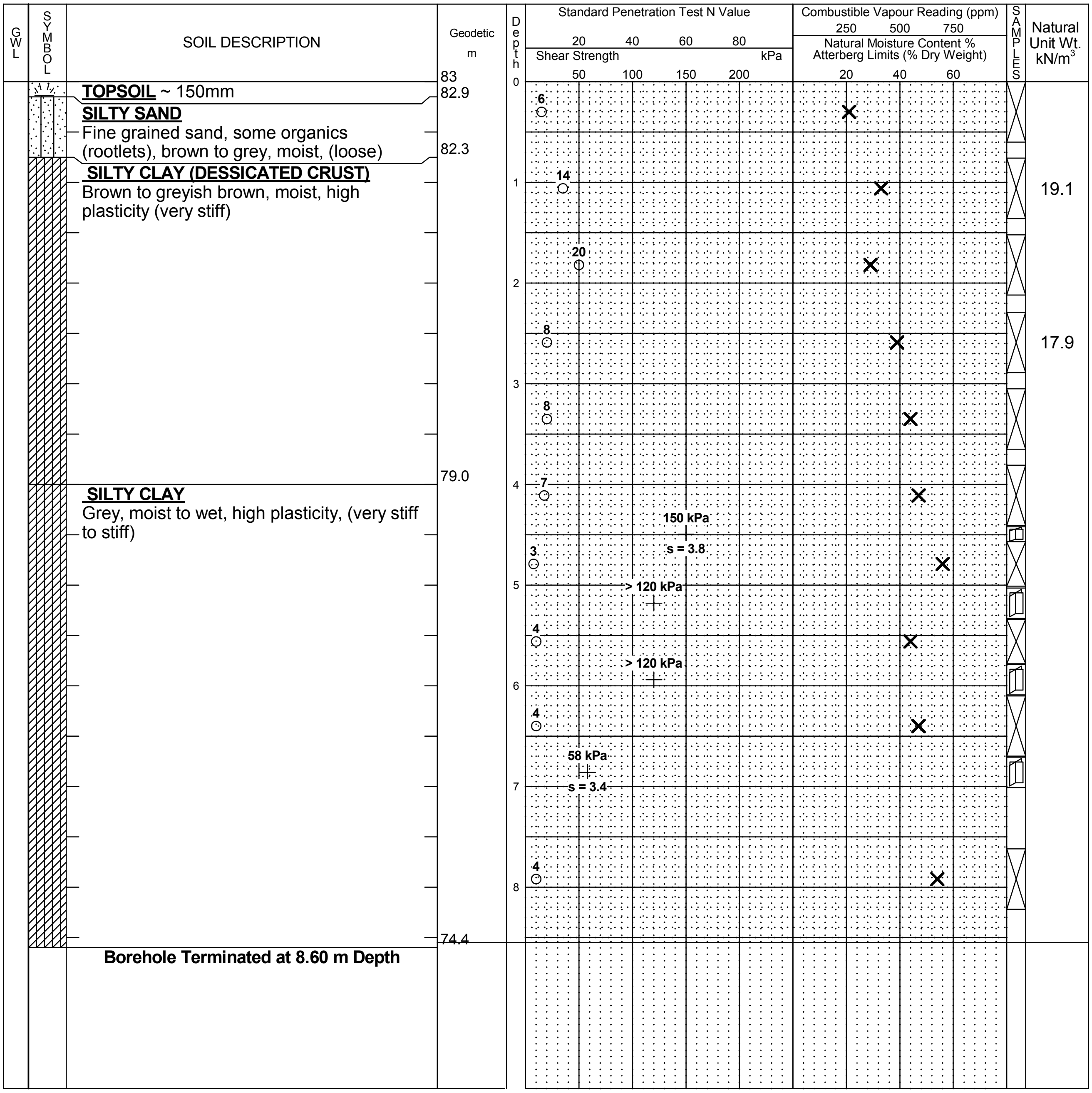
Date Drilled: 'August 18, 2016

Drill Type: 'Trackmount CME 55

Datum: Geodetic

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		



TOPSOIL ~ 150mm

SILTY SAND
Fine grained sand, some organics (rootlets), brown to grey, moist, (loose)

SILTY CLAY (DESSICATED CRUST)
Brown to greyish brown, moist, high plasticity (very stiff)

SILTY CLAY
Grey, moist to wet, high plasticity, (very stiff to stiff)

Borehole Terminated at 8.60 m Depth

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled With Cuttings Upon Completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-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 BH-5

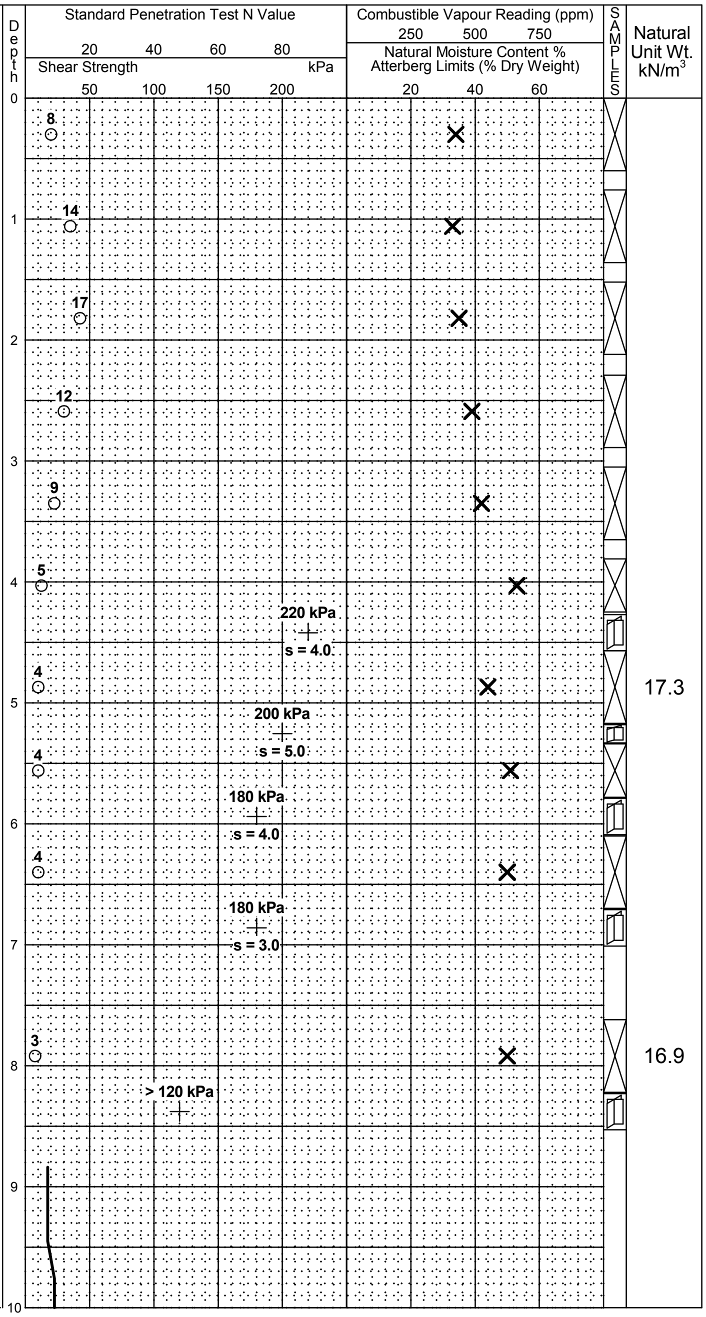
Project No:	OTT-00234493-A0
Project:	Preliminary Geotechnical Investigation. Proposed Residential Subdivision
Location:	1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON
Date Drilled:	'August 18, 2016
Drill Type:	'Trackmount CME 55
Datum:	Geodetic
Logged by:	
Checked by:	

Figure No. 7

Page. 1 of 2

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		

Geodetic m	SOIL DESCRIPTION
81	
80.9	**TOPSOIL** ~ 50mm
	SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity (hard)
77.5	**SILTY CLAY** Grey, moist to wet, high plasticity, (hard to very stiff)
72.4	**Borehole Terminated at 8.60 m Depth**
72.3	**INFERRED OVERBURDEN** Cone advanced from 8.6 m to to refusal at 20.9 m depth



Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled With Cuttings Upon Completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-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 BH-5

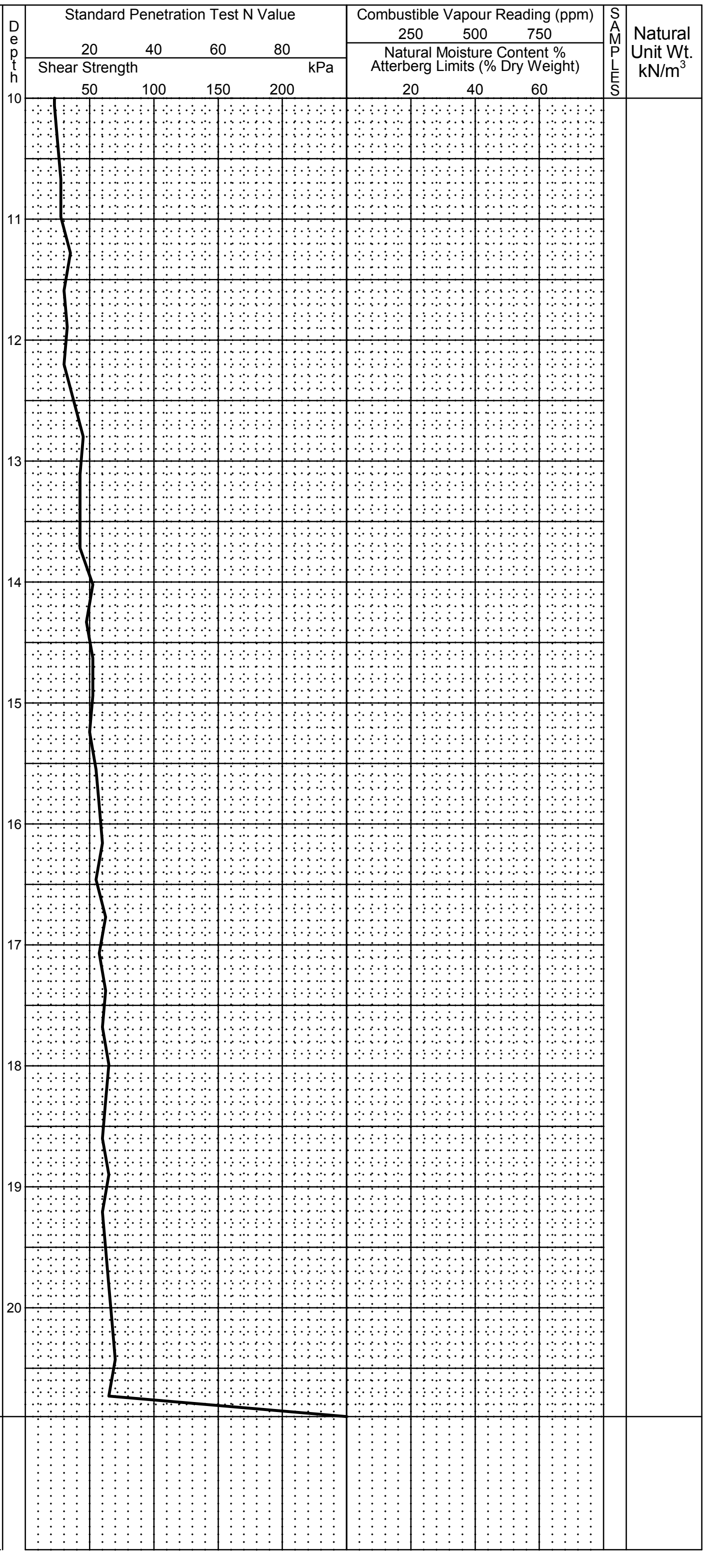
Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 7

