- **The Rousseau Group Inc.**

Geotechnical Investigation

Type of Document
Final

Project Name
Proposed Residential Development
2421 and 2431 New Street
Burlington, Ontario

Project Number
HAM-00801287-A0 Rev. 1

Prepared By:
EXP Services Inc.
80 Bancroft Street
Hamilton, ON L8E 2W5
Canada

Date Submitted
05.13.2019

Table of Contents

1 Introduction 3
2 Field Investigation 3
3 Subsurface Conditions 4
3.1 Stratigraphy 4
3.1.1 Pavement Structure 4
3.1.2 Fill 4
3.1.3 Sand / Sand & Silt 5
3.1.4 Bedrock 5
3.2 Groundwater 5
4 Engineering Discussion & Recommendations 6
4.1 Building Foundation Recommendations 6
4.2 Excavations 6
4.2.1 Temporary Shoring 7
4.3 Groundwater Control 7
4.4 Building Floor Slab-on-Grade and Permanent Drainage 8
4.5 Backfill 8
4.6 Earthquake Considerations 9
4.6.1 Subsoil Conditions 9
4.6.2 Site Classification 9
4.7 Pavement Design 9
4.7.1 General Comments on Permeable Pavement 11
5 General Comments 11

List of Appendices

Appendix

Drawings ..... **A**
Laboratory Test Results ..... **B**

1 Introduction

This report presents the results of the geotechnical investigation carried out 2421 and 2431 New Street in Burlington, Ontario. The three existing single storey commercial structures will be demolished to make way for the construction of a new residential building. The proposed development will consist of an 11 storey condominium with 2 levels of underground parking. A Phase II Environmental Site Assessment (ESA) was also carried out by EXP at the subject property with the results presented under separate cover. The investigation was authorized by Mr. Charles Mady on behalf of The Rousseau Group (TRG) Inc.

The purpose of the investigation was to determine the subsoil and groundwater conditions at the site by putting down a limited number of boreholes and based on an assessment of the factual borehole data, provide an engineering report containing general geotechnical recommendations pertinent to the proposed development.

The comments and recommendations given in this report are based on the assumption that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or the requirement of additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2 Field Investigation

The fieldwork for this investigation was carried out between November 27 and 29, 2017. The boreholes were completed by a combination of auger and split spoon techniques using truck mounted equipment owned and operated by a specialized drilling subcontractor. Prior to the commencement of the drilling operations, private and public underground services were cleared to minimize the risk of contacting any such services during the drilling operations.

Four (4) boreholes, numbered BH-201 to BH-204, were advanced in accessible areas at the site. The boreholes were each advanced into the underlying shale bedrock and were terminated at depths ranging from 8.9 to 9.2 m below grade. The approximate borehole locations are shown on Drawing No. 1 in Appendix A.

The soil samples were obtained using a 51 mm (2 inch) outside diameter split spoon sampler in conjunction with Standard Penetration Tests (ASTM D1586) at the depths noted on the Borehole Logs in Appendix A. The Standard Penetration Test (SPT) N values were recorded and used to provide an assessment of the compactness condition of the in-situ soils.

The retained soil samples were logged in the field and then carefully packaged and transported to our Hamilton laboratory for detailed visual, textural and olfactory classification. Moisture content determinations were carried out on all soil samples and the results are summarized on the borehole logs in Appendix A. The bedrock was cored using HQ-sized equipment and the retrieved samples were preserved in core boxes and transported to the Hamilton laboratory for detailed review.

Groundwater observations were and measurements were carried out during and upon completion of the drilling operations. Groundwater monitoring wells (50 mm diameter) were installed in all Boreholes to facilitate long-term groundwater level measurements and were protected with flush-mount protective casings.

Ground surface elevations at the borehole locations were surveyed by EXP and referenced to a temporary benchmark (TBM), described as follows:

TBM: The centre of the manhole located near the southwest corner of the property, approximately 4 m north of the sidewalk on the north side of New Street.

Elevation: 90.21 m (as obtained from the drawing entitled *Topography of 2421 and 2431 New Street, City of Burlington,* 17-218TP01, dated November 29, 2017 provided by the Client)

3 Subsurface Conditions

Details of the subsurface conditions encountered during the drilling program are summarized on the borehole logs included in Appendix A.

The logs include textural descriptions of the subsoil and indicate the soil boundaries inferred from non-continuous sampling and observations during drilling. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The "Notes on Sample Description" preceding the logs form an integral part of and should be read in conjunction with this report.

3.1 Stratigraphy

Below the surficial pavement structure or topsoil, the boreholes generally encountered fill extending to depths ranging from 0.8 to 1.5 m below grade overlying native sand and shale-till complex. Shale bedrock was encountered at all borehole locations at depths ranging from 5.0 to 5.8 m below existing grade. Details of the encountered materials are presented in the following sections.

3.1.1 Pavement Structure

Boreholes BH-201 to BH-203 were advanced in the existing parking area and encountered the surficial pavement structure. The pavement structure consisted of approximately 65 to 80 mm of asphaltic concrete overlying 120 to 160 mm of granular fill.

3.1.2 Fill

Silty sand or sandy silt fill materials were encountered at each borehole location below the surficial pavement structure or topsoil and extended to depths ranging from 0.8 to 1.5 m below grade. The fill was brown or light brown and in a moist state and typically contained organic staining and/or rootlets. Moisture contents of the fill materials ranged from 4 to 18 percent of dry mass.

3.1.3 Sand / Sand & Silt

Sand to sand & silt was encountered at all borehole locations below the fill and extended to depths ranging from 5.0 to 5.8 m below grade. The sand contained trace to some silt. The sand and the sand & silt were brown and in a moist state, becoming very moist to wet below a depth 2.3 m. Moisture contents of the stratum ranged from 9 to 25 percent of dry mass. Based on SPT N values ranging from 2 to 38 blows per 305 mm penetration, the stratum is classified as very loose to compact.

Grain size analyses of the sand and sand & silt are included in Appendix B.

3.1.4 Bedrock

The bedrock surface was encountered at depths ranging from 5.0 to 5.8 m below grade. The bedrock was cored for approximately 3.1 to 3.6 m at all borehole locations. The bedrock consisted of red shale with an RQD of 0 to 94 percent, indicating the rock is of *very poor* to *excellent* quality.

