Geotechnical Investigation

811 Gladstone Avenue
Ottawa, Ontario

Ottawa Community Housing Corporation

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1. Introduction

GHD was retained by Mr. Meyerhoffer of Ottawa Community Housing Corporation (OCHC or Client) to undertake a geotechnical investigation for a proposed new residential development hereafter referred to as the Site, located at 811 Gladstone Avenue, in Ottawa, Ontario.

The purpose of the investigation was to complete an evaluation of the subsurface stratigraphy on the proposed development site in order to summarize the subsurface conditions found at borehole locations, and based upon the data, provide recommendations concerning foundation type and associated bearing capacity, drainage requirements, as well as comment 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 advancement of a total of five geotechnical boreholes within the proposed building footprint and installation of three monitoring wells to measure ground water level; three additional boreholes were drilled for environmental assessment purposes.
- Lab Testing | One chemical testing of groundwater for corrosion assessment for ductile iron and concrete.
- 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

The site is currently developed with residential townhouses and associated access road and parking area. The site is bounded by Balsam Street on the north, Rochester Street on the west, Gladstone Avenue on the south and by an Institutional property on the east. The site topography slopes down approximately 1.0 metres (m) from north to south as well as east to west.

We understand that two of the existing buildings on the property have partial basement levels used for the mechanical utility rooms that supply services to all three buildings on the property. It is unknown at this time if the adjacent buildings owned by others have any below ground levels.

It is our understanding that the proposed new development will consist of demolition and removal of the existing townhouses. The plans call for construction of a six storey residential building along the south half of the site and will a full basement for car parking garage and storage over the west 2/3 of the building. The depth of excavation required will be approximately 4 to 5 m below existing grades.

The two to three story townhouses along the north portion of the property will have raised first floors and basement beneath, which results in excavations of about 1.5 to 2 m below existing grades.

There will be a local subgrade storm water storage chamber on site and will require an excavation of about 2 m below existing grades.

There will be associated surface parking areas, access roads and landscaped areas.

GHD understands that the design concept for the new building will be conventional spread and strip footings that are expected to be founded near an approximate depths of 2 m for the townhouses and 4 to 5 m for the six-story apartment building. GHD has not been informed of any special slab on grade floor loading requirements for this residential development and therefore we are assuming 24 kPa floor loading for slab on grade.

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

3. Field Investigation

The fieldwork component of this Geotechnical Investigation consisted of the advancement of five boreholes BH1 to BH5. Three additional boreholes (BH6 to BH8) were drilled for environmental assessment purposes. Boreholes were advanced to depths varying between 2.6 m to 6.2 m below the existing surface grade. Three monitoring wells were installed in boreholes BH1, BH2, and BH5. All monitoring wells were sealed within the bedrock. 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, 25, 28 and 29, 2017 with a truck mounted drill rig, as well as a specialized manual drill rig adapted for soil sampling and diamond coring of bedrock, under the supervision of GHD field staff. Boreholes were advanced into the overburden using Standard Penetration Tests (SPTs) at regular intervals using a 50 mm diameter split-spoon sampler and a 31.8 kg hammer for the manual drill rig and a 63.5 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 in manual drilling is corrected for a hammer weight of 63.5 kg and recorded on the borehole logs as "N" value. All boreholes were advanced into bedrock using HQ diamond coring equipment, in order to confirm the existence of bedrock and comment on rock quality (ASTM D2113). Boreholes without monitoring wells were backfilled with bentonite to the top of bedrock and then with silica sand and auger cuttings to the surface upon drilling completion.

The elevations of the boreholes were determined by GHD field staff using a laser level; and related to a temporary benchmark (TBM) which was the top of a fire hydrant (FM175 D67M) located on the Southwest corner of the Rochester and Balsam intersection This benchmark was assumed to have an assured elevation of 100.00 m. The elevations of the boreholes are for use within the context of this report only.

3.1 Laboratory testing

Analytical testing was carried out on a groundwater 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 consisted of a grassed landscape or asphalt paved surface (fill material) followed by a layer of silty sand and gravel, underlain by limestone bedrock.

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

Boreholes BH1, BH2, BH3, BH4 and BH5 were drilled in a paved area which had an asphaltic concrete surface approximately 90 mm thick at the borehole locations and was followed by a basecourse crushed limestone fill material.

Boreholes BH5, BH6, BH7 and BH8 were located in a grass covered landscaped area of the Site. The grass was supported by a very thin topsoil layer.

4.2 Surficial Fill

A surficial fill material was observed in BH8 beneath the cover materials. The fill material was observed to have a thickness of approximately 2.1 m. The fill material was found to consist of sand and gravel. Fill material was loose in compactness condition and was recovered in moist condition.

4.3 Buried Concrete Structure

A buried concrete layer was found within the diamond coring sample beneath the fill material in borehole BH8 location. The concrete structure was found to be in direct contact with bedrock within the cored samples. The thickness of the concrete was found to be approximately 0.3 m.

