555 Canal Bank Street, Welland

555 Canal Bank Developments GP Inc.

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Proposed Dain City West Development
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Executive Summary

This report presents the results of the geotechnical investigation in support of the proposed development at 555 Canal Bank Street in Welland, Ontario. The project will consist of a residential subdivision which would include single family dwellings and townhouses, as well as the associated roadways and site servicing. A mixed-use block with mid-rise structures (maximum 6 storeys) and an elementary school is also planned.

The drilling and fieldwork were performed in preliminary and supplemental stages on February 11, 13, and 14, 2019; on January 27, 2020; and on July 13, 14, and 16, 2020. The scope of the investigation consisted of twenty-seven (27) boreholes to depths ranging from 6.6 to 12.2 m below grade. Monitoring wells were installed in ten (10) borehole locations. All drilling was completed with CME 75 truck and CME 55 and track mounted drilling rigs. Upon completion of drilling, all boreholes were surveyed by a representative of EXP using Trimble R10 equipment.

Thirteen (13) representative soil samples were subjected to grain size analyses and Atterberg limits testing, with four (4) samples subjected to unidimensional consolidation testing. Moisture contents were determined for all recovered samples.

In general, the subsurface conditions consisted of topsoil or pre-existing pavement structures overlying fill typically extending to 1.5 m below grade or less, with native silty clay below. The native silty clay was typically found to be very stiff to hard near the surface, becoming weaker with depth. The upper crust is suitable for founding low to mid-rise structures. However, as the site grades are planned to be raised, this will induce long-term consolidation settlements of the underlying soils and pre-loading will be required prior to building construction. The settlement analysis and pre-loading recommendations are provided in this report.

A hydrogeological assessment has also been carried out by EXP in coordination with this investigation, the results of which have been presented under a separate cover.

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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 Dain City West development located at 555 Canal Bank Street in Welland, Ontario. This report encompasses and supersedes the findings and recommendations presented in EXP's preliminary report for the site entitled *Preliminary Geotechnical Investigation*, with Project No. HAM-00801631-A0, and dated July 6, 2020. A hydrogeological study was also carried out by EXP at the subject site and the results of which are presented under separate cover.

Details of the proposed development (site grading, building configurations, site servicing, etc.) were in the preliminary stages at the time of this report. However, based on these preliminary drawings and details provided to our office, a residential subdivision consisting of single-family dwellings and townhouses, as well as the associated roadways and site servicing is planned. A mixed-use block with mid-rise structures (maximum of 6 storeys anticipated) and an elementary school is also planned. A stormwater management pond will also be constructed at the southern end of the site.

The geotechnical investigation was authorized by Mr. Jeffrey Swartz on behalf of 555 Canal Bank Developments GP Inc.

The purpose of this investigation was to determine the general subsoil and groundwater conditions at the site by advancing twenty-seven (27) boreholes and based on an assessment of the factual borehole data, provide an engineering report containing general geotechnical recommendations pertinent to the proposed construction at this site.

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 subject property is located south of the east-west aligned CNR line, north of St. Clair Drive, and is bounded to the west by Canal Bank Street and the north-south aligned CNR line to the east.

The overburden at the site consists of glaciolacustrine deeper water clay and silt according to the Ontario Geological Survey Map 2496, Quaternary Geology, Niagara-Welland, Southern Ontario. It should be noted that fill materials (man-made deposits) are present in the footprint of the Former John Deere facility. The bedrock at the site consists of limestone, dolostone, shale, sandstone, gypsum of the Salina Formation from the Upper Silurian Period according to the Ontario Geological Survey Map 2544, Bedrock Geology of Ontario, Southern Sheet, but was not encountered within the explored depths.

2. Field Investigation

The fieldwork for this investigation was carried out in three stages: on February 11, 13 and 14, 2019; on January 27, 2020; and on July 13, 14, and 16, 2020. The borehole drilling and sampling operations were completed by a combination of auger and split-barrel techniques using track mounted drilling equipment owned and operated by specialist drilling subcontractors. Prior to the commencement of drilling operations in each stage, private and publicowned underground services were cleared to minimize the risk of contacting any such services during the drilling operations.

In total, twenty-seven (27) boreholes, numbered BH-01 to BH-27, were advanced at the site, with the borehole logs presented in Appendix A. The approximate borehole locations are shown on Drawing No. 1 in Appendix A.

Soil samples were obtained using a 51 mm (2 inch) outside diameter split-barrel sampler in accordance with the Standard Penetration Test (ASTM D1586) at depths noted on the attached borehole logs in Appendix A. The Standard Penetration Tests (SPT) N values were recorded and used to provide an assessment of the consistency or compactness condition of the in-situ soils. The shear strength of the cohesive soils was evaluated using pocket penetrometer measurements and in-situ shear vane testing. 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.

Groundwater levels within the boreholes were measured prior to backfilling. Monitoring wells were installed in ten (10) borings to facilitate long-term groundwater level measurements and hydrogeological testing. The remaining boreholes were backfilled upon completion of drilling in accordance with O.Reg. 903.

The geodetic ground surface elevations at the borehole locations were surveyed by EXP using Trimble R10 equipment following the investigation.

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

In general, the boreholes encountered topsoil or existing pavement structures for the former John Deere plant at the surface. Fill was encountered in nineteen of the boreholes, extending as deep as 4.6 m below grade. Native silty clay was encountered below the topsoil / pavement structure and fill and extended to the termination in all boreholes. Details of the encountered soils are provided in the subsections below.

3.1.1 Topsoil

A surficial layer of topsoil was encountered at Boreholes BH-01 to BH-03, BH-06, BH-08 to BH-12, BH-19, and BH-25 to BH-27. The thickness of the topsoil at the borehole locations ranged from approximately 50 mm to 150 mm.

It should be noted that the topsoil measurements were carried out at the borehole locations only and were found to be variable. A more detailed analysis (involving test pits) is recommended to accurately quantify the amount of topsoil to be removed for construction purposes. Consequently, topsoil quantities should not be established from the information provided at the widely spaced borehole locations only.

3.1.2 Fill / Reworked Native Soil

Fill (or reworked native soil) was encountered below the surficial topsoil or pavement structure at nineteen of the twenty-seven boreholes locations and extended to depths ranging from approximately 0.3 m to 1.5 m below existing grade. Isolated deeper depths of fill deposits were found in Boreholes BH-19 and BH-20, to depths of 4.6 and 2.3 m below grade, respectively. The fill generally consisted of brown or grey, moist silty clay with traces of sand, gravel, rootlets, and wood fragments. Black organic staining was noted at Boreholes BH-04, BH-08, BH-16, BH-19, BH-20, BH-21, BH-23, and BH-27. Moisture contents of the material ranged from 7 to 39 percent.

3.1.3 Silty Clay

Native silty clay was encountered below the surficial topsoil / pavement structure in Boreholes BH-09, BH-11 to BH-14, BH-18, BH-22, and BH-24, and below the fill at all the remaining borehole locations. The silty clay extended to the borehole termination depths of 6.6 m to 12.8 m below grade at all the borehole locations. The silty clay was brown, generally became greyish brown with depth, and was in a moist to wet state with moisture contents ranging from 8 to 48 percent. SPT N values ranged from 0 to 24 blows per 305 mm penetration. Based on undrained shear strengths ranging from 15 to greater than 225 kPa as determined by pocket penetrometer measurements and in-situ shear vane testing, the silty clay is classified as soft to hard in consistency. It should be noted that the stratum generally became weaker with depth.

Thirteen (13) grain size analyses were conducted on select samples of the stratum. Atterberg limits testing was also conducted on the same samples, indicating the silty clay is of low to high plasticity, varying with clay content. The results of this testing are included in Appendix B and summarized in the table below.

Table 3-1: Summary of Laboratory Testing

Sample Information		Grain Size Analysis Results				Atterberg Limits Results		
Borehole	**Depth (m)**	**Clay (%)**	**Silt (%)**	**Sand (%)**	**Gravel (%)**	**Plastic Limit**	**Liquid Limit**	**Plasticity Index**
BH-01	2.3 – 2.8	37	62	1	0	17	35	18
BH-09	6.1 – 6.6	40	59	1	0	16	35	19
BH-10	7.6 – 8.1	31	50	19	0	15	29	14
BH-11	3.1 – 3.7	71	28	1	0	23	60	37
BH-11	9.2 – 9.8	61	38	1	0	20	47	27
BH-12	6.1 – 6.7	64	34	2	0	20	49	29
BH-12	12.2 – 12.8	39	59	1	1	18	36	18
BH-26	2.3 – 2.9	60	35	5	0	21	59	38
BH-26	4.6 – 5.2	52	46	2	0	20	53	33
BH-26	6.1 – 6.7	43	49	8	0	21	51	30
BH-27	0.8 – 1.4	44	50	6	0	20	58	38
BH-27	1.5 – 2.1	51	45	4	0	22	57	35
BH-27	4.6 – 5.2	59	38	3	0	21	55	34

Unidimensional consolidation tests were also carried out on four undisturbed samples. The results of this testing are attached in Appendix C.

3.2 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of the drilling operations. Monitoring wells (50 mm diameter) were installed in Boreholes BH-01 to BH-04, BH-06, BH-08 to BH-10, BH-15, and BH-27 to facilitate long-term groundwater level measurements and hydrogeological testing. Groundwater level measurements are summarized in the table below.

Table 3-2: Groundwater Level Measurements

Borehole No.	**Groundwater Depth / Elevation (m)**								
	Upon Completion	**Feb. 19, 2019**	**Mar. 4, 2019**	**Apr. 15, 2019**	**Jun. 3, 2019**	**Aug. 8, 2019**	**Oct. 11, 2019**	**Jan. 23, 2020**	**Jul. 27, 2020**
BH-01	dry	4.4/172.5	6.7/170.2	5.1/171.8	4.9/172.0	4.8/172.1	4.6/172.3	4.4/172.5	3.7/173.2
BH-02	dry	dry	dry	dry	dry	Inaccessible / damaged	Inaccessible / damaged	Inaccessible / damaged	Inaccessible / damaged
BH-03	dry	dry	dry	dry	dry	dry	Inaccessible / damaged	Inaccessible / damaged	Inaccessible / damaged
BH-04	5.3/171.9	4.6/172.6	2.4/174.8	1.2/176.0	1.1/176.1	1.0/176.2	1.4/175.9	1.3/176.0	0.9/176.3
BH-06	dry	2.6/174.6	2.1/175.2	1.4/175.8	1.0/176.3	1.0/176.3	1.4/175.9	1.2/176.0	Inaccessible / damaged
BH-08	dry	5.4/171.6	4.4/172.6	2.5/174.5	1.8/175.2	1.5/175.5	1.5/175.5	1.2/175.7	1.3/175.6
BH-09	dry	dry	dry	dry	8.9/168.3	8.9/168.3	dry	8.8/168.4	dry
BH-10	8.2/168.4	1.8/174.8	1.7/174.9	1.4/175.2	1.3/175.3	1.4/175.2	1.2/175.4	1.2/175.4	1.2/175.4
BH-15	dry	-	-	-	-	-	-	-	0.7/176.5
BH-27	dry	-	-	-	-	-	-	-	2.1/174.7

The groundwater levels are not considered to have stabilized upon completion given the fine-grained nature of the soil. 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

Final details of the proposed development were not available at the time of this report. However, based on preliminary designs, a residential subdivision consisting of single-family dwellings and townhouses, as well as the associated roadways and site servicing is planned. A mixed-use block with mid-rise structures (maximum 6 storeys anticipated) and an elementary school is also planned. A stormwater management pond will also be constructed at the southern end of the site. We offer the following comments and recommendations for the proposed construction.

4.1 Site Grading

Based on the site topography, the proposed draft plan, and preliminary drawing entitled *EG to Deere Pregrade* dated March 18, 2020 and provided to our office on May 29, 2020, regrading (cut and fill operations) will be carried out at the site. The existing site grades will be increased in the order of 1 to 2 m on the west portion of the site, increasing to as much as about 5 m on the east portion, and approximately 5 m along the proposed berm.

The grade raises will induce long-term consolidation settlements of the underlying soils. These expected settlements and preventative measures (i.e. preloading) are discussed in detail in Section 4.2 below. The following procedures are recommended for the construction of fill sections for building and pavement areas at the site, where required:

- All existing structures, pavements, fill, disturbed soils and organic/deleterious materials should be removed from the proposed building and pavement areas. Fill materials in pavement areas may remain in place, subject to being proof-rolled and replaced as directed by a geotechnical representative, but pavements constructed over fill may require more frequent maintenance and experience a reduced service life.

- The exposed subgrade surface should be proof-rolled with a heavy roller or partially loaded truck and reviewed by a geotechnical representative. Any soft areas detected during the proof-rolling process should be sub-excavated and replaced with approved material compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD).

- Low areas can then be brought up to final subgrade level with approved on-site or imported material placed in lifts not exceeding 200 mm. Fill placed in building areas must be placed on native soil and compacted to 100 percent of SPMDD, i.e. engineered fill as detailed in Drawing No. 30 in Appendix A. Fill placed in pavement areas should be compacted to at least 95 percent SPMDD, with the upper 600 mm compacted to at least 98 percent SPMDD. The moisture content of the fill should be at or near its optimum moisture content to ensure the specified densities can be achieved with reasonable compactive effort.

- Re-use of the on-site fill should be at the discretion of the geotechnical consultant during construction. Given the elevated moisture content at some locations/depths, some adjustment of moisture content may be required to facilitate compaction of re-used materials. Re-used materials must also be free from organics and deleterious materials.

- All imported borrow fill material from local sources should be free from organic material and foreign objects (trees, roots, debris, etc.) and should be approved by EXP prior to transport to the site. In addition, the chemical quality of the borrowed fill material should be assessed by EXP in accordance with the current applicable MOE regulations and guidelines.

- All excavation, backfilling and compaction operations should be monitored on a full-time basis by EXP's geotechnical staff to approve materials and to ensure the specified degrees of compaction have been obtained.

4.2 Settlement Analysis and Preloading Requirements

As noted above, four unidimensional consolidation tests were conducted on the native soils. Based on the results of this testing and the expected grade raises across the site, long-term consolidation settlements of the underlying soils were calculated. These calculations were completed for the western portion of the property, which will see grades raised by as much as approximately 1.0 to 2.0 m, and for the eastern portion of the property, which will see grade raises in the order of 2.5 to 5.0 m. For the purposes of analysis, a maximum grade raise of 2.0 m was used for calculating settlements on the western half of the property, and a maximum of 5.0 m was used for the eastern half of the property. The total long-term consolidation settlements expected to occur from this loading are presented below.

Table 4-1: Long Term Consolidation Settlements Due to Engineered Fill Placement

Location	Location of Consolidation Testing	Maximum Depth of Engineered Fill (m)	Total Consolidation Settlement Expected (mm)
Western Half of Property	BH-11	2.0	89
Eastern Half of Property	BH-12	5.0	197

As shown in the settlement analysis presented in Appendix C, these settlements would be ongoing for many years before stabilizing. As the calculated settlements are considerably higher than the typical allowable limits, pre-loading should be carried out prior to building construction. Pre-loading would consist of placing a greater height of fill than the site grading calls for and leaving this additional soil in place for a specified period of time in order to achieve the expected consolidation settlements more quickly. The pre-load soil should be placed in lifts and compacted to at least 90 percent SPMDD. The time required to achieve the minimum required settlement will vary depending on the height of soil used in preloading as illustrated in the table below.

Table 4-2: Preloading Depths and Time to Achieve Minimum Settlement

Location	Height of Preloading (m)	Total Calculated Settlement (mm)	Assumed Maximum Grade Raise (m)	Total Max. Height of Fill Placed (m)	Minimum Required Settlement* (mm)	Time to Achieve Minimum Settlement (Days)
West Half of Property	1	89	2	3	64	230
	2	89	2	4	64	120
	3	89	2	5	64	80
East Half of Property	4	197	5	9	172	950
	5	197	5	10	172	760
	6	197	5	11	172	680

*Minimum required settlement = (Total Settlement) – (25 mm)

As shown above, the required minimum settlements can be achieved on the west side of the development with approximately 80 to 230 days of preloading, depending on the height of pre-loading used. The allowable settlements for the eastern half of the settlement will take approximately 2 years of preloading to achieve (680 to 760 days with 5 to 6 m of preloading).

A settlement monitoring program should be employed to confirm the above analysis and determine when the required settlements have been achieved. It is noted that the above preloading times were estimated based on testing and analysis of limited soil samples. The preliminary results of the settlement monitoring program should be correlated to this analysis and used to refine the estimates given for total settlements and preloading times in the table above. EXP can be contacted to provide this service.

4.3 Site Servicing

4.3.1 Open-Cut Excavations

The depth of the sewer and water lines were not finalized at the time of this investigation, but based on discussions with the client excavations are expected to extend to depths in the order of 2.0 m below the proposed finished grade for watermains (minimum 1.7 m cover), and 2.5 to 6.0 m for storm and sanitary sewers. Based on the results of the investigation, excavations for sewer and watermain installation are expected to encounter the fill materials as well as the native silty clay. Excavations can be carried out in open-cut with conventional excavation equipment.

All excavations must be completed in accordance with the most recent regulations of the Ontario Occupation Health and Safety Act (OHSA). In general, the fill and silty clay may be classified as Type 3 Soil above the groundwater level. The very stiff to hard silty clay may be classified as Type 2 Soil. If encountered, the soft, wet clay at depth may be classified as Type 4 soil, but can be verified by a geotechnical representative during construction. 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 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.

4.3.2 Shoring

Trench boxes may be used to reduce the lateral extent of the excavation in dewatered excavations. Once the final service elevations are known, the excavations, particularly those extending below approximately 4.0 m from existing grade should evaluated for basal stability. Note that trench boxes are not permitted to be used in excavations extending greater than 6 m deep or in Type 4 soil. The lateral earth pressure acting on supported walls may be computed using the following equation, assuming a rectangular pressure distribution and dewatering will be carried out:

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

where p = Lateral earth pressure (kPa)
K = Coefficient of earth pressure
γ = Unit weight of supported soil (assume 19.0 kN/m^3)
h = Depth to point of interest (m)
q = surcharge load acting adjacent to the wall at the ground surface (kPa)

In general, an earth pressure coefficient, K, of 0.45 may be used where movements must be minimized and 0.25 where minor movements can be tolerated.

4.3.3 Groundwater Control

Stabilized groundwater level measurements obtained from the monitoring wells were dry to as high as 0.9 m below existing grade as shown in Table 3-2 above.

Given the fine-grained nature of the encountered soils, the excavations for watermains and sewer service lines not extending below approximately 3 m from existing grade are typically not expected to encounter significant groundwater. Any perched water within the fill or water bearing seams should generally be possible to remove using conventional construction sump pumping techniques. Deeper excavations extending below the saturated soils may require more advanced dewatering techniques. The design of dewatering systems is the responsibility of the contractor.

Reference to the EXP hydrogeological study for the subject site should be made for additional dewatering comments and recommendations. 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.

4.3.4 Pipe Bedding

Based on the borehole data and the anticipated service invert levels, it is expected that the material encountered at the pipe invert will typically consist of native silty clay, with some areas encountering silty clay fill. It is recommended that any encountered fill materials be removed down to competent native soils or approved competent subgrade prior to placement of the pipe bedding material.

The native soils on site are expected to provide adequate support for the pipes. Pipe bedding and cover requirements should be in accordance with the Ontario Provincial Standard Drawing (OPSD) relevant to encountered ground conditions and the type of pipe installed. The pipe bedding should consist of a Clear Stone (OPSS 1004) or Granular A material (OPSS 1010). The cover material should consist of Granular A or a compacted sand material with no sizes greater than 25 mm.

The granular bedding and cover material should be placed in lifts not exceeding 150 mm and compacted to 95 percent of the SPMDD or in accordance with local requirements. In order to minimize the risk of damage to the pipe, the first lift above the pipe should be increased to at least 300 mm in thickness. Particular care should be taken when compacting beneath the pipe haunches. The degree of compaction achieved in the field should be checked by in-situ nuclear density tests.

Any areas where wet or loose subgrade conditions are encountered, the bedding thickness can be increased or a 300 mm thick layer of 19 mm clear stone wrapped in geotextile (e.g. Terrafix 360R or equivalent) may be used.

4.3.5 Thrust Blocks

It is recommended that all thrust blocks for watermains be poured neat against the native soils without the use of forms in order to achieve the maximum restraint with minimum deflection.

Horizontal restraint for the thrust block is provided by the passive earth pressure developed in the soil behind the block and the friction along the base. The passive earth pressure may be estimated using the following equation and geotechnical parameters:

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

where
p = Passive earth pressure (kPa)
K = Coefficient of passive earth pressure
γ = unit weight of soil (assume 18.0 kN/m^3)
h = Depth below ground surface at which the pressure is to be computed (m)

In general, an earth pressure coefficient, K, of 2.2 may be used. An assumed coefficient of sliding friction of 0.30 may be used on the silty clay soils.

4.3.6 Trench Backfilling Operations

The on-site soils are generally considered suitable for re-use as trench backfill. Some water content adjustment of re-used materials may be required for efficient compaction. Any soil containing organics, excessive moisture, or otherwise deleterious material should not be used for backfill. Any shortfall of suitable on-site excavated material can be made-up with imported and approved fill or granular material.

All backfilling and compaction operations should be closely examined by a representative of this office to ensure uniform compaction to specification requirements, especially in the vicinity of manholes and in all areas that are not readily accessible to compaction equipment, etc. All backfill should be placed in maximum 200 mm loose lifts and uniformly compacted to at least 95 percent SPMDD. For trenches below pavement areas, the upper 600 mm of backfill below subgrade level should be compacted to at least 98 percent SPMDD. If following Region of Niagara requirements for trench backfill, Granular A (OPSS 1010) compacted to 100 percent SPMDD is required to be used for the entire trench below roadways, driveways, and sidewalks.

To mitigate the potential for differential settlement, the fill around catch basins and manholes should consist of free draining granular material (e.g. Granular A). To minimize potential problems, backfilling operations should follow closely after excavation and pipe installation so only a minimal length of trench is exposed. This will minimize wetting of the subgrade and backfill materials. Should construction extend to the winter season, frozen material must not be used as backfill.

4.4 Pavement Design and Construction

The recommended pavement structure is provided in the table below and is based on an estimate of the subgrade soil properties determined from visual examination and textural classification of the soil samples. The pavement structure should be considered for preliminary design purposes only and are based on typical Niagara Region and City of Welland standards. If required, a more refined pavement structure design can be performed based on specific traffic data, client input, and design life requirements.

Table 4-3: Recommended Minimum Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Roadways	Residential Driveways	Commercial Driveways	Truck Routes
Asphaltic Concrete (OPSS 1150)	Min. 92.0% MRD[1]	40 mm HL3 75 mm HL8	50 mm HL3F	40 mm HL3F 50 mm HL8 MDBC	40 mm HL3F 50 mm HL8 MDBC
Granular A Base, Crusher Run Limestone (OPSS 1010)	100% SPMDD2	450 mm	200 mm	300 mm	375 mm

[1]Denotes Maximum Relative Density, [2]Denotes Standard Proctor Maximum Dry Density, ASTM-D698

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 subbase 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 sloped to provide effective surface drainage toward catch basins or drainage areas. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas.

Additional comments on the construction of proposed roadways are as follows:

- The subgrade should be prepared in accordance with Section 4.1, Site Grading above. The final subgrade surface should be properly shaped and crowned.

- 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 the road sections, given the fine-grained nature of the subgrade, we recommend subdrains be installed on both sides of the roadways at least 300 mm below the granular subbase. This will mitigate the potential for water to collect in the granular materials, which could result in pre-mature pavement failure during the spring thaw.

- To minimize problems of differential movement between the pavement and catch basins/manholes due to frost action, backfill around these structures should consist of free-draining granular material. The granular material should be compacted to 98 percent SPMDD with a small tamper to avoid damaging the structures. In addition, catch basins should be perforated just above the drain and the holes screened with filter cloth.

- The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, etc. may be required, especially if construction is carried out during unfavourable weather.

4.5 Building Construction

The site is considered suitable for construction of the single-family dwellings and townhouses, as well as the elementary school. Mid-rise structures for the proposed mixed-use block may also be designed for the available bearing resistances below. If higher bearing values are required, foundations for midrise structures would require the use of more elaborate construction methods (raft foundations, deep foundations, etc.) which were not explored in this investigation. The founding level should be kept as high as possible (i.e. not more than one level of basement is preferred) so buildings are founded in the very stiff to hard silty clay (or engineered fill overlying this very stiff to hard layer). This will also help to limit the depth of excavations below the groundwater level.

The proposed site grade raises will induce long-term consolidation settlements of the underlying silty clay. As such, prior to building construction in those areas, pre-loading will be required. Details of the settlements and pre-loading requirements are discussed in Section 4.2 of this report.

4.5.1 Conventional Foundations

Based on the findings of the investigation, the proposed buildings can be supported on conventional footings founded below any topsoil, fill, or disturbed soils on the hard upper crust of the native silty clay at or below the approximate depths and elevations provided in the table below. **Note that the silty clay typically became weaker below depths of 3.1 m below existing grades and as such the geotechnical resistances values provided are based on founding levels above this depth; EXP must be contacted to review the final site grades and proposed foundation elevations once available to ensure the geotechnical resistances provided are applicable**.

Table 4-4: Preliminary Available Geotechnical Resistance

Borehole No.	Preliminary Available Geotechnical Resistance (kPa)	Founding Soils	Recommended Minimum Founding Depth (m)	Elevation (m)
BH-01	200 SLS / 300 ULS	Silty Clay	1.8	175.1
BH-02	200 SLS / 300 ULS	Silty Clay	1.8	175.6
BH-03	200 SLS / 300 ULS	Silty Clay	1.1	176.8
BH-04	200 SLS / 300 ULS	Silty Clay	1.8	175.4
BH-05	200 SLS / 300 ULS	Silty Clay	1.5	175.3
BH-06	200 SLS / 300 ULS	Silty Clay	1.1	176.1
BH-07	200 SLS / 300 ULS	Silty Clay	1.8	175.0
BH-08	200 SLS / 300 ULS	Silty Clay	1.8	175.2
BH-09	200 SLS / 300 ULS	Silty Clay	0.5	176.7
BH-10	200 SLS / 300 ULS	Silty Clay	1.8	174.8
BH-11	200 SLS / 300 ULS	Silty Clay	0.3	176.7

Borehole No.	Preliminary Available Geotechnical Resistance (kPa)	Founding Soils	Recommended Minimum Founding Depth (m)	Elevation (m)
BH-12	200 SLS / 300 ULS	Silty Clay	0.3	176.4
BH-13	200 SLS / 300 ULS	Silty Clay	0.6	176.7
BH-14	200 SLS / 300 ULS	Silty Clay	0.6	177.0
BH-15	200 SLS / 300 ULS	Silty Clay	1.8	175.4
BH-16	200 SLS / 300 ULS	Silty Clay	1.2	176.5
BH-17	200 SLS / 300 ULS	Silty Clay	0.6	176.6
BH-18	200 SLS / 300 ULS	Silty Clay	0.7	176.5
BH-19	200 SLS / 300 ULS	Silty Clay	4.9	174.6
BH-20	200 SLS / 300 ULS	Silty Clay	2.6	175.7
BH-21	200 SLS / 300 ULS	Silty Clay	1.2	176.5
BH-22	200 SLS / 300 ULS	Silty Clay	0.5	176.4
BH-23	200 SLS / 300 ULS	Silty Clay	1.1	175.9
BH-24	200 SLS / 300 ULS	Silty Clay	1.2	175.9
BH-25	200 SLS / 300 ULS	Silty Clay	1.3	175.9
BH-26	200 SLS / 300 ULS	Silty Clay	1.3	175.3
BH-27	200 SLS / 300 ULS	Silty Clay	1.4	175.4

The footing base must be inspected by geotechnical personnel from EXP prior to engineered fill or concrete placement in order to confirm the availability of the geotechnical resistance values provided and the foundation elevation.

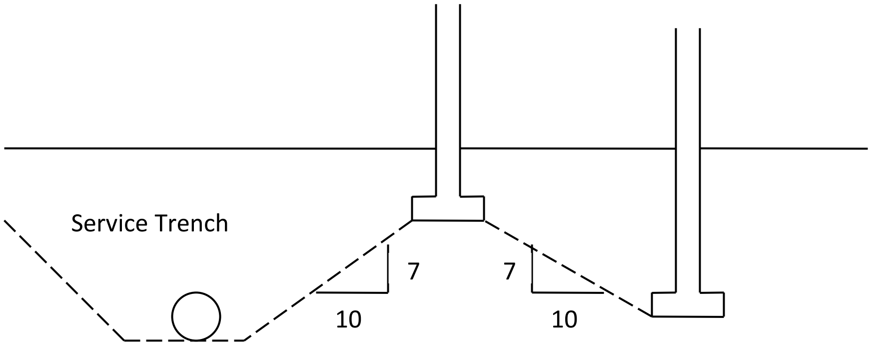
If required, due to the site grades or presence of existing fill materials, the buildings can be supported on conventional footings founded on engineered fill overlying native soil. Engineered fill is to be constructed in accordance with Drawing No. 30 in Appendix A and preloaded as per Section 4.2 of this report. Conventional footings founded on engineered fill overlying native soil may be designed utilizing a geotechnical resistance of 150 kPa at SLS and 225 kPa at ULS.

4.5.2 Potential for Expansive Clays

The above lab testing indicates silty clay materials with clay contents of 31% to 71% and plasticity indexes of 14 to 38. Based on these results and the Canadian Foundation Engineering Manual (Section 15), the in-situ soils would be considered to generally have low potential severity for expansive properties, though the samples with higher clay contents indicate isolated areas of high potential severity for swelling. While expansive clays are encountered infrequently in the Niagara Region, their presence has been documented both by EXP and in other available geotechnical sources from the area. It would be prudent to carry out additional lab testing (swelling tests) undertaken on samples taken from the site to further evaluate the potential for swelling soils and the need for mitigating construction measures.

4.5.3 General Foundation Recommendations

Foundations at different elevations should be located such that higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing. This concept should also be applied to excavations for new foundations in relation to existing footings or underground services.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

All footings 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.

