Geotechnical Investigation

251 Penfield Drive
Ottawa Ontario

Ottawa Community Housing Corporation

GHD | 179 Colonnade Road Suite 400 Ottawa Ontario K2E 7J4 Canada | 11200830 | A1 | Report No 1 | September 30 2019

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1. Introduction

GHD was retained by Ottawa Community Housing Corporation (OCHC) (Client) represented by Mr. Barron Meyerhoffer, to undertake a Geotechnical Investigation, for a proposed new residential development, located at 251 Penfield Drive, in Kanata (Ottawa), Ontario, hereafter referred to as the Site.

The purpose of the investigation was to complete an evaluation of the subsurface stratigraphy on the Site for the proposed residential development and based upon the data, provide recommendations concerning foundation type and associated design bearing pressures , groundwater conditions as well as provide comments on excavation, backfill, pavement design and construction field review.

This report has been prepared with the understanding that the design will be as described in Section 2 and will be carried out in accordance with all applicable codes and standards. Any changes to the project described herein will require that GHD be retained to assess the impact of the changes on the report recommendations provided herein.

The scope of work for GHD consisted of the following activities:

- Underground Service Clearances.
- Fieldwork | The proposed scope included the advancement of a total of seven boreholes within the proposed building and parking lot footprints and installation of two monitoring wells to measure ground water level.
- Lab Testing | Two hydrometer grain size analysis, three Atterberg limit tests, and moisture contents on all collected samples. Chemical testing for corrosion assessment and protection measures for ductile iron and concrete on one collected soil sample.
- Reporting | Preparation of this Geotechnical Report which summarizes the findings of the fieldwork programs and presents recommendations for the design and construction of the structure.

2. Site and Project Description

At the time of the investigation, the site was a vacant landscaped lot. The Site is bounded by Penfield Drive to the South, an existing residential building to the East, residential dwellings to the North, and a park to the West. The site topography slopes down from South to North approximately 1.4 metres (m) with slopes along the East and West sides of the property.

GHD understands that the Client intends to construct an eight unit, split level, one and two storey, slab on grade residential building with no basement. The new development will also include a parking area and an access road. It is our understanding that there is no grade changes at the location of the proposed residential units; however there may be about 1 m of grade raise in the parking area to match the surrounding elevation. We understand that as the result of the grade raise within the parking area a maximum 1 m high retaining wall may need to be constructed. GHD has not received any design drawings or survey plans at the time of issuing this report. Once design drawings and topographical survey plans are available GHD should review the drawings and our recommendations.

The location of the Site is shown on the Site Location Plan attached as Figure 1.

3. Field Investigation

The fieldwork program consisted of the advancement of seven boreholes labelled as BH1, BH1B, BH2, BH2B, and BH3 to BH5. Boreholes were advanced to depths varying between 1.4 m to 6.1 m below the existing surface grade. Two monitoring wells were installed in boreholes BH3 and BH4. All monitoring wells were sealed within the overburden. The location of the boreholes are shown in the Borehole Location Plan attached as Figure 2 at the end of this report.

The borehole drilling fieldwork program was undertaken on August 22, 2019 with a geoprobe drill rig, under the supervision of GHD field staff.

Boreholes were advanced into the overburden using Standard Penetration Tests (SPTs) at regular intervals using a 50 millimeter (mm) diameter split-spoon sampler and a 63.5 kilogram (kg) hammer for a truck mounted drill rig, free falling from a distance of 760 mm, to collect soil samples. The number of drops required to drive the sampler 0.3 m recorded on the borehole logs as "N" value. In-situ Field Vane Test (FVT) was also carried out where applicable. Monitoring wells were installed in two of the boreholes for further groundwater measurement and testing. All boreholes were backfilled with bentonite hole plug and silica sand upon drilling completion. Auger cuttings were spread evenly across the ground surface.

The elevations of the boreholes were determined by GHD field staff using a laser level; and related to an assigned benchmark on Site which was the top of the first floor of the west entrance to the neighboring building at 231 Penfield Drive. This benchmark was assumed to have an arbitrary elevation of 100 m. The elevations of the boreholes are not geodetic and are for use within the context of this report only.

3.1 Laboratory testing

Laboratory testing on recovered soil samples included two Hydrometer Grain Size Analysis, three Atterberg limit tests, and moisture contents on all collected samples. The results from the testing assisted in the subsoil descriptions provided below in Section 4 and on the borehole logs. The laboratory test results are also provided in Appendix B, at the end of this report.

Analytical testing was carried out on a soil sample collected to determine corrosion potential within the subsurface to new ductile iron and buried concrete soils at the site. The results of the chemical analyses are discussed in Section 6.10.

4. Subsurface Conditions

In general, soils encountered at the borehole locations at the south side of the Site and within the building area consisted of a layer of silty sand overlying native silty clay deposit. Toward the north end of the Site and at the location of the access road and parking area, a layer of fill material consisting of a mixture of sand, silt and clay was encountered from the surface to the total depth of the boreholes.

General descriptions of the subsurface conditions are summarized in the following sections, with a graphical representation of each borehole on the Borehole Logs. Notes on Boreholes are provided in Appendix A, at the end of this report.

4.1 Surface Covers

The ground surface at the site consists of topsoil with an approximate thickness of 120 mm. Classification of this material was based solely on visual and textural evidence. Laboratory testing to measure organic content or other constituents was not carried out.

4.2 Fill

A layer of fill was encountered at all borehole locations. The fill material consisted of a silty sand at the south end of the site changing to a clayey silty sand at the north end of the Site. Fill material was found to be very loose to compact, and in a damp to moist condition. The thickness of the fill layer varied from approximately 1.5 m at the south end of the Site to 3.8 m at the north end of the Site. Gradation analysis completed on a selected sample of the fill material from borehole BH1B indicated that the tested sample contained 7 percent gravel, 36 percent sand, 24 percent silt and 33 percent clay. The particle distribution curves are presented in Appendix B.

4.3 Silty Clay

Underlying the fill layer at the borehole BH3 and BH4 locations, a native silty clay deposit was encountered. The deposit contained varying amounts of sand with depth. In general this deposit was found to be stiff to very stiff and was recovered in a damp condition becoming saturated with depth. The shear strength value and the remoulded values determined from field vane test (FVT), indicated that this deposit is classified as sensitive clay.

Gradation analysis conducted on one selected sample of the native fine-grained soils indicated that the tested samples contained 55 percent silt, and 45 percent clay size particles. The fines content (silt and clay particles) of the tested samples was 100 percent. The particle distribution curves are presented in Appendix B.

Atterberg limit tests were conducted on the two selected soil samples. The tested samples had liquid limit value of 61 percent, plastic limit was 20 to 23 percent and plasticity index values ranging between 38 and 41 percent. The natural moisture content of the tested soil samples were 37 and 56 percent.

The water content values of select samples of the fine-grained soils ranged between 11 and 56 percent. The extracted samples were generally described as moist to wet.

4.4 Refusal to Auger Advancement

Practical refusal to auger advancement was encountered in boreholes BH1, BH1B, BH2, BH2B and BH3 that were located within the proposed parking area. The refusal depth ranged from 2.9 meters below ground surface (mbgs) (BH5) to 3.8 mbgs (BH1B). The refusals are assumed to be bedrock however this could not be confirmed as diamond coring was not part of the scope of work for this project.

5. Groundwater

Two monitoring wells were installed as part of the scope of work. Groundwater levels were measured on August 29, 2019, at the monitoring wells. The following Table 5.1 shows the measured water levels.

Table 5.1 Groundwater Observations

Borehole No. (BH)	Depth of Water Below Existing Grade (m)	Elevation (m)*
BH3	3.5	96.8
BH4	3.4	96.9
Notes:*Elevations are not geodetic		

These levels indicated the water is within the native silty clay deposit. However, it should be noted that the groundwater table is subject to seasonal fluctuations and in response to precipitation and snowmelt events.