4.4 Sand and Gravel

In all boreholes, except BH8, a layer of native silty sand and gravel underlay the surface cover. The layer was observed to have a thickness of approximately 0.1 to 0.9 m. The native material was loose to compact in compactness and was recovered in a damp to moist condition.

4.5 Bedrock

Practical refusal to auger advancement was encountered in all boreholes at shallow depths below the existing ground surface and ranged from between 0.4 m at BH7 to 2.2 m at BH8. Refusal to bedrock was confirmed in all boreholes by advancement into the bedrock by diamond coring equipment. The type of bedrock and its quality was confirmed by retrieving samples from all borehole locations by diamond coring techniques. Highly weathered and fractured grey sedimentary rock (Limestone) was encountered at the borehole locations. The quality of this rock was very poor with measured Rock Quality Designation (RQD) values of 29 to 50 within the upper approximately 0.5 to 1.5 m of the bedrock. The quality improves becoming what is considered as good to excellent rock based upon RQD values.

5. Groundwater

Three monitoring wells were installed as part of the scope of work. Groundwater levels were measured on September 8, 2017, at the monitoring wells. The following Table 5.1 shows the measured water levels.

Table 5.1 Groundwater Observations

Borehole Location	Depth of Water Below Existing Grade (m)
BH1	1.9
BH2	2.7
BH5	2.2

These levels indicated the water is within bedrock zone. However, it should be noted that the groundwater table is subject to seasonal fluctuations and in response to precipitation and snowmelt events. Also, it would be expected that water may be perched within the fill materials or the very poor bedrock, especially during and following periods of precipitation and in the spring and fall or other wet seasonal periods.

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 development will consist of demolition and removal of the existing townhouses.
- Six-Storey Building | The proposed structure will consist of a six-storey residential building with one underground level for basement or underground parking and expected excavations in the order of 4 to 5 m below existing grades. Groundwater table is expected to be penetrated with this basement excavation.
- Townhouses | A founding depth for the foundations of about 1.5 m to 2 m below current ground surface and the foundations will be conventional spread and strip type founded within the bedrock. These shallow basement concept should remain above the groundwater level except in wet seasons.
- The slab on grade floor slabs is of a lightly loaded residential type. (i.e., assumed to be approximately 24 kPa)
- No information is available regarding the foundation depth/elevation of the existing buildings or of the off-site adjacent structures.

Based on our understanding of the proposed structure, 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. The most important geotechnical considerations for the design of the proposed buildings are the following:

- Bedrock Excavation | Based on the proposed founding depth of foundations for the structures, bedrock excavation is expected. The upper 0.5 m to 1.5 m of bedrock was found to be weathered and fractured; The bedrock becomes good to excellent quality with depth.
- Existing and Buried Structures | It is important to note that no information was provided regarding the founding depth of the existing building's. It is our understanding that all existing structures will be demolished. Following demolition of the existing structures all foundations and buried structures must be removed from the footprint of the proposed building. Buried concrete was found in borehole BH8 location at about 2.3 m below existing grade. Deep fill layers and further buried structures may exist on site. Contractors and the designers should include some allowance regarding the removal of unknown buried structures or removal of fill materials.
- Frost Susceptibility of the Bedrock | Upper layers of the bedrock were found to be highly fractured and with the shallow groundwater the bedrock may be susceptible to frost action (frost heaving) and requires the same as typical 'soil' frost cover depths and protection. Should construction take place during winter, the exposed surfaces to support foundations must be protected by Contractors against freezing and foundations on bedrock should have adequate soil cover.
- Adjacent Structures | Construction Activity Induced Vibrations | The excavation operations of bedrock will impart vibrations affecting the nearby below grade and above grade structures. The client, designers and contractors should implement measures to reduce risk and severity of damage to adjacent structures.
- Adjacent Structures | Excavation and Dewatering Influences | The presence of the shallow depths to bedrock and type of bedrock, will result in no off-site impact to adjacent buildings due to dewatering effects due to the new building as existing buildings are assumed to be founded on Bedrock. The excavation faces will need to be adequately shored or reinforced with rock bolts for construction period and will be responsibility of Contractor. Long Terms support should not be an issue due to presence of bedrock and distance offset to existing buildings.
- Pre-Construction surveys should be carried out and contractors should incorporate excavation methods to minimize damage to the adjacent structures. This is of particular importance for the institutional building to the east.

6.1 Site Preparation

Site preparation within the new building footprint will involve the demolition and removal of the existing structures, removal of existing vegetation, topsoil and any existing fill materials to expose the bedrock.

In the proposed landscape and pavement areas the site preparation will involve removal of existing structures, existing topsoil and asphaltic concrete. The environmental assessments completed for the site indicate that contaminated soils are present and will require removal. Following the required removals, if soils (fills or native) remain then these may be reviewed by the geotechnical Engineer to determine if they are suitable to remain in place for re-use for the particular area on site. Field verifications should be carried out by qualified geotechnical personnel during construction.