Based on the Ministry of Northern Development and Mines Map 2544, *Bedrock Geology of Ontario, Southern Sheet*, the bedrock in the site vicinity consists of Queenston Shale of the Upper Ordovician period. The upper portion of the bedrock is typically weathered and becomes more sound with depth. Hard limestone, dolostone, and siltstone lenses may also be encountered within the shale.

3.2 Groundwater

Groundwater conditions were monitored in the open boreholes during and upon completion of the drilling operations. Groundwater monitoring wells (50 mm diameter) were installed in all Boreholes to facilitate long-term groundwater level measurements. The groundwater level measurements varied from 2.5 to 3.0 m below grade on December 12, 2014 as shown in the table below:

Table 3-1: Groundwater Level Measurements

Borehole No.		Groundwater Level Measurement (m)		
		December 6, 2017	December 7, 2017	December 12, 2017
BH-201	Shallow	---	2.5	2.5
	Deep	---	2.5	2.5
BH-202	Shallow	2.6	---	2.8
	Deep	2.7	---	2.7
BH-203	Shallow	2.9	---	3.0
	Deep	2.7	---	2.7
BH-204	Shallow	3.0	---	3.0
	Deep	2.9	---	2.9

Seasonal variations in the water table should be anticipated, with higher levels occurring during wet weather conditions (spring thaw and late fall) and lower levels occurring during dry weather conditions.

4 Engineering Discussion & Recommendations

It is understood that the existing buildings will be demolished and an 11 storey residential building with 2 levels of underground parking will be constructed. An associated above ground parking area will also be constructed. Recommendations for the proposed construction are provided in the following sections.

4.1 Building Foundation Recommendations

Foundations for the proposed building with 2 levels of underground parking are expected to extend to depths in the order of 7 m below existing grade. As such, conventional spread and strip footing foundations on sound bedrock are considered feasible at the site.

Conventional spread and strip footing foundations constructed below the weathered shale on sound shale bedrock can be designed with a geotechnical resistance of 3,000 kPa at Ultimate Limit State (ULS). The expected deformation of the shale bedrock is less than the normally tolerated limits of 25 mm and therefore ULS governs. Prior to placement of foundation concrete, all existing fill, native soil, and weathered shale bedrock must be removed down to the sound shale bedrock. The exposed bearing surface is to be inspected by EXP.

4.2 Excavations

Excavations across the site are expected to extend to depths in the order of 7 m below existing grade and will encounter the overburden soils and shale bedrock. Excavation of the soil at this site is not expected to pose any difficulty and can be carried out with heavy hydraulic excavators. Excavations into the shale will require the use of ripping teeth. The use of hydraulic hammers will be required to penetrate the harder sound shale bedrock and layers of limestone which are normally interbedded within the shale. Provisions should be made in the excavation contract to include the use of this equipment for excavations in bedrock.

Excavations for the foundations should be carried out so as to minimize the disturbance of shale at the design foundation elevations. In this regard, it is recommended that a hydraulic hammer be used for foundation excavation.

All excavations must be completed in accordance with the most recent regulations of the Ontario Occupation Health and Safety Act (OHSA). The encountered soils may generally be classified as Type 3 Soil above the groundwater level and Type 4 below the groundwater level. In accordance with the OHSA regulations if the excavation contains more than one type of soil, the soil shall be classified as the type with the highest number. The OSHA requires that excavation slopes be cut at predetermined inclinations, based on the soil types encountered. The need to excavate flatter side slopes if excessively wet or soft/loose materials, or concentrated seepage zones are encountered, should not be overlooked. Water (i.e. surface water runoff) should not be permitted to enter and/or pond within the construction area.

Given the proximity to the adjacent infrastructure, the excavation slopes required by the OHSA will not be possible at all locations and the use of temporary shoring will be required. Comments on temporary shoring methods are provided in the following section.

4.2.1 Temporary Shoring

The shoring method chosen by the structural engineer and/or contractor will depend on the settlement tolerance of the surrounding structures and infrastructure. Consideration should be given to using a shoring method which minimizes groundwater infiltration. Where settlement sensitive structures or services are located within a distance from the excavation equal or less than the excavation depth, the use of a rigid retaining structure will be required. In general, temporary shoring can be laterally restrained with tieback anchors, rakers, struts, or corner braces. The shoring system should be designed in accordance with the guidelines provided in the Canadian Foundation Engineering Manual (CFEM).

EXP should be retained to review the shoring design, to monitor installation and testing of the system, and to monitor the shoring movements during all phases of the excavation. Inclinometers should be installed at locations where buildings or services lie close to the excavation. Careful monitoring is needed in any shored excavation, especially when buildings are located in close proximity. This is necessary to anticipate when and if additional support is needed.

The lateral earth pressures on the temporary shoring system may be calculated using the expression below. Unless dewatering is carried out and an effective drainage system prevents build-up of hydrostatic pressure behind the wall, the additional pressure from groundwater is to be included.

$$p = K(\gamma h+q)$$

where p = lateral earth pressure in kPa acting at depth h (m)

K_a = earth pressure coefficient, 0.3 (sand)

γ = unit weight of soil surrounding the structure, 22 kN/m^3

h = depth to point of interest in m

q = equivalent value of surcharge on the ground surface, kPa

If tieback anchors are to be used for the shoring structure, the anchors should be embedded into compact sand or shale bedrock. A maximum bond stress of 70 kPa may be used for the compact sand and 150 kPa may be used for the shale bedrock. It will be necessary to perform load tests on the tiebacks to confirm the bond stresses assumed in the design of anchors. At a minimum, all anchors should be stressed to 130 percent of their design capacity and then locked-off at the design load. The bond length is that portion of the anchor which extends beyond the no-load zone 60 degrees from the bottom of the shoring.

A passive resistance of 125 kN/cubic meter equivalent fluid pressure acting over two pile diameters may be used for the design of the soldier piles or caissons. The passive resistance should be calculated for a depth beginning 0.3 m below the excavation grade.

