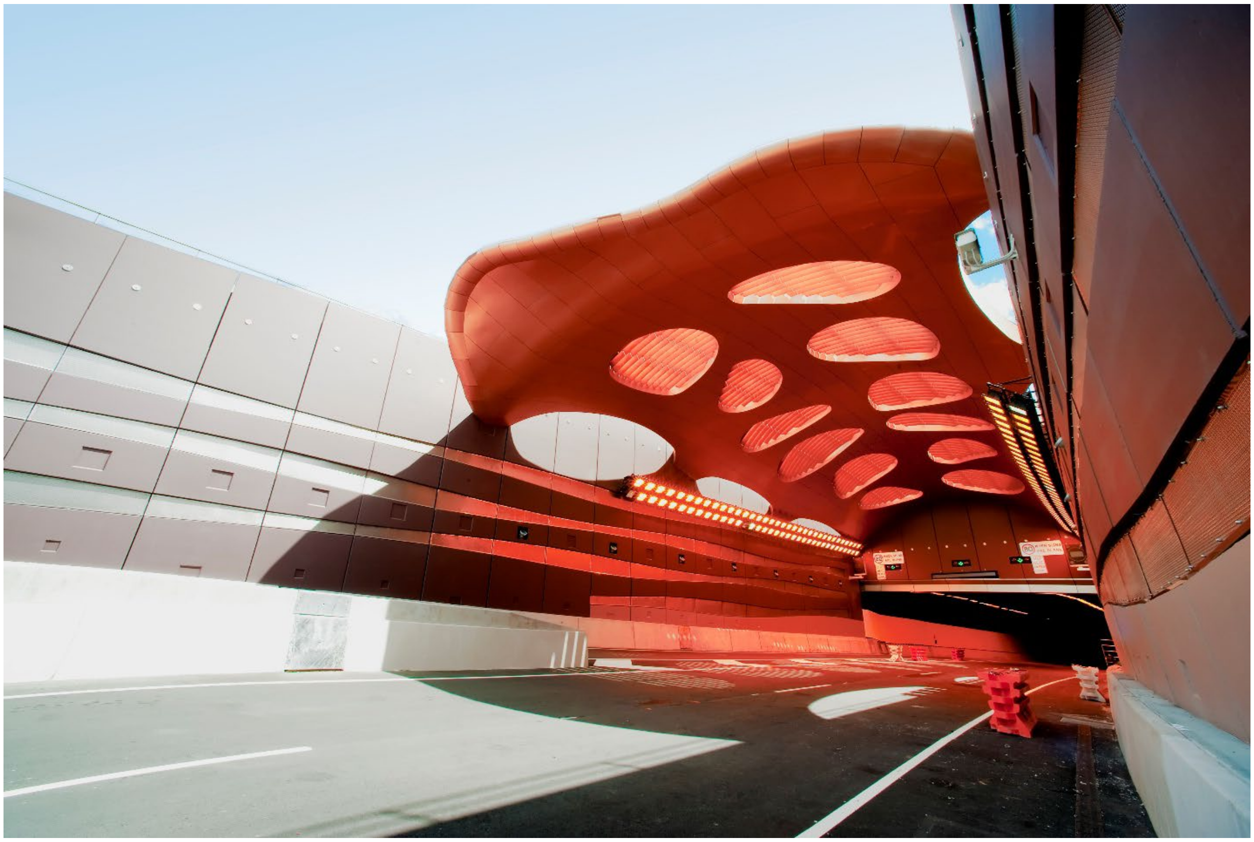
HALTON REGION

1258 REBECCA STREET

PRELIMINARY GEOTECHNICAL INVESTIGATION

FEBRUARY 18, 2021



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PRELIMINARY GEOTECHNICAL INVESTIGATION

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PROJECT NO.: 201-11808-00
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February 05, 2021

Halton Region
Office of the Commissioner
Legislation & Planning Services

Attention: Zach Richards

Dear Mr. Richards,

Geotechnical Services for 1258 Rebecca Street. Oakville, ON.

We are pleased to submit our preliminary geotechnical investigation report conducted for the proposed development of 1258 Rebecca Street. Oakville, ON.

The report is based on information obtained from borehole investigation and laboratory testing conducted on January 4th, 2021. Geotechnical recommendations relevant to the proposed site are also provided within this report.

We trust that this report meets your present requirements. Please contact us if you have any questions.

Yours sincerely,

Liam Gilmour
LG/lg

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REVISION HISTORY

FIRST ISSUE

February 05, 2021	Draft			
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Liam Gilmour	Nick La Posta	Nick La Posta		
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February 18, 2021

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February 18, 2021

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The report is intended to be used in its entirety. No excerpts may be taken to be representative of the findings in the assessment.

The conclusions presented in this report are based on work performed by trained, professional and technical staff, in accordance with their reasonable interpretation of current and accepted engineering and scientific practices at the time the work was performed.

The content and opinions contained in the present report are based on the observations and/or information available to WSP at the time of preparation, using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by WSP and other engineering/scientific practitioners working under similar conditions, and subject to the same time, financial and physical constraints applicable to this project.

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Benchmark and elevations used in this report are primarily to establish relative elevation differences between the specific testing and/or sampling locations and should not be used for other purposes, such as grading, excavating, construction, planning, development, etc.

Design recommendations given in this report are applicable only to the project and areas as described in the text and then only if constructed in accordance with the details stated in this report. The comments made in this report on potential construction issues and possible methods are intended only for the guidance of the designer. The number of testing and/or sampling locations may not be sufficient to determine all the factors that may affect construction methods and costs. We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

Overall conditions can only be extrapolated to an undefined limited area around these testing and sampling locations. The conditions that WSP interprets to exist between testing and sampling points may differ from those that actually exist. The accuracy of any extrapolation and interpretation beyond the sampling locations will depend on natural conditions, the history of Site development and changes through construction and other activities. In addition, analysis has been carried out for the identified chemical and physical parameters only, and it should not be inferred that other chemical species or physical conditions are not present. WSP cannot warrant against undiscovered environmental liabilities or adverse impacts off-Site.

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This limitations statement is considered an integral part of this report.

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APPENDICES

1 INTRODUCTION

WSP Canada Inc. (WSP) was retained by Halton Region to undertake a geotechnical and environmental investigation as part of the planning approvals and pre-development of the site at 1258 Rebecca Street, Oakville, Ontario. The results of the Environmental Investigations are included in a separate report.

The site is located on the south side of Rebecca Street, and Lake Ontario is approximately 700m south of the property. The site is currently unoccupied and has no buildings. It is understood that there is no proposed development currently and this investigating is part of the planning approvals and pre-development of the site.

The investigation was carried out by WSP on January 4th, 2021, and consisted of four (4) boreholes (designated as BH21-01 to BH21-04). The purpose of this geotechnical investigation is to evaluate the subsurface conditions and provide preliminary engineering recommendations for the following:

- Foundations
- Frost Consideration
- Excavations and backfill
- Temporary shoring
- Earth Pressures

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for Halton Region and its architect and structural engineers. Third party use of this report without WSP consent is prohibited.

2 REVIEW OF REGIONAL GEOLOGY

The property is located within a residential/commercial developed neighbourhood with no water features on site. The Site topography is generally flat with a gentle slope to the south, with an elevation of ranging from approximately 88 to 89 masl. The topography in the vicinity of property slopes to the south, towards Lake Ontario. Based on the local topography, the inferred shallow ground water flow direction of the study area is to the south towards Lake Ontario, which is located approximately 700 m south of the Site.

Surficial geology in the vicinity of the Site is described as fine textured glaciolacustrine deposits consisting of silt and clay, minor sand and gravel. The underlying bedrock within the area generally consists of shale, limestone, dolostone, and siltstone of the Georgian Bay Formation; Blue Mountain Formation; Billings Formation; Collingwood Member; Eastview Member.

3 FIELD AND LABORATORY WORK

3.1 FIELD INVESTIGATION AND TESTING

The field investigation consisted of four (4) boreholes (BH21-01 to BH21-04) advanced to depths ranging from 3.2m to 3.3m below existing ground surface at the locations shown on the attached **Drawing 1**. The borehole coordinates and ground geodetic elevations at the locations of the boreholes are presented in the Record of Borehole sheets in **Appendix A**.

Prior to drilling operations, all underground utilities were cleared at the borehole locations by the representatives of the public and private utilities locate companies. The boreholes were drilled with solid stem augers under the direction and supervision of WSP personnel. The soil stratigraphy was recorded by observing the quality and changes of augered materials which were withdrawn from the boreholes, and by sampling the soils at regular intervals of depth using a 50mm O.D. split spoon sampler, in accordance with the Standard Penetration Test (ASTM D 1586) method. This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 0.3m depth into the undisturbed soil (SPT 'N'-values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT 'N' values are indicated on the Borehole Records (Refer to **Appendix A**). Soil samples were visually classified in the field and later re-evaluated by an engineer in our laboratory.

Upon encountering bedrock, boreholes were drilled to auger refusal and rock coring was not performed. As such, our current recommendations are based off an estimated value of bedrock in the area, and all design parameters related to bedrock will need to be confirmed during detailed design.

