284 – 292 Plains Road East, Burlington, ON

Auburn Developments

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Geotechnical Investigation Report

Project Name:
Proposed Multi-Storey Residential Development
284 – 292 Plains Road East
Burlington, Ontario

Project Number:
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1. Introduction and Background

This report presents the results of the geotechnical investigation carried out at the site of the proposed development at 284 – 292 Plains Road East in Burlington, Ontario. The proposed development is expected to consist of a residential building of six storeys plus a penthouse level, and two levels of underground parking.

The investigation was authorized by Mr. Jamie Crich on behalf of Auburn Developments.

The purpose of this investigation was to determine the subsoil and groundwater conditions at the site by advancing four (4) boreholes and based on an assessment of the factual subsurface data, provide an engineering report containing general geotechnical recommendations pertinent to the proposed construction. Additional fieldwork and testing were carried out at the site by EXP as part of a hydrogeological study, the results of which are presented under a separate cover.

The comments and recommendations given in this report assume 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.

1.1 Site Description & Geological Setting

The site is located on the south side of Plains Road East, approximately 65 m west of Filmandale Road in Burlington, Ontario. The site is currently occupied by two (2) vacant residential/commercial buildings. A shed is also currently located at the southeast corner of 284 Plains Road East, and a garage at the southeast corner of 292 Plains Road East. The vacant portions of the site are surfaced with paved parking areas and driveways, as well as grassed and landscaped areas with some large trees. The site is bound by Plains Road East to the north and residential/commercial properties on the remaining sides.

Based on the Ministry of Natural Resources Map 2509, Quaternary Geology, Hamilton Area, the overburden at the site predominantly consists of glaciolacustrine sand and gravel deposits. The underlying bedrock consists of shale, limestone, dolostone, and siltstone from the Queenston Formation of the Upper Ordovician Period as indicated on the Ministry of Northern Development and Mines Map 2544, Bedrock Geology of Ontario, Southern Sheet.

2. Field Investigation

EXP advanced a total of four (4) boreholes at the site, numbered BH-01 to BH-04. The approximate borehole locations are shown on Drawing No. 1 in Appendix A. Boreholes BH-01, BH-03, and BH-04 were advanced to practical auger termination at depths ranging from 11.3 to 15.9 m below existing grade. Borehole BH-02 was advanced to refusal on the bedrock surface at a depth of approximately 20.7 m below existing grade; the bedrock was then cored to a depth of 24.9 m below grade.

The fieldwork for this investigation was carried out on August 17, 18, and 19, 2020. Drilling and sampling operations were completed by a combination of auger and split-spoon techniques using track mounted drilling equipment owned and operated by a specialist drilling subcontractor. Prior to the commencement of the drilling, the public and

private-owned underground services were located to minimize the risk of contacting any such services during the investigation.

Soil samples were obtained using a 51 mm (2 inch) outside diameter split-spoon sampler driven in conjunction with Standard Penetration Test procedure (ASTM D1586) at the depths noted graphically on the borehole logs. The retained soil samples and bedrock cores were logged in the field and then carefully packaged and transported to our Hamilton laboratory for detailed visual, textural and olfactory classification. The Standard Penetration Test (SPT) N values and pocket penetrometer measurements were recorded and used to provide an assessment of the compactness condition or consistency of the in-situ soils.

Groundwater levels within the boreholes were measured prior to backfilling. Four (4) 50 mm diameter monitoring wells (included one nested well) equipped with a flush-mount or monument cap were installed to allow for stabilized groundwater level measurements and subsequent hydrogeological testing. The remaining boreholes were backfilled upon completion of drilling in accordance with O.Reg. 903.

The boreholes were located in accessible areas on site by EXP field personnel. Ground surface elevations at the borehole locations were referenced to a site-specific geodetic benchmark, described as follows:

TBM:	Top of the catch basin lid in the shoulder of the eastbound lane of Plains Road East approximately 115 m west of Filmandale Road.
Geodetic Elevation:	99.36 m (based on the topographic survey prepared by David B. Searles Surveying Ltd., File No. 143-0-17, dated February 1, 2018)

3. Subsurface Conditions

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

The logs include textural descriptions of the subsoil and groundwater conditions 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 borehole logs form an integral part of and should be read in conjunction with this report.

3.1 Soil Stratigraphy

Surficial topsoil or the pavement structure was encountered at all borehole locations, with reworked soil or possible fill encountered immediately below in all boreholes, extending to depths of 0.8 to 1.8 m. The undisturbed native soil at the site consisted of sand overlying silty clay till. The silty clay till transitioned to till-shale complex, with the underlying shale bedrock contacted at a depth of 20.7 m below grade. The bedrock was confirmed by coring in Borehole BH-02. Details of the encountered materials are provided in the following subsections.

3.1.1 Pavement Structure

Boreholes BH-01, BH-03, and BH-04 were advanced through the existing paved parking areas and driveways, which consisted of approximately 50 to 75 mm of asphalt overlying approximately 140 to 200 mm of granular fill. The granular fill consisted of crushed limestone.

3.1.2 Topsoil

Surficial topsoil was encountered at Borehole BH-02, and was noted to have a thickness of approximately 175 mm. It is noted that topsoil thicknesses are expected to vary throughout the site.

3.1.3 Possible Reworked Soil or Fill

A layer of previously reworked soil or possible fill was encountered below the pavement structure or topsoil in all borehole locations, extending to depths of 0.8 to 1.8 m. The material consisted of silty sand with trace gravel, and with traces of rootlets and organic staining noted in Boreholes BH-01 and BH-02. The soil was reddish brown and in a moist state, with moisture contents ranging from 5 to 13 percent.

3.1.4 Sand

Native sand was encountered at all borehole locations below the fill, extending to depths ranging from 9.2 to 10.7 m below grade. The sand contained some silt and trace gravel, was brown, and was typically in a moist to wet state, with moisture contents of the stratum ranging from 5 to 21 percent. In Boreholes BH-01, BH-02, and BH-03, at depths of approximately 6.1 to 9.2 m, the sand became wet and was noted to exhibit flowing behaviour, with moisture contents as high as 21 percent. Wet, flowing sands inside the augers led to the cutting head seizing and the termination of Borehole BH-03 at a depth of 11.3 m. Based on SPT N values ranging from 1 to over 100 blows per 305 mm penetration, the sand is classified as very loose to very dense in compactness condition. In general, the sand is loose to very loose, becoming compact below 2.3 to 4.6 m, and dense to very dense below 3.1 to 7.6 m.

One (1) grain size analysis was conducted on a select sample of the stratum, with the complete laboratory results included in Appendix B and summarized in the table below.