4.3 Groundwater Control

The samples retrieved from the boreholes were noted to become very moist to wet below a depth of 2.3 m. Groundwater levels in the monitoring wells on site ranged from 2.5 to 3.0 m below grade on

December 12, 2017. Excavations extending below these depths will require potentially significant dewatering methods or implementation of a shoring method which minimizes groundwater infiltration. Reference should be made to the EXP hydrogeological study for the subject property for additional groundwater and dewatering comments and recommendations.

Dewatering requirements will be governed by the time of year the construction is performed. It is the responsibility of the contractor to propose a suitable dewatering system based on the time of construction and the groundwater levels. The method used should not undermine adjacent structures.

4.4 Building Floor Slab-on-Grade and Permanent Drainage

It is understood that the proposed building will include 2 levels of underground parking. The basement floor slab-on-grade can be supported on the encountered native soils or shale bedrock. It is recommended that the exposed subgrade be examined by a geotechnical engineer prior to constructing the floor slab-on-grade. Any soft or loose areas encountered during the review should be sub-excavated and replaced with approved fill placed in lifts not exceeding 200 mm and compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD) within 2 percent of the optimum moisture content.

The floor slab should be cast on a moisture barrier consisting of 19 mm clear stone with a thickness of at least 200 mm. The clear stone layer will minimize the capillary rise of moisture from the subsoil to the floor slab (moisture barrier). Adequate saw cuts should be provided in the floor slab as directed by the structural engineer to help control cracking.

For the basement slab-on-grade, the installation of a perimeter drainage and underfloor drainage system at 3 m intervals is required. The exterior grade should be sloped to ensure positive drainage of surface water away from the structure and reduce groundwater infiltration adjacent to the foundations.

4.5 Backfill

Backfill used to satisfy under slab requirements and service trenches, etc., should be compactible fill, i.e., inorganic soil with its moisture content close to its optimum moisture content as determined in the standard Proctor test. Fill placed below concrete slab areas should be compacted to at least 98 percent Standard Proctor Maximum Dry Density (SPMDD) in lifts not exceeding 200 mm.

To minimize potential problems, trench backfilling operations should follow closely after excavation so that only minimal length of trench slope is exposed. This will minimize wetting of the subgrade material. Should construction extend to the winter season, particular attention should be given to ensure that frozen material is not used as backfill.

The majority of excavated material will likely consist of fill and native sand and sand & silt. In general, the excavated material may be reused for backfill subject to the removal of any topsoil or other obviously unsuitable material. However, given the relatively high groundwater level at the site, moisture content adjustment (drying) of re-used soils will likely be required.

In general, the overburden soils are not free draining and therefore should not be used where this characteristic is required, or in confined areas. Imported granular material conforming to OPSS Granular B Type I or II would be suitable for these purposes.

All backfilling and compaction operations must be closely examined by a qualified geotechnical consultant to ensure uniform compaction to specification requirements, especially in the vicinity of manholes and catch basins, and in all areas that are not readily accessible to compaction equipment.

4.6 Earthquake Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading are presented in the following sections.

4.6.1 Subsoil Conditions

The subsoil and groundwater information at this site have been examined in relation to Section 4.1.8.4 of the OBC 2012. The proposed building is anticipated to be founded within the shale bedrock. There have been no shear wave velocity measurements carried out at this site and therefore, N values, shear strengths, and EXP's knowledge of the subsurface conditions in the area have been used to determine the site classification. Based on a weighted average SPT N value of 84, this corresponds to an average shear wave velocity of approximately 800 m/s.

4.6.2 Site Classification

Based on the above assumptions and interpretations and the known soil conditions, the Site Class for this site is "B" as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012.

4.7 Pavement Design

It is our understanding that this project will include the construction of new driveway and parking areas. The recommended pavement structure is provided in Table 4-1 below and is based on an estimate of the subgrade soil properties determined from visual examination and textural classification of the soil samples and traffic requirements. Consequently, the recommended pavement structure should be considered for preliminary design purposes only.

Table 4-1: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Medium Duty Traffic	Heavy Duty Traffic
Asphaltic Concrete (OPSS 310)	Min 92.0% Maximum Relative Density (MRD)	40 mm HL3 50 mm HL8	40 mm HL3 80 mm HL8
Granular A Crushed Limestone (OPSS 1010)	100% SPMDD[1]	150 mm	150 mm
Granular B Type II (OPSS 1010)	100% SPMDD[1]	200 mm	350 mm

[1]Denotes Standard Proctor Maximum Dry Density, ASTM-D698

The granular base and sub-base must be placed in maximum 200 mm lifts and compacted to 100 percent of the Standard Proctor Maximum Dry Density (SPMDD) at moisture content within 2 percent of the optimum moisture content. The subgrade should be compacted to 98 percent SPMDD for at least

the upper 500 mm. The recommended pavement structures outlined assumes adequate provision for drainage.

The foregoing design assumes construction is carried out during dry periods and the subgrade is stable under the load of construction equipment. If construction is carried out during wet weather, and heaving or rolling of the subgrade is experienced, additional thickness of sub-base course material may be required.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over emphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. Subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening.

Additional comments on the construction of the paved areas are as follows:

- The location and extent of sub-drainage required within the paved areas should be reviewed by this office in conjunction with the proposed site grading. In view of the fine-grained nature of the subgrade soils, perimeter subdrains should be provided.
- To minimize problems of differential movement between the pavement and catch basins/manholes due to frost action, the backfill around the structures should consist of free draining granular.
- The most severe loading conditions on light duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as half loads during paving, etc. may be required, especially if construction is carried out during unfavourable weather.
- As part of the subgrade preparation, proposed paved areas should be stripped of obviously unsuitable material. Fill required to raise the grades to design elevations should be inorganic and have moisture contents within 2 percent of the optimum value determined from the Standard Proctor Test. The subgrade should be properly shaped, crowned, and then proof-rolled in the full-time presence of a representative of this office. Soft or spongy subgrade areas should be sub-excavated and properly replaced with suitable approved backfill compacted to at least 95 percent SPMDD.

4.7.1 General Comments on Permeable Pavement

Permeable pavements, an alternative to traditional impervious pavement, allow stormwater to drain between the individual paver units or the porous material and into a stone reservoir where it is infiltrated into the underlying native soil or temporarily retained in the void spaces of the stone layer. They can be used for low traffic roads, parking lots, driveways, pedestrian plazas and walkways.