Bedrock removal is expected for this project for underground services and footing excavations. The excavation operations of bedrock is expected to impart vibrations. Contractors must use techniques and methods to prevent settlement of adjacent ground, structural damage to adjacent buildings and minimize aesthetic impacts (e.g., paint/drywall cracks, pavement cracking). It is recommended that the specifications have pre-condition surveys of the adjacent structures that may be affected and are subject to construction vibrations. Submittal of contractors for methods of excavations, blasting, vibration monitoring, and soil and groundwater management plans in the form of written plans are recommended to be requested by the owner's design consultant team prior to construction to allow adequate time for review and discussion.

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', as defined by the OHSA Regulations for Construction.

Bedrock removal is expected since footing excavations are expected to penetrate to at least 1.5 m to 2 m for the Townhouses, approximately 2 to 3 m for the Stormwater Storage chamber, and 4 to 5 m for the six-storey building's basement.

The soil overburdens and some heavily weathered bedrock are considered to be Type 3 Soils as per the Occupational Health and Safety Act and should be sloped back at 1:1 or supported by a shoring system. The less weathered rock may either be shored and contractors may take some risk but as a minimum the weathered rock should be planned to be cut back at a 30 degree from vertical and/or required support by shoring or a rock protection system of rock bolts-mesh-shotcrete.

Sound rock may be planned to be excavated at near vertical.

Alternatives to sloped or cut back overburden and weathered bedrock is the use of shoring. Shoring methods would be expected to vary from combinations and use of Soil Nailing, Shotcrete and rock bolts, to methods Secant Pile wall.

The more sound bedrock would be excavated at near vertical due to its quality and type of rock. However, other factors may require coverage of the rock face. Local fractures, shear zones or weathered areas may require treatments ranging from rock bolting to rock bolting with mesh and shotcrete. Also depending upon the method and commercial products used for perimeter drainage and waterproofing. The Tender and Specifications should allow for unit price submissions from contractors during the Tender and have allowances in the contract.

The excavation of the bedrock will require the use of line drilling in combination with pneumatic or hydraulic breakers such as hoe rams or heavy excavation equipment equipped for rock excavation. Excavation that may involve controlled blasting techniques and/or line drilling. Local by-laws should be confirmed that this will be allowed. Line drilling on a closely spaced pattern may also be an option to assist excavation methods and prevent over breakage issues, especially around the perimeter or to create local excavations for elevator pits, footings, etc. The use of cutter heads with assist is doing final "shaving" of the rock in areas that would be beneficial to have a smooth face.

Excavations must be planned in advance to ensure the foundations of the adjacent structures, and roadways are not undermined during excavation. Any excavation methodology is subject to the laws and blasting restrictions that are in effect for the area.

The excavation operations of bedrock will impart vibrations affecting the surrounding buildings. It is recommended that the specifications have pre-condition surveys as well as submittal of plans for excavations, blasting, vibration monitoring, and soil and groundwater management plans of the adjacent structures that may be affected and are subject to those vibrations if bedrock mechanical or blast excavation is utilized. Excavations must be planned in advance to ensure the foundations of the adjacent structures as well as the granular structure of the adjacent roadways are not undermined during excavation.

It is recommended that the client's design team include in the specification package, requirements for the successful contractor to submit written Plans for Excavation as well as Soil and Groundwater Management for review by the client design team.

Water quantities expected to enter open excavations during construction will be discussed in the Hydrogeological Assessment and PTTW/EASR submission. These will need to be completed as separate submissions. Volumes will depend on seasonal conditions, depth of excavations, and the duration that excavations are left open.

The excavation of the weathered bedrock may require pneumatic or hydraulic breakers such as hoe rams or heavy excavation equipment equipped for rock excavation. Excavation of more sound rock will require more rigorous methods that may involve controlled blasting techniques and/or line drilling. Line drilling on a closely spaced pattern in combination with the use of hoe ram or other breaking type equipment may also be an option. Any excavation methodology is subject to the laws and blasting restrictions that are in effect for the area.

The client's design team should provide vibration limits for the adjacent off-site residential and institutional buildings and underground structures. The contractors plan should include methodology for how they will control vibrations and adjust their excavation methodology in the event of vibration exceedances. Local municipal guidelines should act as a minimum standard but designers should determine if the standard's criteria is sufficient to protect the buildings.

Surface water and groundwater seepage is expected in the excavated areas, especially within the overburden and weathered rock. Water quantities will depend on seasonal conditions, depth of excavations, and the duration that excavations are left open. Conventional construction dewatering techniques should be taken during construction, such as pumping from sumps and or ditches. Contractors will need to use techniques and methods to minimize disturbance to soils.

GHD did not complete a hydrogeological assessment of this site as part of the scope of work as it was not requested at the time. This is considered necessary to assess of a Permit to take water (PTTW) or submission on the Ontario Environmental Activity and Site Registry (EASR). It is recommended that the client have the necessary Hydrogeological assessments done prior to project Tendering for support of a PTTW or EASR application.

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 of the proposed residential building will be bearing on bedrock and will be supported by conventional spread footings.