6. Discussion and Recommendations

The recommendations in this report are based on GHD's understanding of the proposed development, which is outlined as follows:

- The proposed structure will consist of one to two storey slab-on-grade residential units. GHD understands that no underground levels (basement or parking) are proposed.
- A founding depth for the foundations of about 1.5 m below current ground surface and the foundations will be conventional pad and strip type.
- Floor slab is lightly loaded (less than 24 kilopascal [kPa]).
- No grade raises are planned at or adjacent to the proposed residential units. A grade raise of approximately 1 m is expected within the parking area.
- We understand construction of a retaining wall up to 1 m high may be required as part of the grade raise within the parking area.

Based on the subsurface conditions encountered in the boreholes, and assuming them to be representative of the subsurface conditions across the Site, the following recommendations are provided. Significant geotechnical considerations for design and construction of the proposed structure are:

- Soil Disturbance | The clayey soil on Site are subject to softening if disturbed or exposed to standing water for an extended period of time. Contractors will need to employ suitable measures, such as mud slab to protect the approved subgrade from disturbance during footing construction.

- Grade Raise | Based on the existing grade of the Site, GHD assumes that an approximately 1.0 m grade raise is expected at the proposed new parking lot location at the north end of the Site in order to match the grade of the adjacent property. This grade raise will result in minor settlements of approximately 15 mm. If grade raises more than 1 m is expected GHD should be informed to revise our analysis and further geotechnical fieldwork and laboratory testing may be required depending on the amount of the proposed grade raise.

6.1 Site Preparation

Site preparation within the new building footprint will involve the removal of existing vegetation, topsoil and any existing fill materials to expose the native silty clay. The exposed surface should be examined by geotechnical personnel to assess the competency.

Any identified local anomalies should be excavated and replaced with suitable engineering fill. The backfilling material and the placement and the compaction of the material should follow the instructions provided in Section 6.10.1 of this report.

The soils at this location are subject to strength loss upon disturbance, especially when these soils are subjected to elevated moisture content. Disturbed soils will not be suitable and will need to be removed. Specifications should make some allowance for this issue, but contractors will need to use construction practices, methods and equipment that minimize the risk of disturbance. It is recommended that a mud slab be placed on the approved subgrade to prevent disturbance and protect the bearing surface during footing construction.

The construction should ensure control of surface water, directing it away from excavations. An adequate ditching and pumping system may be necessary in order to collect any surface runoff or groundwater infiltration.

In the proposed pavement areas the site preparation will involve removal of existing topsoil. Existing fills may remain in place under the proposed pavement areas as long as they are proven competent. The exposed subgrade surface should be compacted following excavation, proof rolled and examined by geotechnical personnel to assess the competency and any identified local anomalies (over size materials) or soft spots should be subsequently excavated, replaced with suitable fill, and compacted. Field verification should be carried out by qualified geotechnical personnel during construction. Detailed recommendations regarding the pavement subgrade preparation is provided in Section 6.12 of this report.

6.2 Excavation and Dewatering

All excavations should be completed and maintained in accordance with the Occupational Health and Safety Act (OHSA) requirements. The following recommendations for excavations should be considered to be a supplement to, not a replacement of, the OHSA requirements.

Based on the results of the investigation, overburden soil material within excavation would be considered as 'Type 3 Soils', above groundwater level and 'Type 4' at and below groundwater level as defined by the OHSA Regulations for Construction.

It is recommended that the client's design team request in the specification package that contractors submit Excavation Plans and Soil Management Plan for review by the client design team.

As the depth of excavation is expected to be approximately 1.5 mbgs and the recorded water levels range from 3.4 to 3.5 mbgs, groundwater seepage is not expected in the excavations. Water quantities expected to enter the open excavation will depend on seasonal conditions and the duration that excavations are left open.

The clayey soil on Site are subject to softening if disturbed or exposed to standing water for an extended period of time. Contractors will need to employ suitable measures, such as mud slab to protect the approved subgrade from disturbance during footing construction.

6.3 Foundations

The Ontario Building Code (OBC 2012) requires buildings to be designed using Limit States Design values (LSD) of Serviceability Limit States (SLS) and Ultimate Limit States (ULS). It is expected that the foundation for the proposed residential units will be bearing on the native silty clay and will be supported by conventional spread footings.

The recommended bearing pressures are 100 kPa for SLS conditions and 150 kPa for factored ULS condition. This applies for strip footings up to a maximum of 1.0 m wide and pad footings up to 2.0 m in dimension. These values assume footings are founded at a depth of 1.5 mbgs and will bear on the very stiff to stiff native silty clay.

The factored ULS values include the geotechnical resistance factor (ϕ) of 0.5.

If footings are set at varying levels and/or constructed adjacent to utility trenches, they should be constructed such that the higher footings are set at a level below an imaginary line constructed 10H:7V from the base of the lower excavation as stated previously. Step footings should be constructed such that they do not exceed a slope of 2H:1V along their length.

It is recommended that GHD be retained to complete a review for compliance with our recommendations and during construction to verify suitability of subgrade materials.

6.4 Floor Slabs

Conventional slab-on-grade construction is considered suitable for the proposed building. We understand that the building will have light floor loadings only, i.e., considered to be less than 24 kPa. Higher loading requirements will require additional consultation and analysis.

Preparation of the subgrade as discussed in Section 6.1 and 6.2 would include removal of unsuitable overburden materials to expose suitable subgrade and/or the design subgrade level. Any local weakened areas should be excavated and replaced with suitable fill and compacted. Field verification should be carried out by geotechnical personnel during construction.

A layer consisting of Granular 'A' at least 200 mm thick should be placed immediately below the floor slabs to support the slab-on-grade. This layer should be compacted to 100 percent of its SPMDD and placed on approved subgrade surfaces.

If floor coverings are to be used on slab-on-grades then, a vapour barrier is recommended to be incorporated beneath the slab and should be specified by the architect. Floor toppings may also be impacted by curing and moisture conditions of the concrete. Floor finish manufacturer's specifications and requirements should be consulted and procedures outlined in the specifications should be followed.

The slabs should not be tied into the foundation walls. The placement of construction and control joints in the concrete should be in accordance with generally accepted practice.

6.5 Frost Protection

All exterior footings associated with the heated building must be provided with at least 1.5 m of soil cover or its equivalent in insulation, in order to provide adequate protection against detrimental frost action. This cover depth requirement must be increased to 1.8 m for footings for unheated or isolated structures such as signs, entrance canopy, or piers.

Should construction take place during winter, the exposed surfaces to support foundations must be protected by Contractors against freezing.

6.6 Lateral Earth Pressure

Retaining walls at grade changes with adjacent properties are expected at the north end of the Site at the location of the proposed new parking lot. The walls should be designed for lateral pressures resulting from the following sources:

- Unit weight of the backfilled soil.
- Temporary and permanent vertical loads on the completed ground surface.

6.6.1 Static Conditions

The following soil parameters can be used for designing of the retaining walls for lateral earth pressures.

Table 6.1 Soil Parameters and Earth Pressure Coefficients

Soil	Density 'γ' (kN/m³)	Angle of internal Friction	Rankin Earth Pressure Coefficients[(1)] [(2)]		
		φ	Ka	Ko	Kp
Compacted granular backfill such as an OPSS "Granular BI or BII" type product	21	32	0.31	0.47	3.3
Notes: [(1)] Assumes level/flat backfill surface [(2)] For Temporary soils support shoring is required, designers should refer to the CFEM for design assistance					

The existing fill and native materials are not recommended to be used as backfill material for the retaining walls.

For yielding walls the active earth pressure coefficients Ka is recommended to be used.

For non-yielding wall the at-rest Ko should be used.

The resultant of the applicable static or at-rest force is assumed to act at 1/3H above the base of the wall where H is the height of the wall for the permanent wall with free drain backfill material.