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
		INFERRED OVERBURDEN Cone advanced from 8.6 m to to refusal at 20.9 m depth *(continued)*	71
			60.1
		Refusal to Cone Penetration at 20.9 m Depth	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled With Cuttings Upon Completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-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 BH-6

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Figure No. 8

Page. 1 of 1

Date Drilled: 'August 16, 2016

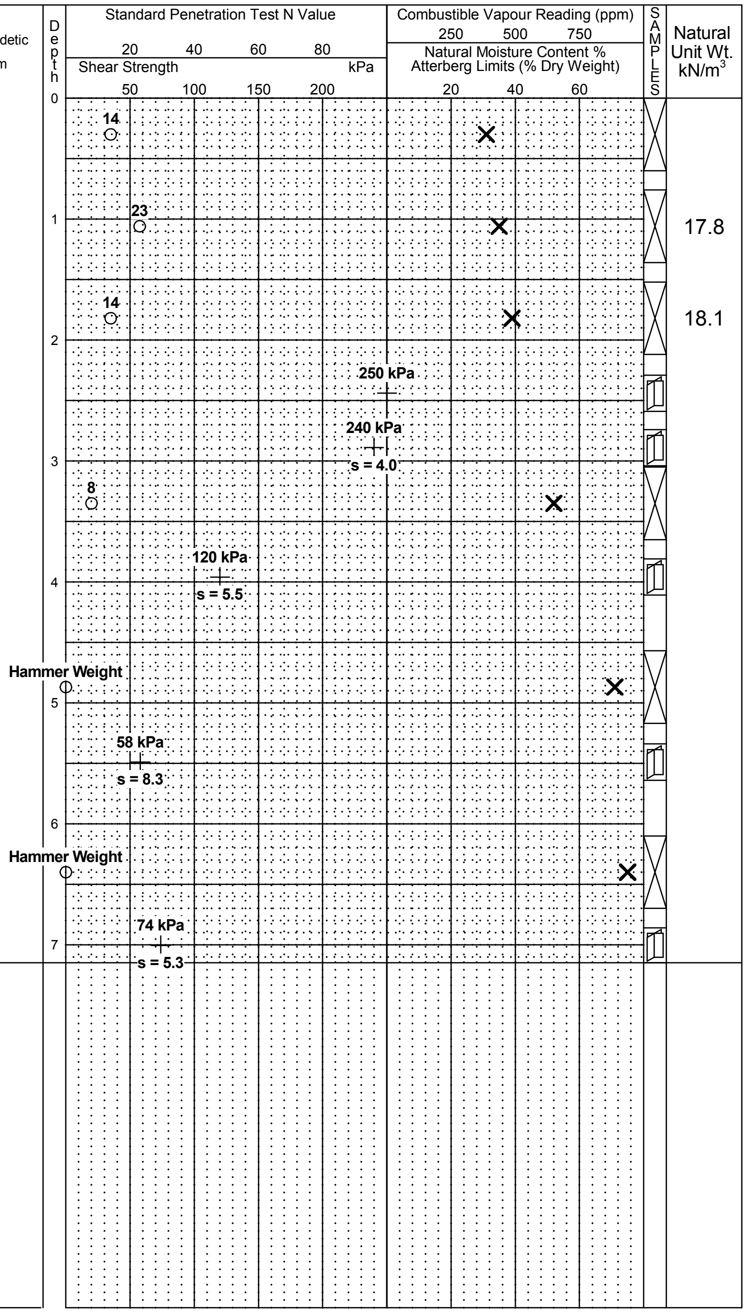
Drill Type: 'Trackmount CME 55

Datum: Geodetic

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
			86.6	0
		TOPSOIL ~ 50mm	86.5	
		FILL Sand and gravel mixed with silty sand, brown, moist, (compact)	85.9	
		SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity (hard)	83.6	
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff)	79.5	
		Borehole Terminated at 7.2 m Depth		



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole Backfilled With Cuttings Upon Completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-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 BH-7

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Figure No. 9

Page. 1 of 1

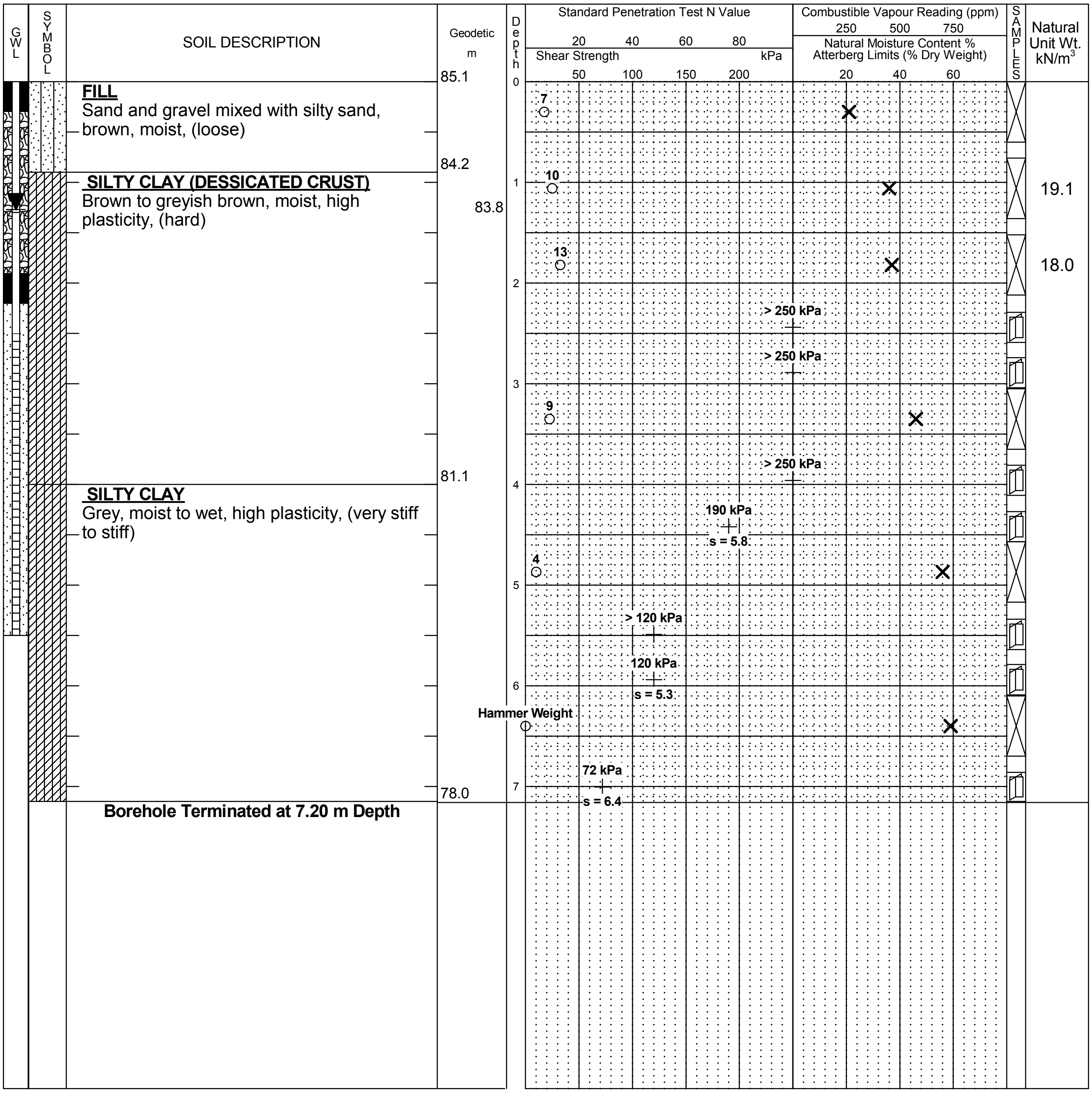
Date Drilled: 'August 16, 2016

Drill Type: 'Trackmount CME 55

Datum: Geodetic

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲



FILL
Sand and gravel mixed with silty sand, brown, moist, (loose)

SILTY CLAY (DESSICATED CRUST)
Brown to greyish brown, moist, high plasticity, (hard)

SILTY CLAY
Grey, moist to wet, high plasticity, (very stiff to stiff)

Borehole Terminated at 7.20 m Depth

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 19 mm diameter piezometer was installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00234493-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
'Sept 10, 2016	1.3	

CORE DRILLING RECORD

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

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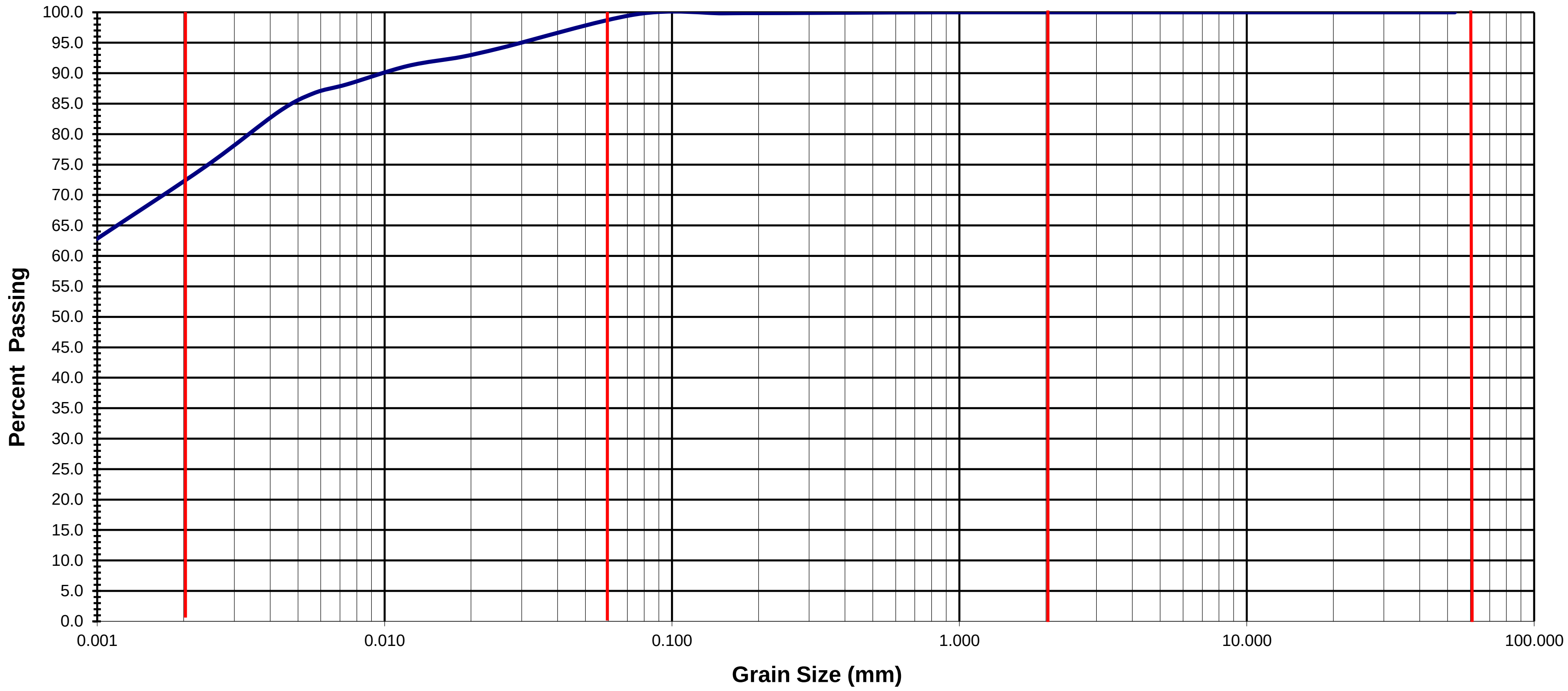
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