The overflow requirements, outlet details, and storage capacity for permeable pavements is determined by the civil consultant based on the site design criteria and subgrade infiltration properties. The encountered native subgrade soil was predominantly sand with varying amounts of silt and has an estimated average percolation rate (T-time) of 10 mins/cm or an infiltration rate of 60 mm/hr, which would generally be considered sufficiently permeable for permeable pavement design. The estimated average infiltration rate is based on the single well response tests (SWRT) completed as part of the Preliminary Hydrogeological Investigation carried out by EXP and the grain size analyses included in Appendix B. In-situ permeability testing may be conducted at the locations where permeable pavements are proposed if more precise infiltration rates are required.

The base of the permeable pavement reservoir should be at least 1.0 m above the seasonally high groundwater level. The groundwater level measurements obtained as part of the current geotechnical investigation and as presented in the Preliminary Hydrogeological Investigation ranged from 2.2 m to 3.1 m below existing grade.

EXP can be contacted to review the proposed permeable pavement design to ensure structural adequacy once the products, materials, and storage capacity are known.

5 General Comments

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions of the subject property. The conclusions presented in this report reflect site conditions existing at the time of the investigation.

EXP Services Inc. should be retained for a 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, EXP Services Inc. 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 results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

More specific information, with respect to the conditions between samples, or the lateral and vertical extent of materials, may become apparent during excavation operations. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent; should this occur, EXP Services Inc. should be contacted to assess the situation and additional testing and reporting may be required. EXP Services Inc. has qualified personnel to provide assistance regarding future geotechnical and environmental issues related to this property.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

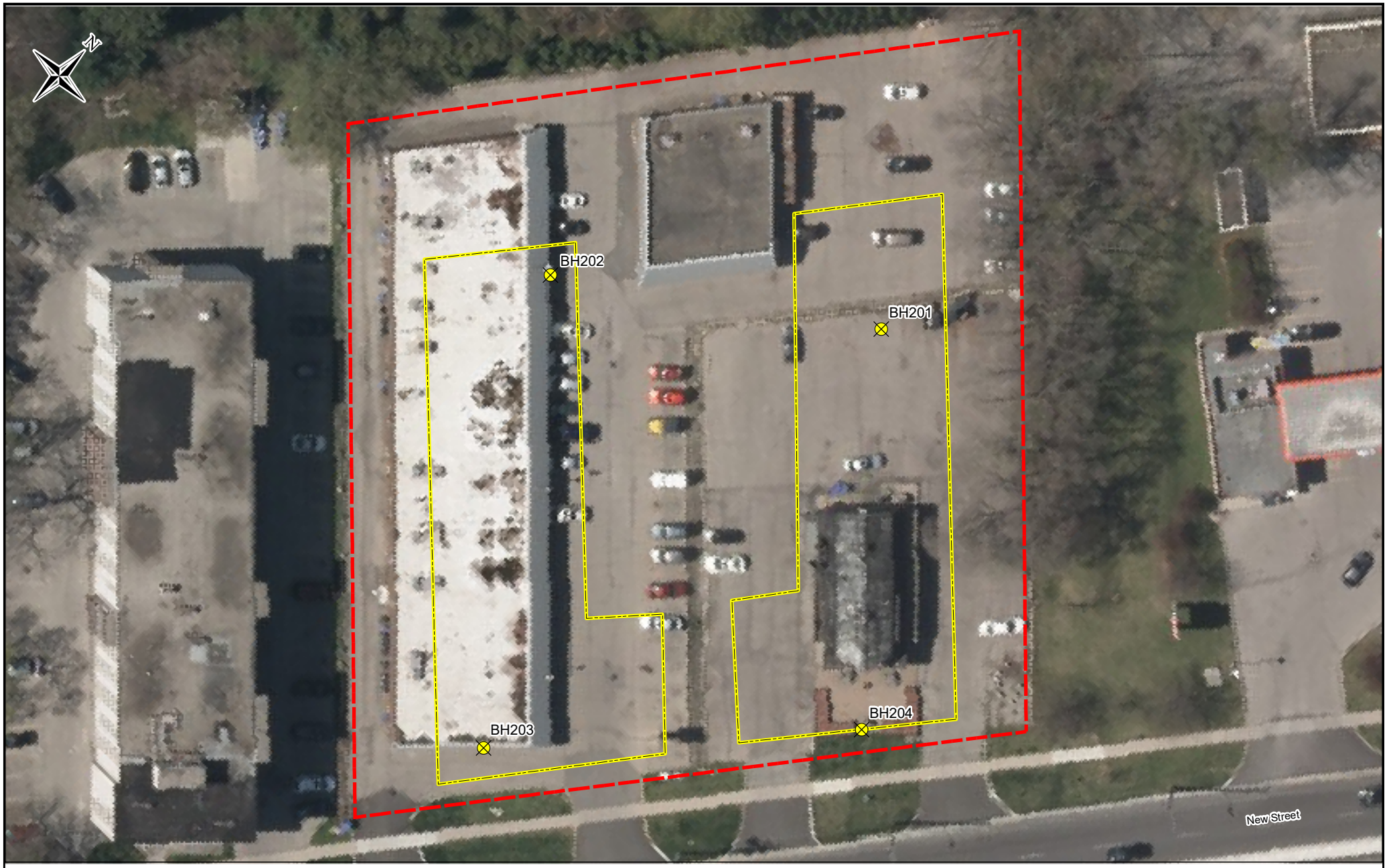
EXP Services Inc.



Matt LiVecchi, P.Eng.
Geotechnical Project Manager

Jeffrey Golder, P.Eng.
Manager, Hamilton Geotechnical Services

Appendix A – Drawings



Legend

Site Boundary | Proposed Building | Geotechnical Borehole

0 5 10 20 30 40 50 m

EXP Services Inc.
80 BANCROFT STREET
HAMILTON, ONTARIO
L9B 1A5
T - (905) 573-4000
F - (905) 573-9693

PROJECT TITLE:

GEOTECHNICAL INVESTIGATION
2421 AND 2431 NEW STREET
BURLINGTON, ONTARIO

DRAWING TITLE:

BOREHOLE LOCATION PLAN

PROJECT No.: HAM-00801287-A0	DWN: EE
SCALE: AS NOTED	CHKD: JG
DATE: DECEMBER 2017	DWG. No.: 1

Notes on Sample Descriptions

1. All sample descriptions included in this report follow the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), as outlined in the Canadian Foundation Engineering Manual. Note, however, that behavioral properties (i.e. plasticity, permeability) take precedence over particle gradation when classifying soil. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

UNIFIED SOIL CLASSIFICATION

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



EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

ISSMFE SOIL CLASSIFICATION

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

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

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 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.