Provided that the soil is not disturbed due to groundwater, precipitation, traffic, etc., and the aforementioned bearing pressure is not exceeded, then total and differential settlements should be small and within the normally tolerated limits of 25 mm and 19 mm, respectively.

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.

4.5.4 Excavations

It is anticipated excavations for the proposed buildings will extend to in the order of 1.5 to 3.0 m below the final site grades. Excavations are generally anticipated to encounter the fill materials and native silty clay. Excavations can be carried out in open-cut with conventional excavation equipment.

All excavations must be completed in accordance with the most recent regulations of the Ontario Occupation Health and Safety Act (OHSA). In general, the fill, disturbed native materials, and firm to stiff silty clay may generally be classified as Type 3 Soil above the groundwater level. The very stiff to hard silty clay may be classified as Type 2 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 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.

4.5.5 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, in kPa acting at depth h;
K = Coefficient of earth pressure, assume to be 0.40
γ = Unit weight of backfill, assume 21.0 kN/m^3
h = Depth to point of interest (m); and
q = Surcharge load acting adjacent to the wall at the ground surface in kPa

The above expression assumes that the perimeter drainage system prevents the build-up of hydrostatic pressure behind the wall and free-draining granular material will be used for backfilling adjacent to the wall.

4.5.6 Floor Slab Construction & Permanent Drainage

Floor slabs for proposed structures may be constructed as a slab-on-grade on native soils or on engineered fill. No significant problems are anticipated for slab-on-grade construction provided the procedures outlined in Section 4.1 of the report are adhered to. A 200 mm layer of 19 mm clear stone should be placed between the prepared subgrade and the floor slab to serve as a moisture barrier. Perimeter drainage will be required for buildings with basements. Perimeter drains can be eliminated if the ground floor slab is at least 300 mm above the exterior grade.

Underfloor drainage may also be required depending on the building configuration, site grades, etc. and EXP should be contacted to review this requirement once the development plan is determined.

Around the perimeter of the buildings, the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and to reduce groundwater infiltration adjacent to the foundations.

4.6 Earthquake Considerations

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

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 subsoil generally consisted of fill and firm to hard silty clay. Foundations are anticipated to be founded on the native silty clay or engineered fill overlying native silty clay. The reported N values for the native soil below the anticipated founding level ranged from 0 to 24 blows per 305 mm penetration. Undrained shear strengths from in-situ shear vane tests and pocket penetrometer readings ranged from 15 to 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.

4.6.2 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.

4.6.3 Depth of Boreholes

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicated that to determine the site classification, the average properties in the top 30 m are to be used. The site classification recommendation is based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions and bedrock depths in the area.

4.6.4 Site Classification

Based on the above assumptions and interpretations and the known soil conditions, the Site Class for this site is "E" as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012 for foundations constructed on native soil in accordance with this report. An improved site classification may be possible if shear wave velocity testing is carried out and EXP can be contacted to provide this service, if required.

4.7 Stormwater Management Pond

It is understood that the project will include the construction of a stormwater management pond (SWP). Based on preliminary design details, the proposed SWP will be constructed at the southern end of the site, with the base of the pond at approximately 4 to 5 m below present grades. Boreholes BH-10, BH-26, and BH-27 were advanced within the general area of the proposed SWP, with monitoring wells installed in each of Boreholes BH-10 and BH-27. These boreholes generally encountered surficial silty clay fill over native silty clay. The groundwater levels, as reported above, ranged from approximately 1.2 m to 2.1 m below existing grade. Preliminary comments on the pond construction are provided in the following sections. EXP should be contacted to review the final pond details once available to confirm the below recommendations and provide any additional comments.

4.7.1 Pond Excavation & Construction

The proposed pond excavation depths are expected to extend to 4 to 5 m below existing grade. Excavations above the groundwater level are not expected to encounter significant issues and can be carried out with a heavy hydraulic excavator. Some groundwater infiltration from surface run-off and perched water within the fill may be encountered but should be possible to control using conventional construction pumping techniques. More significant dewatering methods may be required for deeper excavations below the groundwater level. Reference to the EXP hydrogeological study for the site should be made for the safe excavation depths.

Based on the shallow groundwater conditions encountered at the sites of the proposed SWP and to maintain separation between the stormwater and groundwater, a drainage system in combination with a synthetic or compacted clay liner should be anticipated. It should be noted that drawing down the groundwater level can result in settlements of structures within the zone of influence. Depending on the pond base elevation, dewatering restrictions, and available drainage courses, more elaborate groundwater control methods may be required, e.g. structural concrete slab and tie-downs.

4.7.2 Compacted Clay Liner

4.7.2.1 Material Specifications

All materials to be used in the construction of a clay liner shall be analyzed for particle size distribution and Atterberg Limits.

If the distribution of the particle sizes and the Atterberg Limits fall within the ranges given below, the material is considered acceptable for compacted clay liner construction without the need for additional laboratory testing, provided it is installed using the recommended equipment and procedures. The use of materials as defined herein are expected to produce a clay liner with a hydraulic conductivity of 1×10^{-8} meters/second or less.

Acceptable particle size ranges (by weight):

- Percent fines ≥ 50%
- Clay Content ≥ 20%
- Sand Content ≤ 45%

Where the fines are defined as the soil fraction which passes through a No. 200 (75 µm) US Standard sieve, and clay and sand are defined in the ASTM D2487 standard.

Acceptable Atterberg Limits:

- Plasticity Index (PI) ≥ 0.73(LL - 20)
- Liquid Limit (LL) ≥ 40

As noted above and appended to this report, grain size analyses and Atterberg limits were conducted on seven (7) plastic samples from Boreholes BH-10, BH-26, and BH-27, which were advanced in the area of the proposed SWP. The results of this testing indicated Liquid Limits ranging from 29 to 60, and Plasticity Indexes ranging from 14 to 39. Clay contents of the tested samples ranged from 31 to 60 percent. Based on these results and the above requirements, the encountered materials at the site are generally considered of medium to high plasticity and would be suitable for use in the clay liner, but would contain some layers of lower plasticity silty clay that may be unsuitable. Selective sorting of the material and additional laboratory testing of stockpiled soil may be required to confirm its suitability for use in the liner.

When the material to be used for the construction of a compacted clay liner does not meet all the criteria above, additional testing is required to demonstrate that the "as-constructed" clay liner will have a field hydraulic conductivity of $1x10^{-8}$ meters/second or less. Laboratory hydraulic conductivity shall be determined following ASTM 5084 on no less than three samples after compaction to at least 95 percent SPMDD.

Alternatively, material for the clay liner may be made from 1 part bentonite powder and 3.5 parts Granular A (OPSS 1010) by volume. Mixing of the material shall be carried out in an approved mechanical mixer.

4.7.2.2 Construction Equipment

The recommended compaction equipment for the construction of a clay liner is the sheepsfoot roller. Many different models of sheepsfoot roller compactors are available. Only those meeting the following criteria shall be considered acceptable:

- *Soil Contact Pressures* – The compaction equipment or rollers shall be ballasted to attain soil contact pressures of at least 2400 kPa.

- *Soil Contact Pressure Measurement* – Contact pressure shall be measured by dividing the total mass of the roller by either the total area of the maximum number of tamping feet in one row parallel to the axis of the roller; or by calculating 55 of the total foot area, whichever is the greater.

- *Tamping Feet Requirement* – The tamping feet shall be 200 mm to 250 mm in length from the cylindrical surface of the roller. The tamping feet shall have a face area between 4500 and 6000 mm^2. The compactor feet shall be spaced to provide at least 4 tamping feet for each 0.25 m^2 of cylindrical surface.

- *Equipment Cleaning* – The roller shall be equipped with cleaning fingers to prevent the accumulation of material between the tamping feet and to allow full penetration of the feet through the lift being compacted.

4.7.2.3 Liner Placement and Compaction

The following comments regarding liner placement and compaction should be adhered to:

- *Compaction Specification* - The clay liner shall be compacted to at least 95 percent SPMDD at a moisture content between 90 and 120 percent of optimum.

- *Moisture Management* – If additional moisture is required for compaction, water shall be applied by sprinkling directly on the liner material. The quality of the water shall be subject to the approval of the Engineer and shall be free from undesirable quantities of organic matter and mineral salts. Water application pressure shall be controlled to prevent erosion of the liner and to prevent freestanding water on the surface.

- *Liner Lift Placement* – Liner materials shall be spread by a motor grader or other means approved by Engineer to obtain a uniform lift thickness prior compaction. In the bottom of the excavation, the liner material shall be first placed in the lowest elevations.

- *Lift Thickness* – The foot length of the compaction equipment will govern the thickness of the loose lift that can be compacted. The thickness of each uncompacted lift shall be at least 25 mm less than the foot length of the compaction equipment. This is to ensure full penetration through the uncompacted lift and into the previous compacted liner layer or subgrade on the first pass of the compaction roller feet.

- *Bonding Between Lifts* – If the surface prior to placing the next lift is too hard for the feet on the sheepsfoot roller to penetrate, the surface shall be scarified.

- *Foreign Materials* – All rocks greater than 75 mm in diameter, roots and organic debris shall be removed from the liner material prior to compaction.

- *Construction Below Freezing* – Excavation and compaction shall be completed only when soil temperatures are above freezing.

- *Overlap of Equipment Passes* – The overlap between equipment passes shall not be less than 10 percent of width of the equipment being used to ensure lateral bonding between placed materials.

- *Liner Desiccation* – Each compacted lift shall be protected from drying out to prevent cracking due to shrinkage.

4.7.3 Synthetic Liner

A synthetic liner (e.g. Bentofix NWL or equivalent) or a composite liner can be considered. The liner should be installed on a properly prepared subgrade in accordance with the liner manufacturer’s specifications. If the pond slopes are to be vegetated, it will be necessary to specify a liner that will exhibit sufficient friction to ensure topsoil will not slide off the liner when the pond is in service.

The liner should be installed in accordance with the following procedures:

1. After stripping, the exposed subgrade should be inspected and approved by a geotechnical representative from this office. The groundwater table should be lowered to at least 0.5 m below the pond base level.

2. A 150 mm diameter perforated drain should be installed in a drainage blanket and discharged to a sump or the nearest suitable creek. The water in the sump should not exceed elevation of the base of the pond to prevent uplift.

3. A minimum 150 mm levelling sand layer should be placed over the drainage blanket prior to installing the synthetic liner.

4. A minimum 200 mm of top dressing should be placed over the synthetic liner, which will act as a “reminder layer”. The rock layer is to mark the location of the liner for future maintenance operations. As an alternative to rip-rap, 300 mm of native compacted soil may be used if orange plastic “safety fencing” or another highly-visible, continuous marker is embedded 150 mm above the membrane. This will ensure that during maintenance operations machinery operators know when they have reached the bottom of the pond and do not over-excavate and damage the liner.

5. Alternatively, a geo-cell confinement system could be installed over the liner and infilled with topsoil above the permanent water elevation. As penetrations through the liner would not be allowed, the geo-cell system would need to be anchored at the top of the pond in a keyway and supported by tendons that extend through

the geo-cell webbing. Geo-Web cellular confinement or a similar system could be considered for this purpose.

4.7.4 Pond Grading and Surface Treatment

Loose and unsuitable materials should be removed from pond slope areas. Provided an adequate drainage system is implemented, the sides of the ponds may be sloped at 3 horizontal to 1 vertical or flatter. The slopes should be surface compacted with a heavy roller and should be grassed to prevent surface erosion. Armour stone or rip-rap with filter cloth backing should be placed on the slope face where extensive wet/saturated layers are encountered to prevent side sloughing.

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.

Jeff Golder

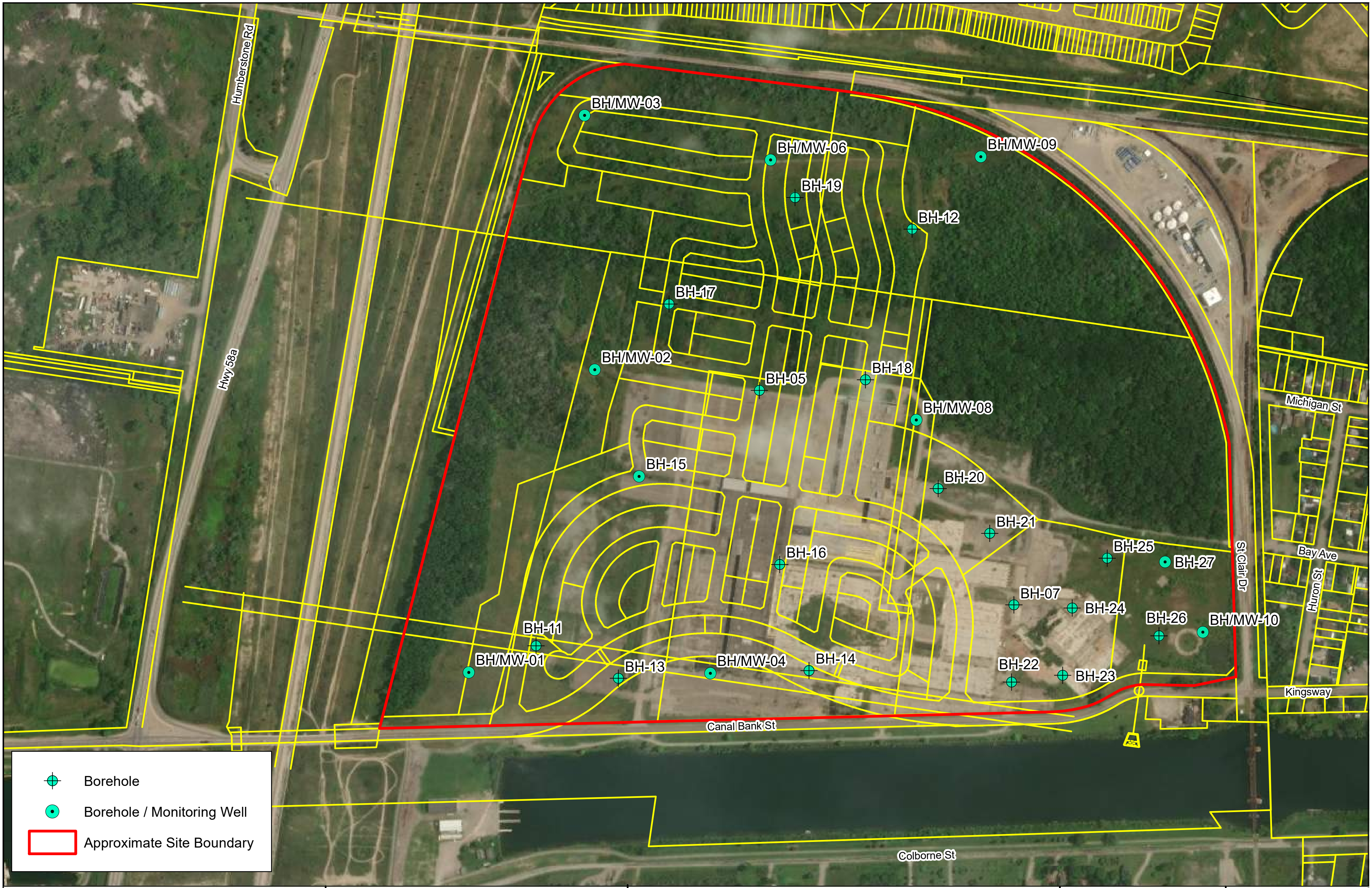
Matt LiVecchi, P.Eng.
Geotechnical Project Manager

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

LICENSED PROFESSIONAL ENGINEER
M. M. LIVECCHI
100189574
PROVINCE OF ONTARIO

Appendix A

Drawings & Borehole Logs



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TRUE
NORTH

0 50 100 150 200 250 m

TITLE AND LOCATION:

BOREHOLE / MONITORING WELL
LOCATION PLAN
555 Canal Bank Street
Welland, Ontario

PROJECT No.: HAM-00801631-I0	DWN: AC
SCALE: AS NOTED	CHKD: ML
DATE: AUGUST 2020	DWG. No.: 1

Drawing 2A

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

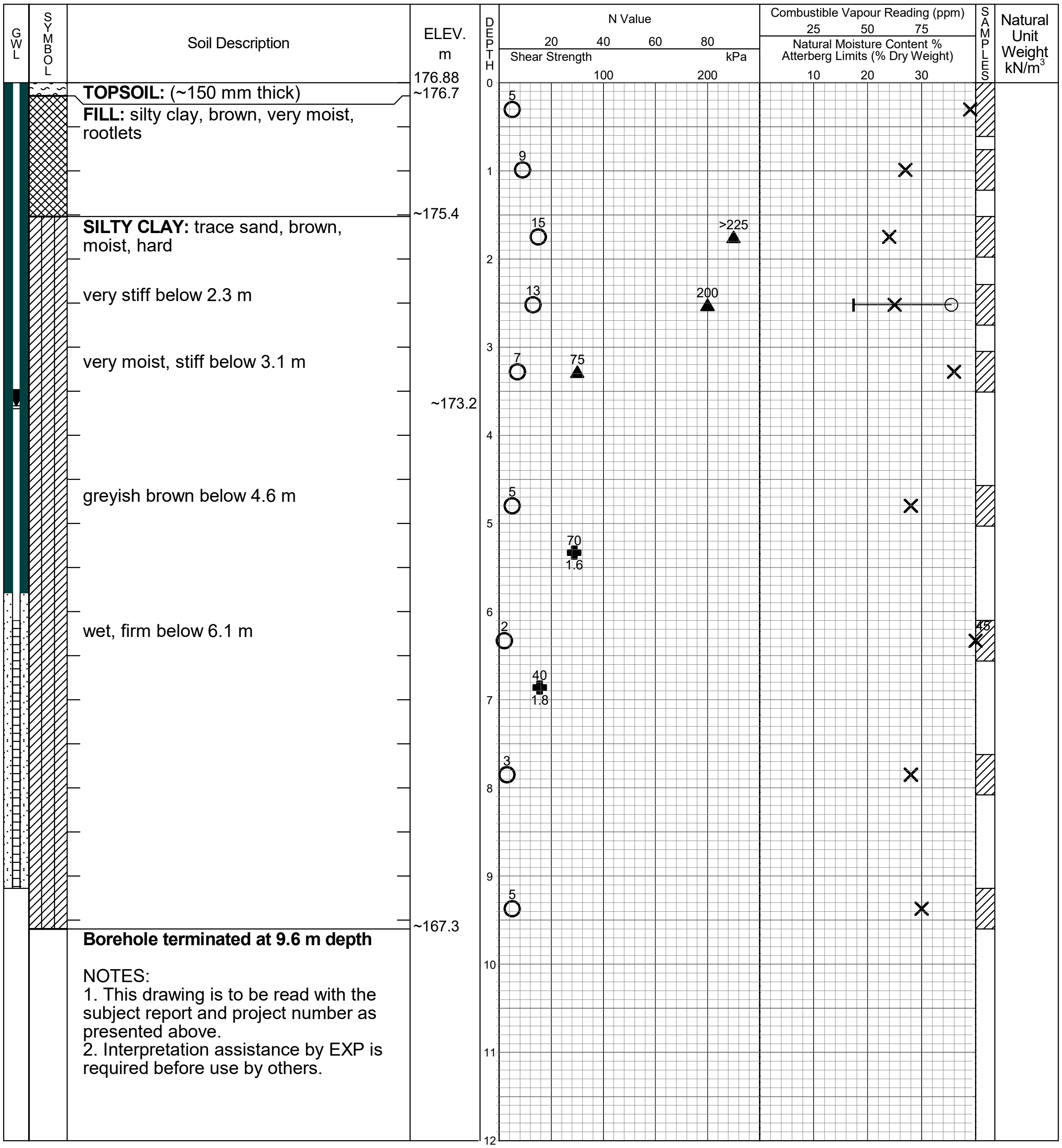
$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

Log of Borehole BH-01

Project No. HAM-00801631-A0
Drawing No. 3
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 14, 2019
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ELEV. m	Soil Description
176.88	
~176.7	**TOPSOIL:** (~150 mm thick)
	FILL: silty clay, brown, very moist, rootlets
~175.4	**SILTY CLAY:** trace sand, brown, moist, hard
	very stiff below 2.3 m
	very moist, stiff below 3.1 m
~173.2	
	greyish brown below 4.6 m
	wet, firm below 6.1 m
~167.3	**Borehole terminated at 9.6 m depth**

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 8/7/20

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

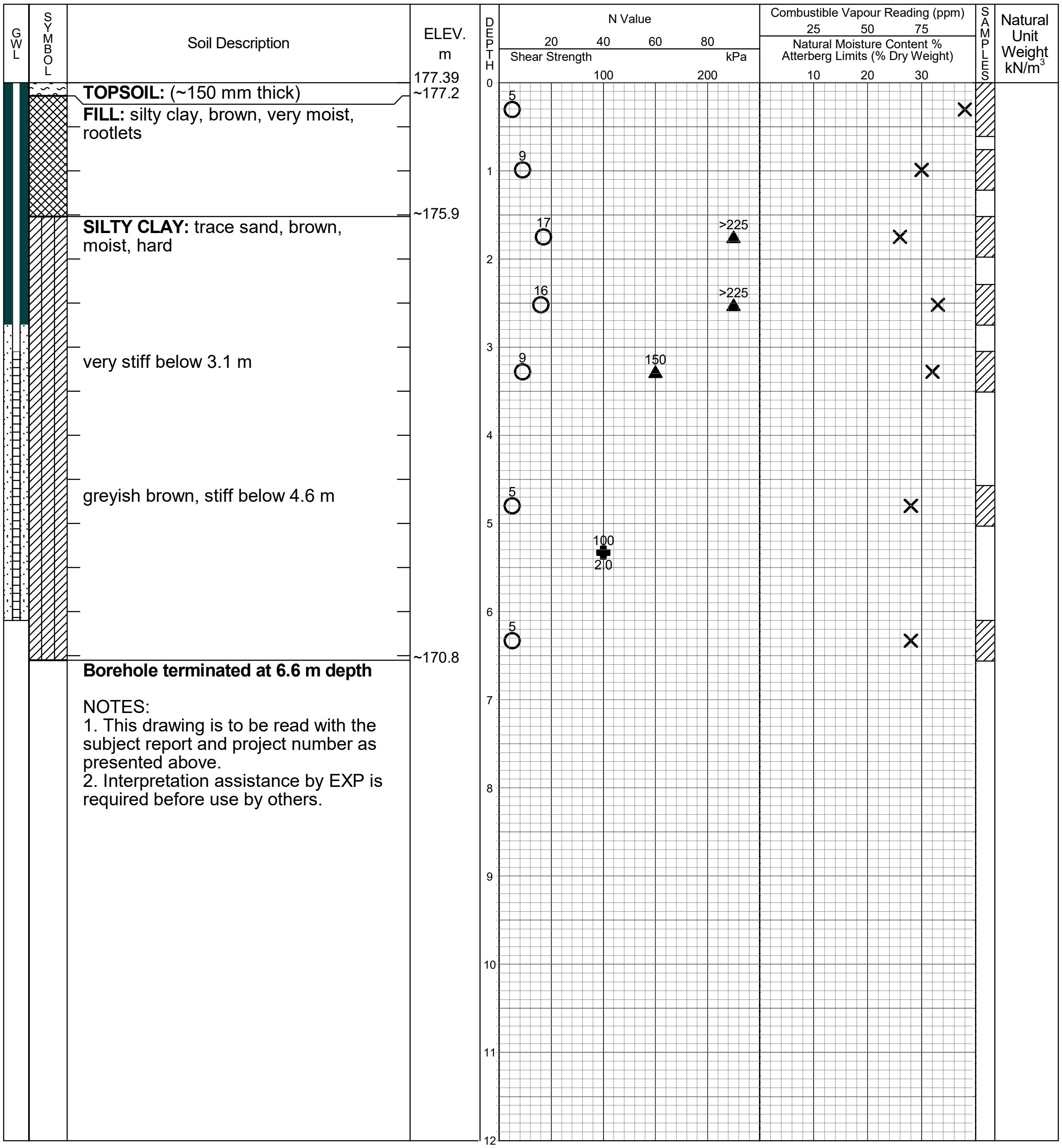
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	9.6
February 19, 2019	4.4	N/A
March 4, 2019	6.7	N/A
April 15, 2020	4.9	N/A
July 27, 2020	3.7	N/A

Log of Borehole BH-02

Project No. HAM-00801631-A0
Drawing No. 4
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 13, 2019
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	⊠	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├─○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ELEV. m	Soil Description
177.39 ~177.2	**TOPSOIL:** (~150 mm thick)
	FILL: silty clay, brown, very moist, rootlets
~175.9	**SILTY CLAY:** trace sand, brown, moist, hard very stiff below 3.1 m greyish brown, stiff below 4.6 m
~170.8	**Borehole terminated at 6.6 m depth**

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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Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.6
February 19, 2019	no free water	N/A
March 4, 2019	no free water	N/A
April 15, 2020	no free water	N/A

Log of Borehole BH-03

Project No. HAM-00801631-A0 | Drawing No. 5

Project: Proposed Subdivision | Sheet No. 1 of 1

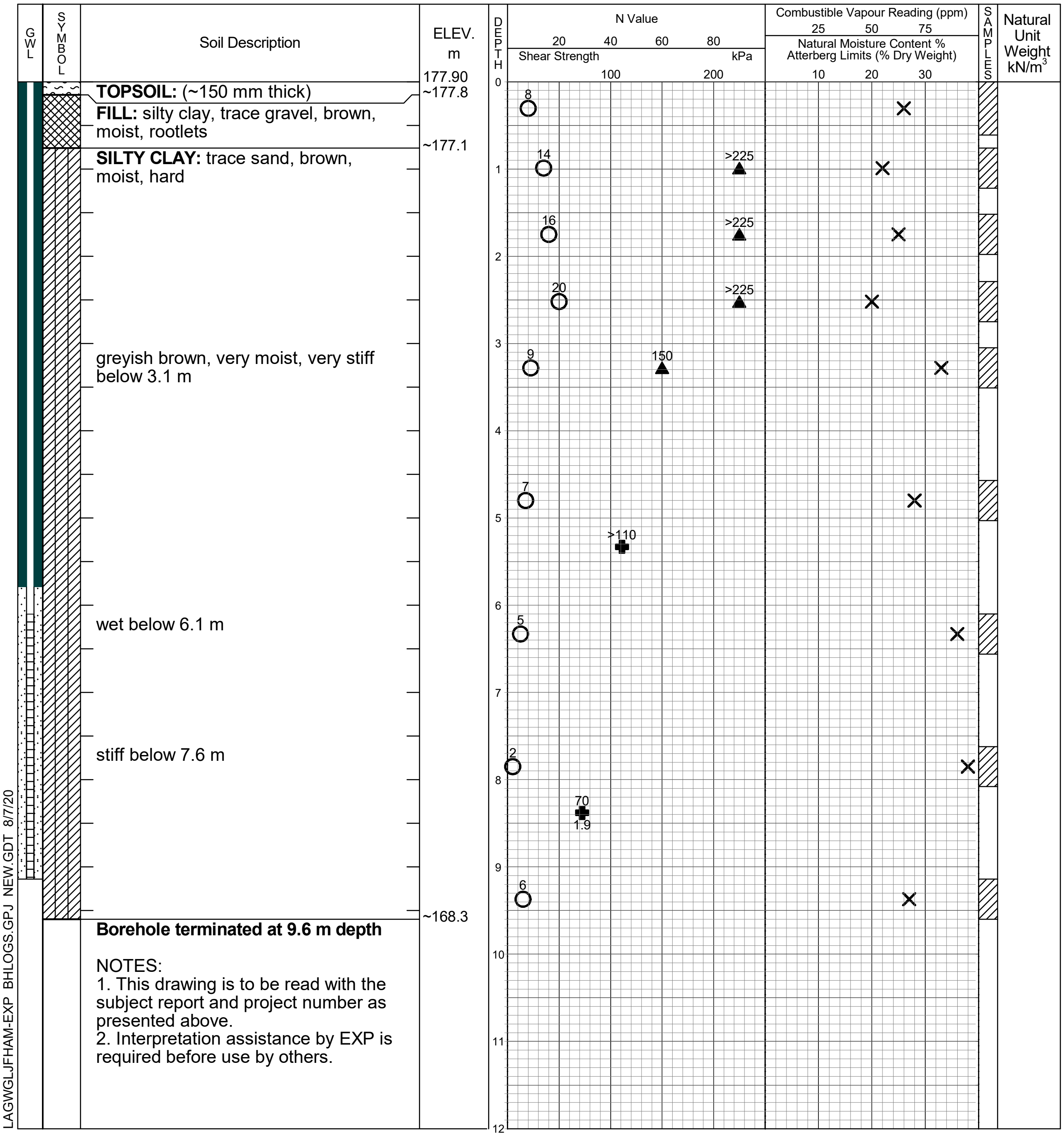
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 13, 2019

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



TOPSOIL: (~150 mm thick)

FILL: silty clay, trace gravel, brown, moist, rootlets

SILTY CLAY: trace sand, brown, moist, hard

greyish brown, very moist, very stiff below 3.1 m

wet below 6.1 m

stiff below 7.6 m

Borehole terminated at 9.6 m depth

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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Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	9.6
February 19, 2019	no free water	N/A
March 4, 2019	no free water	N/A
April 15, 2020	no free water	N/A