Water level observations were made during drilling and were dry upon completion of drilling. Additionally, monitoring wells were installed at all borehole locations to permit long-term groundwater monitoring.

3.2 GEOTECHNICAL LABORATORY TESTING

Following drilling, the soil samples were taken to WSP's laboratory where they were re-examined. Representative soil samples were selected for geotechnical index testing. The testing program consisted of the measurement of the natural moisture content of all available soil samples and results are presented on the respective Borehole Records. In addition, grain size analyses on two (2) selected samples and consistency (Atterberg) limits for two (2) cohesive soil samples were performed and the results are included in **Appendix B**.

4 SUBSURFACE CONDITIONS

The borehole location plan is shown in **Drawing No. 1** and the subsurface conditions in the boreholes are presented on individual borehole records (Refer to **Appendix A**) and are summarized in the following sections. An explanation of the terms used in the records of boreholes are presented in **Appendix A**.

The general subsurface soil profile consists of topsoil over fill materials which are overlain by native silty clay till, which is underlain by shale bedrock. The details of the subsurface soil layers are summarized in the following sections.

4.1 SURFICIAL MATERIAL

A 50 mm to 130 mm layer of topsoil was encountered in boreholes BH21-01 to BH21-02, and BH21-04. In BH21-03, surficial fill was observed.

It should be noted that the thickness of the surficial material observed at the borehole locations may not be representative for the site and should not be relied on to estimate quantities for stripping and/or design purposes.

4.2 FILL MATERIAL

Below the surficial material (and from the surface in Borehole BH21-03), fill material consisting of silty clay was encountered in all boreholes, extending to depths ranging from 0.8 m to 1.5 m. The SPT ‘N’ values within the fill material ranged from 5 to 17 blows per 300 mm of penetration, corresponding to firm to very stiff consistency.

The natural moisture contents were measured in the test samples ranging from 5% to 43%.

4.3 SILTY CLAY

Below the fill material, a native deposit of silty clay was encountered in all boreholes, extending to depths ranging from 1.9 m to 2.6 m. The SPT ‘N’ values within the silty clay ranged from 10 to 45 blows per 300 mm of penetration corresponding to a stiff to hard consistency. The natural moisture contents were measured in the test samples ranging from 8% to 24%.

Two (2) laboratory particle size distribution analyses were conducted on selected samples obtained from the silty clay. The results are provided in Table 4.1 below, according to the Unified Soil Classification System (USCS):

Table 4.1 Cohesive Earth Fill Particle Size Distribution Analysis Results

Borehole No.	Sample I.D.	% Gradation				Primary Soil Classification
		Gravel	Sand	Silt	Clay	
BH21-02	SS3	0	3	63	34	Silty Clay, trace sand
BH21-03	SS2	0	3	56	41	Silty Clay, trace sand

The results of the analyses are shown on the borehole logs in Appendix A. The particle size distribution curves are provided in Appendix B.

Atterberg Limits tests were carried out on the above samples. The results are summarized in Table 4.2 below.

Table 4.2 Summary of Atterberg Limits – Silty Clay

Borehole No.	Sample I.D.	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
BH21-02	SS3	34	21	13
BH21-03	SS2	33	20	13

The Atterberg limits tests performed on the samples indicate a clay of low plasticity, CL, in accordance with the Unified Soil Classification System. The results of the analyses are summarized on the borehole logs in Appendix A and the plasticity chart is provided in Appendix B.

4.4 INFERRED BEDROCK

Inferred bedrock was encountered in all boreholes at a depth ranging from 1.9 m to 2.6 m below existing ground surface at the individual borehole locations. The inferred top of bedrock level varies from Elev. 85.9 m to 86.5m. The inferred bedrock surface elevation should not be considered accurate to better than ±0.5m since the contact with the overlying overburden is not distinct and weathering in the upper bedrock obscures the contact zone. Table 4.3, below, lists the depths at which bedrock was inferred at each borehole.

Table 4.3 Inferred Bedrock Surface Depths/Elevation at Borehole Locations

Borehole No.	Inferred Bedrock Surface Depth (m) (±0.5m)	Inferred Bedrock Surface Elevation (m) (±0.5m)	Inferred Bedrock Formation
BH21-01	1.9	86.5	Queenston Shale
BH21-02	2.6	86.1	Queenston Shale
BH21-03	2.4	85.9	Queenston Shale
BH21-04	2.0	86.3	Queenston Shale

Visual examination of the recovered rock fragments indicates that of the Queenston Formation consisting of highly weathered, reddish brown, very weak to medium strong shale.

4.4.1 BEDROCK IN GREATER TORONTO AREA

Refer to **Appendix A** for general comments for bedrock in Greater Toronto Area.

4.5 GROUNDWATER CONDITONS

Upon completion of drilling all boreholes were dry. An additional groundwater measurement in the monitoring wells on January 14, 2021; the groundwater levels are summarized in Table 4.4. It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations and fluctuations in response to major weather events.

Table 4.4 Summary of Groundwater Observations

MONITORING WELL	WELL DEPTH (mbgs)	GROUND SURFACE ELEVATION (masl)	DATE OF WATER LEVEL MEASUREMENT	DEPTH OF GROUNDWATER LEVEL (mbgs)	GROUNDWATER ELEVATION (masl)
BH21-01	2.85	88.4	2021-01-14	2.77	85.65
BH21-02	2.88	88.7	2021-01-14	1.82	86.89
BH21-03	2.82	88.4	2021-01-14	DRY	DRY
BH21-04	2.96	88.3	2021-01-14	2.38	85.94

5 DISCUSSION AND RECOMMENDATIONS

This section of the report provides our interpretation of the factual geotechnical data obtained from the investigation and provides preliminary recommendations and comments related to the geotechnical aspects of design of foundations and general site development. The recommendations provided are intended for design guidance for this site. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Parties requiring information on specific aspects of construction must make their own interpretation of the subsurface conditions as it affects their proposed design, construction methods, costs, equipment selection, scheduling, etc.

5.1 FOUNDATIONS

It is understood that this investigation is for the planning approval and pre-development of the site and the proposed development is unknown.

Based on the subsurface information from the boreholes, the following bearing capacities have been calculated in Table 5.1 below.

Table 5.1 Inferred Bedrock Surface Depths/Elevation at Borehole Locations

Borehole No.	Founding / Excavation Depth (m)	Founding / Excavation Elevation (m)	Founding Soil	SLS Bearing Capacity* (kPa)	ULS Bearing Capacity (kPa)
BH21-01	1.0	87.4	Native silty clay	120	180
BH21-02	1.7	87.0		120	180
BH21-03	1.0	87.4		120	180
BH21-04	1.0	87.3		120	180

*The SLS bearing capacity was determined for anticipated settlement of 25 mm.

Foundations at or below elevation 85m may be designed for a preliminary Serviceability Limit State (SLS) bearing resistance of 3.0 MPa and Ultimate Limit State (ULS) bearing resistance of 5.0 MPa. Footings designed to the specified bearing capacity at the serviceability limit states (SLS) are expected to settle less than 25 mm total and 19 mm differential. All footing bases must be inspected to confirm the design bearing values. As noted earlier, these values should be considered preliminary and should be confirmed with representative testing and analysis completed on bedrock core samples.

A preliminary modulus of subgrade reaction (ks) value of 100MN/m^3 may be used for structural design for slabs resting on sound shale bedrock.

Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 2 horizontal to 1 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

The shale base of the excavation should be protected from slaking degradation once exposed using a 50mm thick lean mix concrete mudslab.

It should be noted that the recommended bearing capacities on shale bedrock have been approximated by WSP from regional bedrock information for the preliminary design stage only. Modifications to the recommendations may be necessary as additional information becomes available. For example, when more information is available with respect to the subsurface conditions when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections by WSP to validate the information for use during the construction stage.

5.2 FROST CONSIDERATIONS

The design frost penetration depth at the subject site is determined as 1.2 m. All unheated foundation elements, pile caps and slab on grade must be provided with a minimum of 1.2 m of soil cover or equivalent insulation for protection against frost heaving.

The design depth of frost penetration should also be considered in the design of frost tapers for slabs on grade, side platform, pavements, bridge abutments and retaining walls.

5.3 EXCAVATIONS, BACKFILL AND GROUNDWATER CONTROL

It is anticipated that most excavations / trenches will penetrate below the groundwater levels measured at the site. As such dewatering will be required. If shoring is installed at the site, the shoring method selected and designed by the contractor will aid in determining the methodology and extent of dewatering as well as the type of Ministry of Environment permit required (EASR vs PTTW), if any. In order to provide detailed dewatering requirements, an additional assessment is recommended to be completed once site-specific design details, including excavation depths, are available.

Based on the subsurface conditions noted in the boreholes, the excavations will be carried out through the fill soils, stiff to hard silty clay, and weathered to sound shale bedrock. Obstructions, cobbles and boulders should be expected in the overburden. Excavation of the shale, if required, can be carried out using the heaviest available single tooth ripper equipment. The stronger limestone and siltstone beds are frequent and therefore it will be necessary to utilize hoe rams to "open" the hard layers of limestone/siltstone for the ripper. The removal of the underlying fresh and stronger rock and especially the interbedded limestone and siltstone layers and zones where rock quality is "fair", "good" or "excellent" (i.e. RQD > 50%), will be arduous and time consuming.

