DCR Phoenix Group of Companies

Updated 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
February 12, 2021 (supersede November 7, 2016 report)

DCR Phoenix Group of Companies

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Attention: Mike Boucher, Manager of Planning

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Executive Summary

Two preliminary geotechnical investigations were undertaken at the site of the proposed residential subdivision to be located on the east side of Old Montreal Road at the civic address of 1154 – 1208 in the City of Ottawa, Ontario. The first preliminary investigation was undertaken in 2016 and results reported under EXP Project No. OTT-00234493-A0 dated November 7, 2016. Since that time, the design of the project advanced and another geotechnical investigation was undertaken in 2017/2018 to accommodate the changes made. However, prior to submission of the revised report, we were informed to hold the report in abeyance as it had been decided to revisit the project development plan. Recently, EXP was provided with the revised development plan which also includes additional structures to be located in the central area of the site and requested to submit a preliminary geotechnical investigation report for submission to the City of Ottawa for draft plan approval. This preliminary geotechnical report has been updated to the extent possible based on the currently available information. An additional geotechnical investigation will be undertaken at the site subsequent to the approval of the draft plan to cover areas which were not previously investigated or areas where insufficient geotechnical information is currently available to facilitate the design of the structures according to the most recent development plan. The concerns of the City of Ottawa and Rideau Valley Conservation Authority received to date have been addressed in this revised preliminary geotechnical investigation report. The exception to this is the observed erosion of the toe of the slope in the vicinity of Section B-B by RVCA personnel which would be addressed in the final report. Both the previous preliminary geotechnical investigations were requested by Mike Boucher of Phoenix Homes Group of Companies.

Based on the current development, it is proposed to construct four blocks of four-storey apartment buildings, three blocks of five-storey apartment buildings, two blocks of back to back terrace houses and seven blocks of townhomes.

The topography of the site consists of a topographic high in the central part of the site with a 20 m to 12 m high slope in the westerly direction to Montreal Road. A 12 m to 20 m deep ravine is located along the east boundary of the site. Both the slope to Montreal Road and the ravine slopes are covered with vegetation.

Preliminary grading plan indicates that the grade in the western half of the site will be lowered by a maximum of 7 m to 8 m. In addition, the grade at the site will be raised by a maximum of 2.5 m in the eastern half of the site in some areas. An approximately 1.3 m to 7 m high retaining wall extending from close to north property boundary to south property boundary is to be constructed to accommodate the difference in grade between two halves of the site.

The fieldwork for the two preliminary geotechnical investigations comprised of drilling 12 boreholes to 7 m to 23.3 m depth. The boreholes revealed that the predominant surficial soil at the site is hard to very stiff silty clay which extends to a depth of 3 m to 5.6 m. It is underlain by very stiff to stiff silty clay which extends to refusal at a depth of 13.6 m to 23.3 m in the deep boreholes. The refusal was likely met on shale bedrock. The stabilized groundwater table is estimated to be at a depth of 2.5 m to 5.5 m below existing ground surface, i.e. Elevation 67.9 m to 82.1 m.

The results of the two consolidation tests undertaken at the site, indicate that the silty clay at the site is over consolidated by 520 kPa to 78 kPa. It is considered that a maximum grade raise of 2.5 m is feasible at the site.

Based on the currently available geotechnical information, it is most likely that the proposed apartment buildings, back to back terrace housing, and townhomes can be founded on spread and strip footings set on the silty clay crust or the underlying very stiff to stiff silty clay. A Serviceability Limit State (SLS) bearing pressure and factored geotechnical resistance at Ultimate Limit State of 100 to 200 kPa and 150 to 300 kPa respectively are expected to be available at the site. The exception to this is Blocks 5, 6 and 8 where the geotechnical information is currently lacking. The SLS bearing capacity and factored geotechnical resistance at ULS will be established during the additional investigation.

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

Excavations for construction of the apartment building with two basement levels will extend to a depth of approximately 7 m below the finished floor slabs of the buildings and will extend to below the groundwater table. Excavations for installation of the services will extend to a depth of 3 m to 4 m below the roadways. Excavations for construction of the basements of the buildings will most likely require a shoring support system which can be best determined once the final concept plan and design is completed. Excavations for installation of the services may be undertaken as open cut provided they are cut back at an inclination of 45 degrees above the groundwater table. Below the groundwater table, these excavations are expected to slough and may eventually stabilize at a slope between 2H:1V and 3H:1V. If open cut excavations are not feasible due to space restrictions or if cutting back the excavation would result in a very large cut, the excavations may be undertaken within the confines of a trench box for site services and within a pre-engineered shoring system designed and installed in accordance with OHSA 213/91 for buildings to be located on site. Seepage of surface and subsurface water into the excavations should be anticipated. However, it should be possible to remove this water by pumping from sumps located at low points. A hydrogeological study would be required at the site in order to establish the quantity of water to be pumped, assess type of permit required for taking water, and the best method of groundwater control in the areas of deep excavation.

The pavement structures of the access roads and driveways are given on Table V. 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. This designation will be reviewed subsequent to an additional investigation. Currently available information indicates that the on-site soils are not 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. City of Ottawa 2017 guidelines regarding planting of small and medium size trees in subdivisions should be followed.

A slope stability analysis of the slope to the ravine located along the east 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 and D-D. A reiterative analysis of Section A-A was undertaken and gave a geotechnical setback of 24 m for factor of safety of 1.5. A reiterative analysis of Section D-D was performed and gave a geotechnical setback of 19 m for a factor of safety of 1.1 for total strength analysis with seismic loading. 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 east slope for Sections B and C and 35 m and 30 m respectively for Sections A-A and D-D from the crest of the east slope. A slope is also located in the west half of the site and slopes to the west property boundary at Old Montreal Road. This slope is at an inclination of 4.2H:1V or flatter. Based on the currently proposed development plan, most of this slope will be excavated during site re-grading. The stability of this slope will be reviewed during the final geotechnical investigation. The limit of hazardous lands is shown on Figure 2. No development should take place beyond the limit of hazardous land.

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

As indicated above, an additional investigation is required to address areas where there is a gap of geotechnical data and in order to address the comments received from the RVCA and the City of Ottawa.

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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 and the results of the investigation reported under Project OTT-00234493-A0 dated November 7, 2016. Since that time, the design of the proposed development had advanced and therefore an additional geotechnical investigation was undertaken on the site in 2017/2018.

The report is based on the results of these two investigations and incorporates the currently proposed development plan. The proposed development would consist of a residential subdivision with associated roadways and utilities.

Prior to reporting results of the 2017/2018 investigation, the geotechnical investigation report was temporarily shelved as EXP was advised that it has been decided to make further changes to the design. Changes have now been made and EXP was provided with the concept plan as well as the grading plan and requested to submit a preliminary geotechnical investigation report which reflects the current concept plan. Therefore, this report supersedes the November 7, 2016 report. It is noted that the most current plan also includes Blocks 5, 6 and 8 which are located in an area which could not be investigated previously as it was occupied by residences. Subsequent to approval of the concept plan, a more detailed geotechnical investigation would be undertaken at the site and recommendations provided from a geotechnical perspective.

The site location is shown on Figure 1. This work was authorized by Mike Boucher of Phoenix Homes Group of Companies.

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 installation of utilities on the site;

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

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

j) Comment on subsurface concrete requirements; and,

k) Assess the stability of the slopes of the valley located on the east 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 in two stages. The first stage was completed 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 second and additional geotechnical investigation was undertaken between February 5 and 7, 2018 and comprised of extending Borehole 7 of the preliminary investigation to 13.6 m depth and drilling of five additional boreholes (Borehole Nos. 8 to 12 inclusive) to 7 m to 13.1 m depth. The locations of all the boreholes are shown on the Site Plan, Figure 2.

The fieldwork was undertaken with track-mounted drill rigs 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, 3, 9, 10 and 12. 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, 7 and 8 to 12 inclusive. 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 two previous investigations, the site was used for residential and agricultural purposes. The surrounding properties are mostly residential and agricultural. The site is irregular and covers a total area of 14.6 hectares (36 acres).

The topography of the site consists of a topographic high in the central part of the site with a steep slope downwards to the west to Old Montreal Road. The crest of this slope is at Elevation 85.0 m to 82.0 m whereas its toe is at Elevation 71.0 m to 73.0 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 covered with vegetation.

A slope is located on the east side of the site to a deep ravine. The crest of this 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.

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

4 Project Description

Current and updated project concept plan calls for the development of the site with 4 blocks of 4-storey residential buildings containing 46 to 48 units, three blocks of 5-storey residential buildings containing 54 to 90 units, 2 blocks of back to back terrace residences, and 7 blocks of townhomes. Due to the large variation in the ground surface elevations throughout the site, extensive re-grading will be required at the site. Preliminary site grading plans indicate that approximately western half of the site (from Montreal Road to the proposed north-south retaining wall) is to be cut by up to 8 m depth approximately. East of the proposed retaining wall, the maximum cut and fill are expected to be 2 m to 3 m.

Based on the preliminary information received from IBI Group, the following founding levels of the structures were estimated as listed below.

Block Number	Structure Description	Estimated Underside of Footing Elevation-USF (m)
Block 1	4-storey apartment	72.0
Block 2	4-storey apartment	71.50
Block 3	4-storey apartment	70.60
Block 4	4-storey apartment	70.50
Block 5	4-storey apartment	71.30
Block 6	5-storey apartment	74.75
Block 8	5-storey apartment	79.90
Block 10	Back to back terraces	85.15
Block 11	Back to back terraces	84.66
Block 12 to 18	Townhomes	82.3 to 84.6

As part of the site regrading, a north-south retaining wall is to be constructed across the site. The finished grade of this wall behind Block 5 will be at Elev. 78.50 in front of the wall and up to Elev. 85.70 behind the wall. The grade of the retaining wall located behind Block 6 will be at Elev. 81.90 in front of the wall and at Elev. 83.20 m at the top of the wall. The type of wall is not known at this stage and will be established as part of the detailed design.

5 Subsurface Soil and Groundwater Description

A detailed description of the subsurface soil and groundwater conditions established from the boreholes are given on the attached borehole logs, Figure Nos. 3 to 15. 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.

5.1 Topsoil

A 25 mm to 200 mm thick topsoil layer was contacted at the location of all the boreholes except Borehole Nos. 7, 11 and 12.

5.2 Silty Sand / Gravelly Sand

The topsoil in Boreholes 1, 2, 4 and 8 is underlain by a thin layer of silty sand to gravelly sand which extends to 0.5 m to 0.7 m depth (Elevation 82.3 m to 84.5 m). The layer is loose with N values of 6 to 7. Its moisture content is 16 to 30 percent.

5.3 Sand Fill

The surficial soil in the vicinity of Borehole No. 11 is sand fill which also underlies the topsoil in Borehole Nos. 3, 6, and 7. It extends to 0.3 m to 2.3 m depth (Elev. 72.6 to 85.9 m). The natural moisture content of the fil varies from 21 percent to 30 percent.

5.4 Silty Clay Crust

The topsoil in Borehole No. 5, 9 and 10, the fill in Borehole Nos. 3, 6, 7, and 11, and the silty sand in Borehole Nos. 1, 2, 4 and 8 are underlain by desiccated silty clay crust, which extends to 3 m to 5.6 m depth (Elev. 66.3 m to 83.6 m). The natural moisture content and unit weight of the crust vary from 30 to 63 percent and 17.2 to 19.1 kN/m^3 respectively.

The crust is very stiff to hard as indicated by its undrained shear strength, which varies from 106 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 16).

The liquid and plastic limits of the clay were established as 64.5 to 67.5 percent and 26.1 to 26.6 percent respectively, indicating that the crust is inorganic clay of high plasticity.

5.5 Grey Silty Clay

The silty clay crust is underlain by grey silty clay which extends to the entire depth investigated in Borehole Nos. 2, 4, 6 and 8 to 12, i.e. 7.0 m to 13.1 m depth (Elevation 63.4 to 78.9 m) and to a depth of 11.2 to 20.9 m in Borehole Nos. 1, 3 and 7 (Elevation 73.9 m to 63.4 m).

The natural moisture content and unit weight of the silty clay varies from 48 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. 17 and 18). The liquid and plastic limits of the silty clay vary from 52.7 to 68.6 percent and 24.6 to 29.5 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. 19 and 20 and have been summarized as Table II. A review of Figure 19 indicates that the desiccated silty clay crust is over-consolidated by 520 kPa approximately and has a recompression (c_{cr}) and compression index (c_c) of 0.153 and 1.07 respectively. Figure No, 20 indicated that the e grey silty clay is over-consolidated by 78 kPa approximately. Its recompression and compression index are 0.11 and 1.36 respectively.

5.6 Silty Sand Till

The grey silty clay in Borehole Nos. 1, 3 and 7 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 13.6 m depth in Borehole No. 7 (Elev. 71.5 m), 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, 5 and 7 were likely met on bedrock but not confirmed by core drilling techniques. The silty sand till is compact to very dense as indicated by its standard penetration resistance values (N values) which vary from 11 to 64. The natural moisture content of the till is 4 to 10 percent. A grain-size analysis performed on a sample of the till from Borehole No. 1 yielded a soil composition of 3 percent clay, 12 percent silt, 46 percent sand and 39 percent gravel (Figure 21).

5.7 Bedrock

As indicated above, refusal to dynamic cone penetration test or to augering was met in three of the boreholes at depths of 13.6 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.

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Table I: Summary of Subsurface Conditions

Soil Type	MC (%)	γ (kN/m³)	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	30-63	17.2 - 19.1	2 - 23	106 - >250	38	26.1	64.5	72	26	2	-
Clay	48-74	15.5 - 17.3	HW - 6	50 -220	54-70	24.6-28.3	52.7-62.7	61-69	30-37	1-2	-
Silty Sand Till	4-10	-	11 - 64	-	-	-	-	3	12	46	39
Mc = Natural Moisture Content, Cu= Undrained Shear Strength, HW = Hammer Weight, PL= Plastic Limit, LL= Liquid Limit											

Table II: 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

5.8 Groundwater

Water level observations were made in open boreholes during drilling and in the standpipes/monitoring wells installed in Borehole Nos. 1, 3 and 7 to 12 subsequent to completion of the drilling. The observations made have been tabulated on Table III.

Table III: 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 February 15, 2018	1.5 1.3	84.3 84.5
3	August 17, 2016	September 10, 2016 February 15, 2018	2.5 1.5	81.8 82.8
7	August 16, 2016	September 10, 2016	1.3	83.8

Table III: Groundwater Observations in Boreholes				
Borehole No.	**Date Drilled**	**Observation Date**	**Groundwater Depth (m)**	**Elevation to Groundwater Table (m)**
8	February 7, 2018	February 15, 2018	0.8	85.1
9	February 6, 2018	February 15, 2018	1.0	81.2
10	February 6, 2018	February 15, 2018	0.7	76.4
11	February 5, 2018	February 15, 2018	1.1	71.8
12	February 7, 2018	February 15, 2018	1.2	70.7

A review of Table III indicates that the perched water table in Boreholes 1, 3, 7 and 8 to 12 is at a depth of 0.7 m to 1.5 m below the existing ground surface, i.e. Elev. 70.7 m to 85.1 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 2.5 m to 5.5 m below the existing ground surface, i.e. Elev. 82.1 m to 67.9 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.

6 Site Re-grading

The investigation has revealed the site to be underlain by a deep deposit of clay (in the order of 11.2 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. Preliminary grading plan indicates that 2 to 2.5 m of fill will be placed in the eastern half of the site. The silty clay in this area has an upper desiccated crust which varies in thickness from 3.5 m to 5.0 m depth. In addition, the structures to be located in this area will be lightly loaded. It is therefore considered that the grade at the site in this area may be raised by up to 2.5 m.

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

7 Foundation Considerations

The investigation has revealed that the geotechnical conditions at the site are suitable for construction of the proposed structures with one or two levels of basement on spread and strip footing foundations. The site contains surficial desiccated very stiff to hard silty clay, which extends to 3.0 m to 5.6 m depth. It is underlain by very stiff to stiff silty clay. As indicated previously, lowest level footings of the residences will be founded between Elev. 70.5 m and 87.5 m. The SLS and ULS bearing pressures available at the proposed founding levels have been listed on Table IV.

Table IV: SLS and ULS Bearing Pressure of Soil at Founding Level

Block No.	Closest Borehole No.	Lowest Footing Founding Elevation (m)	SLS Bearing Pressure (kPa)	Factored ULS Bearing Pressure (kPa)	Proposed Structure
1	10	72.0	150	225	4-storey apartment
2	10 & 11	71.5	100	150	4-storey apartment
3	11	70.6	200	300	4-storey apartment
4	12	70.5	200	300	4-storey apartment
5	5	71.3	*	*	5-storey apartment
6	9	74.75	*	*	5-storey apartment
8	None	79.9	*	*	5-storey apartment
10	8	85.15	150	225	Back to back terrace homes
11	8	84.66	150	225	Back to back terrace homes
12	1	85.3 – 85.6	100	150	Townhomes
13-14	2 & 8	84.3 – 85.65	100	150	Townhomes
15-18	None	83.3 – 84.25	100	150	Townhomes
NOTE:	*Will be established during final geotechnical investigation. However, it is anticipated that spread and strip footings will be feasible for these structures also with bearing pressures similar to the ones listed on the table.				

The footings of the residences will be located at different levels. The higher footings should be located below a line drawn up at 10H:7V from the lower footings to prevent transference of stresses from the higher footings to the lower ones. The lower footings should be constructed before the higher footings to prevent the latter from being undermined during subsequent construction.

The recommended bearing capacities have been obtained by EXP from the boreholes which in some cases are located some distance away from the structures. The bearing capabilities listed are for the preliminary design stage only. Additional boreholes would have to be drilled, evaluated, and the bearing capacities listed above would have to be revised for the final report. 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. Placement of 50 mm concrete mud slab is recommended on the surface of the approved silty clay to prevent disturbance of the founding surface from the elements and from workers.

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.

8 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 engineered 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 22). 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.

9 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:

$$P = K_0 H (q + \tfrac{1}{2} \gamma H)$$

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. The value of the K would be computed in the final report once the design grades and the inclinations of the retained slopes are known.

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 or sloped surface of the backfill against the subsurface walls. These loads should be taken into consideration when designing the subsurface walls. The resultant load should be assumed to act at 0.6 H from the bottom of the wall.

10 Retaining Walls

Preliminary subdivision design indicates that a terraced retaining wall is to be located approximately 6 m east of Block 5 and will extend beyond Block 5 for some distance in the north as well as south directions. The grade in front of the wall will be at Elevation 78.2 m to 78.5 m and at the top of the wall at Elevation 81.50 to Elevation 85.70 m. Therefore, the height of the wall will vary from 3.5 m to 7.2 m.

Another retaining wall is to be located east of Block 6. The grade in front of this wall is to be set at Elevation 81.9 m and the top of the wall be subjected to lateral static earth as well as lateral dynamic earth forces during a seismic event.

Seismic loading will result in an increase in active lateral earth pressure and a decrease in passive lateral earth pressure on the wall. The seismic lateral earth pressure coefficient given below has been derived based on a design zonal acceleration ratio of 0.32 in the horizontal direction and 0.21 in the vertical direction applicable to Ottawa.

The dynamic pressure distribution is an inverted triangle with maximum pressure at the top of the wall and a minimum at the bottom of the wall. Therefore, the resultant of earthquake pressure on the retaining wall is assumed to be applied at a height of 0.6 H above the base of the wall where H is the height the wall. The total active pressure distribution can be separated into static component and dynamic components and may be determined as follows (Mononobe and Matsuo, 1929):

$$\sigma_{AE}(z) = k_a \gamma z + (K_{AE} - k_a)\gamma(H - z)$$

Where $\sigma_{AE}(z)$ = the total combined active earth pressure (dynamic and static), (kPa);

z = depth below the top of the retaining wall, m;

K_a = static active earth pressure coefficient;

K_{AE} = combined (static and dynamic) active earth pressure coefficient;

γ = unit weight of the free draining backfill soil (KN/m^3); and,

H = Total height of the wall (m).