Test Method ASTM D-422

Modified M.I.T. Classification

CLAY	SILT			SAND			GRAVEL			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



Exp Project No.:	**OTT-00234493-A0**	**Project Name :**	**Preliminary Geotechnical Invsetigation. Proposed Residential Subdvision**				
Client :	**DCR/Phoenix Group of Companies**	**Project Location :**	**1154 - 1208 Old Montreal Road, City of Ottawa, Ontario**				
Date Sampled :	**August 15, 2016**	**Borehole.:**	**1**	**Sample No.:**	**SS3**	**Depth (m) :**	**1.5-2.1**
Sample Description :	**Silty Clay. Trace Sand**					**Figure :**	**10**

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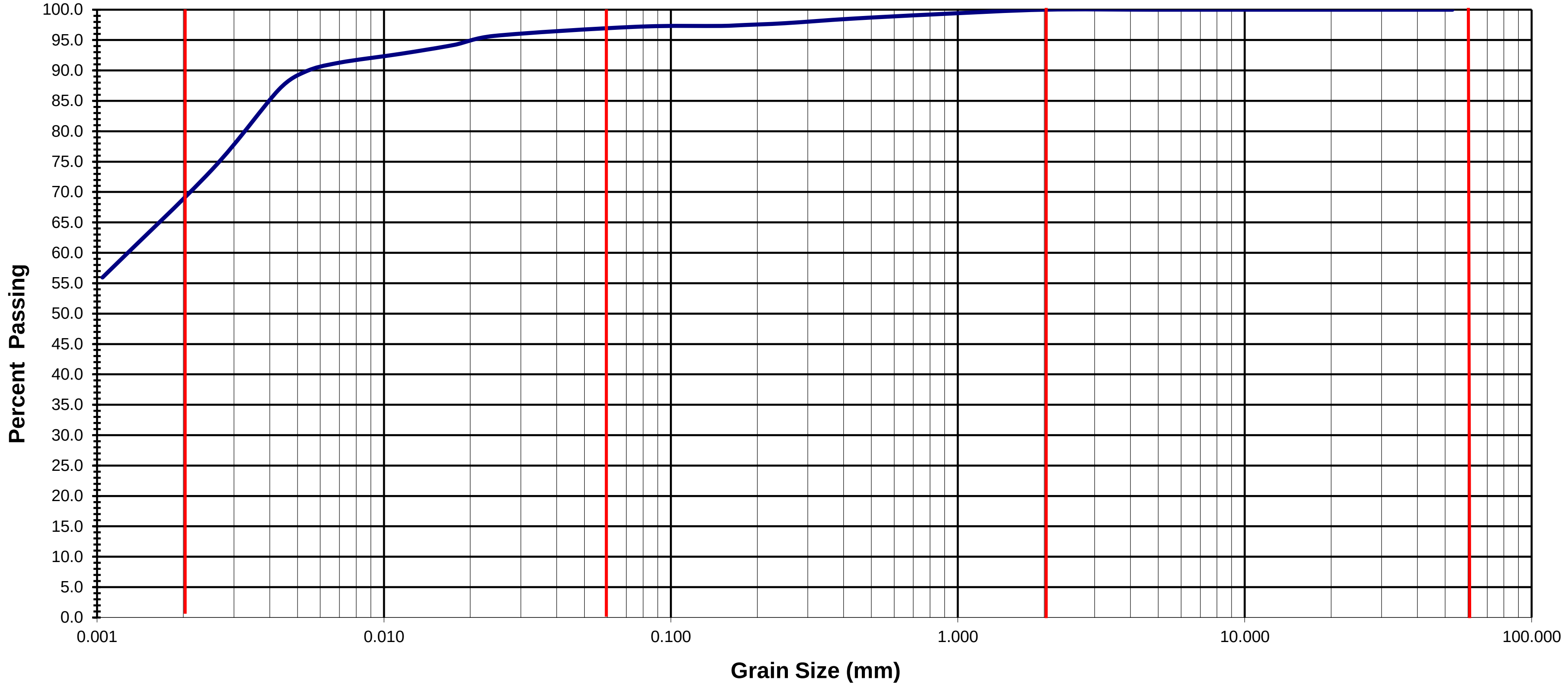
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

Test Method ASTM D-422

Modified M.I.T. Classification

CLAY	**SILT**			**SAND**			**GRAVEL**			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



Exp Project No.:	**OTT-00234493-A0**	**Project Name :**	**Preliminary Geotechnical Invsetigation. Proposed Residential Subdvision**		
Client :	**DCR/Phoenix Group of Companies**	**Project Location :**	**1154 - 1208 Old Montreal Road, City of Ottawa, Ontario**		
Date Sampled :	**August 15, 2016**	**Borehole.:**	**1**	**Sample No.: SS8**	**Depth (m) : 9.1-9.8**
Sample Description :	**Silty Clay. Trace Sand**				**Figure : 11**

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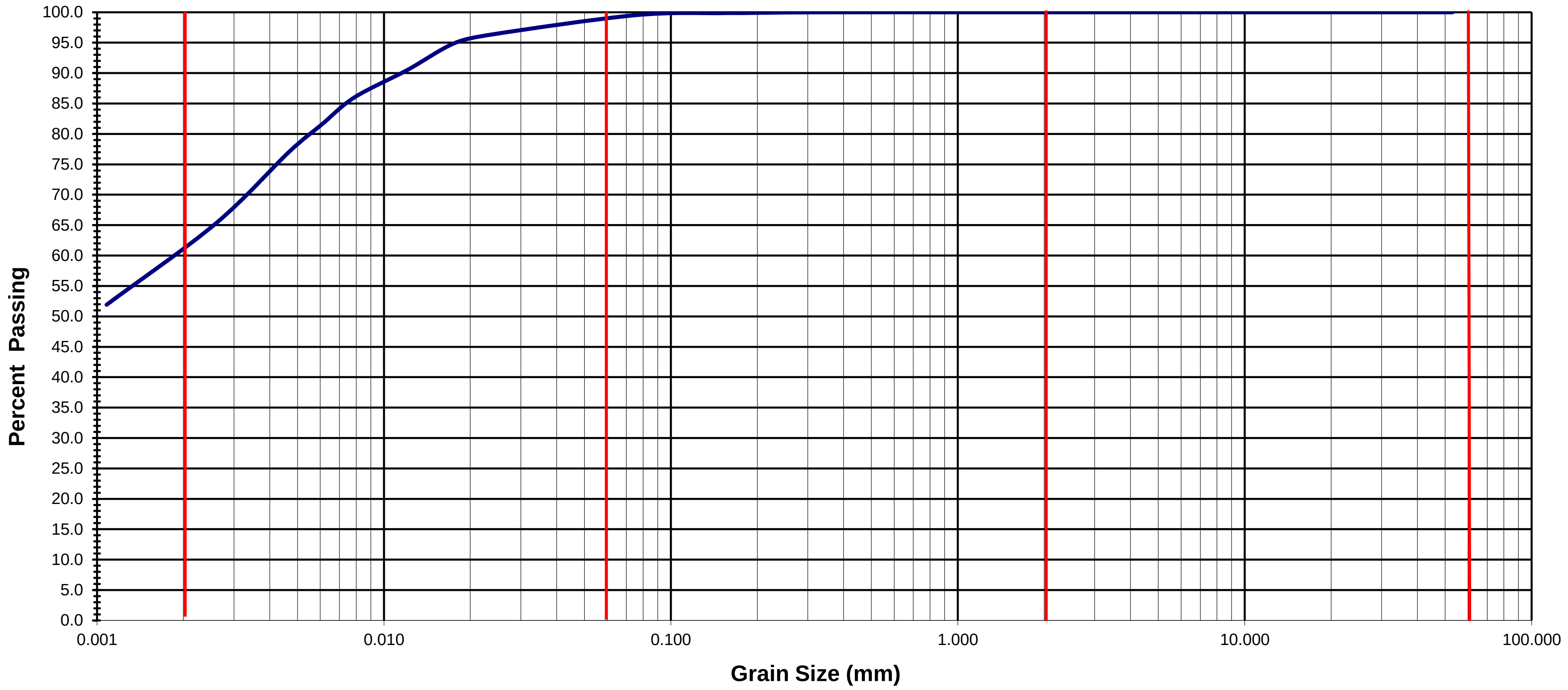
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

Test Method ASTM D-422

Modified M.I.T. Classification

CLAY	SILT			SAND			GRAVEL			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



Exp Project No.:	**OTT-00234493-A0**	**Project Name :**	**Preliminary Geotechnical Invsetigation. Proposed Residential Subdvision**				
Client :	**DCR/Phoenix Group of Companies**	**Project Location :**	**1154 - 1208 Old Montreal Road, City of Ottawa, Ontario**				
Date Sampled :	**August 15, 2016**	**Borehole.:**	**1**	**Sample No.:**	**SS12**	**Depth (m) :**	**15.2-15.8**
Sample Description :	**Silt - Clay**					**Figure :**	**12**