All excavations must be properly designed and carried in accordance with the most recent Occupational Health and Safety Act (OHSA) and by an experienced shoring design engineer and excavation contractor. The effects of construction equipment and stockpiling of excavated soils at the crest of excavations should be considered in the design of excavations (i.e., appropriate surcharge loadings must be added to the lateral earth pressure distribution). In general, the site soils above the groundwater table are considered to be Type 3 as per OHSA; soils below the groundwater table should be considered as Type 4 soils.

The native soils free from topsoil, organics and contamination may be used as trench backfill at the site, provided the moisture content of the excavated native soil is within 2 percent of its optimum moisture content. Depending on the time of construction and weather, some excavated material may be too wet to compact and will require aeration prior to its use. Loose lifts of soil, which are to be compacted, should not exceed 200 mm.

Imported granular fill, which can be compacted with hand held equipment, should be used in confined areas. Underfloor fill should be compacted to at least 98 percent of Standard Proctor Maximum Dry Density (SPMDD). The excavated soils are not considered to be free draining. Where free draining backfill is required, i.e. backfill behind foundation walls and in footing trenches, imported granular fill such as OPSS Granular "B" should be used.

5.4 TEMPORARY SHORING

Consideration may be given to shoring the excavation and to support adjacent structures, if required, with soldier pile and lagging or secant pile systems. These systems must be selected in connection with the design/implementation of dewatering systems. We note that the design of the shoring system is beyond the scope of this geotechnical investigation.

Excavated material should be stockpiled not closer than 5m to the crest of the excavation slopes. For lateral earth pressure recommendations on temporary shoring, please refer to **Drawing No. 2**. The shoring system must be designed in accordance with the 4th Edition of the Canadian Foundation Engineering Manual. The surcharge loading from adjacent structures must be considered.

5.5 LATERAL EARTH PRESSURE

5.5.1 LATERAL EARTH PRESSURE IN OVERBURDEN SOILS ON PERMANENT FOUNDATION WALLS

The earth pressure distribution on the permanent subsurface wall can be taken as hydrostatic, i.e. which is increasing uniformly with depth according to the formula:

$$p_h = K_o.\gamma.h + K_o.q + \gamma_w.h_w$$

where

p_h = horizontal pressure at depth h (kN/m²)

γ = unit weight of soil, taken as 21.5 kN/m^3 above the groundwater table; this value will reduce to 11.5 kN/m^3 below the groundwater table

γ_w = unit weight of water, taken as 9.81 kN/m^3

h = depth below ground surface (m)

h_w = depth below water table

q = surcharge load at ground surface (kPa)

K_o = coefficient of lateral earth pressure at rest for a horizontal ground surface condition,

taken as 0.5 for non-yielding rigid walls.

Below the groundwater table, the submerged unit weight of the soil should be used and the full hydrostatic water pressure should be added.

If the ground surface is not horizontal, the uneven portion can be treated as an equivalent surcharge.

5.5.2 LATERAL EARTH PRESSURE IN BEDROCK ON FOUNDATION WALLS

Structures which extend below the surface of the bedrock and the walls of which are poured in direct contact with the bedrock will be subject to “rock squeeze”. This practise is not recommended.

Although in-situ stress measurements were not made at this site, it is known that bedrock belonging to the Queenston Formation contains high horizontal stresses, the magnitude of which varies between 1.7 and greater than 6.9 MPa. As a result of the relief of this high horizontal stress, significant elastic displacements occur during and after the excavation. Of these, the long term, time dependant displacements are of greater importance. These are estimated to be of the order of 0.05% of the height of the excavation per log cycle of time (e.g. 2.5mm per log cycle of time (in days) for a 5m deep excavation or about a total of 11mm over a period of 50 years). Approximately 50% of the displacement (i.e. 5mm) is expected to occur during the first 100 days following excavation.

It is recommended that the walls not be designed to resist these displacements. Rather a layer of compressible material must be placed between the structure and the rock. This compressible layer could be either a synthetic material (e.g. suitable expanded polystyrene) or sand backfill. Properties and proposed thicknesses of the compressible material should be submitted to a qualified engineer to evaluate its stiffness and assess its suitability. Certain rigid polystyrene insulation products are considered to be excessively stiff for this application.

If the rock squeeze is allowed to dissipate by delaying construction of permanent concrete walls or by applying a compressible void foam, the lateral earth pressures acting on the bedrock portion of the wall below the overburden for concrete cast against the rock with no backfill can be assumed to be a uniform pressure equal to the maximum overburden lateral earth pressure calculated at the overburden to rock interface, plus the hydrostatic forces.

If the rock squeeze is allowed to dissipate by backfilling the zone between the wall and the excavated face of bedrock with sand, the lateral earth pressures acting on the bedrock portion of the wall below the overburden for concrete cast against the rock with backfill can be assumed to increase uniformly with depth and may be calculated in the same manner as is outlined in Section 5.5.1.

5.6 EARTHQUAKE CONSIDERATIONS

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4A of the Ontario Building Code (2012). The classification is based on the determination of the average shear wave velocity in the top 30 meters of the site stratigraphy, where shear wave velocity measurements have been taken or alternatively estimated on the basis of rational analysis of un-drained shear strength or penetration resistance.

For seismic design purposes, the preliminary site designation for seismic analysis is Class C (OBC 4.1.8.4 Table 4.1.8.4.A.). The seismic site class should be confirmed with MASW testing.

6 GENERAL COMMENTS AND LIMITATIONS OF REPORT

WSP should be retained for to perform additional geotechnical analysis and also for general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, WSP will assume no responsibility for interpretation of the recommendations in the report.

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

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP at the time of preparation. Unless otherwise agreed in writing by WSP, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

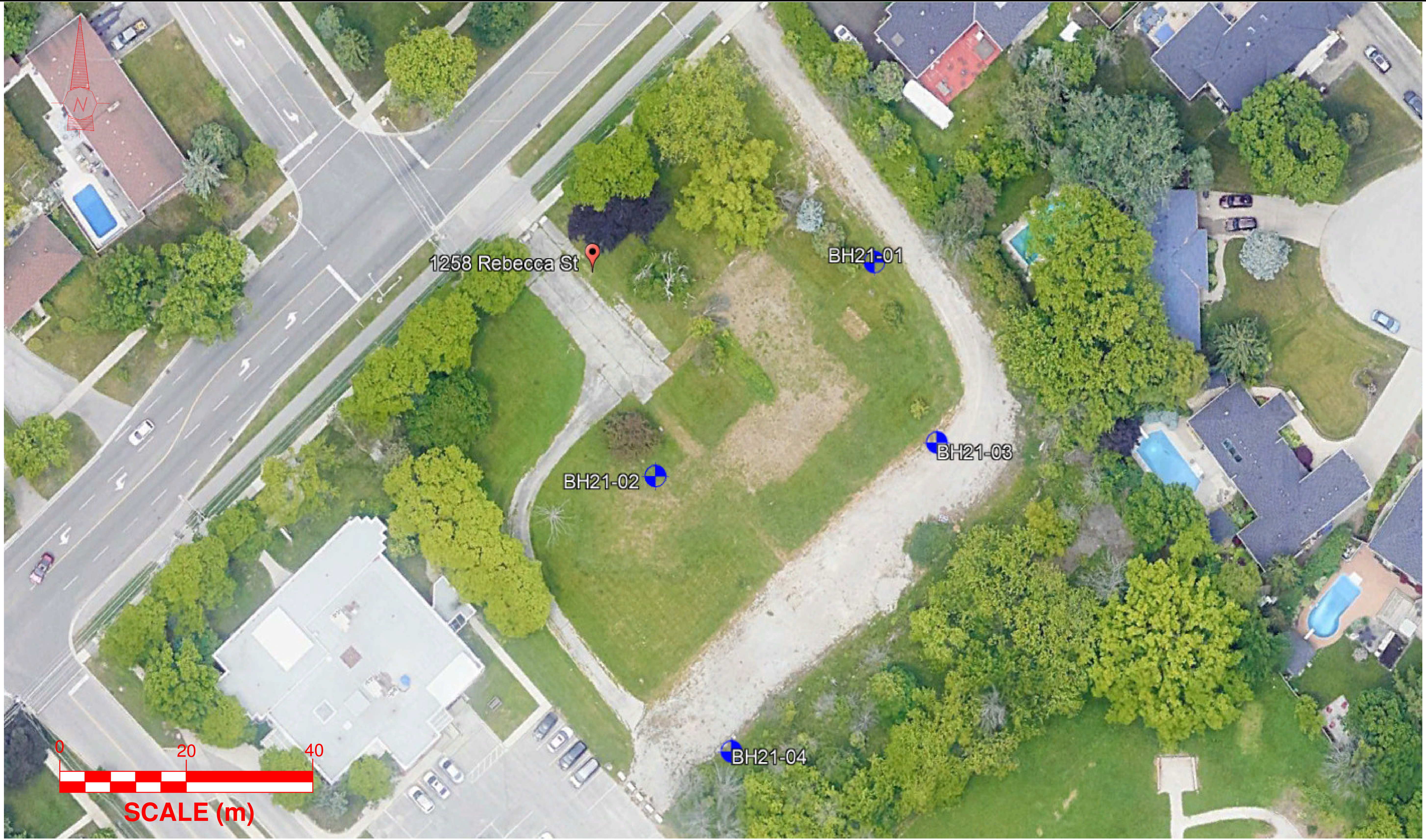
Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP Consultants Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on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