Notes On Soil Descriptions

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay and Silt	<0.060 mm	"trace" (e.g. Trace sand)	1% to 10%
Sand	0.060 to 2.0 mm	"some" (e.g. Some sand)	10% to 20%
Gravel	2.0 to 75 mm	adjective (e.g. sandy, silty)	20% to 35%
Cobbles	75 to 200 mm	"and" (e.g. and sand)	35% to 50%
Boulders	>200 mm		

The compactness of Cohesionless soils and the consistency of the cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil		
Compactness	**Standard Penetration Resistance "N" Blows / 0.3 m**	**Consistency**	**Undrained Shear Strength (kPa)**	**Standard Penetration Resistance "N" Blows / 0.3 m**
Very Loose	0 to 4	Very soft	<12	<2
Loose	4 to 10	Soft	12 to 25	2 to 4
Compact	10 to 30	Firm	25 to 50	4 to 8
Dense	30 to 50	Stiff	50 to 100	8 to 15
Very Dense	Over 50	Very Stiff	100 to 200	15 to 30
		Hard	>200	>30

5. **ROCK CORING**

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundless of the rock mass. It is obtained from the rock cores by summing the length of the core covered, counting only those pieces of sound core that are 100 mm or more length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

Log of Borehole BH-201

Project No. HAM-00801287-A0 Drawing No. 3

Project: Proposed Residential Development Sheet No. 1 of 1

Location: 2421 and 2431 New Street, Burlington, ON

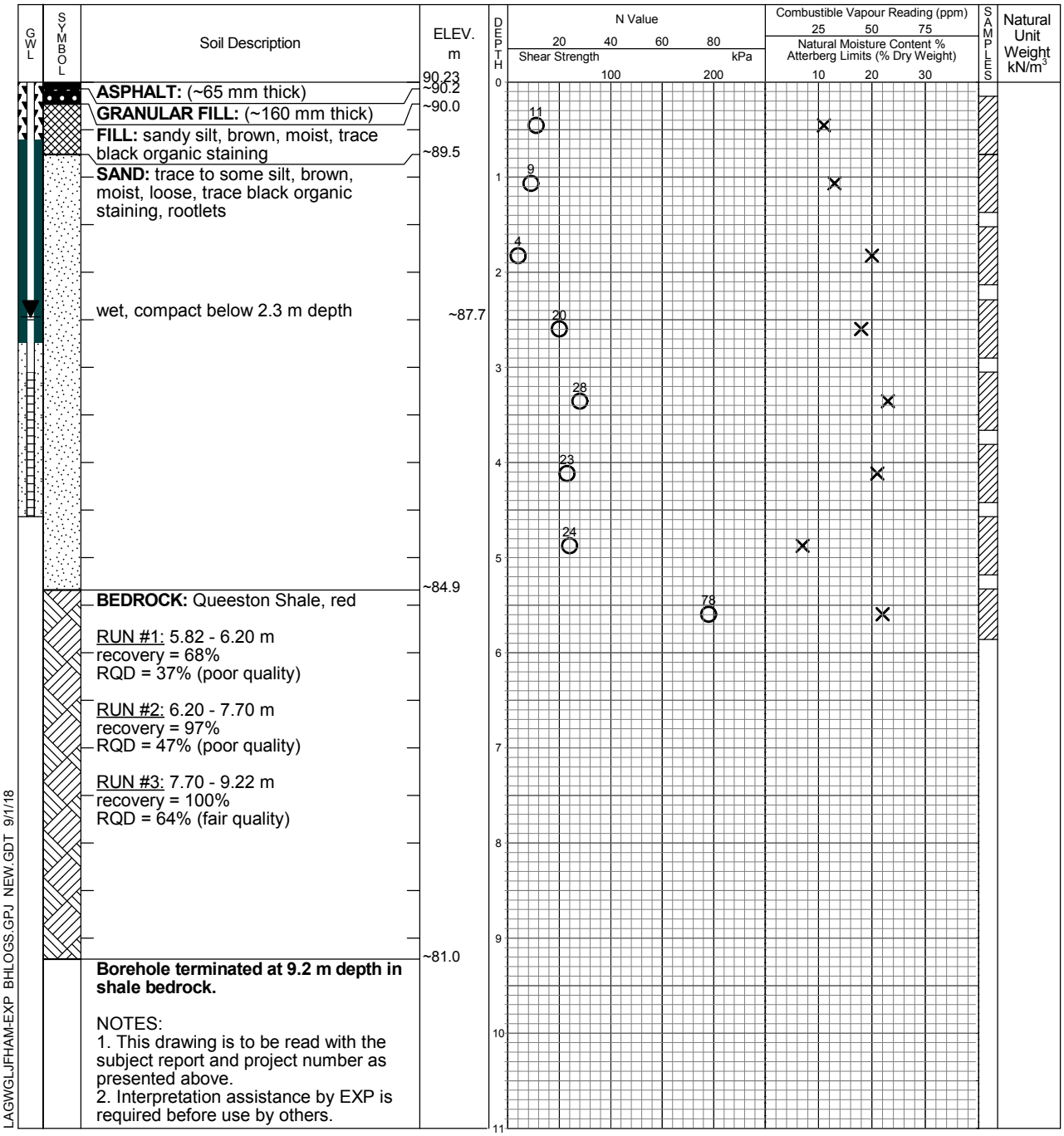
Date Drilled: November 28, 2017

Drill Type: CME 75 Truck Mount. Hollow Stem.