These statements are based on the assumption that there is a perimeter drainage system installed at the base of the retaining walls draining under gravity to a frost free outlet, to prevent the build-up of hydrostatic pressure behind the wall; hydrostatic pressures may not be included in the design.

6.7 Slope Stability

Topographical survey of the Site was not available at the time of issuing this report. GHD's understanding of ground surface elevation is based on the limited elevation survey that was carried out by GHD as part of the geotechnical drilling fieldwork and to determine ground surface elevation at the borehole locations and the surrounding area. Based on the elevation survey of the boreholes and adjacent areas, GHD assumes that an approximately 1.0 m grade raise is expected at the proposed new parking lot location at the north end of the Site in order to match the grade of the adjacent property. GHD anticipates a retaining wall may be required at the north end of the parking lot where the grade drops. Since the expected grade raise is 1.0 m or less, a global stability analysis is not required at this time.

Once the design for the retaining wall is complete, it is recommended that GHD reviews the design to provide comment.

6.8 Permanent Drainage

6.8.1 Underfloor Drainage-Slab-on-Grade – no Basement

Under floor drains are not considered necessary for a structure without basement and a floor slab set above the surrounding grades.

6.8.2 Perimeter drainage

Perimeter drainage around the exterior of the walls of the proposed building and the retaining walls is recommended. The drain should be connected to a frost-free outlet for year round drainage.

6.9 Corrosion Potential of Soils

Analytical testing was carried out on one soil sample collected to determine corrosion potential of the subsurface soils at the site. The selected soil sample was tested for pH, resistivity, chlorides, sulphates, and redox potential. The test results are summarized in the following table.

Table 6.2 Corrosion Parameter Results

Sample ID	BH3- SS3
pH	7.34
Resistivity (ohm-cm)	10,800
Redox Potential (mV)	270
Chloride (%)	0.003
Sulfide (µg/g)	<0.20
Sulphate (%)	<0.01

The American Water Works Association (AWWA) publication 'Polyethylene Encasement for Ductile-Iron Pipe Systems' ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. Soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the results obtained for the sample submitted, the Site soils are not considered to be potentially corrosive to ductile iron pipe.

Table 3 of the Canadian Standards Association (CSA) document A23.1-04/A23.2-04 'Concrete Materials and Methods of Concrete Construction/Methods of Test and Standard Practices for Concrete' divides the degree of exposure into the following three classes:

Table 6.3 Classes of Exposure

Degree (Class) of Exposure	Water Soluble (SO_4) in Soil Sample (%)
Very Severe (S-1)	>2.0
Severe (S-2)	0.20 - 2.0
Moderate (S-3)	0.10 - 0.20

A review of the analytical test results shows the sulphate content in the tested samples was found to be less than 0.10 percent. Based upon the test results, the degree of exposure of the subsurface concrete structures to sulphate attack is low. Therefore, normal General Use (GU) hydraulic cement can be used for the below grade concrete structures.

6.10 Building Backfill

The placement and compaction of the materials that will support the foundations and floor slabs, or any interior backfill must be treated as Engineered Fill.

6.10.1 Engineered Fill

The fill operations for Engineered Fill must satisfy the following criteria:

- Engineered Fill must be placed under the continuous supervision of the Geotechnical Engineer.
- Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade proof rolled, and approved. Any deficient areas should be repaired.
- Prior to the placement of Engineered Fill, the source or borrow areas for the Engineered Fill must be evaluated for its suitability. Samples of proposed fill material must be provided to the Geotechnical Engineer and tested in the geotechnical laboratory for Standard Proctor Maximum Dry Density (SPMDD) and grain size, prior to approval of the material for use as Engineered Fill. The Engineered Fill must consist of environmentally suitable soils (as per industry standard procedures of federal or provincial guidelines/regulations), free of organics and other deleterious material (building debris such as wood, bricks, metal, and the like), compactable, and of suitable moisture content so that it is within -2 percent to +0.5 percent of the Optimum Moisture as determined by the Standard Proctor test. Imported granular soils meeting the requirements of Granular 'A', or 'B' Type II OPSS 1010 criteria would be suitable.
- The Engineered Fill must be placed in maximum loose lift thicknesses of 0.2 m. Each lift of Engineered Fill must be compacted with a heavy roller to 100 percent SPMDD.

- Field density tests must be taken by the Geotechnical Engineer, on each lift of Engineered Fill. Any Engineered Fill, which is tested and found to not meet the specifications, shall be either removed or re-compacted and retested.

6.10.2 Exterior Foundation Wall Backfill

Where applicable and/or if necessary, any backfill placed against the foundation walls should be free draining granular materials meeting the grading requirements of OPSS 1010 for Granular 'B' Type I specifications up to within 0.3 m of the ground surface. The upper 0.3 m should be a low permeable soil to reduce surface water infiltration. Foundation backfill should be placed and compacted as outlined below.

- Free-draining granular backfill should be used for the foundation wall.
- Backfill should not be placed in a frozen condition, or placed on a frozen subgrade.
- Backfill should be placed and compacted in uniform lift thickness compatible with the selected construction equipment, but not thicker than 0.2 m. Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures.
- At exterior flush door openings the underside of sidewalks should be insulated, or the sidewalk should be placed on frost walls to prevent heaving. Granular backfill should be used and extended laterally beneath the entire area of the entrance slab. The entrance slab should slope away from the building.
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to at least 98 percent of its SPMDD.
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95 percent of its SPMDD.
- In areas on the building exterior where an asphalt or concrete pavement will not be present adjacent to the foundation wall, the upper 0.3 m of the exterior foundation wall backfill should be a low permeable soil to reduce surface water infiltration.
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall.

6.11 Underground Services

6.11.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials that may be associated with the development.

- Bedding for buried utilities should be OPSS Granular 'A', and placed in accordance with City of Ottawa specifications.
- The cover material should be a sand material or Granular 'A' and the dimensions should comply with City of Ottawa standards.
- The bedding material and cover materials should be compacted as per City of Ottawa standards and to at least 95 percent of its SPMDD.

- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.11.2 Service Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches under landscaped areas, the backfill should be placed and compacted in uniform thickness compatible with the selected compaction equipment and not thicker than 200 mm. Each lift should be compacted to a minimum of 95 percent SPMDD. The backfill placed in the upper 300 mm below a pavement subgrade elevation should be compacted to a minimum of 100 percent SPMDD.
- To reduce the potential for differential frost heave, the selected backfill materials should reasonably match the existing soil profile within the frost penetration zone (1.8 m below finished grade) except that fill with organic matter should not be reused in trenches. Alternatively, if imported backfill, including granular materials, are used then the excavation sides should have frost tapers as per OPSD 800 series which essentially indicates that there should be a back slope of 10:1 (H:V) from the bedding grade to the finished grade.

6.12 Pavement Sections

Access driveways and parking areas are expected to be constructed over existing fill. In order to prepare the site for the pavement area, it is necessary that the area be stripped of any existing cover materials such as surficial topsoil and associated root-mat other deleterious materials deemed unsuitable by geotechnical personnel to expose a suitable subgrade. The exposed subgrade should be proof rolled in the presence of a Geotechnical Engineer. Any areas where "soft spots", rutting, local anomalies, or appreciable deflection are noted should be excavated and replaced with suitable fill and use of geotextiles may be warranted for strength improvement. The fill should be compacted to at least 95 percent of its SPMDD.

Based on the existing grade of the Site, GHD assumes that an approximately 1.0 m grade raise is expected at the proposed new parking lot location at the north end of the Site in order to match the grade of the adjacent property. Refer to Section 6.9.1 for recommended backfill material. This grade raise will result in minor settlements of approximately 15 mm.

The pavement sections described in the table below are recommended for areas subjected to parking lot and access road. Pavement materials and workmanship should conform to the appropriate Ontario Provincial Standard Specifications (OPSS).