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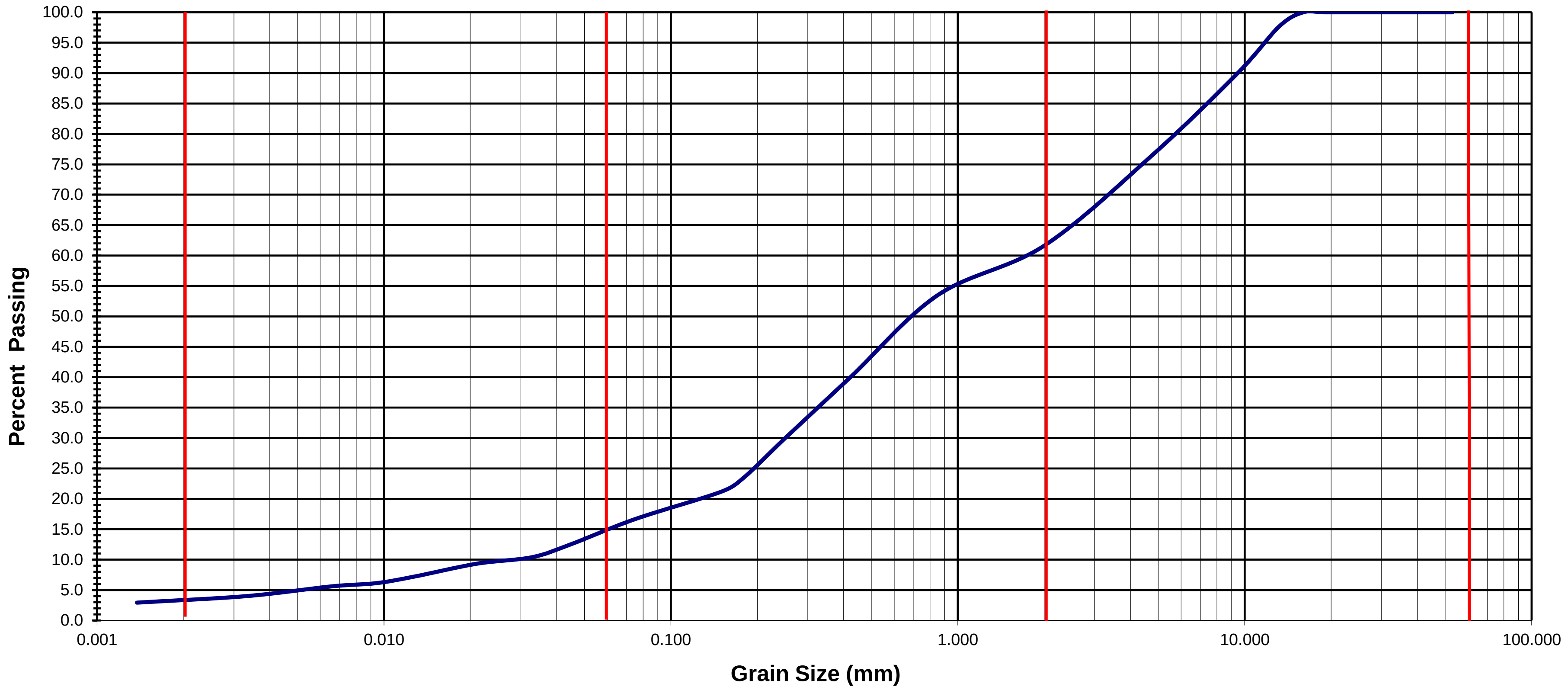
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

Test Method ASTM D-422

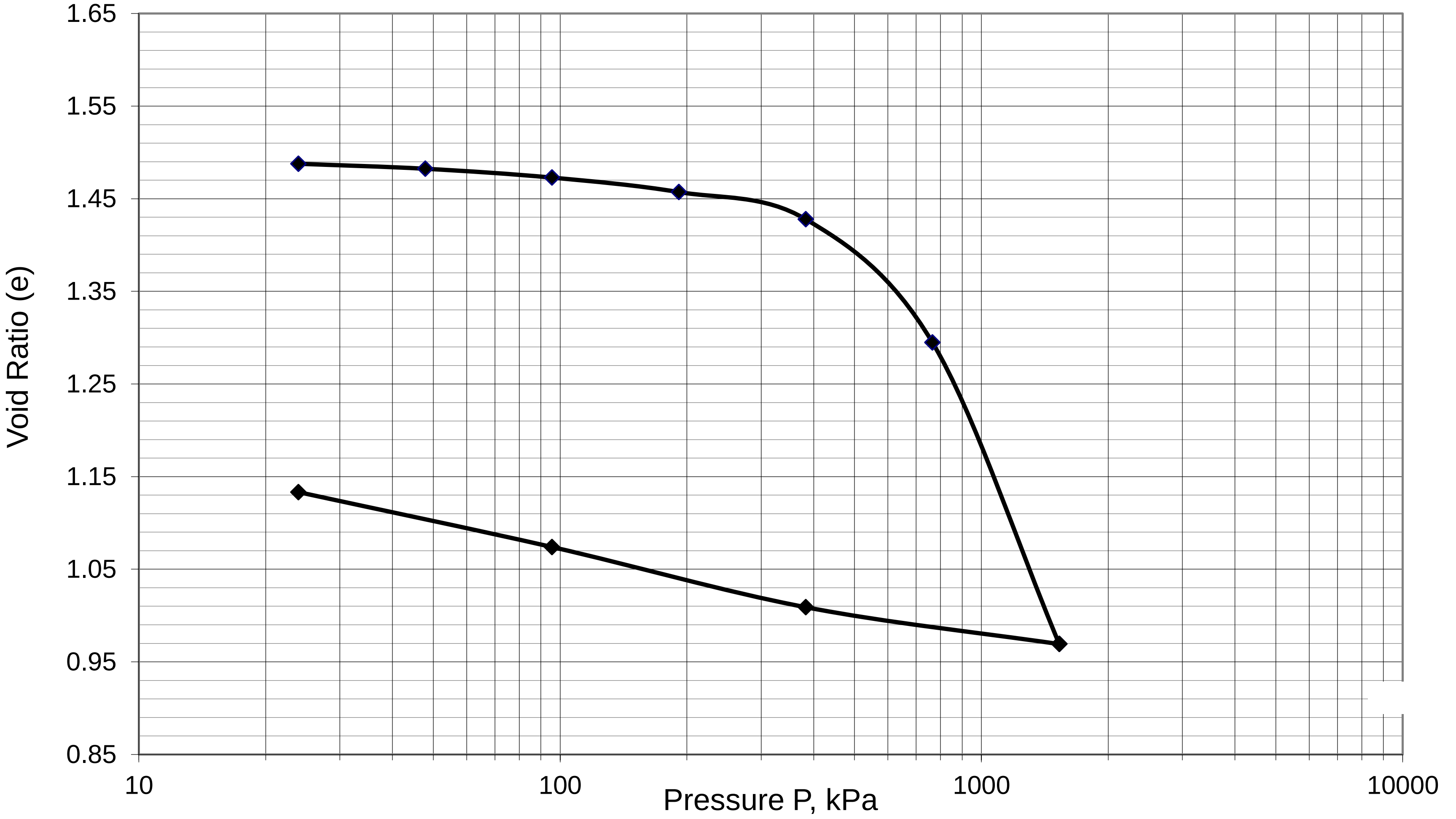
Modified M.I.T. Classification

CLAY	SILT			SAND			GRAVEL			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



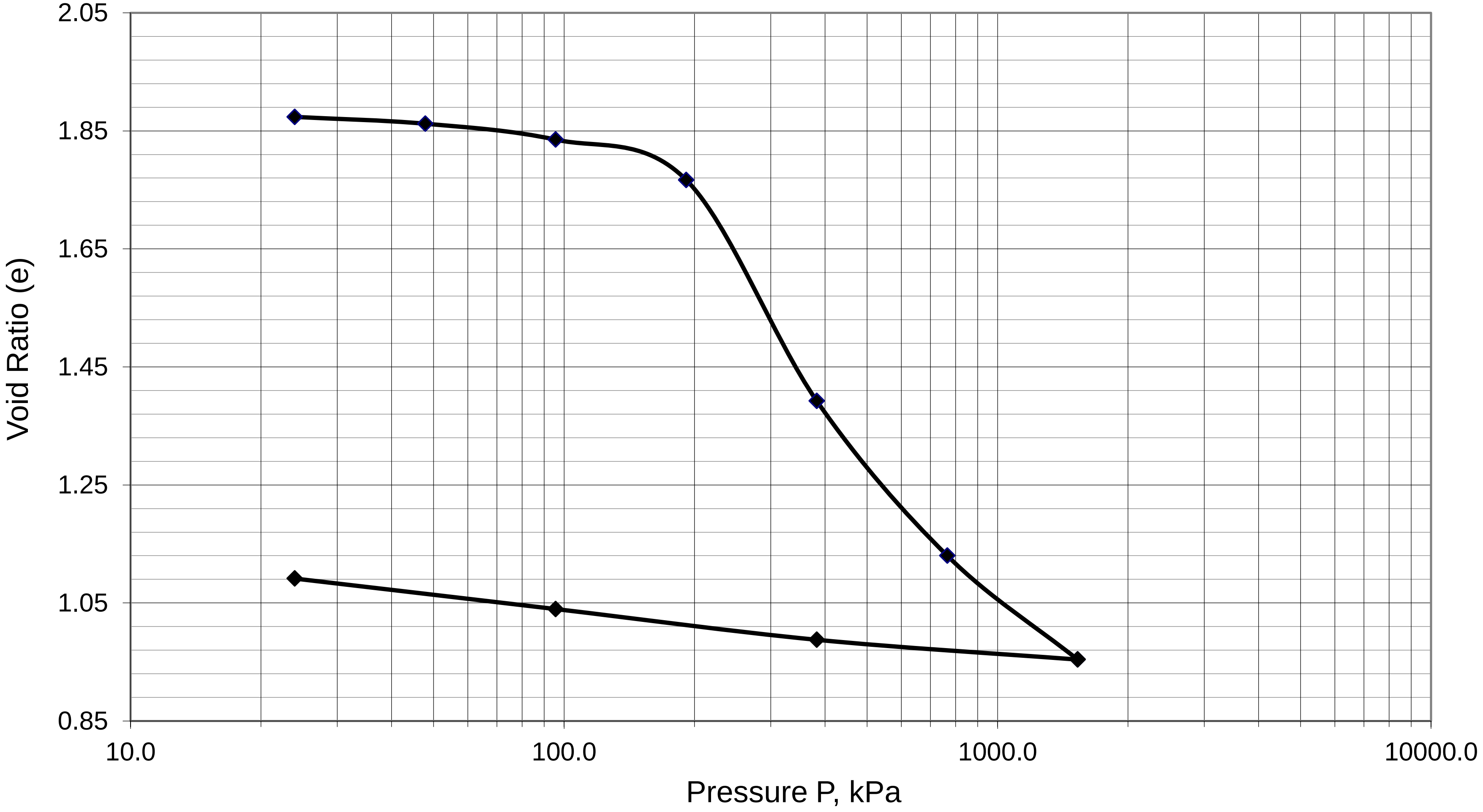
Exp Project No.:	**OTT-00234493-A0**	**Project Name :**	**Preliminary Geotechnical Invsetigation. Proposed Residential Subdvision**				
Client :	**DCR/Phoenix Group of Companies**	**Project Location :**	**1154 - 1208 Old Montreal Road, City of Ottawa, Ontario**				
Date Sampled :	**August 15, 2016**	**Borehole.:**	**1**	**Sample No.:**	**SS15**	**Depth (m) :**	**19.8-20.4**
Sample Description :	**Sand and Gravel, Some Silt, Trace Clay**					**Figure :**	**13**