Table 3-1: Summary of Grain Size Analysis

Sample	Clay (%)	Silt (%)	Sand (%)	Gravel (%)
BH-04 SS6	7	14	77	2

3.1.5 Silty Clay Till

Silt clay till was encountered at all borehole locations below the sand, extending to the borehole termination depths of 11.3 to 15.9 at Boreholes BH-01, BH-03, and BH-04 and to the till-shale complex at 20.0 m at Borehole BH-02. The silty clay till contained trace to some sand, with occasional gravel, was grey, and in a moist state, with moisture contents of the stratum ranging from 7 to 19 percent. SPT N values ranged from 21 to greater than 100 blows per 305 mm penetration. Based on estimated undrained shear strengths greater than 225 kPa as determined by pocket penetrometer measurements, the silty clay till is classified as hard in consistency.

One (1) grain size analysis was conducted on a select sample of the stratum as summarized in the table below. Atterberg limits testing was also conducted on the same sample, indicating the silty clay is of medium plasticity. The results of this testing are included in Appendix B.

Table 3-2: Summary of Grain Size Analysis

Sample	Clay (%)	Silt (%)	Sand (%)	Gravel (%)
BH-01 SS11	31	53	13	3

3.1.6 Till-Shale Complex

The till-shale complex is defined as the transition zone between the till and the underlying weathered shale bedrock and frequently contains shale inclusions. The till-shale complex was encountered in Borehole BH-02 at a depth of 20 m between the silty clay till and shale bedrock. The till-shale complex was red and grey and in a damp state with a moisture content of 9 percent. Based on an SPT N value of 89 blows per 305 mm penetration and an estimated undrained shear strength of greater than 225 kPa, the stratum is classified as hard in consistency.

3.1.7 Bedrock

Sampling spoon refusal on the assumed bedrock surface was encountered in Borehole BH-02 at a depth of approximately 20.7 m below grade. The bedrock was then confirmed by coring to an approximate depth of 24.9 m.

Based on review of the retrieved rock sample and Ontario Geological Survey Map 2544, Bedrock Geology of Ontario, Southern Sheet, the bedrock consists of red shale of the Queenston Formation. The upper portion of Queenston Shale bedrock is typically highly to moderately weathered to a depth of 600 mm to 1.5 m. Though not encountered within the rock sampled, hard limestone lenses are common within the shale and have been encountered by EXP at various sites in the Halton Region. These hard limestone layers can result in contractual problems for excavations and drilling. The bedrock surface depth and rock quality are summarized in the following table. Unconfined compressive strength testing was conducted on three (3) samples, with the results also summarized in the table below and complete results included in Appendix B. Photographs of the rock cores are included in Appendix C.

Table 3-3: Depth and Elevation of Bedrock Surface

Borehole No.	Depth of Bedrock Surface (m)	Elevation of Bedrock Surface (m)	Rock Quality Designation (RQD)	Unconfined Compressive Strength (MPa)
BH-02	20.7	77.8	60% - 96%	49 - 88

3.2 Groundwater Conditions

Groundwater conditions were monitored in the open boreholes during and upon completion of the investigation, but groundwater levels are not anticipated to have stabilized during the short term of the investigation. The soils were noted to become wet at depths of 4.6 to 7.6 m. 50 mm diameter groundwater monitoring wells were installed to facilitate long-term groundwater level measurements and hydrogeological testing at Boreholes BH-02, BH-03, and

BH-04, with nested shallow and deep monitoring wells at BH-03. Groundwater level measurements are summarized in the table below.

Table 3-4: Groundwater Level Measurements at Monitoring Well Locations

Borehole No.	Groundwater Depth / Elevation (m)			
	Upon Completion	August 21, 2020	August 24, 2020	September 3, 2020
BH/MW-02	6.1 / 93.06	5.01 / 93.46	5.01 / 93.46	5.05 / 93.42
BH/MW-03 (Shallow)	4.6 / 93.64	4.88 / 93.38	4.90 / 93.36	4.93 / 93.33
BH/MW-03 (Deep)	4.6 / 93.66	4.90 / 93.36	4.92 / 93.34	4.95 / 93.31
BH/MW-04	4.6 / 94.42	4.70 / 94.32	5.57 / 93.45	5.52 / 93.50

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. Discussion and Recommendations

Based on updated conceptual information provided by the client on September 7, 2021, the proposed development is expected to consist of a building with six storeys plus a penthouse level, and two levels of underground parking. The underground parking garage would occupy the majority of the site, with minor greenspaces planned along Plains Road East and in the southwest corner of the property. We offer the following comments and recommendations for the proposed construction.

4.1 Building Foundation Recommendations

Final details of the proposed building was not available, but it is understood that the building configuration is anticipated to consist of two levels of underground parking (corresponding to a founding level in the order of 6 m below grade). Several options for founding the proposed building are considered feasible, including conventional footings, a raft foundation, or deep foundations as detailed in the subsections below.

4.1.1 Conventional Foundations

At the anticipated founding level noted above, the soils typically consisted of dense to very dense sand over hard silty clay till, and as such conventional spread and strip footing foundations may be considered to support the proposed building.

Conventional spread or strip footings constructed on the undisturbed native sand at or below approximately 6 m from existing grade can be designed using a geotechnical resistance of 300 kPa at Serviceability Limit State (SLS) and 450 kPa at Ultimate Limit State (ULS), subject to review by EXP during construction. The footing base must be evaluated by geotechnical personnel from EXP prior to concrete placement to confirm the availability of the geotechnical resistance values provided and the foundation elevation.

Provided that the ground is not disturbed due to groundwater, precipitation, traffic, etc., and the aforementioned geotechnical resistance values are not exceeded, then total and differential settlements should be within the normally tolerated limits of 25 mm and 19 mm, respectively.

4.1.2 Raft Foundation

The anticipated underside of a raft foundation would be approximately Elev. 92 to 94 m. As such, the raft would likely be supported within the sand stratum, depending on site grading and the design's final founding elevations.

In the design of raft foundations, the traditional approach of the structural engineer has been to request a value for the coefficient of subgrade reaction for the supporting foundation soil. However, unlike strength and compressibility, the modulus of subgrade reaction is not a fundamental soil property, but rather a simplified method of representing the soil-structure interaction for a specific foundation configuration and loading condition.

For the design of a raft foundation established on the dense to very dense sand, the vertical modulus of subgrade reaction can be taken as 50 MPa/m. A geotechnical reaction at SLS of 300 kPa and a factored geotechnical resistance at ULS of 450 kPa can be used for foundations constructed on undisturbed native soil below approximately 6 m from existing grade, subject to review by EXP prior to placing foundation concrete. The total and differential settlements of well designed and constructed raft foundation are expected to be within the normally tolerated limits of 50 and 19 mm, respectively.

4.1.3 Deep Foundations

Alternatively, a deep foundation scheme consisting of caissons may be considered, particularly if additional storeys are planned. Caissons founded on the slightly weathered to sound shale bedrock below any highly weathered or loose rock (at least 1 m below the bedrock surface level) can be designed with an end-bearing resistance of 1,500 kPa ULS/SLS. The actual founding depth of the caissons are subject to verification by EXP during construction.