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office

DRAWINGS

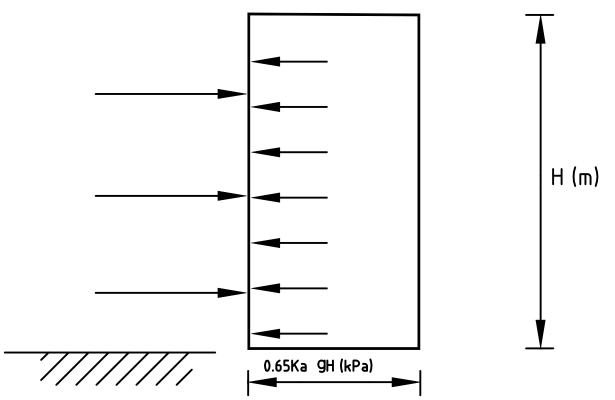
BOREHOLE LOCATION PLAN (DRAWING 1)
EARTH PRESSURE DISTRIBUTION (DRAWING 2)



LEGEND

⊕ **Boreholes drilled January 2021**

Client:	**Halton Region**			Project No:	**201-11808-00**	Figure No:	**1**
Drawn:	**LG**	Approved:		Title:	**BH Location Plan**		
Date:	**Nov-20**	Scale:	**N/A**	Project:	**1258 Rebecca Street, Oakville**		
Original Size:	**N/A**	Rev:	**N/A**		2 INTERNATIONAL BLVD, UNIT 201 TORONTO, ON M9W 1A2		

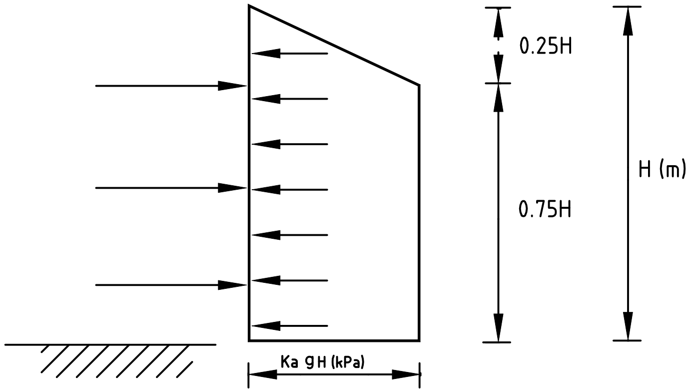


g = unit weight of soil

g' = submerged unit weight of soil (i.e. below ground water level)

Ka = Active Earth Pressure

IN COMPACT TO VERY DENSE NON-COHESIVE SOILS (SANDS AND SILTS)

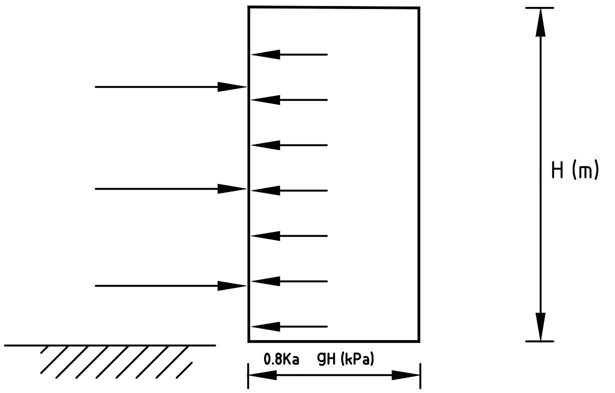


g = unit weight of soil

g' = submerged unit weight of soil (i.e. below ground water level)

Ka = Active Earth Pressure

IN VERY STIFF TO HARD COHESIVE CLAYS OR CLAYEY SOILS

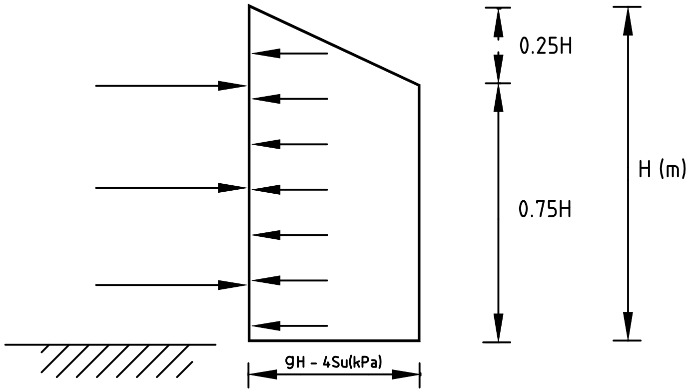


g = unit weight of soil

g' = submerged unit weight of soil (i.e. below ground water level)

Ka = Active Earth Pressure

IN LOOSE OR DISTURBED NON-COHESIVE SOILS (SANDS AND SILTS)



g = unit weight of soil

g' = submerged unit weight of soil (i.e. below ground water level)

Su = Undrained Shear Strength

IN VERY SOFT TO FIRM COHESIVE CLAYS OR CLAYEY SOILS

Notes:

1. Check system for partial excavation condition.
2. If the free water level is above the base of the excavation, the hydrostatic pressure must be added to the above pressure distribution.
3. If surcharge loadings are present near the excavation, these must be included in the lateral pressure calculation.

2 INTERNATIONAL BLVD, SUITE 201
TORONTO, ONTARIO CANADA M9W 1A2
TEL.: 416-798-0065 | FAX: 416-798-0518 | WWW.WSP.COM

CLIENT: REGION OF HALTON

TITLE: EARTH PRESSURE DISTRIBUTION ON BRACED EXCAVATIONS

PROJECT:

PROJECT NO:

DRAWING NO:

DRAWN BY: ZMO

CHECKED BY: SD

DATE: AUG 11, 2020

SCALE: N.T.S

ORIGINAL SIZE: LETTER

REV. #: N/A

APPENDIX

A

BOREHOLE EXPLANATION FORMS AND BOREHOLE LOGS

General Comments – Bedrock in Greater Toronto Area

The bedrock that makes spread footings or caissons a popular choice for high-rise foundation support is a shale or shale limestone composition. The highest member, the Queenston Formation, is generally found west of Toronto, while the Georgian Bay Formation underlies most of Metro Toronto, with the Collingwood Formation east of Toronto. The Queenston is, relatively speaking, the weaker of the three formations that are likely to support caissons or footings.

The Georgian Bay as well as the Queenston and Collingwood Formation are of Middle Ordovician Age. It is defined as the rock unit that overlies the bluish grey shales of the Collingwood Formation and is in turn overlain by the red shale of the Queenston Formation. The Georgian Bay Formation consists of bluish and grey shale with interbeds of sandstone, limestone and dolostone. Towards the west where the Georgian Bay formation underlies the Queenston Formation, the limestone content increases significantly and limestone and/or sandstone may comprise as much as 70 to 90 percent of the bedrock. The hard layers are usually less than about 100 to 150 mm thick but some layers are much thicker. The thicker layers have been observed to be as much as 750 to 900 mm at some sites. The layers are actually lenses and they can vary significantly in thickness over short distances.

The upper portion of the bedrock is commonly weathered for a depth of 600 to 1000 mm and within this weathered zone hard limestone layers or lenses are common. These hard limestone layers can result in contractual problems for augers, and can provide misleading bedrock elevations. Where the weathering is more extensive a shale till layer may be found above the bedrock. In the sound bedrock, the limestone, sandstone, dolostone is hard to very hard.

Stress relief features such as folds and faults are common in the bedrock. In these features, the rock is heavily fractured and sheared, and contains layers of shale rubble and clay. Weathering is much deeper than the surrounding rock in these features and often there is a lateral migration of the stress relief features resulting in sound unweathered bedrock overlying fractured and weather bedrock. The stress relief features are usually in the order of 4 to 6 m wide, but the depth can vary from 4 to 5 m to in excess of 10 m. These features occur randomly.

The bedrock contains significant high locked in horizontal stresses. These stresses can impose significant loads on tunnel walls but the slower rate of construction for basements allows for a relaxation of these stresses and they are not normally a problem for basement construction.

Groundwater seepage below the top 1000 mm is generally small, however, at several locations in Toronto and Mississauga large quantities have been encountered.

Bedding joints in the bedrock are very close-to-close, smooth planar in the shale and rough planar in the limestone. Significant vertical jointing is common.

Where the bedrock was cored, a detailed description of the rock core is appended to the borehole log.