Consolidation Test Results - BH 2 -TW5



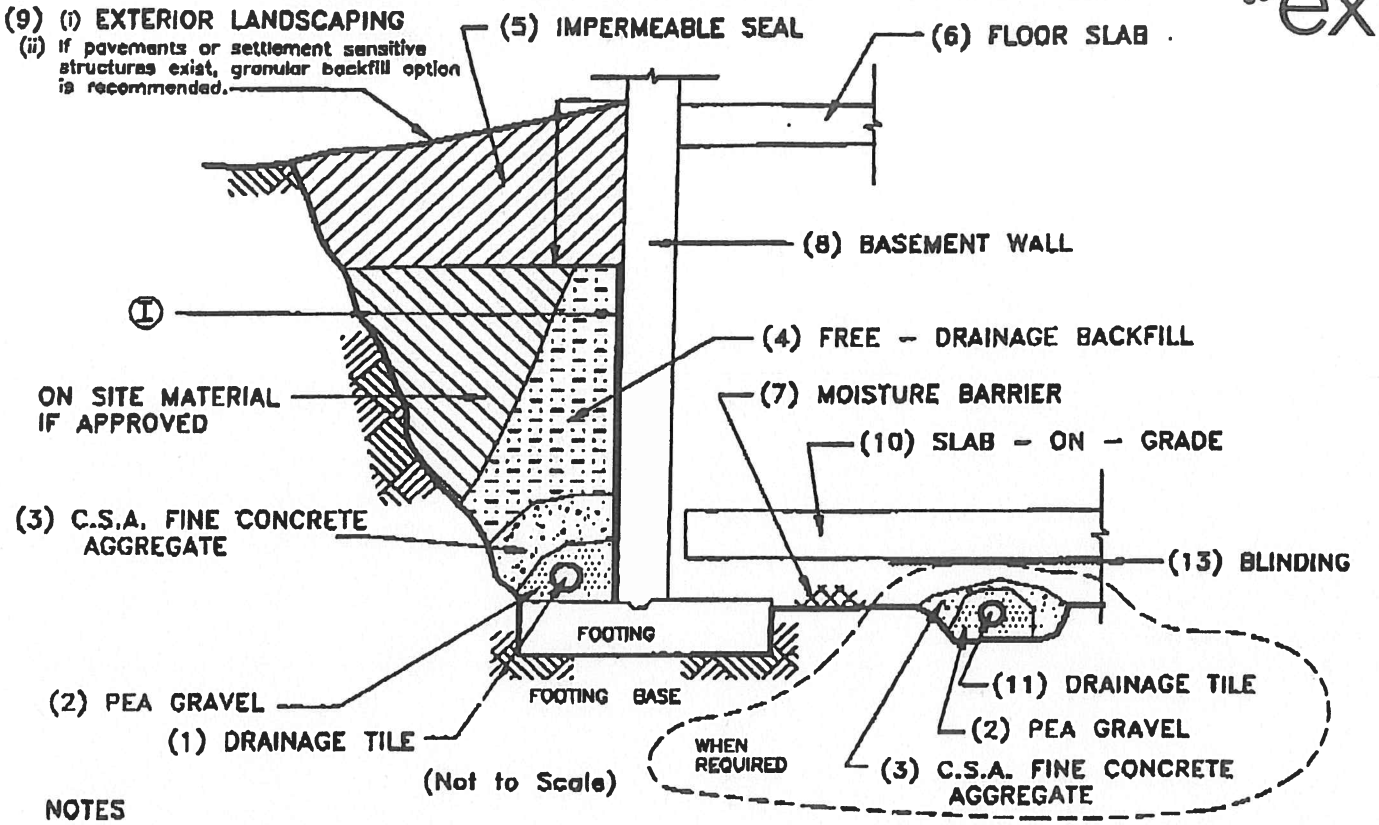
Borehole	**BH-2**	P'o	**75.4**	Ccr	**0.153**
Sample No.	**TW5**	P'c	**595.0**	Cc	**1.070**
Sample Depth (m)	**4.3**	OC Ratio	**7.9**	Wo (%)	**53**
Sample Elev. (m)	**81.1**	Initial Void Ratio	**1.497**	Unit Wt.(KN/m3)	**Crust 18, Grey Clay 16.5**
Project Number	**OTT-00234493-A0**	**Sample Description**	**Silty Clay**	**Figure Number**	**14**

Consolidation Test Results - BH3 - TW10



Borehole	**BH-3**	P'o	**114.0**	Ccr	**0.110**
Sample No.	**TW-10**	P'c	**192.0**	Cc	**1.360**
Sample Depth (m)	**7.9**	OC Ratio	**1.7**	Wo (%)	**71.0**
Sample Elev. (m)	**76.4**	Initial Void Ratio	**1.892**	Unit Wt.(KN/m3)	**Crust 18, Grey Clay 16.5**
Project Number	**OTT-00234493-A0**	**Sample Description**	**Clay - Grey**	**Figure Number**	**15**

BASEMENT DRAINAGE DRAWING



(Not to Scale)

NOTES

OPTION A – GRANULAR BACKFILL

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

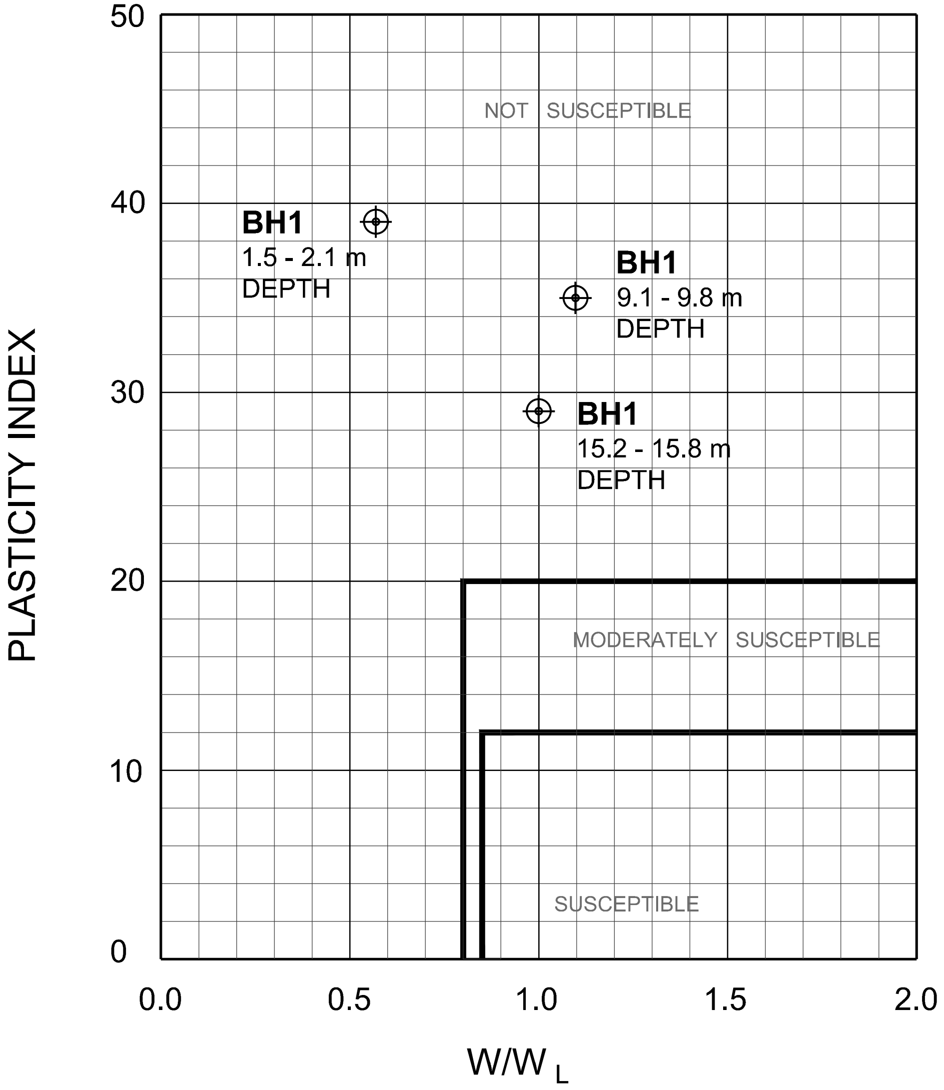
NOTE: A) Underfloor drainage can be deleted where not required (see report).

OPTION B – CORE DRAIN

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

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

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



BRAY ET AL. (2004) CRITERIA FOR LIQUEFACTION
ASSESSMENT OF FINE-GRAINED SOILS

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scale N.T.S.
date OCT. 31, 2016
drawn by M.N.

CLIENT: DCR PHOENIX GROUP OF COMPANIES

TITLE: PRELIMINARY GEOTECHNICAL INVESTIGATION
1154 - 1208 OLD MONTREAL ROAD, OTTAWA, ON

project no. OTT-00234493-A0

FIG 17

PROJECT No.: OTT-00234493-A0
1154-1208 Old Montreal, Ottawa, Ontario
Section A-A
Effective Stress Analysis