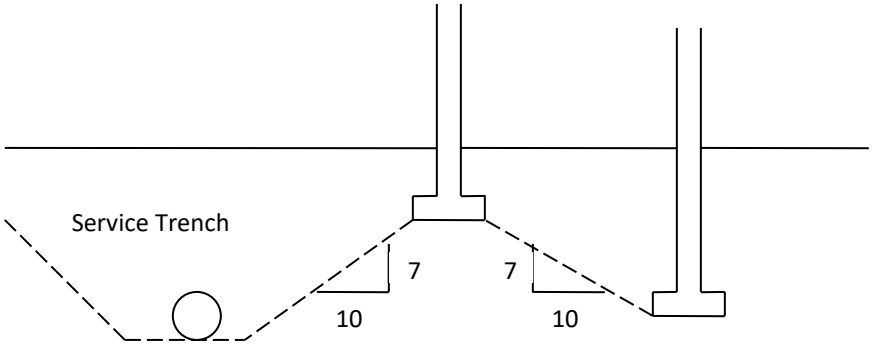
The caissons will require temporary liners for installation to prevent the soil from caving and thus minimize the possible formation of voids below the floor slab, and to help control any water/soil seepage into the caissons. The liners must be tightly sealed into the silty clay till or weathered bedrock to prevent the infiltration of soil and groundwater into the hole. Once the caissons have been drilled to the final founding elevation and the rock conditions confirmed by EXP, it is recommended that the base be cleaned by placing about 0.3 to 0.5 m of concrete into the final base and mixing it with the loose material present at the base. All concrete and loose soil should then be removed prior to placing the reinforcing cage and the structural concrete.

As noted above, wet flowing sand conditions were encountered above the silty clay till. Prior to withdrawal of the liner, the contractor should be prepared to place concrete by 'tremmie' method if the liner cannot form a seal to prevent groundwater infiltration. An experienced contractor should be employed to ensure the above procedures are followed and no 'necking' or voids in the concrete occurs in the caisson shaft during the concrete pour.

Concrete being placed into the caissons should have a slump of about 150 mm in order to minimize the risk of necking in the shaft. Once the method of construction is established the concrete mix must be reviewed by this office.

4.2 General Foundation Recommendations

Conventional foundations in soil at different elevations should be located such that higher footings are set below a line drawn up at 10:7, horizontal to vertical from the near edge of the lower footing. This requirement is not applicable for foundations in sound bedrock. This concept should also be applied to excavations for new foundations in relation to existing foundations or underground services.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

All foundations and grade beams for caissons exposed to freezing conditions must be provided with a minimum of 1.2 m of earth cover or equivalent insulation for frost protection, depending on the final grade requirements.

The recommended geotechnical resistances have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, it should be appreciated that modifications to bearing levels may be required if unforeseen subsoil conditions are revealed after the excavation is exposed to full view or if final design decisions differ from those assumed in this report. For this reason, this office should be retained to review final foundation drawings and to provide field inspections during the construction stage.

4.3 Excavations

Excavations for two underground levels are expected to extend to depths in the order of 6 m below existing grade. Excavations will be carried out through the existing fill and into the native sand. Excavation of the soil at this site is generally not expected to pose any difficulty and can be carried out with heavy hydraulic excavators. Groundwater infiltration will be a concern as outlined in Section 4.6 and the EXP hydrogeological study (provided under separate cover).

All excavations must be completed in accordance with the most recent regulations of the Ontario Occupation Health and Safety Act (OHSA). The encountered fill and loose to compact sand may generally be classified as Type 3 Soil above the groundwater level, though the very loose sand or areas below the groundwater level where the sand exhibits flowing behaviour would be classified as Type 4 soil. 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 OHSA requires that unsupported 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.

It is important to note that soils encountered in the construction excavations may vary significantly across the site. Our preliminary soil classifications are based solely on the materials encountered in the boreholes advanced at the site. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, we recommend that EXP be contacted immediately to evaluate the conditions encountered.

4.4 Temporary Shoring

Given the anticipated excavation depths and extents as well as the presence of adjacent infrastructure and dwellings, temporary shoring is expected to be required. The shoring method chosen by the structural engineer and/or contractor will depend on the settlement tolerance of the surrounding structures and infrastructure. 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. Given the presence of permeable soils and excavations below the groundwater level, a method of shoring that limits groundwater infiltration may be preferable.

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).

Lateral support for the shoring system can be obtained by using rakers extending into the excavation and founded on undisturbed native soil. Alternatively, if the raft foundation option is chosen, the middle portion of the raft may be constructed and used to support the rakers.

Or, tieback anchors may be used and should be installed in the soil behind the excavation to a sufficient distance to allow mobilizing the desired lateral load resistance. The recommended configuration of the no-load zone is shown in Figure 26.16 of the Canadian Foundation Engineering Manual (CFEM), 4th Edition. The minimum horizontal spacing between anchors should be 1.2 m to ensure that group effects between adjacent ground anchors are minimized and that anchor intersection (due to drilling deviations) is avoided. Group effects will reduce the load-carrying capacity of individual ground anchors. The anchors may be installed in the compact to dense sand using an equivalent bond stress of 100 kPa. Higher bond values may be considered if the soil anchors are constructed using re-groutable techniques. The recommended design parameters should be confirmed by load testing a number of anchors to 200 percent design load in accordance with the current edition of the CFEM. As a minimum for this site, at least two 200 percent anchor load tests for each soil stratum should be carried out to verify the capacity of the anchors. The design for the production anchors should then be modified based on the test results, where necessary. All remaining anchors must be installed using similar procedures and proof tested to 1.5 times the design load.

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.

4.5 Lateral Earth Pressure

The lateral earth pressure acting on the foundation walls may be calculated using the following equation:

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

where
- p = lateral earth pressure intensity at depth h (kPa)
- K = earth pressure coefficient (assume 0.40)
- γ = unit weight of retained soil, assume 21.0 kN/m^3 for granular backfill
- h = depth to point of interest (m)
- q = surcharge load acting adjacent to the wall at the ground surface (kPa)

If the building is constructed as a tank without drainage, lateral hydrostatic and uplift pressures below the slab will need to be accounted for using the expression below.

$$p = K[(\gamma h_w) + (\gamma' (h - h_w))] + (\gamma_w (h - h_w)) + K q$$

where
- p = lateral earth pressure and hydrostatic pressure in kPa acting at depth h (kN/m^2)
- K = active earth pressure coefficient, assume 0.30
- γ_w = unit weight of water, 9.8 kN/m^3
- γ = unit weight of soil surrounding the structure, assume 21.0 kN/m^3
- γ' = effective unit weight of retained soil, assume 11.2 kN/m^3
- h = depth to point of interest (m)
- q = equivalent value of surcharge on the ground surface (kPa)

4.6 Groundwater Control

All of the boreholes were recorded as wet at depths of 4.6 to 6.1 m below grade upon completion of drilling and prior to coring. Groundwater levels in the monitoring wells on site ranged from 4.7 to 5.6 m below grade. Excavations extending below these depths will require dewatering 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.