Log of Borehole BH-04

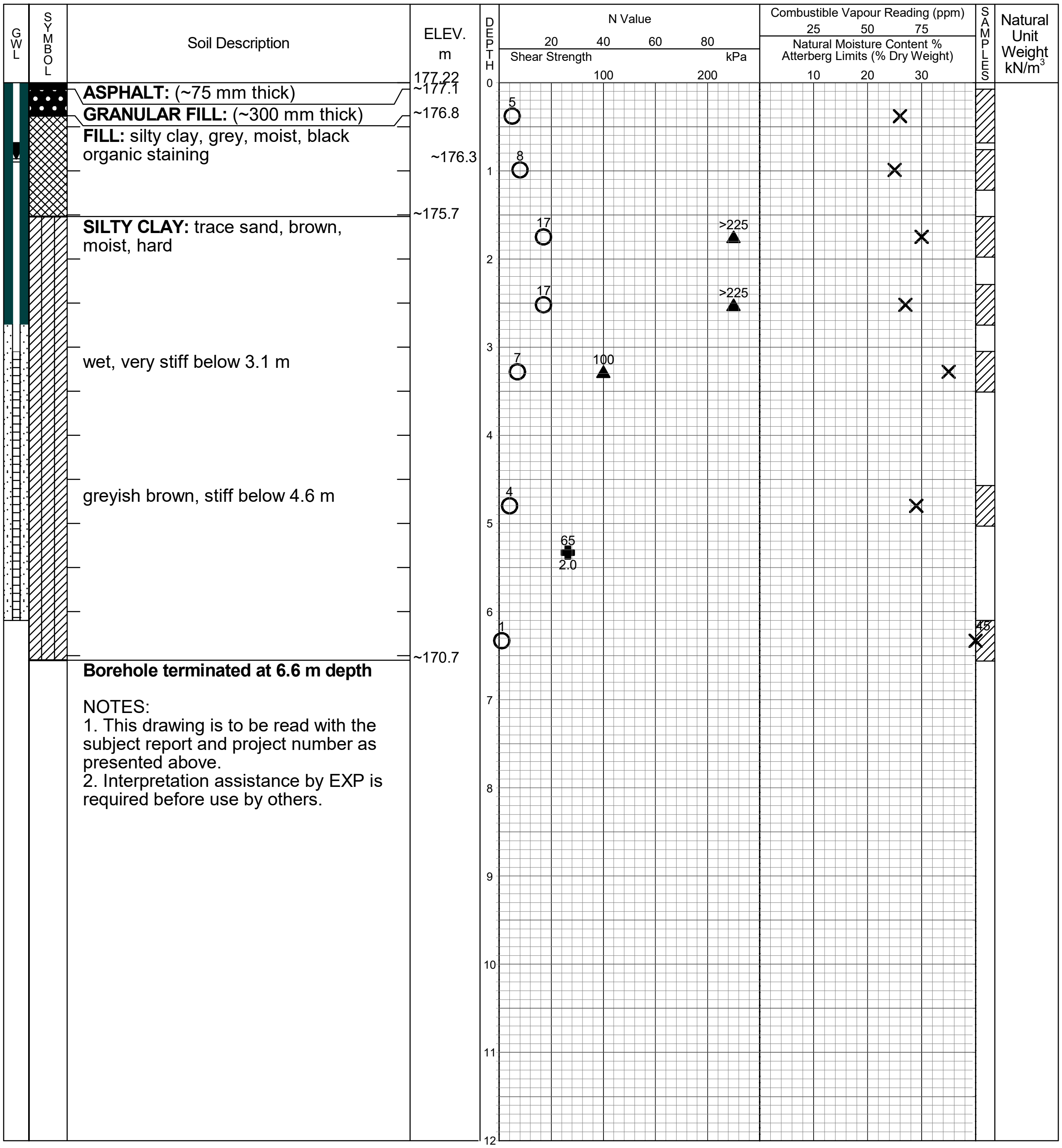
Project No.	HAM-00801631-A0	Drawing No.	6
Project:	Proposed Subdivision	Sheet No.	1 of 1
Location:	555 Canal Bank Street, Welland, ON		

Date Drilled: February 14, 2019

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	―	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~300 mm thick)

FILL: silty clay, grey, moist, black organic staining

SILTY CLAY: trace sand, brown, moist, hard

wet, very stiff below 3.1 m

greyish brown, stiff below 4.6 m

Borehole terminated at 6.6 m depth

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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Time	Water Level (m)	Depth to Cave (m)
on completion	5.3	6.6
February 19, 2019	4.6	N/A
March 4, 2019	2.4	N/A
April 15, 2020	1.2	N/A
July 27, 2020	0.9	N/A

Log of Borehole BH-05

Project No. HAM-00801631-A0

Drawing No. 7

Project: Proposed Subdivision

Sheet No. 1 of 1

Location: 555 Canal Bank Street, Welland, ON

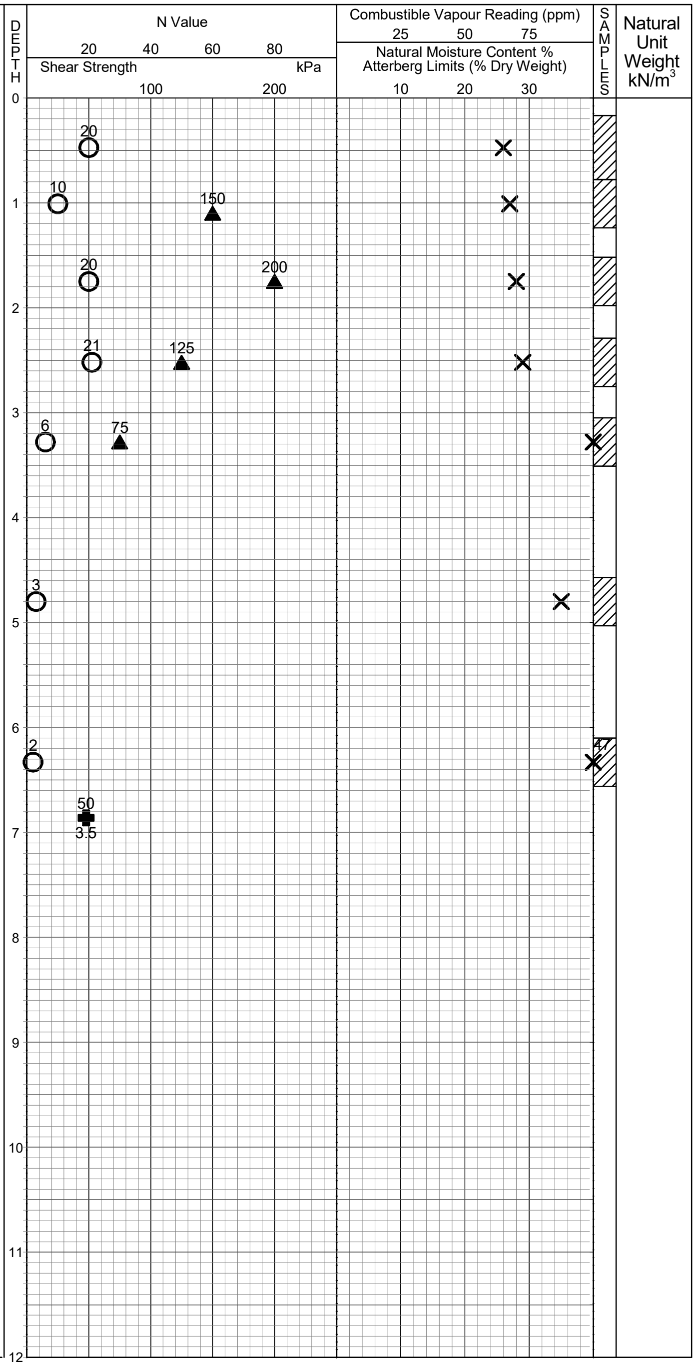
Date Drilled: February 13, 2019

Drill Type: CME 75 Truck Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

Soil Description	ELEV. m
	176.77
ASPHALT: (~170 mm thick)	~176.6
GRANULAR FILL: (~340 mm thick)	~176.3
FILL: silty clay, brown, moist	~175.7
SILTY CLAY: trace sand, brown, moist, very stiff trace gravel below 1.5 m wet, stiff below 3.1 m greyish brown below 4.6 m	~169.9
Borehole terminated at 6.9 m depth	



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.

Time	Water Level (m)	Depth to Cave (m)
on completion	4.0	6.6

exp.

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Log of Borehole BH-06

Project No. HAM-00801631-A0

Drawing No. 8

Project: Proposed Subdivision

Sheet No. 1 of 1

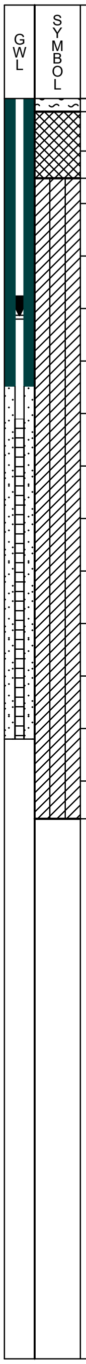
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 13, 2019

Drill Type: D-50 Track Mount. Solid Stem.

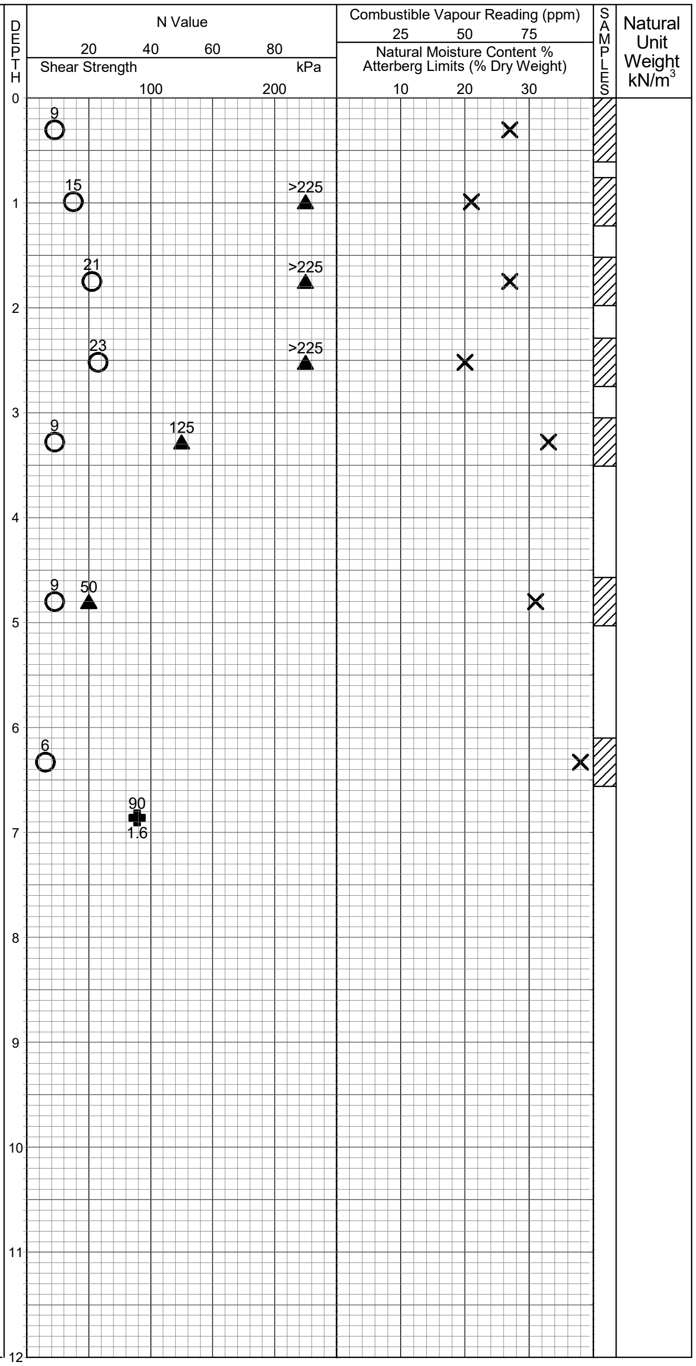
Datum: Geodetic

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	▲



Soil Description	ELEV. m
	177.24
TOPSOIL: (~125 mm thick)	~177.1
FILL: silty clay, brown, moist, rootlets	
	~176.5
SILTY CLAY: trace sand, brown, moist, hard, rootlets	
	~175.1
very moist, very stiff below 3.1 m	
greyish brown, stiff below 4.6 m	
wet below 6.1 m	
	~170.4
Borehole terminated at 6.9 m depth	

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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Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.6
February 19, 2019	2.6	N/A
March 4, 2019	2.1	N/A
April 15, 2020	1.4	N/A

Log of Borehole BH-07

Project No. HAM-00801631-A0 Drawing No. 9

Project: Proposed Subdivision Sheet No. 1 of 1

Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 11, 2019

Drill Type: D-50 Track Mount. Solid Stem.

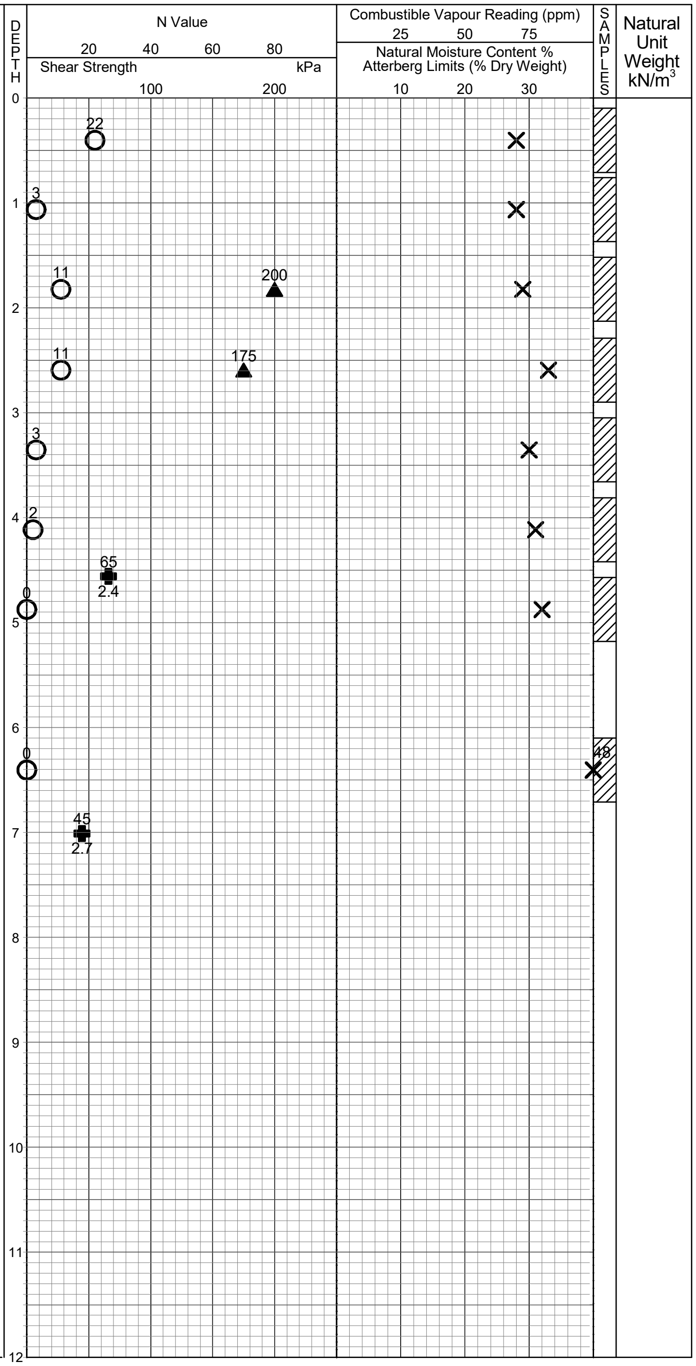
Datum: Geodetic

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	▲

GWL	SYMBOL	Soil Description	ELEV. m
			176.77
		ASPHALT: (~100 mm thick)	~176.7
		GRANULAR FILL: (~300 mm thick)	~176.4
		FILL: silty clay, grey, moist, rootlets, wood fragments	
		SILTY CLAY: trace sand, brown, moist, very stiff	~175.3
		very moist below 2.3 m	
		grey, stiff below 3.1 m	
		wet, firm below 6.1 m	
		Borehole terminated at 7.0 m depth	~169.8

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.



Time	Water Level (m)	Depth to Cave (m)
on completion	4.6	6.6

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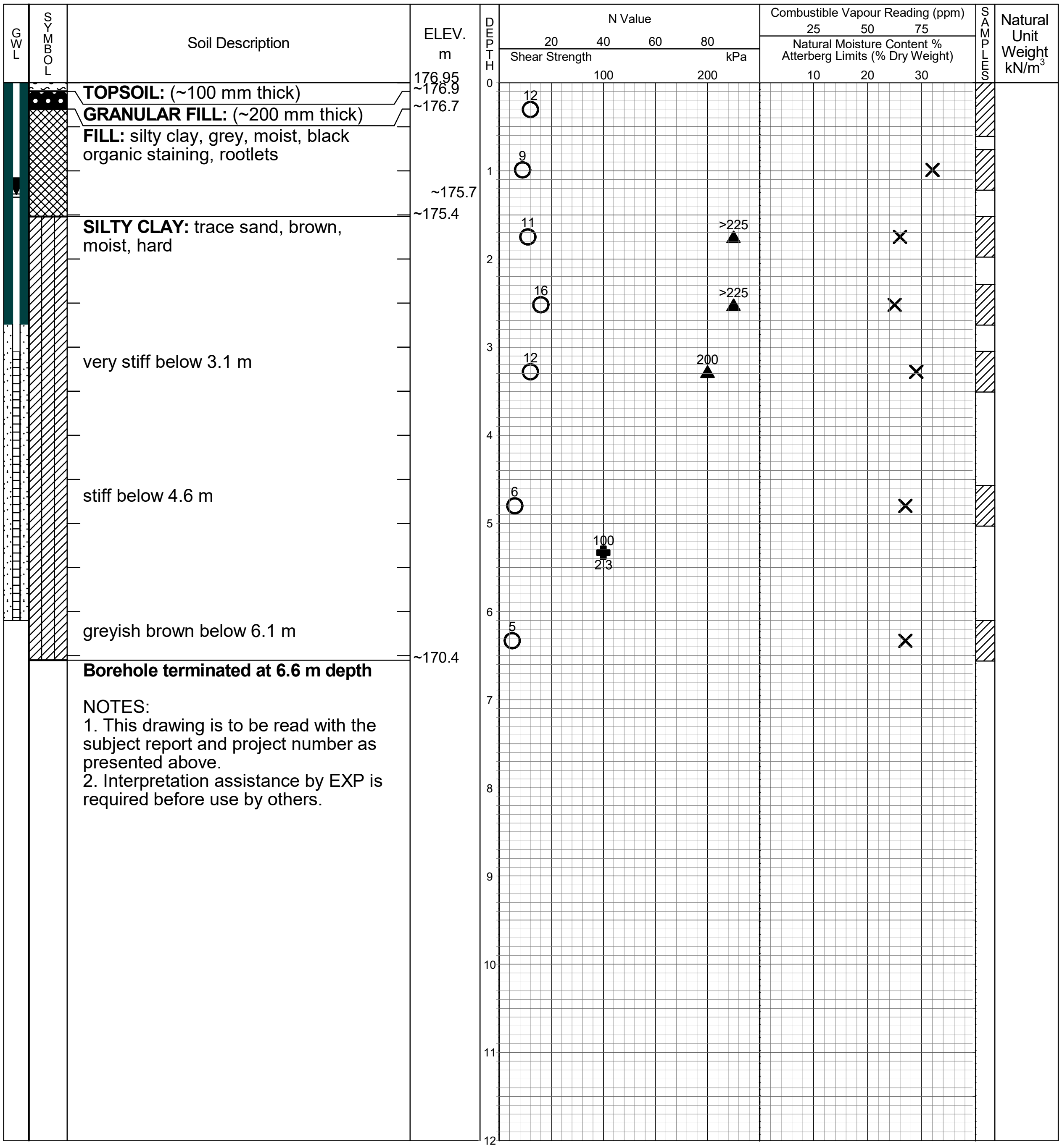
Log of Borehole BH-08

Project No. HAM-00801631-A0
Drawing No. 10
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 11, 2019
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



Soil Description	ELEV. m
	176.95
TOPSOIL: (~100 mm thick)	~176.9
GRANULAR FILL: (~200 mm thick)	~176.7
FILL: silty clay, grey, moist, black organic staining, rootlets	~175.7
SILTY CLAY: trace sand, brown, moist, hard	~175.4
very stiff below 3.1 m	
stiff below 4.6 m	
greyish brown below 6.1 m	~170.4

Borehole terminated at 6.6 m depth

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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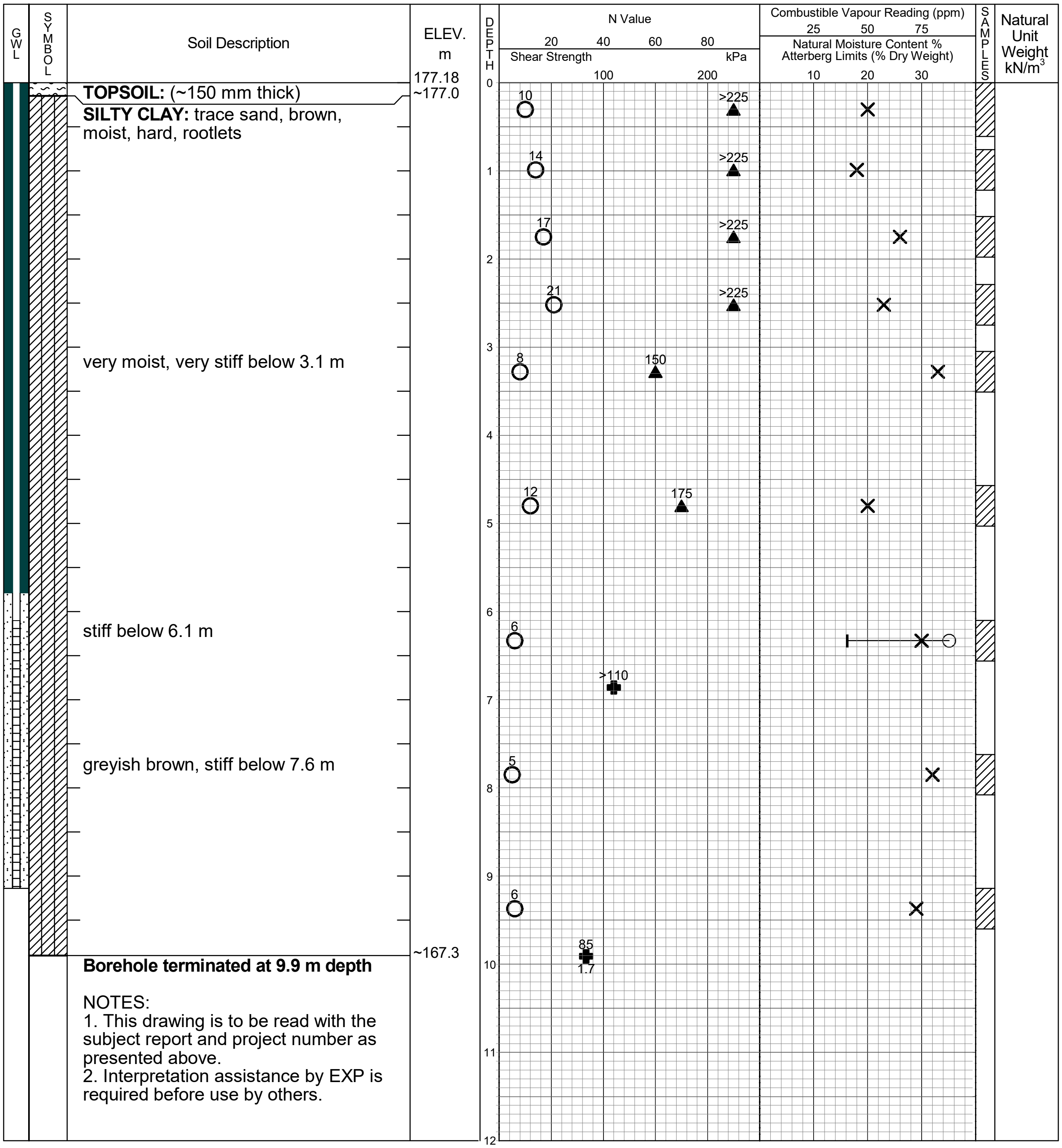
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.6
February 19, 2019	5.4	N/A
March 4, 2019	4.4	N/A
April 15, 2020	2.5	N/A
July 27, 2020	1.3	N/A

Log of Borehole BH-09

Project No. HAM-00801631-A0
Drawing No. 11
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 11, 2019
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



TOPSOIL: (~150 mm thick)

SILTY CLAY: trace sand, brown, moist, hard, rootlets

very moist, very stiff below 3.1 m

stiff below 6.1 m

greyish brown, stiff below 7.6 m

Borehole terminated at 9.9 m depth

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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Telephone: 905.573.4000
Facsimile: 905.573.9693

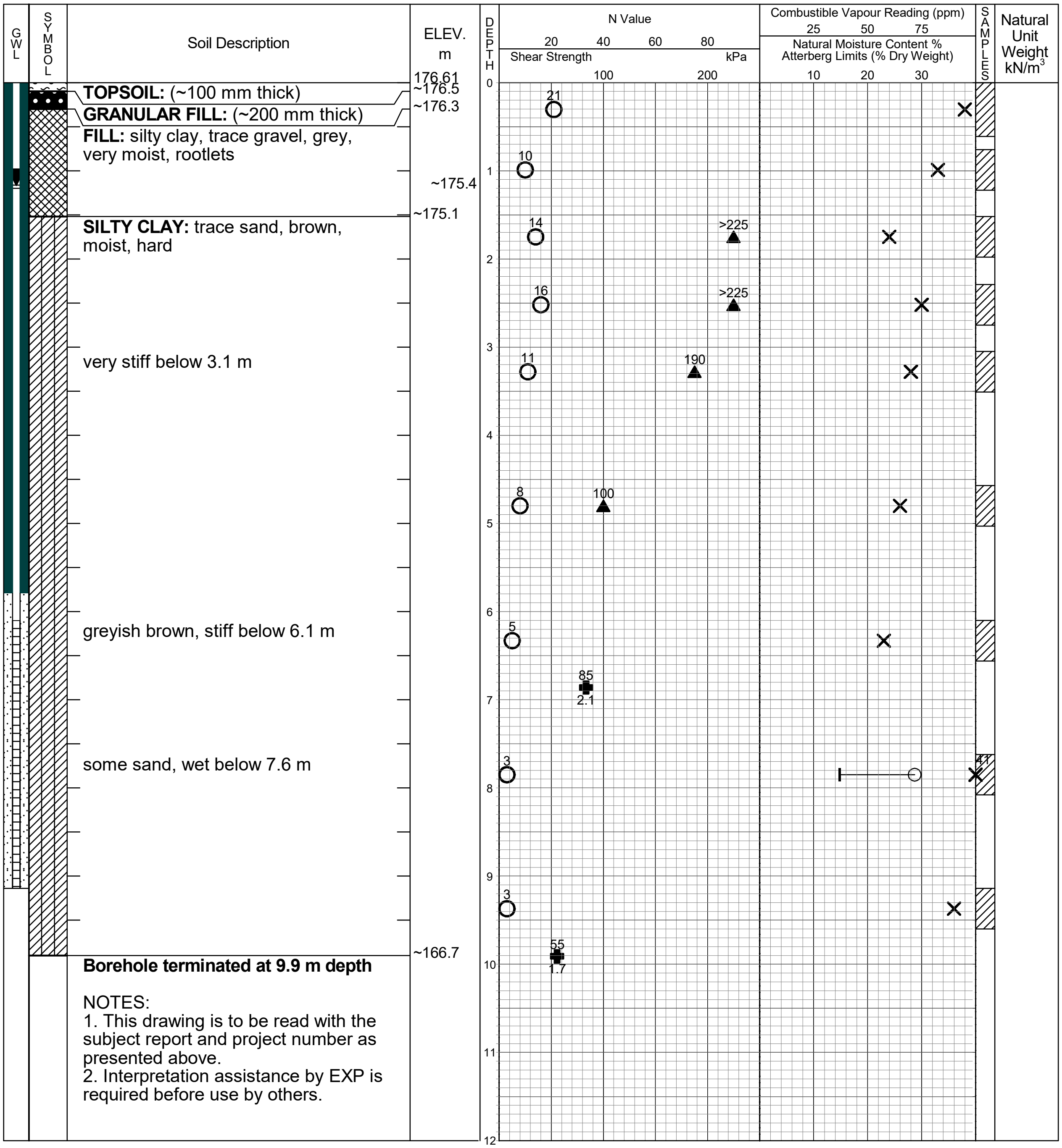
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	9.6
February 19, 2019	no free water	N/A
March 4, 2019	no free water	N/A
April 15, 2020	no free water	N/A
July 27, 2020	no free water	N/A

Log of Borehole BH-10

Project No. HAM-00801631-A0
Drawing No. 12
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: February 11, 2019
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ELEV. m	Soil Description
176.61 ~176.5	**TOPSOIL:** (~100 mm thick)
~176.3	**GRANULAR FILL:** (~200 mm thick)
	FILL: silty clay, trace gravel, grey, very moist, rootlets
~175.4	
~175.1	**SILTY CLAY:** trace sand, brown, moist, hard
	very stiff below 3.1 m
	greyish brown, stiff below 6.1 m
	some sand, wet below 7.6 m
~166.7	**Borehole terminated at 9.9 m depth**

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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Time	Water Level (m)	Depth to Cave (m)
on completion	8.2	9.6
February 19, 2019	1.9	N/A
March 4, 2019	1.8	N/A
April 15, 2020	1.4	N/A
July 27, 2020	1.2	N/A

Log of Borehole BH-11

Project No. HAM-00801631-A0
Drawing No. 13
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: January 27, 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	▲

GWL | SYMBOL | Soil Description | ELEV. m

177.00
~177.0 **TOPSOIL:** (~50 mm thick)