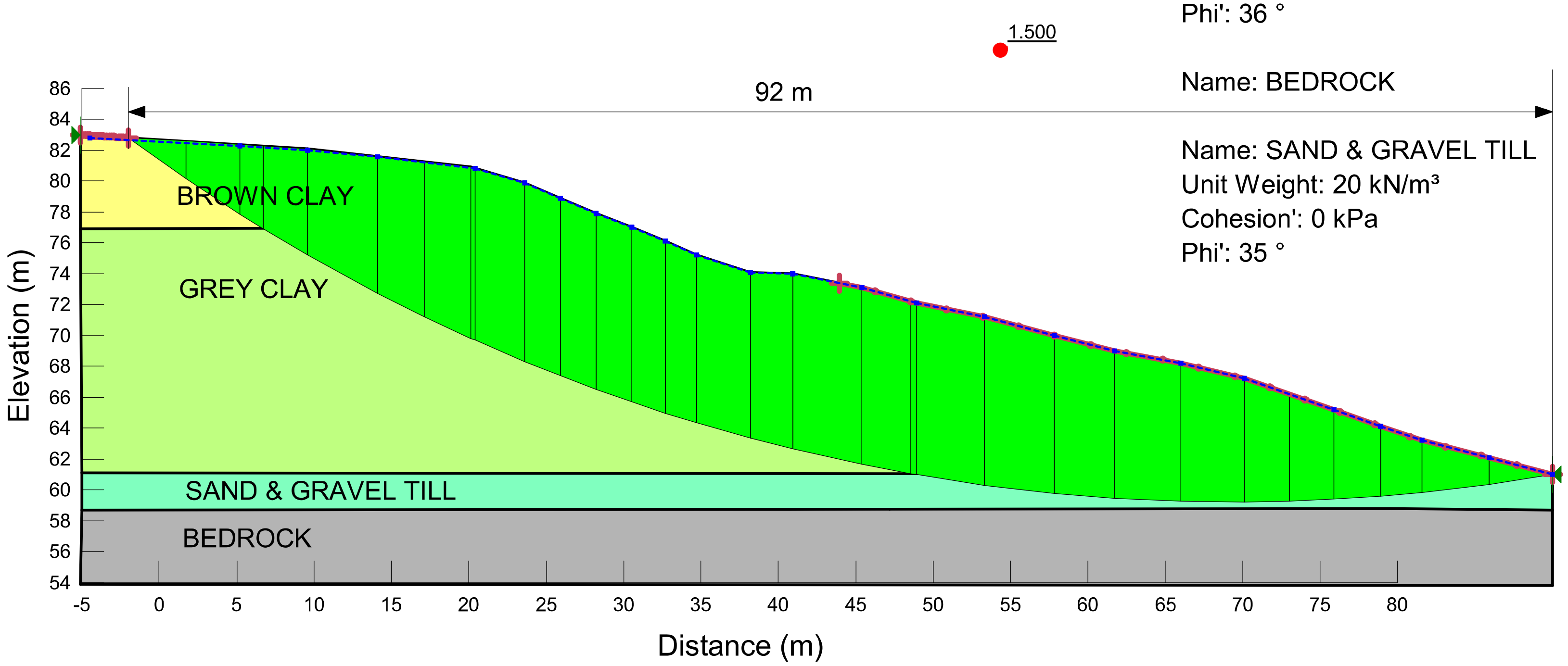
Figure No. 18

Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

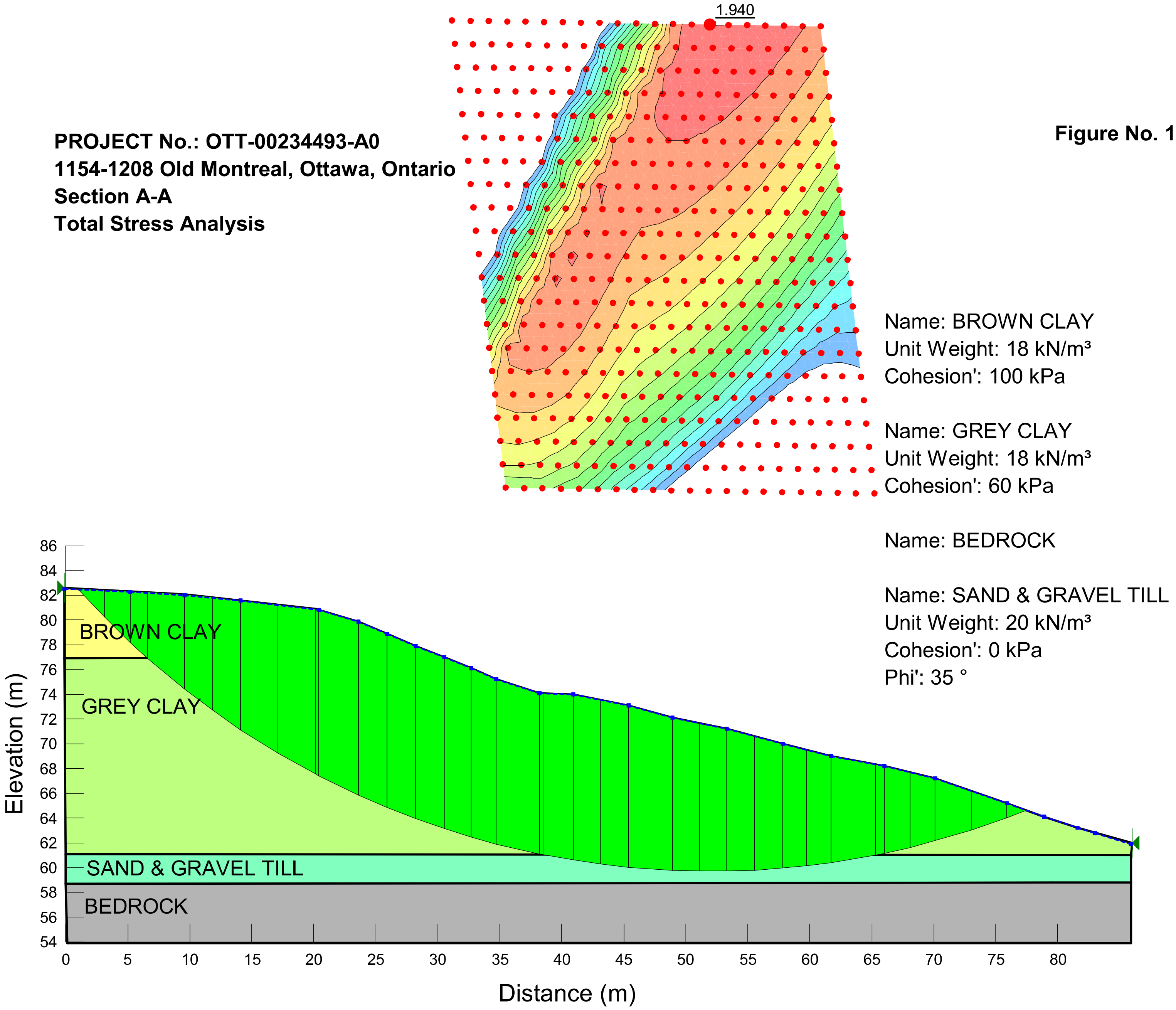
Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



PROJECT No.: OTT-00234493-A0
1154-1208 Old Montreal, Ottawa, Ontario
Section A-A
Total Stress Analysis

Figure No. 19



Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 100 kPa

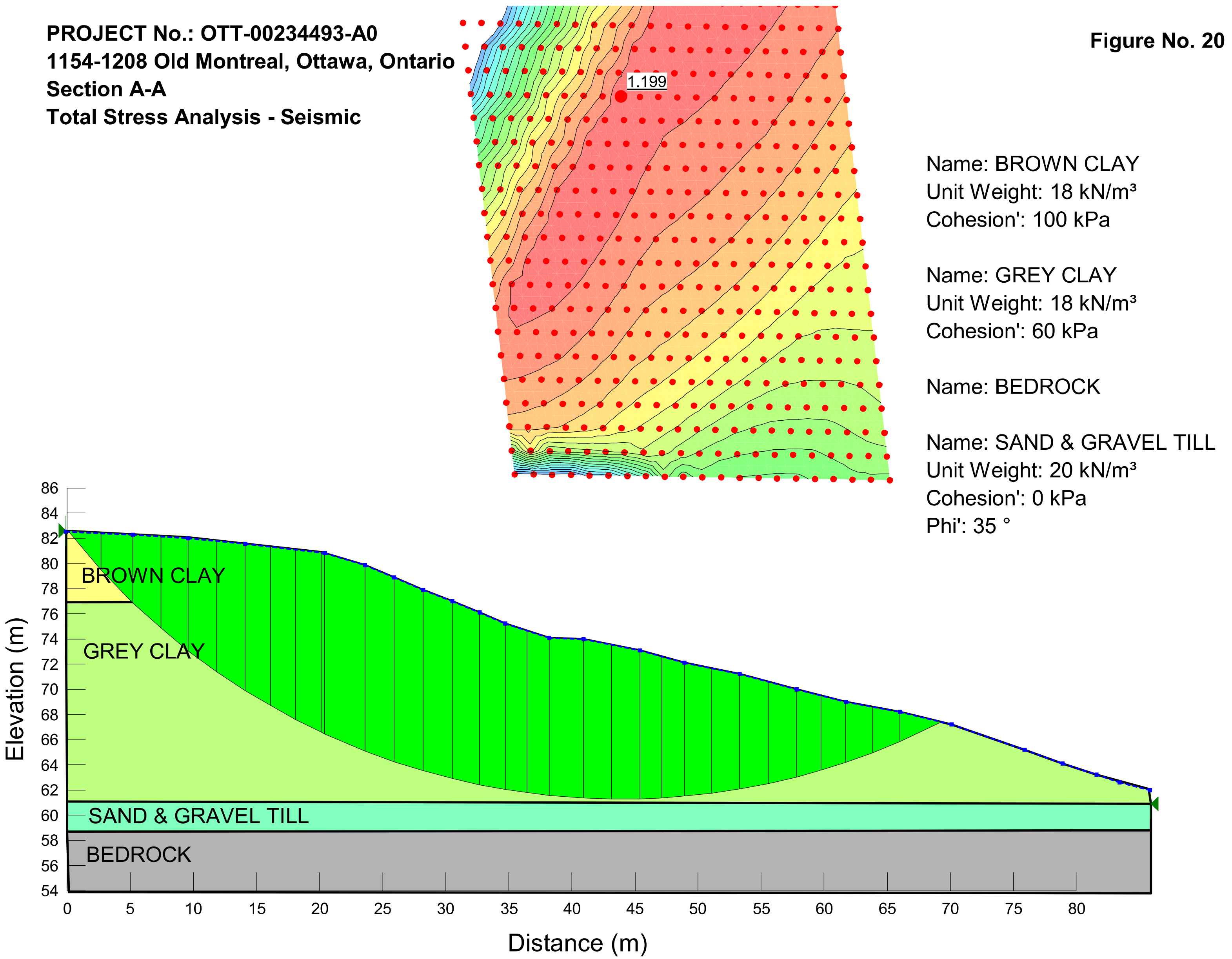
Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

PROJECT No.: OTT-00234493-A0
1154-1208 Old Montreal, Ottawa, Ontario
Section A-A
Total Stress Analysis - Seismic

Figure No. 20



Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 100 kPa

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

Project No.: OTT-00234493-A0
1154-1208 Old Montreal, Ottawa, Ontario
Section B-B
Effective Stress Analysis

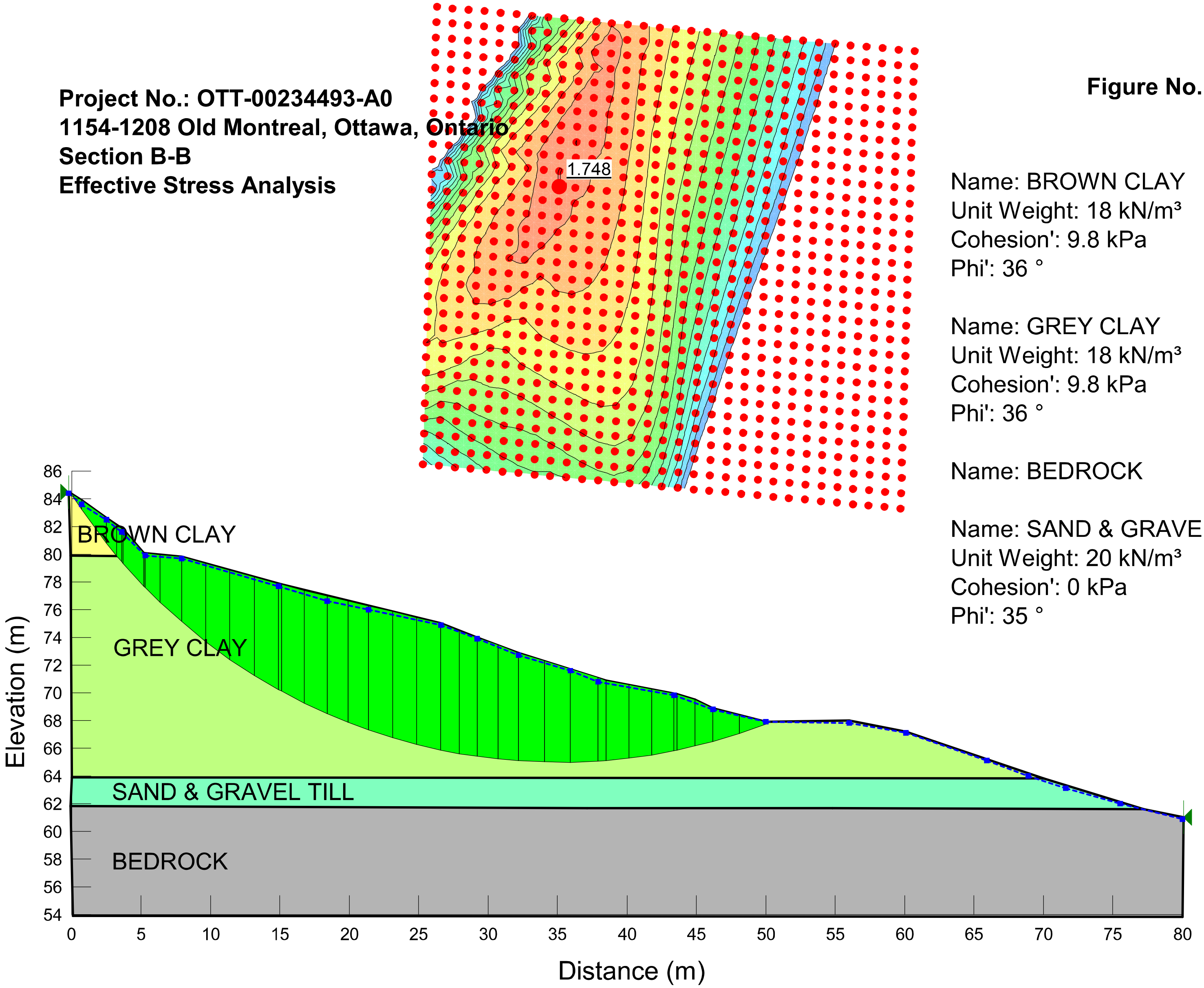
Figure No. 21

Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



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Section B-B
Total Stress Analysis