4.7 Building Floor Slab-on-Grade and Permanent Drainage

The drainage requirements and slab configuration will vary depending on whether the building is constructed to be watertight, the chosen foundation configuration, as well as servicing requirements.

The underground parking garage floor slab-on-grade can be supported on the native sand, or if a raft foundation is used, the floor slab-on-grade can be constructed on 200 mm of 19 mm clear stone overlying the raft to allow for installation of underfloor drainage and servicing (if required).

It is recommended that the exposed subgrade be examined by a geotechnical engineer prior to constructing the floor slab-on-grade. Any loose or disturbed material 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 subgrade 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. The installation of a perimeter drainage and underfloor drainage system at 3 m intervals is recommended given the groundwater level at site. Alternatively, if a drainage system is not used and the building is instead designed to be watertight, the floor slab/raft may be waterproofed and designed for hydrostatic pressure. 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.

4.8 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 100 percent Standard Proctor Maximum Dry Density (SPMDD) in lifts not exceeding 200 mm.

To minimize potential problem, any 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 silty sand fill or native sand. In general, the excavated material may be reused for backfill subject to the removal of any topsoil or other obviously unsuitable material. However, moisture content adjustment of re-used soils might be required, particularly for any saturated sand soils excavated at depth.

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.

4.9 Earthquake Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading are presented in the subsections below.

4.9.1 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. Foundations are anticipated to be founded on the dense sand overlying hard silty clay till. The reported N

values for these strata below the expected founding level ranged from 19 to over 100 blows per 305 mm penetration. Undrained shear strengths of the silty clay till from pocket penetrometer were greater than 225 kPa.

There have been no shear wave velocity measurements carried out at this site and therefore, N values and EXP's knowledge of the soil conditions in the area have been used to determine the site classification.

4.9.2 Corrected N-Values N60

The Average Standard Penetration Resistance shown in Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 refers to N60 which is defined as "Average Standard Penetration Resistance for the top 30 m, corrected to a rod energy efficiency of 60% of the theoretical maximum". It should be noted that the drillers in the area do not have their rod energy efficiencies measured and therefore, computed N60 values are not available for this site.

4.9.3 Site Classification

Based on the above assumptions and interpretations and the known soil conditions, the Site Class for this site is "D" as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. It should be noted that an improved site classification may be achievable if shear wave velocity testing is carried out. EXP can be contacted to provide this service if required.

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 in regard to 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.

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Appendix A

Drawings & Borehole Logs



EXP Services Inc.
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LEGEND:

APPROXIMATE BOREHOLE LOCATION

TITLE AND LOCATION:

BOREHOLE LOCATION PLAN
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL DEVELOPMENT
284 - 292 PLAINS ROAD EAST, BURLINGTON, ON

JOB NO.: BRM-00258353-A0	DRAWN BY: ML
SCALE: NTS	CHECKED BY: JG
DATE: September 2020	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-01

Project No. BRM-00258353-A0 Drawing No. 3

Project: Proposed Multi-Storey Residential Development Sheet No. 1 of 1

Location: 284 - 292 Plains Road East, Burlington, ON

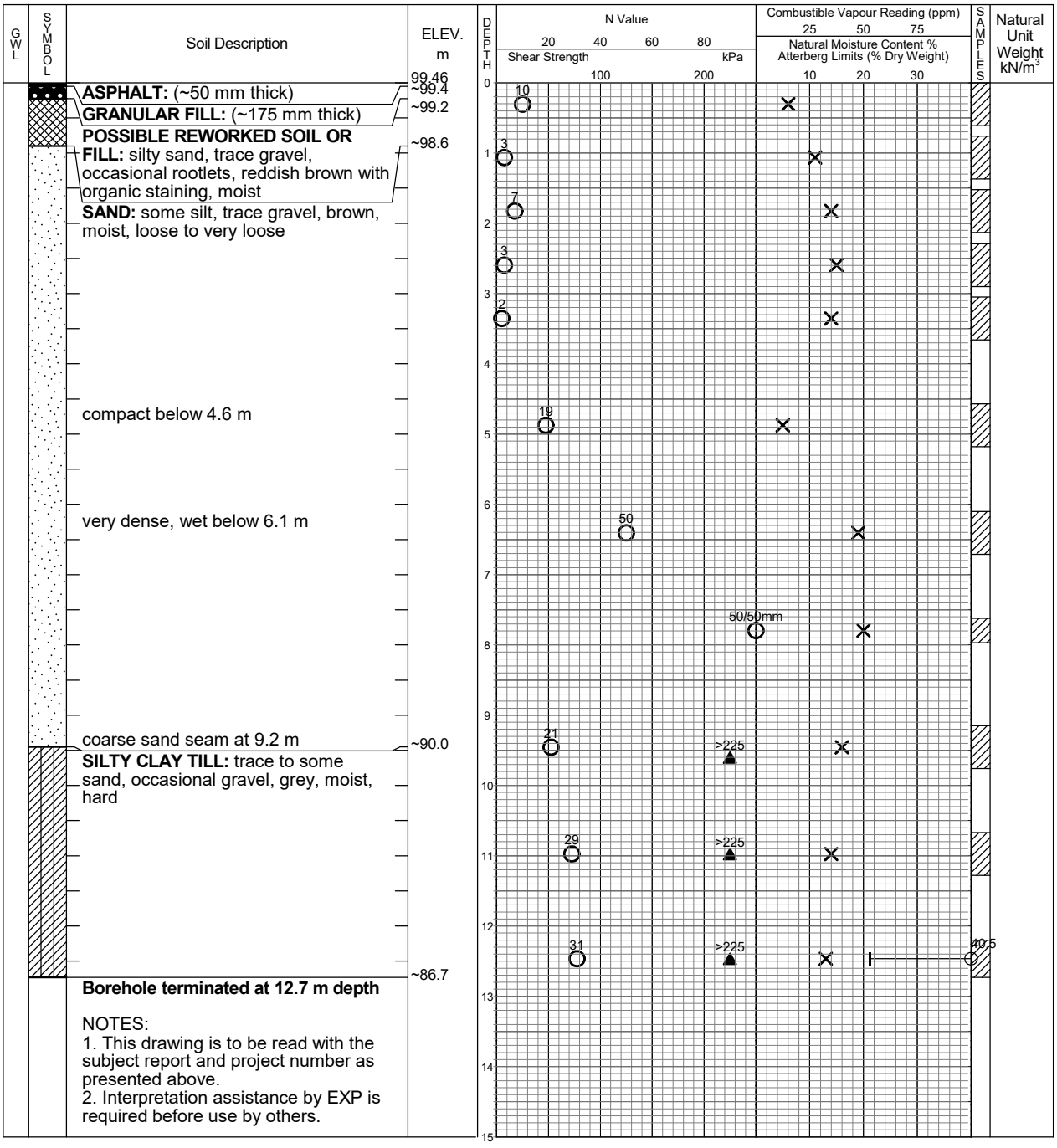
Date Drilled: August 19, 2020

Drill Type: D-50 Track Mount. Solid 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