SILTY CLAY: trace sand, brown, moist, hard

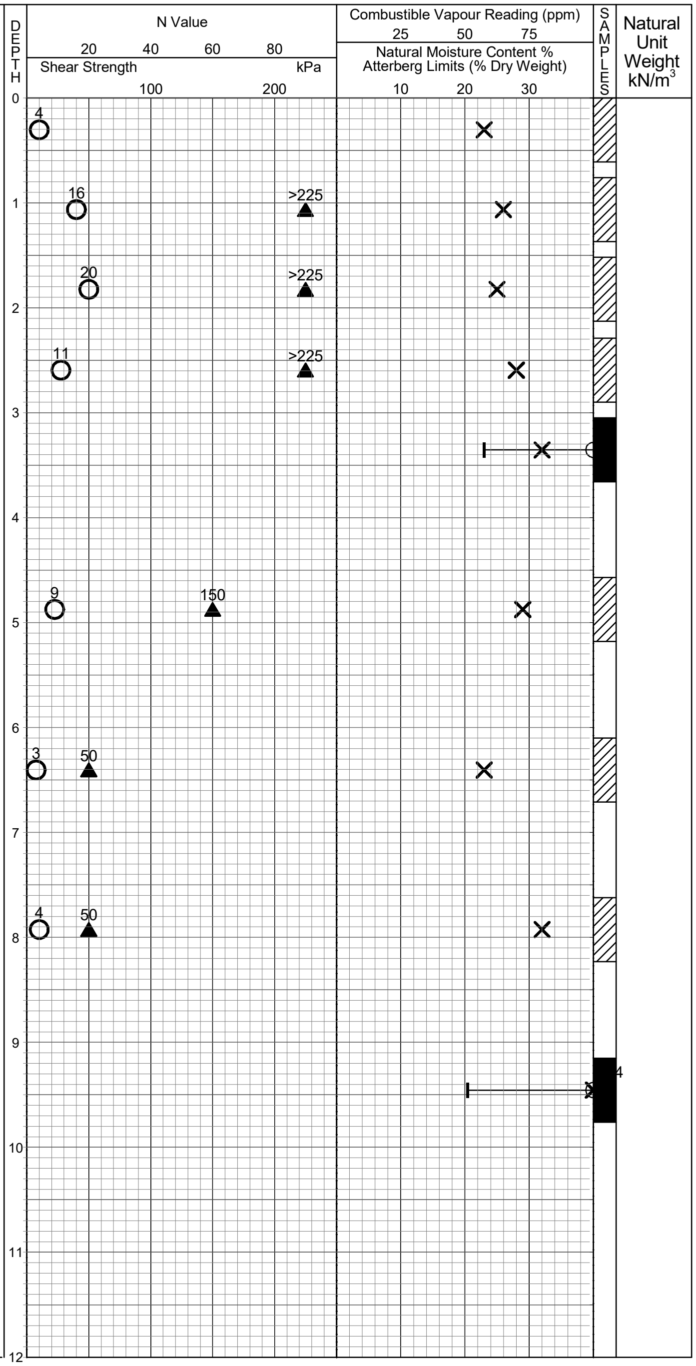
very stiff below 4.6 m

greyish brown, stiff below 6.1 m

wet below 9.2 m

~167.2 **Borehole terminated at 9.8 m depth**

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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Facsimile: 905.573.9693

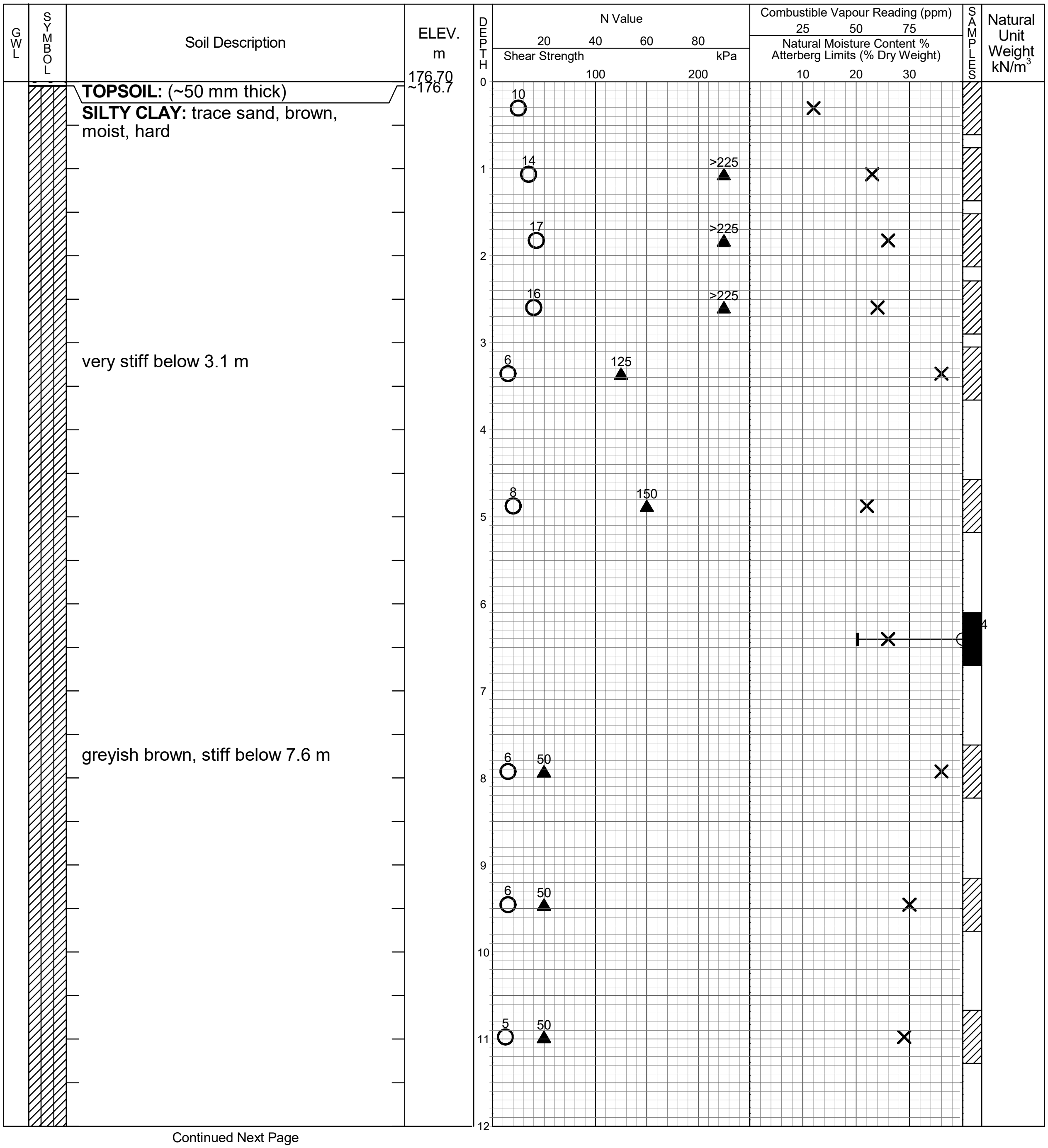
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	9.8

Log of Borehole BH-12

Project No. HAM-00801631-A0
Drawing No. 14
Project: Proposed Subdivision
Sheet No. 1 of 2
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: January 27, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



TOPSOIL: (~50 mm thick)

SILTY CLAY: trace sand, brown, moist, hard

very stiff below 3.1 m

greyish brown, stiff below 7.6 m

Continued Next Page

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Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	12.8

Log of Borehole BH-12

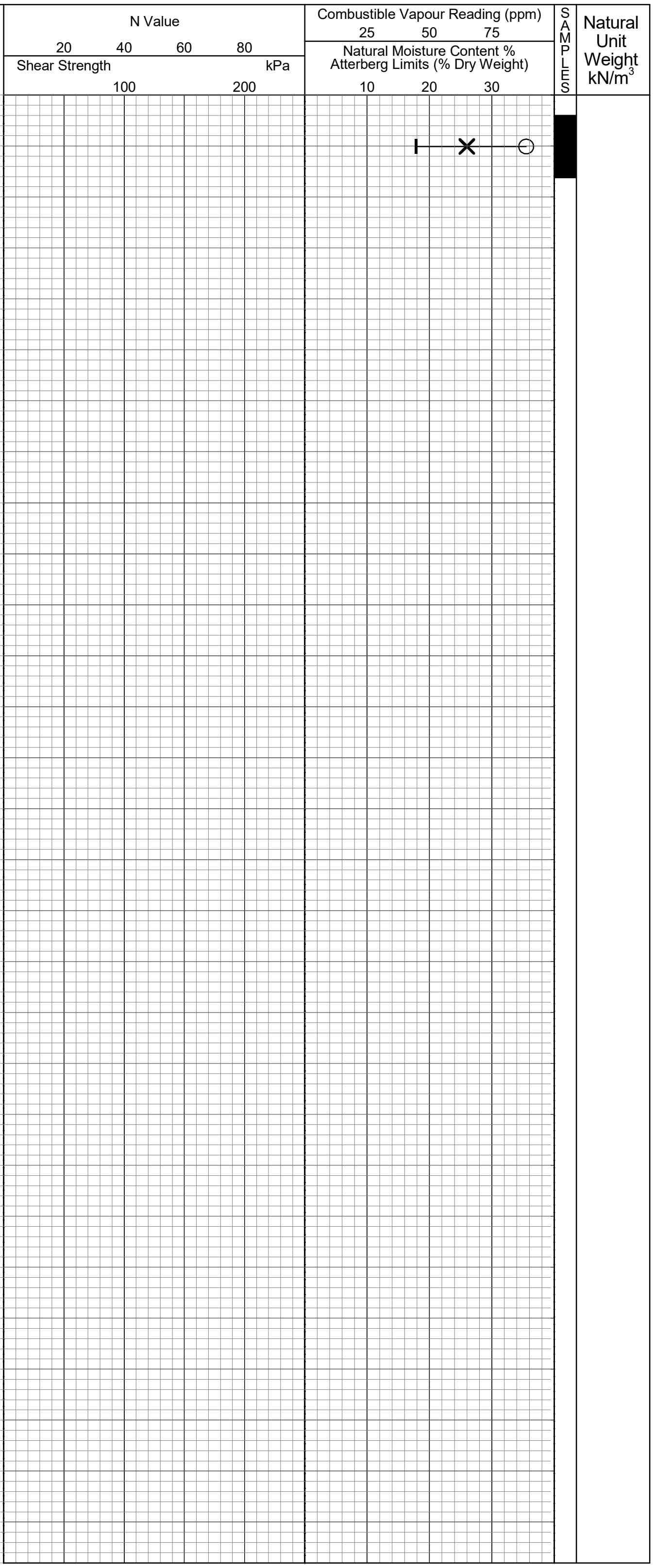
Project No. HAM-00801631-A0

Drawing No. 14

Project: Proposed Subdivision

Sheet No. 2 of 2

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH
			164.70	12
	▨		~163.9	
		Borehole terminated at 12.8 m depth 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.		13



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Telephone: 905.573.4000
Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	12.8

Log of Borehole BH-13

Project No. HAM-00801631-A0

Drawing No. 15

Project: Proposed Subdivision

Sheet No. 1 of 1

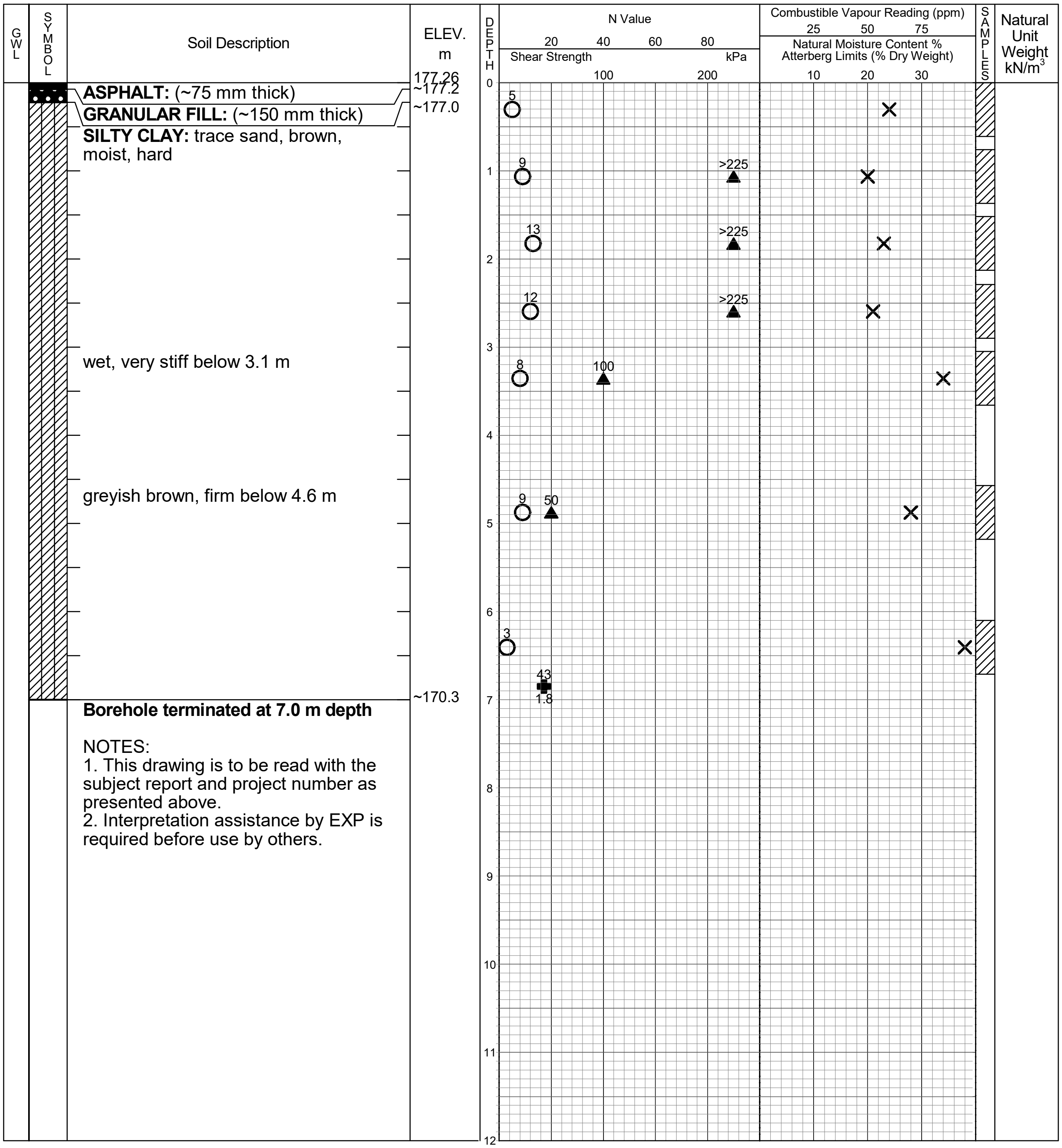
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 16, 2020

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	⊠	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~150 mm thick)

SILTY CLAY: trace sand, brown, moist, hard

wet, very stiff below 3.1 m

greyish brown, firm below 4.6 m

Borehole terminated at 7.0 m depth

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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Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-14

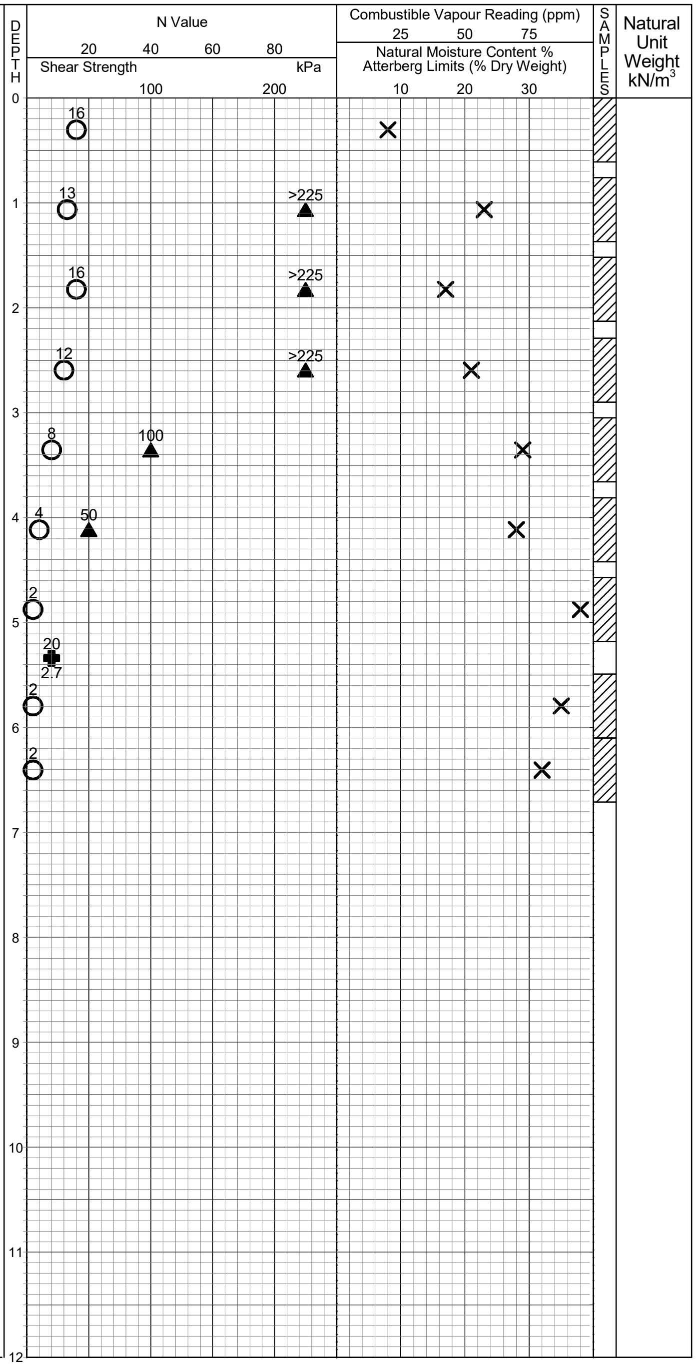
Project No. HAM-00801631-A0
Drawing No. 16
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 14, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
			177.57
		ASPHALT: (~75 mm thick)	~177.5
		GRANULAR FILL: (~200 mm thick)	~177.3
		SILTY CLAY: trace sand, brown, moist, hard	
		very stiff below 3.1 m	
		greyish brown, firm below 3.8 m	
		wet below 4.6 m	
		soft below 5.3 m	
		Borehole terminated at 6.7 m depth	~170.9

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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Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	6.4	6.7

Log of Borehole BH-15

Project No. HAM-00801631-A0 Drawing No. 17

Project: Proposed Subdivision Sheet No. 1 of 1

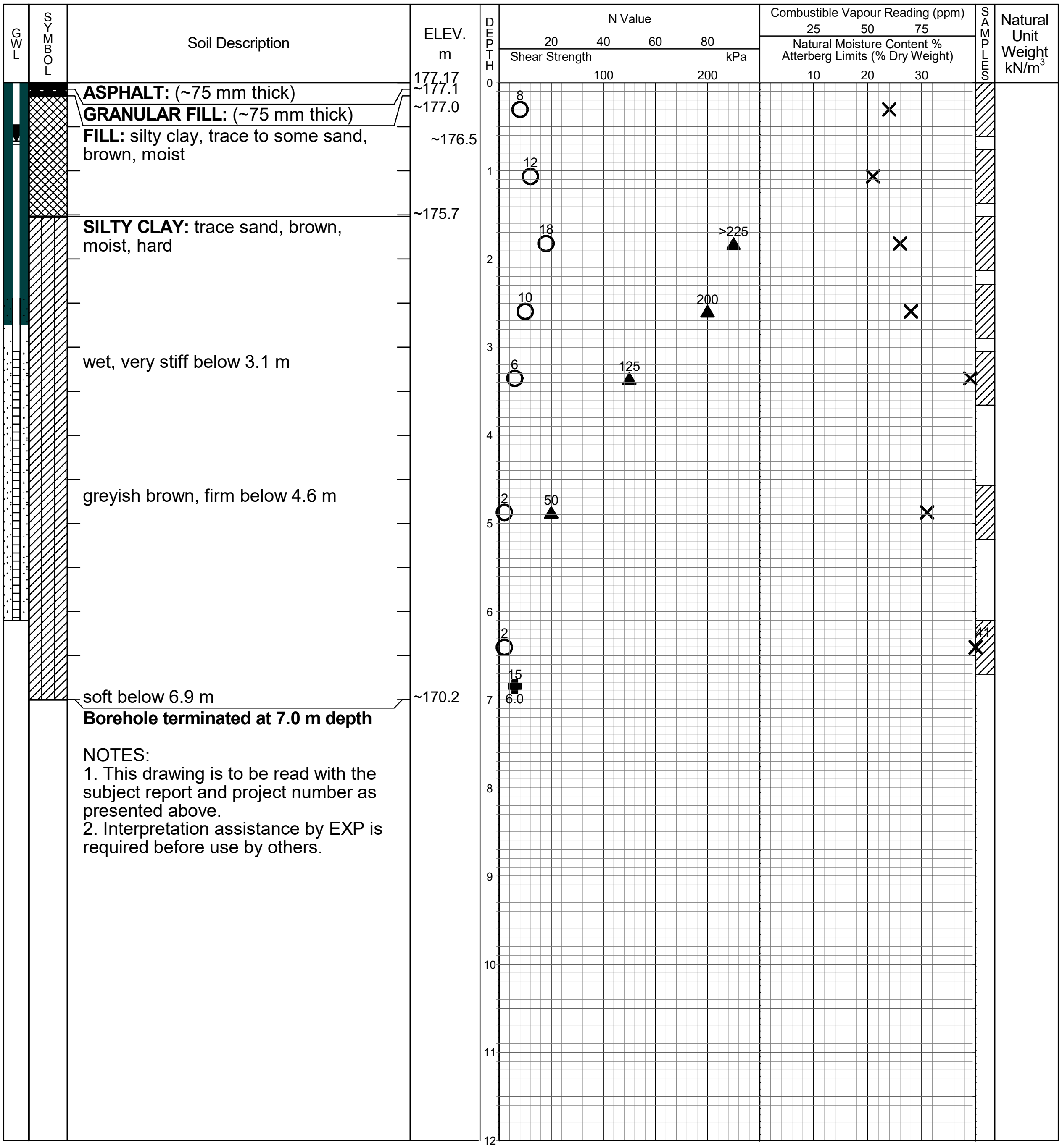
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 13, 2020

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~75 mm thick)

FILL: silty clay, trace to some sand, brown, moist

SILTY CLAY: trace sand, brown, moist, hard

wet, very stiff below 3.1 m

greyish brown, firm below 4.6 m

soft below 6.9 m

Borehole terminated at 7.0 m depth

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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Telephone: 905.573.4000
Facsimile: 905.573.9693

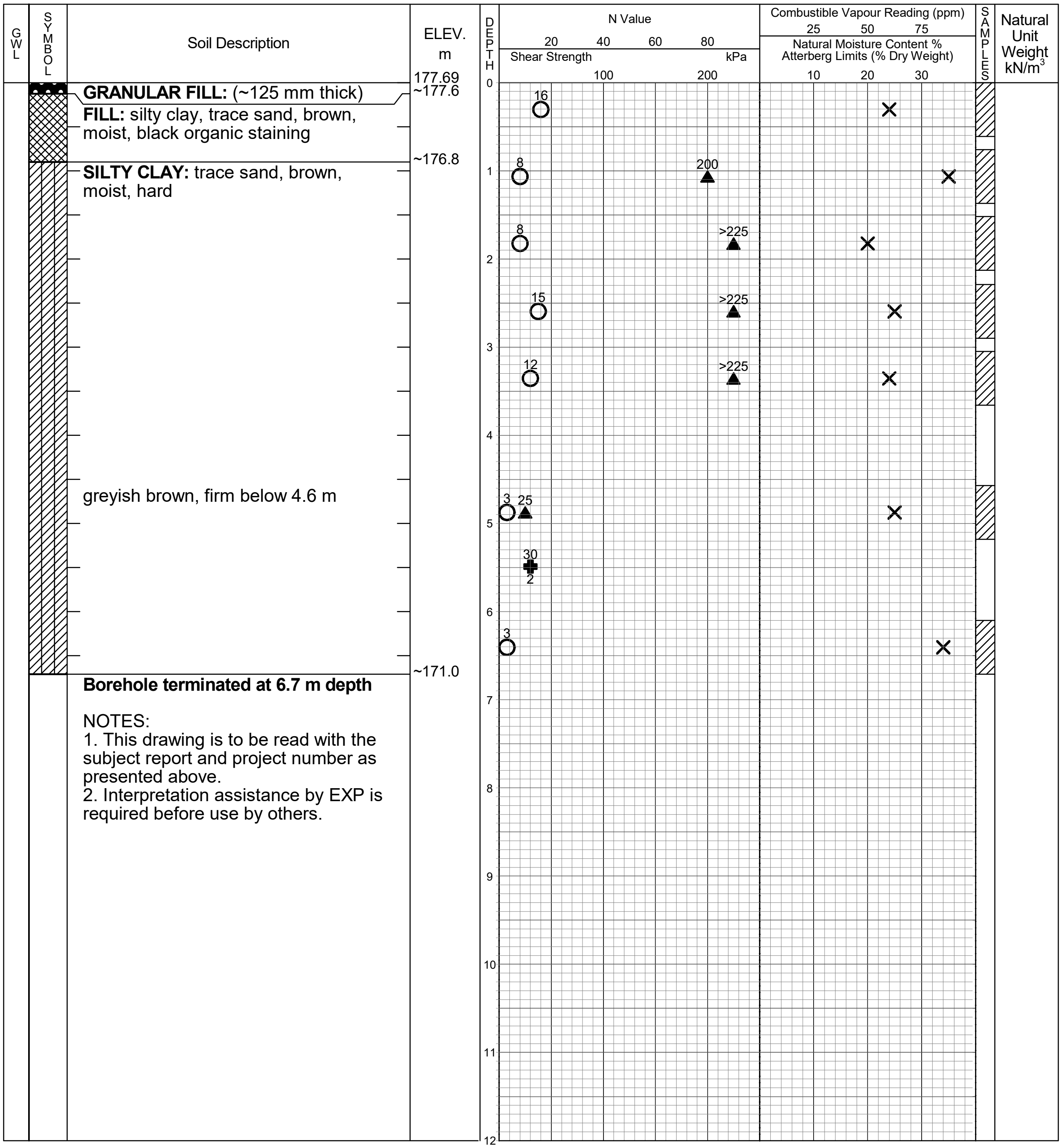
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7
July 27, 2020	0.7	N/A

Log of Borehole BH-16

Project No. HAM-00801631-A0
Drawing No. 18
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 14, 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	▲



Soil Description	ELEV. m
	177.69
GRANULAR FILL: (~125 mm thick)	~177.6
FILL: silty clay, trace sand, brown, moist, black organic staining	~176.8
SILTY CLAY: trace sand, brown, moist, hard greyish brown, firm below 4.6 m	~171.0
Borehole terminated at 6.7 m depth	

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 8/7/20

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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-17

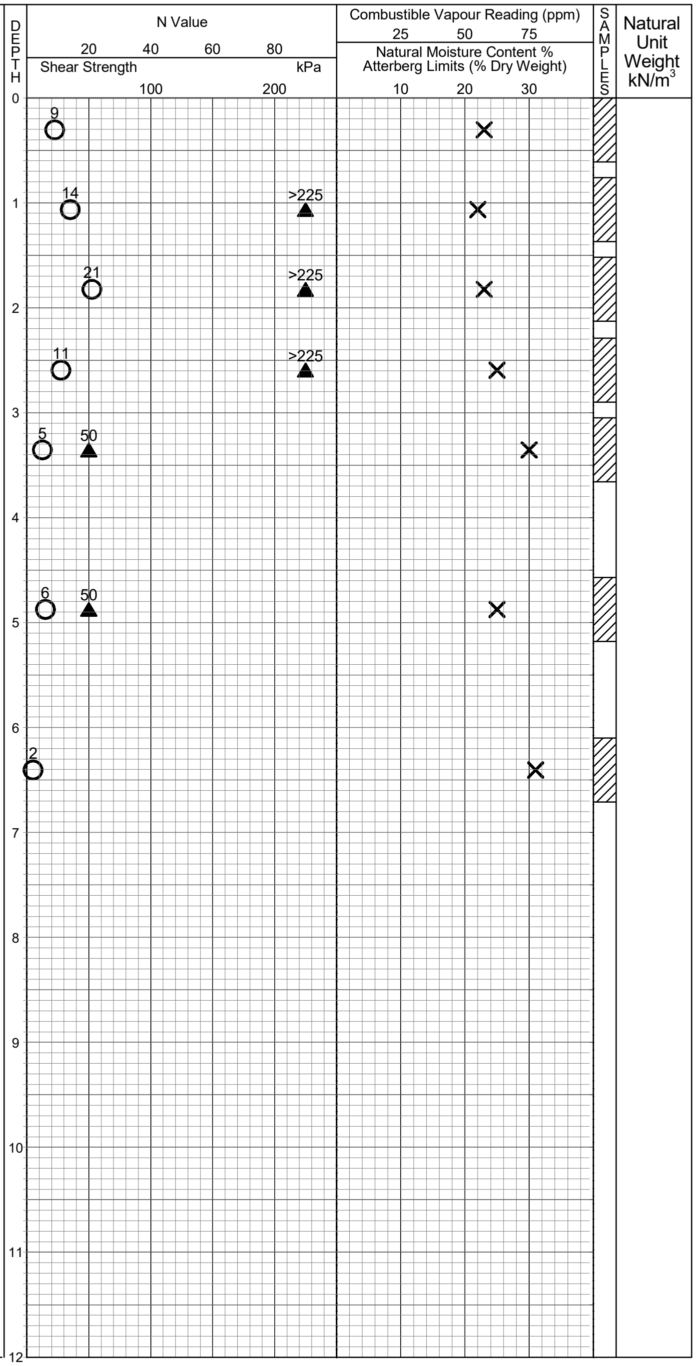
Project No. HAM-00801631-A0
Drawing No. 19
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 13, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
			177.16
		FILL: silty clay, occasional rootlets, brown, moist,	~176.9
		SILTY CLAY: trace sand, brown, moist, hard	
		greyish brown, firm below 3.1 m	
		Borehole terminated at 6.7 m depth	~170.5

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.