Table 6.4 Recommended Pavement Structure

Pavement Layer	Minimum Thickness	Heavy Duty (Access Roads)
HL3 Asphalt	50 mm	40 mm
HL8 Asphalt	n/r	50 mm
Granular 'A' Base Course	150 mm	150 mm

Table 6.4 Recommended Pavement Structure

Pavement Layer	Minimum Thickness	Heavy Duty (Access Roads)
Granular 'B', Type II Sub-Base Course	300 mm	450 mm

In order to accommodate the recommended thicknesses, designers will need to review grades and determine where stripping or filling is necessary. Pavement materials and workmanship should conform to the appropriate OPSS.

Minimum Performance Grade (PG) at 58 – 34 should be used at this Site.

Drainage of the pavement layers is important. The subgrade surface and each layer of the pavement section should be provided with a suitable cross fall (approximately 2 percent) to prevent water from ponding on the pavement surface and beneath the pavement layers. Surface runoff should be directed to storm sewers, or allowed to flow into ditches.

Where the new pavement abuts existing and the subgrade levels vary between the two areas, then a frost transition should be integrated into the subgrade with a 10:1 slope in the subgrade. Sufficient field-testing should be carried out during construction to assess compaction of each lift of the pavement layers. This should be accompanied by laboratory testing of the granular and asphalt materials. All granular base course materials should be compacted to 100 percent of its SPMDD.

Annual or regular maintenance will be required to achieve maximum life expectancy. Generally, the asphalt pavement maintenance will involve crack sealing and repair of local distress.

It should be noted that the pavement sections described within this report represent end-use conditions only, which includes light vehicular traffic and occasional garbage or service trucks. It may be necessary that these sections be temporarily over-built during the construction phase to withstand larger construction loadings such as loaded dump trucks or concrete trucks.

6.13 Construction Field Review

The recommendations provided in this report are based on an adequate level of construction monitoring being conducted during construction phase of the proposed building. GHD requests to be retained to review the drawings and specifications, once complete, to verify that the recommendations within this report have been adhered to, and to look for other geotechnical problems. Due to the nature of the proposed development, an adequate level of construction monitoring is considered to be as follows:

- Prior to construction of footings, the exposed foundation subgrade should be examined by a Geotechnical Engineer or a qualified Technologist acting under the supervision of a Geotechnical Engineer, to assess whether the subgrade conditions correspond to those encountered in the boreholes, and the recommendations provided in this report have been implemented.
- A qualified Technologist acting under the supervision of a Geotechnical Engineer should monitor placement of Engineered Fill underlying floor slabs.

- Backfilling operations should be conducted in the presence of a qualified Technologist on a part time basis, to ensure that proper material is employed and specified compaction is achieved.
- Placement of concrete should be periodically tested to ensure that job specifications are being achieved.

7. Limitation of the Investigation

This report is intended solely for Ottawa Community Housing Corporation and other party explicitly identified in the report and is prohibited for use by others without GHD's prior written consent. This report is considered GHD's professional work product and shall remain the sole property of GHD. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to GHD. Client shall defend, indemnify and hold GHD harmless from any liability arising from or related to Client's unauthorized distribution of the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevations and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of Geotechnical Engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, GHD will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, GHD is the Geotechnical Engineer of record. It is recommended that GHD be retained during construction of all foundations and during earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

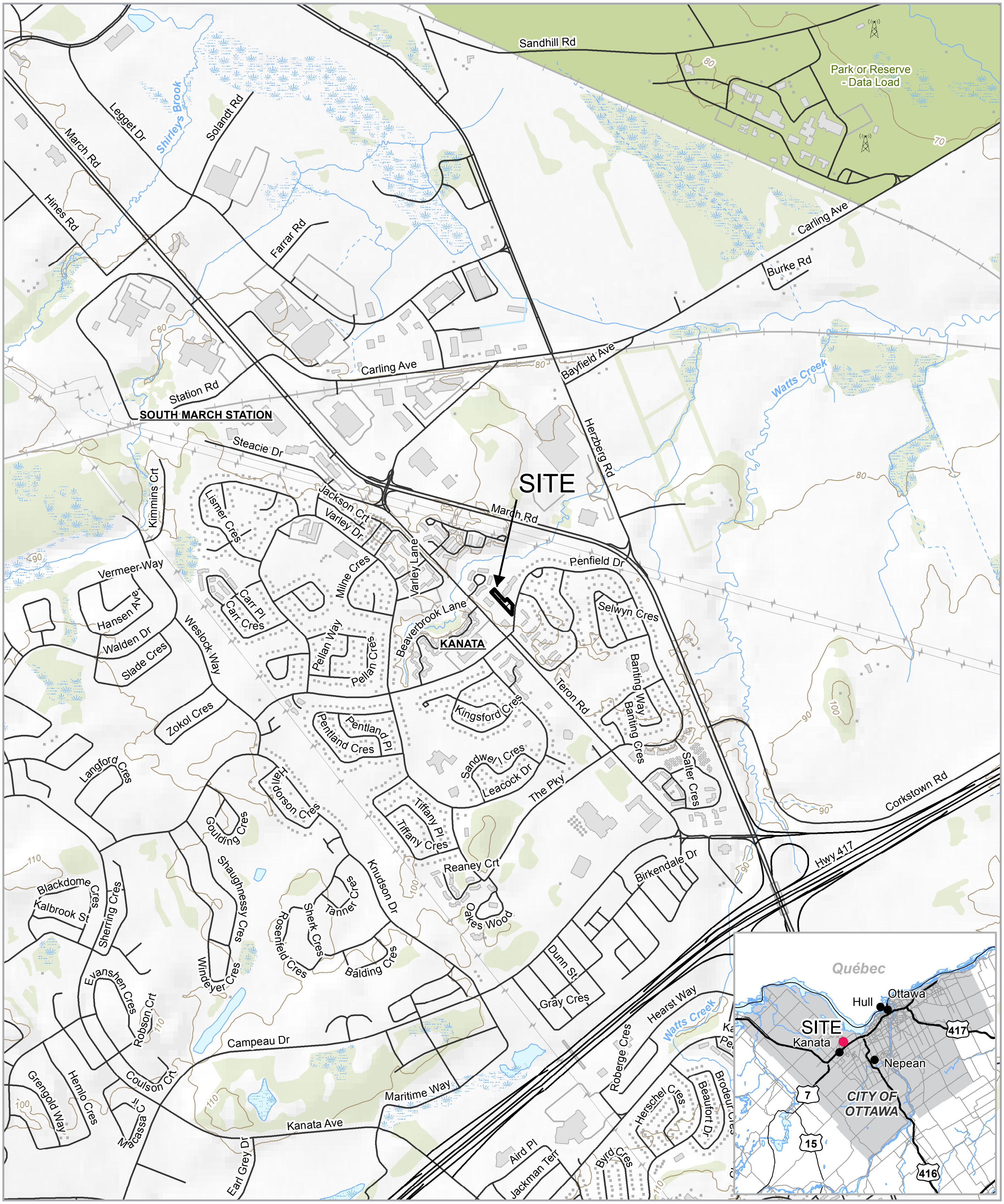
It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the seven test hole locations only. The subsurface conditions confirmed at these seven test locations may vary at other locations. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction, which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

All of Which is Respectfully Submitted,

GHD

Ryan Vanden Tillaart, EIT

Joseph B. Bennett, P. Eng.



Paper Size ANSI A

0 140 280 420 560

Meters

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 18N

N

GHD

OTTAWA COMMUNITY HOUSING CORPORATION
251 PENFIELD DRIVE, OTTAWA, ON
GEOTECHNICAL INVESTIGATION

Project No. **11200830**
Revision No. **-**
Date **Aug 26, 2019**

SITE LOCATION MAP

FIGURE 1

Q:\GIS\PROJECTS\11200000s\11200830\Layouts\201908_GeotechInvestigation\11200830_201908GeotechInvestigation_GIS001.mxd
Print date: 26 Aug 2019 - 15:39

OTTAWA COMMUNITY HOUSING CORPORATION
251 PENFIELD DRIVE, OTTAWA, ON
GEOTECHNICAL INVESTIGATION

Project No. 11200830
Revision No. -
Date Aug 27, 2019

BOREHOLE LOCATION PLAN

FIGURE 2

Data source: Image ©2019 Google, Imagery date: 2018/06/01.