The total passive pressure in front of the wall can be similarly separated into static and dynamic components as follows:

$$\sigma_{PE} = k_p \gamma z + (K_{PE} - k_p)\gamma(h - z)$$

Where σ_{PE} = the total combined passive earth pressure (dynamic and static), (kPa);

z = depth below the ground surface in front of the wall;

K_p = static passive earth pressure coefficient;

K_{PE} = combined (static and dynamic) passive earth pressure coefficient;

γ = unit weight of free draining backfill soil (KN/m^3); and,

h = depth of embedment of the wall (m).

The above earth pressure expressions do not take into account any surcharge applied on the wall or the backfill soil. It also assumes that the backfill against the subsurface walls will be free-draining granular material and drains will be presented at the footing level to prevent building up of hydrostatic pressure against the subsurface walls. The lateral earth pressure parameters of the backfill material are given on Table V. The following assumptions were made during computation of the lateral earth pressure parameters:

i.) The ground surface in front and behind the wall was assumed horizontal;

ii.) The back face of the wall was assumed vertical; and,

iii.) The friction between the wall and the backfill was ignored.

Table V: Lateral Earth Pressure Parameters for Design of Retaining Walls

Soil layer		
Unit Weight of Soil (γ), kN/m^3		22
Angle of Internal Friction (ϕ') (°)		30 °
Coefficient of Earth Pressure at Rest (k_0)		0.5
Active Earth Pressure Coefficient	Static (k_a)	0.33
	Static and dynamic (k_{AE})	0.70
Passive Earth Pressure Coefficient	Static (k_p)	3.0
	Static and dynamic(k_{PE})	2.2

The above expressions assume that a perimeter drainage system together with free draining granular backfill material adjacent to the walls will prevent the buildup of hydrostatic pressure behind the walls. The backfill should be compacted to 95 percent SPMDD. The method of compaction of the backfill is not known. However, it is recommended that a minimum compaction surcharge of 20 kPa should be taken into account when designing the retaining walls.

11 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.

A higher site class may be obtained if a shear wave velocity (MASW) is completed at the site and if required by the structural engineer.

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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.

12 Excavations

Based on the preliminary proposed grades, it appears that the area west of the proposed retaining wall will be cut. The depth of cut will vary from 2.5 m close to the west property boundary, increasing to 7 m approximately at proposed Private Street #1. The cut operation will result in removal of part or all of the desiccated silty clay crust and exposure of the underlying very stiff to stiff silty clay. It is understood that the services will be installed at a depth of 3.5 m to 4 m below the proposed cut/roadway elevations. Excavations for installation of the services will be predominantly in the silty clay. The exception to this is Borehole No. 7 where the excavation will extend below the silty clay into gravelly sand till and likely terminate close to the depth where refusal to augering was met. It is not known whether this refusal was met on bedrock or boulders. However, a 'base-heave' type of failure of the excavation is not anticipated due to the dense nature of the silty sand till. The excavations will extend below the prevailing groundwater table.

Excavations maybe undertaken by conventional heavy equipment capable and must comply with the most recent Occupational Health and Safety Act (OHSA), Ontario Regulations 213/91 (August 1, 1991). Based on the definitions contained in OHSA, the subsurface soils at the site are classified as Type 3 soil.

Excavations in the silty clay above the groundwater table are expected to be stable when cut back at 45 degrees. Excavations in the silty clay below the groundwater table and in the silty sand till will not experience a 'base-heave' type of failure of the excavation. However, sides of the excavation may slough and eventually stabilize at a slope of 2H:1V to 3H:1V. If open-cut excavations are not feasible due to space restrictions, the excavations may be undertaken within the confines of a pre-engineered support system (trench box), which is designed and constructed according to the latest requirements of Occupational Health and Safety Act, Ontario Regulations 213/91 for service trenches and within a engineered shoring support system elsewhere on site. Depending on the depth of the excavation in certain areas of the site for site services, the use of trench boxes as support system may not be feasible, and an engineered shoring support system may be required. This will be best established by contractors bidding on this project.

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. It is also recommended that a 50 mm concrete mud slab should be placed on the surfaces of the approved footings subgrades to prevent their disturbance from the elements and from movement of workers.

Whether a Permit to Take Water will be required or not 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.

Seepage of the 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. In areas of high infiltration or in areas where more permeable soil layers may exist, a higher seepage rate should be anticipated. Therefore, the need of high capacity pumps to keep the excavation dry should not be ignored.

It is anticipated that groundwater will need to be removed from the excavations. It is noteworthy to mention that new legislation came into force in Ontario on March 29, 2016 to regulate groundwater takings for construction dewatering purposes. Prior to March 29, 2016, a Category 2 Permit to Take Water (PTTW) was required from the Ontario Ministry of the Environment and Climate Change (MOECC) for groundwater takings related to construction dewatering, where taking volumes in excess of 50 m3/day, but less than 400 m3/day, and the taking duration was no more than 30 consecutive days. The new legislation replaces the Category 2 PTTW for construction dewatering with a new process under the Environmental Activity and Sector Registry (EASR). The EASR is an on-line registry, which allows persons engaged in prescribed activities, such as water takings, to register with the MOECC instead of applying for a PTTW.

To be eligible for the new EASR process, the construction dewatering taking must be less than 400 m3/day under normal conditions. The water taking can be groundwater, storm water, or a combination of both. It should be noted that the 30-consecutive day limit on the water taking under the old Category 2 PTTW process has been removed in the new EASR process. Also, it should be noted that the EASR process requires two technical studies be prepared by a Qualified Person, prior to any water taking. These studies include a Water Taking Report, which provides assurance that the taking will not cause any unacceptable impacts, and a Discharge Plan, which provides assurance that the discharge will not result in any adverse impacts to the environment. It is noted that if the above conditions cannot be met, a Category 1 PTTW may be required. However, whether a Category I PTTW will be required would depend on construction schedule, method of supporting excavation sides, and depth of excavations, etc. For this purpose, the hydrogeological study will be required. The study would include carefully controlled tests using pumped wells and observation wells which will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems. EXP has qualified persons who can prepare these types of reports, if required. A significant advantage of the new EASR process over the former Category 2 PTTW process, is that the groundwater taking may begin immediately after completing the on-line registration of the taking and paying the applicable fee, assuming the accompanying technical studies have been completed. The former PTTW process typically took more than 90 days, which had the potential to impact construction schedules.

12.1 Excess Soil Management

Ontario Regulation 406/19 made under the Environmental Protection Act (November 28, 2019) is scheduled to be implemented on January 1, 2021. The new regulation will dictate the testing protocol that will be required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols will need to be implemented and followed based on the volume of soil to be managed. The testing protocols are specific as to whether the soils are stockpiled or *in situ*. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

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

The material to be excavated during site regrading, construction of the footings and installation of services (after cut and fill operations) is expected to be silty clay. The 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, backyards etc. The silty clay below the crust may be too wet for adequate compaction ad may be used for general grading purposes throughout the site. It is noted that the silty clay is prone to moisture absorption if stockpiled for re-use on site and therefore must be protected from the elements. Samples of these soils must be collected during the earth work to establish their suitability and their best usage throughout the site.

Any fill that has to be imported to backfill footing trenches, service trenches, inside the buildings, and against subsurface walls should conform to OPSS 1010 for Granular B, Type II. Any fill that has to be imported to backfill service trenches outside the buildings should preferably conform to OPSS 1010 for select subgrade material. If granular fill is used to backfill service trenches, clay dykes would be required to prevent lowering of the groundwater table on the site.

Any fill required for backfilling purposes in the interior and exterior of the buildings for trench backfill that required to be imported and should conform to the following specifications:

- Engineered Fill under footings - OPSS 1010 Granular B Type II – Compacted to 100 percent of the SPMDD;
- Engineered Fill under the floor slab - OPSS 1010 Granular B Type II – Compacted to 98 percent of the SPMDD;
- Backfill material for footing trenches and against foundation walls located outside the building – OPSS 1010 Granular B Type II - Compacted to 95 percent of the SPMDD;
- Trench backfill and subgrade fill in parking area and access roads – OPSS 1010 Select Subgrade Material (SSM) OR on-site dry and compactible material - Compacted to 95 percent of the SPMDD; and,
- Landscaped area, clean fill free of organic and deleterious material placed in 300 mm thick lifts and each lift to compacted to 92 percent of the SPMDD.

To minimize differential settlement of the pavement over service trenches and adjacent areas, the trench backfill material within the frost zone should match the existing material along the trench walls. This will minimize the differential frost heaving of the subgrade soil. Otherwise, frost tapers may be required.

14 Pipe Bedding Requirement

It is recommended that the bedding for the underground services including material specifications, thickness of cover material and compaction requirements conform to City of Ottawa requirements and/or Ontario Provincial Standard Specification and Drawings (OPSS and OPSD). A minimum of 300 mm of OPSS 1010 is recommended for use as a granular bedding on this project and should be placed and compacted to 98 percent of the SPMDD.

Due to the some services will be installed in silty clay below the prevailing groundwater table, it is recommended the pipe bedding in theses areas should consist of 300 mm thick OPSS 1010 Granular B Type II sub-bedding material overlain by 150 mm thick OPSS 1010 Granular A bedding material. The bedding materials should be compacted to at least 98 percent SPMDD.

In areas of high infiltration and as a trench base stabilization techniques, such as removal of loose/soft material, placement of crushed stone sub-bedding (Granular B Type II), completely wrapped in a non-woven geotextile, may also be used if trench base disturbance becomes a problem in wet or soft areas.

If the backfill for the service trenches will consist of granular fill, clay seals should be installed in the service trenches at select intervals as per City of Ottawa Drawing No. S8. The seals should be 1 m wide, extend over the entire trench width and from the bottom of the trench to the underside of the pavement structure. The clay should be compacted to 95 percent SPMDD. The purpose of the clay seals is to prevent the permanent lowering of the groundwater level.

15 Subdivision Roadway

The subgrade at the site will be silty clay or engineered fill. Pavement structure thicknesses required for the access roads and parking areas set on silty clay subgrade were computed and are shown on Table VI. 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 VI: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Driveways	Parking Areas	Access Roads and Fire Route
Asphaltic Concrete (PG 58-34)	92 to 97 % MRD	50 mm HL3	65 mm – SP12.5	50 mm – SP12.5 60 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. Only dry and compactible on-site excavated soil should be used to backfill 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.

16 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 VII.

Table VII: 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.

17 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 2005 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 VIII). 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 VIII: Tree Planting

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

The above policy still applies in cases where a site contains SMC soils with plasticity index greater than 40 percent. For soils containing SMC soils with plasticity index lower than 40 percent, the above policy was revised in 2017 to provide guidelines for planting of small and medium sized trees.

The City of Ottawa 2017 guidelines state that for planting street trees in the road right-of-way where Sensitive Marine Clays (SMC) soils have been identified, the tree to foundation setbacks may be reduced to 4.5 m for small (mature tree height up to 7.5 m) and medium size trees (mature tree height 7.5 m to 14 m) provided all of the following six conditions are met:

(1) The modified plasticity index of the soil between the underside of footing (USF) and depth of 3.5 m generally does not exceed 40%. Clay soils that exceed the 40% plasticity index are considered to have high potential for soil volume change. For these soils, the setbacks and tree planting restrictions remain unchanged from 2005 Clay Soils Policy (i.e. tree setback must be equal to the mature height of the tree, except for small trees where the setback should be 7.5 m).

(2) The USF is 2.1 m or greater below the lowest finished grade. This footing level applies for footings within 10 m of the tree as measured from the centre of the tree trunk and verified by means of the grading plan.

(3) A small size tree and medium size tree must be provided with a minimum of 25 m^3 and 30 m^3 of available soil volume respectively as determined by a landscape architect. The developer will ensure that the soil is generally uncompacted when backfilling in the street tree planting locations.

(4) The tree must be small to medium size as confirmed by a landscape architect in the Landscape Plan.

(5) The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15 m bars in the foundation wall) to provide ductility.

(6) Grading surrounding the tree must promote draining to the tree root zone (in such a manner as not to be detrimental to the tree), as noted in the subdivision Grading Plan.

In addition, the policy has specified the following procedural changes:

(1) One Atterberg Limit and one moisture content test on 150 m spacing and a grain size test for every four boreholes.

(2) One shrinkage test per subdivision.

(3) The USF will be verified by means of the Grading Plan.

(4) Reinforcement of the foundation walls will be confirmed by the geotechnical report.

(5) The geotechnical report must contain a separate section on SMC soils, a signed letter and corresponding map that confirms the locations of the low/medium and/or high sensitivity clay soils as determined by the plasticity tests.

(6) Landscape architect will develop a Landscape Plan that is consistent with recommendations provided in the geotechnical report.

(7) The landscape architect will provide a signed letter indicating that the trees are of small or medium size and have been planted with appropriate soil volumes as per guidelines.

(8) In addition, the city specifies the minimum number of trees that must be provided in a plan of subdivision will be one tree per lot, and two trees per corner lot with exceptions noted in the guidelines.

For complete details, concerned parties should refer to the City of Ottawa publication "Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines."

It is noted that some geotechnical aspects of the policy related to planting of the trees in subdivisions can be addressed in the geotechnical investigation report. Other aspects would have to be addressed once the site grading, location of streets and boulevards have been finalized. All the geotechnical aspects that can be addressed in the geotechnical report will be included in the final geotechnical investigation report.

18 Slope Stability

18.1 Slope Stability Analysis of East Slope (Ravine Slope)

The stability of the existing slope to the ravine located at the east property boundary 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 slope and to determine the required set back of the proposed structures from the crest of the slope. 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 IX.

Table IX: 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
D-D	83.0	64.0	19.0	3.42H: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 IX).
2.) The soil stratigraphy for the various cross-sections is shown on Figure Nos. 24 to 35 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 24 to 35 inclusive and have also been tabulated on Table X.

Table X: Results of Slope Stability Analysis

Section	Condition Analysed	Factor of Safety	Required Geotechnical Set Back	Figure No.
A-A	Effective stress analysis and setback determination	1.50	24 m	24
	Total stress analysis	1.94	-	25
	Total stress analysis with seismic loading	1.20	-	26
B-B	Effective stress analysis	1.75	0	27
	Total stress analysis	2.55	-	28
	Total stress analysis with seismic loading	1.63	-	29
C-C	Effective stress analysis	1.59	0	30
	Total stress analysis	2.21	-	31
	Total stress analysis with seismic loading	1.49	-	32

Table X: Results of Slope Stability Analysis				
Section	**Condition Analysed**	**Factor of Safety**	**Required Geotechnical Set Back**	**Figure No.**
D-D	Effective stress analysis	1.46	0	33
	Total stress analysis	1.64	0	34
	Total stress analysis with seismic loading and setback determination	1.10	19 m	35

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. The factor of safety of the slope for seismic loading conditions was greater than 1.1 for all sections of the slope except Section D-D. A reiterative slope stability analysis was undertaken for this cross-section to determine the setback required for factor of safety of 1.1. A geotechnical setback of 19 m was computed for this section.

It is noted that a slope is also located along the west 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 currently proposed development plan, most of this slope will be excavated to lower the grade at the site. The stability of this slope, if required, will be assessed as part of the final geotechnical investigation once the site grades have been finalized.

The slopes located at the east and west sides of the site are covered with vegetation and trees. The vegetation and trees provide stability to the slopes and should not be disturbed in anyway. The exception to this is the west slope which will be excavated.

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 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 as it prevents erosion. Additional vegetation should be planted on the slopes wherever necessary.

18.2 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.

18.3 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. 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 γ 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.

[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.*

19 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/ riverbank 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 west slope of the creek located east of the site was 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 east slope (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 and Section D-D where it is 30 m.

It is noted that RVCA has indicated that a recent examination of the slope revealed some surficial slope erosion / stability issues in the vicinity of Section B-B. Prior to preparations of the final geotechnical report, this area will be reviewed to determine if there has been further deterioration since initial excavation of the slope by EXP personnel in 2016 so that appropriate remedial measures can be recommended.

The required limit of hazardous lands for the slope located on the east side of the site has been tabulated on Table XI 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 XI: Limit of Hazardous Lands at Cross-Section Locations

Section	Geotechnical Setback from Crest of Slope (m)	Erosion Set Back (m)	Available Erosion Allowance in Valley Floor (m)	Erosion Access Allowance (m)	Total Setback from Crest of 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
D-D	19	5	0	6	30

20 General Comments

The comments provided in this report are preliminary and subject to revision on completion of a final geotechnical investigation. They 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 preliminary design purposes.

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

exp.

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Canada

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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

Filename: r:\230000\234000\234493 - 1154-1208 old montreal road\drawings 2021\1154-1208 old montreal road.dwg
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Last Plotted: 2/9/2021 3:36:17 PM Plotted by: NugentM
Pen Table:: omm pen setting july 20.ctb
References: 385039a.dwg; 385039b.dwg; 385040D.DWG; MTL RD_SP-1_2021_01_29_Rev16.dwg

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 TOPO PLAN OBTAINED FROM CITY OF OTTAWA. PROPOSED SUBDIVISION PLAN OBTAINED FROM M. DAVID BLAKELY ARCHITECT INC. , PRELIMINARY DEVELOPMENT PLAN, SP-1, REV. 16

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

SCALE

HORZ 1:750

0 5m 10m 30m
HORIZONTAL 1:750

LEGEND

BH1 EL. 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 PLAN

PROJ NO.: OTT-00234493-A0
SURVEY
DATE: FEB. 9, 2021
DRAWING NO.: FIG 2

Notes On Sample Descriptions

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

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

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

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

Log of Borehole BH-01

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

Date Drilled: 'August 15, 2016

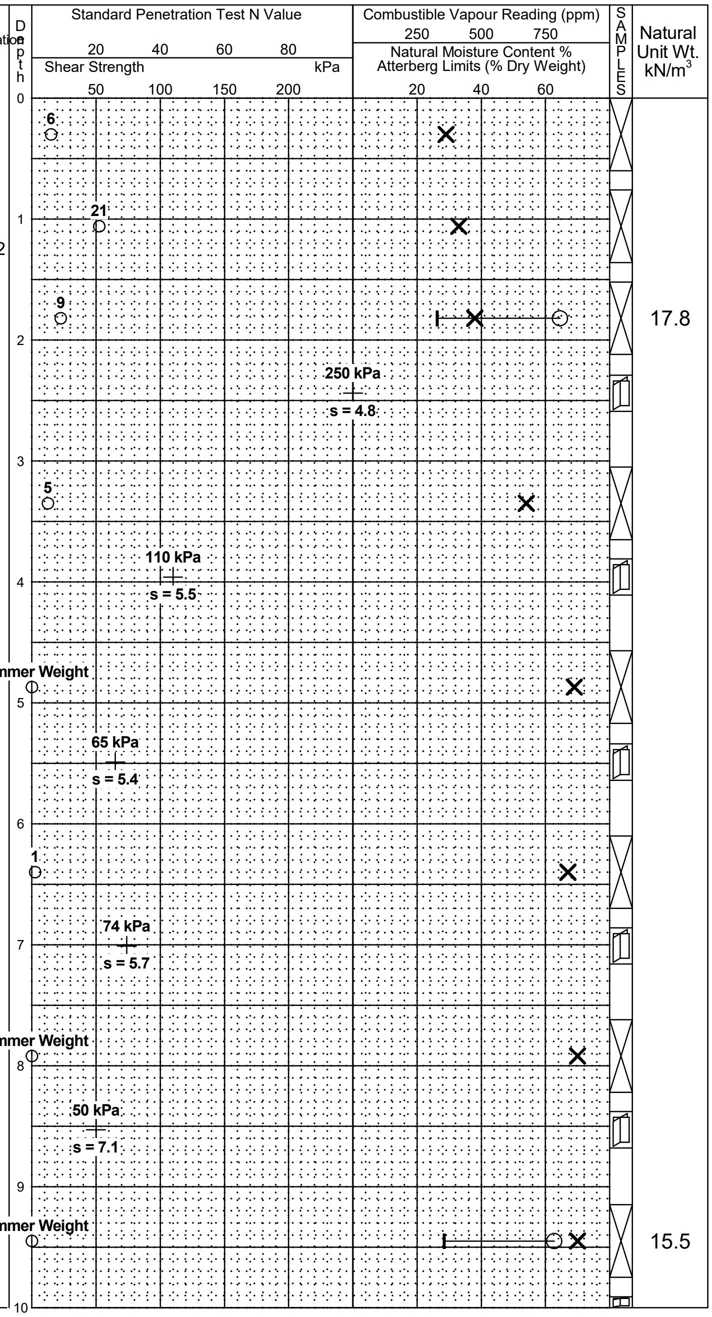
Drill Type:

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		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			85.82	0
		TOPSOIL ~ 220mm	85.6	
		SILTY SAND Fine grained sand, some organics (rootlets), brown to grey, moist, (loose)	85.1	
		SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity, (very stiff to hard)	84.52	1
		SILTY CLAY Grey, moist to wet, high plasticity, (stiff)	81.8	4



Continued Next Page

NOTES:
1. Borehole/Test Pit 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	
Feb 15, 2018	1.3	

CORE DRILLING RECORD

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

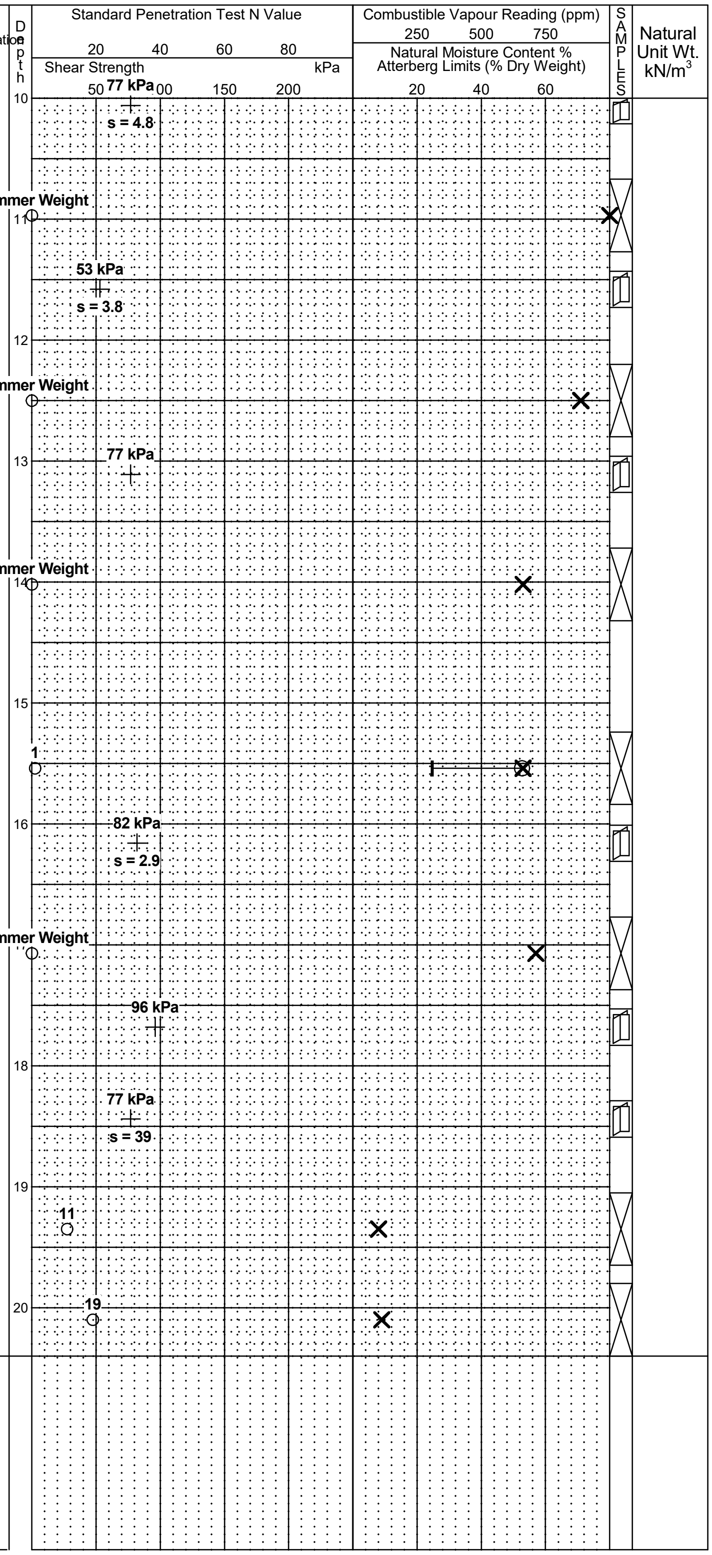
Log of Borehole BH-01

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 3

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			75.82	10
		SILTY CLAY Grey, moist to wet, high plasticity, (stiff) *(continued)*		
			66.9	
		GRAVELLY SAND TILL Some silt, trace clay, grey, wet, (compact)		
			65.4	
		Borehole Terminated at 20.4 m Depth		



NOTES:
1. Borehole/Test Pit 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	
Feb 15, 2018	1.3	

CORE DRILLING RECORD

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

Log of Borehole BH-02

Project No: OTT-00234493-A0

Figure No. 4

Page. 1 of 1

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

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

Date Drilled: 'August 16, 2016

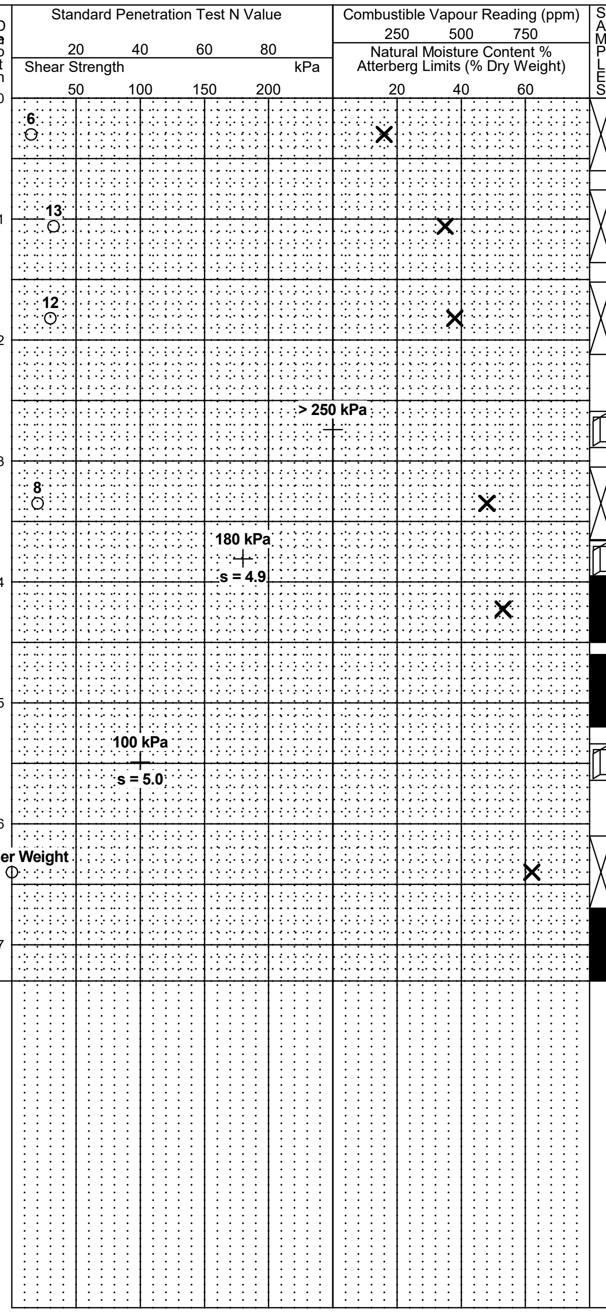
Drill Type:

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		

GWL	SOIL DESCRIPTION	Geodetic Elevation m	Depth (m)	Natural Unit Wt. kN/m³
		85.4	0	
	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)		1	17.9
			2	17.7
	SILTY CLAY Grey, moist to wet, high plasticity, (stiff)	81.4	4	
	Borehole Terminated at 7.3 m Depth	78.1		



NOTES:
1. Borehole/Test Pit 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-03

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

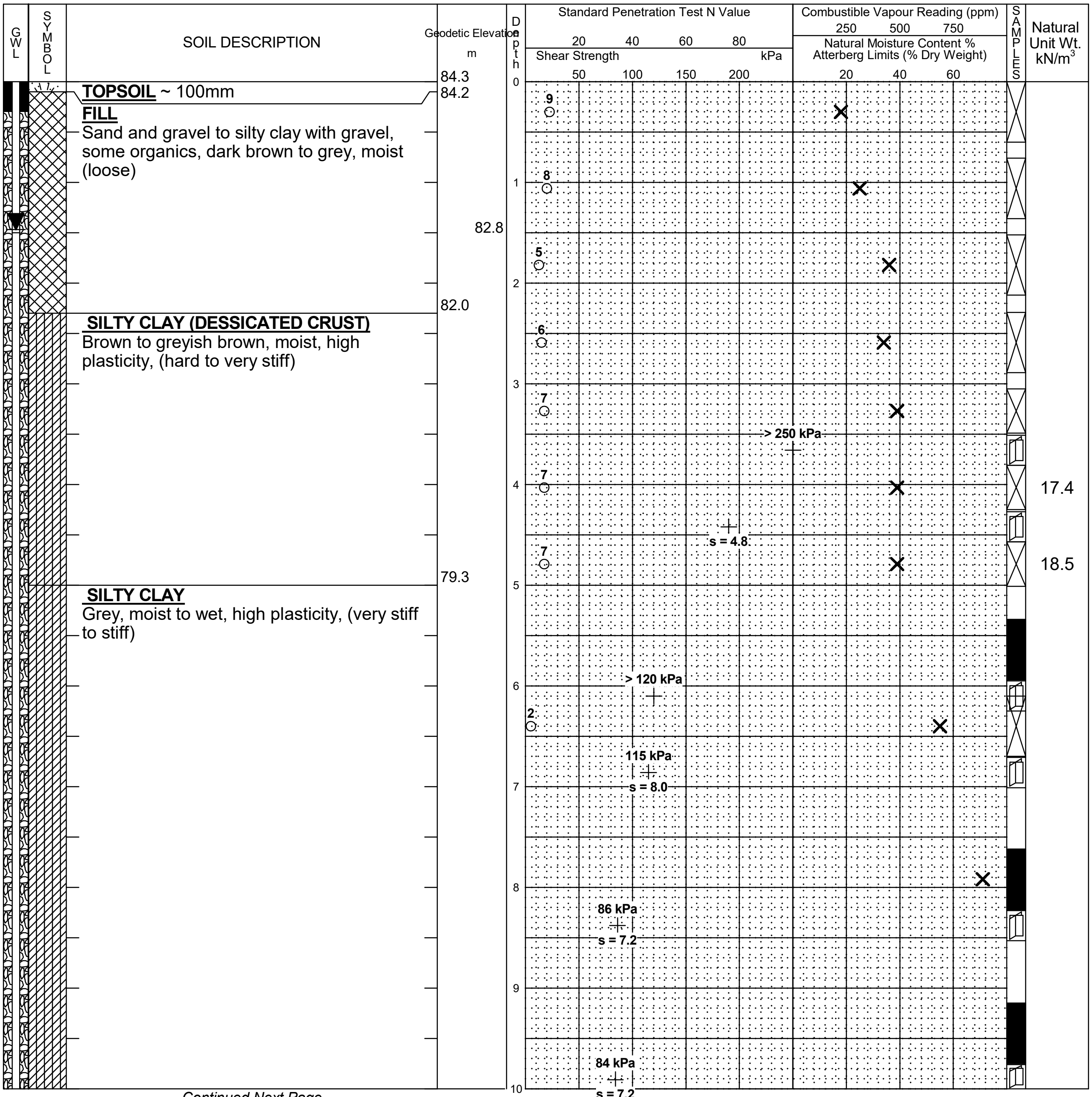
Date Drilled: 'August 17, 2016

Drill Type:

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	▲



Continued Next Page

NOTES:
1. Borehole/Test Pit 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	
Feb 15, 2018	1.5	

CORE DRILLING RECORD

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

Log of Borehole BH-03

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 5

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic 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
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff) *(continued)*	74.3	10				
				11	1; 72 kPa, s = 5.5			
				12	Hammer Weight			
				13	91 kPa, s = 5.4			
				14	Hammer Weight; 103 kPa, s = 7.2			
				15	86 kPa, s = 6.0			
				16				
				17	Hammer Weight			
				18	106 kPa, s = 5.5			
				19				
				20	Hammer Weight			
		GRAVELLY SAND TILL Some silt, trace clay, grey, wet, (compact)	63.4	21	20			
				22				

Continued Next Page

NOTES:
1. Borehole/Test Pit 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	
Feb 15, 2018	1.5	

CORE DRILLING RECORD

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

Log of Borehole BH-03

Project No: OTT-00234493-A0

Figure No. 5

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic 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
		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/Test Pit 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	
Feb 15, 2018	1.5	

CORE DRILLING RECORD

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

Log of Borehole BH-04

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

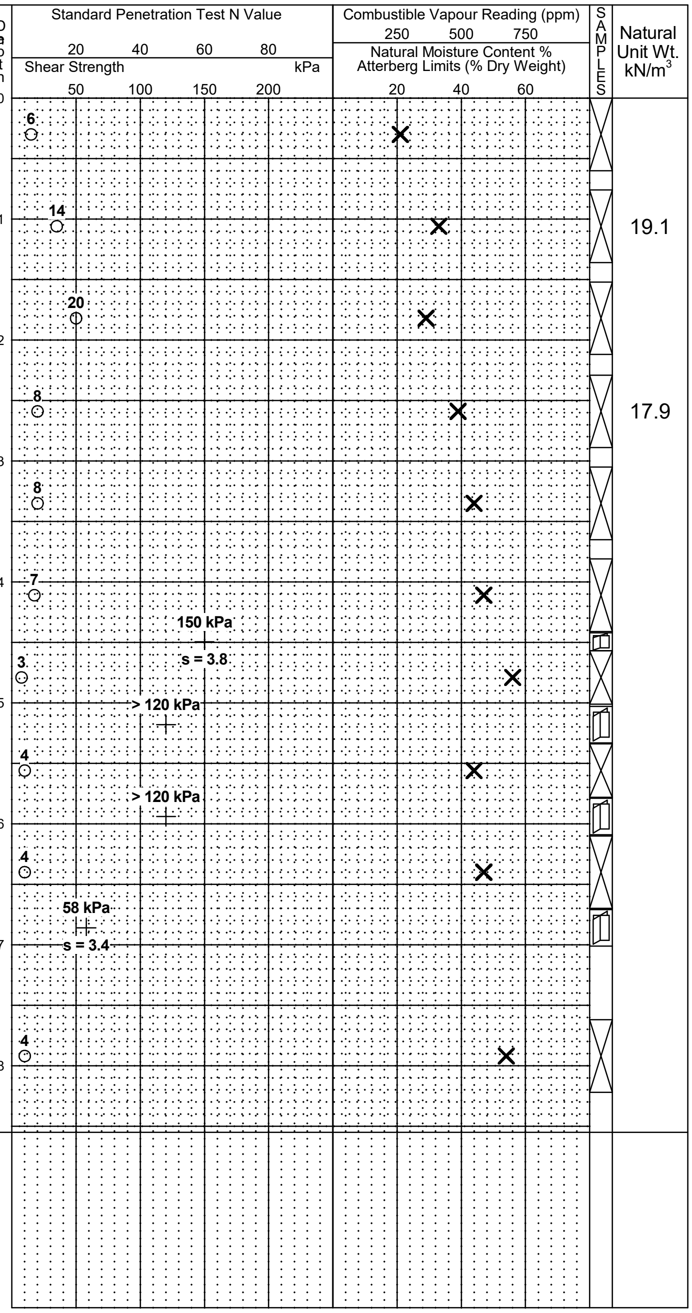
Drill Type:

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 Elevation m	Depth
			83	0
		TOPSOIL ~ 150mm	82.9	
		SILTY SAND Fine grained sand, some organics (rootlets), brown to grey, moist, (loose)	82.3	
		SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity (very stiff)		1, 2, 3
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff)	79.0	4, 5, 6, 7, 8
			74.4	
		Borehole Terminated at 8.60 m Depth		



NOTES:
1. Borehole/Test Pit 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-05

Project No: OTT-00234493-A0

Figure No. 7

Page. 1 of 2

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:

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 Elevation (m)	SOIL DESCRIPTION	Depth (m)	SPT N Value	Shear Strength (Vane)	Natural Moisture Content	Natural Unit Wt. kN/m³
81 / 80.9	**TOPSOIL** ~ 50mm	0				
	SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity (hard)		8		×	
		1	14		×	
			17		×	
			12		×	
			9		×	
77.5	**SILTY CLAY** Grey, moist to wet, high plasticity, (hard to very stiff)					
		4	5		×	
				220 kPa, s = 4.0		
			4		×	17.3
		5		200 kPa, s = 5.0		
			4		×	
				180 kPa, s = 4.0		
		6	4		×	
				180 kPa, s = 3.0		
		8	3		×	16.9
				> 120 kPa		
72.4 / 72.3	**Borehole Terminated at 8.60 m Depth**					
	INFERRED OVERBURDEN Cone advanced from 8.6 m to to refusal at 20.9 m depth	9–10				

Continued Next Page

NOTES:
1. Borehole/Test Pit 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

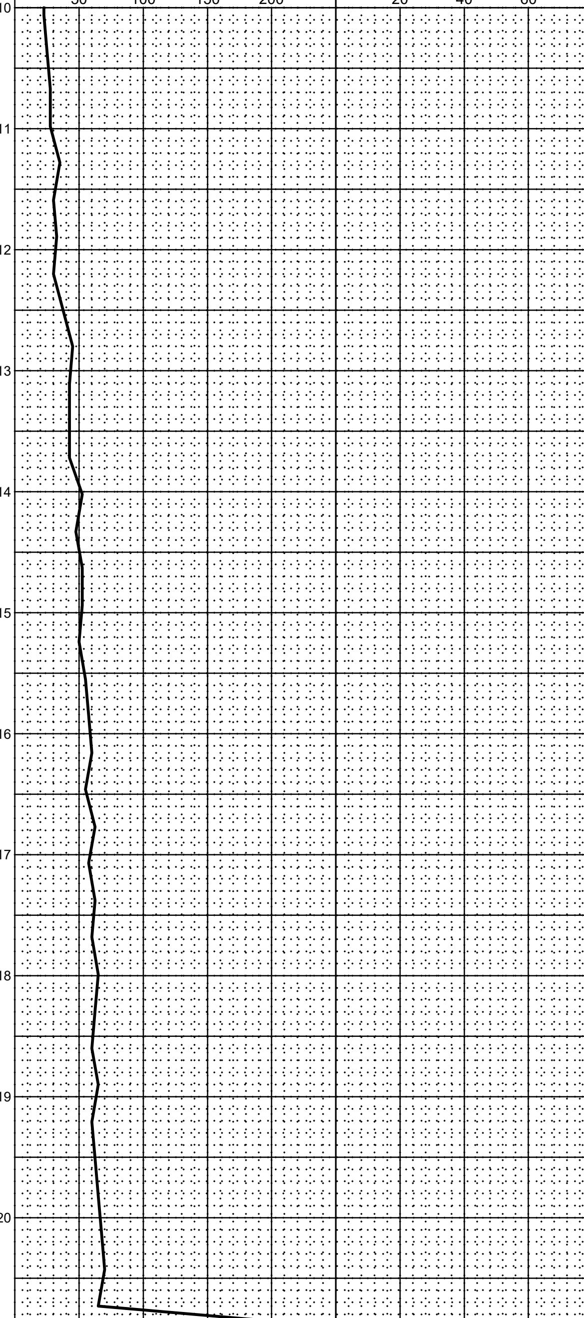
Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-05