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 9/14/22

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

Time	Water Level (m)	Depth to Cave (m)
on completion	6.1	no cave

Log of Borehole BH-02

Project No. BRM-00258353-A0 Drawing No. 4

Project: Proposed Multi-Storey Residential Development Sheet No. 1 of 2

Location: 284 - 292 Plains Road East, Burlington, ON

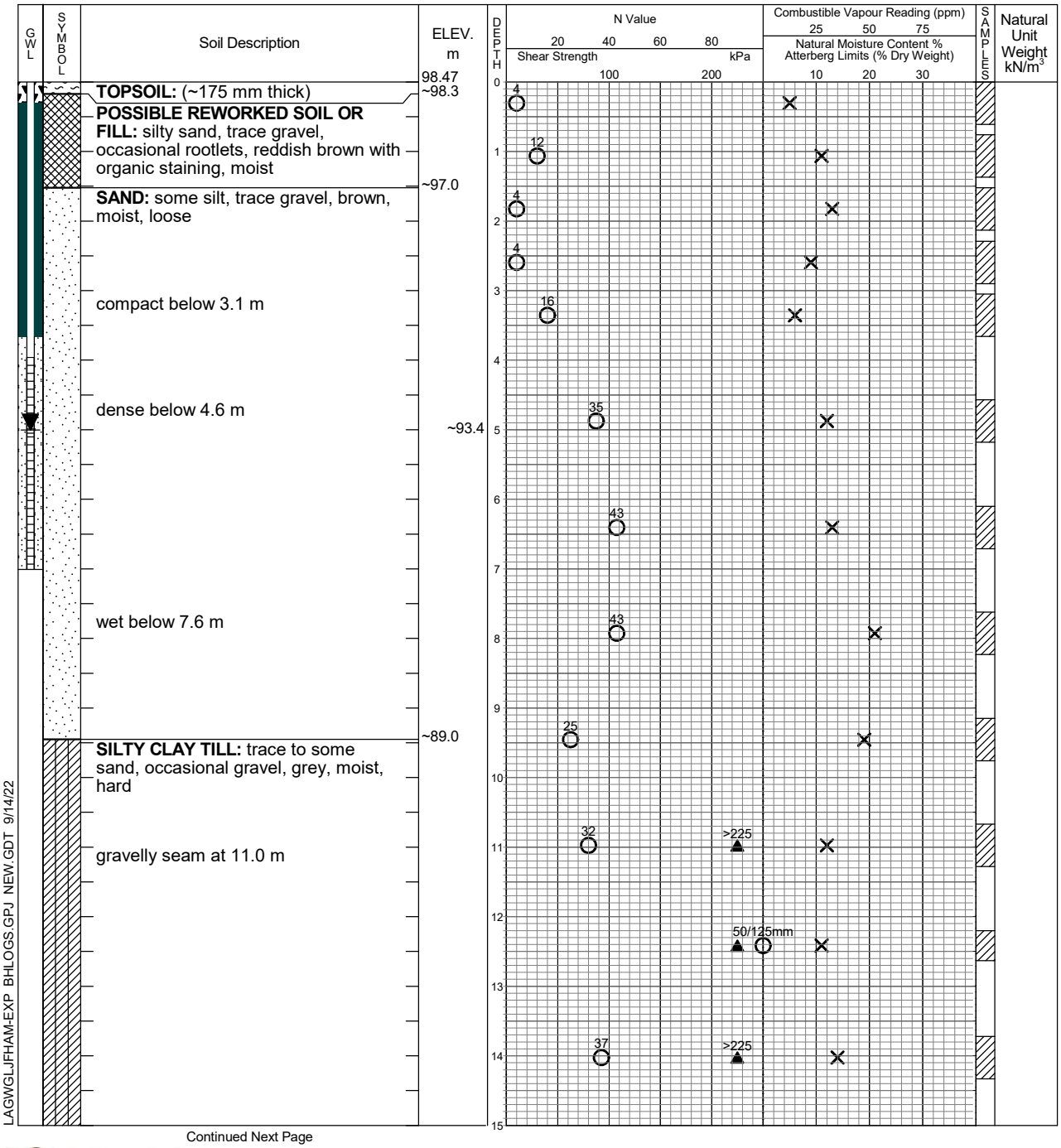
Date Drilled: August 17, 2020

Drill Type: D-50 Track Mount. Solid 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 9/14/22

Continued Next Page

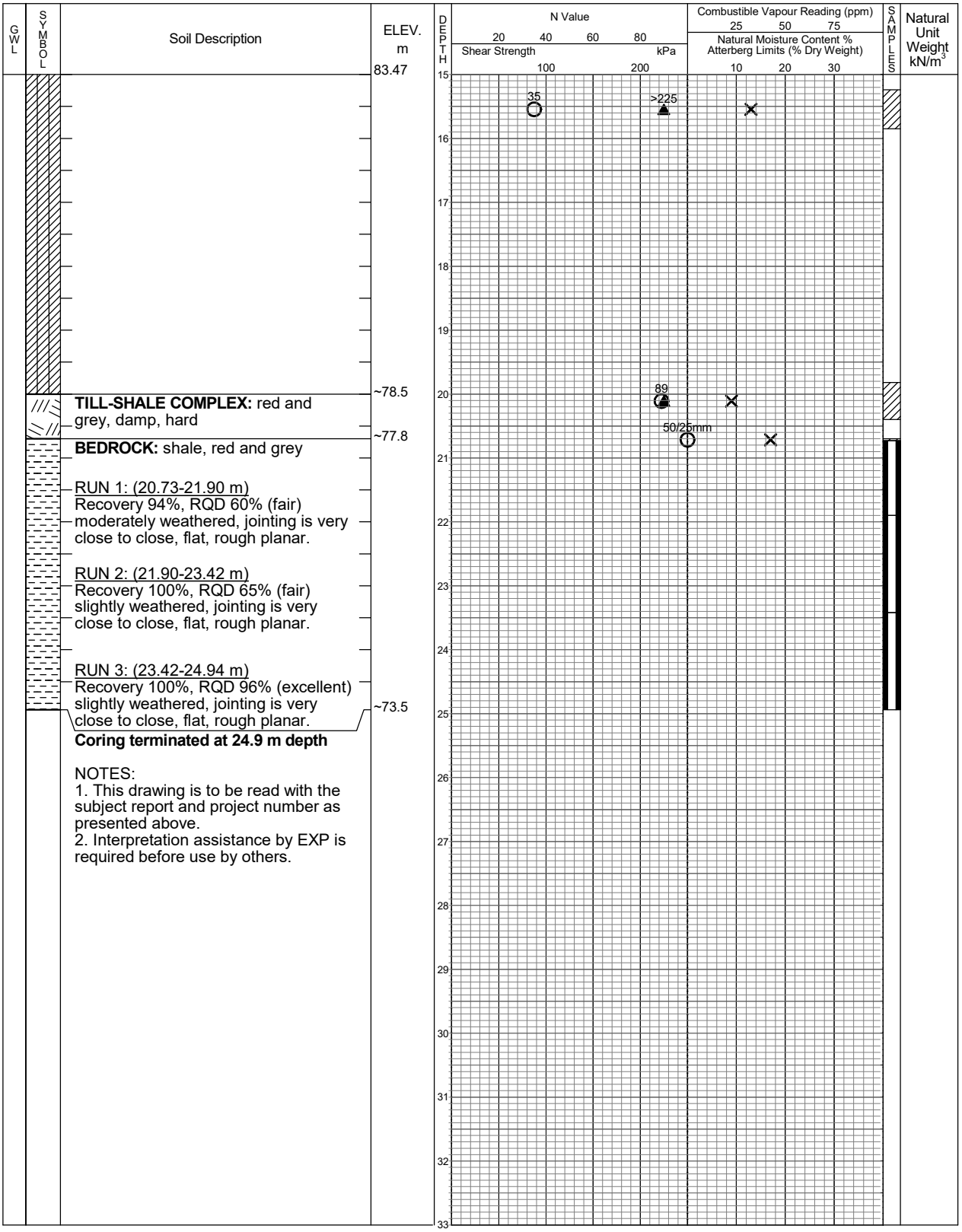
EXP Services Inc.
Hamilton, Ontario
Telephone: 905.573.4000
Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	4.6	no cave
August 21, 2020	5.01	
August 24, 2020	5.01	
September 3, 2020	5.05	