Datum: Geodetic

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer



LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/1/18

Time	Water Level (m)	Depth to Cave (m)
Dec. 7, shallow	2.5	
Dec. 7, deep	2.5	
Dec. 12, shallow	2.5	
Dec. 12, deep	2.5	

EXP Services Inc.
Hamilton, ON
Telephone: 905.573.4000
Facsimile: 905.573.9693

Log of Borehole BH-202

Project No. HAM-00801287-A0 Drawing No. 4

Project: Proposed Residential Development Sheet No. 1 of 1

Location: 2421 and 2431 New Street, Burlington, ON

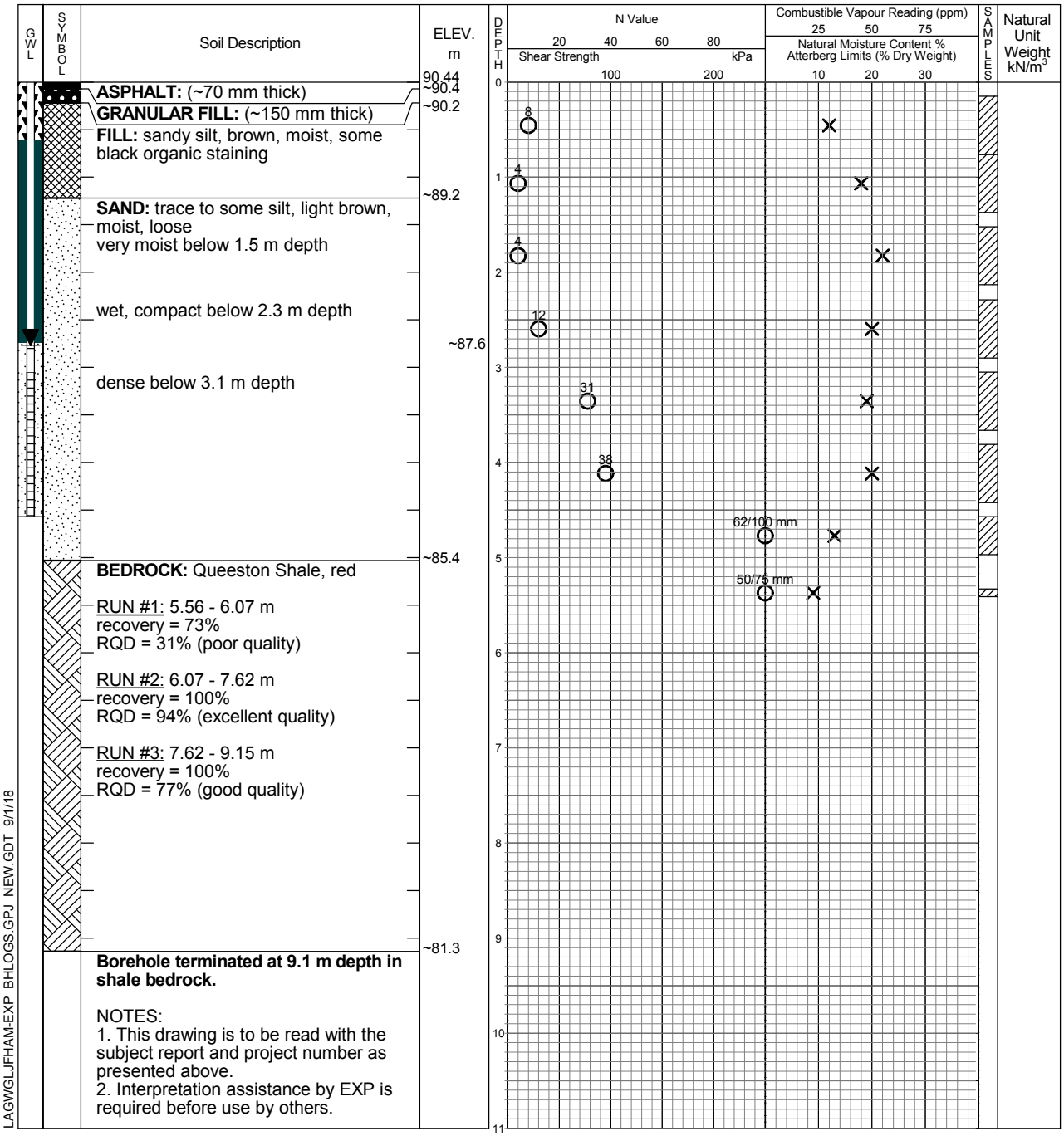
Date Drilled: November 27, 2017

Drill Type: CME 75 Truck Mount. Hollow Stem.

Datum: Geodetic

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test (S)

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer



ASPHALT: (~70 mm thick)

GRANULAR FILL: (~150 mm thick)

FILL: sandy silt, brown, moist, some black organic staining

SAND: trace to some silt, light brown, moist, loose
very moist below 1.5 m depth

wet, compact below 2.3 m depth

dense below 3.1 m depth

BEDROCK: Queeston Shale, red

RUN #1: 5.56 - 6.07 m
recovery = 73%
RQD = 31% (poor quality)

RUN #2: 6.07 - 7.62 m
recovery = 100%
RQD = 94% (excellent quality)

RUN #3: 7.62 - 9.15 m
recovery = 100%
RQD = 77% (good quality)

Borehole terminated at 9.1 m depth in shale bedrock.

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.

LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/1/18

EXP Services Inc.
Hamilton, ON
Telephone: 905.573.4000
Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
Dec. 6, shallow	2.6	
Dec. 6, deep	2.7	
Dec. 12, shallow	2.8	
Dec. 12, deep	2.7	