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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-18

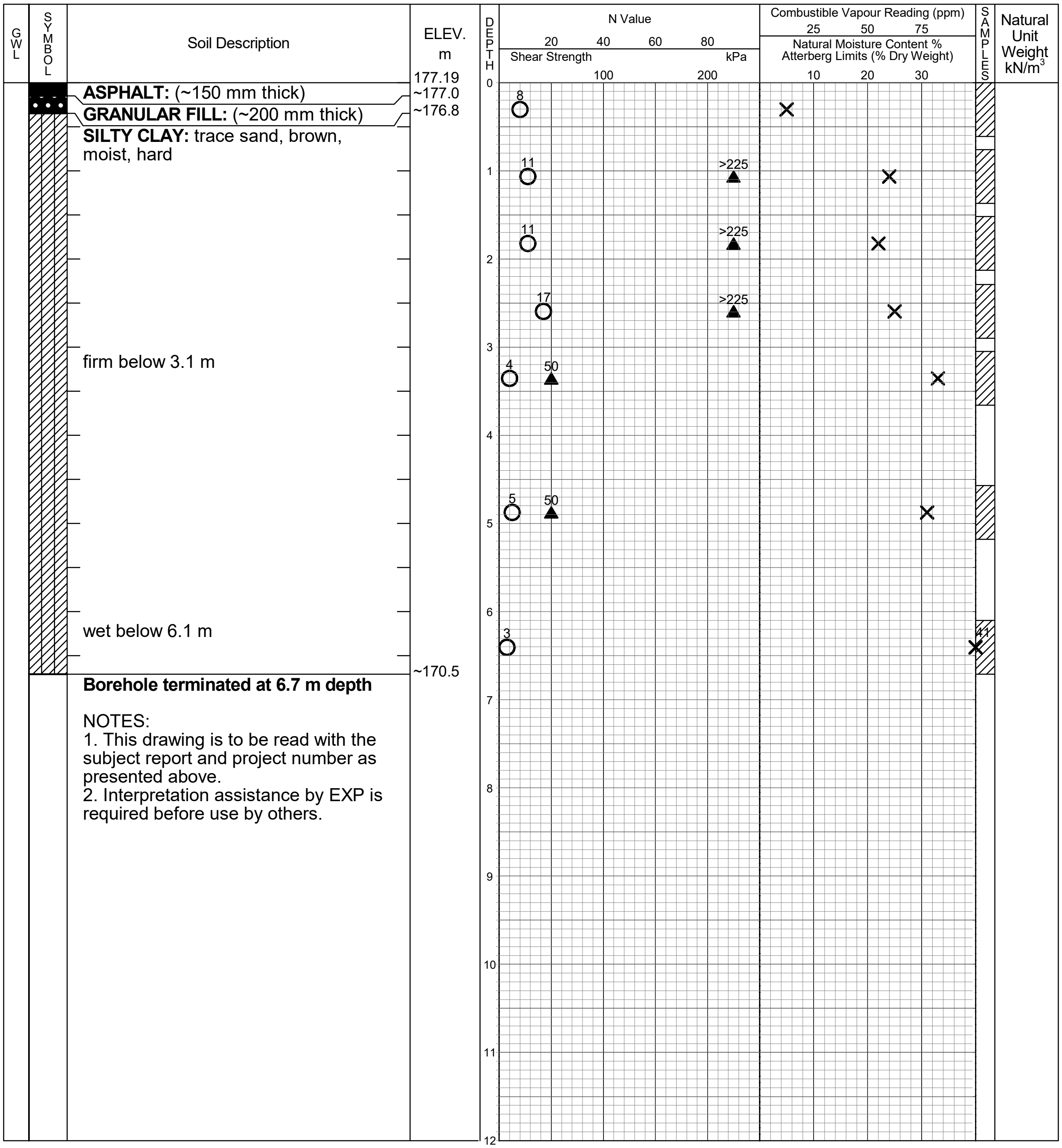
Project No.	HAM-00801631-A0	Drawing No.	20
Project:	Proposed Subdivision	Sheet No.	1 of 1
Location:	555 Canal Bank Street, Welland, ON		

Date Drilled: July 13, 2020

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~150 mm thick)

GRANULAR FILL: (~200 mm thick)

SILTY CLAY: trace sand, brown, moist, hard

firm below 3.1 m

wet below 6.1 m

Borehole terminated at 6.7 m depth

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 8/7/20

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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-19

Project No. HAM-00801631-A0 Drawing No. 21

Project: Proposed Subdivision Sheet No. 1 of 1

Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 13, 2020

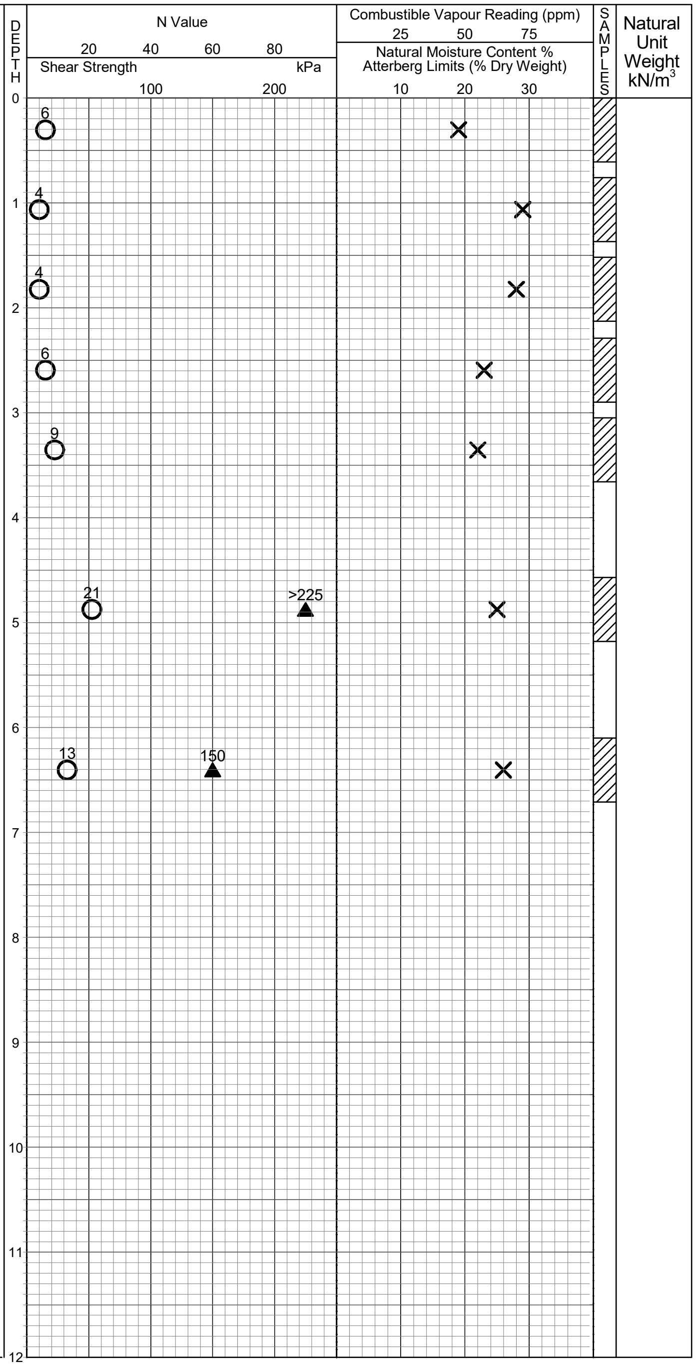
Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	⊠	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
			179.49
		TOPSOIL: (~100 mm thick)	~179.4
		FILL: silty clay, trace to some sand and gravel, brown, moist, black organic staining	
		SILTY CLAY: trace sand, greyish brown, moist, hard very stiff below 6.1 m	~174.9
		Borehole terminated at 6.7 m depth	~172.8

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 8/7/20

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

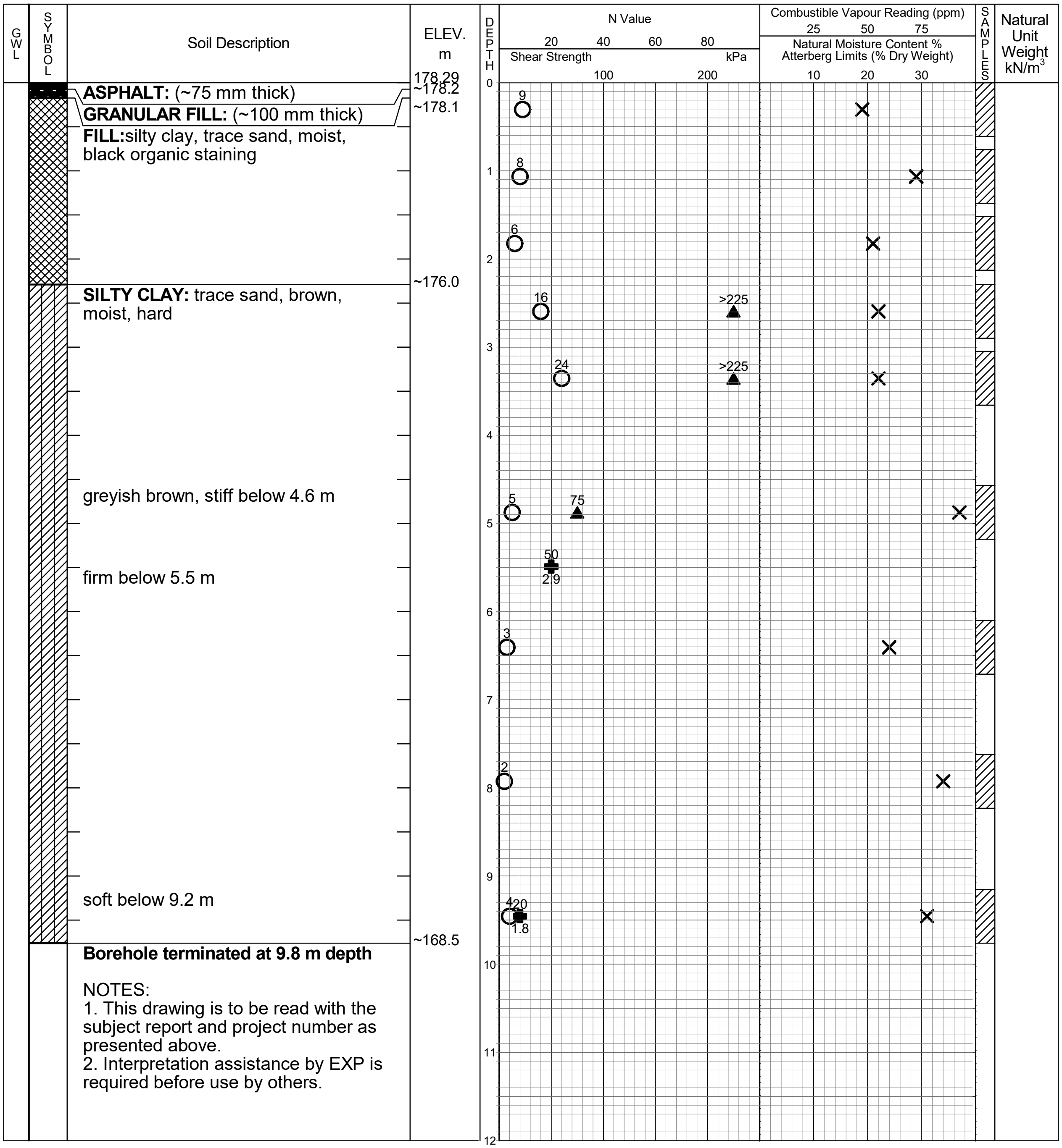
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-20

Project No. HAM-00801631-A0 — Drawing No. 22
Project: Proposed Subdivision — Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 13, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~100 mm thick)

FILL: silty clay, trace sand, moist, black organic staining

SILTY CLAY: trace sand, brown, moist, hard

greyish brown, stiff below 4.6 m

firm below 5.5 m

soft below 9.2 m

Borehole terminated at 9.8 m depth

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 8/7/20

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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	9.8

Log of Borehole BH-21

Project No. HAM-00801631-A0
Drawing No. 23

Project: Proposed Subdivision
Sheet No. 1 of 1

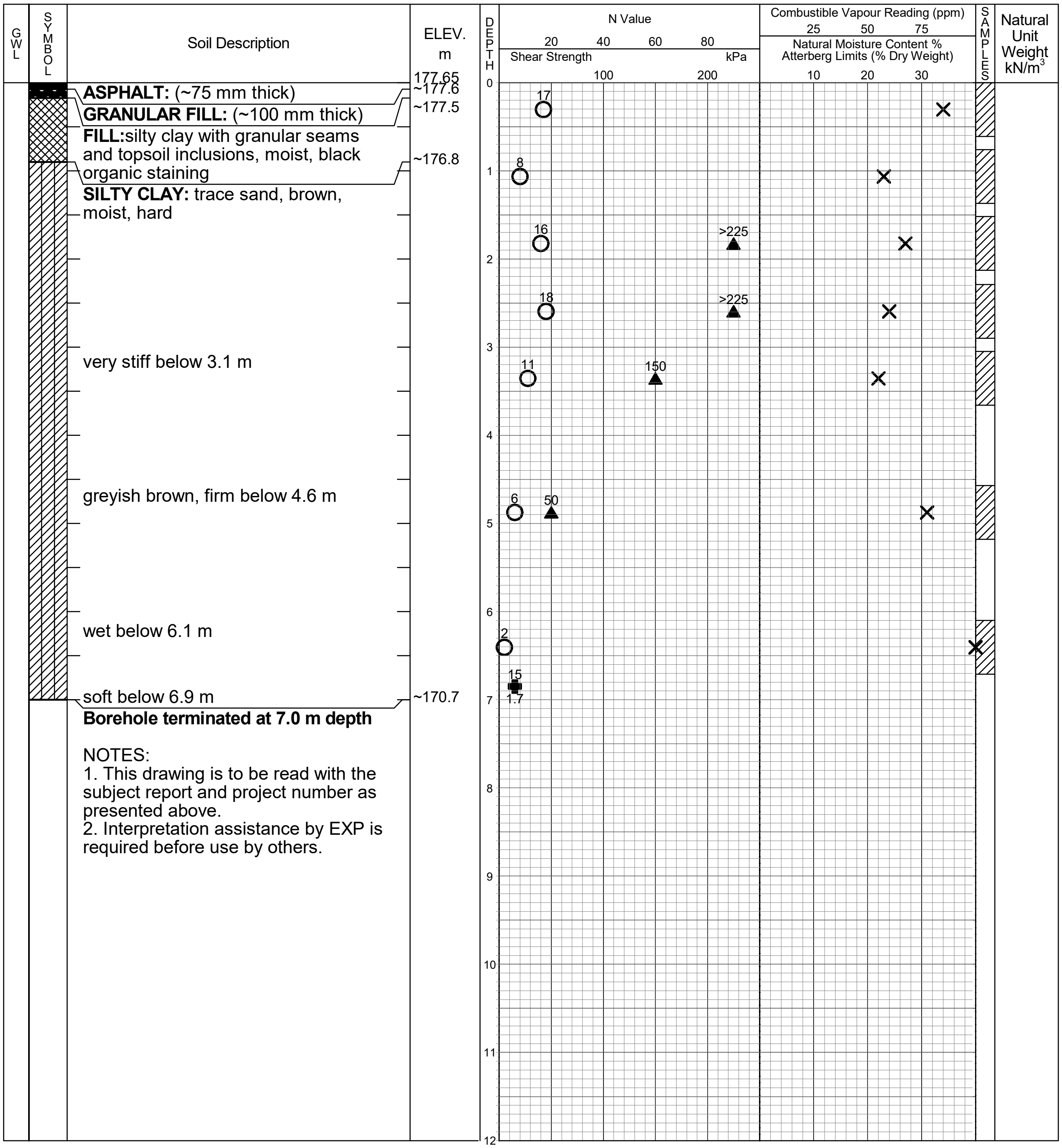
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 13, 2020

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~100 mm thick)

FILL: silty clay with granular seams and topsoil inclusions, moist, black organic staining

SILTY CLAY: trace sand, brown, moist, hard

very stiff below 3.1 m

greyish brown, firm below 4.6 m

wet below 6.1 m

soft below 6.9 m

Borehole terminated at 7.0 m depth

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.

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

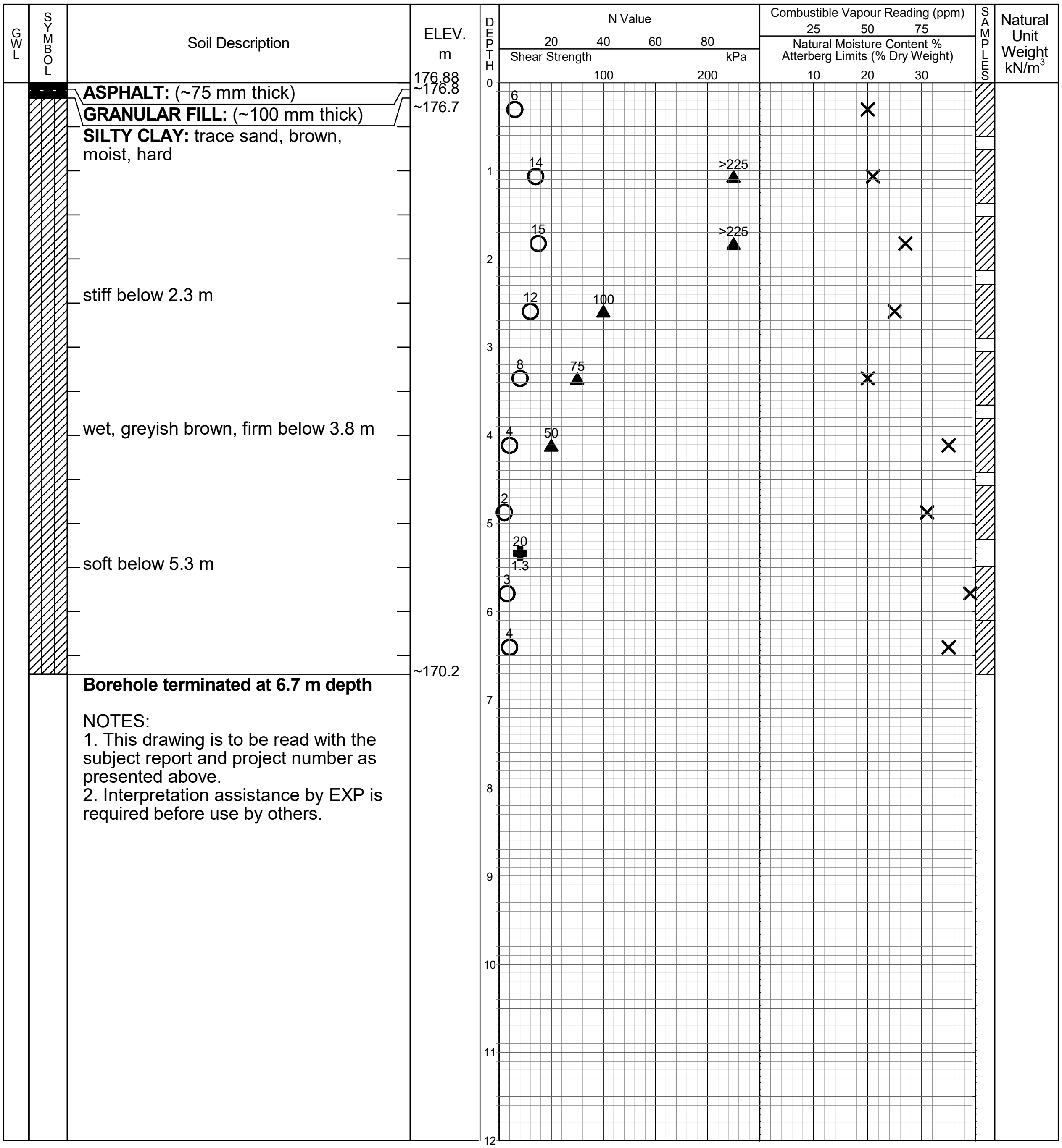
Time	Water Level (m)	Depth to Cave (m)
on completion	6.4	6.7

Log of Borehole BH-22

Project No. HAM-00801631-A0
Drawing No. 24
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 14, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~100 mm thick)

SILTY CLAY: trace sand, brown, moist, hard

stiff below 2.3 m

wet, greyish brown, firm below 3.8 m

soft below 5.3 m

Borehole terminated at 6.7 m depth

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.

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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

LAGWGLJFHAM-EXP BHLOGS.GPJ NEW.GDT 8/7/20

Log of Borehole BH-23

Project No. HAM-00801631-A0
Drawing No. 25

Project: Proposed Subdivision
Sheet No. 1 of 1

Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 14, 2020

Drill Type: D-50 Track Mount. Solid Stem.

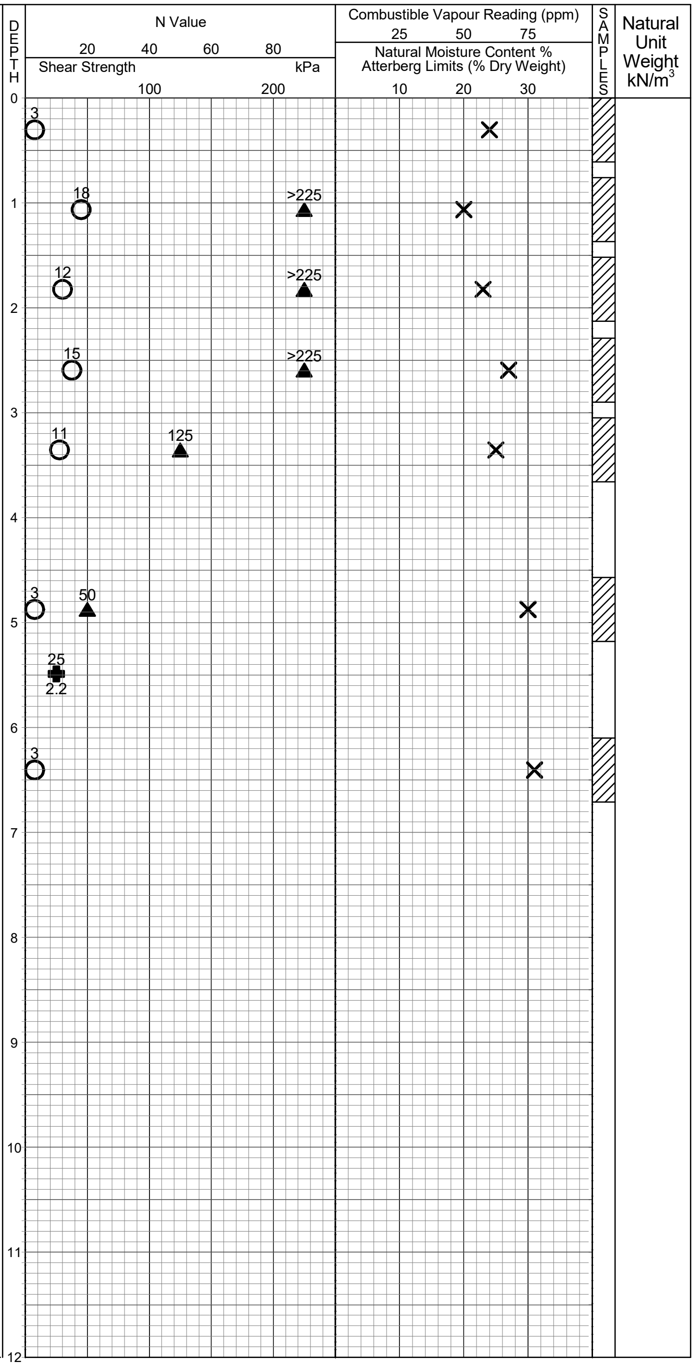
Datum: Geodetic

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	▲

GWL	SYMBOL	Soil Description	ELEV. m
			176.99
	▩	**FILL:** silty clay, brown, moist, black organic staining	
	▨	**SILTY CLAY:** trace sand, brown, moist, hard	~176.2
	▨	very stiff below 3.1 m	
	▨	greyish brown, firm below 4.6 m	
		Borehole terminated at 6.7 m depth	~170.3

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.



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

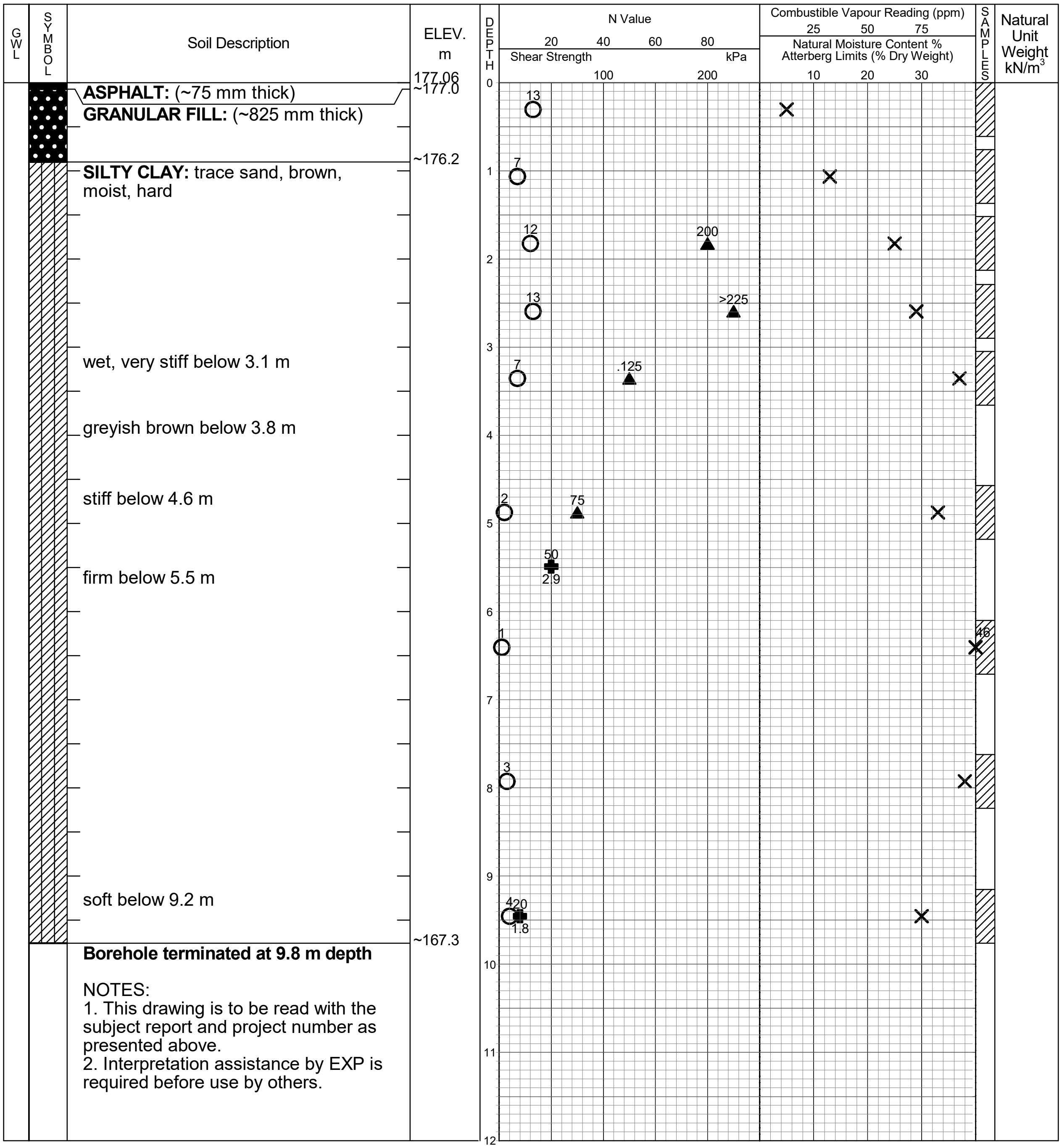
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-24

Project No. HAM-00801631-A0
Drawing No. 26
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 14, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ASPHALT: (~75 mm thick)

GRANULAR FILL: (~825 mm thick)

SILTY CLAY: trace sand, brown, moist, hard

wet, very stiff below 3.1 m

greyish brown below 3.8 m

stiff below 4.6 m

firm below 5.5 m

soft below 9.2 m

Borehole terminated at 9.8 m depth

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.

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

Time	Water Level (m)	Depth to Cave (m)
on completion	7.9	9.8

Log of Borehole BH-25

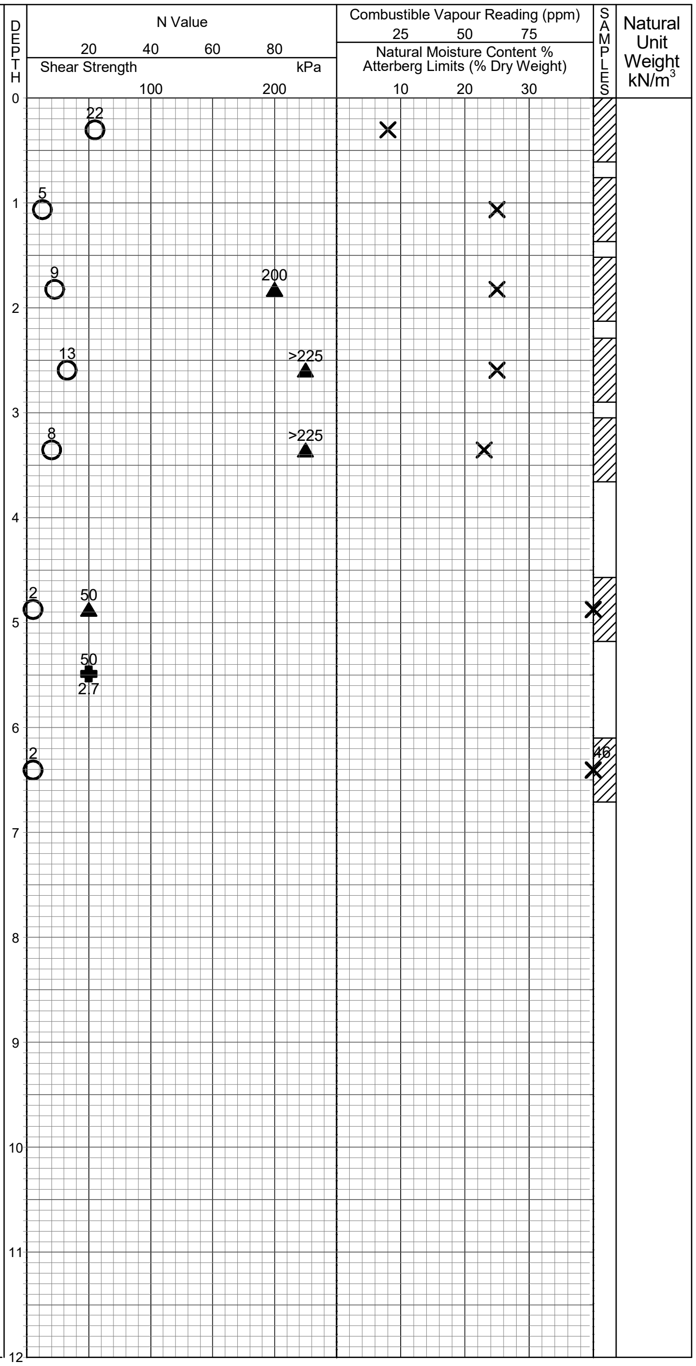
Project No. HAM-00801631-A0
Drawing No. 27
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 16, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	□
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	ELEV. m	Soil Description
	177.23	
	~177.1	**TOPSOIL:** (~150 mm thick)
		FILL: silty clay, trace sand and gravel, brown, moist
	~176.2	**SILTY CLAY:** trace sand, brown, moist, hard
		greyish brown below 3.1 m
		wet, stiff below 4.6 m
	~170.5	**Borehole terminated at 6.7 m depth**

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.