Appendices

Appendix A
Borehole Logs and Notes on Boreholes

REFERENCE No.: 11200830-A1 ENCLOSURE No.: 1

BOREHOLE No.: BH1

ELEVATION: 98.88 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 251 Penfield Drive, Ottawa, ON

DESCRIBED BY: R.Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 22 August 2019 DATE (FINISH): 22 August 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	98.88		GROUND SURFACE					
			TOPSOIL					
	98.8		FILL - Silty sand, some gravel, loose, brown, damp, wood chips and grass rootlets		SS1	58		7
	98.1		FILL - Silty clayey sand, trace gravel, compact, brown and grey, damp to moist Spoon and auger refusal encountered at 1.4 mbgs Possible boulder		SS2	54		16
	97.5		Borehole terminated at 1.4 mbgs					

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

REFERENCE No.: 11200830-A1 ENCLOSURE No.: 2

BOREHOLE No.: BH1B

ELEVATION: 98.99 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 251 Penfield Drive, Ottawa, ON

DESCRIBED BY: R.Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 22 August 2019 DATE (FINISH): 22 August 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	98.99	GROUND SURFACE				
	98.9	TOPSOIL				
		FILL - Silty sand, some gravel, loose, brown, damp	SS1	58		8
	98.2	FILL - Silty clayey sand, trace gravel, very loose to loose, grey, moist to saturated	SS2	25		8
			SS3	33		PH
		Organic staining at 2.4 mbgs	SS4	25		6
		wood chips encountered	SS5	17		7
	95.2	Spoon and auger refusal encountered at 3.8 mbgs				
		Borehole terminated at 3.8 mbgs				

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

REFERENCE No.: 11200830-A1 ENCLOSURE No.: 3

GHD	BOREHOLE No.: BH2 ELEVATION: 99.71 m	BOREHOLE LOG Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 251 Penfield Drive, Ottawa, ON

DESCRIBED BY: R.Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 22 August 2019 DATE (FINISH): 22 August 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

SCALE FOR TEST RESULTS: 50kPa, 100kPa, 150kPa, 200kPa; 10, 20, 30, 40, 50, 60, 70, 80, 90

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	State	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	99.71	GROUND SURFACE					
	99.6	TOPSOIL					
		FILL - Silty sand, some gravel, compact, brown, damp	SS	SS1	71		13
0.5	99.2	FILL - Sand, very loose, brown, damp					
	99.1	FILL - Silty clayey sand, trace gravel, very loose to compact, brown and grey, damp					
1.0			SS	SS2	8		1
1.5		becoming moist					
			SS	SS3	38		14
2.0							
2.5		becoming saturated	SS	SS4	17		1
3.0	96.7	Borehole terminated at 3.0 mbgs					

Depth scale: 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

NOTES:
mbgs: meters below ground surface
Drill not plumb; drill moved to BH2B location; refusal encountered at 3.0 mbgs

REFERENCE No.: 11200830-A1 ENCLOSURE No.: 5

BOREHOLE LOG

BOREHOLE No.: BH3
ELEVATION: 100.28 m

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation
PROJECT: Geotechnical Investigation
LOCATION: 251 Penfield Drive, Ottawa, ON
DESCRIBED BY: R.Vanden Tillaart CHECKED BY: B. Vazhbakht
DATE (START): 22 August 2019 DATE (FINISH): 22 August 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Monitor Well	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	100.28	GROUND SURFACE					
	100.2	TOPSOIL	100.20				
0.5		FILL - Silty sand, some gravel, compact, brown, damp, construction debris (brick)	Sand	SS1	67		23
1.0	99.5	FILL - Clayey silty sand, trace gravel, compact, brown, damp	0.91	SS2	63		15
1.5	98.8	SILT AND CLAY- trace sand, stiff, brown and grey, damp					
2.0			Riser	SS3	100		9
2.5			Bentonite	SS4	100		8
3.0		becoming moist					
3.5			WL 3.49 8/29/2019	SS5	100		3
4.0				SS6	100		1
4.5			4.27 Sand 4.57				
5.0		becoming saturated		SS7	100		1
5.5			Screen	SS8	100		PH
6.0	94.2	Borehole terminated at 6.1 mbgs	6.10				
6.5							

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface
Pocket penetrometer values for GHD internal use only

REFERENCE No.: 11200830-A1 ENCLOSURE No.: 6

BOREHOLE No.: BH4

ELEVATION: 100.27 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 251 Penfield Drive, Ottawa, ON

DESCRIBED BY: R.Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 22 August 2019 DATE (FINISH): 22 August 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Monitor Well
	100.27	GROUND SURFACE	
		TOPSOIL	100.20
	100.1	FILL - Silty sand, some gravel, compact, brown, damp	Sand
0.5			
	99.5	FILL - Clayey silty sand, trace gravel, compact, brown, damp	0.91
1.0			
1.5	98.7	SILT AND CLAY- trace sand, stiff to very stiff, brown and grey, moist to saturated	Riser
2.0			
2.5			Bentonite
3.0			
3.5			WL 3.39 8/29/2019
4.0			4.27
4.5			Sand 4.57
5.0			
5.5			Screen
6.0	94.2	Borehole terminated at 6.1 mbgs	6.10
6.5			

SAMPLE DATA

Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)	Test Results
SS1	75		13	
SS2	79		10	
SS3	88		8	
SS4	100		6	
SS5	100		4	
FV6				S=7.3
SS7	100		PH	
FV8				S=4.5

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface
Pocket penetrometer values for GHD internal use only

REFERENCE No.: 11200830-A1 ENCLOSURE No.: 7

BOREHOLE No.: BH5
ELEVATION: 99.36 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation
PROJECT: Geotechnical Investigation
LOCATION: 251 Penfield Drive, Ottawa, ON
DESCRIBED BY: R.Vanden Tillaart CHECKED BY: B. Vazhbakht
DATE (START): 22 August 2019 DATE (FINISH): 22 August 2019

LEGEND

- ☒ SS Split Spoon
- GS Auger Sample
- ▨ ST Shelby Tube
- ▼ Water Level
- ○ Water content (%)
- ├─┤ Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	State	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	99.36	GROUND SURFACE					
		TOPSOIL					
	99.2	FILL - Silty sand, some gravel, loose, brown, damp, construction debris (brick)	☒	SS1	71		8
	98.6	FILL - Silty clayey sand, trace gravel, very loose to loose, brown and grey, damp becoming saturated	☒	SS2	58		9
		becoming moist	☒	SS3	75		8
		Wood chips encountered					
			☒	SS4	33		1
	96.5	Spoon and auger refusal encountered at 2.9 mbgs Borehole terminated at 2.9 mbgs					

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

Notes on Borehole and Test Pit Reports

Soil description :

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey sols is measured by the value of undrained shear strength (Cu).

