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Geotechnical Investigation
Proposed Commercial Development

839, 853 and 869 Hurontario Street & 7564 Poplar Sideroad, Collingwood, Ontario
G2S21366D-Rev. 1

Charis Developments Ltd.
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Attention: Steve Assaff, President

July 2025

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Appendices

1. Introduction

G2S Consulting Inc. (G2S) was retained by Charis Developments Ltd. (the Client) to complete an additional Geotechnical Investigation for the properties located at 839, 853 and 869 Hurontario Street & 7564 Poplar Sideroad, Collingwood, Ontario, hereinafter referred to as the ‘Site’.

G2S had previously submitted a geotechnical report for the Site titled ‘G2S21366C, titled “Geotechnical Investigation Proposed Commercial Development, 839 and 869 Hurontario Street, Collingwood, Ontario”, dated March 17, 2022.

Based on the updated Conceptual Site Plan provided by the client and dated May 9, 2025, the properties at 853 Hurontario and 7564 Poplar Sideroad were added to the proposed development.

It is understood that the proposed development will include the construction of a commercial development (the Gateway Centre), which will contain five (5) slab-on-grade Commercial buildings, three (3) storey commercial/office building, and twelve (12) storey mixed use building with two (2) levels of underground parking. The proposed development will also contain the associated at grade parking areas, and underground utilities.

The general location of the Site is shown on the Site Key Plan included on Drawing No. 1 in Appendix A.

2. Site and Project Overview

2.1 Site Description

The Site is near rectangular shaped, comprising an approximate plan area of 3.9 hectares (9.6 acres) in size, and is located approximately 2.7 km south of Georgian Bay, in an area consisting of residential, commercial, and vacant land uses. The Site is bound by Poplar Sideroad to the south, Hurontario Street to the west, residential development to the east, and a tributary to Pretty River to the north, followed by residential and commercial developments. The Site is generally vacant, undeveloped land, with a single-family dwelling occupying the northwest corner of the Site, which constitutes the property of 853 Hurontario Street.

At the time of the investigation, a gravel fill pad was noted in the centre portion of the Site. We understand that the granular pad was constructed of imported fill in December 2007/January 2008.

2.2 Proposed Development

It is G2S' understanding that, per the Site Plan provided by the client and dated April 08, 2025, the proposed development will include the construction of a commercial plaza including the following phases:

- Phase 1
    - Four (4) retail buildings (Total Building height ~7000mm)
    - Associated site servicing and above ground parking
- Phase 2
    - One (1) grocery store (Building height ~7200mm)
    - Two units retail building (Building height ~7000mm)
    - Associated site servicing and above ground parking
- Phase 3
    - Multi-storey mixed-use building with commercial on ground floor and residential above (Building height ~42500mm) (12 storey)
    - Retail building with offices above (Building height 11000mm) (3 storey)
    - Two (2) levels of underground parking
    - Associated site servicing

The purpose of this geotechnical investigation was to determine the subsurface conditions at twenty-three (23) borehole locations (BH101 – BH123), which were completed between September 30, 2021, and January 7, 2022, and two (2) borehole locations (BH201 – BH202), which were completed on June 4, 2024. Further, the investigation was carried out to interpret the findings from theses boreholes with respect to the design and construction of the underground services, foundations and related earthworks for this project from a geotechnical point-of-view.

This report is based on the above summarized project description, and on the assumption that the design and construction will be performed in accordance with applicable codes and standards. Any significant deviations from the proposed project design may void the recommendations given in this report. If significant changes are made to the proposed design, then this office must be consulted to review the new design with respect to the results of this investigation. The information contained in this report does not reflect upon the environmental aspects of the Site and therefore have not been addressed in this document.

3. Investigation Methodology

A total of twenty-five (25) sampled boreholes were advanced at the locations illustrated in the attached Drawing No. 1, Borehole and Monitoring Well Location Plan, in Appendix A. The borings were put down uncased using solid stem continuous flight auger equipment. The drilling and sampling operations were carried out under the direction and supervision of a G2S staff member. Boreholes BH101 to BH123 were advanced to depths of between approximately 2.1 to 8.2 m below the existing grade (mbeg), while BH201 and BH202 were advanced to depths of approximately 6.9 and 11.1 mbeg on completion of drilling, the boreholes were backfilled in general accordance with Ontario Regulation 903.

Representative samples of the subsoils were recovered from the borings at selected depth intervals using split barrel sampling equipment driven in accordance with the requirements of Standard Penetration Resistance Testing (SPT). After undergoing a general field examination, the soil samples were preserved and transported to the soil laboratory for visual, tactile, and olfactory classifications. Routine moisture content tests were performed on the soil samples recovered from the boring. The bedrock was cored at one (1) borehole location (Borehole BH201) using HQ-sized equipment and the retrieved samples were preserved in core boxes and transported to the Burlington laboratory for detailed review.

Details of the conditions encountered in the boreholes, together with the results of the field and laboratory tests, are presented in Borehole (BH) Logs BH101 to BH123, BH201, and BH202, included in Appendix B. It is noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and therefore should not be construed as the exact plans of geological change.

Elevations at the ground surface of the borehole locations were interpolated from the provided Topographic Survey Plan entitled “Plans of Survey of Part of Lot 40 Concession 8 Geographic Township of Nottawasaga Now in the Town of Collingwood County of Simcoe”, by J.D. Barnes, dated September 13, 2023. This topographic survey plan was later utilized to produce the Borehole and Monitoring Well Location Plan.

4. Subsurface Conditions

The subsurface soil conditions have been evaluated in the twenty-five (25) boreholes investigated by G2S at the Site for the purpose of this report. It should be considered that the subsurface conditions may not be consistent between and beyond the locations investigated at the Site. The soil descriptions outlined in the following stratigraphic summary are based on our interpretation of non-continuous samples of soil obtained from boreholes.

The subsurface conditions encountered at the borehole locations are summarized as follows:

4.1 Topsoil/Surficial Granular

A surficial veneer of topsoil and organic material with thicknesses ranging between approximately 75 and 360 mm, was encountered in BH101, BH103 to BH108, BH113 to BH115, BH117, BH118, BH121, BH122, and BH202. A surficial layer of granular fill was encountered at the top of BH201 with an approximate thickness of 50 mm

It should be noted that the depth of topsoil must be expected to vary across the Site, particularly in areas of mature trees, from the depths encountered at the borehole locations. If required, a more detailed analysis such as test pits can be carried out to accurately quantify the amount of topsoil to be removed for construction purposes. In this report the term “topsoil” has been used from a geotechnical point of view and does not necessarily reflect the suitability of the material to support plant growth. If it is to be used for landscaping or agricultural purposes, its suitability should be confirmed by tests on representative samples for organic and nutrient content and therefore its ability to support plant growth.

4.2 Fill

Fill material was encountered below the topsoil in BH101, BH103, BH104, BH106 to BH108, BH115, BH118, BH122, at the surface in BH116, and below the surficial granular in BH 201. The fill layer extended to depths ranging from approximately 0.8 and 1.5 mbeg (~Elev. 194.6 to 193.3 m). This layer of fill consisted generally of clayey silt and contained traces to interbedded layers of sand.

Granular fill material (sand and gravel) was contacted at the surface in BH102, BH109 to BH112, BH119, BH120, and BH123. The granular fill extended to depths ranged between 0.9 and 1.6 mbeg (~Elev. 194.9 and 194.0 m). The granular fill was generally brown in color and contained some cobble and boulder size particles. The lower portion of the granular fill was mixed with clayey silt in BH102 and BH109.

With moisture content ranging between 4.0 and 26%, the fill layer was found to be in moist to wet condition.

4.3 Upper Clayey Silt

Clayey silt was found beneath the fill in BH101 to BH104, BH106 to BH109, BH111, BH115, BH116, BH118 to BH120, BH122, BH123 and BH201, below the topsoil in BH105, BH113, BH114, BH117, BH121, and BH202. The clayey silt deposit was found underlaying a silt deposit in BH110. The clayey silt extended to depths ranging between approximately 1.5 and 4.6 mbeg (~Elev. 193.9 and 190.4 m). BH116 to BH121, and BH123 were terminated in this deposit. The upper clayey silt was brown to grey in color and contained traces of sand. With “N” values ranging from 1 to 18 blows per 300 mm of penetration, the clayey silt deposit was classified as very soft to very stiff in consistency. The moisture content for the clayey silt ranged between 18 and 29%, indicating moist conditions. Based on three grain size analyses tests, the clayey silt contained 24 to 34% clay, 64 to 72% silt, and 2 to 3% sand, and 0 to 1% gravel. Based on the laboratory results for three clayey silt samples, the liquid limit and the plastic limit for the clayey silt ranged between 20 and 28, 14 and 18%, respectively, indicating low plasticity. The results of the grain size analysis and the Atterberg Limits are indicated on the borehole logs and the grain size distribution graphs, as well as the plasticity index chart are included in Appendix B.

4.4 Silt

Silt deposit was encountered below the clayey silt in BH101 to BH106, BH108, BH109, BH111, BH113 to BH115, BH122, and below the fill in BH110 and BH112. In BH110 the silt was found interbedded by the native clayey silt deposit with a thickness of approximately 1.3 m. The silt extended to depths ranging from 3.1 and 6.1 mbeg (~Elev. 192.0 and Elev. 189.6 m). The silt deposit was generally greyish brown in color and contained traces of gravel and some clay. With “N” values ranging between 0 and 17 blows per 300 mm of penetration, the native silt deposit was classified as very loose to compact in compactness. The moisture content for the silt ranged between 5 and 30%, indicating moist to wet conditions. Grain size analysis and Atterberg limits determination testing were carried out on selected soil samples of the silt deposit. Based on the laboratory testing, the silt consisted of 0 to 2% gravel, 6 to 9% sand, 68 to 77% silt, and 15 to 24% clay. Based on the laboratory results for two silt samples, the liquid limit for the silt ranged between 20 and 22%, and the plastic limit was ranging between 15 and 16%, indicating low plasticity. The results of the grain size analysis and the Atterberg limits are indicated on the borehole logs and the grain size distribution graphs, as well as the plasticity index chart are included in Appendix B.

4.5 Lower Clayey Silt

Lower clayey silt deposit was encountered beneath the silt in BH113 and BH122, below (~Elev. 191.8 – 190.5 m) and extending to depths ranging between 4.6 and 6.1 mbeg (~Elev.190.3 and 189.0 m). The lower clayey silt was brown to grey in color and contained traces of gravel and sand. With the hammer dropping under its own weight during the Standard Penetration Test and undrained shear strength in the order of 12 kPa, as determined by the field shear vane testing, the lower clayey silt deposit was classified as very soft in consistency. The moisture content for the clayey silt ranged between 21 and 23%, indicating moist conditions. Grain size analysis and Atterberg Limits determination testing were carried out on selected soil samples of the clayey silt deposit. Based on a two grain size analyses tests, the clayey silt contained 28 to 32% clay, 60 to 68% silt, 3% sand, and 1 to 5% gravel. Based on the laboratory results for two clayey silt samples, the liquid limit for ranged from 21 to 22%. and the plastic limit was in the order of 15%, indicating low plasticity. The results of the grain size analysis and the Atterberg Limits are indicated on the borehole logs and the grain size distribution graphs, as well as the plasticity index chart are included in Appendix B**.**

4.6 Silty Sand Till/Sandy Silt Till

Silty sand till/sandy silt till was encountered below the silt deposit in BH101 to BH106, BH108 to BH112, BH114, and BH115, and below the clayey silt deposit in BH107, BH113 and BH122, BH201, and BH202. The silty sand till/sandy silt till extended to depths ranging between 5.2 and 8.2 mbeg at the termination depths. Borehole Nos. BH101 to BH115, and BH122 were terminated in this deposit. With “N” values ranging between 3 and in excess of 50 blows per 300 mm of penetration, the silty sand till/sandy silt till deposit was classified as very loose to very dense in compactness. The moisture content for the till deposit ranged between 6 to 18% indicating moist to very moist conditions. Grain size analysis and Atterberg limits determination testing were carried out on selected soil samples of the silt deposit. Based on the laboratory testing, the silty sand till/sandy silt till deposit consisted of 10 to 33% gravel, 36 to 60% sand, 7 to 35% silt, and 0 to 19% clay. The results of the grain size analysis are indicated on the borehole logs and the grain size distribution graphs, as well as the plasticity index chart are included in Appendix B.

4.7 Sandy Gravel Till

Sandy gravel till deposit was encountered beneath the sandy silt till in both boreholes BH201 and BH202. The gravel extended to depths ranging from approximately 6.9 to 7.5 mbeg. The sandy gravel till deposit was generally grey in color containing trace silt. With” N” values in excess of 50 blows per 300 mm of penetration, this deposit was classified as very dense in compactness

4.8 Limestone/Dolostone Bedrock

Limestone/dolostone bedrock was encountered at depths ranging from approximately 6.9 to 7.5 mbeg (~Elev. 188.1 to 187.8 m). Bedrock was proven by coring in Borehole BH201 between 7.5 and 11.1 mbeg (187.8 and 184.2 m) and was inferred by auger and sampler refusal in BH202. The approximate depth and elevation of the limestone/dolostone bedrock surface at the borehole locations are presented in Table 1 below:

Table 1: Approximate Depth and Elevation of Limestone/Dolostone Bedrock Surface

Borehole ID	Depth of Dolostone Bedrock Surface Below Existing Grade (m)	Approximate Relative Elevation* of Bedrock Surface (m)	Remarks
BH201	7.5	187.8	Proven by Coring between 7.5 and 11.1 mbeg (~Elev. 187.8 to Elev. 184.2 m)
BH202	6.9	188.1	Inferred by Auger

Due to the method of drilling and sampling, the surface elevation of the bedrock at the location, which was proven by auger, can be different than indicated on the borehole log. Typically, the presence of boulders above the actual bedrock surface may give a false indication of the bedrock level. From our observation of the recovered samples, a review of published information, and past experience in the area, the bedrock is a limestone of the Simcoe Groupe from the Middle Ordovician period. The bedrock in the area may also contain interbedding dolostone, shale, arkose, and sandstone. The Limestone bedrock is generally considered very competent in terms of the foundation/excavation requirements for the proposed project. The upper level of Limestone bedrock is generally grey to dark grey in colour, fine grained and argillaceous, typically considered good to excellent relative to the Rock Quality Designation.