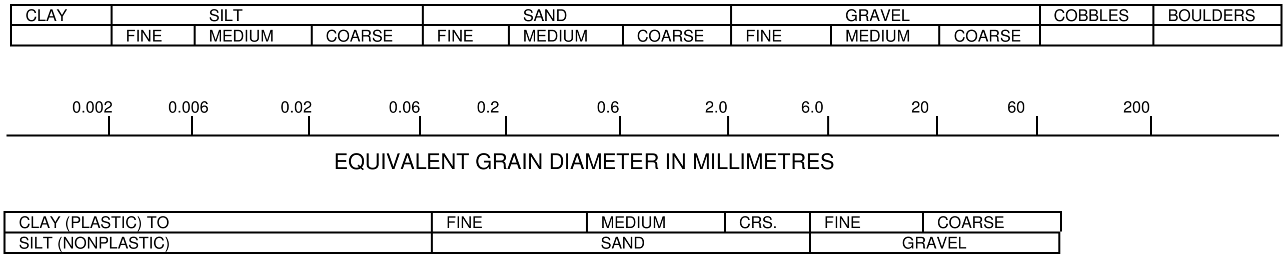
Design features related to the bedrock are discussed in other sections of this report, and these general comments must be considered with these comments.

Methane gas exists in the bedrock, normally below the top 1000 mm and more concentrated with depth. Appropriate care and monitoring is essential in all confined bedrock excavations, particularly caissons and tunnels.

Notes on Sample Descriptions

1. All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by WSP also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION



UNIFIED SOIL CLASSIFICATION

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

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

Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60^{o} cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	>300 mm
Cobbles	75 mm-300 mm
Gravel	4.75 mm-75 mm
Sand	0.075 mm-4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-40%
And (e.g. sand and gravel)	40-60%

Soil Description

a) Cohesive Soils

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

b) Cohesionless Soils

Density Index (Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

wsp

LOG OF BOREHOLE BH21-1

1 OF 1

PROJECT: Preliminary Geotechnical Investigation
CLIENT: Region of Halton
PROJECT LOCATION: 1258 Rebecca Street
DATUM: Geodetic
BH LOCATION: N 4808044.462 E 605443.165

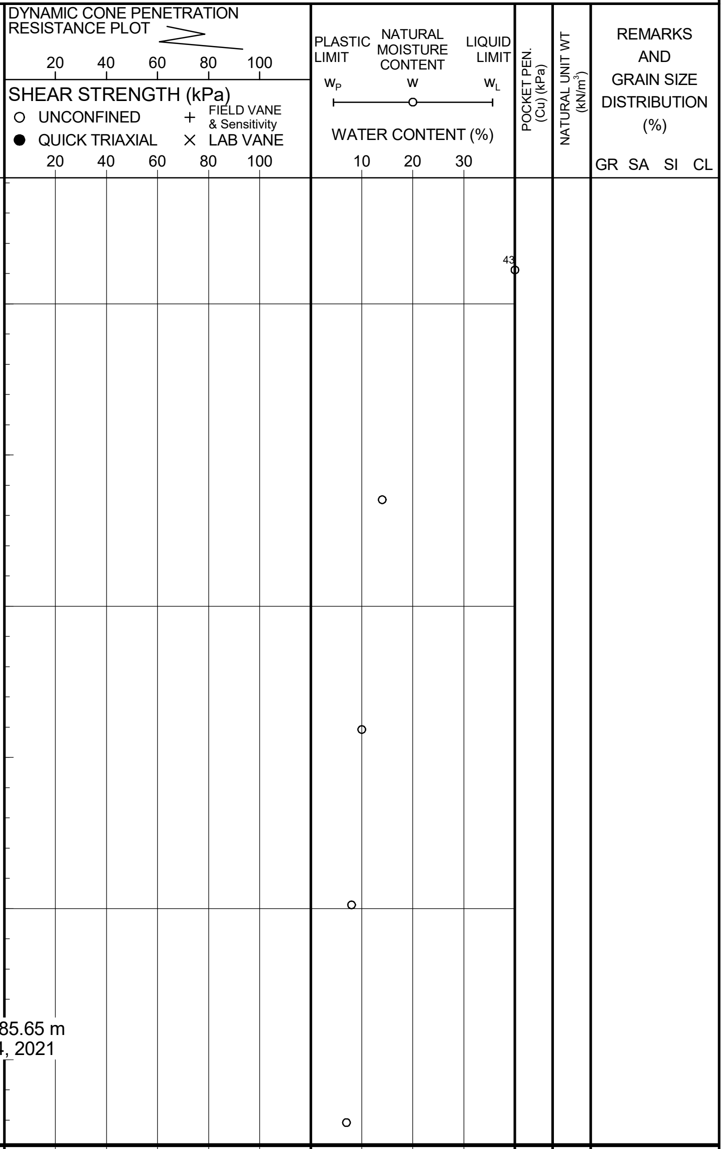
Method: Solid Stem Auger
Diameter:
Date: Jan/04/2021 to Jan/04/2021

REF. NO.: 201-11808-00
ENCL NO.: 1
ORIGINATED BY LG

(m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m
88.42	Ground Surface			
88.34 0.00	76mm TOPSOIL			
0.08	Reddish brown, moist, firm, SILTY CLAY, trace sand, trace gravel; FILL	1	SS	5
87.66 0.76	Reddish brown, moist, very stiff to hard, SILTY CLAY, trace sand; (CL)	2	SS	13
		3	SS	68
86.49 1.93	INFERRED SHALE BEDROCK	4	SS	100/ 230mm
		5	SS	55
85.22 3.20	**END OF BOREHOLE**			

Notes:
1) Borehole terminated at proposed depth
2) Borehole was open and dry upon completion of drilling

W. L. 85.65 m
Jan 14, 2021



WSP-SOIL-ROCK-MAY-29-2017.GLB WSP SOIL LOG 2016 WITH PID 201-11808-00 DRAFT.GPJ 1/20/21

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

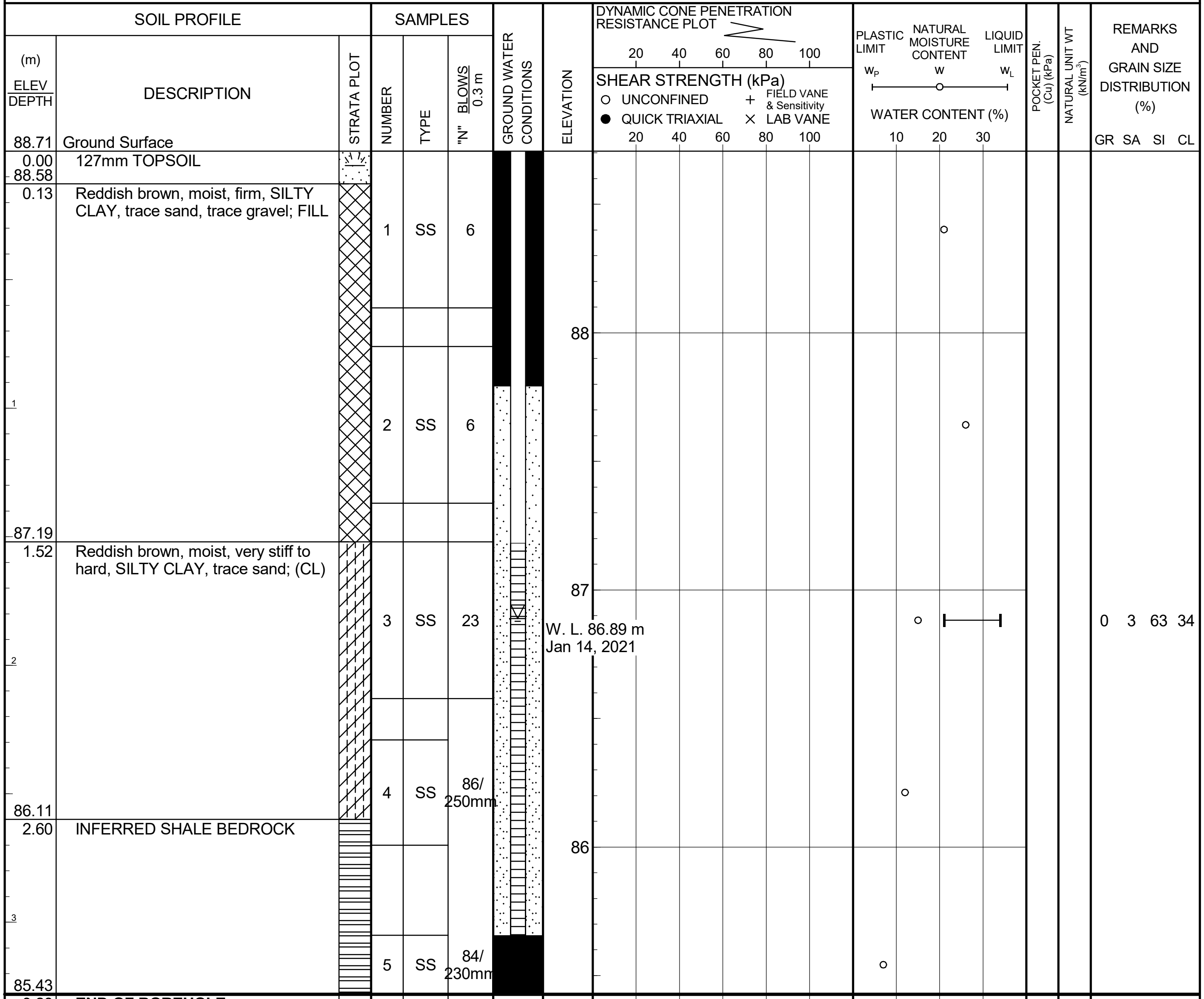
GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
$○^{8}$=3% Strain at Failure