Log of Borehole BH-203

Project No. HAM-00801287-A0 Drawing No. 5

Project: Proposed Residential Development Sheet No. 1 of 1

Location: 2421 and 2431 New Street, Burlington, ON

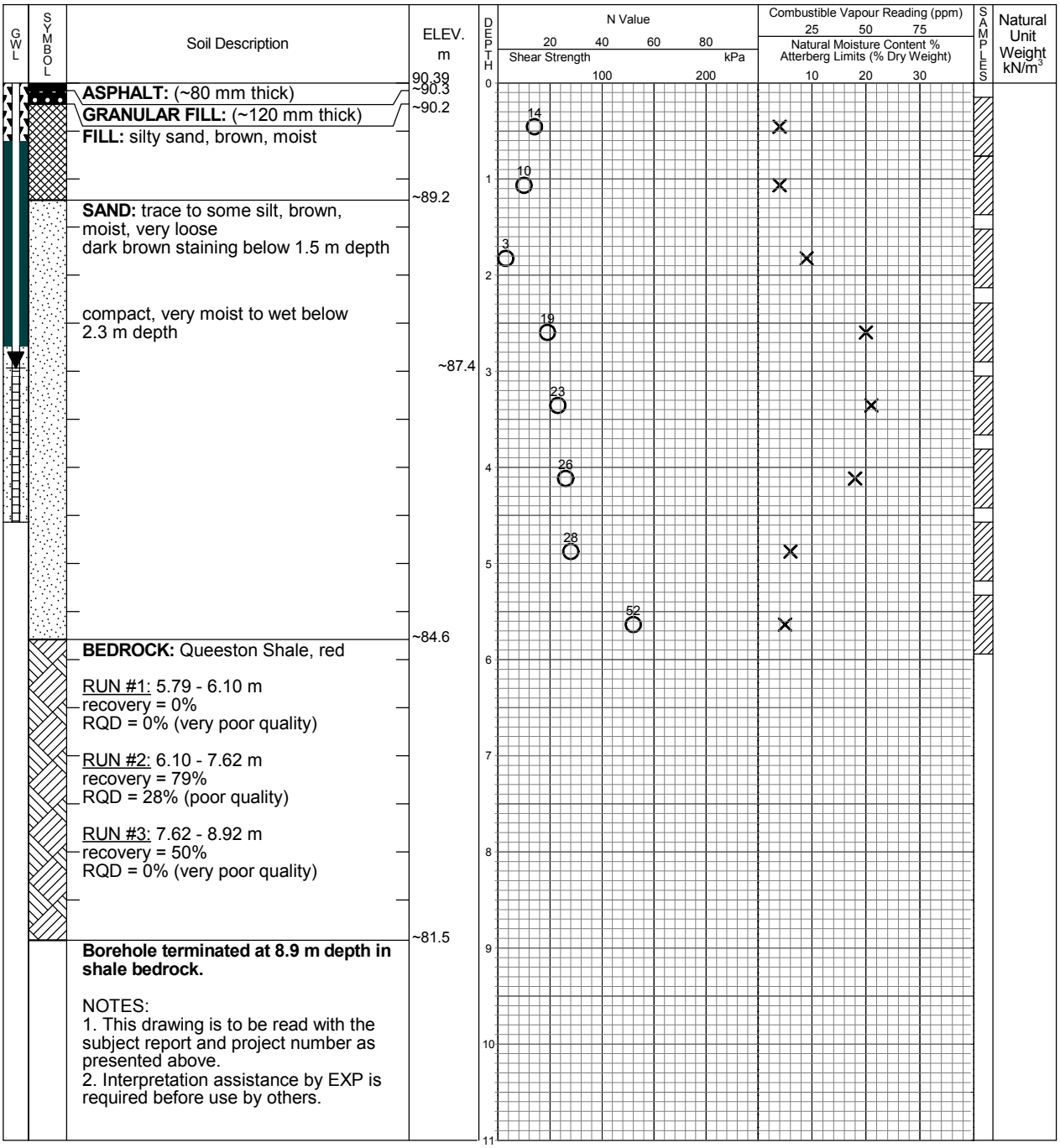
Date Drilled: November 27, 2017

Drill Type: CME 75 Truck Mount. Hollow Stem.

Datum: Geodetic

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer



LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/1/18

Time	Water Level (m)	Depth to Cave (m)
Dec. 6, shallow	2.9	
Dec. 6, deep	2.7	
Dec. 12, shallow	3.0	
Dec. 12, deep	2.7	