Based on the recorded conditions within the boreholes, it is recommended that the shallow pad and strip footings be founded at a depth of about 1.5 m below ground surface within the weathered limestone bedrock. The recommended bearing pressures for strip and pad footings, founded at a depth of about 1.5 metres below ground surface (mbgs) on the limestone bedrock is 500 kPa under factored ULS conditions. There is no corresponding SLS value for footings set on bedrock, as settlement is considered to be nil for the footings founded on bedrock. The factored ULS value includes the geotechnical resistance factor (Φ) of 0.5. The minimum founding sizes should be 0.75 m for pad footings and 0.5 m widths for strip footings on bedrock using the bearing pressure.

Higher capacities for footings bearing on sound bedrock are available if designs assessed will result in a more economical design, but further investigation and recommendations would be required. The available factored ULS value may be in the order of 2000 kPa.

Based on the existence of mud seam recorded in the coring of the bedrock, it is our recommendation that rock probing be completed at the time of construction to evaluate the bedrock for mud seams within the footing areas for the 6 Storey Building. One mud seam was found in borehole BH4. Mud seams can be inherent in the sedimentary type limestone depths. Based on the existence of mud seam layers in bedrock, it is our recommendation that rock probing be completed at the time of construction to evaluate the bedrock beneath footing subgrade for mud seams. This "probing" may consist of contractors being required to drill a 50 mm diameter hole, 1.5 m below the base of the exposed footing subgrade. These probe holes should then be assessed by the Geotechnical Engineer to confirm the absence/presence of mud seams and then recommend whether additional deeper excavation to remove the rock down to the underside of the mud seam is required if the mud seam is deemed significant by the Geotechnical Engineer. Structural engineers should determine the remedial approach for foundation support if this over excavation is required. Remedial approach options may be bulk concrete backfill, extending the foundation walls or other. Designers/Owners should account for this work and unit rates for over excavation and remedial approach in the Tender and Specification documents. If the mud seam are greater than 1 m below underside of footing level and/or thin enough then the Geotechnical assessment during construction may allow the mud seam to be left in-place.

Excavations for footings and other adjacent structures (sump pits, sewer trenches, etc.) set within bedrock at various levels, including step footings, should be positioned such that they do not encroach within the 1V:1H zone of influence of an adjacent footing. Step footings should be designed in a manner that the average slope of the benching is no steeper than 1V:2H along the length and the height of the bench is less than 0.3 m.

6.4 Floor Slabs

Conventional slab-on-grade construction is considered suitable for the proposed building. We are assuming 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. The subgrade surface may need to be compacted following excavation. 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.

For the Townhouse structures slab-on-grades should an underfloor weeping tile and vapour barrier to be incorporated beneath the slab and should be specified by the architect.

For the 6 Storey building with basement, a heavy duty vapour or waterproofing membrane should be incorporated as well as rigorous underfloor weeping tile network.

Underfloor weeping networks should be connected to dedicated sumps that are separate from perimeter weeping tile systems.

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.

Designers should consider concrete slab crack control measures and whether the slabs should be tied into the foundation walls. The designers and contractors must carefully plan the placement of construction and control joints in the concrete and should be in accordance with generally accepted practice.

6.5 Frost Protection

The bedrock is a sedimentary rock with fractures and the water table is close to the surface. Therefore, the bedrock may be susceptible to frost action and frost heave. 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 subgrade surfaces must receive adequate temperature protection by Contractors to protect against freezing for the duration of the construction period.

6.6 Seismic Site Classification

In accordance with OBC-2012, the building and its structural elements must be designed to resist a minimum earthquake force based upon the borehole drilling program that was undertaken as part of this Geotechnical Investigation, this Site is recommended to have a Site Classification 'C', with respect to Table 4.1.8.4.A of the National Building Code of Canada 2010.

Higher Site Class of A or B are likely available, but would require confirmation by additional investigation using geophysical methods, in order to measure the shear wave velocity within the soil and rock mass. Structural designers should review the savings of using Site Class A or B versus C to justify the cost of the additional geophysical investigation.

6.7 Corrosion Potential of Soils

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

Table 6.1 Corrosion Parameter Results

Sample ID	MW4
pH	8.27
Resistivity (ohm-cm)	610
Sulphate (µg/L)	143
Chloride (µg/g)	163

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 considered to be potentially corrosive to cast iron pipe. Therefore protective measures, such as sacrificial cathode protection should be considered.

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 6.2 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 0.02 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.

6.8 Building Backfill

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

6.8.1 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 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.

6.8.2 Exterior Foundation Wall Backfill

6.8.2.1 Townhouses

Conventional residential backfilling requirements are recommended for the Townhouse buildings. 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.

6.8.2.2 Six-Storey Building

The client and designers have the option to pour the foundation walls against shoring or bedrock faces or alternatively allow for an offset or space sufficient to place backfill as outlined in Section 6.9.2.1.