Classification (Unified system)

Clay	< 0.002 mm		
Silt	0.002 to 0.075 mm		
Sand	0.075 to 4.75 mm	fine	0.075 to 4.25 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel	4.75 to 75 mm	fine	4.75 to 19 mm
		coarse	19 to 75 mm
Cobbles	75 to 300 mm		
Boulders	>300 mm		

Terminology

"trace"	1-10%
"some"	10-20%
adjective (silty, sandy)	20-35%
"and"	35-50%

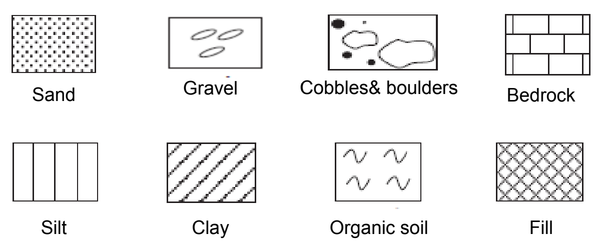
Relative density of granular soils	**Standard penetration index "N" value** (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Consistency of cohesive soils	**Undrained shear strength (Cu)** (P.S.F)	(kPa)
Very soft	<250	<12
Soft	250-500	12-25
Firm	500-1000	25-50
Stiff	1000-2000	50-100
Very stiff	2000-4000	100-200
Hard	>4000	>200

Rock quality designation

"RQD" (%) Value	Quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

STRATIGRAPHIC LEGEND



Samples:

Type and Number

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
SSE, GSE, AGE: Environmental sampling
ST: Shelby tube
PS: Piston sample (Osterberg)
AG: Auger
RC: Rock core
GS: Grab sample

Recovery

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index
R: Refusal to penetration
N_c: Dynamic cone penetration index
Cu: Undrained shear strength
Pr: Pressure meter
k: Permeability
ABS: Absorption (Packer test)

LABORATORY TESTS:

I_p: Plasticity index
W_l: Liquid limit
Wp: Plastic limit
H: Hydrometer analysis
GSA: Grain size analysis
A: Atterberg limits
w: Water content
γ: Unit weight
C: Consolidation
CS: Swedish fall cone
CHEM: Chemical analysis
O.V.: Organic vapor

Appendix B
Laboratory Testing Results

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Ottawa Community Housing Corporation	**Lab no.:**	G-19-006
Project/Site:	251 Penfield, Ottawa, On	**Project no.:**	1120830-A1

Borehole no.:	BH3 SS5	Sample no.:	N/A	Depth:	10' - 12'
Soil description:				Date sampled:	22-Aug-19

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	1
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	1	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	35	27	17
Water Content:			
Tare no.	S12	S13	S14
Wet soil+tare, g	41.84	40.67	40.94
Dry soil+tare, g	34.01	33.09	33.24
Mass of water, g	7.83	7.58	7.70
Tare, g	21.47	21.51	21.77
Mass of soil, g	12.54	11.58	11.47
Water content %	62.4%	65.5%	67.1%

Plastic Limit (PL) - Water Content:

Tare no.	S21	S22
Wet soil+tare, g	26.34	27.12
Dry soil+tare, g	25.40	26.01
Mass of water, g	0.94	1.11
Tare, g	21.42	21.56
Mass of soil, g	3.98	4.45
Water content %	23.6%	24.9%
Average water content %	24.3%	

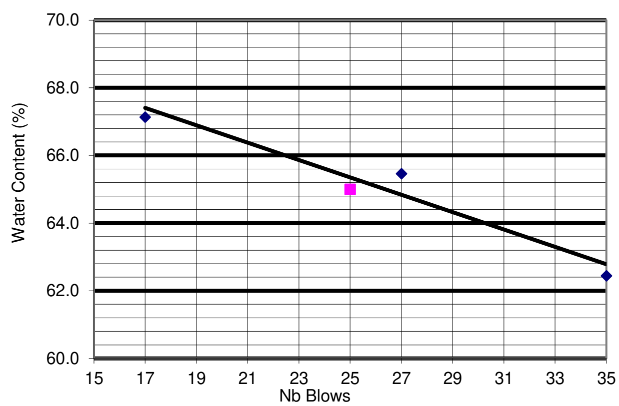
Natural Water Content (W^n):

Tare no.	S40
Wet soil+tare, g	69.70
Dry soil+tare, g	54.10
Mass of water, g	15.60
Tare, g	21.80
Mass of soil, g	32.30
Water content %	48.3%

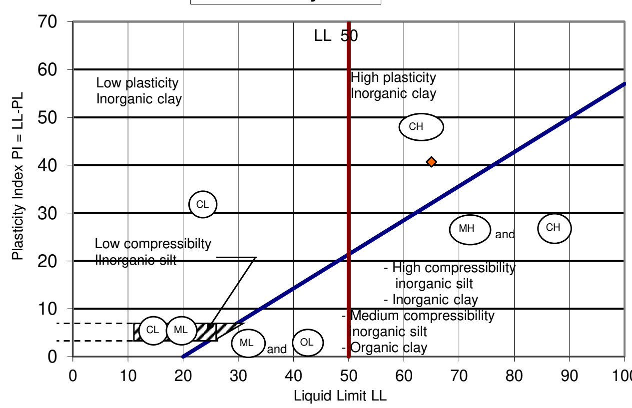
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☐ Dry preparation
- ☑ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
65	24	41	48

Remarks:

Performed by:	E. Bennett/A. Elhaddad	**Date:**	September 5, 2019
Verified by:		**Date:**	September 5, 2019

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Ottawa Community Housing Corporation	**Lab no.:**	G-19-005
Project/Site:	251 Penfield, Ottawa, On	**Project no.:**	11200830-A1

Borehole no.:	BH4SS4	Sample no.:	N/A	Depth:	5' - 7'
Soil description:				Date sampled:	

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	1
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	1	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	27	21	18
Water Content:			
Tare no.	S23	S28	S29
Wet soil+tare, g	39.91	41.92	38.73
Dry soil+tare, g	32.99	34.32	32.17
Mass of water, g	6.92	7.60	6.56
Tare, g	21.53	21.85	21.59
Mass of soil, g	11.46	12.47	10.58
Water content %	60.4%	60.9%	62.0%

Plastic Limit (PL) - Water Content:

Tare no.	S5	S6
Wet soil+tare, g	26.61	26.45
Dry soil+tare, g	25.68	25.59
Mass of water, g	0.93	0.86
Tare, g	21.67	21.81
Mass of soil, g	4.01	3.78
Water content %	23.2%	22.8%
Average water content %	23.0%	

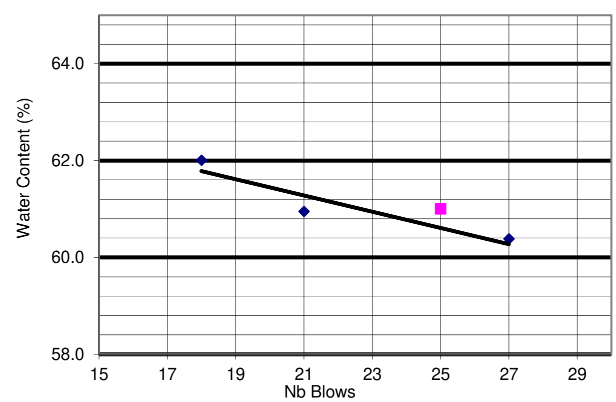
Natural Water Content (W^n):

Tare no.	S10	
Wet soil+tare, g	66.80	
Dry soil+tare, g	54.60	
Mass of water, g	12.20	
Tare, g	21.90	
Mass of soil, g	32.70	
Water content %	37.3%	

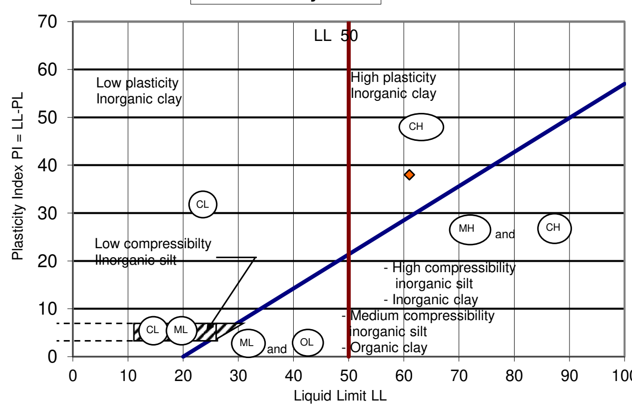
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☐ Dry preparation
- ☑ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
61	23	38	37

Remarks:

Performed by:	E. Bennett/A. Elhaddad	**Date:**	August 5, 2019
Verified by:	[signature]	**Date:**	August 5, 2019