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

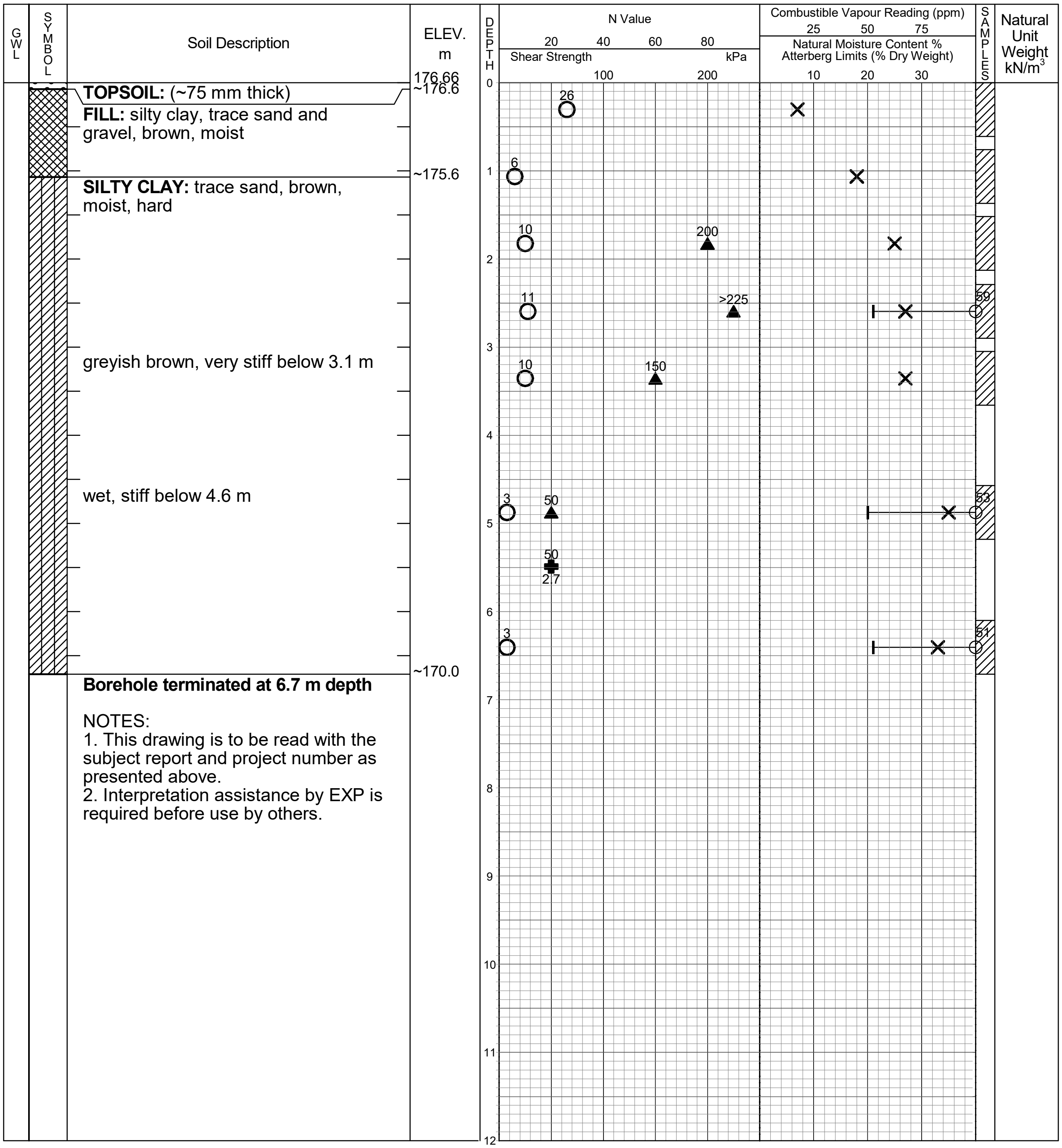
Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-26

Project No. HAM-00801631-A0
Drawing No. 28
Project: Proposed Subdivision
Sheet No. 1 of 1
Location: 555 Canal Bank Street, Welland, ON

Date Drilled: July 16, 2020
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	□
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



176.66
~176.6
TOPSOIL: (~75 mm thick)

FILL: silty clay, trace sand and gravel, brown, moist

~175.6
SILTY CLAY: trace sand, brown, moist, hard

greyish brown, very stiff below 3.1 m

wet, stiff below 4.6 m

~170.0
Borehole terminated at 6.7 m depth

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.

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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7

Log of Borehole BH-27

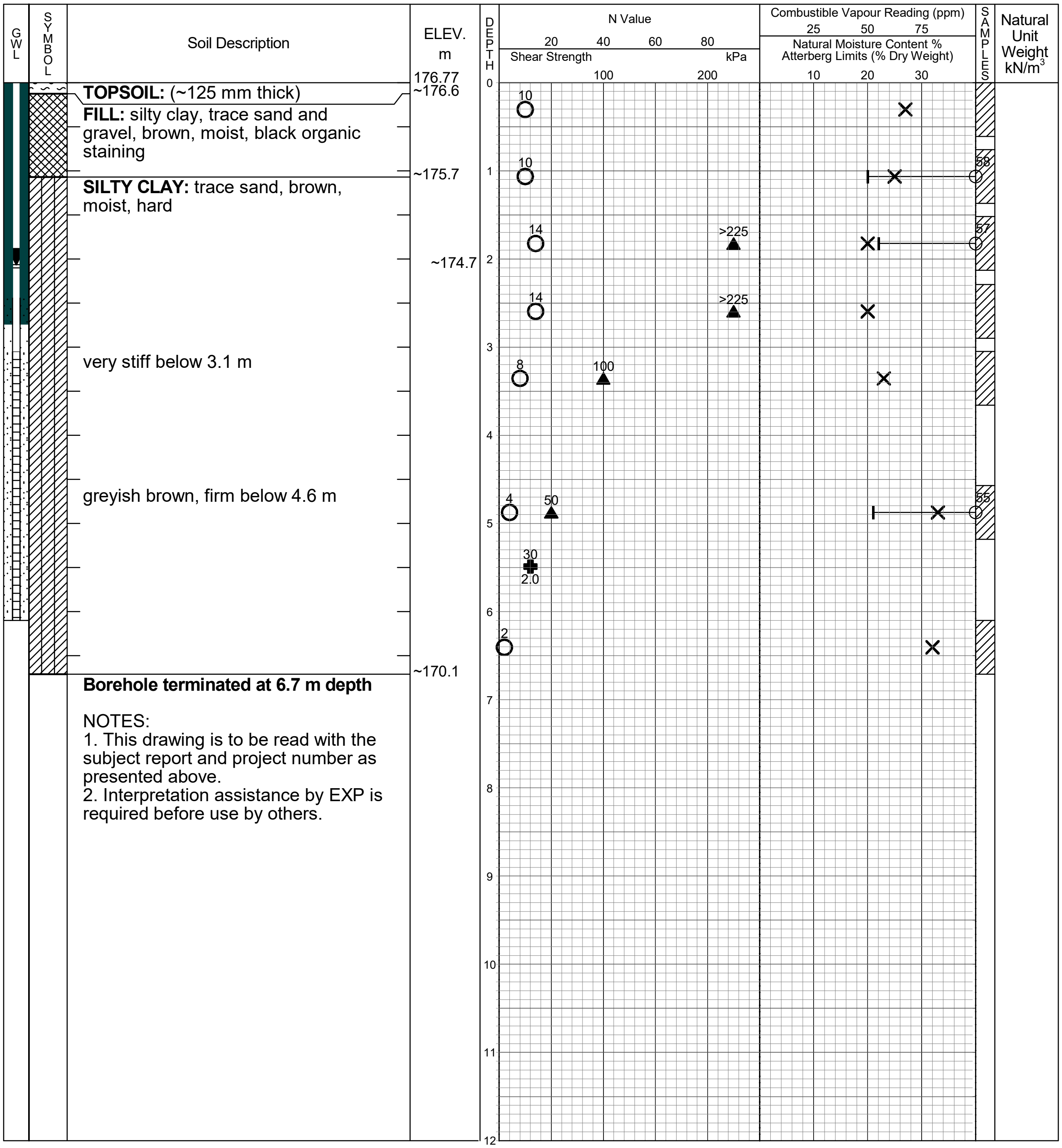
Project No.	HAM-00801631-A0	Drawing No.	29
Project:	Proposed Subdivision	Sheet No.	1 of 1
Location:	555 Canal Bank Street, Welland, ON		

Date Drilled: July 14, 2020

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



TOPSOIL: (~125 mm thick)

FILL: silty clay, trace sand and gravel, brown, moist, black organic staining

SILTY CLAY: trace sand, brown, moist, hard

very stiff below 3.1 m

greyish brown, firm below 4.6 m

Borehole terminated at 6.7 m depth

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 8/10/20

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

Time	Water Level (m)	Depth to Cave (m)
on completion	no free water	6.7
July 27, 2020	2.1	N/A

GENERAL REQUIREMENTS FOR ENGINEERED FILL

1. The area must be stripped of all topsoil, fill, organic stained or disturbed native material and proof-rolled. Soft/loose spots must be dug out. The stripped native subgrade must be examined and approved by an EXP Services Inc. (EXP) engineer prior to placement of engineered fill.

2. The structure/building footprint, including basements, garages, etc. must be defined by offset stakes that remain in place until the foundations and service connections are all constructed. Confirmation that the foundations are within the pad, service lines are in place, and that the grade conforms to drawings, must be obtained by the owner in writing from the surveyor and EXP. Without this confirmation, no responsibility for the performance of the structure can be accepted by EXP.

3. The approved engineered fill must be compacted to 100% Standard Proctor Maximum Dry Density throughout. Granular fill conforming to OPSS Granular B is preferred.

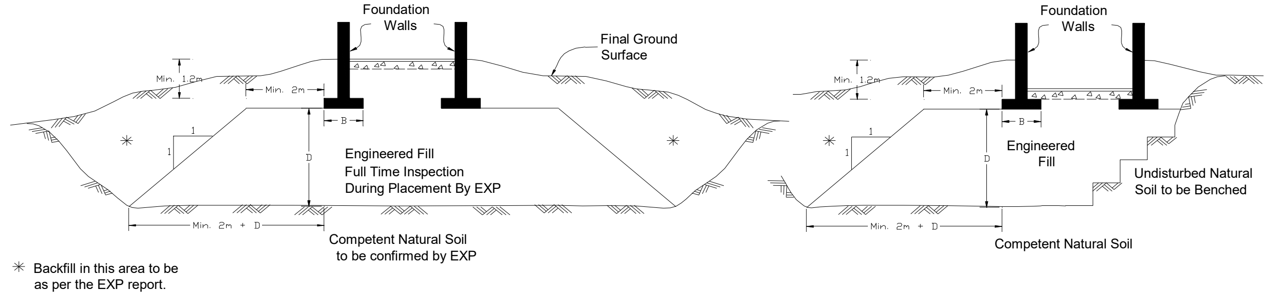
4. Full-time geotechnical inspection by EXP during placement of engineered fill is required.

5. The fill must be placed such that the specified geometry is achieved. Refer to sketches below for minimum requirements. Take careful note that the projection of the compacted pad beyond the footing at footing level is a minimum of 2 m. The base of the compacted pad extends 2 m plus the depth of excavation beyond the edge of the footing.

6. A minimum footing width of 500 mm (20 inches) is suggested and footings must be provided with nominal steel reinforcement.

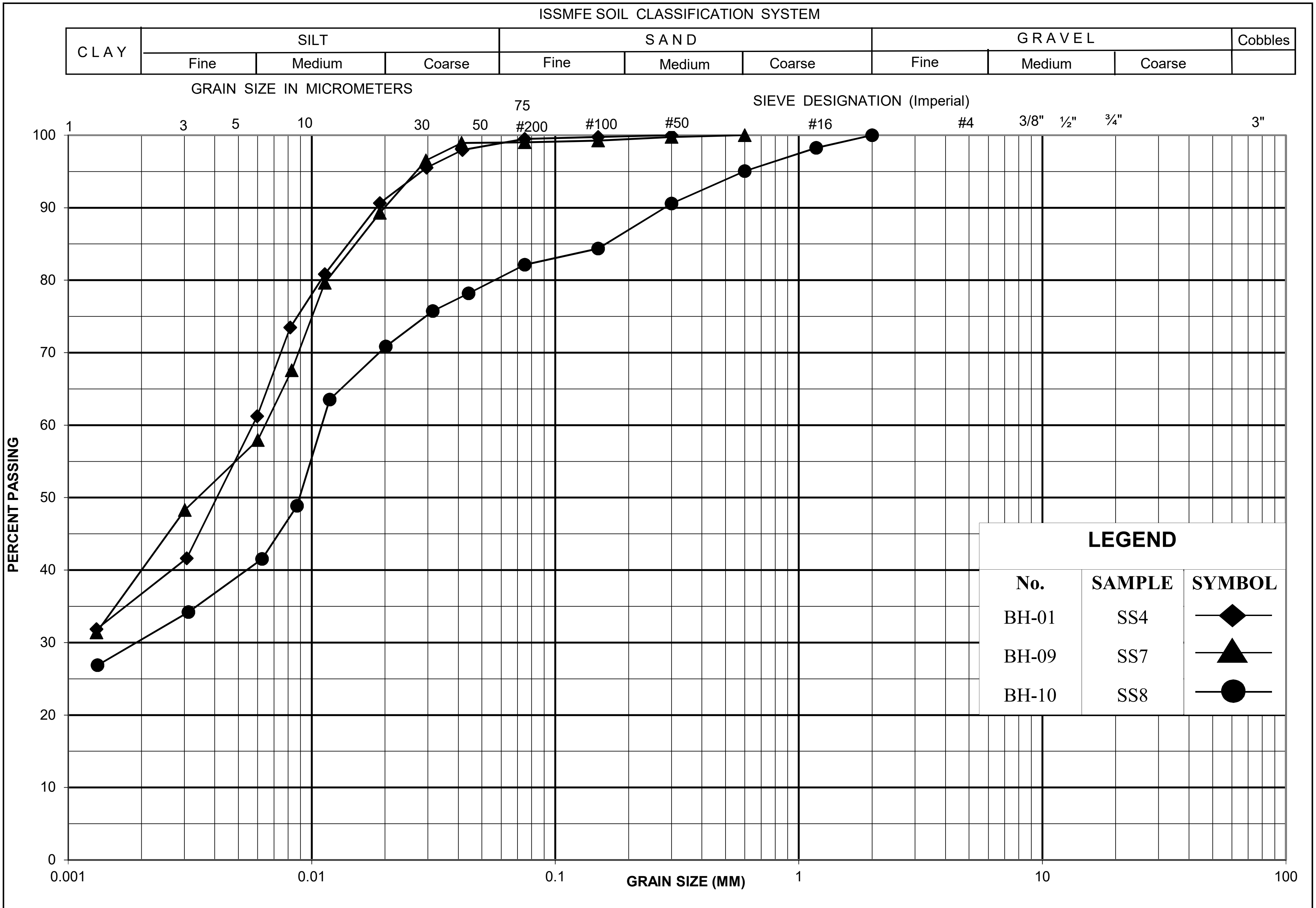
7. All excavations must be done in accordance with the Ontario Occupational Health and Safety Act.

8. These guidelines are to be read in conjunction with the EXP report attached. Foundations built on engineered fill constructed in accordance with the above requirements and the attached EXP report may support a geotechnical resistance of 150 kPa at Serviceability Limit States (SLS) and 225 kPa at Ultimate Limit States (ULS).



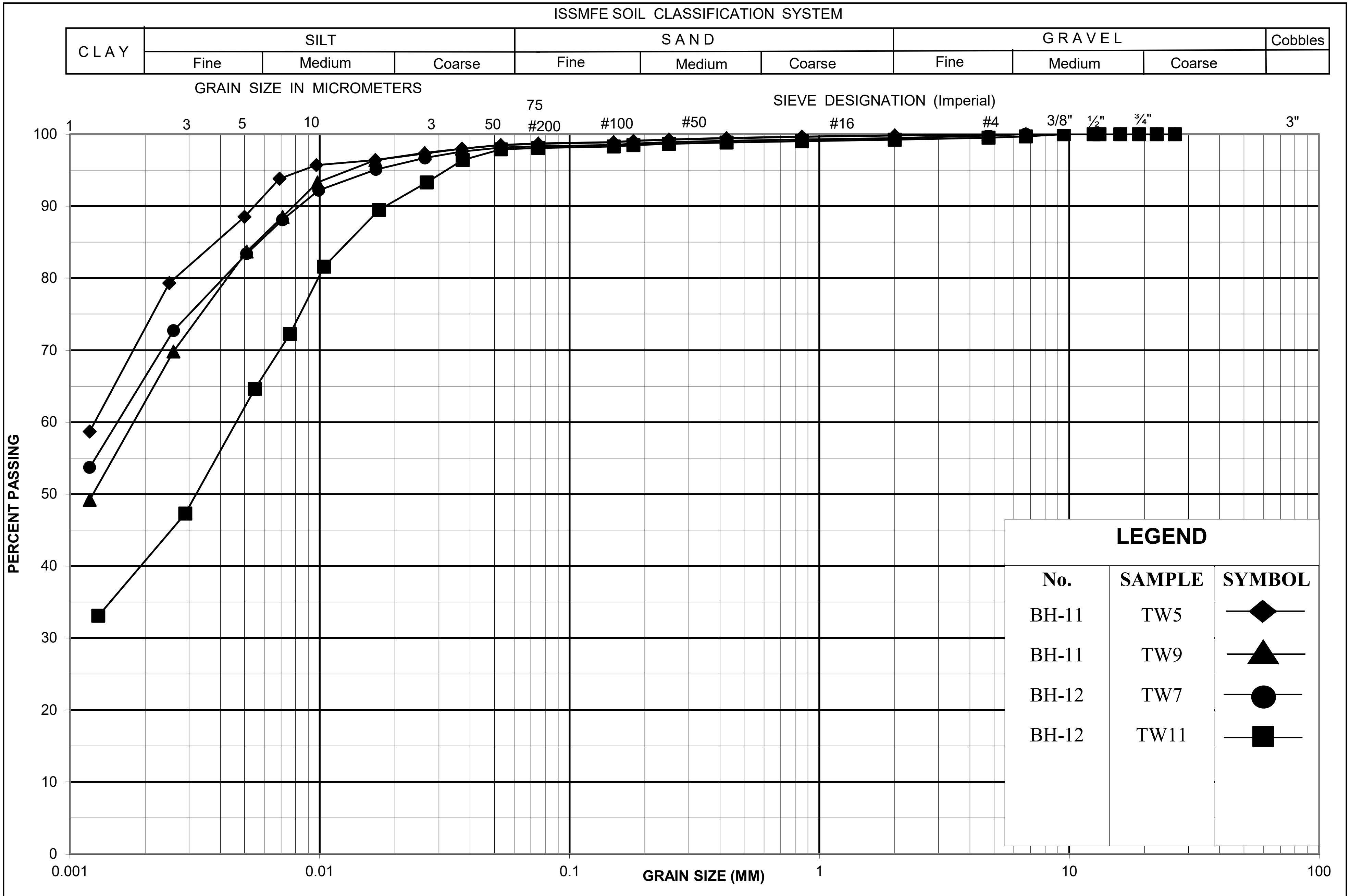
Appendix B

Laboratory Testing



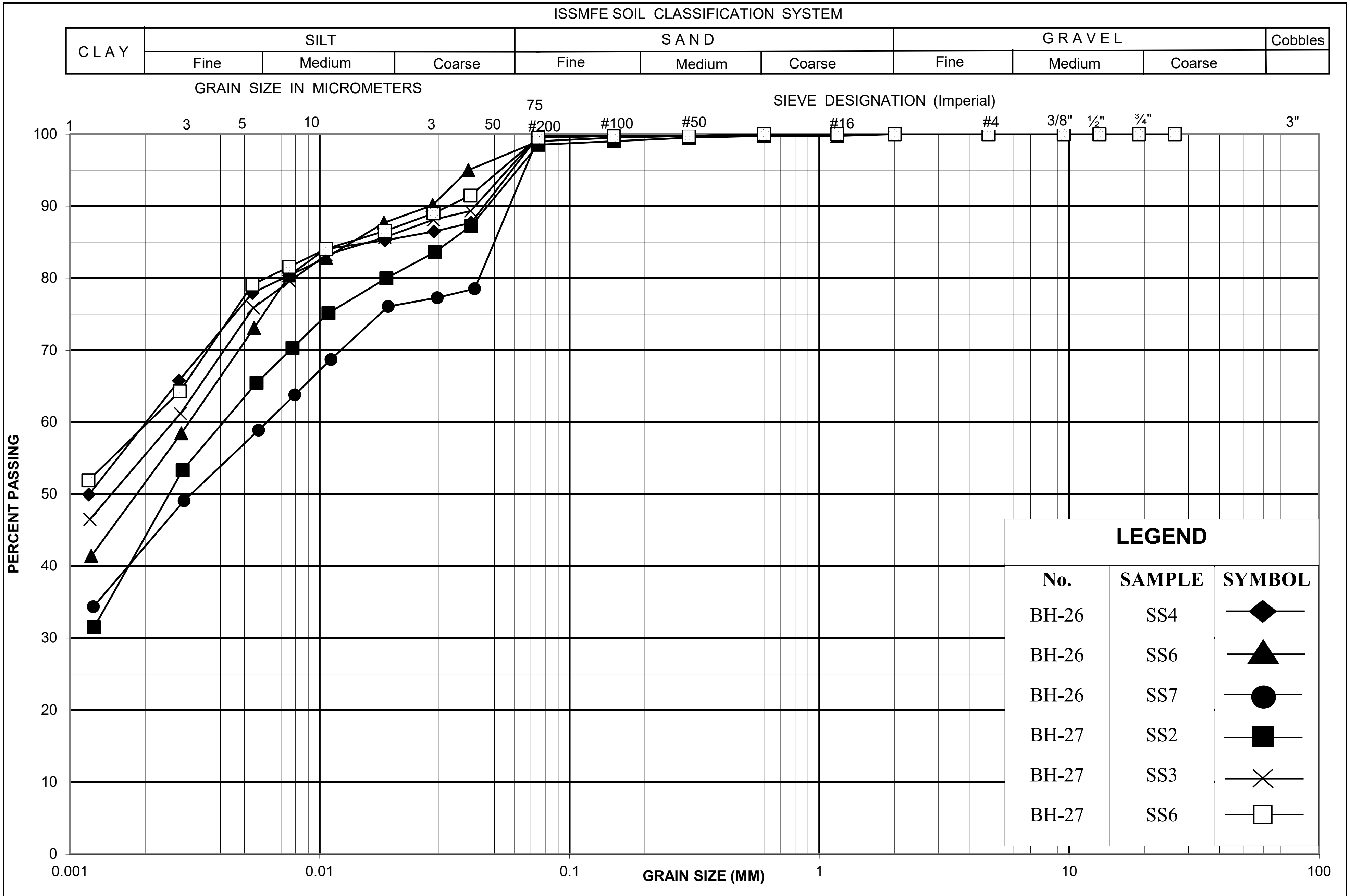
GRAIN SIZE DISTRIBUTION

DRAWING NO.: B1
PROJECT NO.: HAM 00801631-A0
DATE: March 2019



GRAIN SIZE DISTRIBUTION

DRAWING NO.:	B2
PROJECT NO.:	HAM-00801631-I0
DATE:	August 2020



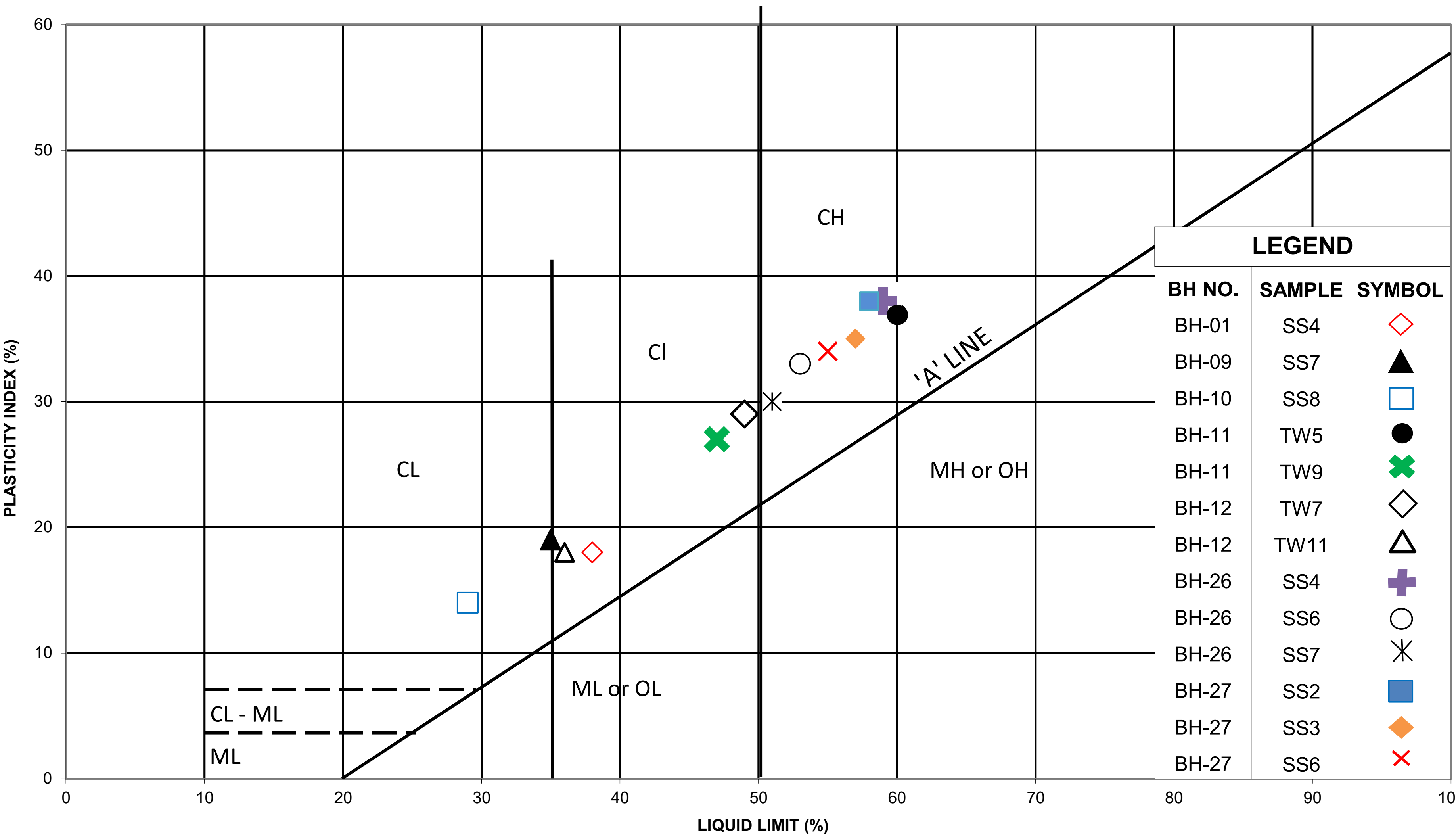
exp.