6.9 Lateral Earth Pressure

Permanent basement/underground parking walls are to be considered as retaining walls and should be designed to withstand lateral earth pressures. It is assumed that hydraulic pressures are not applicable as drainage systems are proposed. If this changes the client and designers should seek further advice from GHD. There may also be retaining walls at grade changes with adjacent properties. 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

6.9.1 Static Conditions

The following soil parameters can be used for designing of the retaining walls for lateral earth pressures within the depth of any soil overburdens or if excavations result in the requirement to use soil/granular backfills to be placed. The weathered bedrock(i.e., top ~1 m) should be considered as part of the soil overburden when considering lateral earth pressures

Table 6.3 Soil Parameters and Earth Pressure Coefficients

Soil	Density 'γ' (kN/m^3)	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

Table 6.3 Soil Parameters and Earth Pressure Coefficients

Soil	Density 'γ' (kN/m^3)	Angle of internal Friction	Rankin Earth Pressure Coefficients(1) (2)		
		φ	Ka	Ko	Kp

(1) Assumes level/flat backfill surface

(2) If temporary soil support shoring is required, designers should refer to the CFEM for design assistance and to Section 6.7.3.

- 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.

It is noted that for the temporary shoring system that will support the existing fill and upper weather bedrock Section 26.10.3 of CFEM 2006 should be used by designers regarding the distribution of the forces. The soils encountered in the boreholes consist mainly of granular soils. If the shoring must support existing structures then the stiffness of the shoring system must be addressed by the designers and Ko is recommended. The contractor must also ensure installation procedures minimize risk of lateral movements especially where structures are being supported by the shoring system.

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.

6.9.2 Lateral Rock Pressures

The weathered bedrock(i.e., top ~1 m) should be considered as part of the soil overburden when considering lateral earth pressures.

The sound bedrock is predominantly sound limestone deposit with some shale interbeds. Typically the rock is expected to be fairly sound, competent bedrock below the upper weathered zone.

The sound bedrock would be able to be cut at near vertical and should stay stable. There was no reported fault or shear zones noted in the borehole logs.

The bedrock will be able to be cut near vertical.

The minimal depth into rock, it is typically considered that any in-situ stresses are released a short time following excavation. There will be nil to minimal pressures from the rock on the permanent walls for such shallow excavations into the bedrock. For this site and project as described above, regarding the pressure exerted on basement walls within the bedrock depth, it is recommended that K=0 (i.e., rock is self-supporting)

During construction, in spite of the quality of rock found within the boreholes, there may be local fissures and fractures oriented in such a way to create conditions of possible block failure. Some allowance should be included with the Project Specifications and Tender documents to allow for design and contractor installation of rock bolts for temporary excavation stability concerns during construction.

6.9.3 Dynamic Condition

These pressures are not considered for the structures under Part 9 of the Ontario Building, i.e., the Townhouse structures.

Also it is expected that the six-storey structure will have basement walls will have not granular backfill and therefore these dynamic forces are not applicable. If backfill is used between the basement walls and bedrock, then GHD should be consulted for further advice and recommendations.

6.10 Permanent Drainage

6.10.1 Townhouse Buildings

Both perimeter and under floor drainage is considered necessary for this structure with underground levels unless the building is treated to create a waterproofed "bathtub" in which case additional review and recommendations are required.

6.10.2 Six-Storey Building

It is recommended that Composite Drainage Blanket (CDB) or geodrain is used for the perimeter walls. There are several commercially available products available. The CDB should be connected by a collection piping system and drained to a frost-free outlet for year round drainage. The perimeter system should not be connected to the interior under-floor drainage system.

Underfloor drainage network is also recommended and should be connected to a frost free sump (separated from the perimeter drainage system and sump) with discharge to the municipal sewerage system.

As portions of the structure will be below the water table, it is also recommended that the exterior walls be protected with a waterproofing membrane applied to the wall in addition to the CDB.

Under floor and perimeter drains are considered necessary for all structures proposed, i.e., both townhouse and six-storey structure.

The drains should be connected to a frost-free outlets for year round drainage.

Elevator pits should have drainage weepers and waterproofing design measures. If drainage weepers are not practical then the pits will need to be designed to resist hydraulic buoyancy pressures.

If elevator pistons are used then the designers of these shafts and installations will need to also consider buoyancy issues. Installation of these will also need to consider groundwater control and buoyancy during installation. This may need additional investigation as the GHD mandate did not include deep enough boreholes to address the elevator piston shaft installation.

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 pavement 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 settlement and frost heave 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) within the frost zone below finished grade.

6.12 Pavement Sections

Access driveways and parking areas are expected to be constructed over existing fill or bedrock. 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.

The pavement sections described in the table below are recommended for areas subjected to parking lot and heavy truck traffic. 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
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:

- It is recommended that GHD be retained to review design drawings and specifications prior to the tender to ensure our recommendations have been interpreted and that there are no additional geotechnical recommendations required.
- 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 (OCHC) 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 eight test hole locations only. The subsurface conditions confirmed at these eight 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.