Based on the rock core samples, which were obtained from Borehole BH201, the bedrock consisted of grey to dark grey limestone/dolostone, moderately weathered to unweathered in general, with highly fractured zones between 7.6 – 7.8, 8.4 – 8.5, 9.5 – 9.6, and 9.7 – 9.8 mbeg at the location of BH201.

The Total Core Recovery (TCR) was 98% to 100% with a recorded Rock Quality Designation (RQD) for each core run ranging between 0% and 89%, indicating very poor to good quality. The discontinuities observed in the rock cores were typically bedding planes with flat orientation. Vertical or dipping discontinuities were noted at depths of 7.6, 8.3, 8.4, 8.9, 9.5, and 9.9. The spacing of the discontinuities ranged from very close to moderate, and joints filling was generally classified as slightly altered, sandy and silty minor clay, or oxidation.

Laboratory Unconfined Compressive Strength (UCS) tests were performed on three (3) selected samples of the rock cores and, with results ranging from 37.7 to 73.6 MPa, indicated medium to high strength. The UCS laboratory test results report along with photographs for the retrieved core samples are included in Appendix C. Details of the rock coring are included on the borehole log in Appendix B.

4.9 Groundwater Observations

The groundwater level observations during the drilling operation have been recorded as footnotes on the borehole logs. Free water was reported in BH 101, BH102, BH103, BH104, BH106, BH107, BH108, BH111, BH112, BH113, BH114, and BH115 at depths ranging between 0.3 and 4.4 mbeg. Sidewall caving was reported in BH102, BH103, BH106, BH108, BH111, BH112, and BH115 at approximate depths of 4.6, 4.3, 4.3, 4.4, 5.8, 3.0, and 3.9 mbeg, respectively. Groundwater monitoring wells were installed in Borehole Nos. BH105, BH110, and BH122 during the initial geotechnical investigation on September 30th and October 1st, 2021, to facilitate the prolonged groundwater monitoring. Further, four (4) additional monitoring wells were installed on January 7, 2022, in the immediate vicinity of BH101, BH103, BH106, and BH115 to provide relevant information for the hydrogeology study at the Site. Those monitoring wells were identified as BH101A, BH103A, BH106A, and BH115A, respectively. Two (2) additional monitoring wells were installed on June 4, 2024, to provide relevant groundwater information pertinent to the construction of Phase 3 from the project. The results of groundwater monitoring to date are summarized in Table No. 2A and 2B below:

Table 2A: Groundwater Observations – 2021- 2024

Sample Location	Installation Date	Ground Surface Elevation	Well Depth from Ground Surface (m)	Screened Interval Elevation (m) and Depth (m bgs)	Groundwater Elevation and Depth (mbeg)								
					Oct 13, 2021	Nov 3, 2021	Jan 21, 2022	Apr 5, 2022	June 5, 2024	June 20, 2024	June 26, 2024	July 19, 2024	May 16, 2025
BH/MW101	Jan. 7/22	194.8	4.8	193.0 – 190.0 (1.8 – 4.8)	-	-	193.7 (1.1)	193.7 (1.1)	193.7 (1.1)	--	193.6 (1.2)	193.7 (1.1)	193.8 (1.0)
BH/MW103	Jan. 7/22	194.4	4.6	192.8 – 189.8 (1.6 – 4.6)	-	-	193.7 (0.7)	193.8 (0.6)	193.6 (0.8)	--	193.5 (0.9)	193.7 (0.7)	193.7 (0.7)
BH/MW105	Oct. 1/21	194.6	4.8	192.9 – 189.9 (1.8 – 4.8)	194.7 **(-0.1)**	195.0 **(-0.3)**	Frozen	194.8 **(-0.1)**	194.6 (0)	194.2 (0.4)	194.5 (0.1)	194.6 (0)	194.7 (-0.1)
BH/MW106	Jan. 7/22	194.5	4.6	192.9 – 189.9 (1.6 – 4.6)	-	-	Frozen	194.7 **(-0.2)**	194.5 (0)	--	194.4 (0.1)	194.4 (0.1)	194.6 (-0.01)
BH/MW110	Oct. 1/21	195.9	5.3	193.6 – 190.6 (2.3 – 5.3)	193.9 (2.0)	194.9 (0.9)	194.4 (1.4)	194.5 (1.3)	194.6 (1.3)	194.4 (1.5)	194.6 (1.3)	194.6 (1.3)	194.6 (1.3

Sample Location	Installation Date	Ground Surface Elevation	Well Depth from Ground Surface (m)	Screened Interval Elevation (m) and Depth (m bgs)	Groundwater Elevation and Depth (mbeg)								
					Oct 13, 2021	Nov 3, 2021	Jan 21, 2022	Apr 5, 2022	June 5, 2024	June 20, 2024	June 26, 2024	July 19, 2024	May 16, 2025
BH/MW122	Sept. 30/21	195.1	5.3	189.8 – 192.8 (2.3 – 5.3)	194.7 (0.4)	195.0 (0.10)	195.6 **(-0.7)**	194.7 (0.4)	--	--	194.3 (1.0)	--	194.8 (0.3)

Bolded value level is above the ground surface and may be due to an unstable groundwater level and/or upward hydrogeologic gradient

Table 2B: Groundwater Observations – 2024

Sample Location	Installation Date	Ground Surface Elevation	Well Depth from Ground Surface (m)	Screened Interval Elevation (m) and Depth (m bgs)	Groundwater Elevation and Depth (mbeg)			
					Jun 20, 2024	Jun 26, 2024	Jul 19, 2024	May 16, 2025
BH/MW201	Jun. 4, 2024	195.3	7.6	190.7 - 187.7 (4.6 – 7.6)	194.2 (1.1)	194.7 (0.6)	194.8 (0.5)	194.9 (0.4)
BH/MW202	Jun. 4, 2024	195.0	6.9	191.2 – 188.1 (3.8 – 6.9)	194.3 (0.6)	194.6 (0.4)	194.6 (0.4)	194.7 (0.3)

Some infiltration of groundwater through the more permeable seams of the native soils and surface runoff should be anticipated during the excavation operations. Surface water should be directed away from the excavations. It is noted that the static groundwater level fluctuates based on seasonal conditions experienced and may at times be slightly shallower than noted above during the 'wet' periods of the year (i.e., spring melt). Refer to Appendix B for the list of abbreviations and borehole logs.

5. Geotechnical Considerations

5.1 Site Preparation

At the time of this report, the final grading plan of the proposed development was not yet available to G2S. However, based on the updated Site Plan and the proposed development plan as outlined in Section 2.1, it is likely that engineered fill will be utilized to accommodate the design grades for the proposed buildings. Prior to any earthwork, it will be necessary to remove some or most of the vegetation and topsoil from the Site. All topsoil and any near-surficial soil containing high amounts of topsoil and/or organic material should be removed in areas that are to be developed.

Any engineered fill must be placed and uniformly compacted in maximum lift thicknesses of 300 mm for earth fill and 200 mm for commercially sourced granular material. Each lift of the engineered fill must be uniformly compacted to at least 100 percent of Standard Proctor Maximum Dry Density (SPMDD). The placement water content of the engineered fill material should be maintained within ± 3 percent of the laboratory optimum water content in order to achieve an acceptable degree of compaction.

The limits of any engineered fill placed during this operation can best be determined by the geotechnical engineer at the time of construction. If engineered fill will be used to support foundations or pavements, it must extend laterally at sufficient distance to develop adequate lateral resistance.

All aspects of engineered fill construction including final excavation, material selection, placement and compaction must be tested by the geotechnical engineer at the time of placement and compaction. In-situ density (compaction) testing is required during construction for any and all engineered fill placement.

5.2 Foundation Recommendations

5.2.1 Methodology

The shallow foundations are to be designed applying the Limit State Design (LSD) methodology described in the current Canadian Foundation Engineering Manual (CFEM). Both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) were considered.

For design purposes to address the ULS, the ultimate (unfactored) bearing capacity of the foundation soil (R_n) was calculated. The allowable (factored) bearing capacity (ΦR_n) was computed by multiplying R_n with a reduction factor $\Phi=0.5$, in accordance with National Building Code of Canada (NBCC) (2020) and CFEM (2023).

The foundation designer needs to ensure that the factored bearing capacity is greater than the factored applied pressure at foundation level ($\alpha_i S_{ni}$). Hence, the following formula applies:

$\Phi R_n \geq \sum \alpha_i S_{ni}$

If the foundation is subjected to vertical forces that act eccentric to the centroid of the foundation, the size of the foundation used in the bearing capacity equation is reduced to the following:

$B' \times L' = (B-2e_B) \times (L-2e_L)$

where:

B, L: actual foundation dimensions

B’, L’: reduced dimensions to be used in the bearing capacity equation

e_B, e_L: eccentricities due to applied forces (loading) from the centroid in dimensions B and L respectively

Foundations subject to moments M_B and M_L in the B and L directions and vertical load V acting through the centroid are equivalent to a loading system with V acting at eccentricities $e_B=M_B/V$ and $e_L=M_L/V$.

CFEM (2006) emphasizes that this equation is an approximate but reasonable approach provided that the eccentricity acts within the middle third of the foundation, i.e., eccentricity, $e < B/6$. In addition, in case of inclined loading, appropriate factors need to be considered in accordance with CFEM (2006).

The serviceability limit state (SLS) bearing pressure is considered by calculating the settlements (immediate, consolidation and total) due to foundation load. It is expected that the structural team will compare the settlements due to foundation and fill loads against allowable foundation settlement values.

It is noted that the overall settlement and/or heave experienced by the foundations may depend on other factors such as the quality of subgrade preparation and weather conditions at the time of construction, ground freeze and thaw, dynamic loading, among other factors.

5.2.2 Strip and Spread Footing – (Building 01, 02, 05, 06, 07, and 08)

The proposed structures can be supported on conventional spread and strip footings founded on the native clayey silt/silt deposits below any fill or disturbed soils. Bearing resistance of 100 to 180 kPa at SLS and 200 to 360 kPa at ULS could be utilized for the foundation design. The geotechnical resistance of a sustained load at Serviceability Limit State (SLS) should be within the normally tolerated limits of total and differential settlement of 25, and 19 mm, respectively, and a maximum footing size of 1.5 m. Should a different footing size being used, G2S should be contacted to provide an update for the bearing resistance and the associated settlement.

Prior to placement of foundation concrete, all existing fill, organics, and any other deleterious material must be removed down to the undisturbed native soils. The exposed footing base is to be inspected by G2S.

Alternatively. The proposed structures can be supported by conventional spread and strip footings founded on engineered fill, utilizing design bearing resistance ranging between 120 and 150 kPa at SLS and 180 to 225 kPa at ULS.

The available bearing resistance and the relevant approximate founding elevations are presented in Table No. 3 below:

Table 3: Bearing Resistance for Conventional Spread Footings

Phase	Building Location	Borehole ID	Material	Bearing Resistance (kPa)	Recommended Founding Depth (m)	Approximate Founding Elevation (m)
Phase 1	Building 01	BH121	Engineered Fill	120 SLS/180 ULS	-	194.6
	Building 02	BH105	Clayey Silt	120 SLS/240 ULS	0.8	193.8
		BH118	Engineered Fill	150 SLS/225 ULS	-	194.5
	Building 07	BH112	Silt	100 SLS/200 ULS	1.7	194.2
		BH113	Clayey Silt	180 SLS/360 ULS	0.8	194.1
	Building 08	BH109	Clayey Silt	120 SLS/240 ULS	1.7	194.1
		BH111	Engineered Fill	150 SLS/225 ULS	-	195.2
		BH122	Engineered Fill	150 SLS/225 ULS	-	195.2
Phase 2	Building 05 (Grocery Store)	BH106	Clayey Silt	180 SLS/360 ULS	1.0	193.5
		BH107	Engineered Fill	150 SLS/225 ULS	-	194.8
		BH108	Clayey Silt	180 SLS/360 ULS	1.0	194.4
		BH117	Clayey Silt	120 SLS/240 ULS	0.3	194.1
	Building 06	BH115	Clayey Silt	180 SLS/360 ULS	1.0	194.3
		BH114	Clayey Silt	180 SLS/360 ULS	1.0	194.0

It should be noted that a weaker zone within the silt deposit was encountered in some boreholes and this layer was typically found below approximate Elevations 192.8 to 191.0 m.