LOG OF BOREHOLE BH21-2

PROJECT: Preliminary Geotechnical Investigation
CLIENT: Region of Halton
PROJECT LOCATION: 1258 Rebecca Street
DATUM: Geodetic
BH LOCATION: N 4808009.416 E 605416.745

Method: Solid Stem Auger
Diameter:
Date: Jan/04/2021 to Jan/04/2021

REF. NO.: 201-11808-00
ENCL NO.: 2
ORIGINATED BY LG



ELEV (m) DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
88.71	Ground Surface				
0.00	127mm TOPSOIL				
88.58 0.13	Reddish brown, moist, firm, SILTY CLAY, trace sand, trace gravel; FILL	1	SS	6	
		2	SS	6	
87.19 1.52	Reddish brown, moist, very stiff to hard, SILTY CLAY, trace sand; (CL)	3	SS	23	0 3 63 34
		4	SS	86/ 250mm	
86.11 2.60	INFERRED SHALE BEDROCK	5	SS	84/ 230mm	
85.43 3.28	**END OF BOREHOLE**				

Notes:
1) Borehole terminated at proposed depth
2) Borehole was open and dry upon completion of drilling

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3, \times^3$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

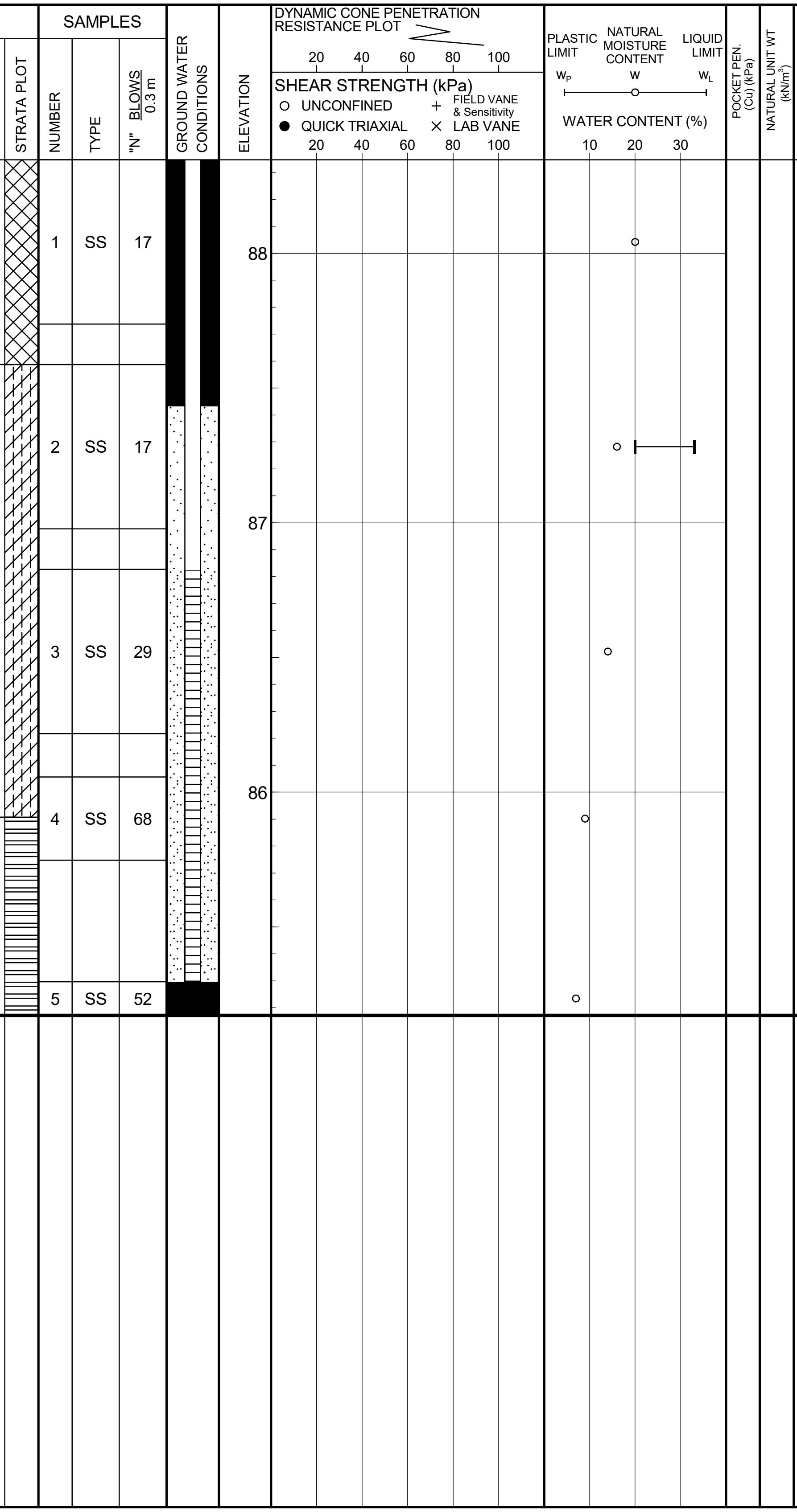
LOG OF BOREHOLE BH21-3

PROJECT: Preliminary Geotechnical Investigation
CLIENT: Region of Halton
PROJECT LOCATION: 1258 Rebecca Street
DATUM: Geodetic
BH LOCATION: N 4808020.833 E 605456.283

Method: Solid Stem Auger
Diameter:
Date: Jan/04/2021 to Jan/04/2021

REF. NO.: 201-11808-00
ENCL NO.: 3
ORIGINATED BY LG

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GR	SA	SI	CL
88.35	Ground Surface							
0.00	Reddish brown, moist, very stiff, SILTY CLAY, trace sand, trace gravel; FILL 100mm sand and gravel layer	1	SS	17				
87.59 0.76	Reddish brown, moist, very stiff, SILTY CLAY, trace sand; (CL)	2	SS	17	0	3	56	41
		3	SS	29				
85.91 2.44	INFERRED SHALE BEDROCK	4	SS	68				
		5	SS	52				
85.17 3.18	**END OF BOREHOLE** Notes: 1) Borehole terminated at proposed depth 2) Borehole was open and dry upon completion of drilling							



REMARKS AND GRAIN SIZE DISTRIBUTION (%)

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

WSP-SOIL-ROCK-MAY-29-2017.GLB
WSP SOIL LOG 2DIG WITH PID 201-11808-00_DRAFT.GPJ 1/20/21

LOG OF BOREHOLE BH21-4

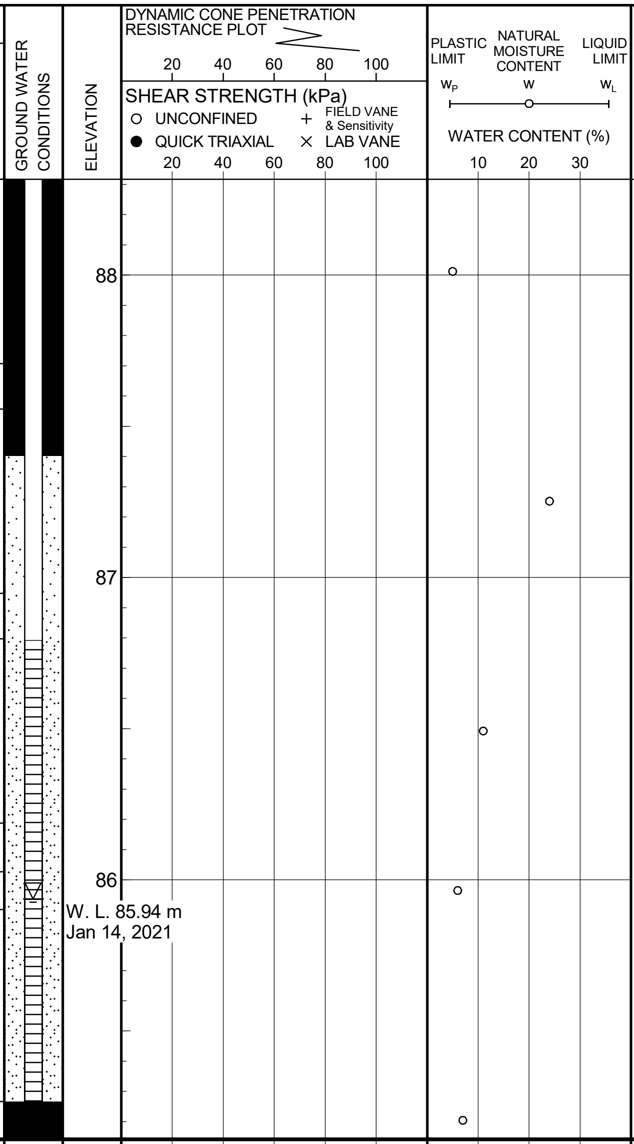
PROJECT: Preliminary Geotechnical Investigation
REF. NO.: 201-11808-00
CLIENT: Region of Halton
Method: Solid Stem Auger
ENCL NO.: 4
PROJECT LOCATION: 1258 Rebecca Street
Diameter:
ORIGINATED BY LG
DATUM: Geodetic
Date: Jan/04/2021 to Jan/04/2021
BH LOCATION: N 4807972.716 E 605433.078