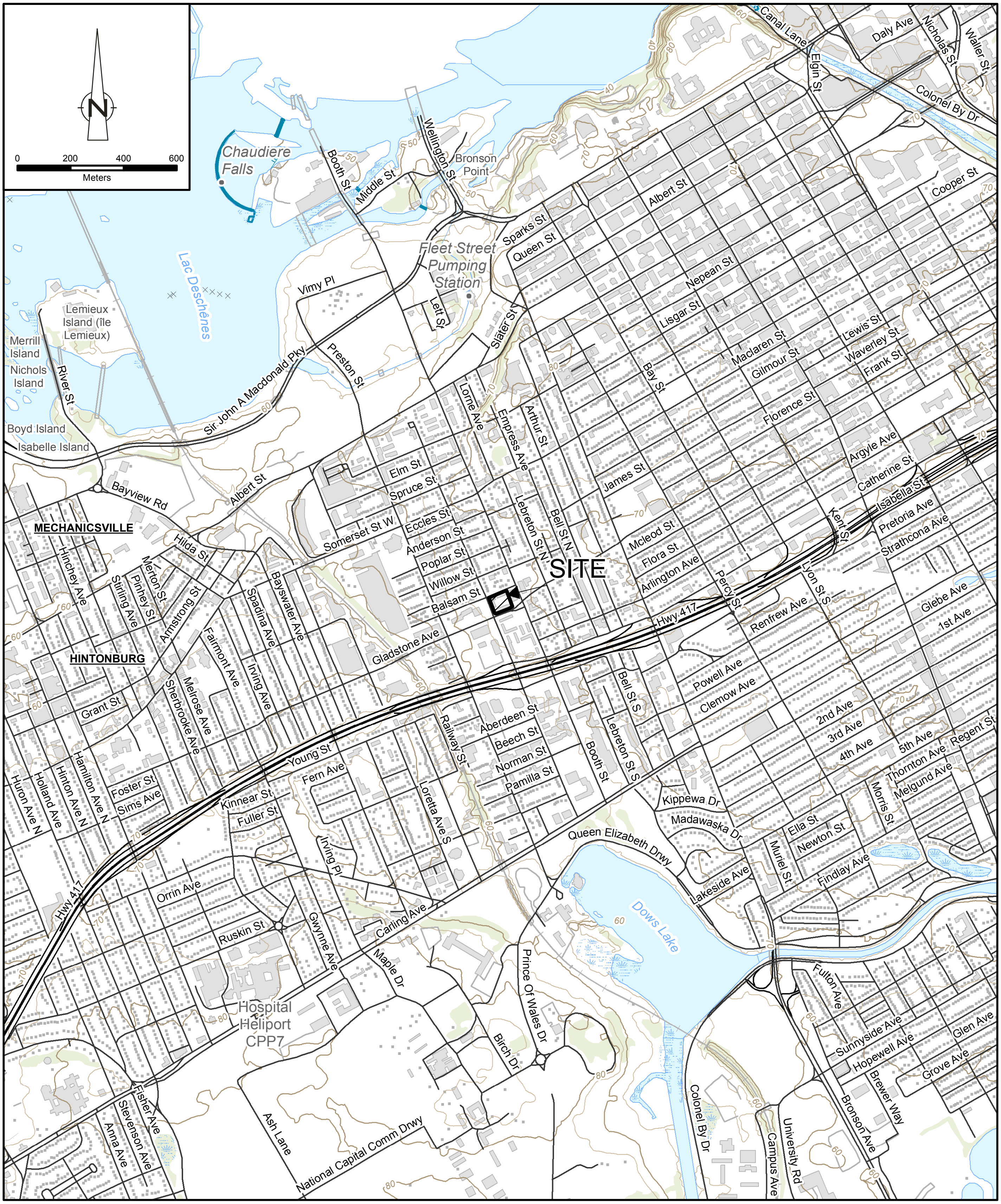
All of Which is Respectfully Submitted,

GHD

LICENSED PROFESSIONAL ENGINEER
11140575-A
March 1, 2019
B. VAZHBAKHT
100171356
PROVINCE OF ONTARIO

Bahareh Vazhbakht, P. Eng.

Joseph B. Bennett, P. Eng.



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Coordinate System: NAD 1983 UTM Zone 18N

GHD

OTTAWA COMMUNITY HOUSING
811 GLADSTONE AVENUE, OTTAWA, ON
GEOTECHNICAL INVESTIGATION

11140575-A1
Sep 5, 2017

SITE LOCATION MAP

FIGURE 1

GIS File: Q:\GIS\PROJECTS\11140000s\11140575\Layouts\002\11140575-A1(002)GIS-OT001.mxd

Source: Microsoft product screen shot(s) reprinted with permission from Microsoft Corporation, July 2013

0 5 10 15
Meters

Coordinate System:
NAD 1983 UTM Zone 18N

N

GHD

OTTAWA COMMUNITY HOUSING
811 GLADSTONE AVENUE, OTTAWA, ON
GEOTECHNICAL INVESTIGATION

BOREHOLE LOCATION PLAN

11140575-A1
Sep 19, 2017

FIGURE 2

GIS File: Q:\GIS\PROJECTS\11140000s\11140575\Layouts\002\11140575-A1(002)GIS-OT002.mxd