Log of Borehole BH-02

Project No. BRM-00258353-A0 Drawing No. 4

Project: Proposed Multi-Storey Residential Development Sheet No. 2 of 2



LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/14/22

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

Time	Water Level (m)	Depth to Cave (m)
on completion	4.6	no cave
August 21, 2020	5.01	
August 24, 2020	5.01	
September 3, 2020	5.05	

Log of Borehole BH-03

Project No. BRM-00258353-A0 Drawing No. 5

Project: Proposed Multi-Storey Residential Development Sheet No. 1 of 1

Location: 284 - 292 Plains Road East, Burlington, ON

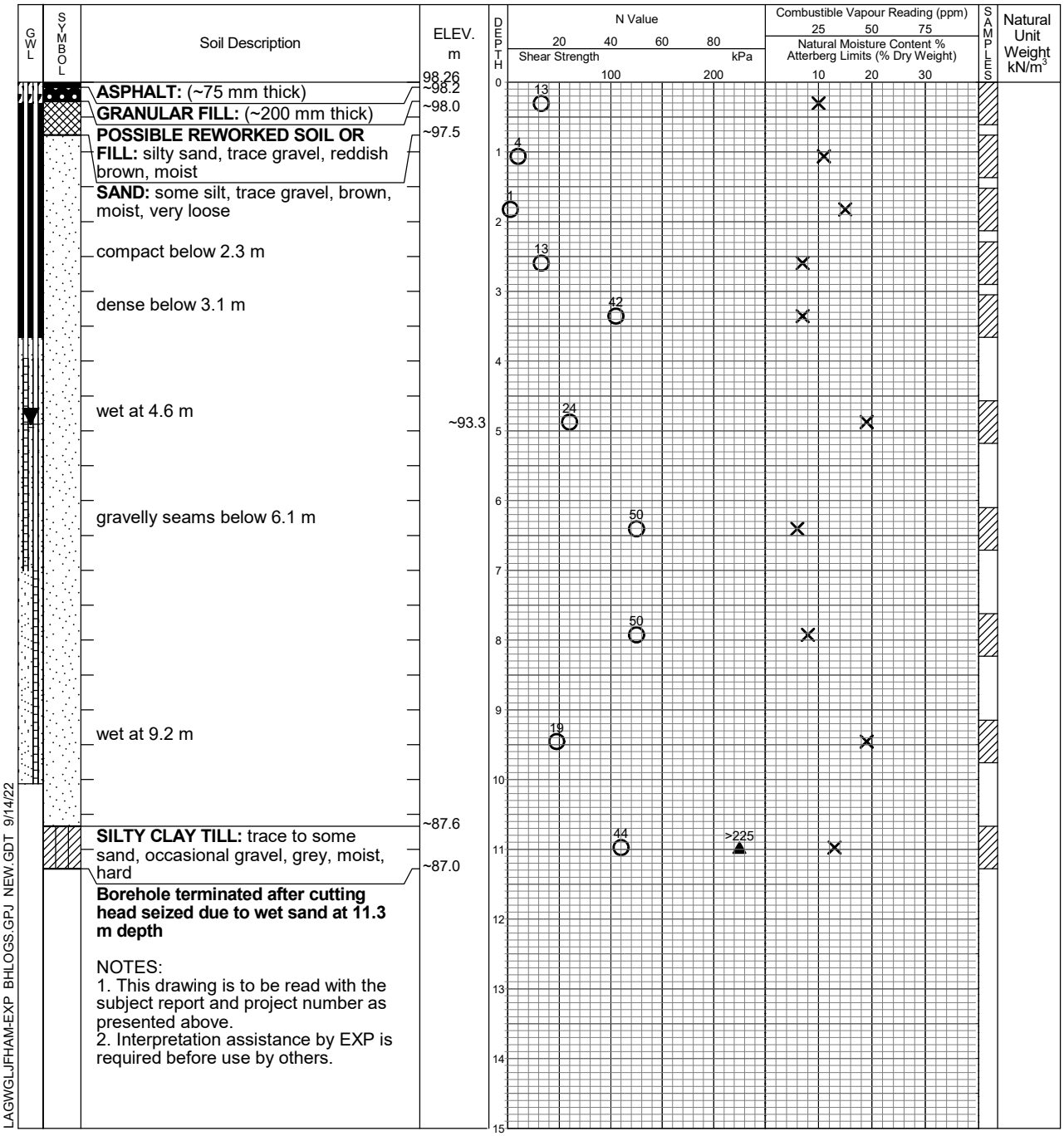
Date Drilled: August 18, 2020

Drill Type: D-50 Track Mount. Solid 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



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.