Settlement is expected to occur within the weaker deposit following the fill placement. Settlement analysis was carried out for the area below each building pad to explore the consolidation characteristics of the weak silt deposit based on the thickness of the engineered fill material, which is anticipated to be placed. The results of the settlement analysis are summarized in Table No. 4 below:

Table 4: Estimated Settlement Following Engineered Fill Placement

Building Location	Borehole ID	Engineered Fill Thickness (m)	Estimated Settlement (mm)		
			Im.	Cons.	T
Building 01 & 02	BH111, BH105	1.7	6	37	43
Building 05	BH106, BH107	1.75	5	25	30
Building 06	BH114, BH115	1.7	5	29	34
Building 07	BH111, BH112,	1.2	4	21	25
Building 08	BH109, BH111, BH122	1.2	4	21	25

Im: Immediate Settlement
Cons.: Consolidation Settlement
T: Total Settlement

The weaker deposit is predominantly silt and most of the consolidation is expected to occur within six months to one year from the time of fill placement. However, preloading is expected to accelerate the consolidation process. Engineered fill with an average thickness of 1.7 m is expected to be placed in the area of Buildings (01&02), 04, 05, and 06, and based on the settlement analysis results, the total settlement within the weaker silt deposit in these areas, was estimated as 43, 48, 30, and 34 mm, respectively. Therefore, it is our recommendation to preload these areas with a minimum of 1.0 m of additional fill.

The settlement analysis for the placement of approximately 1.2 m in thickness of engineered fill in the area of Buildings 07 and 08, indicated an expected total settlement of 25 mm within the weaker silt deposit below 3.1 mbeg (~Elev. 192.8 – 192.0 m). The anticipated settlement should be within the acceptable limits. However, any increase in the engineered fill thickness may cause settlement exceeding these acceptable limits. As such, this office should be notified if the values of the engineered fill thickness have increased more than the values noted above. It is also recommended that the start of construction of foundations be prolonged as much as possible after placement of the fill, especially at areas where relatively high load is anticipated, such as the area of Buildings 04 and 05, to make sure that the majority of the settlement/soil consolidation has occurred prior to the footing construction. The expected settlement of the engineered fill under its own weight should be between 0.5% to 1.0% of the height of the material placed. The majority of the settlement for engineered fill consisting of cohesive material is expected to occur within 2 to 3 months of the fill placement. Furthermore, we would recommend that settlement monitors be installed to monitor the rate of settlement and to determine if and when the majority of the settlement (90%) has taken place.

In addition to preloading, wick drains can also be used to accelerate the consolidation process and significantly reduce the consolidation time for loose cohesive soils to shorten the drainage path for the pore water and hence accelerate consolidation process. Wick Drain is a special purpose strip drain used for consolidation of soft, compressible soils. The wick drain has two components, a core which serves as a water conduit for the pore water and a geotextile filter fabric, which allows water to pass into the core while restricting the movement of soil particles which might clog the core. The drain is fed down through a hollow mandrel mounted on an excavator or crane, which is connected at the bottom to an expendable anchor plate. A vibratory hammer or static method is used to drive the mandrel to design depth. It's then removed, leaving the wick drain in place. The wick drain is then cut at the ground surface. The vertical wick drains are installed in a pattern to provide short drainage paths for pore water, which accelerates the consolidation process and consequently shortens the construction schedule.

5.2.3 Helical Piles (Buildings 01, 02, 05, and 06)

If higher bearing resistance is required or the project schedule wouldn't allow the pre-loading, Helical piles could be an option to support the proposed structures below the weaker silt zone and into the native silty sand till/sandy silt till.

Helical piles are a proprietary foundation system, and therefore, the actual allowable bearing resistance and pile capacity must be determined by the pile designer based on site-specific conditions and the proposed load requirements as defined by the client. The final capacity will depend on several factors, including pile size, the number of helices, soil density, and installation torque. This capacity estimates assume typical installation conditions and soil characteristics, but it is essential to note that site-specific testing and design must verify the final capacities.

The load capacity of helical piles is heavily influenced by the density of the till, the pile installation method, and the depth of embedment. The use of grouted columns may also be considered to enhance pile capacity. Grouting can significantly increase both the skin friction and overall load-bearing resistance of the pile. Additionally, pre-drilling or other ground modification techniques may be employed to optimize performance in challenging soils.

The helical pile design should be carried out as per the latest edition of the Canadian Foundation Engineering Manual (CFEM). Helical pile installation is considered feasible for the Site; however, the following should be considered during construction:

- Pile installation monitoring and inspection by qualified geotechnical personnel is required during the installation of all helical pile foundations.
- The upper helix of the pile system should be founded below the frost penetration depth to provide sufficient resistance to frost heaving/adfreezing action.
- Although cobbles and boulders were not encountered in the drilled borehole, however, if encountered, they may hinder the progress of the pile installation. Therefore, it would be prudent that the helical pile supplier/installer be consulted regarding the general subsoil conditions and the feasibility of helical pile system installation at the Site.
- It is also recommended that field load tests be carried out to determine the piles' actual load capacity in compression and tension, confirm the theoretical design foundation, and allow Site specific correlation between capacity and installation torque.

The test pile should be loaded to at least two times the design load. Piles that have been tested to their ultimate capacity should not be used as production piles. Pile test methods and apparatus should conform to ASTM D1143-81.

5.2.4 Ground Improvement technique (Buildings 01, 02, 05, and 06)

Ground improvement system such as Controlled Modulus Columns (CMC) may be considered as a form of ground improvement to provide the required bearing resistance for the anticipated structural load for the proposed building.

Controlled Modulus Columns are injected concrete columns (semi-rigid inclusions), which are installed with a hollow full-lateral-displacement auger. The columns are developed by grouting under controlled limited pressure through the stem of the augers while the augers are being extracted. The building load will be transferred to the installed columns through a load transfer platform consisting of Granular B type II material compacted to 98% of the material Standard Proctor Maximum Dry Density (SPMDD) as specified by the system installer.

The CMC Ground improvement technique is a proprietary system and the system design and specification including the actual available bearing resistance for the system should be determined by the system supplier.

5.2.5 Strip and Spread Footing – BH201 & BH202 - (Building 03 and 04)

It is understood that consideration is being given to the construction of two levels of underground parking below the entire area of Phase 3 of the development including Buildings 03 and 04. As such the foundation level for the proposed Buildings 03 and 04 would be set at approximately 7 to 7.5 mbeg (~Elev. 188.2 – 187.7 m). The proposed structures can be founded on the competent bedrock below any highly weathered or fractured zones below approximately 7.8 mbeg (~Elev. 187.7 m) and designed for bearing resistance of 2000 kPa at Ultimate Limit State (ULS). The geotechnical resistance of a sustained load at SLS should be within the normally tolerated limits of 25 millimetres of settlement. The settlement of foundations placed on competent limestone bedrock is expected to be negligible. As such, the bearing resistance SLS is not provided.

5.3 Foundation Construction

The footing beds in the clayey silt/silt will be prone to disturbance from construction, foot traffic and precipitation. It would be prudent to consider the placement of a 50 mm concrete 'mud' slab over the footing bases once evaluated. This will protect the footing beds from disturbance and provide a clean working surface for the placement of formwork and reinforcing steel.

In areas where it will be necessary to provide adjacent footings at different founding elevations, the lower footing should be constructed before the higher footing, if possible. To limit stress transfer from higher footings to lower footings, the higher footing should be set below a line drawn up from the edge of the lower footing at 10 horizontal to 7 vertical. The footings to be constructed adjacent to the existing structure should 'match' the level of the existing foundations.

All footings exposed to the environment must be provided with a minimum of 1.5 m of earth cover or equivalent insulation to protect against frost damage. This frost protection would also be required if construction were undertaken during the winter months. All footings and foundations should be designed and constructed in accordance with the current Ontario Building Code. We would recommend the placement of a 50 mm thick high-density sheet of Styrofoam insulation against the exterior of the foundation walls, followed by the placement of a 10-mil sheet of 'double' polyethylene (fold' placed at 'top') to prevent frost heaving/adfreezing action.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this report and to allow changes to be made if subsurface conditions differ from the conditions identified at the borehole locations.

5.4 Seismic Design Parameters

The structure shall be designed according to Section 4.1.8 of the current Ontario Building Code, Ontario Regulation 332/12. The seismic parameter provided below are based on the subsurface soil conditions encountered at the Site as well as the depth of the investigation. The conducting of site-specific shear wave velocity testing may allow for the upgrade of the site class.

It should be noted that the values of the provided seismic design parameters were based on the 2%-in-50-year seismic hazard values and are provided in accordance with Article 4.1.8.4. of the National Building Code 2020 (NBC). The structural engineer responsible for the project should review the earthquake loads and effects.

5.4.1 Mixed use Building & Retail/Office Building (Buildings 03 and Building 04)

Based on the subsurface soil conditions encountered in the current investigation, the applicable Site Classification for the seismic design is Site Class B – bedrock, based on the average soil characteristics for the Site. The seismic data as per the 2020 National Building Code interpolated seismic hazard values, are as follows:

Sa(0.2) [g]	Sa(0.5) [g]	Sa(1.0) [g]	Sa(2.0) [g]	Sa(5.0) [g]	Sa(10.0) [g]	PGA [g]	PGV [m/s]
0.151	0.0878	0.0482	0.0231	0.00617	0.00233	0.061	0.0567

5.4.2 Slab on Grade Structures (Buildings 01, 02, 05, 06, 07, and 08)

Based on the subsurface soil conditions encountered in the previous investigation, the applicable Site Classification for the seismic design is Site Class D – stiff soil, based on the average soil characteristics for the Site. The seismic data as per the 2020 National Building Code interpolated seismic hazard values, are as follows:

Sa(0.2) [g]	Sa(0.5) [g]	Sa(1.0) [g]	Sa(2.0) [g]	Sa(5.0) [g]	Sa(10.0) [g]	PGA [g]	PGV [m/s]
0.237	0.253	0.156	0.0758	0.0202	0.00638	0.128	0.16

5.5 Floor Slab Considerations

The floor slab may be constructed using conventional slab-on-grade techniques on a prepared subgrade. The exposed subgrade surface should then be well compacted (proofrolled) in the presence of a representative of G2S. Any soft ‘spots’ delineated during this work must be sub-excavated and replaced with quality backfill material compacted to 100 percent of its Standard Proctor Maximum Dry Density (SPMDD). Imported granular fill is preferred due to its relative insensitivity to weather conditions, its relative ease in achieving the required degree of compaction, and its quick response to applied stresses. For slab-on-grade placed on bedrock surface, the exposed subgrade should be prepared by carefully examining the surface for any loose or unsuitable material (i.e., weak, disturbed, fractured rock, etc.).

As with all concrete floor slabs, there is a tendency for the floor slabs to crack. The slab thickness, concrete mix design, amount of steel and/or fibre reinforcement and/or wire mesh placed into the concrete slab, if any, will therefore be a function of the owner's tolerance for cracks in, and movements of, the slabs-on-grade, etc. The 'saw-cuts' in the concrete floors, for crack control, should extend a minimum of 1/3 the thickness of the slab.

A moisture barrier will be required under the floor slabs, such as the placement of at least 200 mm of well-compacted 19 mm clear crushed stone. At a minimum, the moisture barrier material should contain no more than 10 percent passing the No. 4 sieve

Curing of the slab-on-grade must be carefully specified to ensure that slab curl is minimized. This is especially critical during the hot summer months of the year when the surface of the slab tends to dry out quickly while high moisture conditions in the moisture barrier or water trapped on top of any 'poly' sheet at the saw cut joints and cracks, and at the edges of the slabs, maintains the underside of the slab in moist conditions.

It is also important that excess free water not be added to the concrete during its placement as this could increase the potential for shrinkage cracking and curling of the slab. Based on the conditions encountered in the boreholes, backfill recommendations, and the floor slab considerations, a modulus of sub-grade reaction, k_{v1}, of 25 MPa/m and 45 MPa/m (based on a loaded area of 300 mm x 300 mm) can be used for the design of the slab-on-grade floor slab placed on engineered fill and bedrock surface, respectively.

5.6 Lateral Earth Pressure and Perimeter Drainage

The following soil properties may be considered for the design of structures subject to an unbalanced earth load. These properties have been estimated based on our review and laboratory testing of the soil samples, which were recovered during the geotechnical investigation, as well as the type of backfill material, which is expected to be used in construction.

Table 5: Soil Properties for Design of Earth Retaining Structures

Material	φ		K_a	K_o	K_P
Fill: Earth fill – Stiff to very stiff/ compact	28	19.0	0.36	0.53	2.78
Fill: OPSS 1010 Granular B - compact	34	21.0	0.28	0.44	3.54
Clayey Silt/Silt – Soft to Compact	30	19.5	0.33	0.50	3.00
Till – Dense/Very Stiff	36	22	0.26	0.41	3.85

Φ = Angle of Internal Friction (degrees), γ = Bulk Unit Weight of Soil (kN/m^3), K_a = Active Earth Pressure Coefficient, K_o = At-rest Earth Pressure Coefficient, K_P = Active Earth Pressure Coefficient.