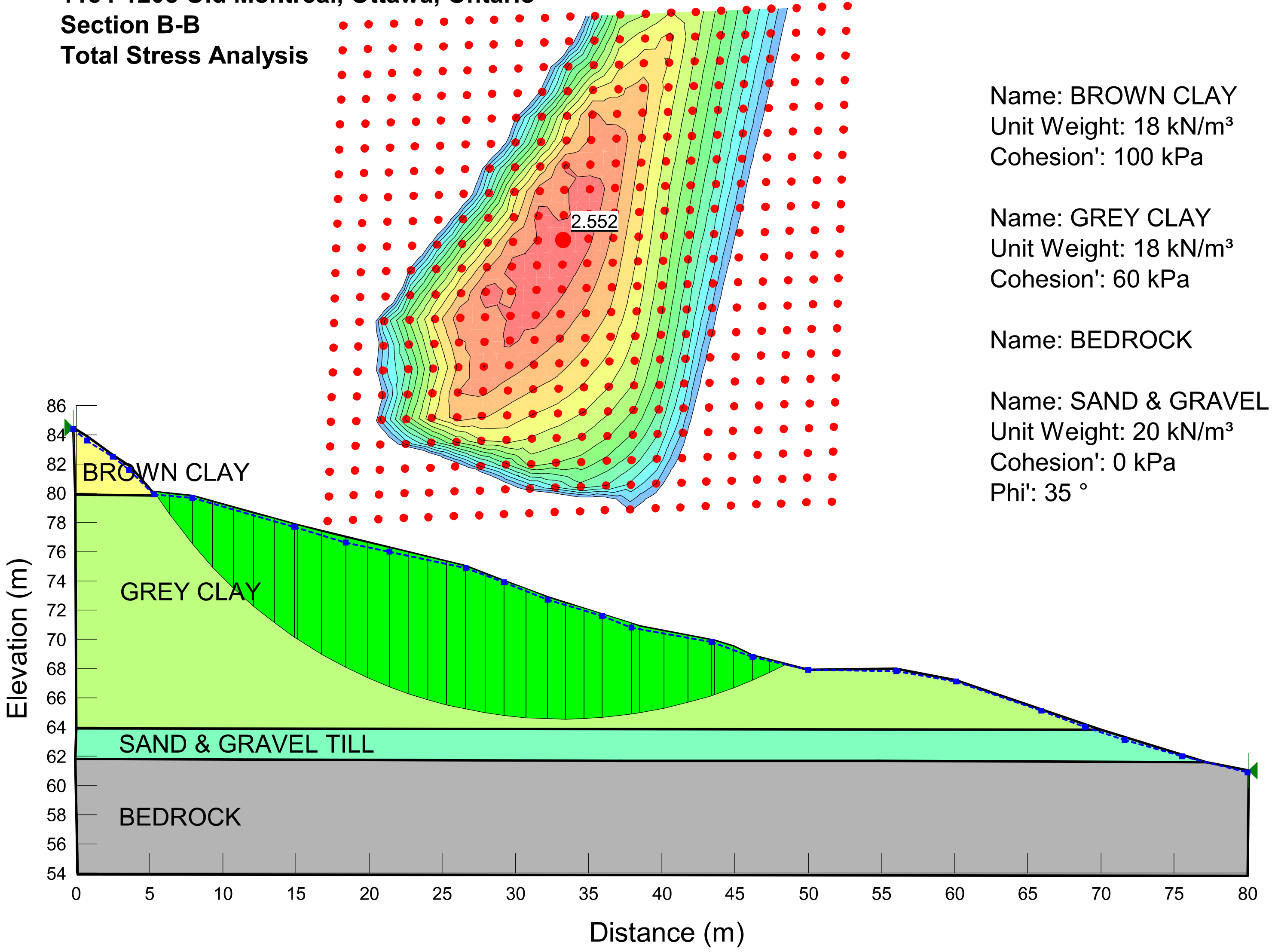
Figure No. 22

Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 100 kPa

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

Name: SAND & GRAVEL TILl
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



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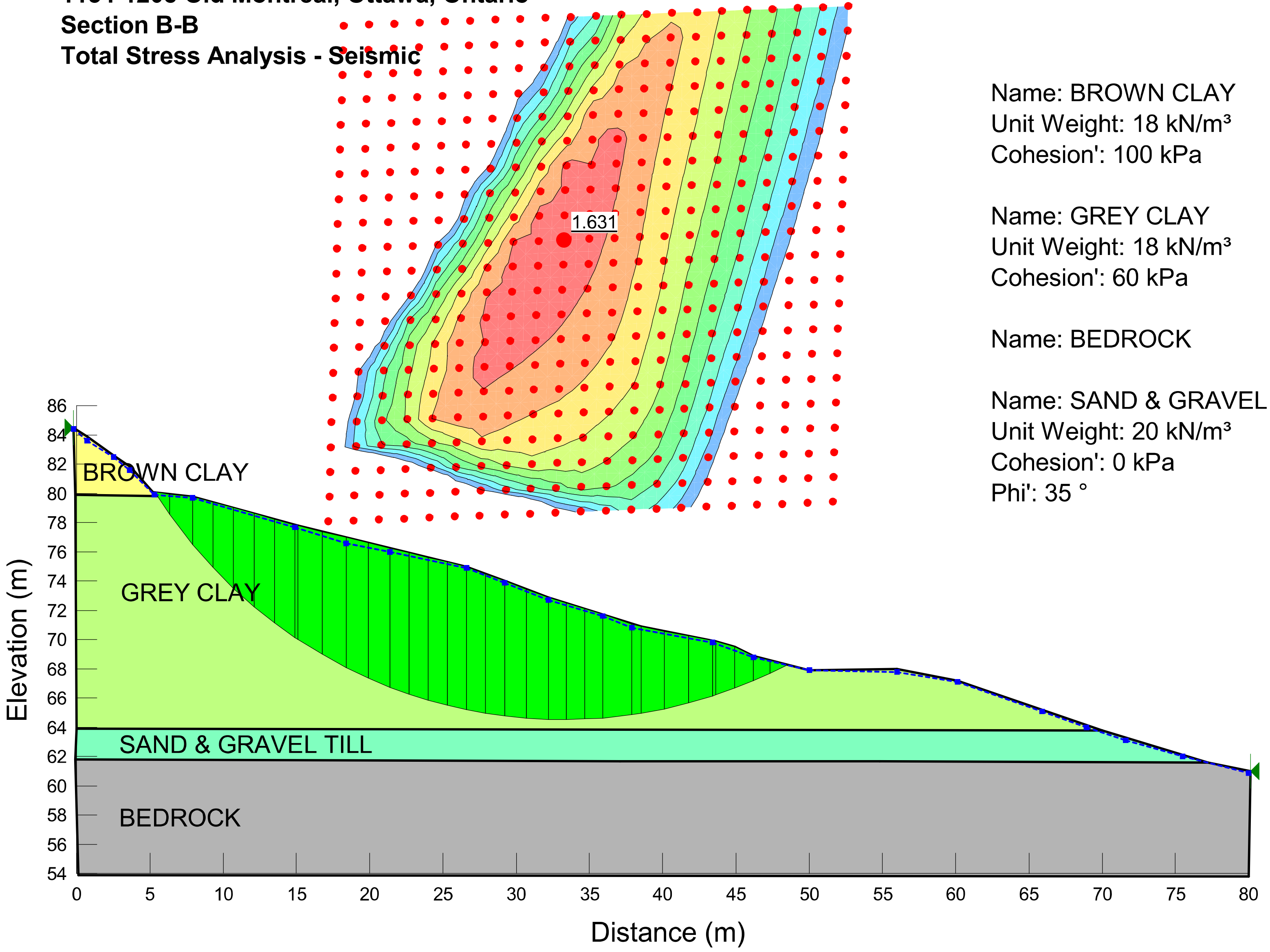
1154-1208 Old Montreal, Ottawa, Ontario
Section B-B
Total Stress Analysis - Seismic