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

Time	Water Level (m)	Depth to Cave (m)
on completion	4.6	no cave
Aug 21/20 (shallow)	4.88	
Aug 21/20 (deep)	4.90	
Aug24/20 (shallow)	4.90	
Aug 24/20 (deep)	4.92	

Log of Borehole BH-04

Project No. BRM-00258353-A0 Drawing No. 6

Project: Proposed Multi-Storey Residential Development Sheet No. 1 of 2

Location: 284 - 292 Plains Road East, Burlington, ON

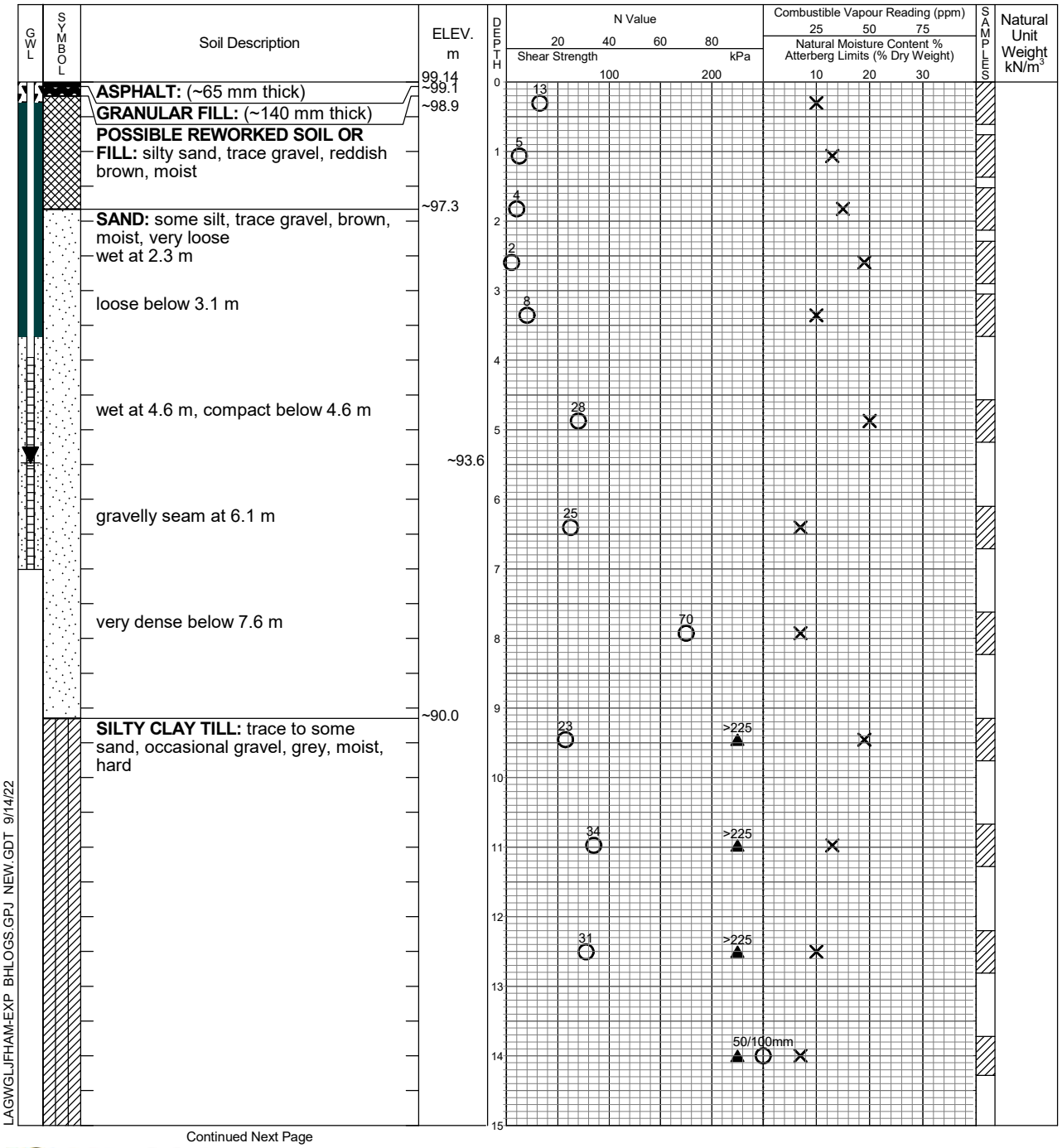
Date Drilled: August 19, 2020

Drill Type: D-50 Track Mount. Solid 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



LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/14/22

Continued Next Page

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

Time	Water Level (m)	Depth to Cave (m)
on completion	4.6	no cave
August 21, 2020	4.70	
August 24, 2020	5.57	
September 3, 2020	5.52	

Log of Borehole BH-04

Project No. BRM-00258353-A0 Drawing No. 6

Project: Proposed Multi-Storey Residential Development Sheet No. 2 of 2

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	N Value / Shear Strength kPa	Combustible Vapour Reading (ppm) / Natural Moisture Content % / Atterberg Limits (% Dry Weight)	SAMPLES	Natural Unit Weight kN/m³
			84.14	15	20 40 60 80; 100 200; 25 50 75; 10 20 30			
				16	33 (○) >225 (▲)	×		
			~83.3	17				
		Borehole terminated at 15.9 m depth		18				
		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.		19				
				20				
				21				
				22				
				23				
				24				
				25				
				26				
				27				
				28				
				29				
				30				
				31				
				32				
				33				

LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 9/14/22

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

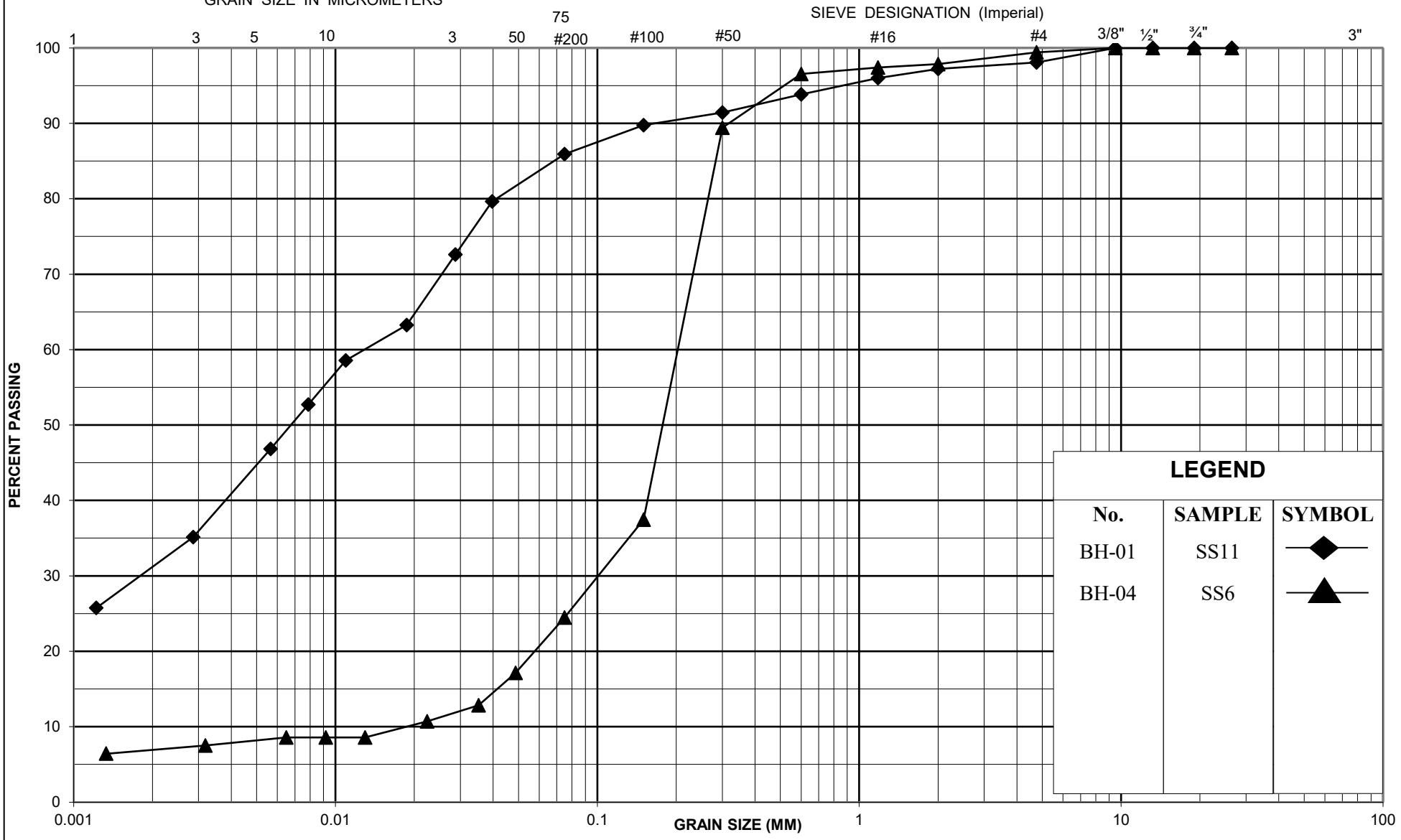
Time	Water Level (m)	Depth to Cave (m)
on completion	4.6	no cave
August 21, 2020	4.70	
August 24, 2020	5.57	
September 3, 2020	5.52	