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT
88.32	Ground Surface	
0.00	50mm TOPSOIL	
88.27 0.05	Reddish brown, moist, firm, SILTY CLAY, trace sand, trace gravel; FILL 175mm sand and gravel layer	
87.56 0.76	Reddish brown, moist, stiff to hard, SILTY CLAY, trace sand; (CL)	
86.32 2.00	INFERRED SHALE BEDROCK	
85.14 3.18	**END OF BOREHOLE** Notes: 1) Borehole terminated at proposed depth 2) Borehole was open and dry upon completion of drilling	

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m
1	SS	7
2	SS	10
3	SS	45
4	SS	52
5	SS	51



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

REMARKS AND GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3 , × 3 : Numbers refer to Sensitivity

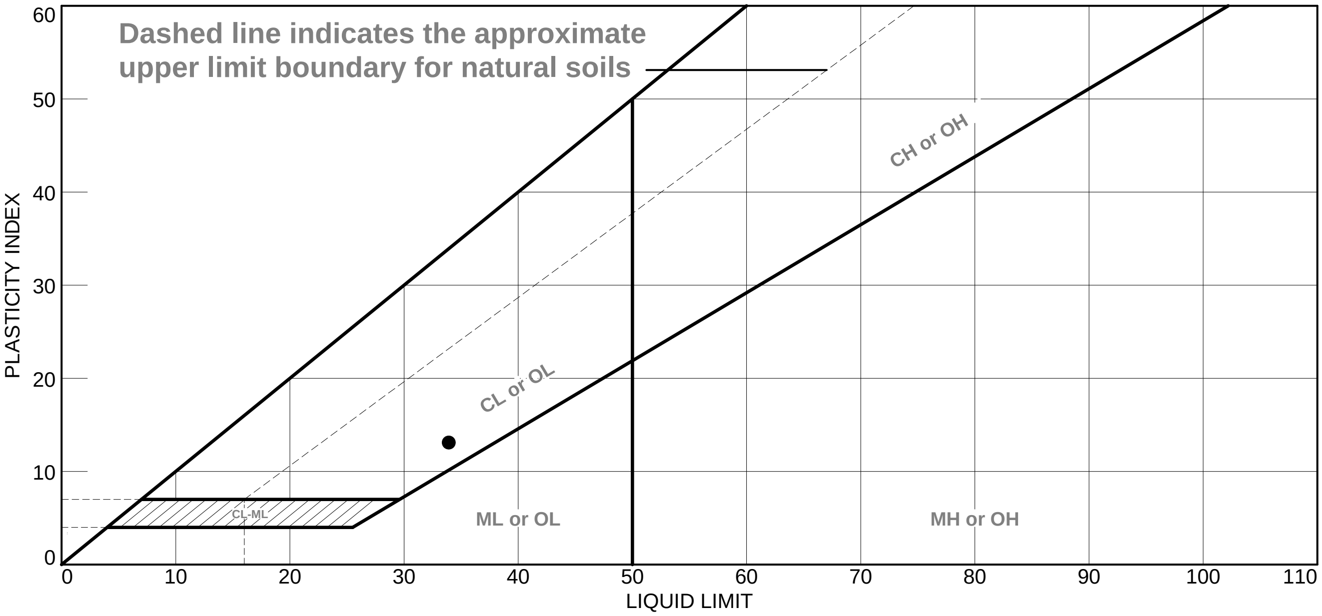
○ 8=3% Strain at Failure

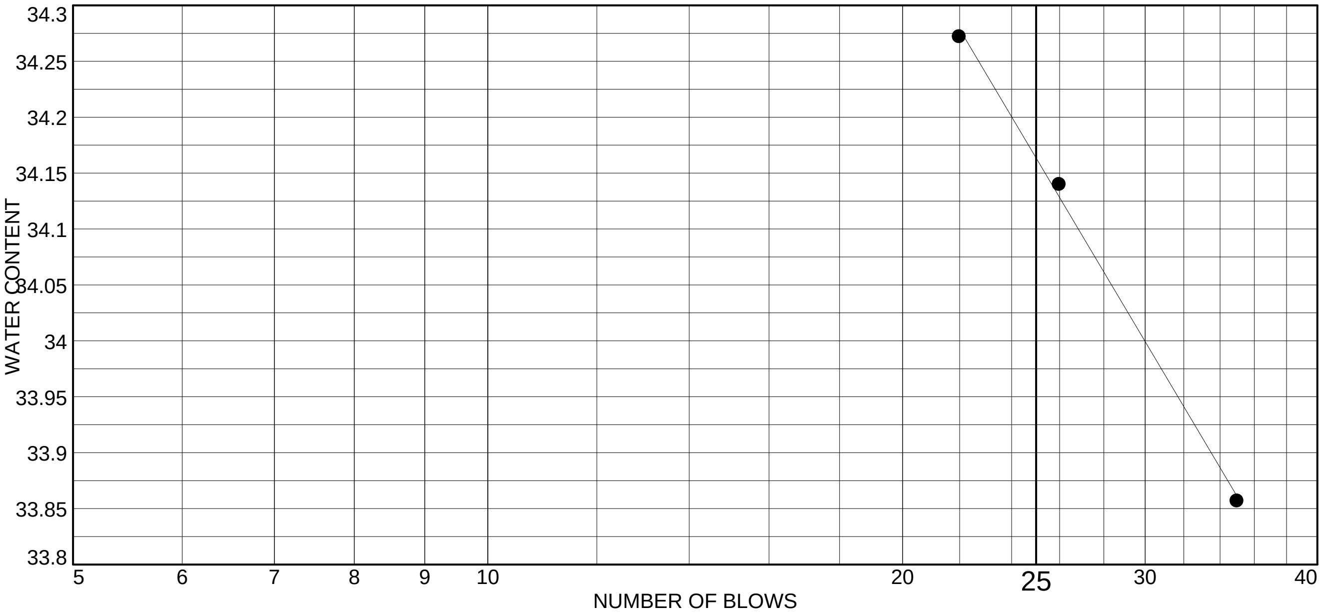
APPENDIX

B

GEOTECHNICAL LABORATORY TEST RESULTS

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty Clay	34	21	13	99	97	CL

Project No. 201-11808- **Client:** Region of Halton

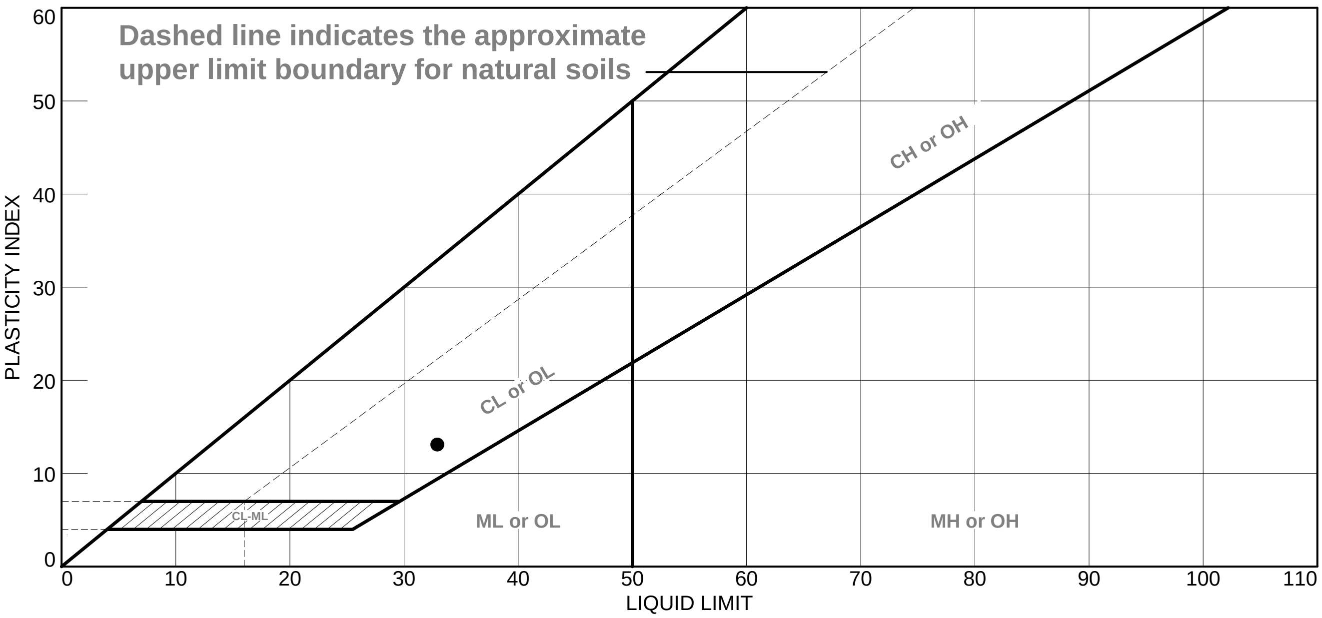
Project: 1258 Rebecca street

Sample Number: 20-02_SS3

Remarks:

Figure R1602-A-1

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty Clay	33	20	13	99	97	CL

Project No. 201-11808- **Client:** Region of Halton

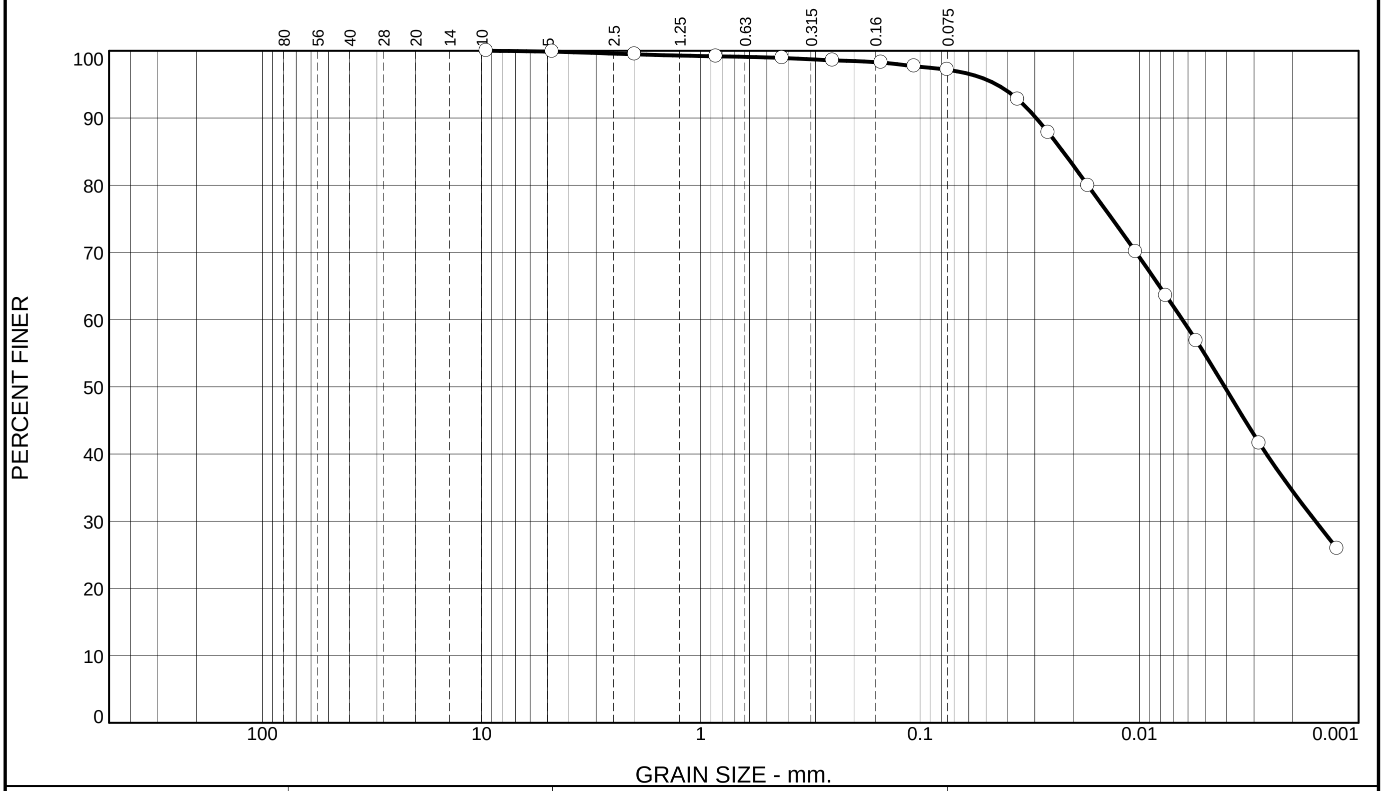
Project: 1258 Rebecca street

Sample Number: 20-03_SS2

Remarks:

Figure R1602-A-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	1	0	2	63	34

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
9.50	100		
4.75	100		
2.00	99		
0.85	99		
0.425	99		
0.25	99		
0.15	98		
0.106	98		
0.075	97		
0.0358 mm.	93		
0.0260 mm.	88		
0.0171 mm.	80		
0.0104 mm.	70		
0.0076 mm.	64		
0.0055 mm.	57		
0.0028 mm.	42		
0.0013 mm.	26		

* (no specification provided)

Soil Description

Silty Clay

Atterberg Limits

PL= 21 LL= 34 PI= 13

Coefficients

D_{90}= 0.0296 D_{85}= 0.0223 D_{60}= 0.0064
D_{50}= 0.0041 D_{30}= 0.0016 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(13)

Remarks

Sample Number: 20-02_SS3

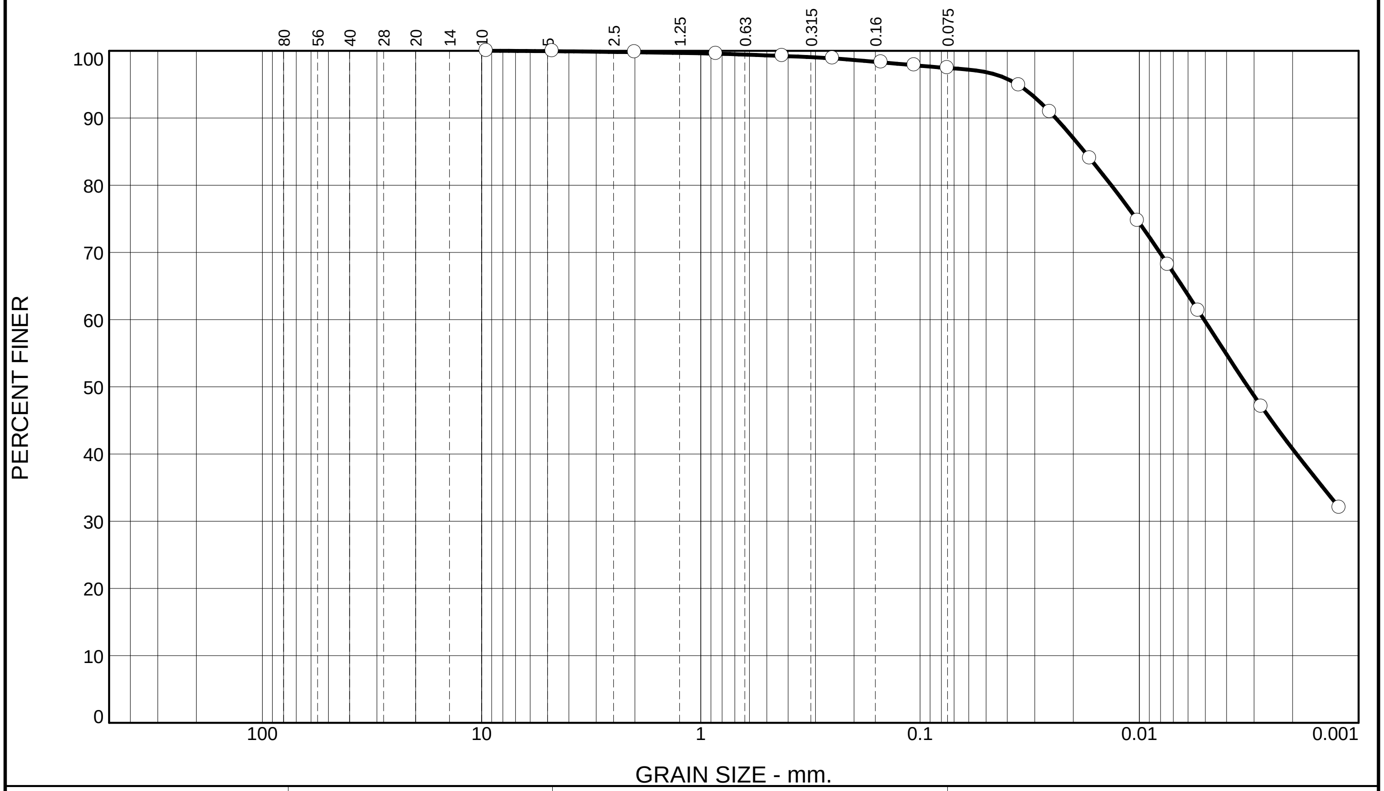
Date: Jan. 11,2021

Client: Region of Halton
Project: 1258 Rebecca street

Project No: 201-11808-00 **Figure** R1602-H-1

Tested By: Bruce Shan/Bonnie Wang

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	1	2	56	41

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
9.50	100		
4.75	100		
2.00	100		
0.85	100		
0.425	99		
0.25	99		
0.15	98		
0.106	98		
0.075	97		
0.0354 mm.	95		
0.0256 mm.	91		
0.0168 mm.	84		
0.0102 mm.	75		
0.0074 mm.	68		
0.0054 mm.	61		
0.0028 mm.	47		
0.0012 mm.	32		

* (no specification provided)

Soil Description

Silty Clay

Atterberg Limits

PL= 20 LL= 33 PI= 13

Coefficients

D_{90}= 0.0241 D_{85}= 0.0178 D_{60}= 0.0051
D_{50}= 0.0032 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(13)

Remarks

Sample Number: 20-03_SS2

Date: Jan. 11,2021

Client: Region of Halton
Project: 1258 Rebecca street

Project No: 201-11808-00 **Figure** R1602-H-2

Tested By: Bruce Shan/Bonnie Wang