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 7

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic 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
		INFERRED OVERBURDEN Cone advanced from 8.6 m to to refusal at 20.9 m depth *(continued)*	71	10 – 20.9				
		Refusal to Cone Penetration at 20.9 m Depth	60.1					

NOTES:
1. Borehole/Test Pit 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 OTT-234439 - OLD MONTREAL ROAD.GPJ TROW OTTAWA.GDT 1/26/21

Log of Borehole BH-06

Project No: OTT-00234493-A0

Figure No. 8

Page. 1 of 1

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

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

Date Drilled: 'August 16, 2016

Drill Type:

Datum: Geodetic

Logged by: Checked by:

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

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth (m)	Standard Penetration Test N Value / Shear Strength (kPa)	Combustible Vapour Reading (ppm) / Natural Moisture Content % / Atterberg Limits (% Dry Weight)	SAMPLES	Natural Unit Wt. kN/m³
			86.6	0				
		TOPSOIL ~ 50mm	86.5		14	×	Split spoon	
		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)		1	23	×	Split spoon	17.8
					14	×	Split spoon	18.1
				2	250 kPa		Auger	
					240 kPa, s = 4.0		Auger	
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff)	83.6	3	8	×	Split spoon	
				4	120 kPa, s = 5.5		Auger	
				5	Hammer Weight	×	Split spoon	
					58 kPa, s = 8.3		Auger	
				6	Hammer Weight	×	Split spoon	
			79.5	7	74 kPa, s = 5.3		Auger	

Borehole Terminated at 7.2 m Depth

NOTES:
1. Borehole/Test Pit 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-07

Project No: OTT-00234493-A0

Figure No. 9

Page. 1 of 1

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

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

Date Drilled: 'August 16, 2016

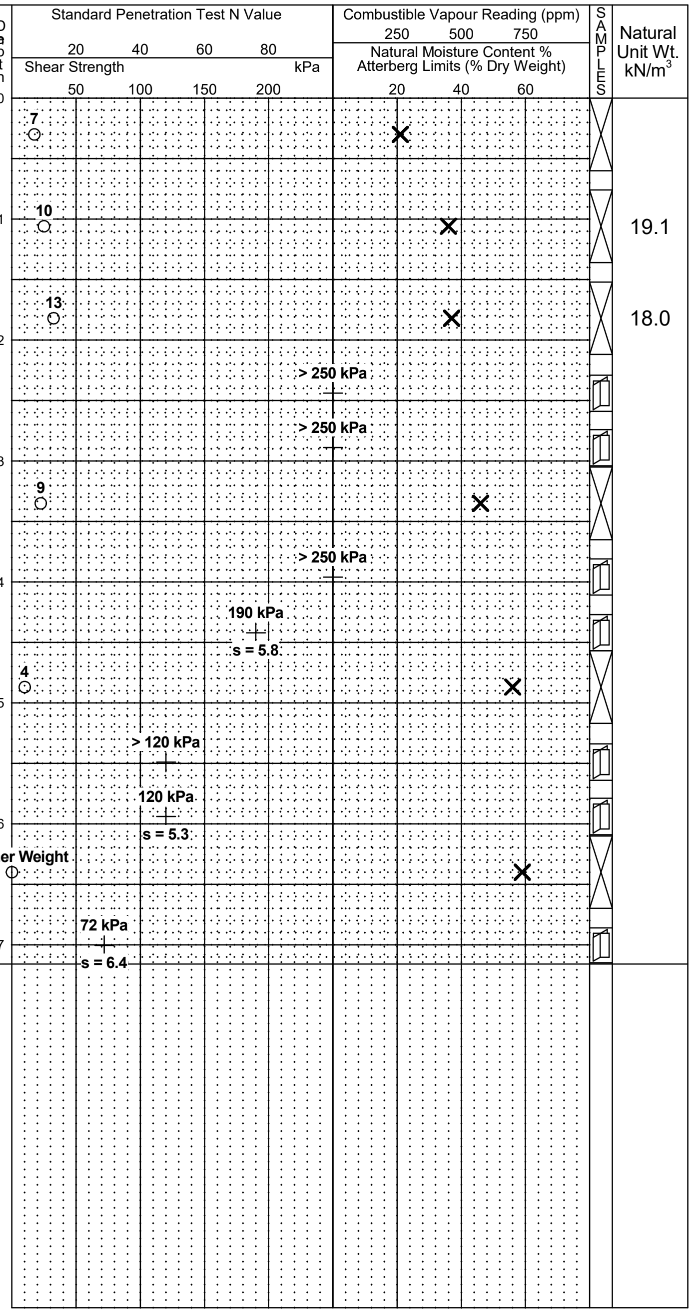
Drill Type:

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		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			85.1	0
		FILL Sand and gravel mixed with silty sand, brown, moist, (loose)		
			84.2	1
		SILTY CLAY (DESSICATED CRUST) Brown to greyish brown, moist, high plasticity, (hard)		
			83.8	2
				3
			81.1	4
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff)		5
				6
			78.0	7
		Borehole Terminated at 7.20 m Depth		



NOTES:
1. Borehole/Test Pit 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 %

Log of Borehole BH-07B

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. 10

Page. 1 of 2

Date Drilled: 'February 5, 2018

Drill Type:

Datum: Geodetic

Logged by: Checked by:

Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	X
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	Geodetic 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
		INFERRED OVERBURDEN Augered through overburden to 6.1 m depth	85.1	0				
		SILTY CLAY Grey, wet, high plasticity, (very stiff)	79.0	6.1	N = 5 (≈6.4 m); Vane 115, S = 4.8 (≈6.8 m); N = 3 (≈7.9 m); Vane 101, S = 6.0 (≈8.4 m); N = 2 (≈9.5 m); Vane 115, S = 4.4 (≈9.9 m)	Moisture content plotted at ≈6.4 m, ≈7.9 m, ≈9.5 m	Split spoon and Shelby tube samples	16.5 (≈6.4 m); 16.2 (≈9.5 m)

Continued Next Page

NOTES:
1. Borehole/Test Pit 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)
'Feb 5, 2018	dry	13.1

CORE DRILLING RECORD

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

Log of Borehole BH-07B

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 10

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic 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
		SILTY CLAY Grey, wet, high plasticity, (very stiff) *(continued)*	75.1	10				
				11	6			
		GRAVELLY SAND TILL Some silt, trace clay, grey, moist, (very dense)	73.9					
				12	64			
				13				
			71.5					

Borehole Terminated at 13.6 m Depth Upon Auger Refusal

NOTES:
1. Borehole/Test Pit 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)
'Feb 5, 2018	dry	13.1

CORE DRILLING RECORD

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

Log of Borehole BH-08

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. 11

Page. 1 of 1

Date Drilled: 'February 7, 2018

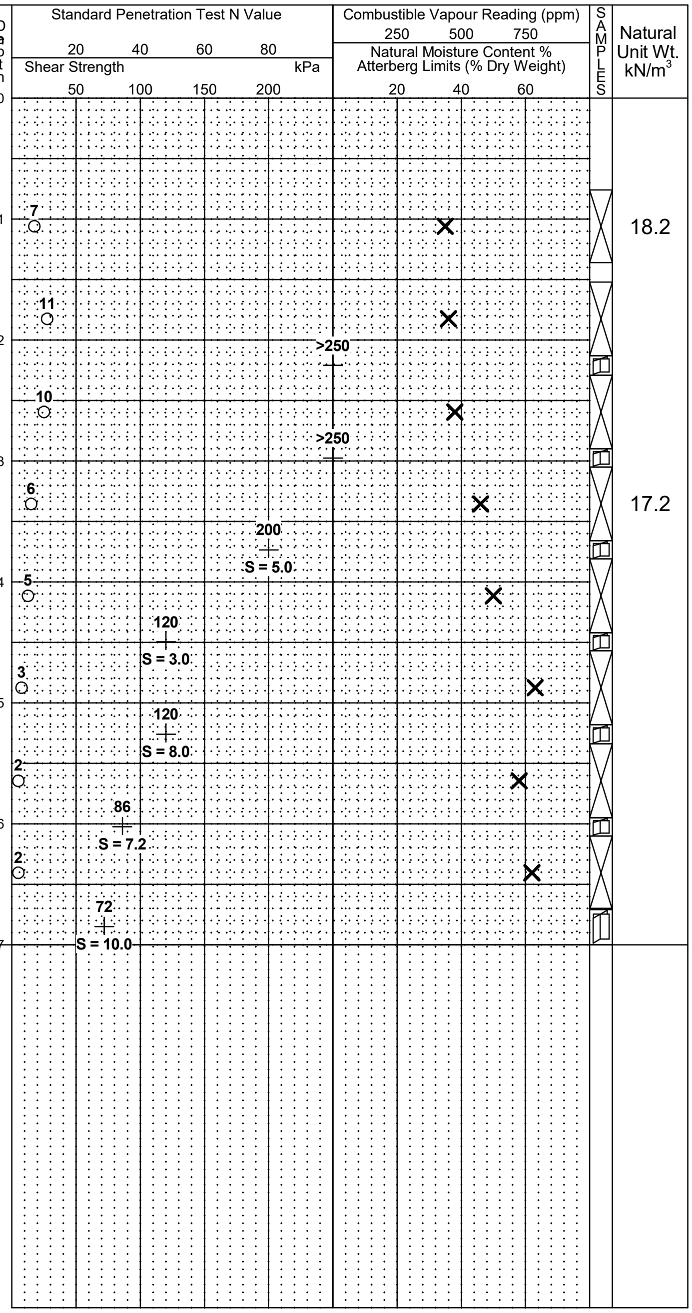
Drill Type:

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		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			85.9	0
		TOPSOIL ~200mm	85.7	
		SILTY SAND Fine grained sand, greyish brown, moist -Upper 250 mm frozen	85.4	
▼		**SILTY CLAY (DESSICATED CRUST)** Greyish brown, moist, high plasticity, (hard to very stiff)	85.1	
		SILTY CLAY Grey, wet, high plasticity, (stiff)	80.5	
		Borehole Terminated at 7.0 m Depth	78.9	7



NOTES:
1. Borehole/Test Pit 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)
'Feb 7, 2018	dry	6.1
Feb 15, 2018	0.8	

CORE DRILLING RECORD

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

Log of Borehole BH-09

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. 12

Page. 1 of 2

Date Drilled: 'February 6, 2018

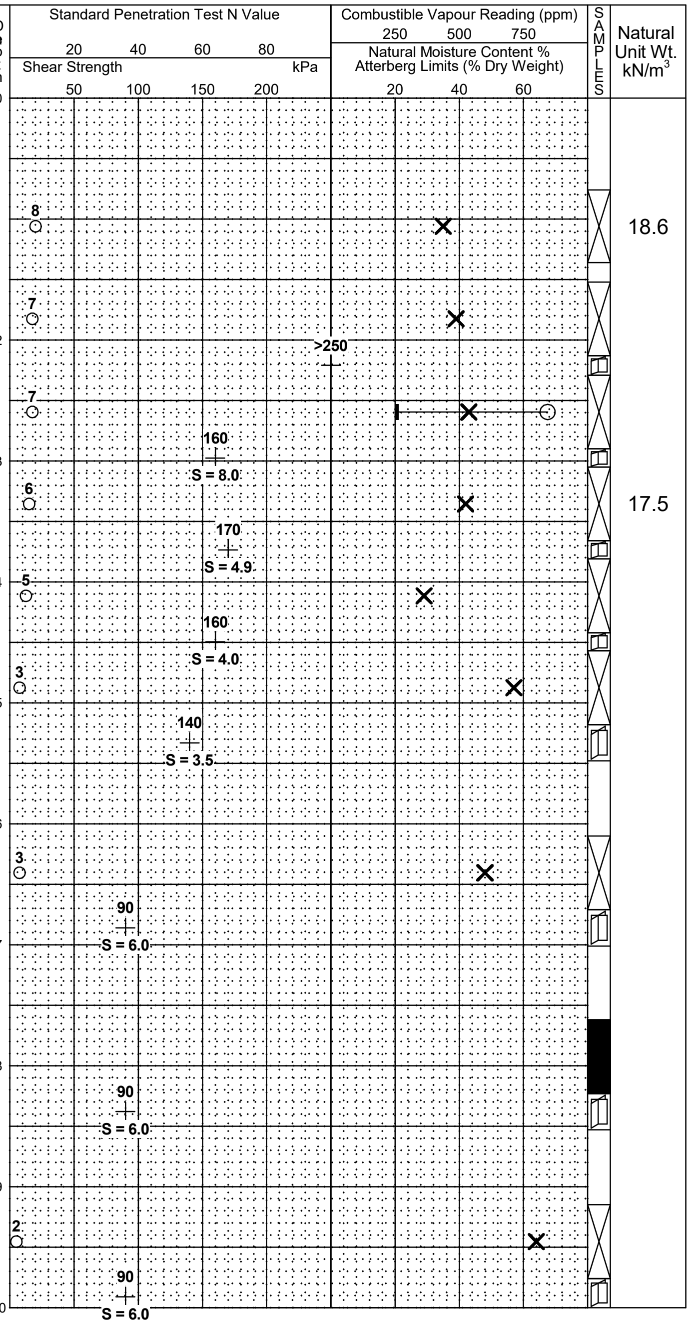
Drill Type:

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		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			82.2	0
		TOPSOIL ~150mm	82.1	
		SILTY CLAY (DESSICATED CRUST) Greyish brown, moist, high plasticity, (hard to very stiff) -Upper 200 mm frozen		
		SILTY CLAY Grey, wet, high plasticity, (very stiff to stiff)	76.7	



Continued Next Page

NOTES:
1. Borehole/Test Pit 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)
'Feb 6, 2018	dry	12.2
Feb 15, 2018	1.0	

CORE DRILLING RECORD

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

Log of Borehole BH-09

Project No: OTT-00234493-A0

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 12

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic 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
		SILTY CLAY Grey, wet, high plasticity, (very stiff to stiff) *(continued)*	72.2	10				
				11	2; 95; S = 6.3			
				12	2; 120; S = 6.0			
			69.1	13				
		Borehole Terminated at 13.1 m Depth						

NOTES:
1. Borehole/Test Pit 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)
'Feb 6, 2018	dry	12.2
Feb 15, 2018	1.0	

CORE DRILLING RECORD

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

Log of Borehole BH-10

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. 13

Page. 1 of 1

Date Drilled: 'February 6, 2018

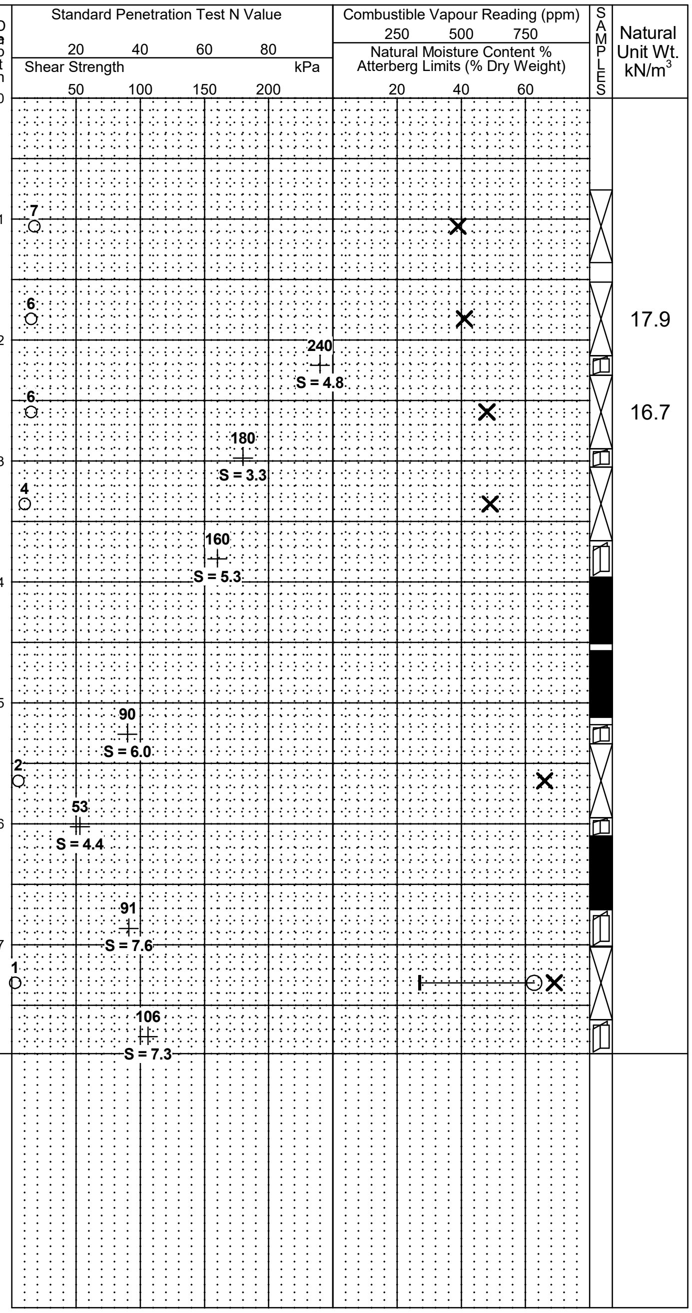
Drill Type:

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		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
		TOPSOIL ~200mm	77.1	0
		SILTY CLAY (DESSICATED CRUST) Greyish brown, moist, high plasticity, (hard to very stiff) -Upper 300 mm frozen	76.9	
			76.1	1
		SILTY CLAY Grey, wet, high plasticity, (stiff to very stiff)	71.8	
		Borehole Terminated at 7.9 m Depth	69.2	



NOTES:
1. Borehole/Test Pit 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)
'Feb 6, 2018	dry	7.0
Feb 15, 2018	0.7	