Appendix B

Laboratory Test Results

ISSMFE SOIL CLASSIFICATION SYSTEM

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



GRAIN SIZE DISTRIBUTION

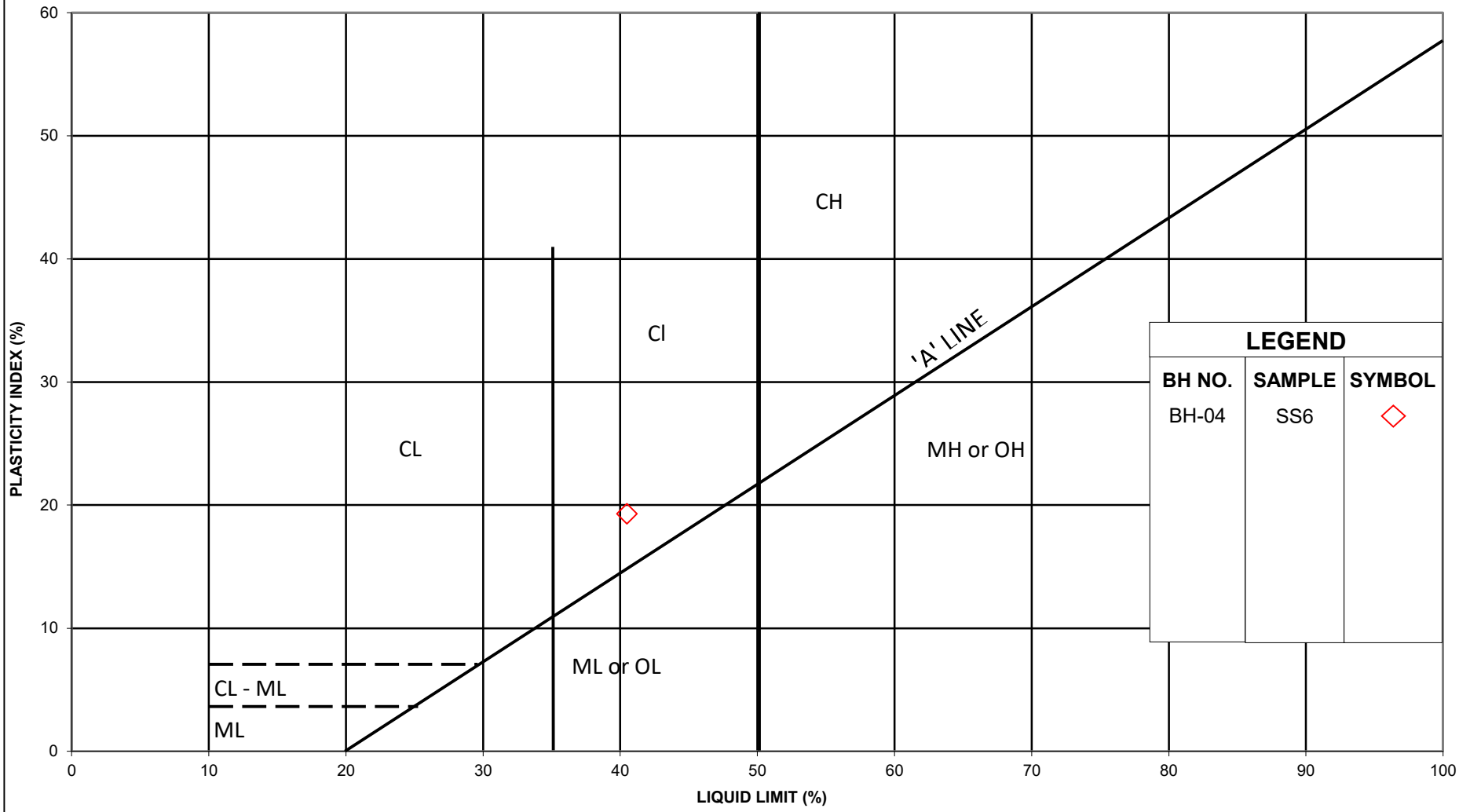
DRAWING NO.: B1

PROJECT NO.: BRM-00258353-A0

DATE: September 2020

PROPOSED MULTI-STOREY RESIDENTIAL DEVELOPMENT

284 - 292 Plains Road East, Burlington, ON



exp.

PLASTICITY CHART

SILTY CLAY (CI)

DRAWING NO.:	B2
PROJECT NO.:	BRM-00258353-A0
DATE:	September 2020

Table No.1- Compressive Strength Results

Sample No.	Location	Diameter, D (mm)	Area (mm^2)	Length of Core (mm)	Unit Weight kg/m^3	Correction Factor	Break Load (kN)	Compressive Strength (MPa)
1	BH-02 (21.3 – 21.4 m)	47	1734	52	3600	0.89	96.13	49.3
2	BH-02 (22.1 – 22.2 m)	47	1734	57	2617	0.93	164.70	88.3
3	BH-02 (23.9 – 24.0 m)	47	1734	91	2663	0.99	133.36	76.1

Test procedure in general accordance with A23.2-14C: Method for Compressive Strength Testing of Drilled Cores

Appendix C

Core Photographs



Borehole BH-02: Run 1 to Run 3 (20.73 to 24.94 m)