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Ottawa Community Housing Corporation	**Lab no.:**	G-19-005
Project/Site:	251 Penfield, Ottawa, On	**Project no.:**	11200830-A1

Borehole no.:	BH4SS7	Sample no.:	N/A	Depth:	15' - 17'
Soil description:				Date sampled:	22-Aug-19

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	1
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	1	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	30	25	19
Water Content:			
Tare no.	S37	S38	S39
Wet soil+tare, g	42.94	37.12	36.97
Dry soil+tare, g	35.00	31.12	30.94
Mass of water, g	7.94	6.00	6.03
Tare, g	21.65	21.23	21.27
Mass of soil, g	13.35	9.89	9.67
Water content %	59.5%	60.7%	62.4%

Plastic Limit (PL) - Water Content:

Tare no.	S1	S2
Wet soil+tare, g	27.04	27.14
Dry soil+tare, g	26.21	26.26
Mass of water, g	0.83	0.88
Tare, g	22.00	21.76
Mass of soil, g	4.21	4.50
Water content %	19.7%	19.6%
Average water content %	19.6%	

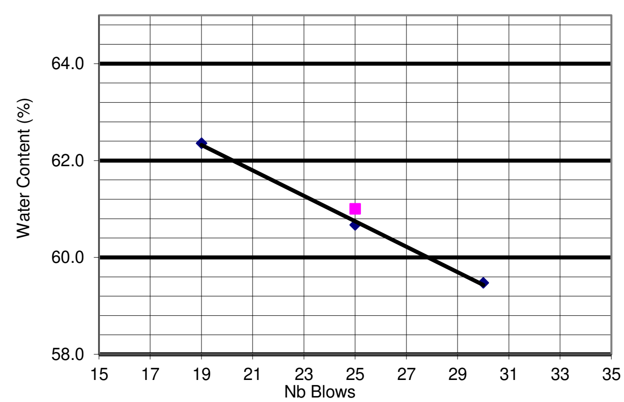
Natural Water Content (W^n):

Tare no.	S12	
Wet soil+tare, g	70.90	
Dry soil+tare, g	53.10	
Mass of water, g	17.80	
Tare, g	21.40	
Mass of soil, g	31.70	
Water content %	56.2%	

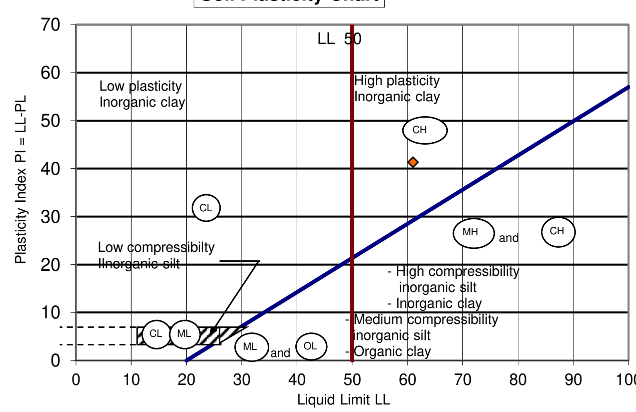
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☐ Dry preparation
- ☑ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
61	20	41	56

Remarks:

Performed by:	E. Bennett/A. Elhaddad	**Date:**	September 5, 2019
Verified by:	E/ A	**Date:**	September 5, 2019

Moisture Content of Soils
(ASTM D2216)

Client: Ottawa Community Housing Corporation
Project: 251 Penfield, Ottawa, On
Location: 251 Penfield, Ottawa, On

Lab No.: G-19-005
Project No.: 11200830-A1

Apparatus Used for Testing

Oven no.: 1 Scale no.: 1

Sample No.	BH1-GS1	BH1-SS2	BH1B-SS1	BH1B-SS2	BH1B-SS3	BH1B-SS4	BH1B-SS5	
Container no.	S15	S26	S1	S5	S2	S20	S28	
Mass of container + wet soil (g)	70.3	69.0	70.1	69.1	78.5	68.7	68.1	
Mass of container + dry soil (g)	63.7	60.9	64.3	61.2	64.3	57.8	55.4	
Mass of container (g)	21.5	21.4	22.0	21.7	21.8	21.9	21.7	
Mass of dry soil (g)	42.2	39.5	42.3	39.5	42.5	35.9	33.7	
Mass of water (g)	6.6	8.1	5.8	7.9	14.2	10.9	12.7	
Moisture content (%)	15.6	20.5	13.7	20.0	33.4	30.4	37.7	

Sample No.	BH2-GS1	BH2-GS2	BH2-GS3	BH2-GS4				
Container no.	S37	S3	S23	S38				
Mass of container + wet soil (g)	69.6	69.8	70.4	68.3				
Mass of container + dry soil (g)	65.3	62.8	62.3	56.1				
Mass of container (g)	21.7	21.9	21.6	21.3				
Mass of dry soil (g)	43.6	40.9	40.7	34.8				
Mass of water (g)	4.3	7.0	8.1	12.2				
Moisture content (%)	9.9	17.1	19.9	35.1				

Remarks:

Performed by: A.Elhaddad **Date:** August 26, 2019

Verified by : E/2 **Date:** September 5, 2019

Moisture Content of Soils
(ASTM D2216)

Client:	Ottawa Community Housing Corporation	**Lab No.:**	G-19-005
Project:	251 Penfield, Ottawa, On	**Project No.:**	11200830-A1
Location:	251 Penfield, Ottawa, On		

Apparatus Used for Testing

Oven no.: 1 Scale no.: 1

Sample No.	BH3-SS1	BH3-SS2	BH3-SS3	BH3-GS1	BH3-GS2	BH3-GS3	BH3-GS4	BH3-GS5
Container no.	S41	S29	S6	S19	S14	S43	S18	S40
Mass of container + wet soil (g)	65.9	75.9	75.6	73.6	73.6	77.7	76.2	69.7
Mass of container + dry soil (g)	61.4	68.4	64.5	59.6	57.5	61.2	62.1	54.1
Mass of container (g)	21.7	21.7	21.8	21.5	21.7	21.6	21.8	21.8
Mass of dry soil (g)	39.7	46.7	42.7	38.1	35.8	39.6	40.3	32.3
Mass of water (g)	4.5	7.5	11.1	14.0	16.1	16.5	14.1	15.6
Moisture content (%)	11.3	16.1	26.0	36.7	45.0	41.7	35.0	48.3
Sample No.	BH4-SS1	BH4-SS2	BH4-SS3	BH4-SS4	BH4-SS5	BH4-SS7		
Container no.	S39	S24	S25	S10	S13	S12		
Mass of container + wet soil (g)	66.7	70.4	77.6	66.8	69.3	70.9		
Mass of container + dry soil (g)	62.1	62.8	66.1	54.6	54.3	53.1		
Mass of container (g)	21.3	22.1	21.5	21.9	22.0	21.4		
Mass of dry soil (g)	40.8	40.7	44.6	32.7	32.3	31.7		
Mass of water (g)	4.6	7.6	11.5	12.2	15.0	17.8		
Moisture content (%)	11.3	18.7	25.8	37.3	46.4	56.2		

Remarks:

Performed by:	A.Elhaddad	**Date:**	August 26, 2019
Verified by :	[signature]	**Date:**	September 5, 2019

Moisture Content of Soils
(ASTM D2216)

Client:	Ottawa Community Housing Corporation	**Lab No.:**	G-19-005
Project:	251 Penfield, Ottawa, On	**Project No.:**	11200830-A1
Location:	251 Penfield, Ottawa, On		