CORE DRILLING RECORD

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

LOG OF BOREHOLE OTT-234439 - OLD MONTREAL ROAD.GPJ TROW OTTAWA.GDT 1/26/21

Log of Borehole BH-11

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. 14

Page. 1 of 1

Date Drilled: 'February 5, 2018

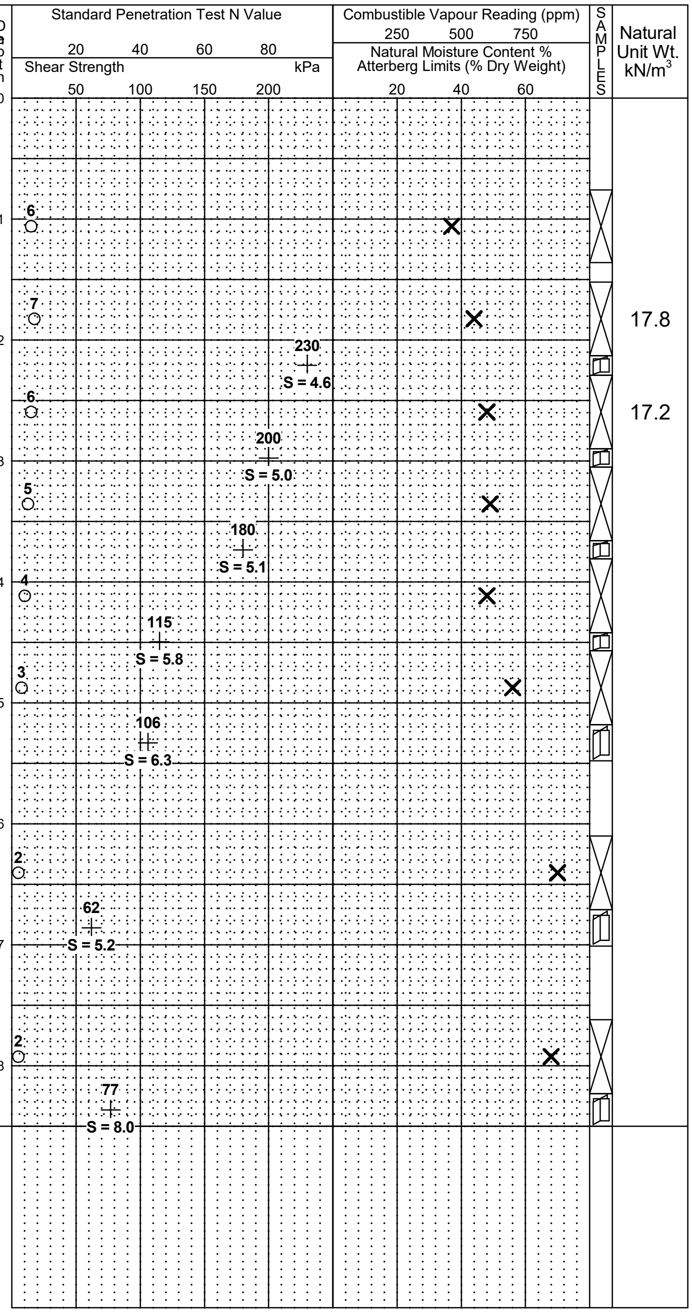
Drill Type:

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 Elevation m	Depth
			72.9	0
		GRANULAR FILL ~300mm	72.6	
		SILTY CLAY (DESSICATED CRUST) Greyish brown, moist, high plasticity, (hard to very stiff)		
		-Upper 900 mm frozen	71.8	1
		SILTY CLAY Grey, wet, high plasticity, (stiff)	67.3	
		Borehole Terminated at 8.5 m Depth	64.4	



NOTES:
1. Borehole/Test Pit 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)
'Feb 5, 2018	dry	7.6
Feb 15, 2018	1.1	

CORE DRILLING RECORD

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

Log of Borehole BH-12

Project No: OTT-00234493-A0

Figure No. 15

Page. 1 of 1

Project: Preliminary Geotechnical Investigation. Proposed Residential Subdivision

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

Date Drilled: 'February 7, 2018

Drill Type:

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		

Depth (m)	Geodetic Elevation (m)	Soil Description	SPT N Value	Shear Strength (kPa)	Natural Moisture Content / Atterberg Limits	Natural Unit Wt. kN/m³
0	71.9	**SILTY CLAY (DESSICATED CRUST)** Greyish brown, moist, high plasticity, (hard to very stiff) -Upper 300 mm frozen				
~1.0	70.7		12		×	18.4
~1.8			9		×	17.9
~2.2				>250 S =		
~2.6			6		×	
~3.0				200 S =		
~3.4			4		×	
~3.8				160 S =		
~4.1			2		×	
~4.5				130 S =		
~4.9			2		├─×─○	
~5.3				130 S =		
5.6	66.3	**SILTY CLAY** Grey, wet, high plasticity, (very stiff)				
~7.0				110 S =		
~7.9			2		×	
~8.3				115 S =		
8.5	63.4	**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 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)
'Feb 7, 2018	dry	7.6
Feb 15, 2018	1.2	

CORE DRILLING RECORD

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

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

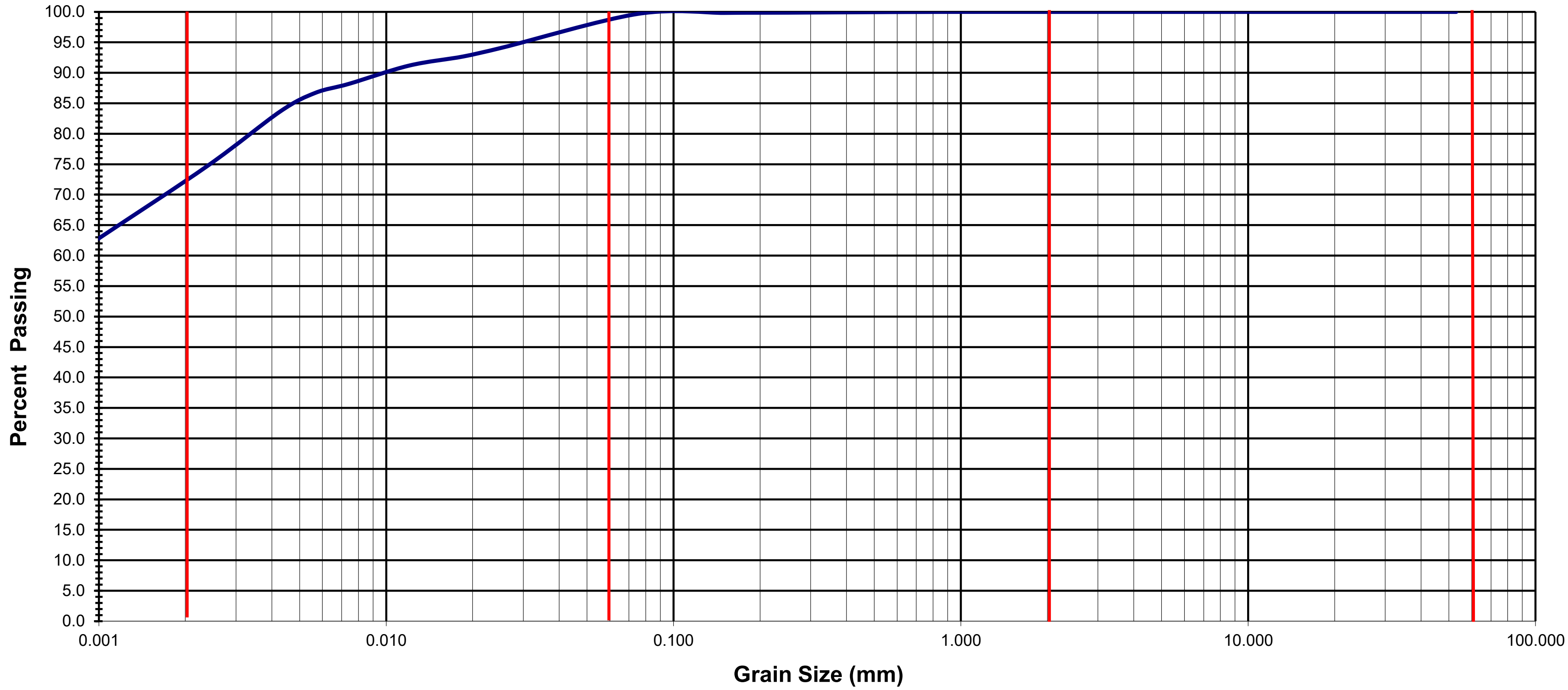
Grain-Size Distribution Curve

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 :**	16

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

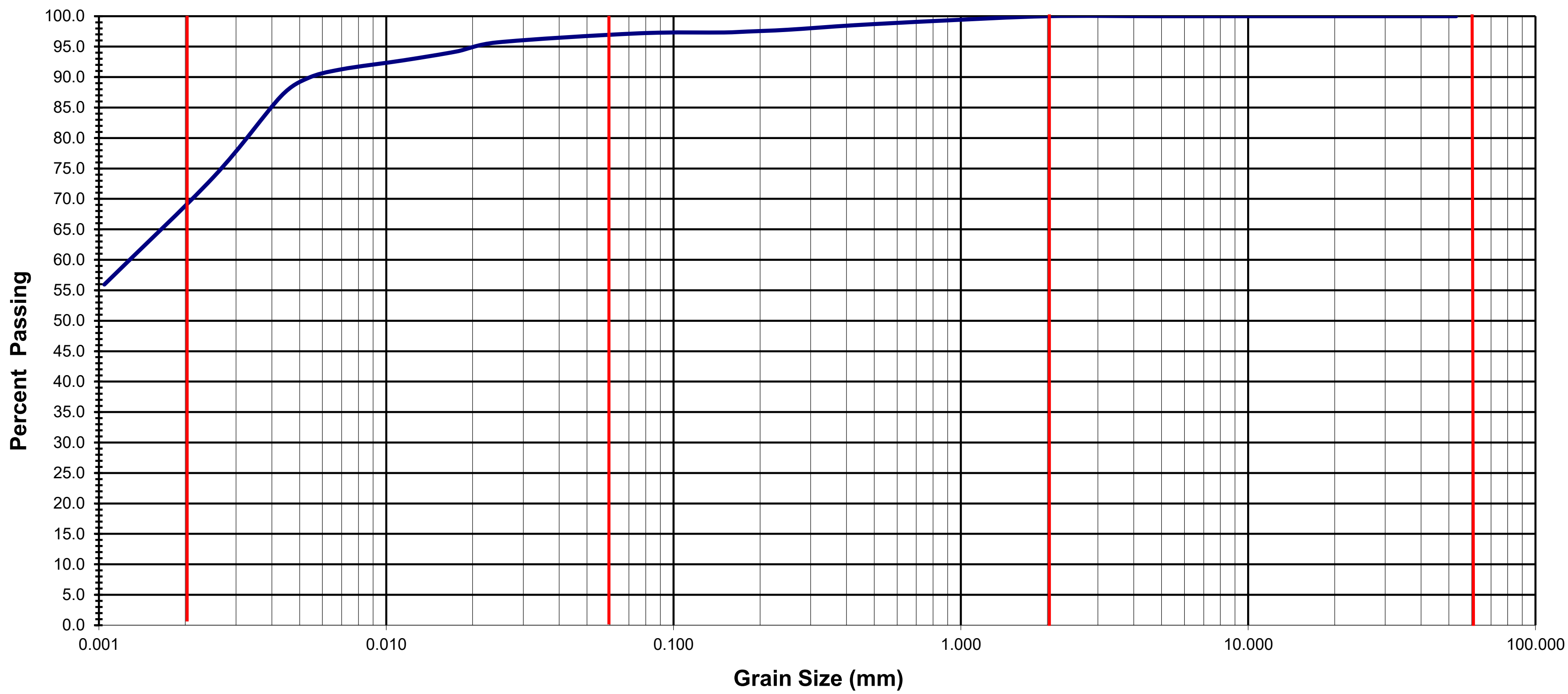
Grain-Size Distribution Curve

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 :**	**17**

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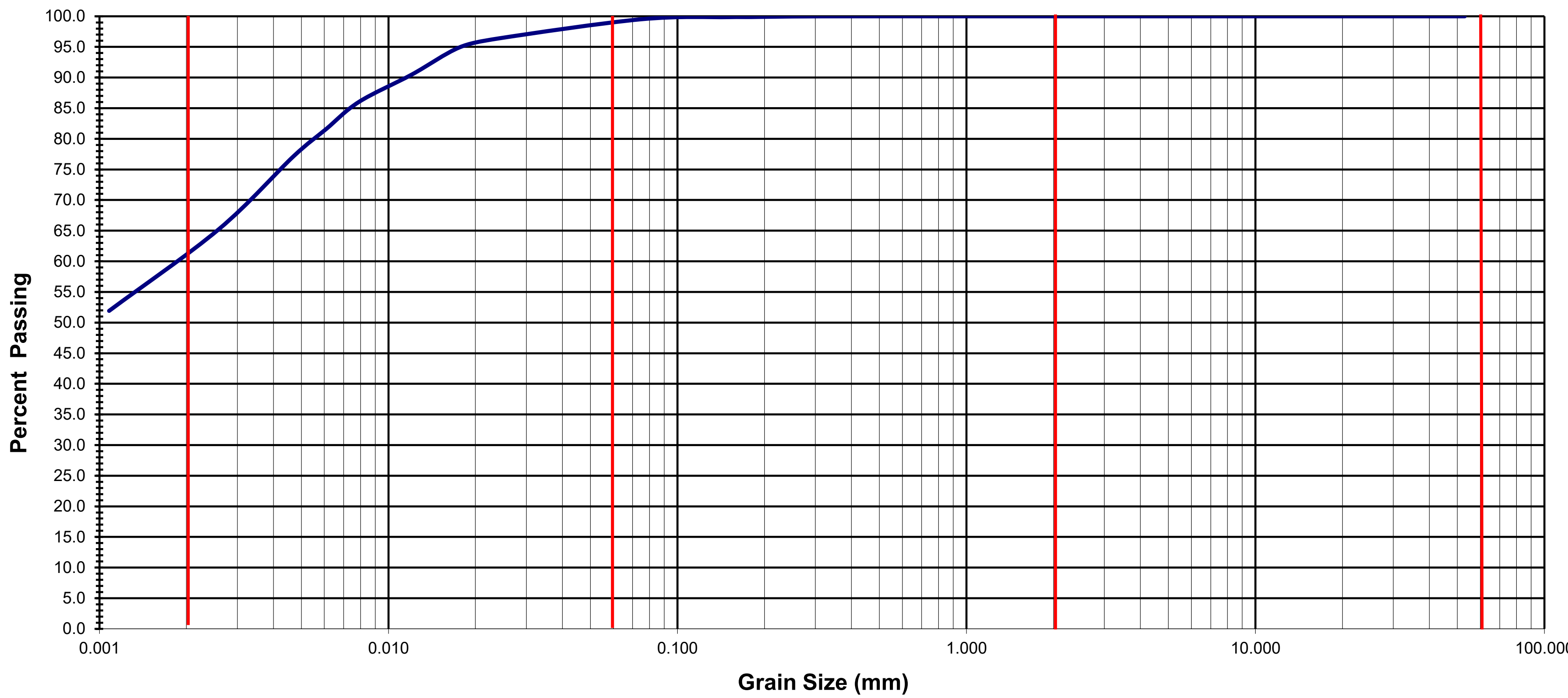
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

Test Method ASTM D-422

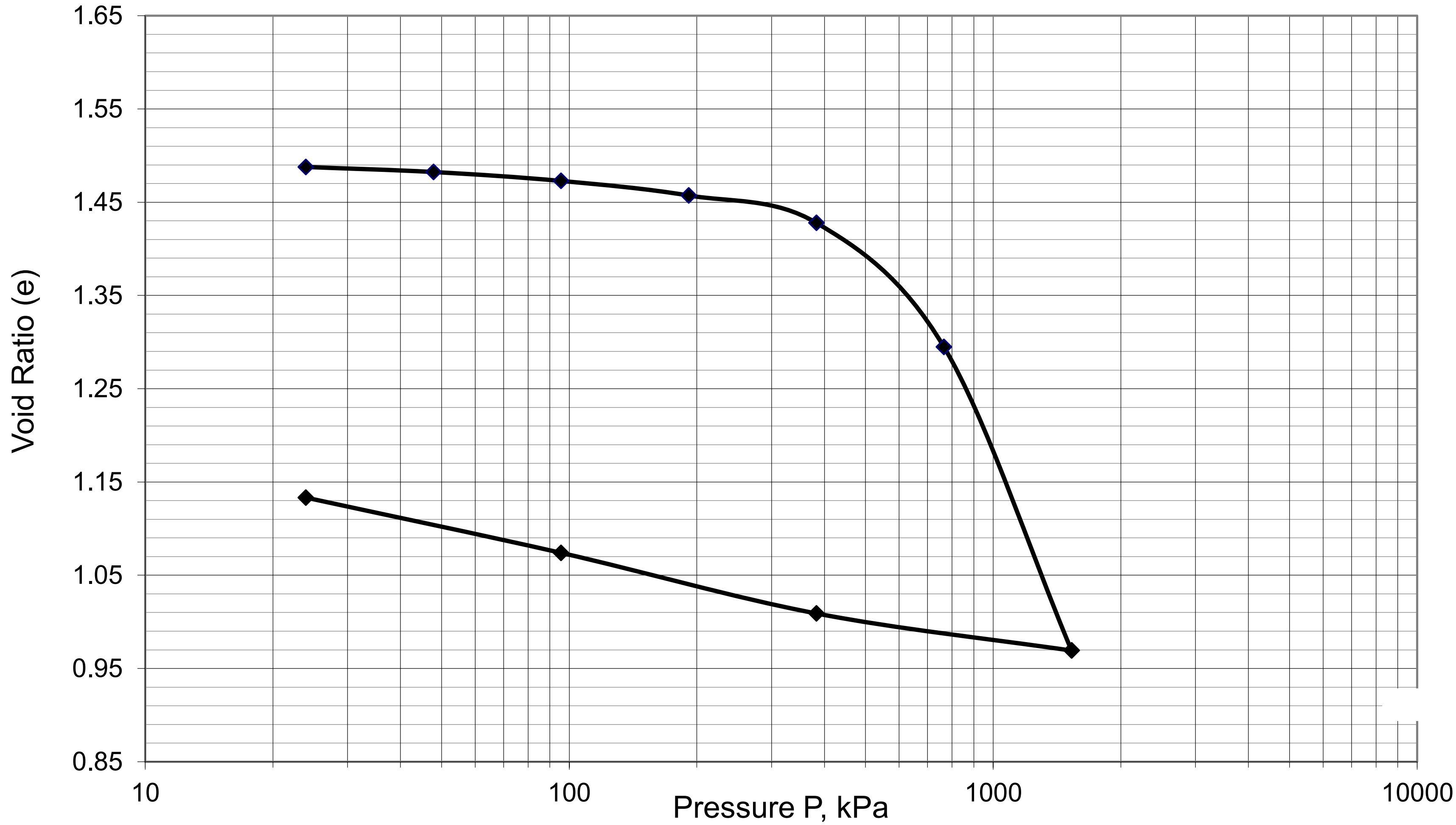
Modified M.I.T. Classification

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



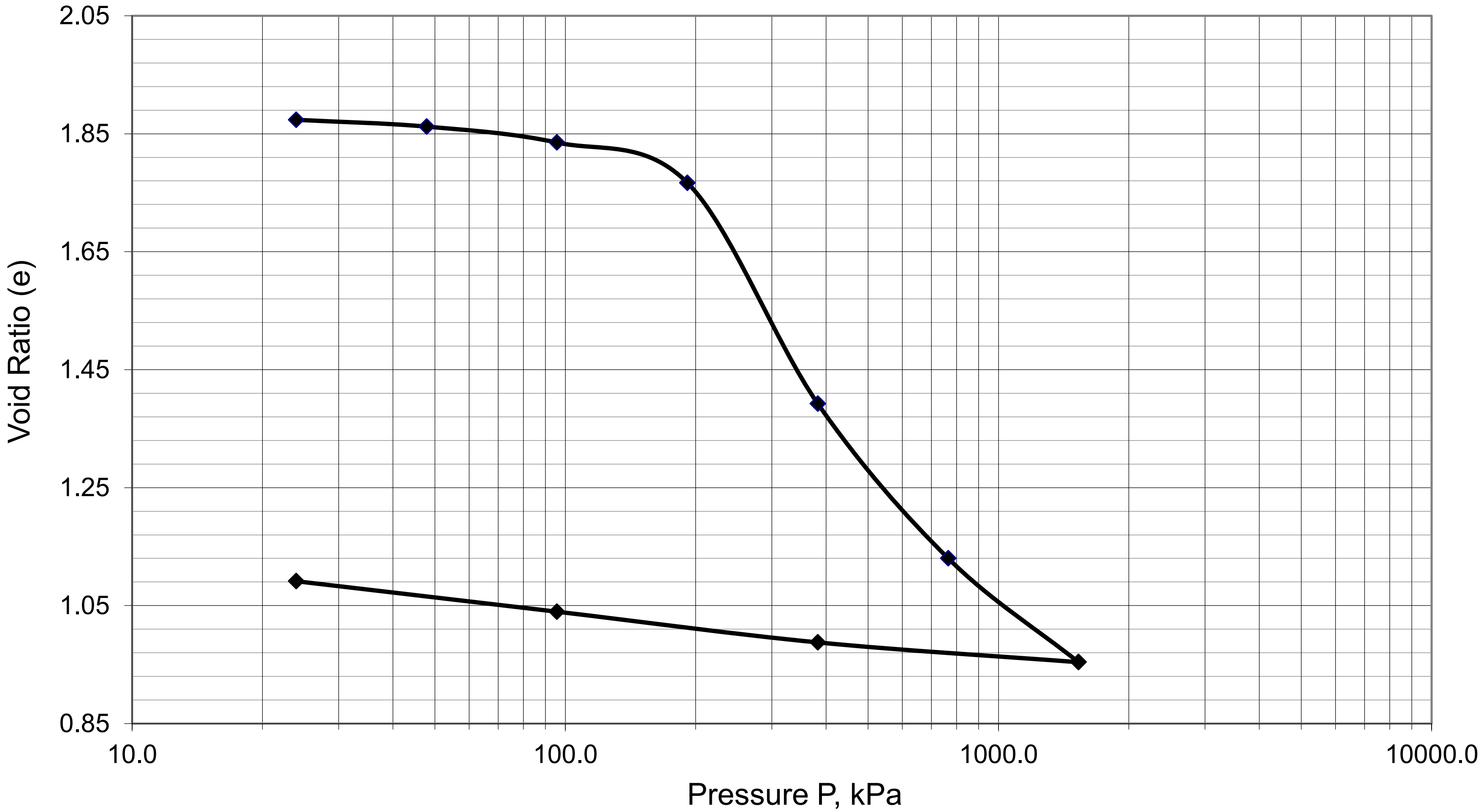
Exp Project No.:	**OTT-00234493-A0**	**Project Name :**	**Preliminary Geotechnical Invsetigation. Proposed Residential Subdvision**		
Client :	**DCR/Phoenix Group of Companies**	**Project Location :**	**1154 - 1208 Old Montreal Road, City of Ottawa, Ontario**		
Date Sampled :	**August 15, 2016**	**Borehole.:** **1**	**Sample No.:** **SS12**	**Depth (m) :**	**15.2-15.8**
Sample Description :		**Silt - Clay**		**Figure :**	**18**