The following equation can be used to calculate earth pressure acting on the retaining walls including the effects of groundwater pressure:

$$p = K (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2$$

Where, p = lateral earth pressure in kPa acting at depth h;

K = earth pressure coefficient;

γ = unit weight of retained soil

h_1 = depth in meters above the water table

γ' = effective unit weight of soil

γ_w = unit weight of water (10 kN/m^3)

h_2 = depth in metres below the water table; and

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

For slab-on-grade buildings, it is recommended that a perimeter drainage system extend in all areas where the floor slab level is less than 0.3 m above the final exterior grade. In addition, a permanent perimeter drainage system should be provided around the structure to prevent the build-up of water against the basement walls. At a minimum, it is recommended that the perimeter weeping tile consists of a 150 m diameter perforated pipe with a geofabric ‘sock’, surrounded with 200 mm of 19-mm clear stone, with the stone in turn encased by a heavy geotextile filter fabric. The suppliers of the geotextile filter fabric should be consulted as to the type best suited for this project. Alternatively, vertical drainage system can be attached to the exterior basement walls such as Terradrain 600 or equivalent, which should cover the entire basement wall. The drainage system should outlet to a gravity storm sewer connection, fitted with a suitable back-flow prevention valve. In the event that a sump pump system is required, it should be constructed with an ‘oversized’ reservoir to limit pumping intervals and include an alarm in the event that the system fails to operate as per design, with a municipal water backup pump to operate during power outages and when maintenance of the main pump is required. If the structure is not designed as a watertight structure, it is recommended to install an underfloor drain. The underfloor drain should be connected to a positive outlet. Elevator pits should be drained separately with an independent lower pumping sump, or it can be designed as a watertight structure.

This office should examine the installation of the perimeter and subfloor drains. The exterior grade around the structure should be sloped away from the structure to prevent the ponding of water against the foundation walls. Additional well graded granular material should be placed and compacted in exterior sidewalk and accessibility ramp areas to reduce the effects of frost heaving. Alternatively, insulation could be placed in these areas, or a structural ‘frost’ slab should be constructed at the doorways.

If a sewer discharge permit/agreement was required by the Town of Collingwood to discharge the private water directly or indirectly into the municipal sewer or connecting the perimeter drainage system to a positive outlet was not possible, the portion of the proposed building, below grade level, could be constructed completely watertight. The basement wall for the watertight structure should be suitably waterproofed, designed, and constructed to withstand hydrostatic water pressure. The building material and the proposed construction method should be selected to accommodate the installation of the waterproofing system.

For construction where foundations are placed directly on bedrock, the factored geotechnical resistance against sliding is a function of the friction between the footing base and the surface of the bedrock and can be expressed as follows:

$$R = \mu (N \tan \varphi)$$

Where,

R = The friction between the footing(s) and the bedrock

N = The normal load acting on the bedrock

$\tan\varphi$ = The friction resistance of the bedrock

μ = The factor of safety for the ultimate limits states design (ULS) for sliding (0.8)

5.7 Excavations and Groundwater Control

Based on the investigation findings, excavation for underground garages, foundations, and site services will be carried out through the fill, native sand/sand and gravel, and into the limestone bedrock. The excavation must be completed in accordance with the current OHSA regulations. For guidance, soft soils and soils below the groundwater level are classified as Type 4. The fill material, loose to compact material could be classified as Type 3 soil. The very dense sand/sand and gravel would be classified as Type 2 soil. The limestone/dolostone would be classified as Type 1 soil. If the excavation contains more than one type of soil, the soil shall be classified as the type with the highest number. Excavation slopes steeper than those required in the Safety Act must be supported or a trench box must be provided, and a senior Geotechnical Engineer from this office should supervise the work. We note that the rate of excavation may be slowed when existing buried services and foundations, and floor slabs of the existing structures are encountered by the contractor.

The excavation of the overburden soils at the Site is not expected to pose any difficulty and can be carried out with heavy hydraulic backhoes. Where workers must enter excavations extending deeper than 1.2 m below grade, the excavation sidewalls must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulation for Construction Projects. In addition, a rock fall protection system should be considered for the protection of the workers.

A large excavator equipped with a tiger-toothed bucket in conjunction with a jackhammer or hoe ram is the preferred method of excavation to shallow depths in rock scaling. Where rock blasting is permitted, conventional rock excavation techniques such as blasting in accordance with Ontario Provincial Standard Specification (OPSS) OPSS120, controlled blasting and trim blasting may be considered. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of the cut and the relative depth of excavation into the bedrock.

Groundwater infiltration through the fill layer, sand/sand and gravel deposit, and the upper portion of bedrock is anticipated. Any water that may seep into the excavations could be removed using conventional construction ‘dewatering’ techniques, such as pumping from sumps and ditches. More water should be expected when connections are made with existing services. Surface water should be directed away from the excavations. The quality of water, the amount of water, and disposal method should be taken into consideration during tendering. In this regard, it is recommended that a number of test excavations be conducted to allow tendering contractors to observe the groundwater conditions firsthand to assess how this will affect their operations. Ontario Regulation 387/04 requires authorization from the Ministry of the Environment, Conservation, and Parks (MECP) for all water takings over 50,000 L/day. Ontario Regulation 63/16 specifies that for temporary construction dewatering at rates between 50,000 and 400,000 L/day an Environmental Activity and Sector Registry (EASR) may be obtained in lieu of a Permit to Take Water (PTTW). Dewatering at rates of more than 400,000 L/day requires a PTTW to authorize groundwater withdrawal.

It should be noted that G2S is carrying out a hydrogeological investigation at the Site and based on the results of this investigation, recommendations pertinent to the type and extent of the groundwater control will be issued under separate cover.

5.8 Pipe Bedding

The base of the excavations in the dewatered native soils should remain firm and stable., should suffice. The bedding material, as typically specified by the Town of Collingwood, should be uniformly compacted to at least 95 percent SPMDD, with special attention paid to compaction under the pipe haunches. It is recommended that the supporting bedding material be extended to at least one pipe diameter on each side and have a minimum thickness of 200 millimetres to achieve uniform pressure conditions around the pipes. The cover and bedding should be placed in lifts not exceeding 150 millimetres and compacted as specified by the local and Regional/OPSS requirements, based upon the size and type of pipe specified. The compaction should be checked during construction by in-situ relative density tests. In sections where wet/saturated or weak subgrade conditions are encountered, a minimum 300 mm thick layer of 19 millimetre Clear Stone or High-Performance Aggregate (HPA), wrapped in geotextile (i.e., Terrafix 360R or equivalent) is recommended.

5.9 Temporary Shoring

A caisson wall or soldier piles/timber lagging system may be used for the shoring system. The shoring method will depend on the settlement tolerance for the adjacent structure and buried utilities. The shoring wall must be designed and constructed as a rigid shoring system, such as a continuous interlocking caisson wall to limit adjacent soil movements/deflections.

The excavation may be supported by a temporary shoring system consisting of timber lagging and soldier piles (Figure No. 1, Appendix D). or a continuous caisson wall (Figure No. 2, Appendix D). The requirement for caisson walls is shown on Figure No. 3 in Appendix D.

The shoring system may be constructed with walers supported by rakers or soil anchors. Tieback agreements will be required for the installation of soil anchors from the neighboring properties. The shoring system must be designed by a professional engineer experienced in shoring design and the shoring system constructed by an experienced contractor. Any surcharge loads must be incorporated into the shoring design.

The structural member stiffness and stability is the responsibility of the shoring design engineer and the shoring contractor. We would recommend that a detailed condition survey for the nearby structures and roadways be conducted prior to the commencement of the excavation operation. In addition, the shoring system must be monitored for any vertical or horizontal movements during the course of construction.

The excavation must provide ‘space’ for the construction of the footings and foundation walls, with an allowance for access by workers. The shoring design should be based on the procedure detailed in the latest edition of the Canadian Foundation Engineering Manual. Lateral earth pressure, K = 0.35 can be used if small lateral deformations are acceptable, or 0.5 in fill against rigid walls.

The native sandy silt till, and limestone bedrock are capable of supporting the proposed raker footings. The raker footings supported on the dewatered, native sandy silt till at 45 degree inclination can be designed for bearing resistance of 600 kPa at ULS, and the competent limestone/dolostone bedrock at 45 degree inclination can be designed for bearing resistance of 2000 kPa at ULS.

Caisson and soldier pile toes will be made in limestone/dolostone bedrock. The horizontal resistance of the soldier pile toes will be developed by embedment below the base of excavation, where resistance is developed from passive earth pressure. The factored vertical bearing resistance for the design of the pile embedded in sound bedrock is 8 MPa. The factored lateral bearing capacity of the sound rock is 1 MPa.

If anchor support is required, the shoring system should be supported by pre-stressed soil anchors extending below the adjacent lands. The effective length of the tie-back anchor is the length extending beyond a line drawn from the base of the shoring and projecting upward at 45° angle. Anchors extended into the clayey silt till may be designed based on skin friction of 50 kPa. Anchors extended into the sandy silt till deposit and limestone bedrock may be designed based on skin frictions ranging between 60 kPa and 1000 kPa. These values depend on the anchor installation method and grouting procedures. Gravity poured concrete can result in low bond values, while pressure-grouted anchors will give higher values and produce a more satisfactory anchor.

It will be necessary to perform load tests on the tiebacks to confirm the bond stresses assumed in the design of anchors. Movement of the shoring system is anticipated. Vertical movements will result from the vertical loads on the piles resulting from the tieback. Horizontal movement will result from the soil and water pressures. The magnitude of this movement can be controlled by sound construction practices. The horizontal and vertical movement of the shoring system must be monitored, especially at locations where settlement sensitive structures are present

5.10 Backfill Considerations

The majority of the excavated material will consist of the existing fill, native clayey silt and silt material. The excavated material, apert from any organics, excessively wet, or other deleterious material, is considered to be suitable for use as service trench backfill and as engineered fill provided that the moisture content can be controlled to within 3 percent of the standard Proctor optimum value. Some moisture content conditioning of the excavated material may be required, depending upon the weather conditions experienced at the time of construction to achieve acceptable compaction densities and minimize long-term settlements. A granular pad was encountered in the area of BH102, BH109 to BH112, BH119, BH120, and BH123. The material consisted of sand and gravel and was noted to contain cobbles and boulders size material. In addition, the granular material was also found to be mixed with clayey silt in the area of BH102 and BH109. The granular material apart from the oversize particles should be suitable for use as service trenches and engineered fill backfill, provided adequate sorting is carried out. The granular backfill could remain in place at the pavement areas subject to proofrolling and evaluation at the time of construction.

We note that where backfill material is placed near or slightly above its optimum content, the potential for long-term settlements due to the ingress of groundwater and collapse of the fill structure is reduced. Correspondingly, the shear strength of ‘wet’ backfill material is also lowered, thereby reducing its ability to support construction traffic, and therefore impacting roadway construction. If the soil is well ‘dry’ of its optimum value, it will appear to be very strong when compacted but will tend to settle with time as the moisture content in the fill increases to equilibrium condition. The soils may require high compaction energy to achieve acceptable densities if the moisture content is not close to their standard Proctor optimum value. It is therefore very important that the placement moisture content of the backfill soils be within 3 percent of its standard proctor optimum moisture content during placement and compaction.

Any imported fill required in service trenches or to raise the subgrade elevation should have its moisture content within 3 percent of its optimum moisture content and meet the necessary environmental guidelines.

The backfilling and compaction operations should be monitored by a representative of G2S to monitor uniform compaction of the backfill material to project specification requirements. Close supervision is prudent in areas that are not readily accessible to compaction equipment, for instance near the end of compaction ‘runs’, and around the foundation walls. Any engineered fill should be compacted to 100 percent SPMDD. A method should be developed to assess compaction efficiency employing the on-Site compaction equipment and backfill materials during construction.

5.11 Pavement Considerations

The pavement areas should be stripped of all topsoil, organic and other unsuitable materials. The exposed subgrade should be proofrolled with 3 to 4 passes of a loaded tandem truck in the presence of a representative of G2S, immediately prior to the placement of the sub-base material. Any areas of distress revealed by this, or any other means, must be subexcavated and replaced with suitable backfill material.

Alternatively, the soft areas may be repaired by the placement of coarse aggregate, such as 50 mm clear crushed stone.

The need for subexcavations of a softened subgrade will be reduced if construction is undertaken during periods of dry weather and careful attention is paid to the compaction operations. At areas of weak subgrade, suitable geogrid product such as Tensar TriAx (TX) geogrid or equivalent can be used to stabilize the subgrade. The fill placed over shallow utilities that cut into or across the paved areas must also be compacted to 100 percent of its SPMDD.

For weak subgrade material (i.e. high plasticity clays or other unsuitable fine-grained materials), chemical subgrade stabilization can be implemented as an alternative to subexcavation and replacement with a processed granular material.

The chemical stabilization process involves the mixing of the subgrade material with hydraulic binders (typically limes, cement, or fly ash) in order to produce changes in the soil's properties, including plasticity, maximum dry density, and optimum moisture content, and therefore enhancing the strength and durability of the subgrade. Typical lime or cement to soil ratio for stabilization may range between approximately 2 and 14% depending on the soil type but is generally between 4 and 10%. The design binder ration should be determined by thorough laboratory testing and confirmed by a geotechnical engineer.

Once the mix design is determined, the binder can be spread on the subgrade, mechanically mixed-in to the appropriate depth using specialized mechanical mixing equipment, and then the subgrade can be recompacted in lifts. Once graded, the material should be allowed to cure fully. Based on the binder type and mix design, consideration should be made to ensure adequate temperature and soil pH is maintained during curing

Good drainage provisions will optimize the long-term performance of the pavement structure. The subgrade must be properly crowned and shaped to promote drainage to the subdrain system. Subdrains should be installed to intercept excess subsurface water and to prevent softening of the subgrade material. Surface water should not be allowed to pond adjacent to the outer limits of the paved area.

The most severe loading conditions on the subgrade typically occur during the course of construction, therefore, precautionary measures may have to be taken to ensure that the subgrade is not unduly disturbed by construction traffic. These measures would include minimizing the amount of heavy traffic travelling over the subgrade, such as during the placement of granular base layers.