GRAIN SIZE DISTRIBUTION

DRAWING NO.: B3
PROJECT NO.: HAM-00801631-I0
DATE: August 2020

PROPOSED DAIN CITY WEST DEVELOPMENT

555 Canal Bank Street, Welland, ON



LEGEND

BH NO.	SAMPLE	SYMBOL
BH-01	SS4	◇
BH-09	SS7	▲
BH-10	SS8	□
BH-11	TW5	●
BH-11	TW9	✖
BH-12	TW7	◇
BH-12	TW11	△
BH-26	SS4	✚
BH-26	SS6	○
BH-26	SS7	✳
BH-27	SS2	■
BH-27	SS3	◆
BH-27	SS6	✕

PLASTICITY CHART

SILTY CLAY (CL-CH)

DRAWING NO.: B4

PROJECT NO.: HAM-00801631-I0

DATE: August 2020

Appendix C

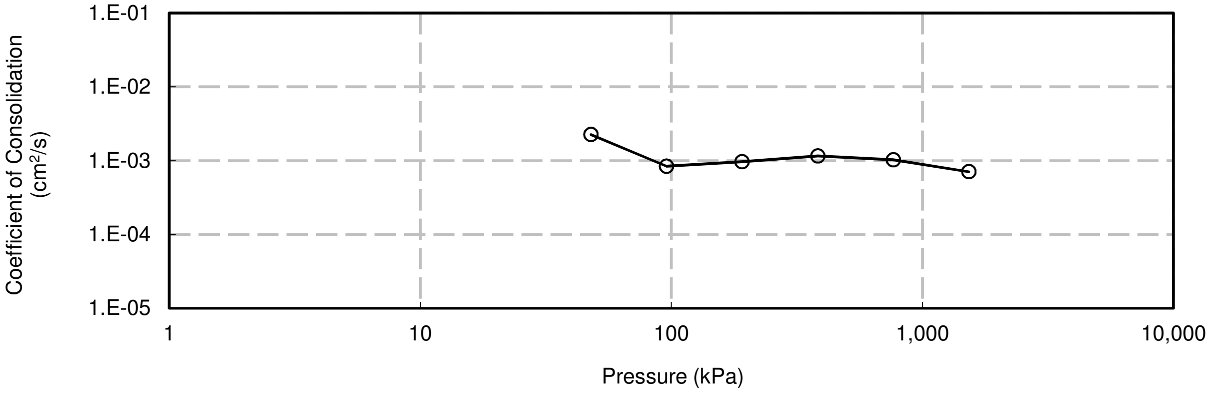
Settlement Analysis

CONSOLIDATION TEST

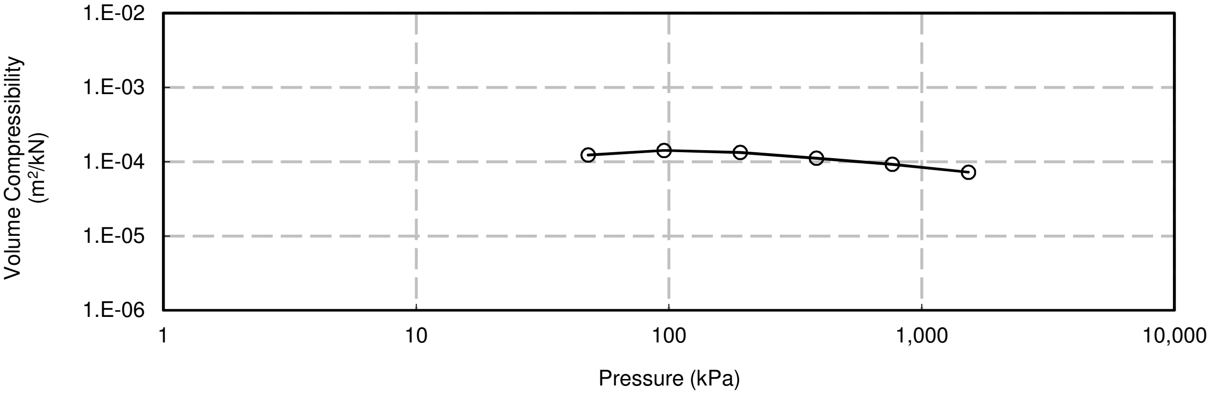
Figure No. 3

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 101, TW 5 - Silty Clay, trace sand

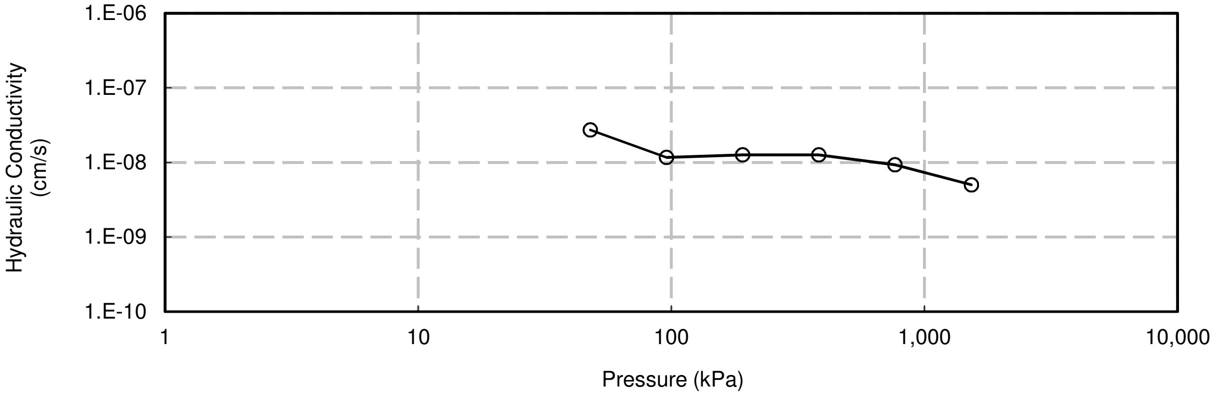
Cv vs Pressure



mv vs Pressure



k vs Pressure



U:\Project\HAM-00801631-E0 555 Canal Bank St, Welland\Consolidation Results_HAM-00801631-E0_R1.xlsx

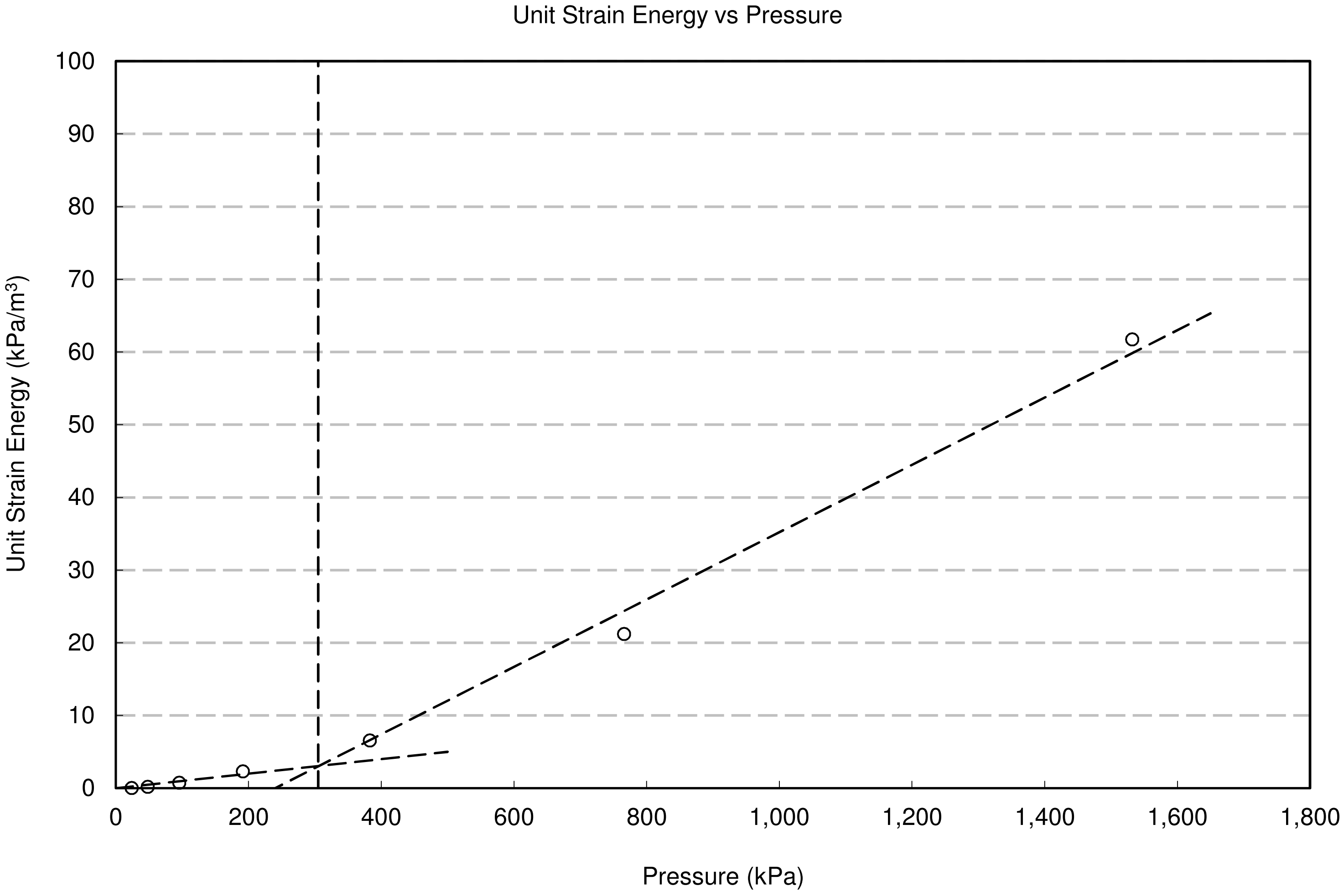
Project No. : HAM-00801631-E0
Date : July 2020

exp.

Prepared By : HW
Checked By : ML

U:\Project\HAM-00801631-E0 555 Canal Bank St, Welland\Consolidation Results_HAM-00801631-E0_R1.xlsx

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 101, TW 5 - Silty Clay, trace sand



Pc = 305 kPa

Project No. : HAM-00801631-E0
Date : July 2020

Prepared By : HW
Checked By : ML

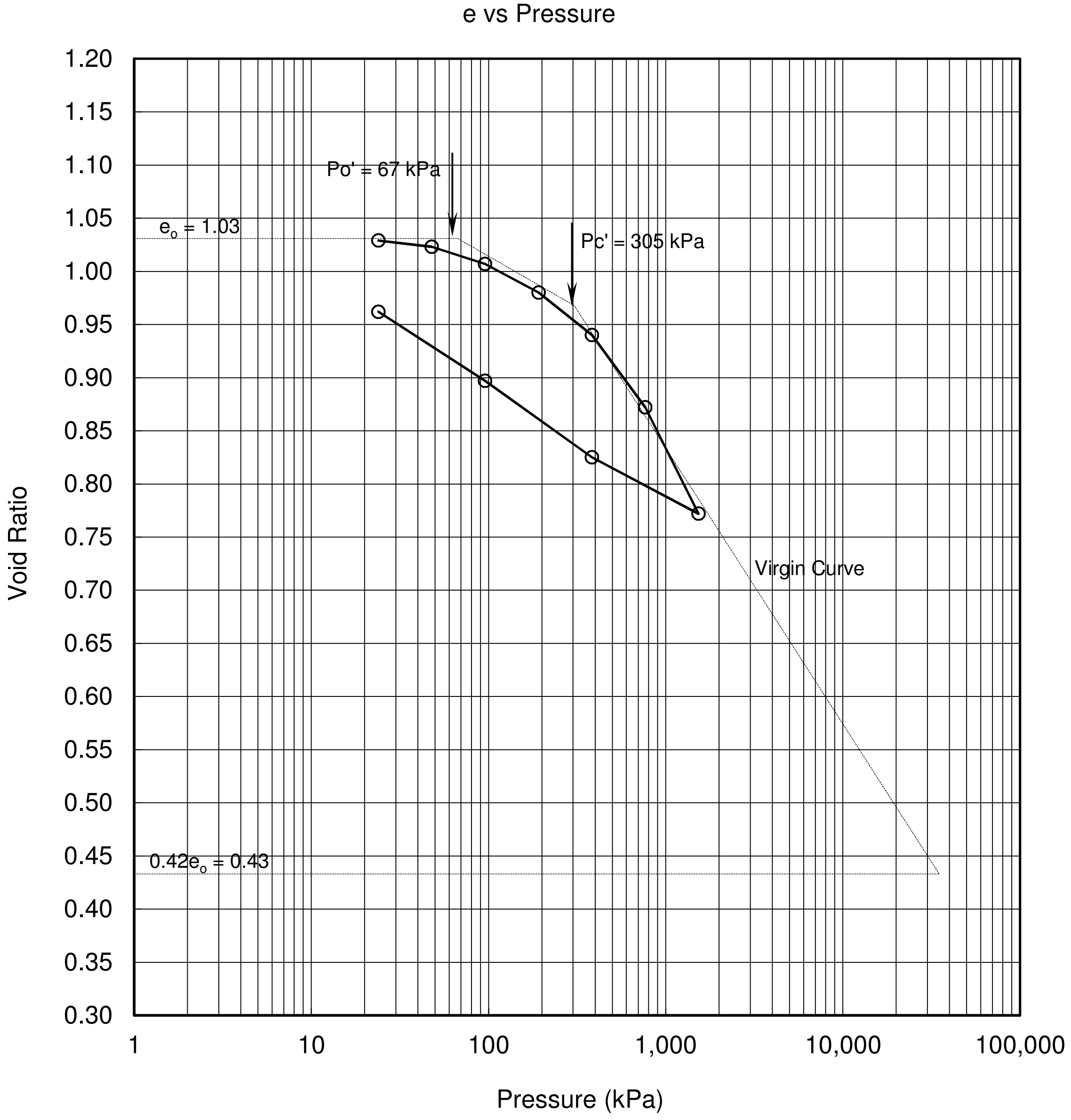
CONSOLIDATION TEST

Figure No. 4

CONSOLIDATION TEST

Figure No. 5

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 101, TW 5 - Silty Clay, trace sand



Soil Type : 1

e_o =	1.03	ω_L =	60%	Po' =	67 kPa
ω =	37%	ω_P =	23%	Pc' =	305 kPa
γ =	18.6 kN/m^3	PI =	37%	Cc =	0.26
Gs =	2.78			Cr =	0.096

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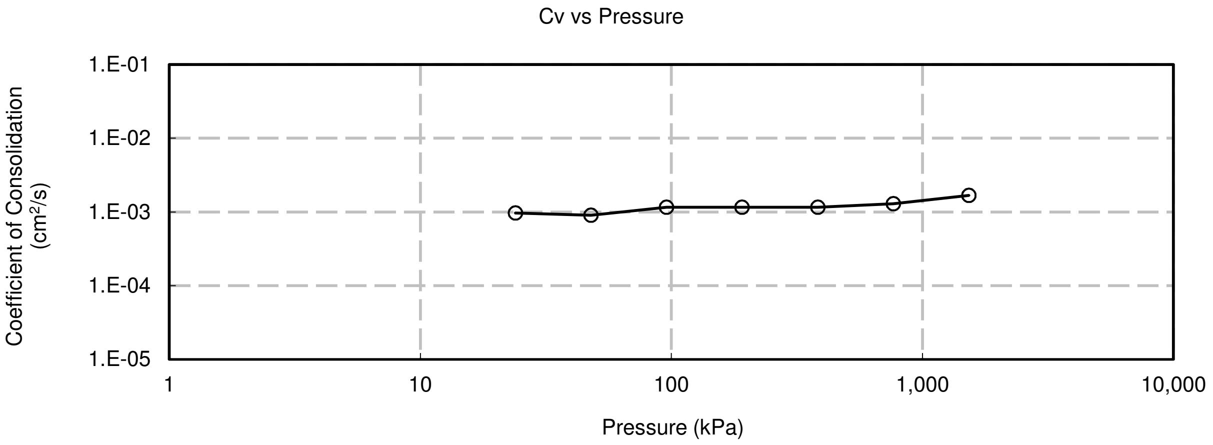
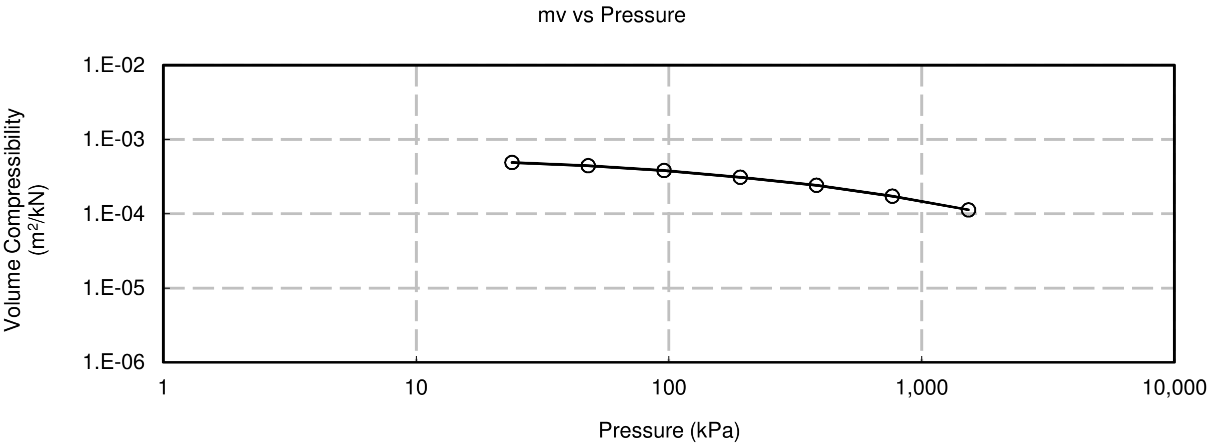
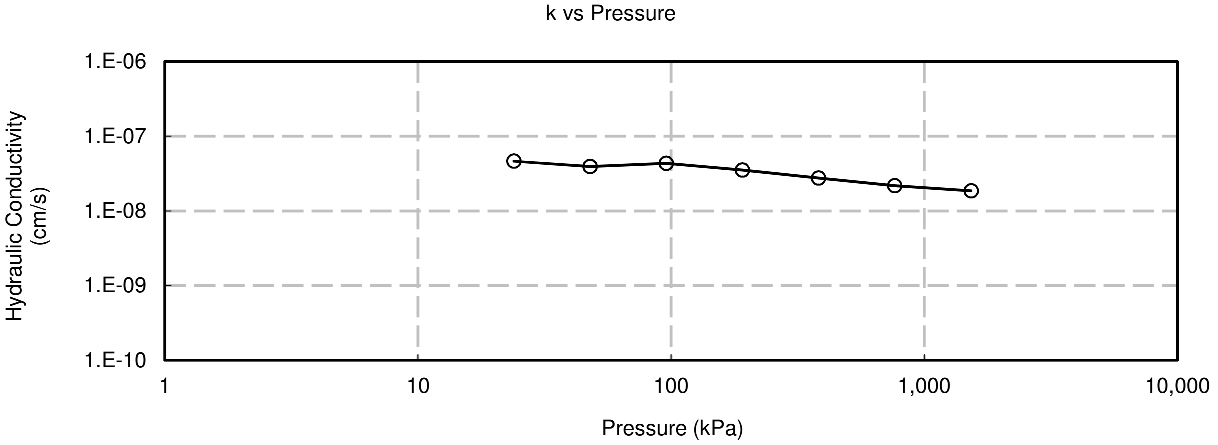
CONSOLIDATION TEST

Figure No. 6

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 101, TW 9 - Silty Clay, trace sand







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Prepared By : HW
Checked By : ML

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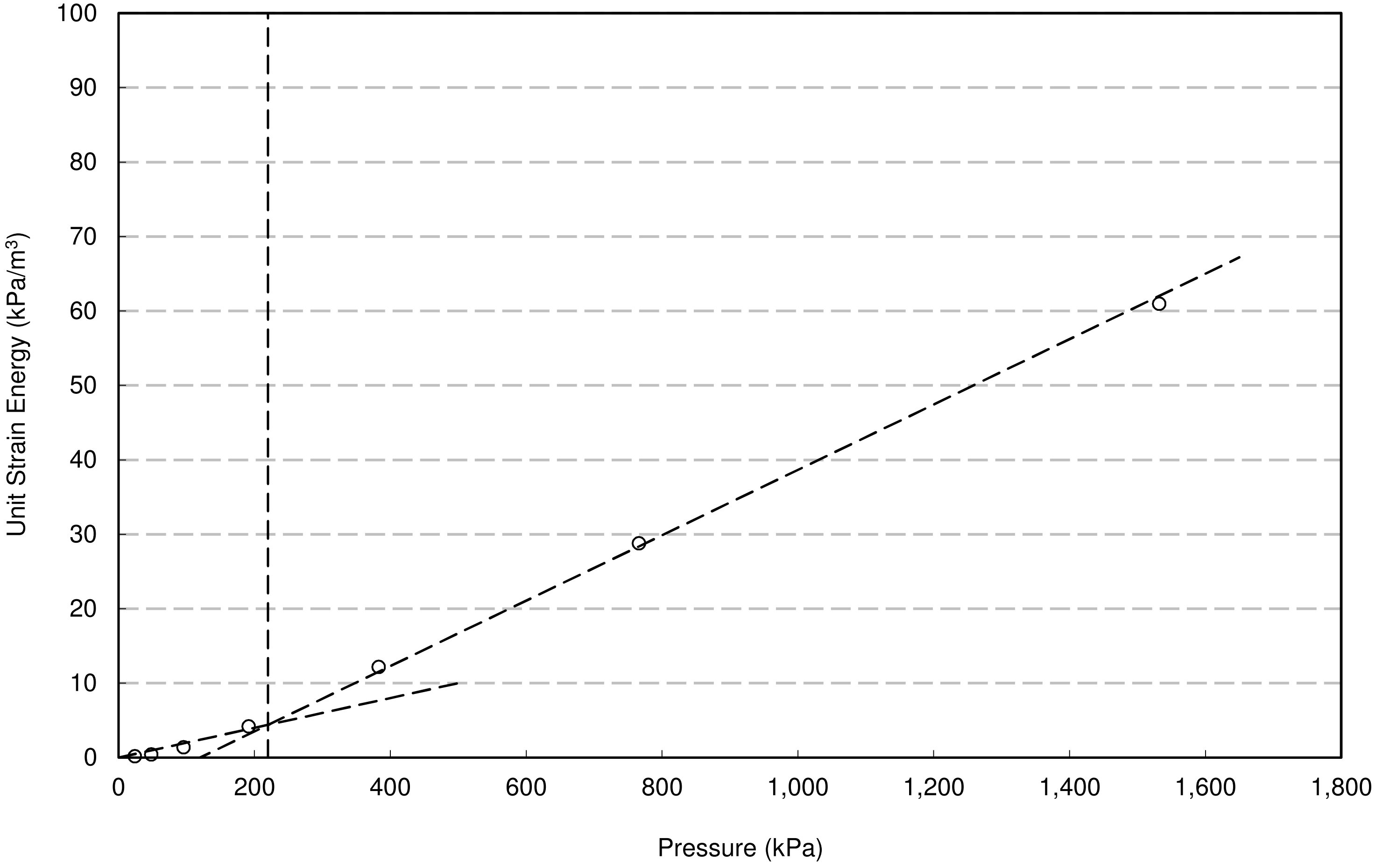
Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON

BH 101, TW 9 - Silty Clay, trace sand

Unit Strain Energy vs Pressure



Pc = 220 kPa

Prepared By : HW

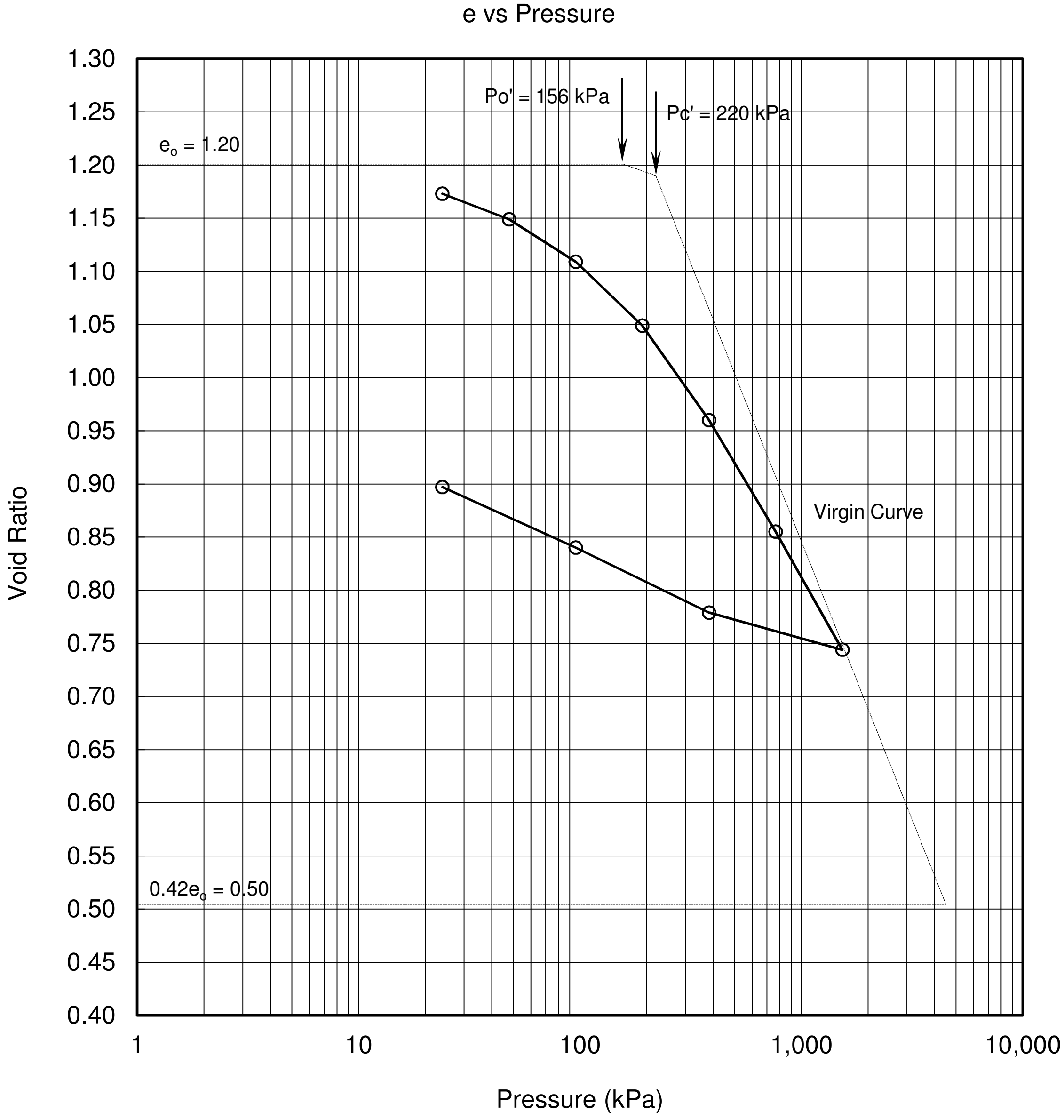
Checked By : ML

CONSOLIDATION TEST

Figure No. 7

CONSOLIDATION TEST	**Figure No. 8**

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 101, TW 9 - Silty Clay, trace sand



Soil Type : Silty Clay, trace sand

e_o =	1.20	ω_L =	47%	Po' =	156 kPa		
ω =	42%	ω_P =	20%	Pc' =	220 kPa		
γ =	18.0 kN/m^3	PI =	27%	Cc =	0.52		
Gs =	2.78			Cr =	0.074		

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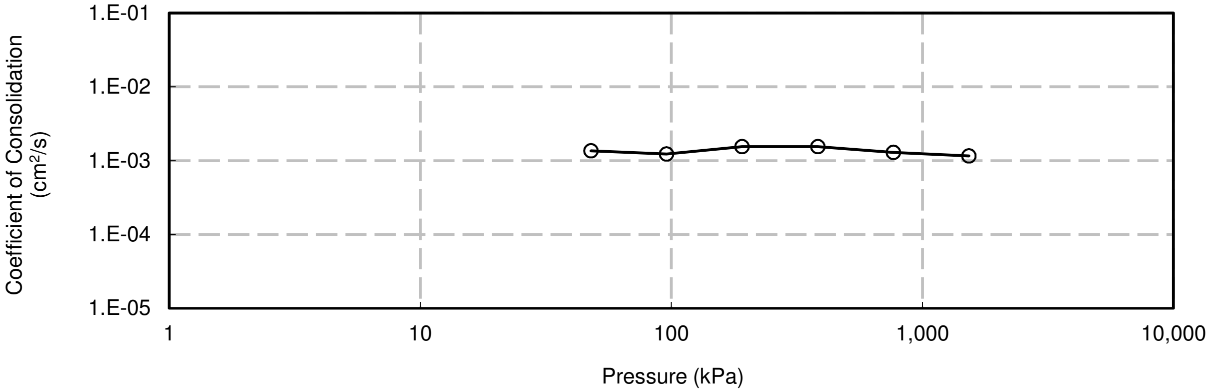
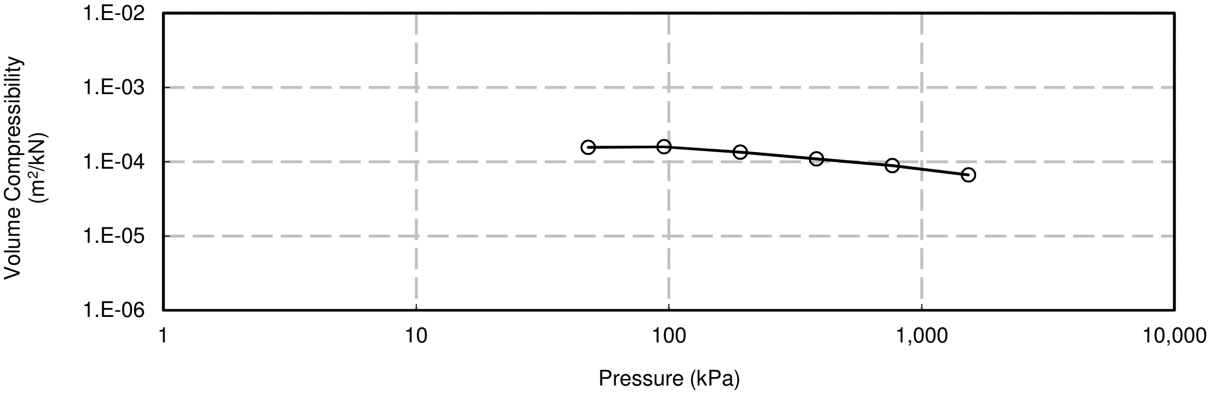
CONSOLIDATION TEST	Figure No. 9

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 102, TW 7 - Silty Clay, trace sand

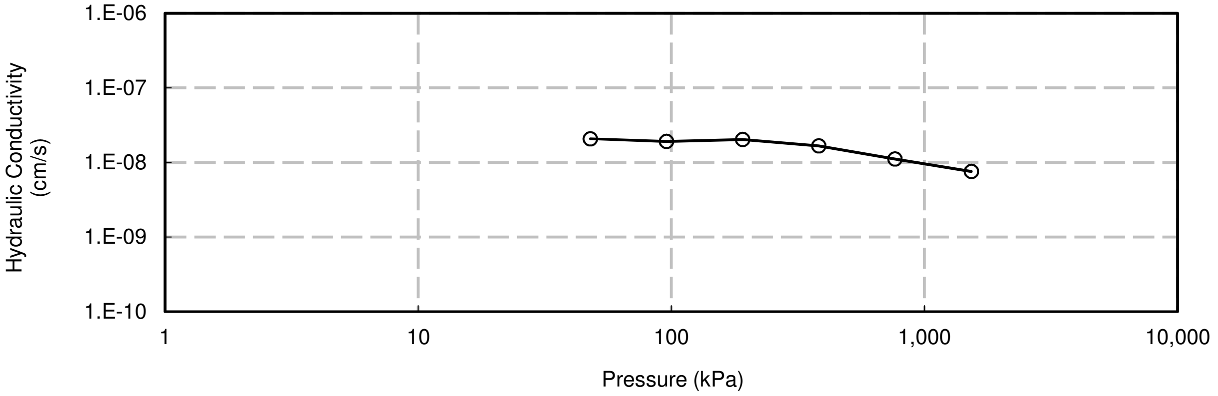
Cv vs Pressure



mv vs Pressure



k vs Pressure



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Date :	July 2020	Checked By :	ML