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	**19**

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	**20**

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Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

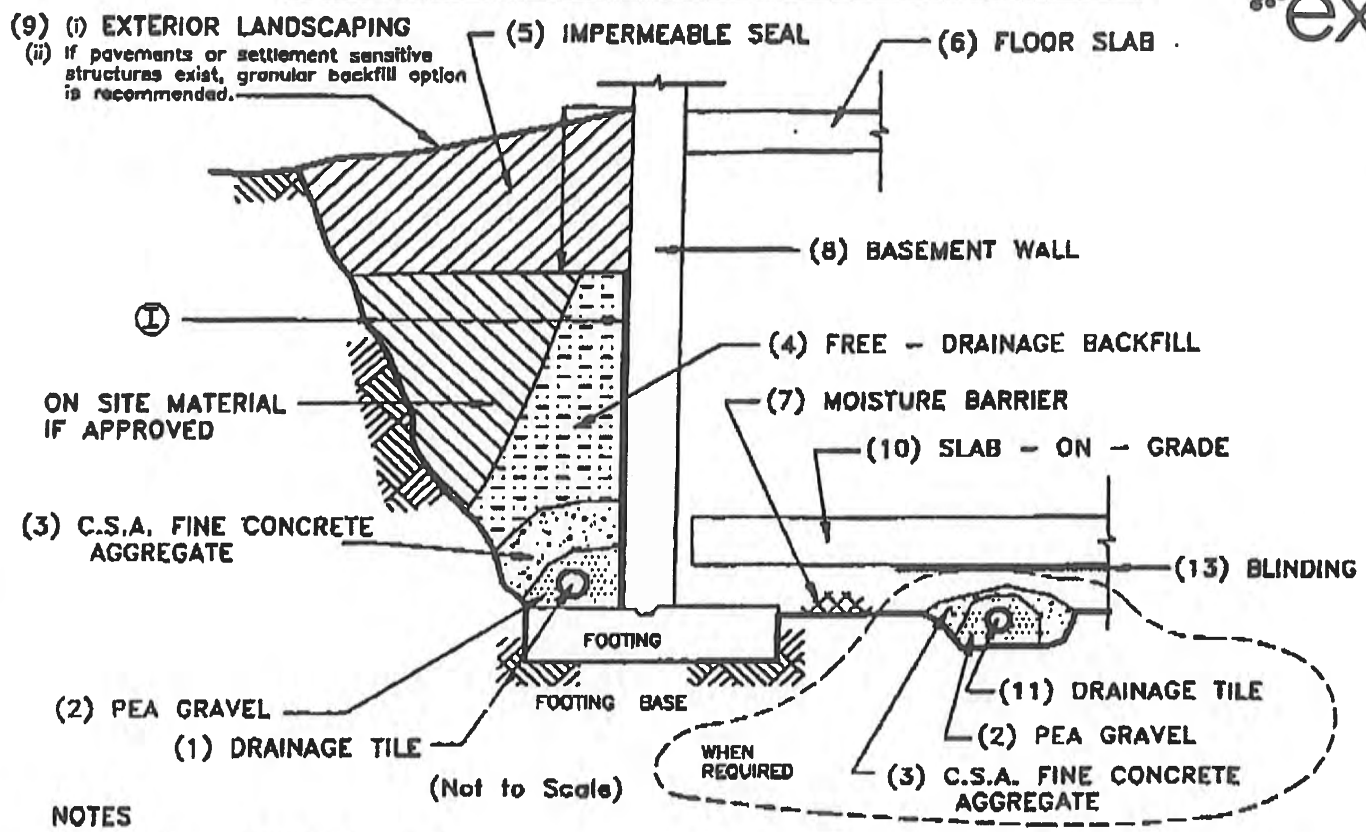
Test Method ASTM D-422

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 :**	**21**

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. Minimum 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.

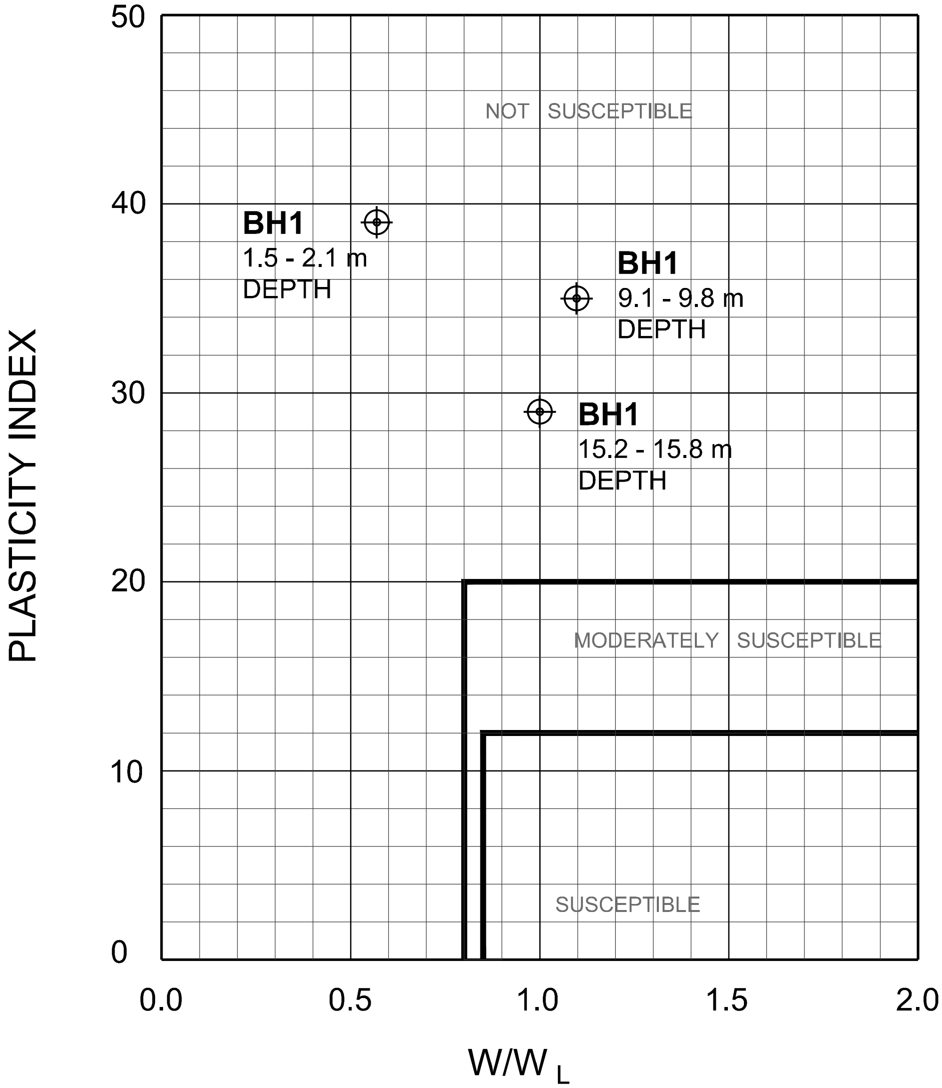
NOTE: A) Underfloor drainage can be deleted where not required (see report).

OPTION B – CORE DRAIN

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

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

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



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

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scale N.T.S.	CLIENT:	DCR PHOENIX GROUP OF COMPANIES	project no. OTT-00234493-A0
date OCT. 31, 2016	TITLE:	PRELIMINARY GEOTECHNICAL INVESTIGATION 1154 - 1208 OLD MONTREAL ROAD, OTTAWA, ON	FIG 23
drawn by M.N.			

Filename: r:\230000\234000\234493 - 1154-1208 old montreal road\fig 17 liquefaction assessment.dwg
Last Saved: 10/31/2016 11:28:17 AM
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PROJECT No.: OTT-00234493-A0
1154-1208 Old Montreal, Ottawa, Ontario
Section A-A
Effective Stress Analysis

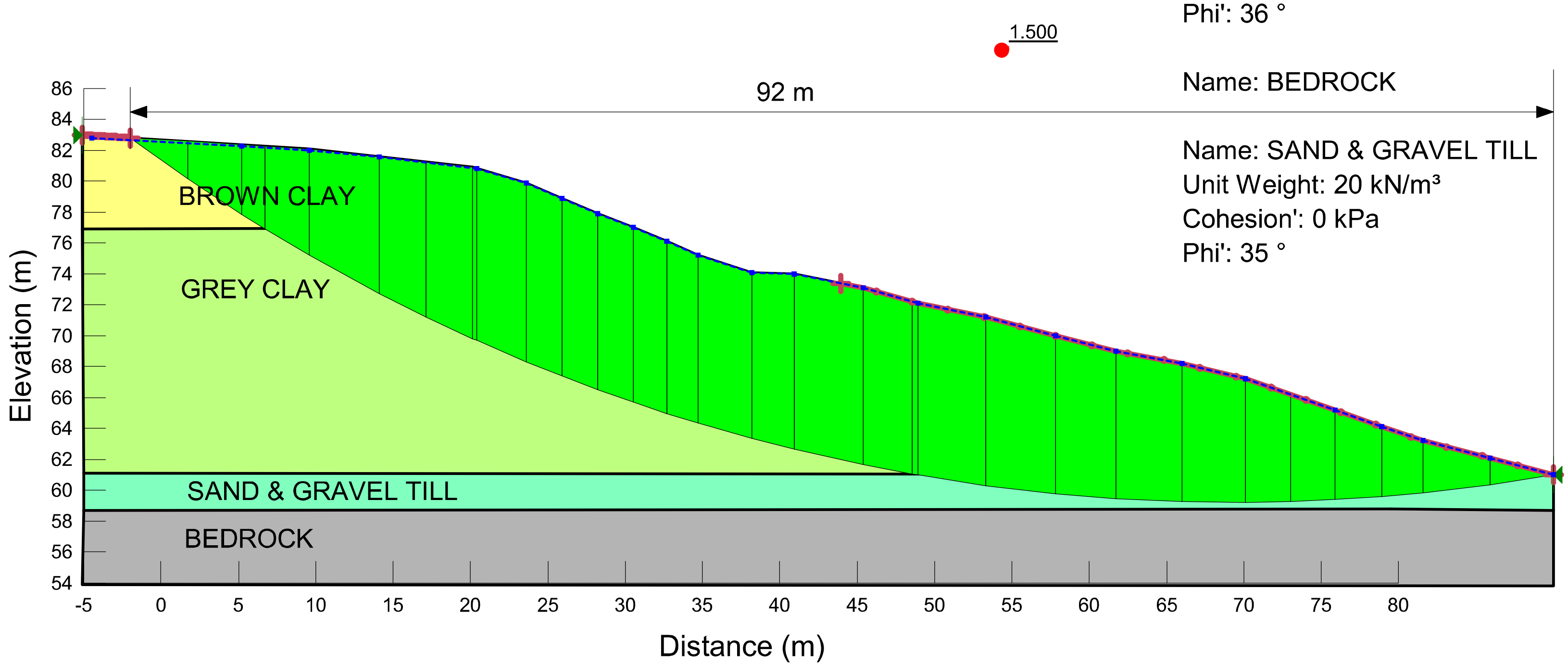
Figure No. 24

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 °

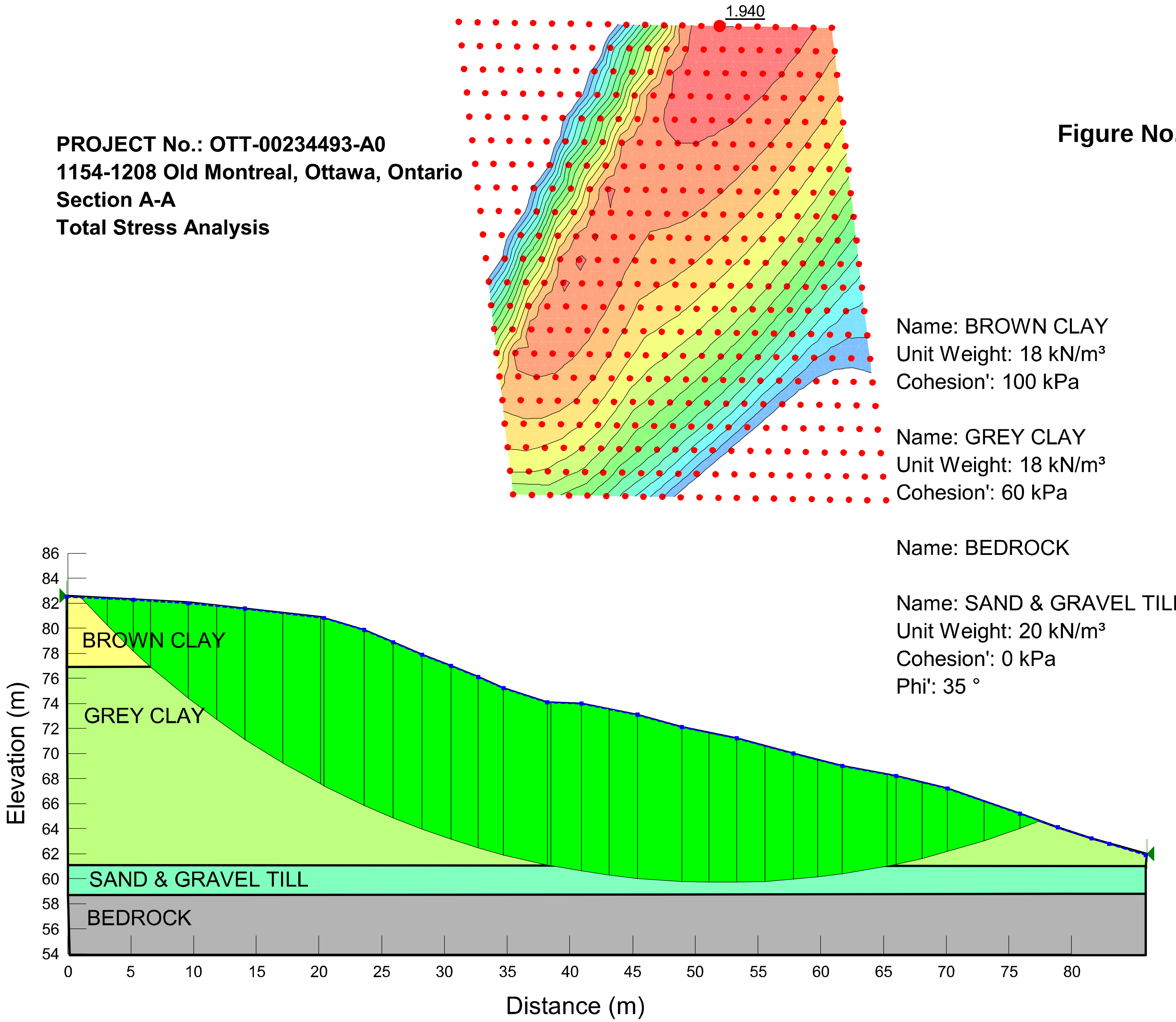
Name: BEDROCK

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



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

Figure No. 25



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

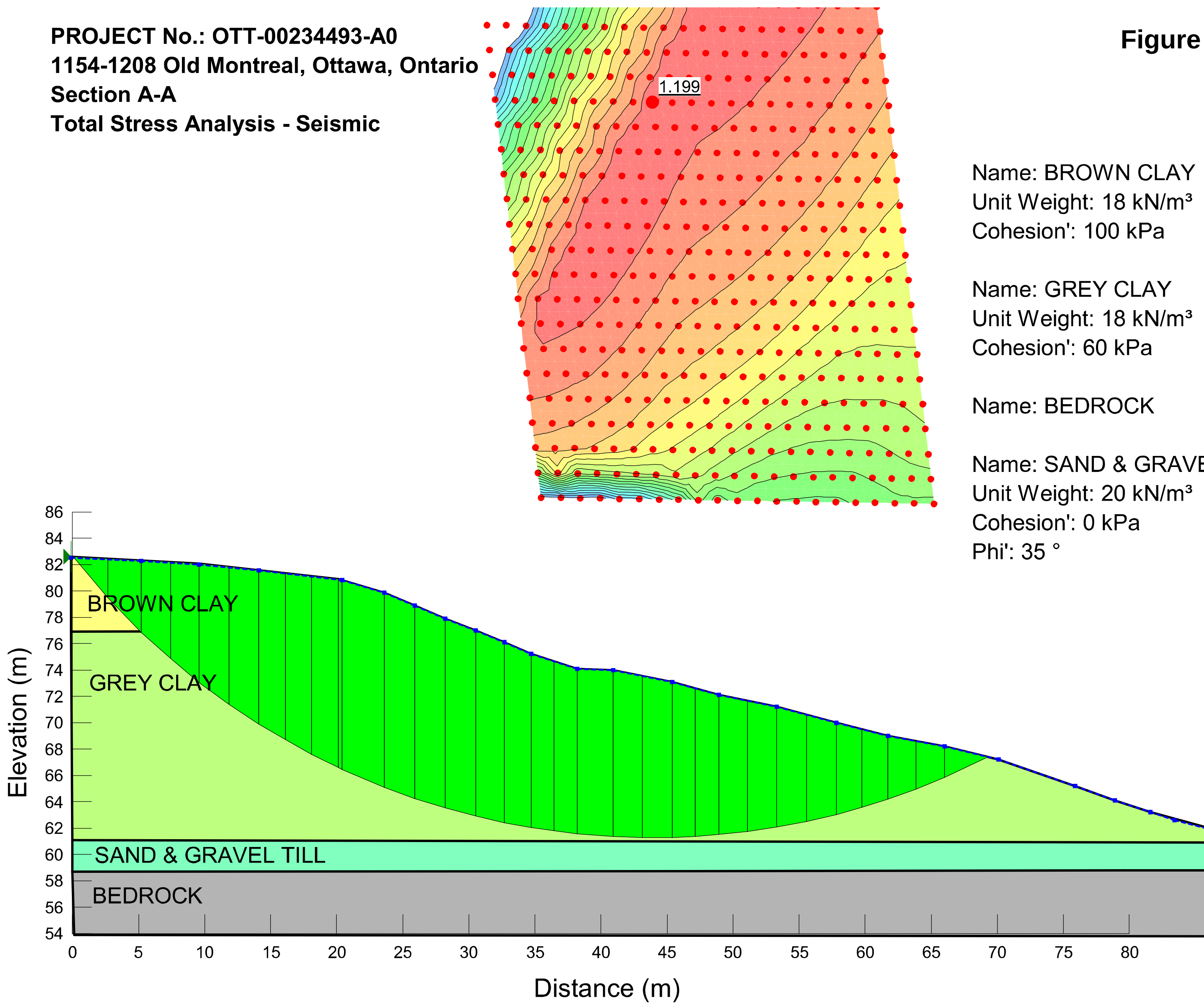
Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