If construction is conducted under adverse weather conditions, additional subgrade preparation may be required. During wet weather conditions, such as typically experienced during the fall and spring months, additional subgrade preparation, such as the provision of an additional depth of Granular B sub-base coarse material would be required. It is also important that the sub-base and base coarse granular layers of the pavement structure be placed as soon after exposure and preparation of the subgrade level as practical.

The suggested pavement structures, for the pavement areas, outlined in Table 6 below are based on subgrade parameters estimated on the basis of visual and tactile examinations of the on-Site soils and past experience. The outlined pavement structures may be expected to have an approximate 15 to 20 years of design life, assuming that regular maintenance is performed. Should a more detailed pavement structure design be required, Site specific traffic information would be needed, together with detailed laboratory testing of the subgrade soils.

Table 6: Suggested Pavement Structure

LAYER DESCRIPTION	COMPACTION REQUIREMENTS	LIGHT DUTY SECTIONS	HEAVY DUTY (TRUCK ROUTE)
Asphaltic Concrete Wearing course OPSS HL 3 or HL 3A	Min 92.0 % *MRD	40 millimetres	40 millimetres
Binder Course OPSS HL 8	Min 92.0 % *MRD	60 millimetres	80 millimetres
Base Course OPSS Granular A	100% **SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular B Type II	100% **SPMDD	300 millimetres	450 millimetres

* MRD denotes maximum relative density, MTO LS-264

** SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

6. Stormwater Underground Storage Facility

It is understood that a stormwater storage facility (SWSF) is considered for the storm water management at the Site. The detailed design, including the location and the proposed invert depth for the SWSF is not available at this time, however, consideration is given to installing the system within the general parking area and rooftop storage at the Grocery Store (Building 5). As per the borehole information at the area of proposed facility (BH110, BH119, BH120), the soil at the proposed founding level consisted of silt overlaying clayey silt material in BH110, and clayey silt in BH119 and BH120. The clayey silt is considered suitable to support the proposed storage facility. Therefore, it is recommended to subexcavate the subgrade to the native clayey silt deposit and backfill the area, if required, with suitable material as per Sections 5.1 and 5.8 of this report. Bearing resistance of 120 kPa at SLS and 240 kPa at ULS could be utilized for the foundation design on the native clayey silt or properly placed fill material. The infiltration rate for the clayey silt material was estimated based on the grain size analysis, which was carried out for soil samples at depths ranging between approximately 1.5 and 3.1 mbeg (~Elev. 194.3 – 191.8 m). The results of the grain size analysis are summarized in Table 7 at Section 6.1 of this report and the grain size distribution graph is included in Appendix B.

The material to be used as clay liner should be compacted to a minimum of 95% of the material Standard Proctor Maximum Dry Density (SPMDD). In addition, the material used in the liner construction should be analyzed as per LS 702/703, (ASTM D4318 and ASTM D7928) and the grain size distribution and the Atterberg Limits for this material should meet the following criteria:

1. Percent of fine particles passing Sieve Size 0.075 mm ≥ 50 percent
2. Percent of particles passing Sieve Size 0.002 mm (clay content) ≥ 20 percent
3. Percent of particles passing 2.0 mm and retained on 0.06 mm ≤ 45 percent

Clay liner material can be accepted based on the results of hydraulic conductivity laboratory testing to be completed for at least three soil samples taken from the already placed material, which should be compacted to a minimum of 95% of the material SPMDD. The hydraulic conductivity for the liner material based on the as built conditions should be in the order of 1x10-8 or less. Alternatively, a mixture of 1 part bentonite to 3.5 parts by volume of Granular 'A' conforming with the OPSS.MUNI 1010 requirements, could be used to produce the clay liner material, provided that an approved mechanical mixer is used in the production of the liner material.

The proposed storage facility should be placed on a 300 mm thick layer of Granular 'A (OPSS.MUNI1010) compacted to a minimum of 98% Standard Proctor Maximum Dry Density. For weak or excessively wet subgrade, 19 mm clear stone completely wrapped in geotextile filter fabric to prevent the migration of fines into the void spaces of the bedding material, can be used beneath the proposed storage facility. Bedding and cover requirements for the system installation should follow the designer and the proprietary system specifications. Discussion and recommendations regarding the buoyant uplift pressure due to the anticipated high groundwater level and its effect on the underground storage system are included in Section 6.2 of this report.

6.1 Soil Coefficient of Permeability

Grain size analysis by hydrometer as per LS 702/703, (ASTM D4318 and ASTM D7928) was conducted on soil samples at depths between 1.5 and 3.1 mbeg of the native clayey silt deposits. The soil samples were recovered from BH109, BH113, and BH201. These samples were tested to provide estimates of the percolation rate and coefficient of permeability for that area and depth. Utilizing the Ontario Building Code, Supplementary Standard SB-6: Percolation Time and Soil Descriptions.

Table 7: Percolation Time and Hydraulic Conductivity Based on Grain Size Analysis

Sample	Layer Description	K (cm/sec)	T (mins/cm)
BH109 – S3	Clayey Silt	10^{-6} and less	Over 50
BH113 – S5	Clayey Silt	10^{-6} and less	Over 50
BH201 – S3	Clayey Silt	10^{-6} and less	Over 50
BH201 – S5	Clayey Silt	10^{-6} and less	Over 50

As shown above, the estimated coefficient of permeability was estimated as 10^{-6} and less and the associated percolation time is over 50 mins/cm. .

The soil properties indicated are representative only of the samples tested. Note that the permeability and percolation rates have been estimated based on an approximate relationship of soil types as determined by the grain size distribution tests conducted and will vary with changing soil properties and level of compactness.

6.2 Hydraulic Uplift

The groundwater table as prevailed by the investigation is located at approximately 0.6 to 1.4 mbeg across the Site. At some locations, groundwater was found at-grade. The groundwater levels are expected to show seasonal fluctuations and vary in response to prevailing weather conditions. The underground structure/storage tanks should be designed to resist uplift from hydrostatic pressure.

The hydraulic uplift pressure, P, below the pond liner can be calculated using the following formula:

$$P = \gamma_w h_w$$

Where:

P = The hydraulic pressure acting on the base of the structure (kPa)

γ_w = Unit weight of water (10 kN/m^3)

h_w = Depth in metres of the liner below the water table

And the corresponding required liner thickness can be calculated as follows:

$$h_L = P / \gamma_L$$

Where:

γ_L = Unit weight of liner in kN/m^3

h_L = Thickness in metres of the liner and any overburden material

The groundwater table may be assumed to be at the ground surface when designing for uplift. Typical techniques to resist uplift pressure are as follows:

- Deadman anchors with straps that go over the top of the tanks. The deadman anchors are typically 0.3 to 0.6 m2 reinforced concrete beams which run the full length of the tanks.
- The deadweight of the empty tanks plus the deadweight of the backfill material above the units exceeds the maximum factored hydrostatic uplift pressure that will act on the structure/tanks.
- Concrete slab to extend below the entire footprint of the structure/tanks and protrudes about 0.3 to 0.6 m beyond either side of the tanks. The tanks can be tied to the concrete slab by straps

Groundwater control and dewatering considerations for construction at the Site will be provided in the hydrogeological report, issued under a separate cover.

7. General Comments

The comments provided in this document are intended only for the guidance of the design team. The subsoil descriptions and borehole information are only intended to describe conditions at the borehole locations. Contractors placing bids for undertaking this project should carry out due diligence to verify the results of this investigation and to determine how the subsurface conditions will affect their operations. The action of stripping topsoil and unsuitable near-surficial soils as well as the selection and placement of engineered fill should be tested by the geotechnical engineer at the time of construction. In-situ density testing should be carried out on any engineered fill placed at the Site.

All foundations should be reviewed on Site by the geotechnical engineer as they are constructed, as required by 4.2.2.3 of the Ontario Building Code (2024). If G2S is not retained to review the foundation bearing conditions or the construction of the foundations in the field, then G2S assumes no responsibility for the performance of the foundations as constructed.

The long-term performance of slabs on grade is dependent on the subgrade support conditions. Subgrades to support slabs on grade should be inspected by the geotechnical engineer prior to final construction. It is important that any engineered fill constructed beneath slabs on grade is carried out as outlined in this report.

8. Limitations

The geotechnical engineering advice and recommendations provided in this report are considered preliminary and were based on the factual information obtained during this investigation.

It may be possible that the subsurface conditions vary between and beyond the investigated borehole locations. For the purpose of this report, it is assumed that the conditions outside of and between the exact borehole locations are similar to the conditions observed in the boreholes. The change in subsurface stratigraphy reported on the borehole logs has also been interpreted based on non-continuous sampling, therefore, changes in stratigraphy as shown on the borehole logs and as discussed in this report should not be regarded as exact lines of geological change. The subsurface conditions at the Site may change with the passage of time and/or by human intervention.

The findings along with the geotechnical engineering advice and recommendations provided in this report are limited to the conditions at the Site at the time of this investigation as described herein. Conclusions presented in this report should not be construed as legal advice. If Site conditions or applicable standards change or if any additional information becomes available at a future date, changes to the findings, conclusions and recommendations in this report may be necessary.

Through any subsurface investigation by boreholes, it may not be possible to identify all aspects of the subsurface conditions at the Site that could affect construction costs, techniques, equipment, and scheduling. Contractors bidding on or undertaking work on the project must be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their interpretation of the subsurface conditions and/or their own investigations.

This report has been prepared for the sole benefit of the Client (Charis Developments Ltd.) and is intended to provide geotechnical engineering advice and recommendations based on the subsurface conditions investigated at the subject Site. This report is the copyright of G2S Consulting Inc. (G2S) and may not be used by any other person or entity without the expressed written consent of the Client and G2S. Any use which a third party makes of this report, or any reliance on decisions made based on it, is the responsibility of such third parties. G2S accepts no responsibility for damages, if any suffered by any third party as a result of decisions made or actions based on this report. It is recognized that The Town of Collingwood, in their capacity as the planning and building authority under Provincial statues, may make use of and rely upon this report cognizant of the limitations thereof, both as are expressed and implied.

Secondary review of this report was completed for general QA/QC and adherence to company standards and does not include a technical review of engineering conclusions and recommendations. This report does not address any environmental conditions such as soil and ground water chemical quality and suitability of excess soils for off-site re-use.

9. Closing Remarks

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

Yours truly,

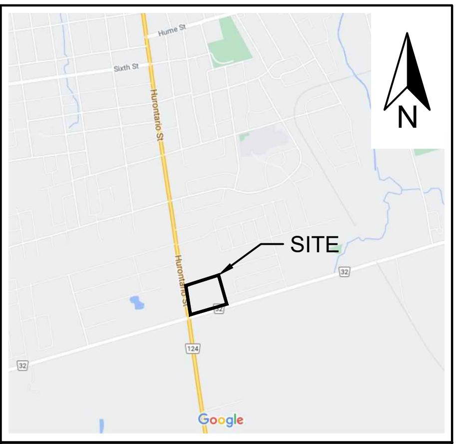
G2S Consulting Inc.

LICENSED PROFESSIONAL ENGINEER
A. ABASS
100071816
July 7, 2025
PROVINCE OF ONTARIO

Ashraf Abass, P. Eng.
Senior Project Engineer

Steve Campbell, P.Geo.
Principal

Appendix A:
Drawings



LEGEND

◉	MONITORING WELL / BOREHOLE LOCATION (CURRENT INVESTIGATION BY G2S COMPLETED IN JUNE 2024)
◕	BOREHOLE LOCATION (PREVIOUS INVESTIGATION BY G2S COMPLETED IN OCTOBER 2021)
⊙	MONITORING WELL LOCATION (PREVIOUS INVESTIGATION BY G2S COMPLETED IN OCTOBER 2021)
195.0	GROUND SURFACE ELEVATION (m)
194.6	GROUNDWATER ELEVATION (JULY 19, 2024)

REFERENCE:

PLAN REPRODUCED FROM AUTOCAD .DWG FILE "THE GATEWAY CENTRE OPTION C4BR5 2025-05-09- SITE PLAN OPTION - C4B BRANDED" PROVIDED BY CLIENT

TITLE:
BOREHOLE AND MONITORING WELL LOCATION PLAN

CLIENT:
CHARIS DEVELOPMENTS LTD.

LOCATION:
853 HURONTARIO STREET
COLLINGWOOD, ONTARIO

PROJECT NO.:	G2S21366
DRAWING:	1
SCALE:	AS SHOWN
DATE:	JULY 2025
DRAWN BY:	DB/SN/VP/NC
FILE NAME:	G2S21366.dwg

G2S

Appendix B:
Borehole Logs and
Grain Size Analysis Graphs
Plasticity Chart (2024)

C o n s u l t i n g I n c .