EXP Services Inc.
Hamilton, ON
Telephone: 905.573.4000
Facsimile: 905.573.9693

Log of Borehole BH-204

Project No. HAM-00801287-A0 Drawing No. 6

Project: Proposed Residential Development Sheet No. 1 of 1

Location: 2421 and 2431 New Street, Burlington, ON

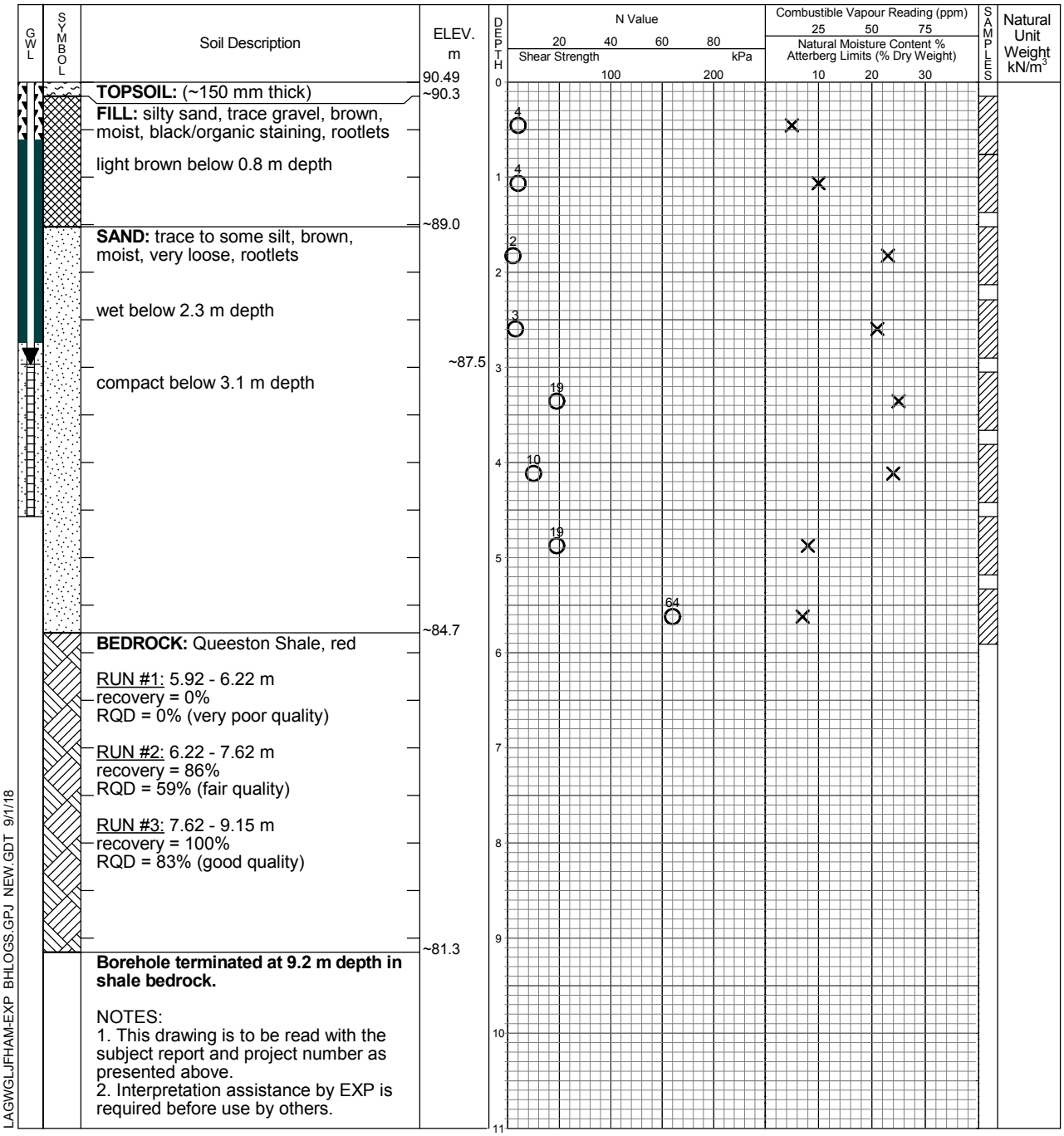
Date Drilled: November 29, 2017

Drill Type: CME 75 Truck Mount. Hollow Stem.

Datum: Geodetic

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer



LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/1/18

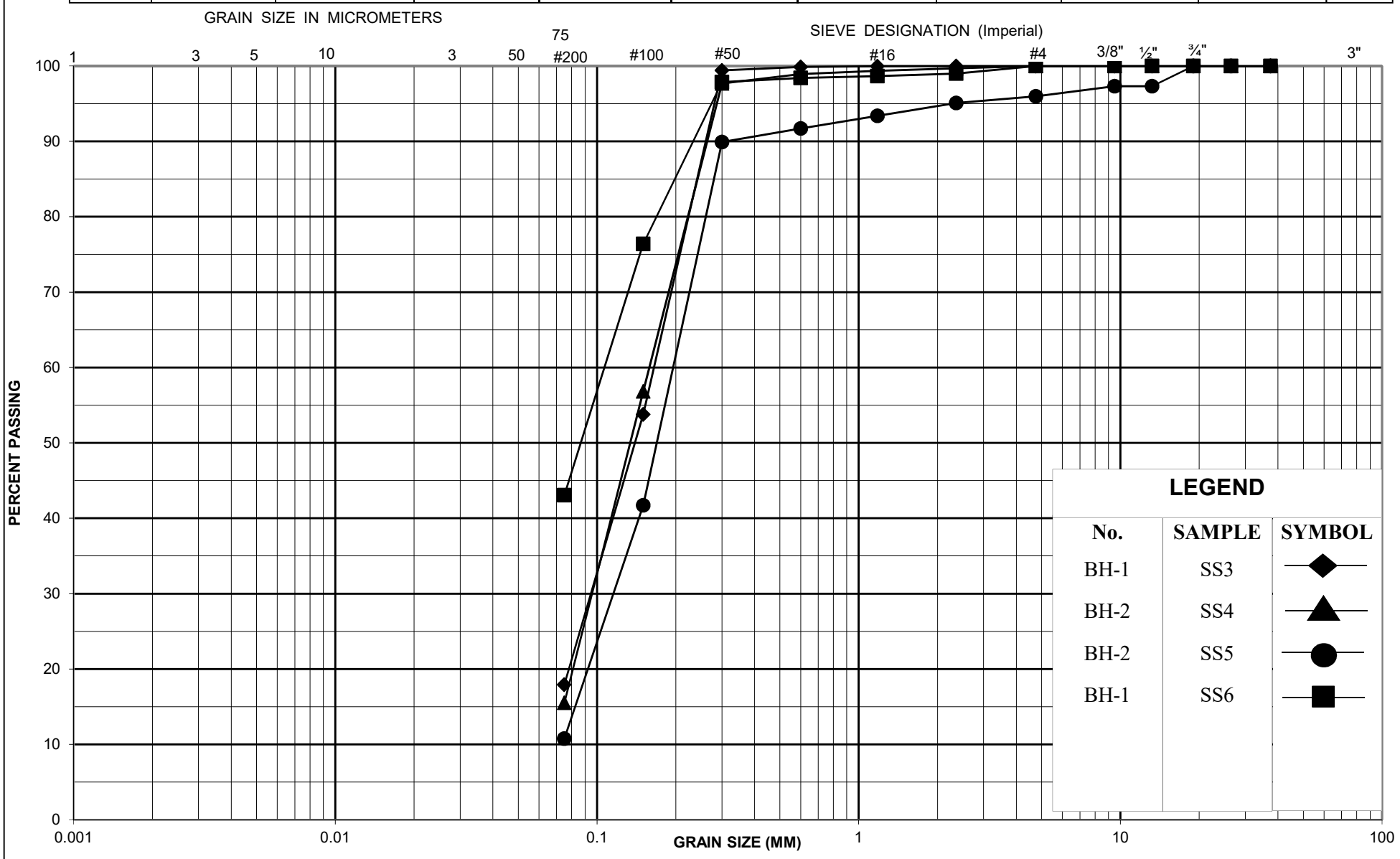
Time	Water Level (m)	Depth to Cave (m)
Dec. 6, shallow	3.0	
Dec. 6, deep	2.9	
Dec. 12, shallow	3.0	
Dec. 12, deep	2.9	

EXP Services Inc.
Hamilton, ON
Telephone: 905.573.4000
Facsimile: 905.573.9693

Appendix B –
Laboratory Test Results

ISSMFE SOIL CLASSIFICATION SYSTEM

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



exp.

GRAIN SIZE DISTRIBUTION

DRAWING NO.:	B1
PROJECT NO.:	HAM-00801287-A0
DATE:	JANUARY 2018