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

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

Figure No. 26



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

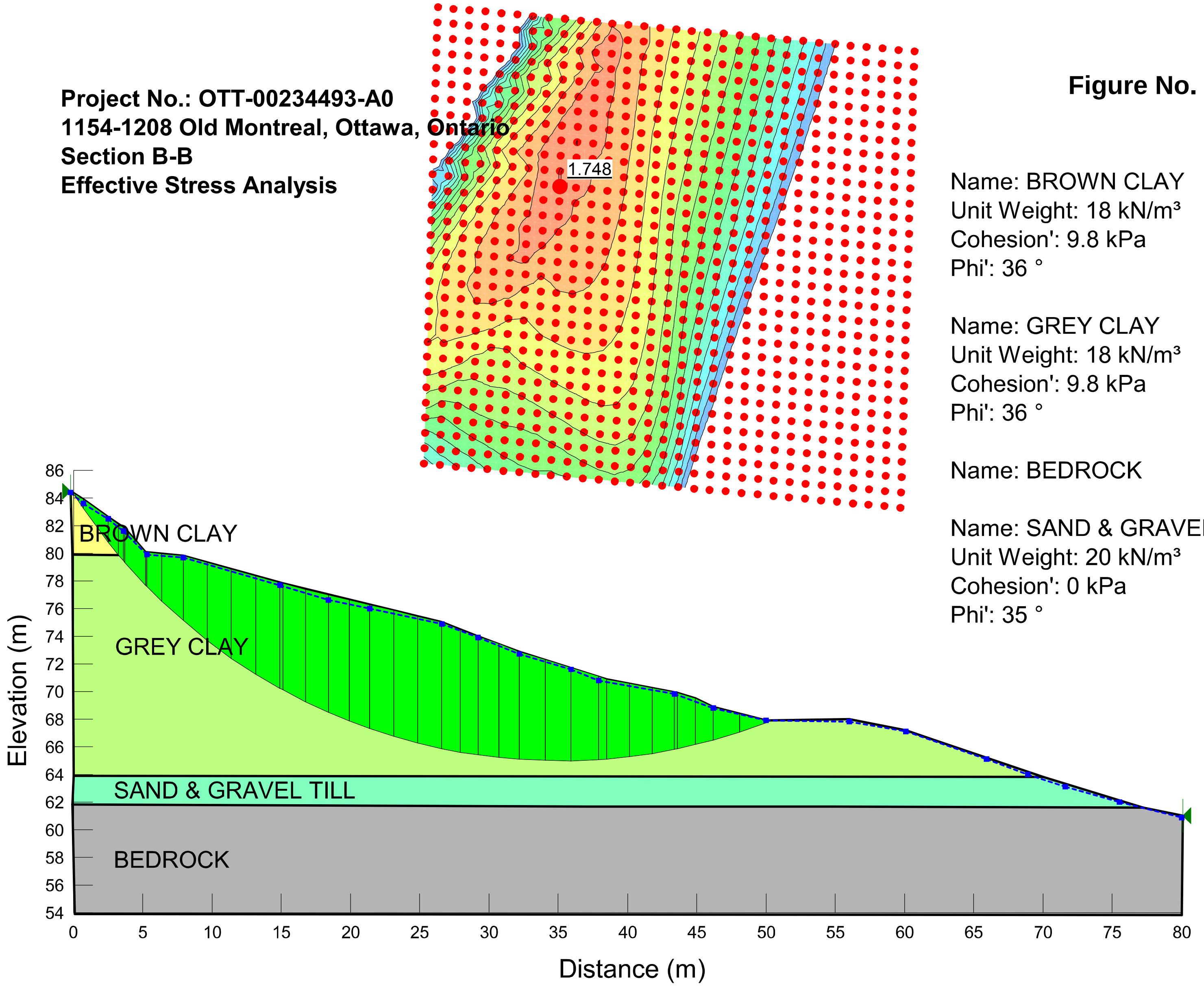
Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

Name: BEDROCK

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

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

Figure No. 27



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 B-B
Total Stress Analysis

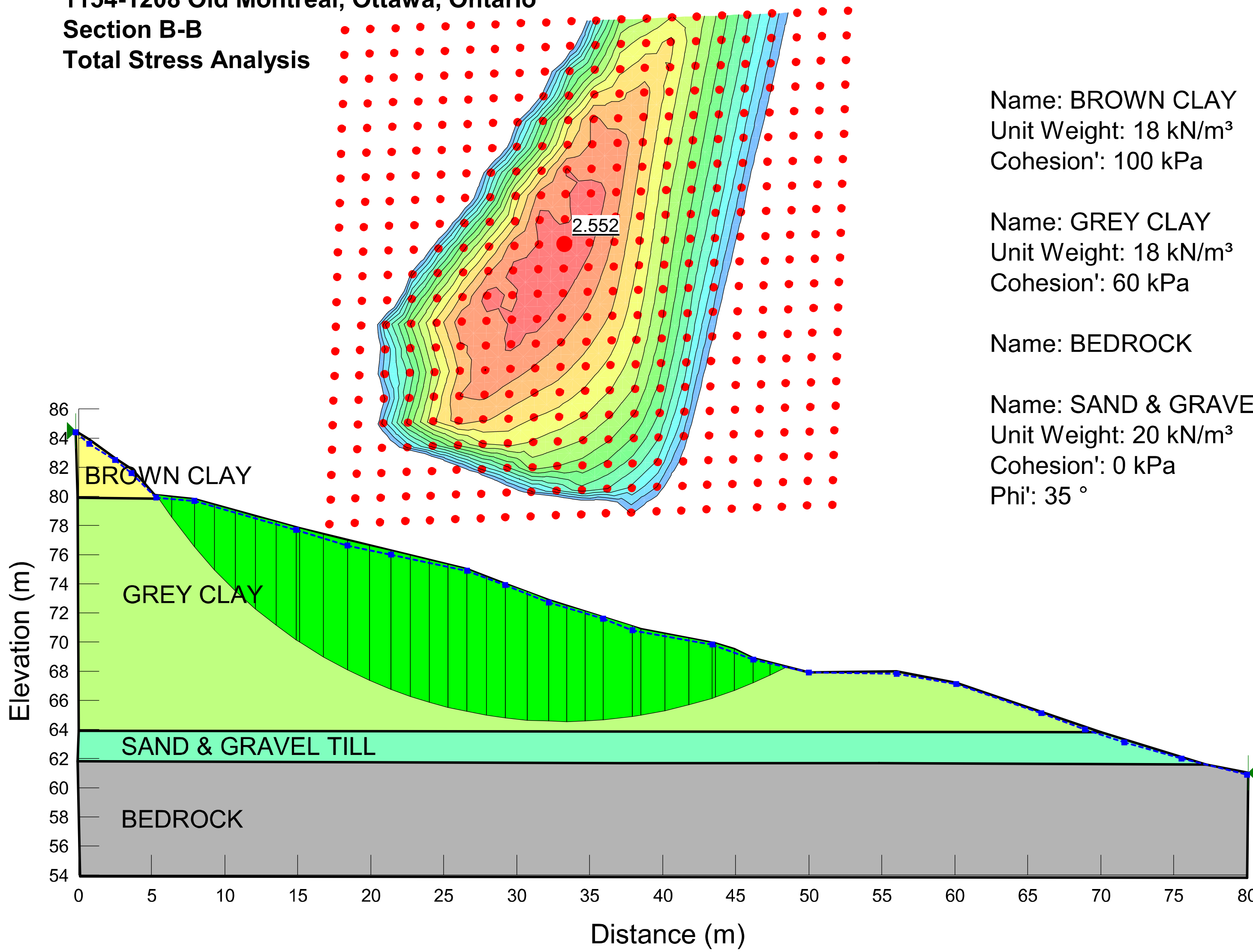
Figure No. 28

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
Total Stress Analysis - Seismic

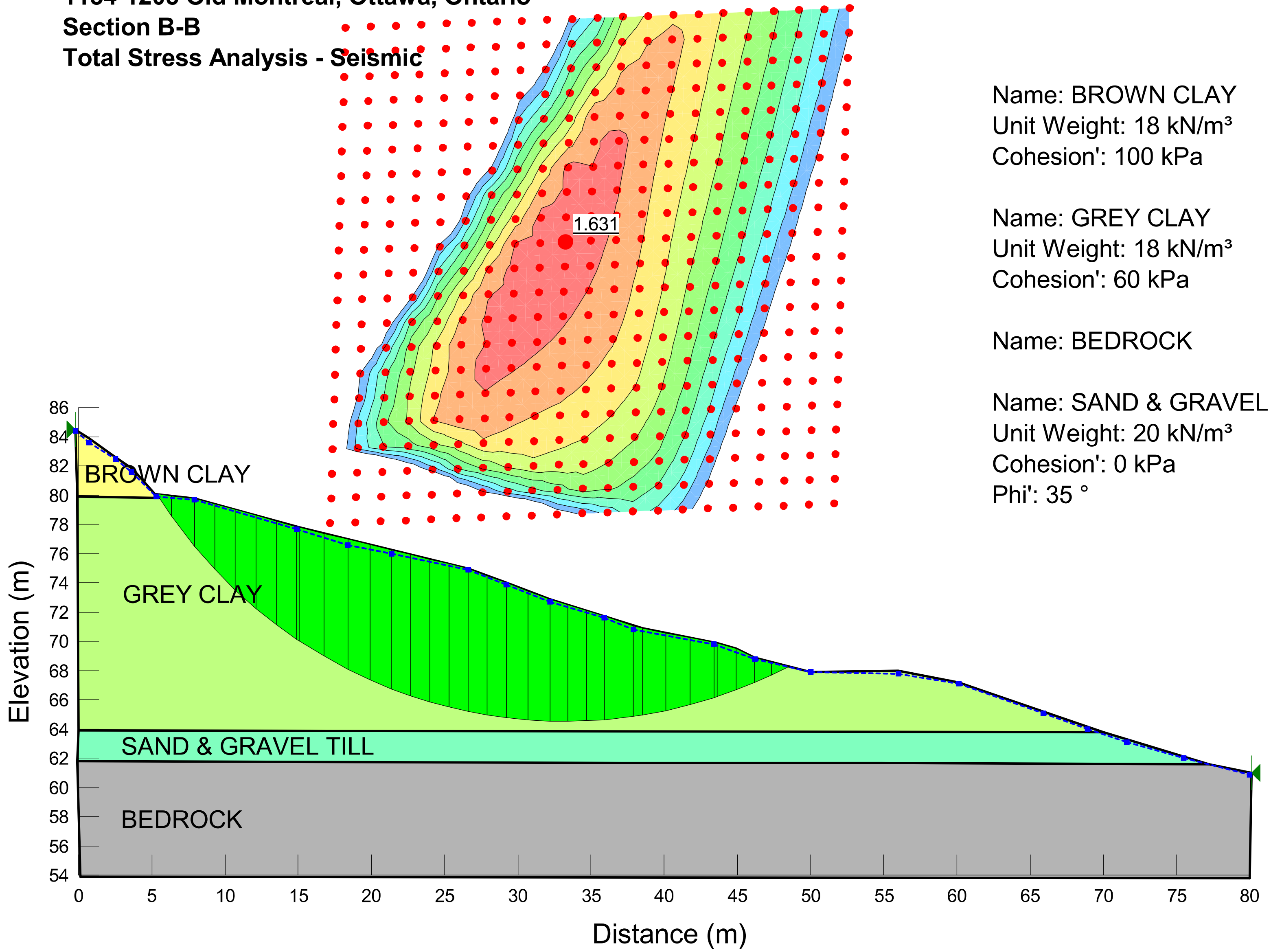
Figure No. 29

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

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

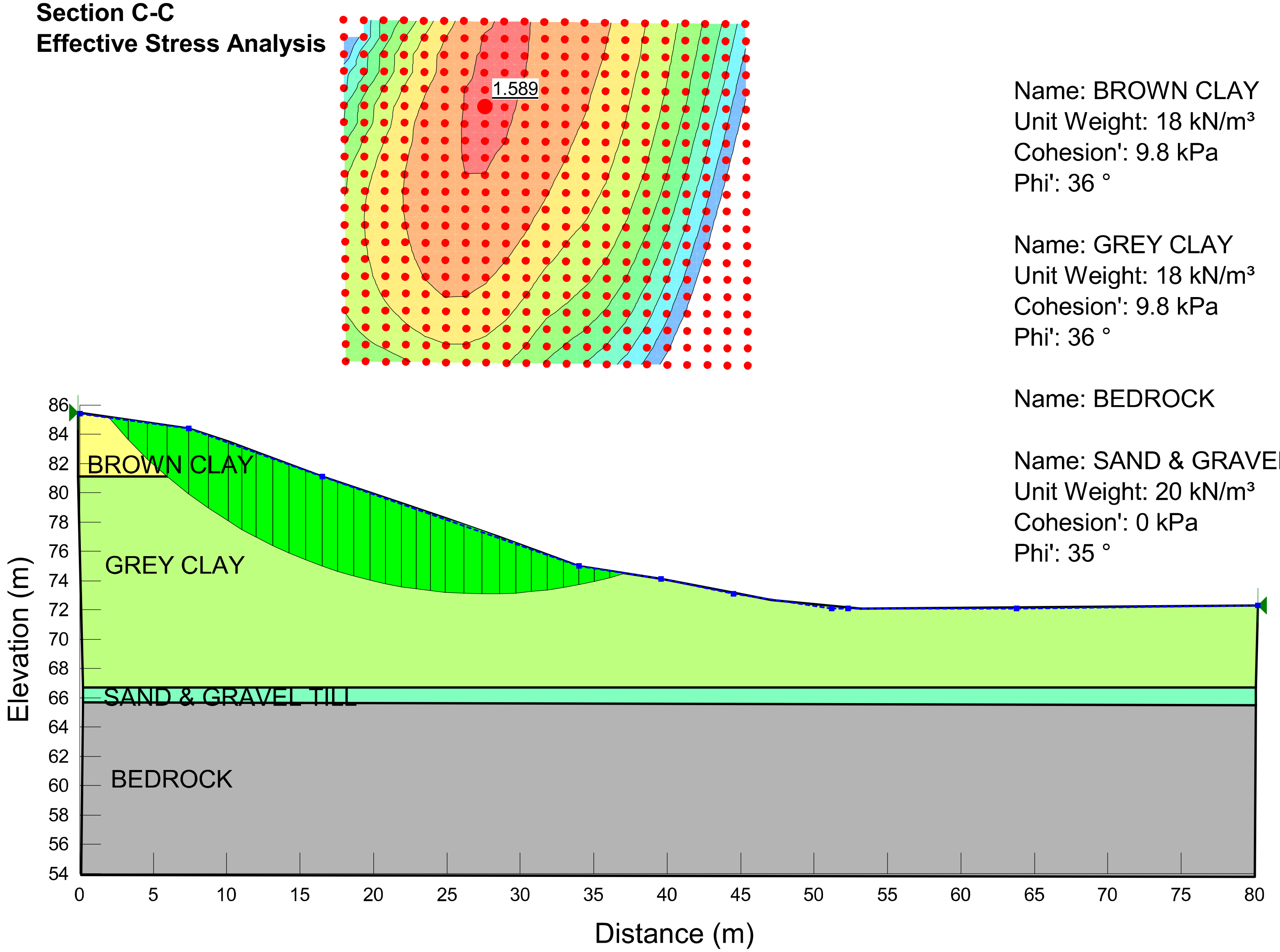
Name: BEDROCK

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 C-C
Effective Stress Analysis

Figure No. 30



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

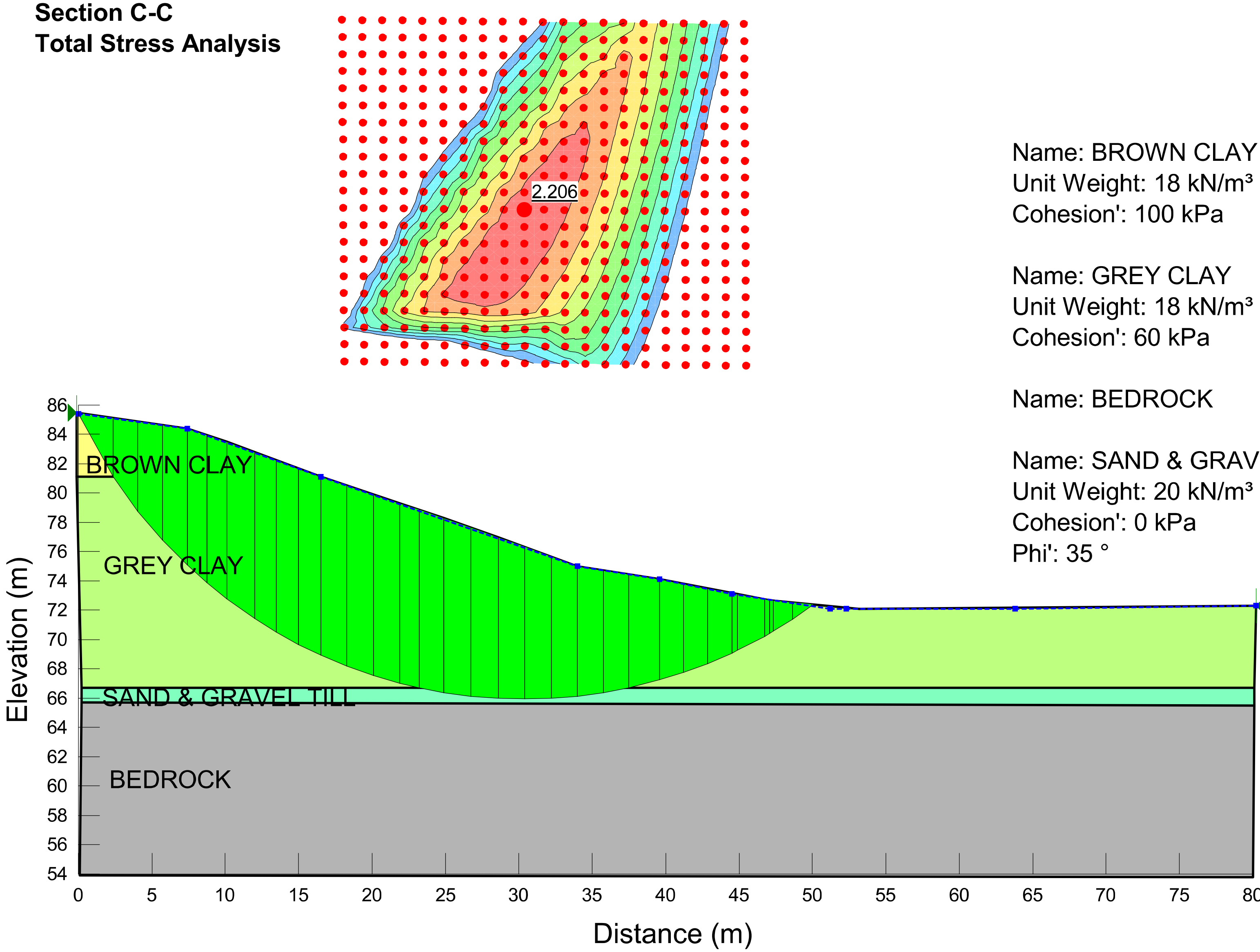
Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 9.8 kPa
Phi': 36 °