LIST OF ABBREVIATIONS

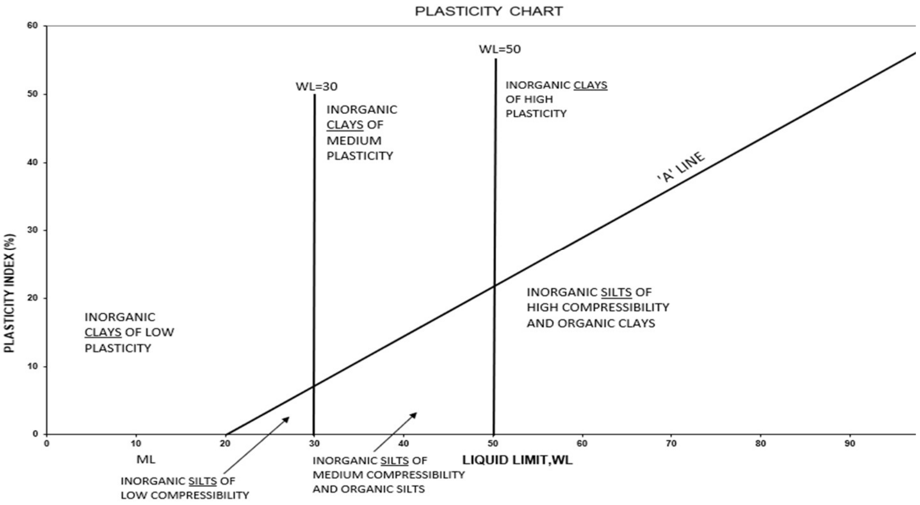
Description of Soil

The consistency of cohesive soils and the relative density or compactness of cohesionless soils are described in the following terms:

COHESIVE SOIL			COHESIONLESS SOIL	
CONSISTENCY	N (blows/0.3 m)	C (kPa)	DENSENESS	N (blows/0.3 m)
Very Soft	0 – 2	0 – 12	Very Loose	0 – 4
Soft	2 – 4	12 – 25	Loose	4 – 10
Firm	4 – 8	25 – 50	Compact	10 – 30
Stiff	8 – 15	50 – 100	Dense	30 – 50
Very Stiff	15 – 30	100 – 200	Very Dense	>50
Hard	>30	>200		
Moisture conditions				
Moist: dark or greyish color, may feel cool upon				
Wet: same as moist with free water seepage when handled				

Abbreviations

SS	Split Spoon Sample
AS	Auger Sample
GS	Grab Sample
DP	Direct Push
S	Sample
RC	Rock Core
FV/VA	Shear Vane (Field)
SPT	Standard Penetration Test
N	Blow counts per 300mm of penetration. (ASTMD1586)
MC	Moisture Content
PL	Plastic Limits
LL	Liquid Limits
PI	Plasticity Index
CF	Continuous Flight
SSA	Solid Stem Auger
HSA	Hollow Stem Auger



Penetration Resistance

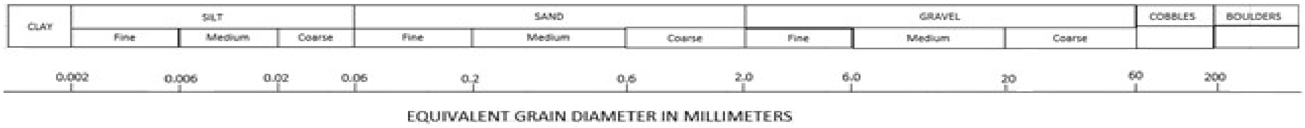
Standard Penetration Resistance N: The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m. The values reported are as noted in the field without corrections.

Soil Classification Dynamic Penetration Resistance: The number of blows required to advance a 51 mm, 60-degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow. Soils descriptions are made per the Canadian Foundations Engineering Manual (CFEM), following the International Society for Soil Mechanics and Foundation Engineering. (ISSMFE)

Notes

Soil samples will be discarded after three months unless directed otherwise by the Client.
Unless the grain size analysis is performed in our lab, soil samples are classified based on visual, tactile, and olfactory examinations, which may not be sufficient for accurate classification or precise grain sizing.

ISSMFE SOIL CLASSIFICATION



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BH/MW NUMBER 201

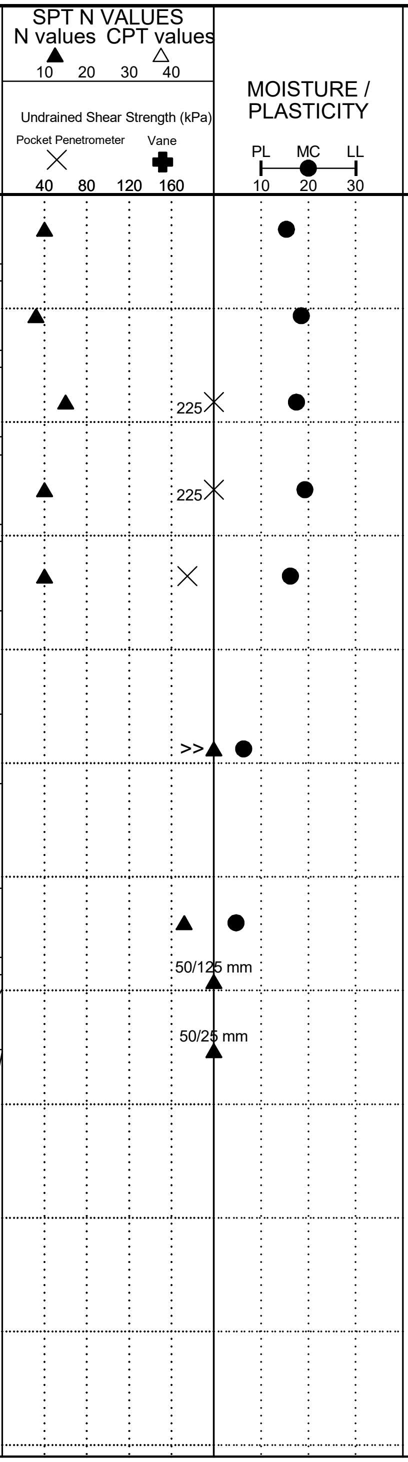
PAGE 1 OF 1

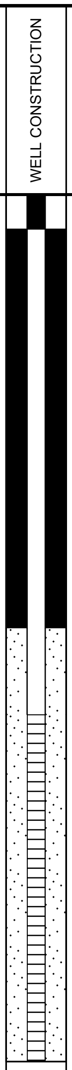
CLIENT Charis Developments Ltd.
PROJECT NAME Geotechnical Investigation for The Gateway Centre
PROJECT NUMBER G2S21366
PROJECT LOCATION 853 Hurontario St, Collingwood, ON
DATE STARTED 24-6-4 **COMPLETED** 24-6-5
GROUND ELEVATION 195.3 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 55 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)
0.05	GRANULAR: ~50 mm	195.25
	FILL: Clayey silt, greyish brown, some sand, some gravel, trace organics, moist	
1.5		193.78
	CLAYEY SILT: Brown and grey, some sand, trace organics, reworked appearance at top portion, moist, very stiff	
2.3		193.01
	becoming greyish brown, very moist, increasing plasticity with depth	
4.6		190.73
	SANDY SILT TILL: Grey, some gravel, trace clay, moist, dense to very dense	
6.9		188.44
	GRAVEL: Grey, trace silt, some sand, wet	
7.5		187.78
7.5	LIMESTONE BEDROCK: Refer to Rock Core Log for bedrock characterization details	187.76
8.0		187.35
8.1		187.25
	RUN 1: Total Recovery (100%) - RQD (26%) Poor Quality Low to Medium Strength	
9.5	RUN 2: Total Recovery (98%) - RQD (0%) Very Poor Quality Medium Strength	185.77
	RUN 3: Total Recovery (100%) - RQD (64%) Fair Quality Very Low to Medium Strength	
11.1	RUN 4: Total Recovery (99%) - RQD (89%) Good Quality Very Low to High Strength	184.20

Borehole terminated at 11.1 m.

NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
S1	SPT	10	0/0	
S2	SPT	8	0/0	
S3	SPT	15	0/0	0 3 71 26
S4	SPT	10	0/0	
S5	SPT	10	0/0	1 3 72 24
S6	SPT	55	0/0	
S7	SPT	43	0/0	
S8	SPT	50	0/0	
S9	SPT	50		
S10	RC			
S11	RC			
S12	RC			
S13	RC			





Stickup protective casing set in concrete
Bentonite seal
Filter sand
Slotted screen

Water Level Readings:

Date	Depth (m)	Elev. (m)
2024-06-20	1.10	194.20
2024-06-26	0.62	194.68
2024-07-19	0.52	194.78

BH/MW NUMBER 202

PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Charis Developments Ltd.

PROJECT NAME Geotechnical Investigation for The Gateway Centre

PROJECT NUMBER G2S21366

PROJECT LOCATION 853 Hurontario St, Collingwood, ON

DATE STARTED 24-6-4 **COMPLETED** 24-6-4

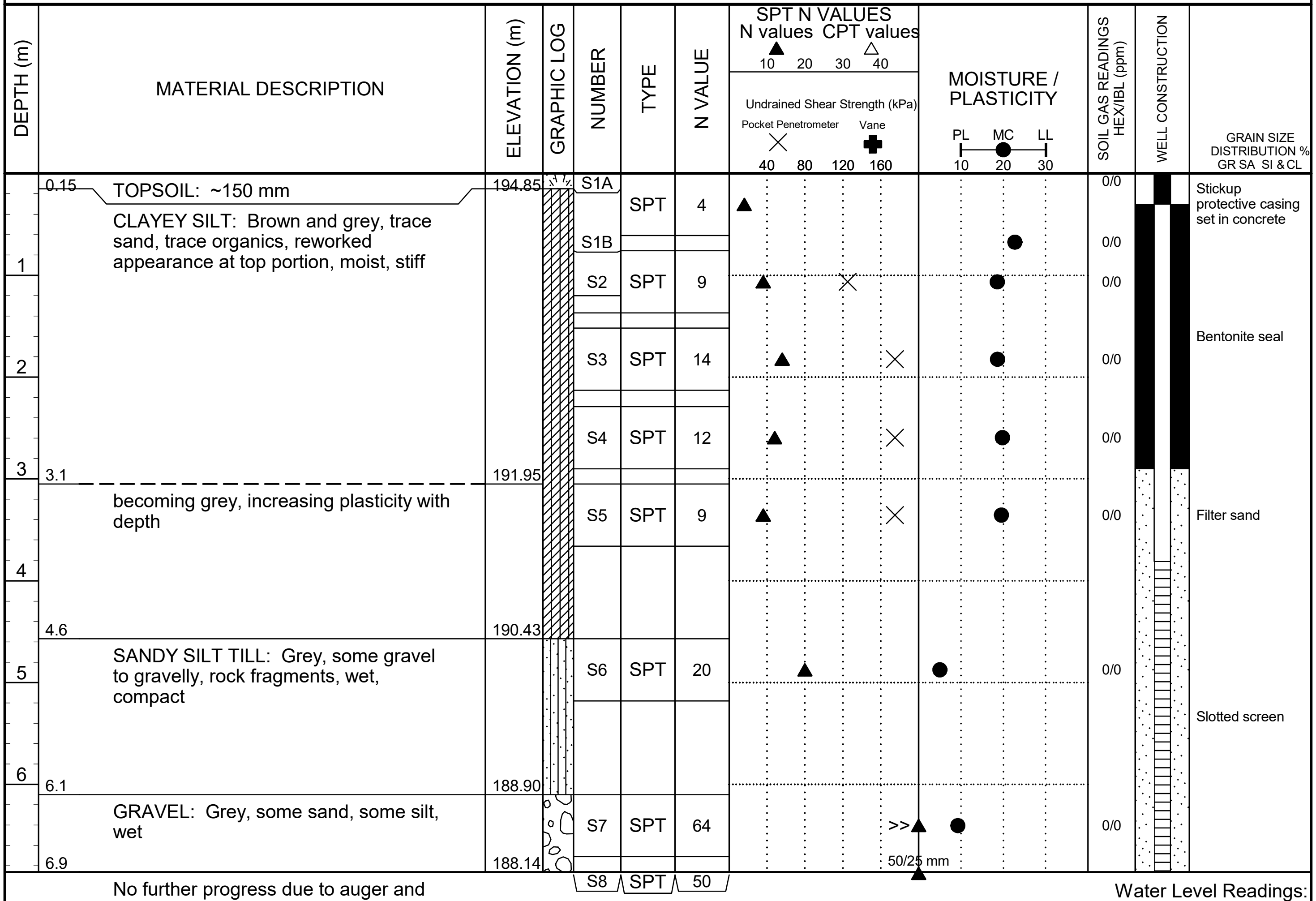
GROUND ELEVATION 195.0 m

DRILLING CONTRACTOR Davis Drilling Ltd.

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD CME 55 Track

NOTES



DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION
0.15	TOPSOIL: ~150 mm	194.85	S1A	SPT	4	0/0	Stickup protective casing set in concrete
	CLAYEY SILT: Brown and grey, trace sand, trace organics, reworked appearance at top portion, moist, stiff		S1B			0/0	
			S2	SPT	9	0/0	
			S3	SPT	14	0/0	Bentonite seal
			S4	SPT	12	0/0	
3.1	becoming grey, increasing plasticity with depth	191.95	S5	SPT	9	0/0	Filter sand
4.6		190.43					
	SANDY SILT TILL: Grey, some gravel to gravelly, rock fragments, wet, compact		S6	SPT	20	0/0	Slotted screen
6.1		188.90					
	GRAVEL: Grey, some sand, some silt, wet		S7	SPT	64	0/0	
6.9		188.14	S8	SPT	50		

No further progress due to auger and sampler refusal on probable bedrock
Borehole terminated at 6.9 m.