Appendices

Appendix A
Borehole Logs and Notes on Boreholes

REFERENCE No.: 11140575-A1 ENCLOSURE No.: 1

BOREHOLE LOG

BOREHOLE No.: BH1-17/MW1-17

ELEVATION: 101.28 m

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis

DATE (START): 22 August 2017 DATE (FINISH): 22 August 2017

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
0.00	101.28	GROUND SURFACE
	101.2	**ASPHALT** (Approximately 0.1 m thick)
	100.9	**FILL-** Gravelly sand, dense, grey, damp.
	100.8	**SAND AND GRAVEL-** Loose, brown, damp. *Auger refusal at 0.475 m, continued with rock coring
		LIMESTONE- Grey, weathered and fractured, fair quality. Water level : 1.89 mbgs *Becoming good *Becoming excellent
	95.1	Borehole ended at approximately 6.15 m in limestone

Monitor Well: 0.05; 0.20; 0.66; Bentonite Seal; 1.27; Solid Pipe; 1.48; WL 1.84 9/8/2017; Sand; Screen; 6.05; 6.15

Sample Data

Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
Auger			
SS1	5/15		
RC1	36/41		60
RC2	60/60		82
RC3	57/60		89
RC4	60/60		93

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

NOTES:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1 ENCLOSURE No.: 2

BOREHOLE No.: BH2-17/MW2-17

ELEVATION: 101.50 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis

DATE (START): 25 August 2017 DATE (FINISH): 25 August 2017

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)
	101.50	GROUND SURFACE					
	101.4	**ASPHALT** (Approximately 0.1 m thick)	0.05				
0.5	101.1	**FILL-** Gravelly sand, loose, grey, damp.	Sand 0.20	SS1	12/20		9
	100.9	**SILTY SAND-** Some gravel, compact, grey with reddish-brown staining, damp. *Auger refusal at 0.6 m, continued with rock coring	0.61				
1.0		**LIMESTONE-** Grey, weathered and fractured, fair quality. *Water was whiteish at start, transitioning to grey. Water level : 2.74 mbgs	Bentonite Seal	RC1	23/23		65
1.5		*Becoming good	1.35 Solid Pipe 1.55				
2.0				RC2	60/60		88
2.5		*Becoming excellent	WL 2.74 9/8/2017				
3.0			Sand				
3.5				RC3	58/58		98
4.0			Screen				
4.5							
5.0				RC4	52/52		100
5.5							
6.0				RC5	28/28		100
	95.3	Borehole ended at approximately 6.2 m in limestone	6.13 6.22				
6.5							

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

NOTES:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1 ENCLOSURE No.: 3

GHD

BOREHOLE No.: BH5-17/MW3-17

ELEVATION: 102.39 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis

DATE (START): 28 August 2017 DATE (FINISH): 28 August 2017

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		STRATIGRAPHY	
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK
meters	102.39		GROUND SURFACE
	102.3		**TOPSOIL-** Silty sand, brown, loose, moist. (Approximately 0.1 m thick)
	102.1		**FILL-** Sand and gravel, brown, loose, damp. Mudseam : 0.9 to 0.91m
	101.7		**LIMESTONE-** Grey, weathered and fractured, poor quality. Water level : 2.17 mbgs
			*Becoming good
			*Becoming excellent
	96.5		Borehole ended at approximately 5.87 m in limestone

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

MONITOR WELL

- 0.05
- 0.20
- 0.61
- Bentonite Seal
- 1.09
- Solid Pipe
- 1.30
- WL 2.17 9/8/2017
- Sand
- Screen
- 5.87

SAMPLE DATA

State	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	SS1	7/24		15
	RC1	60/60		50
	RC2	30/30		80
	RC3	45/45		92
	RC4	27/27		100
	RC5	47/47		100

SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

10 20 30 40 50 60 70 80 90

NOTES:

*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1 ENCLOSURE No.: 4

BOREHOLE No.: **BH3-17**
ELEVATION: 101.08 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation
PROJECT: Geotechnical Investigation
LOCATION: 811 Gladstone Avenue., Ottawa Ontario
DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis
DATE (START): 25 August 2017 DATE (FINISH): 25 August 2017

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)
	101.08	GROUND SURFACE				
	101.0	**ASPHALT** (Approximately 0.1 m thick)				
		FILL- Gravelly sand, loose, grey, damp.				
0.5	100.7	**SILTY SAND-** some gravel, compact, grey, moist.	SS1	11/25		9
	100.3	*Auger refusal at 0.76 m, continued with rock coring				
1.0		**LIMESTONE-** Grey, weathered and fractured, fair quality.	RC1	22/22		64
1.5		*Becoming excellent				
2.0			RC2	57/57		93
2.5						
	98.3	Borehole ended at approximately 2.7 m in limestone				
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:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1 ENCLOSURE No.: 5