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 C-C
Total Stress Analysis

Figure No. 31



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

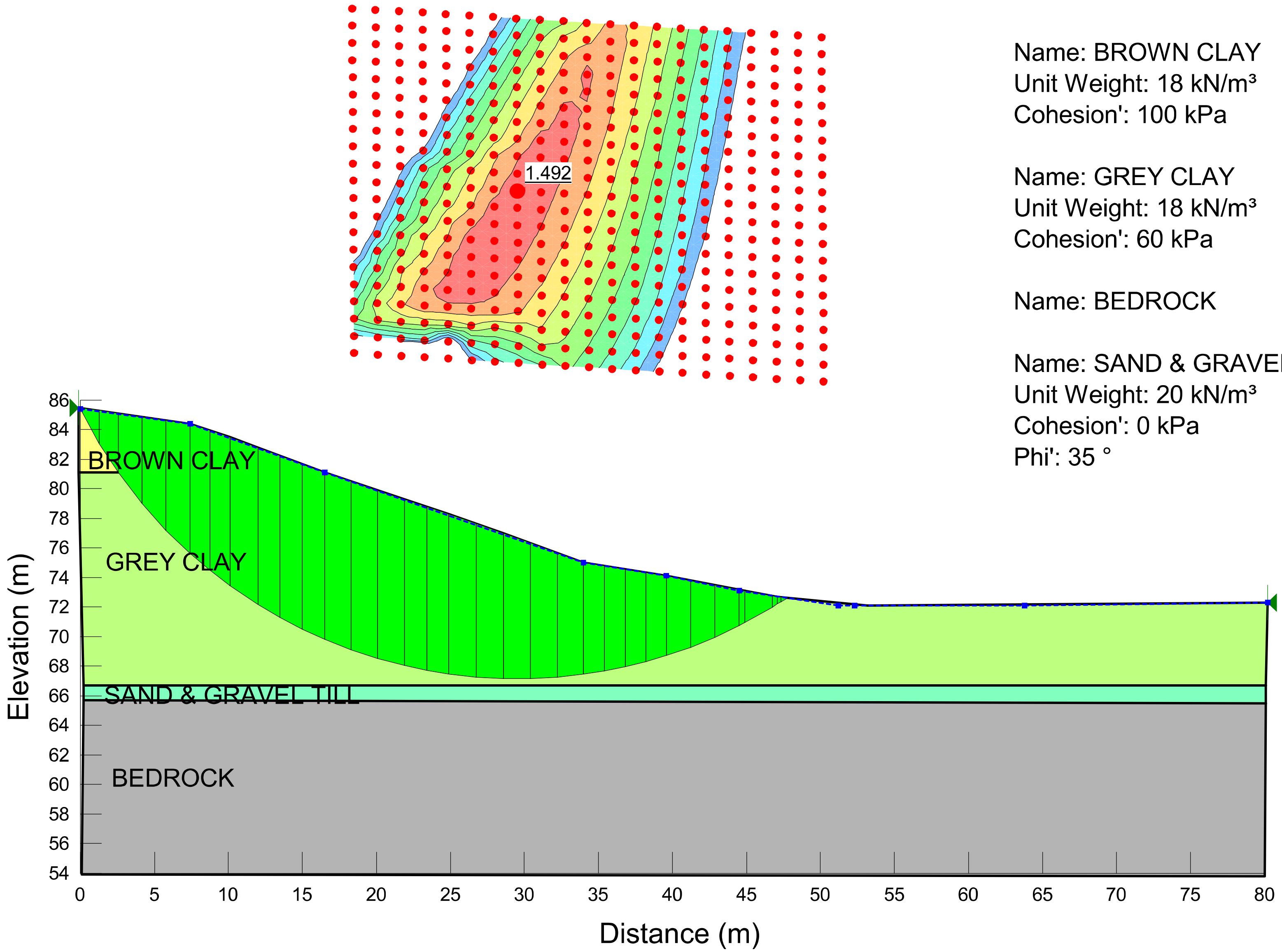
Name: GREY CLAY
Unit Weight: 18 kN/m³
Cohesion': 60 kPa

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 C-C
Total Stress Analysis - Seismic

Figure No. 32



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

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

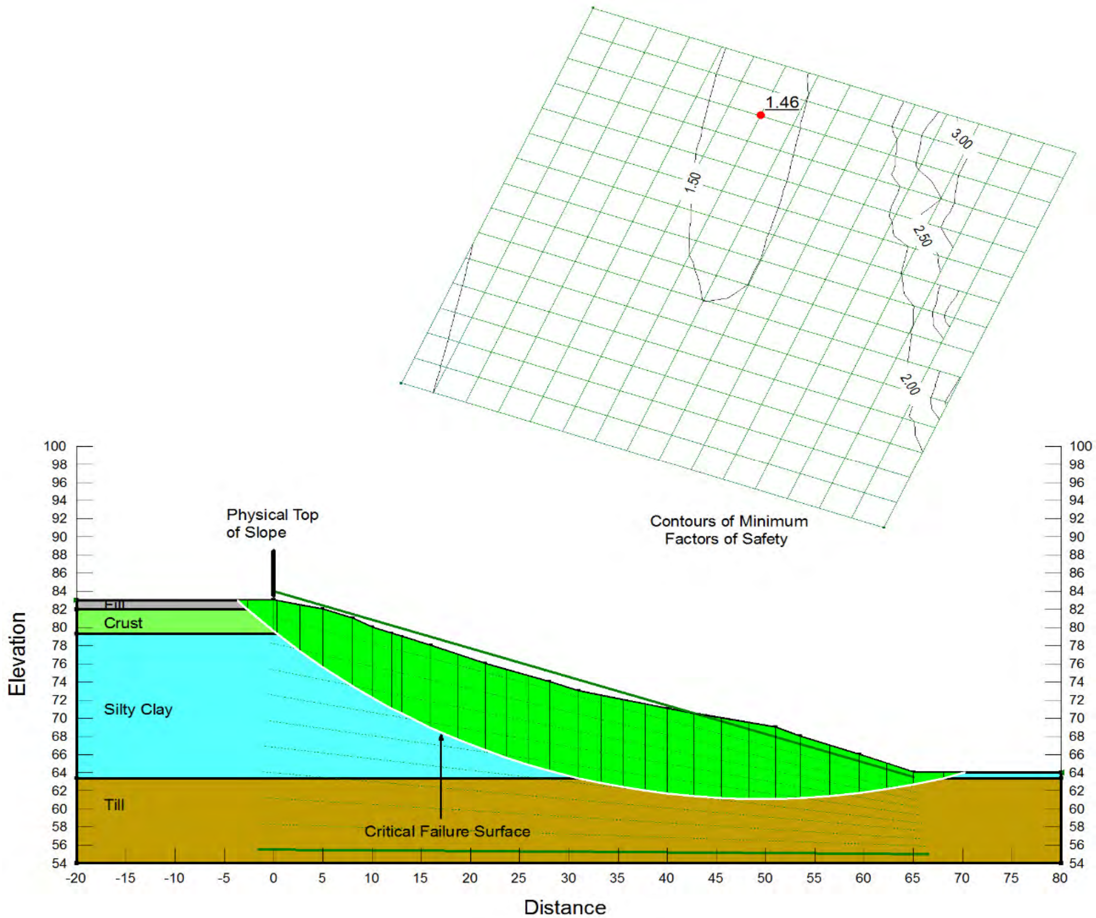
Name: BEDROCK

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

STATIC SLOPE STABILITY ANALYSIS
Effective Stress Analysis
1154 - 1208 Old Montreal Rd, Ottawa

OTT-00234493-A0

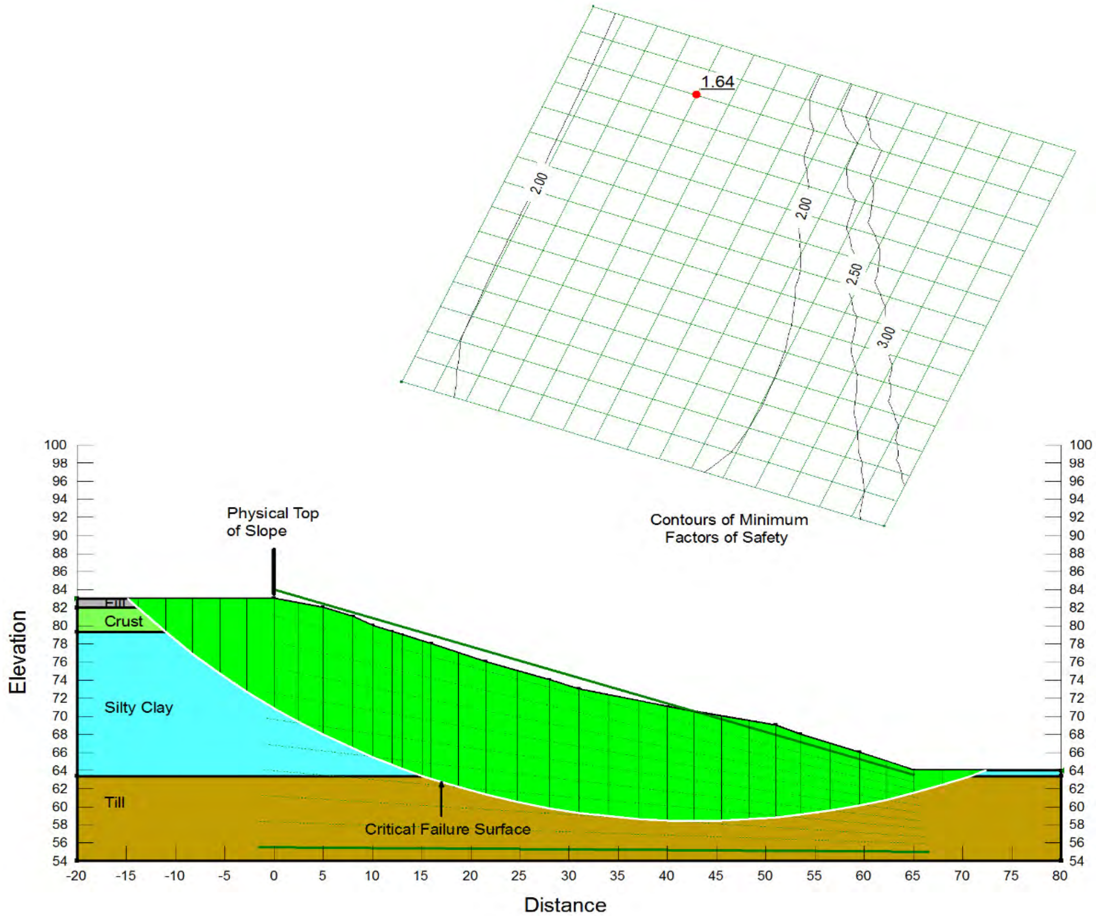
Figure No. 33



Section / Location	:	Section D-D	
Slope	:	Forward	3.4H:1V
Height	:	19.0 m	
Mid-height Berm	:	N/A	
Water Table	:	Pore-water Pressure Ratio, Ru	
Drained Condition	:	Drained	
Analysis Method	:	Morgenstern - Price	
Surcharge	:	N.A.	

Stratum	γ (kN/m^3)	c (kPa)	ϕ (^{o})
Fill	20.0	0	30
Crust	18.0	9.8	36
Silty Clay	18.0	9.8	36
Till	20.0	0	35

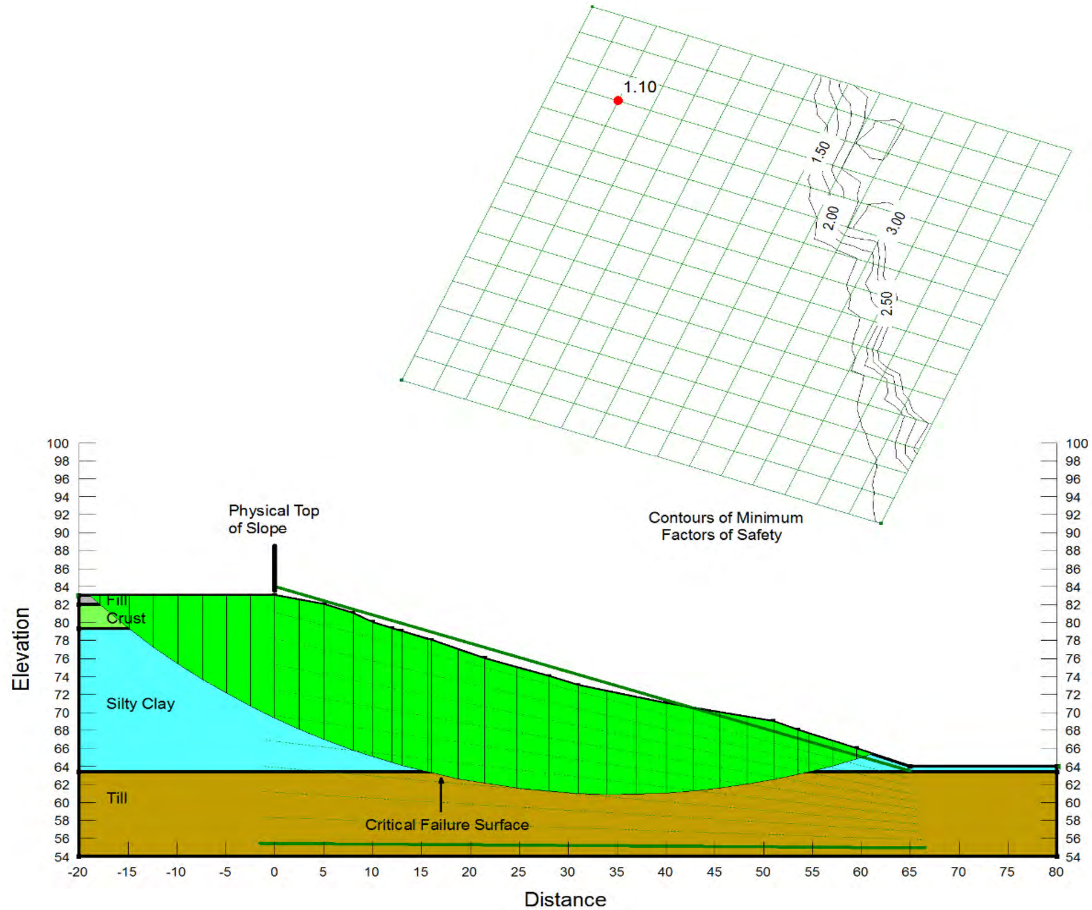
STATIC SLOPE STABILITY ANALYSIS **OTT-00234493-A0**
Total Stress Analysis
1154 - 1208 Old Montreal Rd, Ottawa **Figure No. 34**



Section / Location	:	Section D-D (NW Facing Slope)
Slope	:	Forward 3.4H:1V
Height	:	19.0 m
Mid-height Berm	:	N/A
Water Table	:	Pore-water Pressure Ratio, Ru
Drained Condition	:	Undrained
Analysis Method	:	Morgenstern - Price
Surcharge	:	N.A.

Stratum	γ (kN/m^3)	c (kPa)	ϕ (^{o})
Fill	20.0	0	30
Crust	18.0	100	-
Silty Clay	18.0	60	-
Till	20.0	0	35

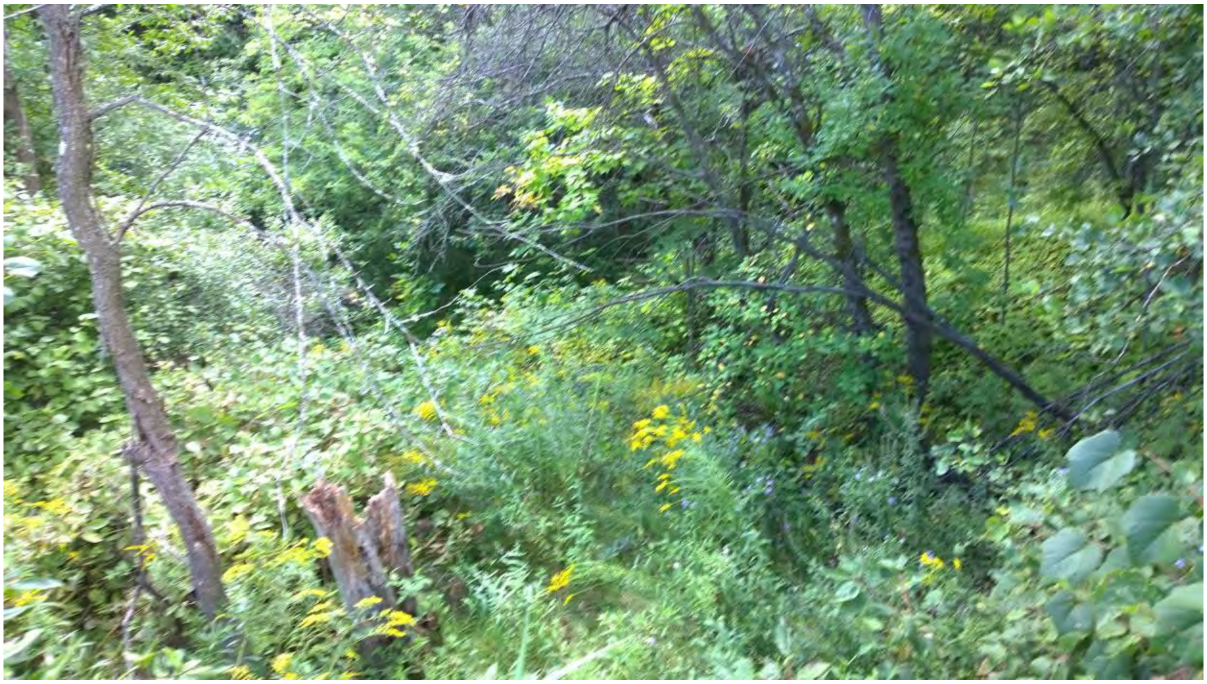
PSEUDO-STATIC SLOPE STABILITY ANALYSIS **OTT-00234493-A0**
Total Stress Analysis with Seismic Loading
1154 - 1208 Old Montreal Rd, Ottawa **Figure No. 35**



Section / Location	:	Section D-D (NW Facing Slope)
Slope	:	Forward 3.4H:1V
Height	:	19.0 m
Mid-height Berm	:	N/A
Water Table	:	Pore-water Pressure Ratio, Ru
Drained Condition	:	Undrained
Analysis Method	:	Morgenstern - Price
Surcharge	:	N.A.

Stratum	γ (kN/m^3)	c (kPa)	ϕ (^{o})
Fill	20.0	0	30
Crust	18.0	100	-
Silty Clay	18.0	60	-
Till	20.0	0	35

Appendix A: Photos of Erosion Along Creek Bank



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

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

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

Photograph No.4

View looking south from north side of pathway near creek (Location 4 on Figure 2)

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

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

Photograph No. 7
View of creek looking west from east bank of creek at Location 7 on Figure 2

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

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

List of Distribution

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