Water Level Readings:

Date	Depth (m)	Elev. (m)
2024-06-20	0.69	194.31
2024-06-26	0.42	194.58
2024-07-19	0.36	194.64

Explanatory Sheet To Core Log

Column No.	Description
1	Elevation of Geotechnical Boundary
2	Depth of Geotechnical Boundary in Borehole
3	Geological Symbol for Rock or Soil Material
4	General Description of Geotechnical Unit: Quantitative description including rock type(s), percentage of rock types, frequency, and sizes of interbeds, colour, texture, weathering, strength and general joint spacing
5-11	Joint (Discontinuity) Characteristics
5	Number of Joints in Set: A rock mass can be intersected by a number of joint sets of varying orientation
6	Joint Type:

B = Bedding Joint	F = Fault
C = Cross Joint	S = Shear Plane

7 Orientation: Only variations in dip can be identified in core; dip direction is obtained from field mapping or orientated core

F = Flat	= 0 – 20°
D = Dipping	= 20 – 50°
V = Vertical	= 50 – 90°

8 Joint Spacing: This is an approximate measure of spacing between joints in specific joint sets

VW = Very Wide	= >3 m
W = Wide	= 1 to 3 m
M = Moderate	= 30 cm to 1 m
C = Close	= 5 to 30 cm
VC = Very Close	= <5 cm

9 Roughness

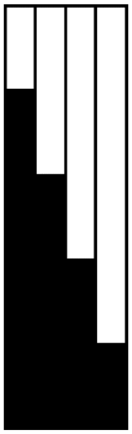
RU = Rough Undulating
RP = Routh Planar
SU = Smooth Undulating
SP = Smooth Planar
LU = Slickensided Undulating
LP = Slickensided Planar

10 Filling:

	Approximate Φ_t
T = Tight, hard, non-softening	25 – 35°
O = Oxidation, surface staining only	25 – 30°
SA = Slightly altered; clay free	25 – 30°
S = Sandy particles; clay free	25 – 35°
Si = Sandy and silty minor clay	20 – 25°
NC = Non softening clays (<5mm)	16 – 24°
SO = Softening clays (<5mm)	12 – 16°
SC = Swelling clay fillings (<5mm)	6 – 12°

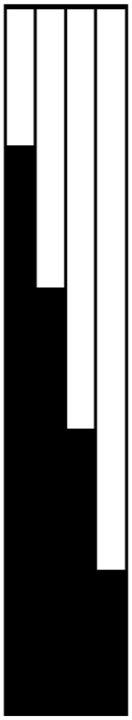
11 Aperture: Estimated size of joint opening

12 Degree of Weathering of Rock Material



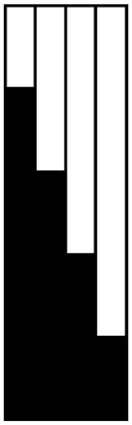
Unweathered	= no signs of discoloration or oxidation
Slightly weathered	= partial discoloration: fractures (joints) typically oxidized
Moderately weathered	= total discoloration
Highly weathered	= total discoloration: typically, friable & pitted
Completely weathered	= resembles soil: rock structure usually preserved

13 Strength of Rock Material



		Approx. Uniaxial Compressive Strength
Very High Strength	= Specimen can only be chipped by a geological hammer	>200 MPa
High Strength	= Specimen requires a number of blows to fracture it: cannot be scrapped with a pocketknife	50 – 100 MPa
Medium Strength	= Specimen can be fractured by a single blow of geological hammer; can be scrapped with pocketknife, not peeled	15 – 50 MPa
Low Strength	= Shallow indentations made with a firm blow of geological hammer; can be peeled with pocketknife with difficult	4 – 15 MPa
Very Low Strength	= Crumbles under firm blow with point of geological hammer; can be peeled by pocketknife	1 – 4 MPa

14 Fracture Frequency: Number of natural joints occurring over a mere length of core. All natural joints are counted irrespective of the number of the number of joint sets:



Fracture Frequency		Joint Spacing
<0.3 /m	=	Very wide = 3 m
0.3 – 1 /m	=	Wide = 1 – 3 m
1 – 3 /m	=	Moderate = 30 cm – 1 m
3 – 20 /m	=	Close = 5 – 30 cm
>20 /m	=	Very close = <5 cm

15 Run Number: Drill run number

16 Core Recovery: Core recovery is the total length of core pieces, irrespective of their individual lengths, obtained in a core run and expressed as a percentage of the length of that core run.

17 Rock Quality Designation (RQD): The total length of those pieces of sound core which are 10 cm or greater in length in a core run expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks

ROD	Rock Mass Classification (After Deere)
0 – 25%	Very poor
25 – 50%	Poor
50 – 75%	Fair
75 – 90%	Good
90 – 100%	excellent

18 Water Recovery: The estimated water returning out of the casing

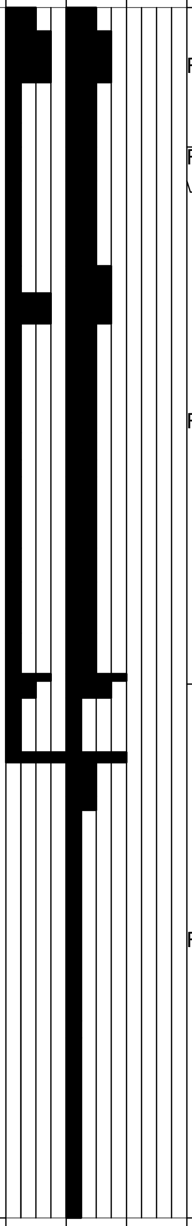
19 Water Colour: The colour of the water returning out the casing

ROCK CORE LOG

BH NO. 201

PROJECT Geotechnical Investigation	**ORIENTATION** Vertical	**ELEVATION (m)** 195.3	**DATUM**	**PROJECT NUMBER** G2S21366
LOCATION 869 Hurontario St & 7564 Poplar Side Road, Collingwood, ON	**DATE STARTED** 06-04-24	**COMPLETED** 06-05-24	**LOGGED BY** DB	**DRAWING NUMBER**
CLIENT Charis Developments Ltd.	**DRILLER** Davis Drilling Ltd.	**DRILL TYPE** HQ Rock Coring	**CORE BARREL** HQ	**SHEET** of

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	JOINT CHARACTERISTICS NO. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE (mm)	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NUMBER	RECOVERY (%)	RQD	WATER RECOVERY (%)	WATER COLOUR
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
187.8			LIMESTONE/DOLOSTONE BEDROCK		S	D	C	SU	O									
					C	F	C	RU	Si					RUN 1	100	26	100	Grey
					B	F	C	RP	SA									
					F	V	C	RU	SA									
	8													RUN 2	98	0	100	Grey
					B	F	C	RU	SA									
					B	D	C	RU	SA									
					F	V	C	RP	SA									
					B	F	C	RP	Si									
					B	F	M	RU	SA					RUN 3	100	64	100	Grey
					C	D	C	RP	SA									
					B	V	VC	RU	SA									
	9																	
					B	F	C	RU	O									
					B	F	C	RU	O									
					B	F	C	RU	Si									
					B	F	C	RU	Si									
					B	F	C	RU	SA									
					B	F	VC	RU	Si									
					B	F	C	RU	SA									
					B	F	VC	RU	SA									
					B	D	C	RP	SA									
					B	D	VC	RP	SA									
	10																	
														RUN 4	99	89	100	Grey
					B	F	M	RU	SA									
					B	F	VC	RU	SA									
	11																	
					B	F	M	RU	SA									
184.2			End of Borehole at 11.1 m															

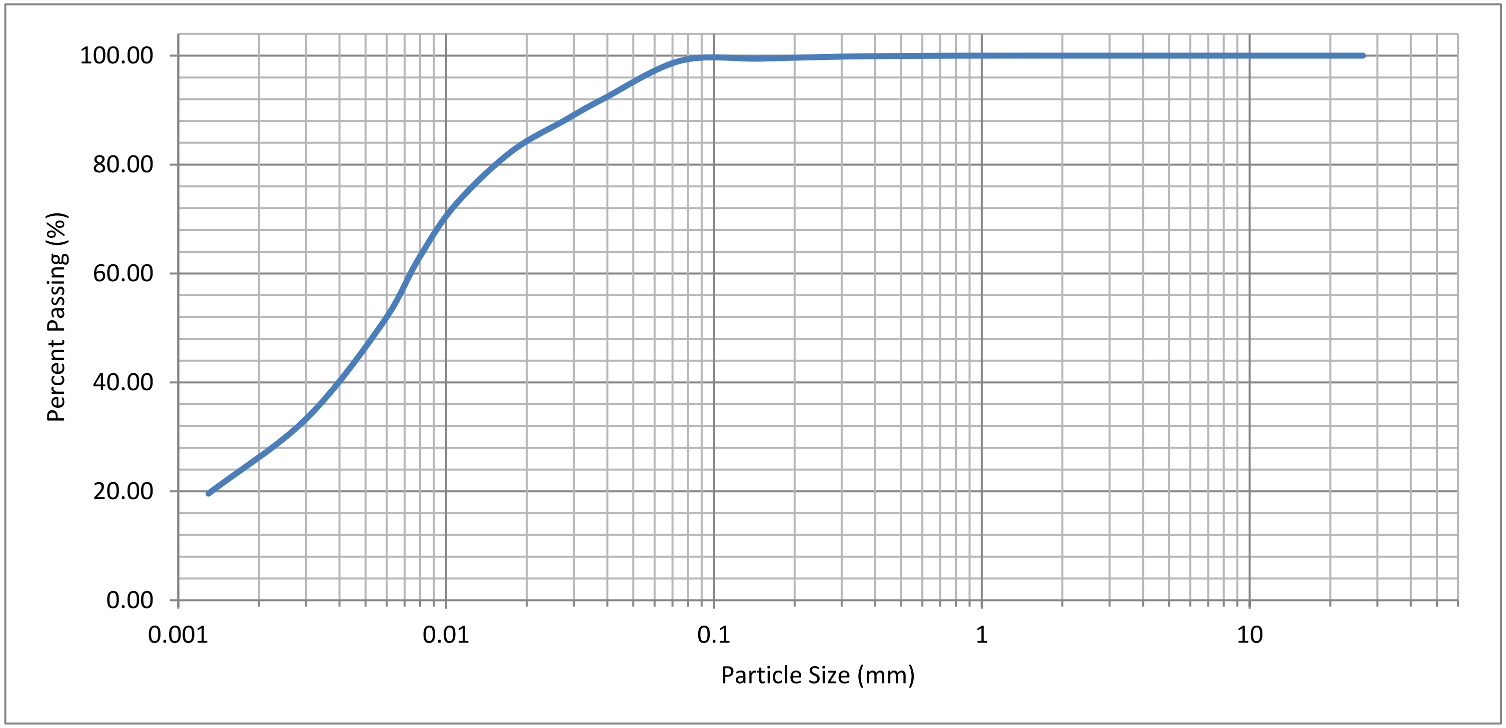


Project No.:	G2S21366D	**Lab No.:**	24132A
Project Name:	839, 853, 869 Hurontario St. & 7564 Poplar Sideroad, Collingwood	**Borehole/Sample No.:**	BH201-S3

ISSMGE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL		
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE

SIEVE SIZE: 1 2 6 20 60#200 #100 #50 #16 #8 #4 3/8" 3/4" 2-1/2"

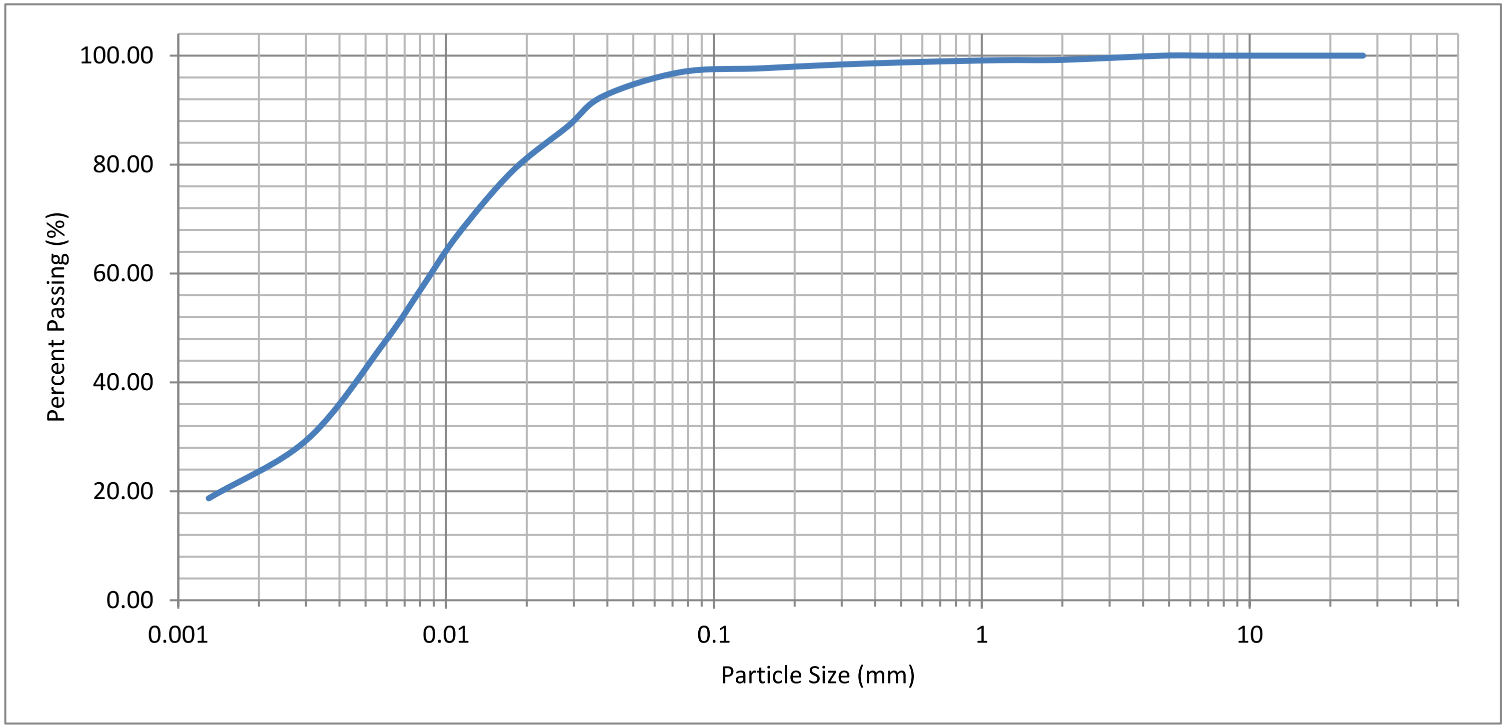


Project No.:	G2S21366D	**Lab No.:**	24132B
Project Name:	839, 853, 869 Hurontario St. & 7564 Poplar Sideroad, Collingwood	**Borehole/Sample No.:**	BH201-S5