Figure No. 23



Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 100 kPa

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

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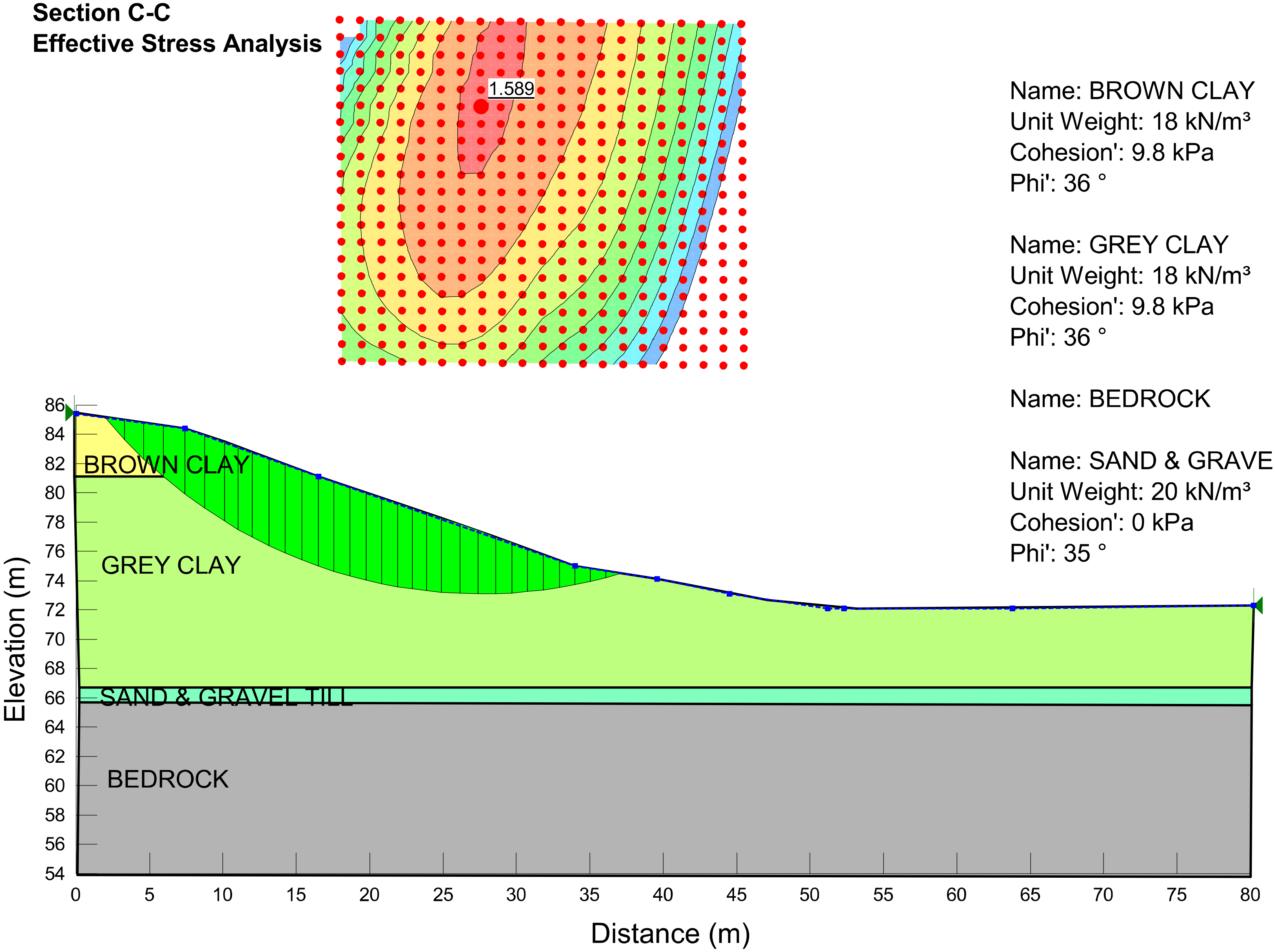
Section C-C
Effective Stress Analysis

Figure No. 24



Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

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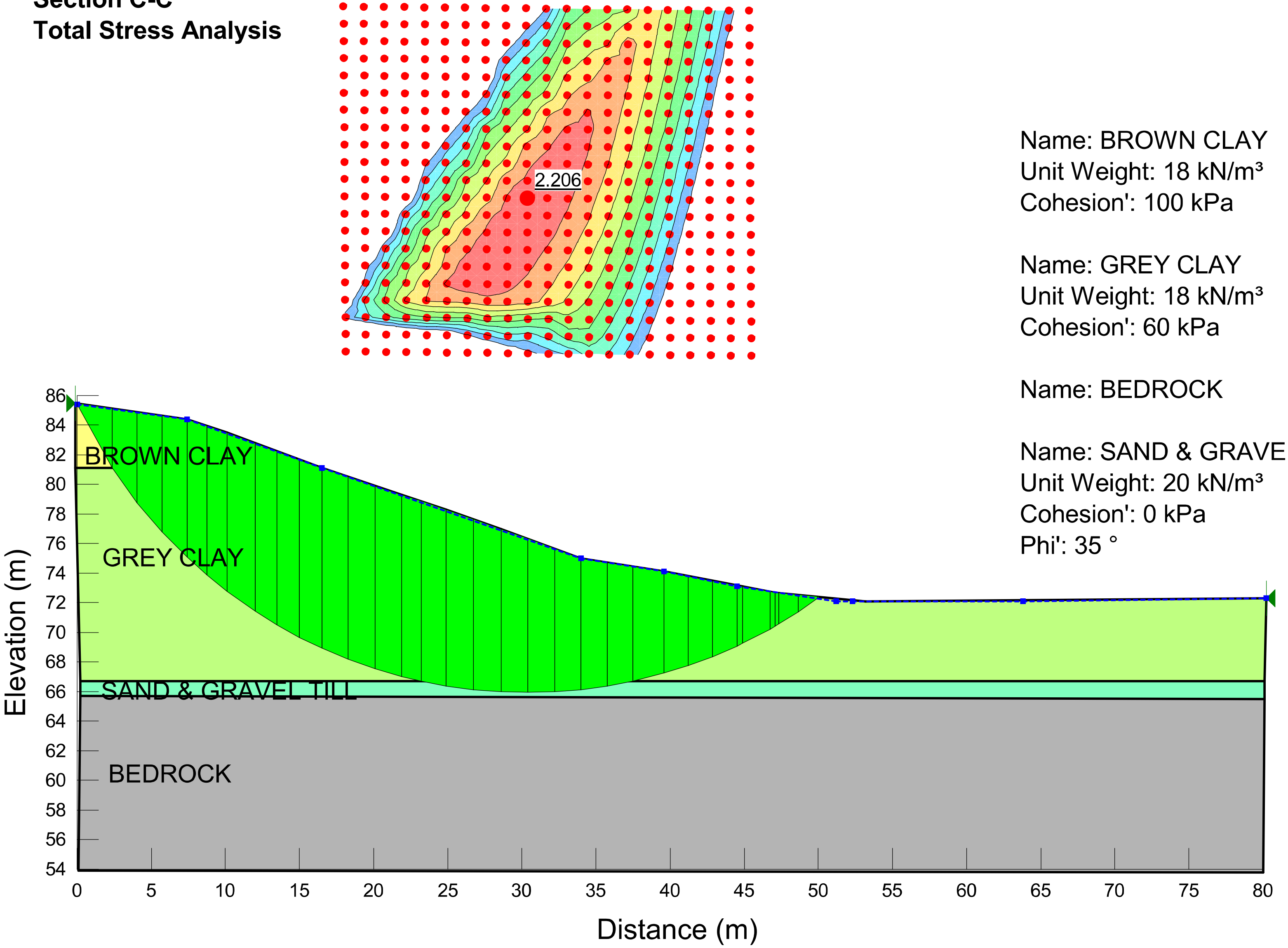
Section C-C
Total Stress Analysis

Figure No. 25



Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 100 kPa

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

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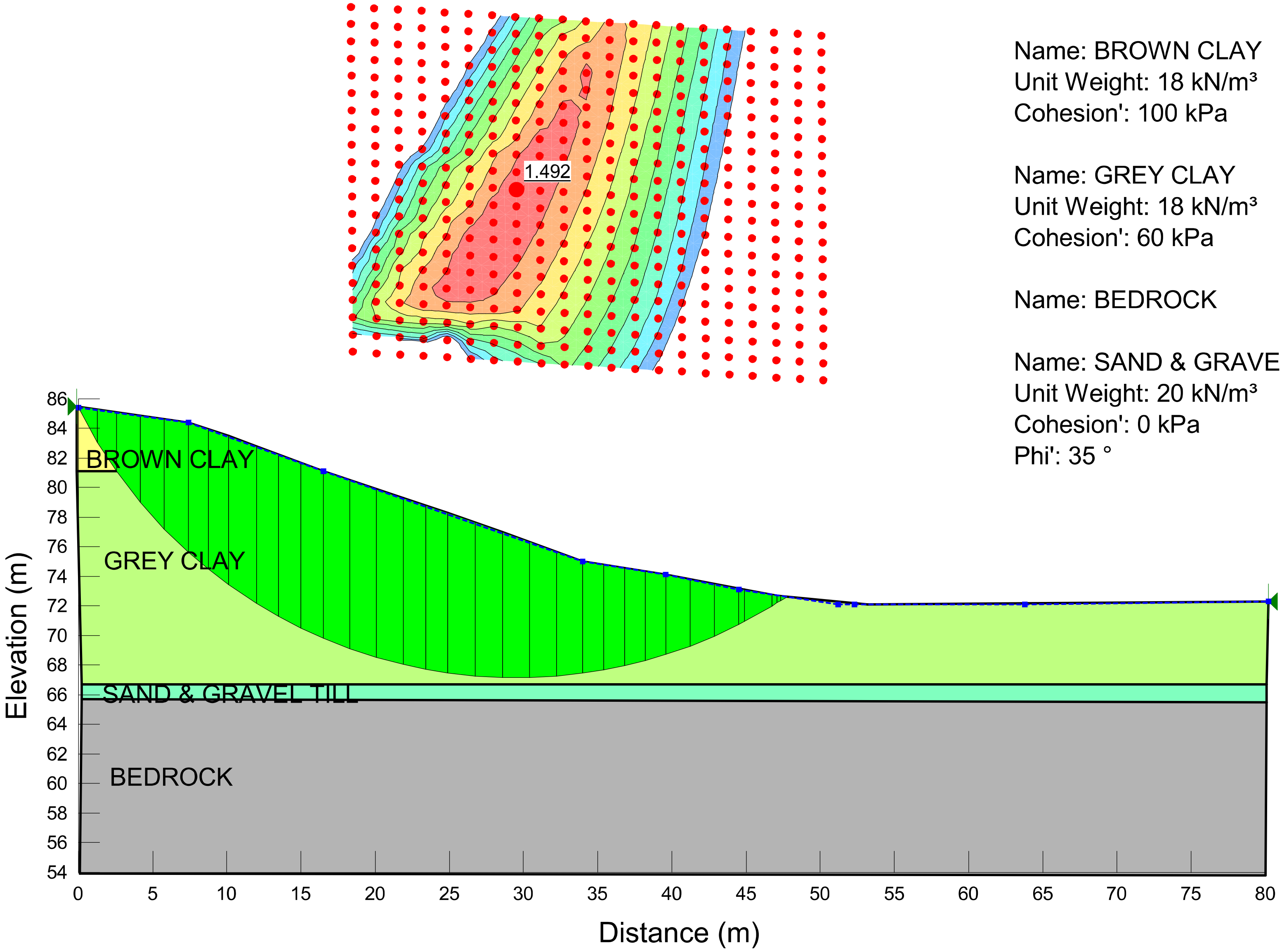
Section C-C
Total Stress Analysis - Seismic

Figure No. 26



Name: BROWN CLAY
Unit Weight: 18 kN/m³
Cohesion': 100 kPa

Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

Name: SAND & GRAVEL TILL
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

Appendix A: Photos of Erosion Along Creek Bank

Photograph No. 1
View of creek west of Pedestrian Bridge (Location 1 on Figure 2)

Photograph No. 2:
View of west end of culvert under the roadway (Location 2 on Figure 2)

Photograph No. 3
View of creek looking west from pathway (Location 3 on Figure 2)

Photograph No. 4
View looking west from east side of pathway near creek (Location 4 on Figure 2)

Photograph No. 5
View of creek looking east from Location 5 on Figure 2

Photograph No. 6:
View along creek bank looking west from north bank at Location 6 on Figure 2

Photograph No. 7
View of creek looking north from south bank of creek at Location 7 on Figure 2

Photograph No. 8
View of toe of slope looking west from Location 8 on Figure 2

Photograph No. 9
View of creek looking south from Location 9 on Figure 2

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