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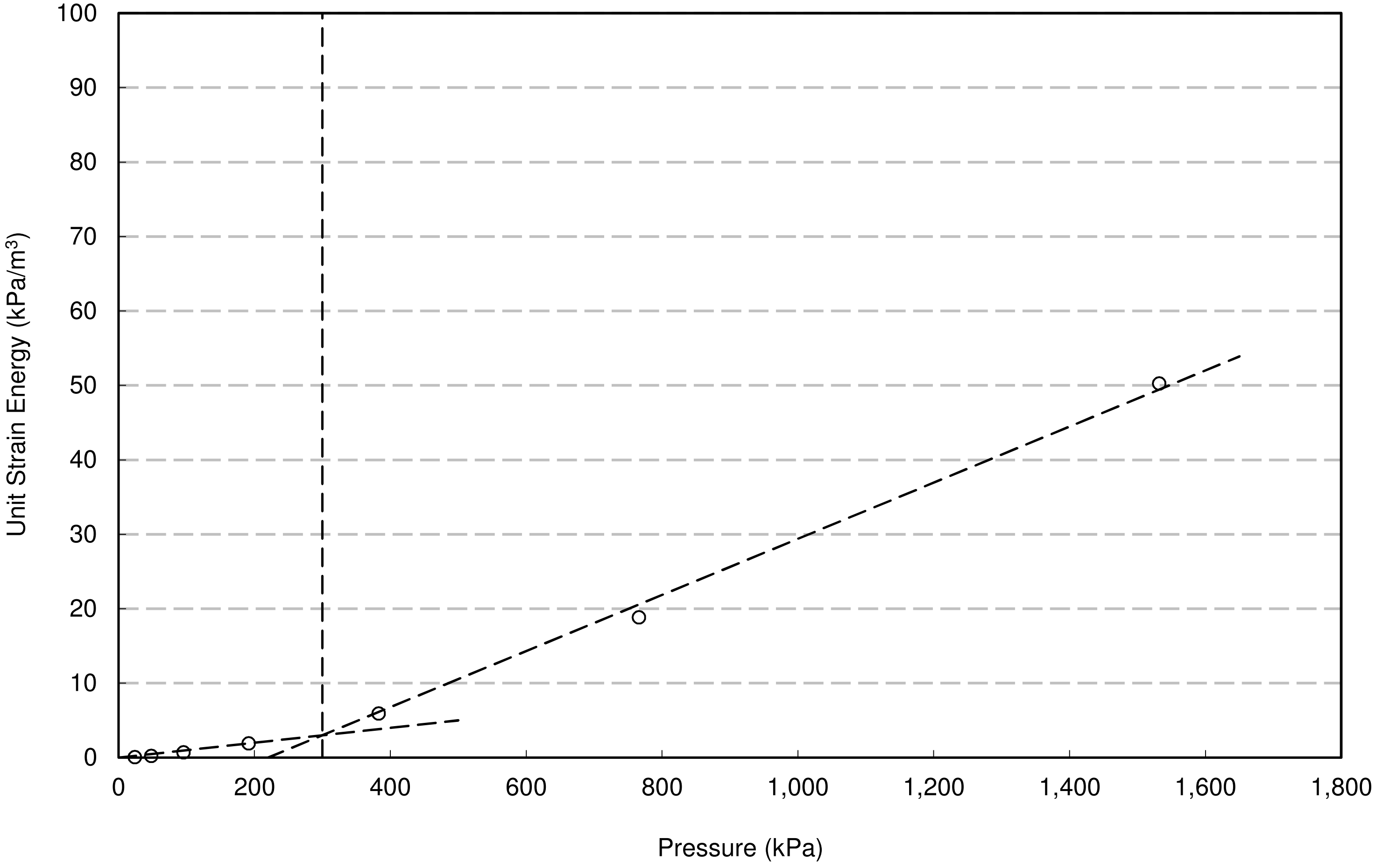
Date : July 2020

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 102, TW 7 - Silty Clay, trace sand

Unit Strain Energy vs Pressure



Pc = 300 kPa

Prepared By : HW
Checked By : ML

CONSOLIDATION TEST

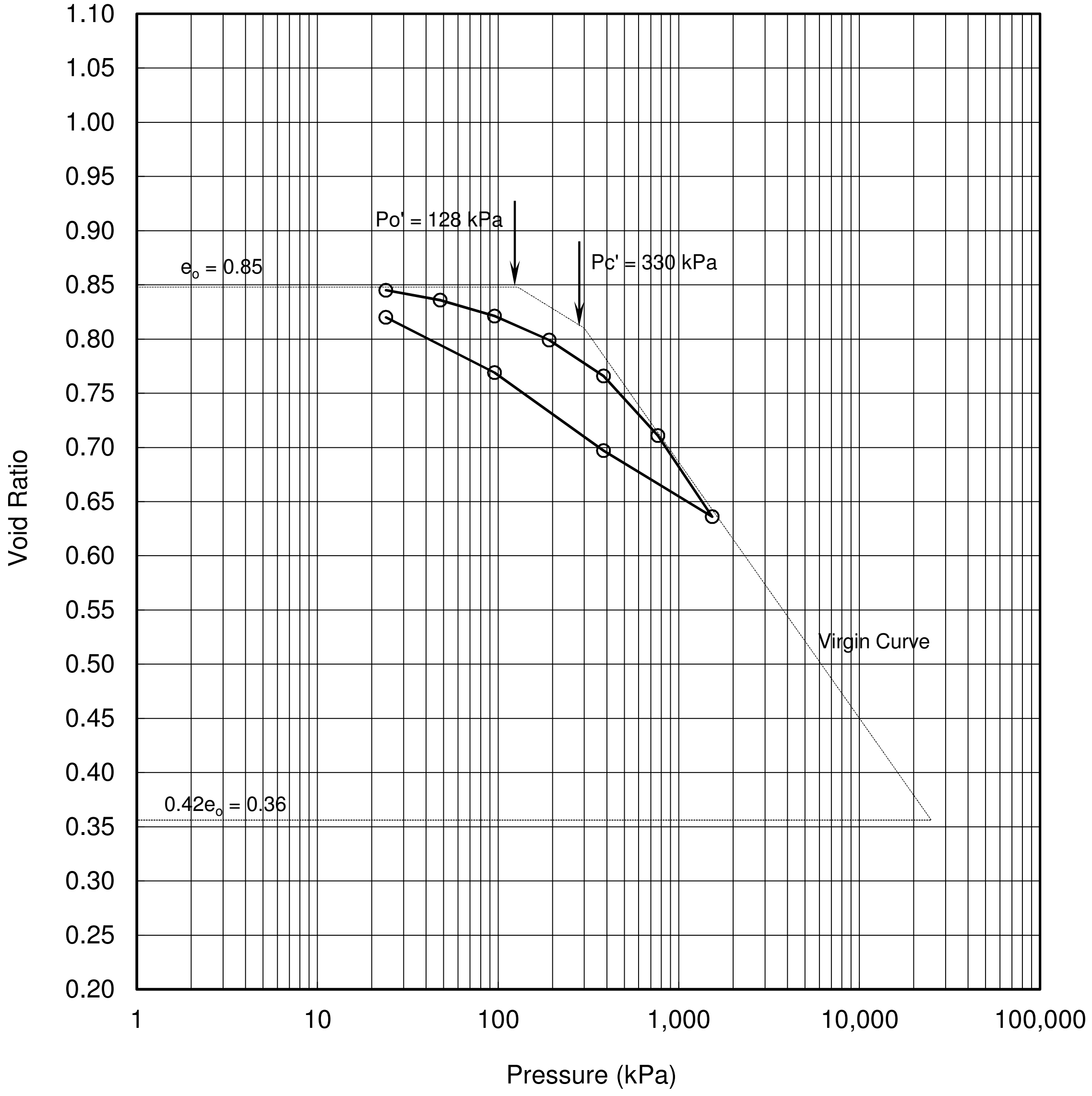
Figure No. 10

CONSOLIDATION TEST

Figure No. 11

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 102, TW 7 - Silty Clay, trace sand

e vs Pressure



Soil Type :	Silty Clay, trace sand						
e_o =	0.85	ω_L =	49%	Po' =	128	kPa	
ω =	29%	ω_P =	20%	Pc' =	300	kPa	
γ =	20.0 kN/m^3	PI =	29%	Cc =	0.24		
Gs =	2.78			Cr =	0.103		

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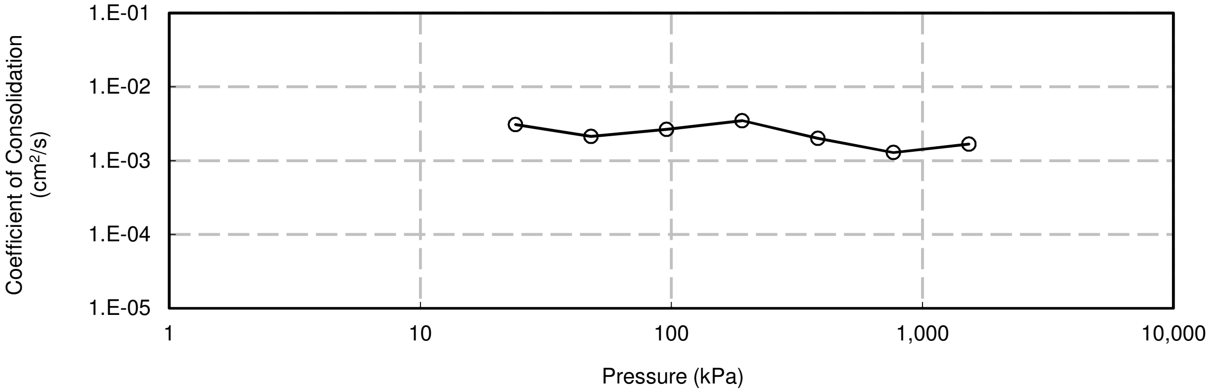
CONSOLIDATION TEST

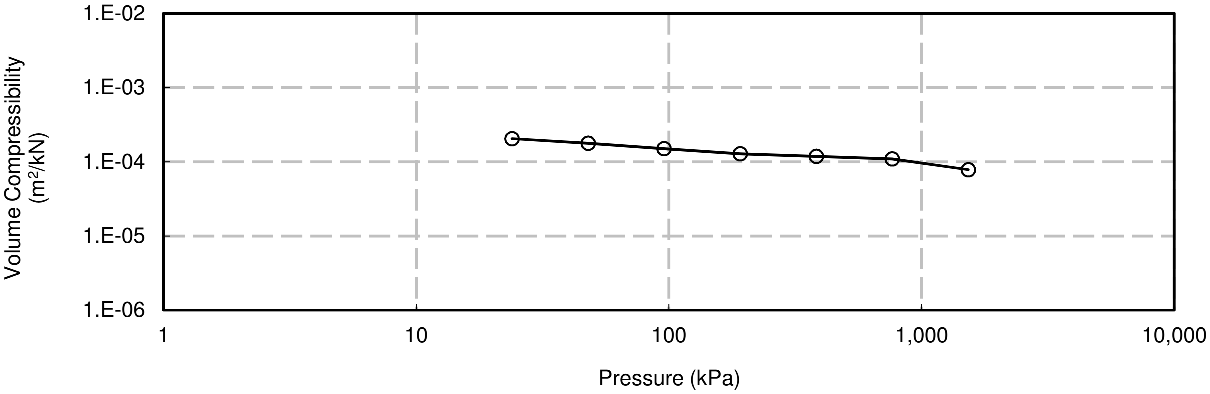
Figure No. 12

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 102, TW 11 - Silty Clay, trace sand

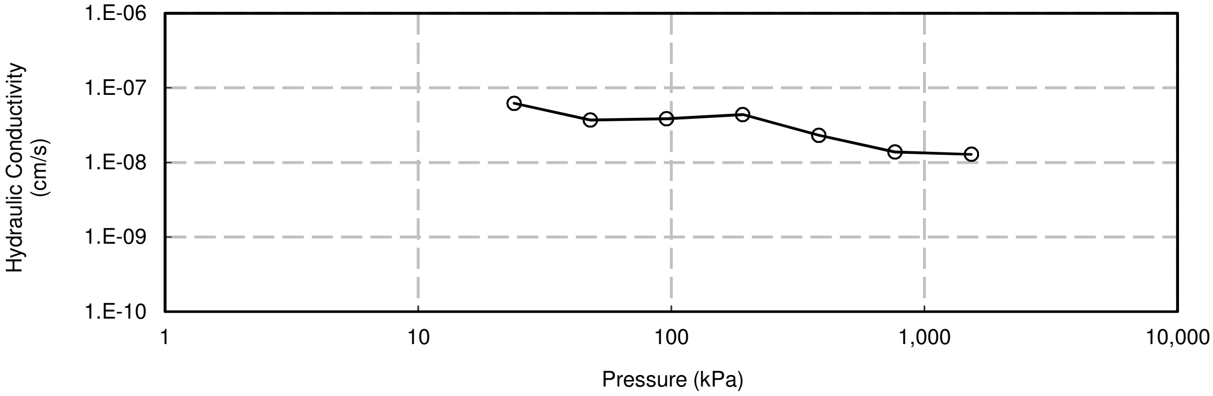
Cv vs Pressure



mv vs Pressure



k vs Pressure



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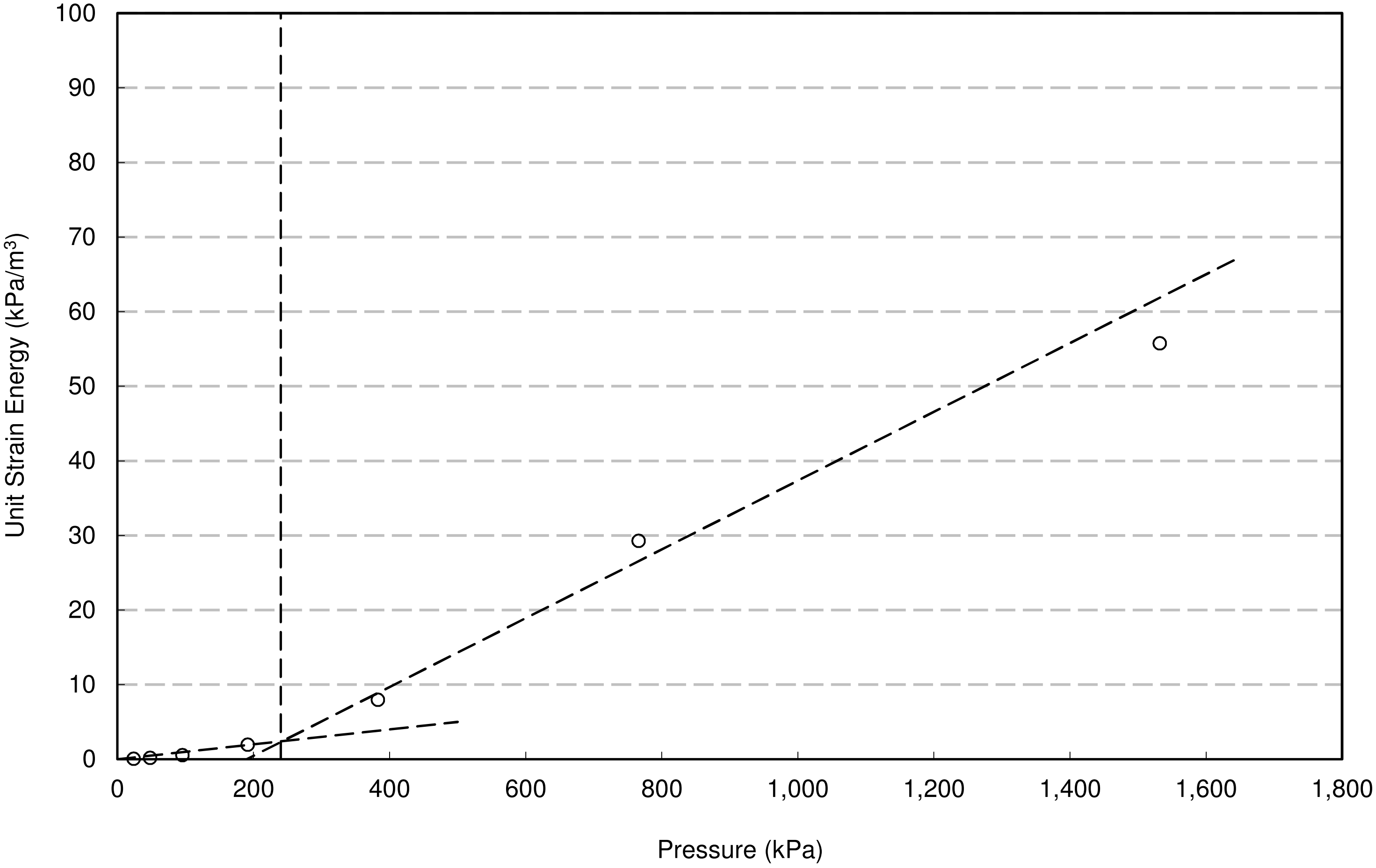
Date : July 2020

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON

BH 102, TW 11 - Silty Clay, trace sand

Unit Strain Energy vs Pressure



Pc = 240 kPa

Prepared By : HW

Checked By : ML

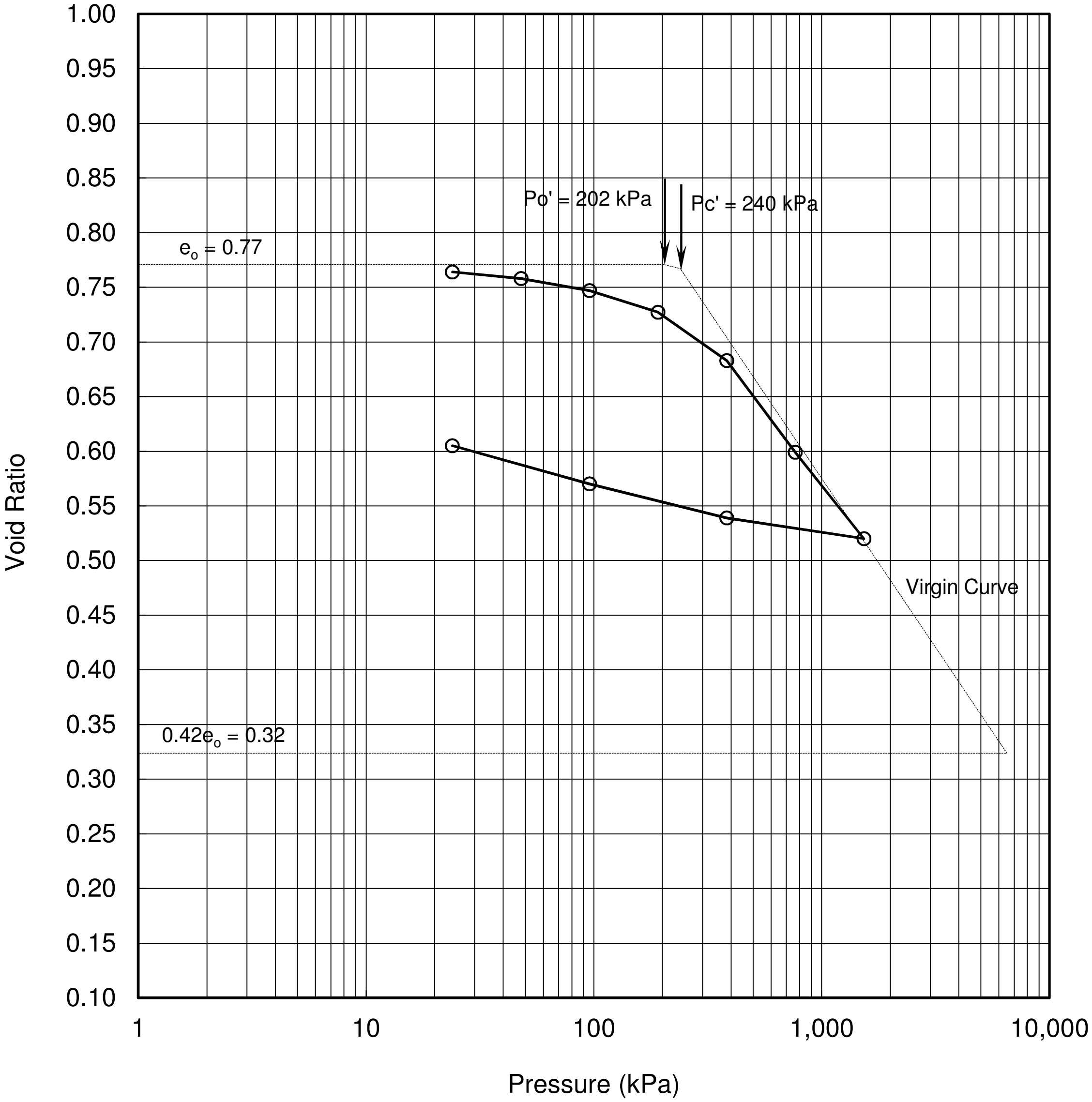
CONSOLIDATION TEST

Figure No. 13

CONSOLIDATION TEST	**Figure No. 14**

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON
BH 102, TW 11 - Silty Clay, trace sand

e vs Pressure



Soil Type : Silty Clay, trace sand

e_o =	0.77	ω_L =	36%	Po' =	202 kPa
ω =	27%	ω_P =	18%	Pc' =	240 kPa
γ =	20.4 kN/m^3	PI =	18%	Cc =	0.31
Gs =	2.75			Cr =	0.054

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Checked By : ML

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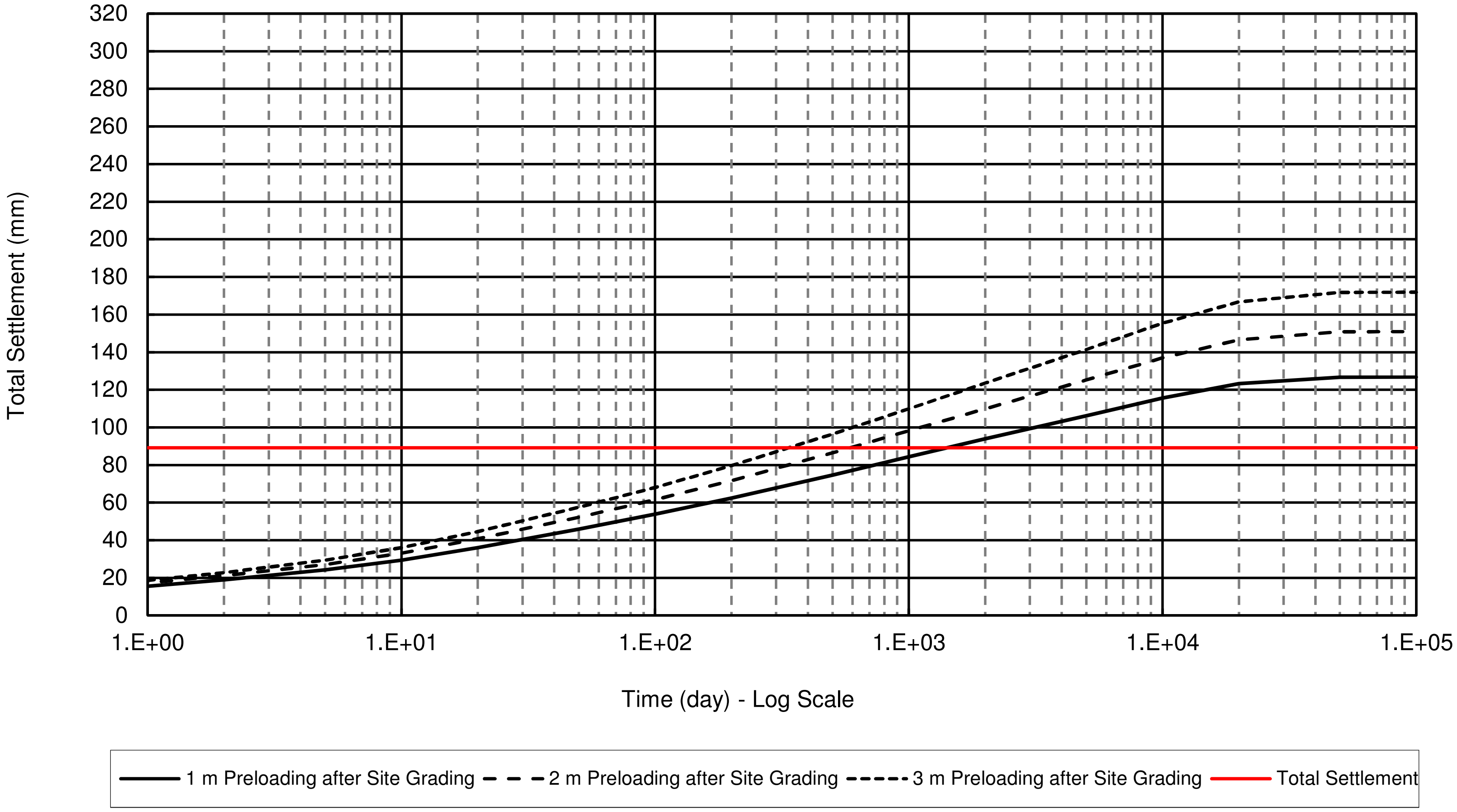
SETTLEMENT VS. LOG TIME

FIGURE 1

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON

Proposed Cut Area (Reference Borehole 101)



Project No. : HAM-00801631-E0

Date : July, 2020

Prepared By : HW

Checked By : PC

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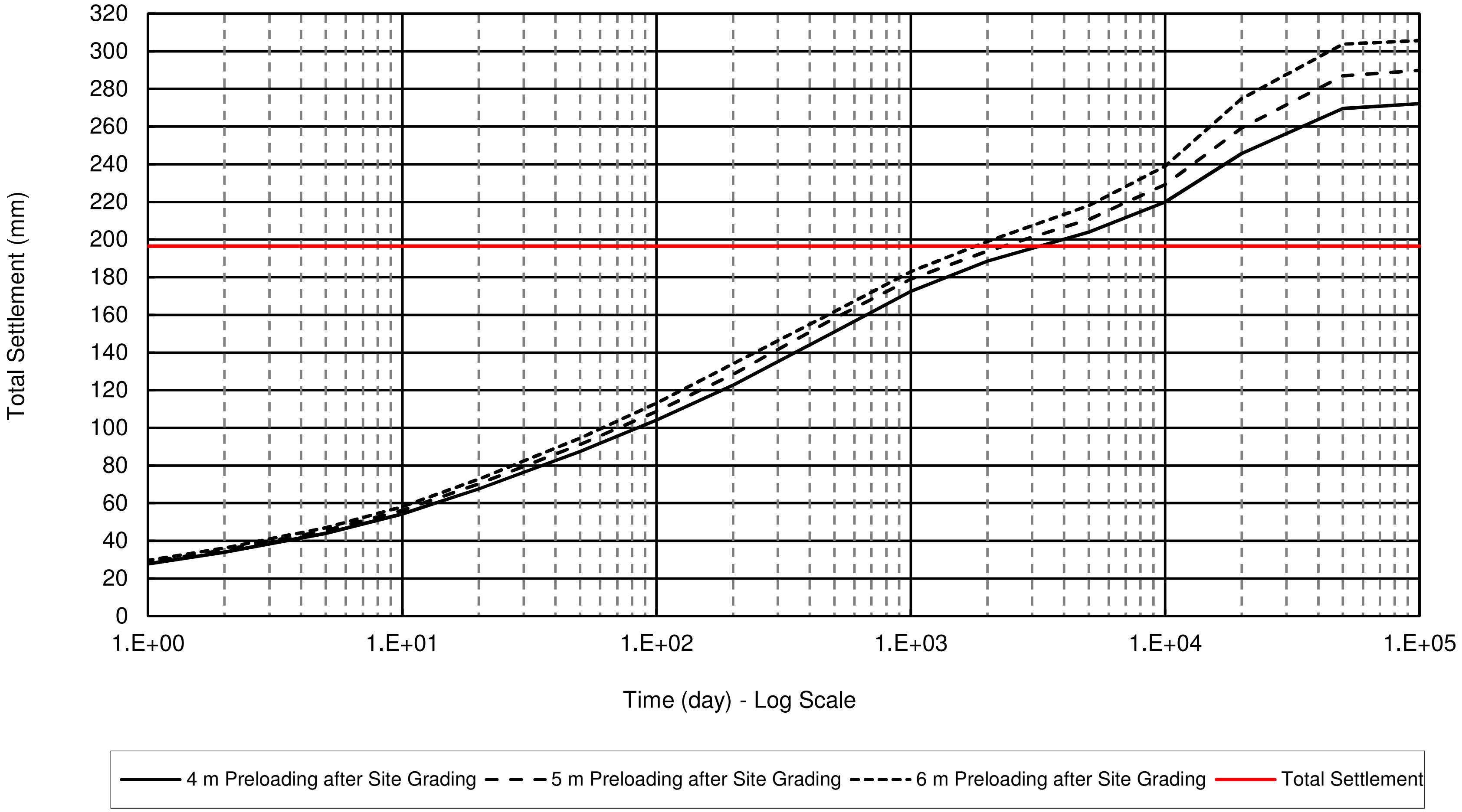
SETTLEMENT VS. LOG TIME

FIGURE 2

Supplemental Geotechnical Investigation, 555 Canal Bank Street, Welland, ON

Proposed Fill Area (Reference Borehole 102)



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Date : July, 2020

Prepared By : HW

Checked By : PC

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West Side (Cur Area, Reference BH 101)

	Total Settlement (mm)	Time (day)	Allowable Settlement (mm)	Time (day)	Note
1 m Preloading after Site Grading	89	1,400	64	230	3 m above Existing Grade
2 m Preloading after Site Grading	89	580	64	120	4 m above Existing Grade
3 m Preloading after Site Grading	89	330	64	80	5 m above Existing Grade

East Side (Fill Area, Reference BH 102)

	Total Settlement (mm)	Time (day)	Allowable Settlement (mm)	Time (day)	Note
4 m Preloading after Site Grading	197	3,200	172	950	9 m above Existing Grade
5 m Preloading after Site Grading	197	2,300	172	760	10 m above Existing Grade
6 m Preloading after Site Grading	197	1,800	172	680	11 m above Existing Grade

Allowable Settlement = Total Settlement - 25 mm

Compaction for site grading 100% SPMDD

Compaction for preloading 90% SPMDD