BOREHOLE No.: BH4-17

ELEVATION: 101.08 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis

DATE (START): 25 August 2017 DATE (FINISH): 25 August 2017

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		STRATIGRAPHY	SAMPLE DATA				
Depth BGS	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery	OVC	Penetration Index / RQD	SCALE FOR TEST RESULTS
meters	101.08	GROUND SURFACE		%	ppm	N	50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90
	101.0	**ASPHALT** (Approximately 0.1 m thick)					
		FILL- Gravelly sand, compact, grey, damp.	SS1	10/16		19	● N = 19
0.5	100.7 / 100.6	**TILL-** Silty sand, some gravel, loose, greyish brown, damp. *Auger refusal at 0.5 m, continued with rock coring					
	100.3	**LIMESTONE-** Grey, weathered and fractured, good quality. Mudseam: 2.38 to 2.4m	RC1	27/27		78	
1.0							
1.5							
2.0			RC1	60/60		85	
2.5	98.4	Borehole ended at approximately 2.7 m in limestone					
3.0							
3.5							
4.0							
4.5							
5.0							
5.5							
6.0							
6.5							

NOTES:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1

ENCLOSURE No.: 6

BOREHOLE No.: **BH6-17**

ELEVATION: 101.70 m

BOREHOLE LOG

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis

DATE (START): 28 August 2017 DATE (FINISH): 28 August 2017

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)
	101.70	GROUND SURFACE				
	101.5	**TOPSOIL-** Silty sand with organics (grass), very loose, dark brown, moist. (Approximately 0.18 m thick)				
		FILL- Silty sand, trace organics, loose, dark brown, moist.	SS1	7/24		4
			SS2	5/20		22
	100.6	*Auger refusal at 1.1 m, continued with rock coring				
		LIMESTONE- Grey, weathered and fractured, poor quality.	RC1	25/25		30
		*Becoming excellent	RC2	47/47		96
	98.8	Borehole ended at approximately 2.9 m in limestone				

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:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1 | ENCLOSURE No.: 7

GHD	**BOREHOLE No.: BH7-17** **ELEVATION:** 101.66 m	**BOREHOLE LOG** Page: 1 of 1

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart | CHECKED BY: S. Wallis

DATE (START): 29 August 2017 | DATE (FINISH): 29 August 2017

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		STRATIGRAPHY		SAMPLE DATA					
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	OVC	Penetration Index / RQD	SCALE FOR TEST RESULTS 50kPa 100kPa 150kPa 200kPa / 10 20 30 40 50 60 70 80 90
meters	101.66		GROUND SURFACE			%	ppm	N	
	101.6		**TOPSOIL-** Silty sand with organics (grass), very loose, dark brown, moist. (Approximately 0.1 m thick)	☒	SS1	12/20		10	● 10
0.5			**FILL-** Sand some gravel, loose, light brown, moist.						
	101.0		*Auger refusal at 0.66 m, continued with rock coring						
1.0			**LIMESTONE-** Grey, weathered and fractured, poor quality.		RC1	14/19		29	
					RC2	15/15		40	
1.5			*Becoming excellent						
2.0									
					RC3	57/56		93	
2.5									
3.0	98.7		Borehole ended at approximately 2.9 m in limestone						
3.5									
4.0									
4.5									
5.0									
5.5									
6.0									
6.5									

NOTES:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

REFERENCE No.: 11140575-A1 ENCLOSURE No.: 8

BOREHOLE No.: BH8-17

ELEVATION: 100.65 m

BOREHOLE LOG

CLIENT: Ottawa Community Housing Corporation

PROJECT: Geotechnical Investigation

LOCATION: 811 Gladstone Avenue., Ottawa Ontario

DESCRIBED BY: R. Vandentillaart CHECKED BY: S. Wallis

DATE (START): 29 August 2017 DATE (FINISH): 29 August 2017

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)
0	100.65	GROUND SURFACE				
	100.5	**TOPSOIL-** Silty sand with organics (grass), loose, dark brown, moist. (Approximately 0.18 m thick)				
		FILL- Sand and gravel, loose, dark brown, moist.	SS1	10/24		4
			SS2	6/24		10
	99.6	**FILL-** Gravel some sand trace silt and clay, loose, greyish brown, damp.	SS3	3/24		34
			SS4	3/18		
	98.4	**CONCRETE**				
	98.1	**LIMESTONE-** Grey, weathered and fractured, poor quality.	RC1	12/24		25
		*Becoming good	RC2	39/39		82
	96.8	Borehole ended at approximately 3.9 m in limestone				

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:
*Surveyed elevations are relative to Fire Hydrant FM175067M (SW Corner of Rochester/Balsam)

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%

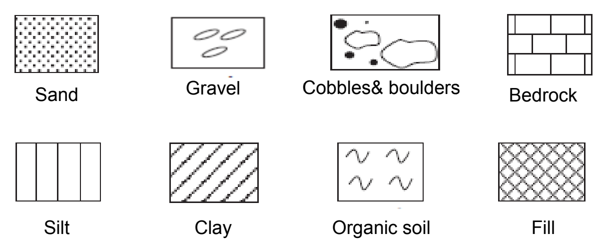
Relative density of granular soils	**Standard penetration index "N" value** (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

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

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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