Apparatus Used for Testing

Oven no.: 1 Scale no.: 1

Sample No.	BH5-SS1	BH5-SS2	BH5-SS3	BH5-SS4				
Container no.	S7	S16	S32	S36				
Mass of container + wet soil (g)	71.1	72.1	68.9	71.1				
Mass of container + dry soil (g)	65.5	62.6	60.5	60.6				
Mass of container (g)	21.7	21.5	21.7	22.0				
Mass of dry soil (g)	43.8	41.1	38.8	38.6				
Mass of water (g)	5.6	9.5	8.4	10.5				
Moisture content (%)	12.8	23.1	21.6	27.2				

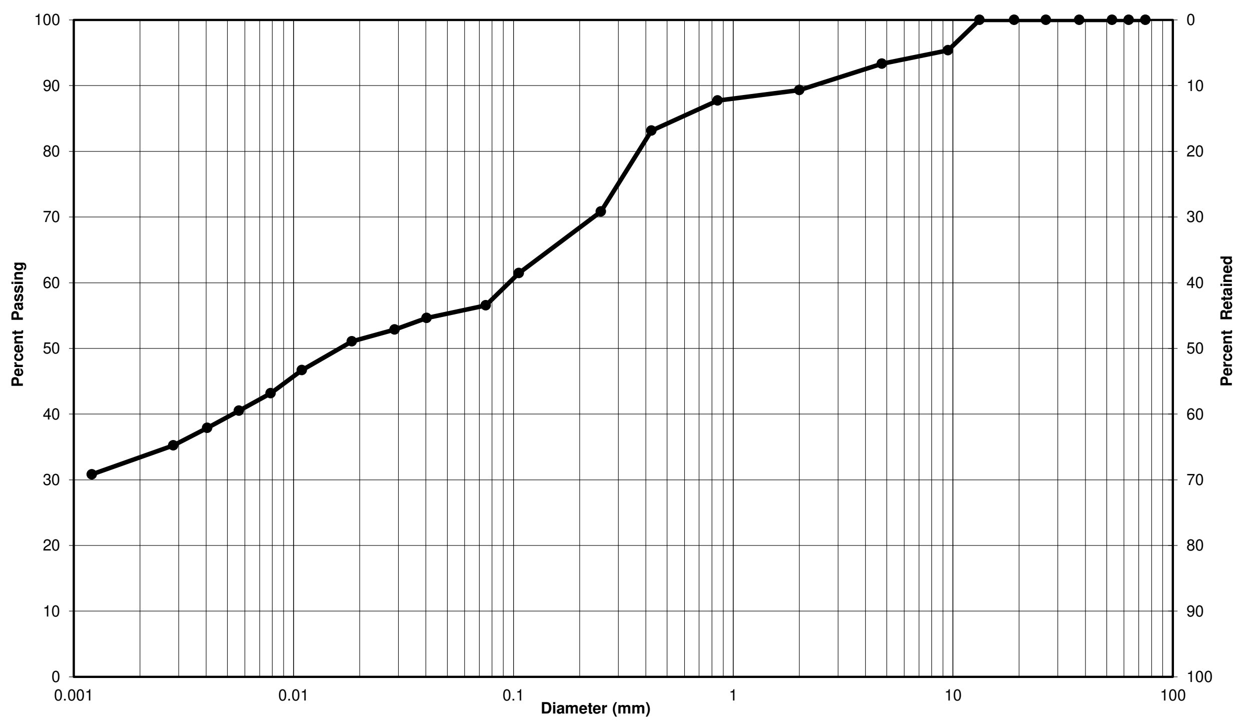
Sample No.								
Container no.								
Mass of container + wet soil (g)								
Mass of container + dry soil (g)								
Mass of container (g)								
Mass of dry soil (g)								
Mass of water (g)								
Moisture content (%)								

Remarks:

Performed by:	A.Elhaddad	**Date:**	August 26, 2019
Verified by :		**Date:**	September 5, 2019

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Ottawa Community Housing Corporation	**Lab No.:**	G-19-005
Project, Site:	251 Penfield, Ottawa, On	**Project No.:**	11200830-A1
Borehole No.:	BH1B - SS3	Sample No.:	N/A
Depth:	5' - 7'	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

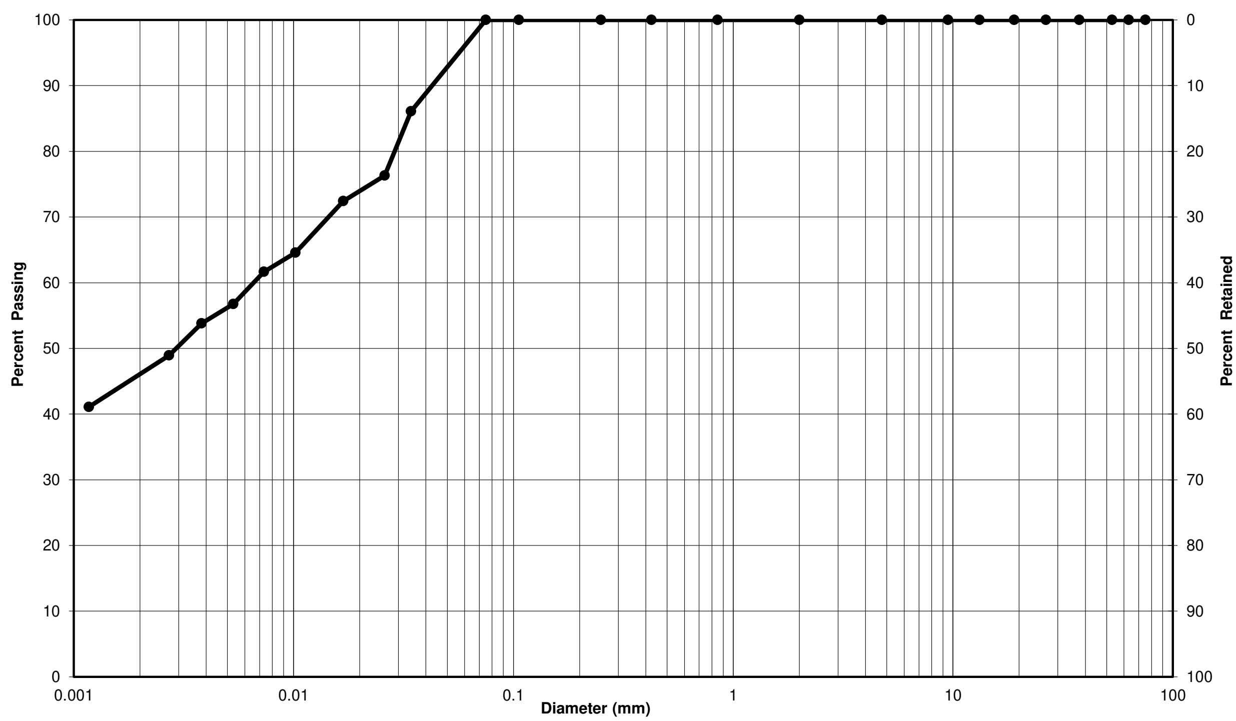
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Silty, Clayey, Sand, trace Gravel	7	36	57
Clay-size particles (<0.002 mm):			33 %

Remarks:

Performed by:	E. Bennett/A. Elhaddad	**Date:**	September 4, 2019
Verified by:		**Date:**	September 4, 2019

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Ottawa Community Housing Corporation	**Lab No.:**	G-19-005
Project, Site:	251 Penfield, Ottawa, On	**Project No.:**	11200830-A1
Borehole No.:	BH4 - SS7	Sample No.:	N/A
Depth:	15' - 17'	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Silt and Clay	0	0	100
Clay-size particles (<0.002 mm):	45 %		

Remarks:

Performed by:	E. Bennett/A. Elhaddad	**Date:**	September 4, 2019
Verified by:		**Date:**	September 4, 2019

GHD-FO-930.103 (On)-Particle-size Soils (USCS - GEO) MTO LS-702(Rev1) 12-08-2016

about GHD

GHD is one of the world's leading professional services companies operating in the global markets of water, energy and resources, environment, property and buildings, and transportation. We provide engineering, environmental, and construction services to private and public sector clients.

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