ISSMGE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL		
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE

SIEVE SIZE: 1 2 6 20 60 #200 #100 #50 #16 #8 #4 3/8" 3/4" 2-1/2"



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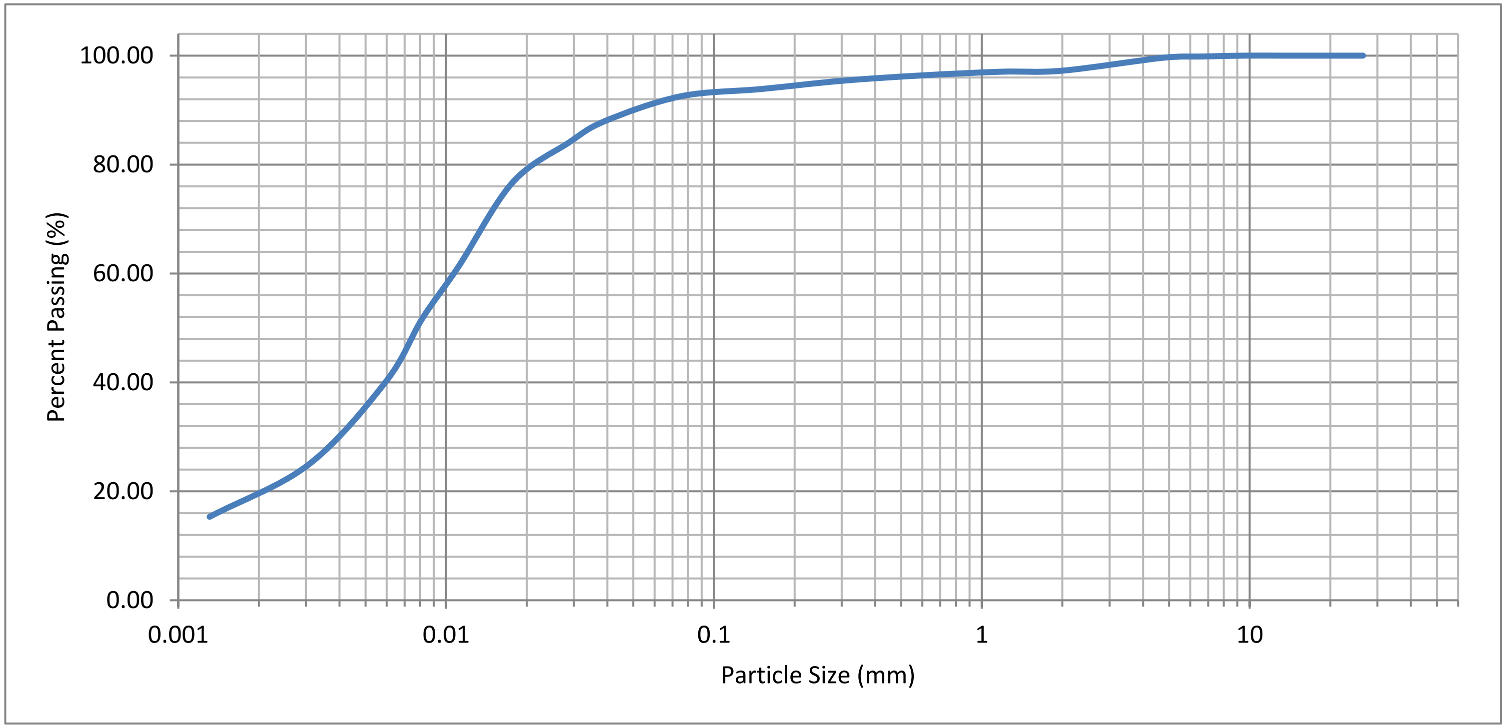
G2Senvironmental.com

Project No.:	G2S21366D	**Lab No.:**	24132C
Project Name:	839, 853, 869 Hurontario St. & 7564 Poplar Sideroad, Collingwood	**Borehole/Sample No.:**	BH202-S5

ISSMGE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL		
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE

SIEVE SIZE: 1 2 6 20 60#200 #100 #50 #16 #8 #4 3/8" 3/4" 2-1/2"



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4361 Harvester Rd., Unit 12, Burlington, ON L7L 5M4
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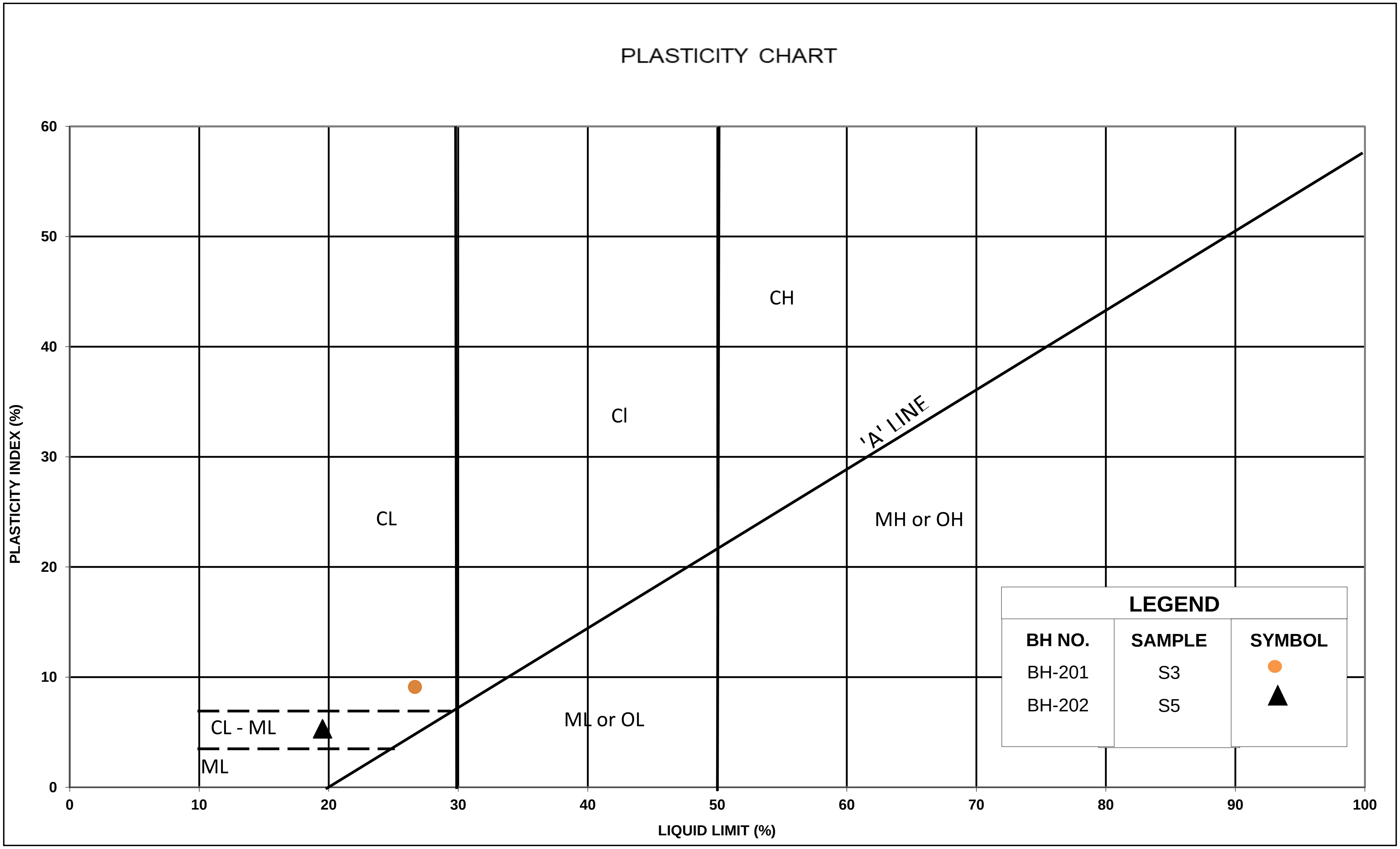
Project No.: G2S21366D **Lab No.:** 24132A&C
Project Name 839, 853, 869 Hurontario St. & 7564 Poplar Sideroad **Borehole/Sample No.:** BH201-S3/BH202-S5


PLASTICITY CHART

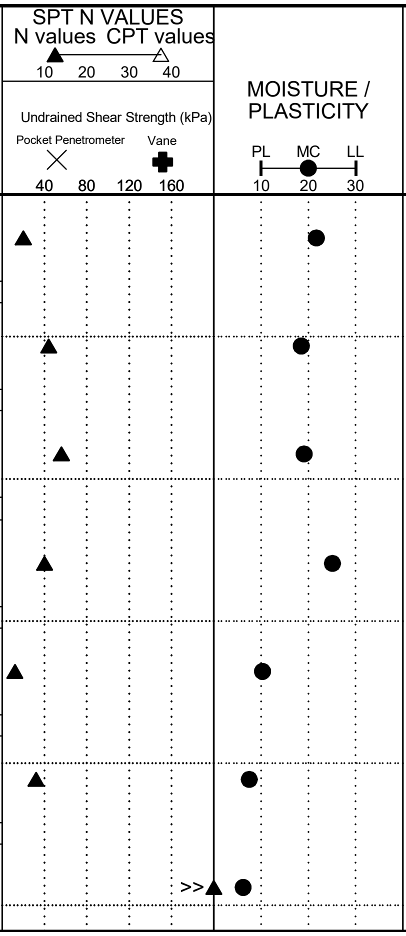
Borehole Logs and
Grain Size Analysis Graphs
Plasticity Chart (2021)

BOREHOLE NUMBER 101

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-22 **COMPLETED** 21-10-22
GROUND ELEVATION 194.77 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.36	TOPSOIL: ~360 mm	194.41		SS1	SPT	5			
1.2	FILL: Sand, brown to dark brown, some silt, some clay, moist, debris	193.55		SS2	SPT	11			
3.1	CLAYEY SILT: Brown to grey, occasional sand seams, some clay, moist to very moist, stiff	191.72		SS3 SS4	SPT SPT	14 10			
4.6	SILT: Grey, some gravel, very moist, very loose to loose	190.17		SS5 SS6	SPT SPT	3 8			
5.2	SANDY SILT TILL: Grey, some gravel, moist, dense	189.59		SS7	SPT	55			



Borehole terminated at 5.2 m.

Upon completion of augering
No cave
Free water at 3.9 m

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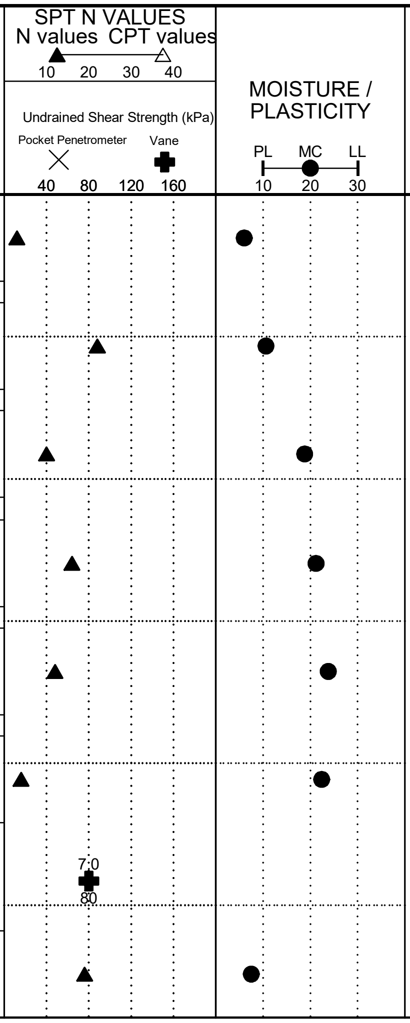
BOREHOLE NUMBER 102

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-21 **COMPLETED** 21-10-21
GROUND ELEVATION 195.51 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.65	FILL: Sand and gravel, brown, some silt, cobble and boulder size material on the surface, moist	194.86		SS1	SPT	3	20/0		
1.5	becoming clayey silt, brown, some gravel, some silt, moist	193.99		SS2	SPT	22	30/0		
2.3	CLAYEY SILT: Brown to grey, some sand, stiff	193.22		SS3	SPT	10	25/0		
5.2	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist to wet, loose to compact	190.33		SS4	SPT	16	10/0		
				SS5	SPT	12	15/0		2 6 77 15
				SS6	SPT	4	0/0		
					VANE				
5.8	SANDY SILT TILL: Grey, some gravel, trace clay, moist, compact	189.72		SS7	SPT	19	5/0		



Borehole terminated at 5.8 m.

Upon completion of augering
Wet cave at 4.6 m
Free water at 0.65 m after 24 hours

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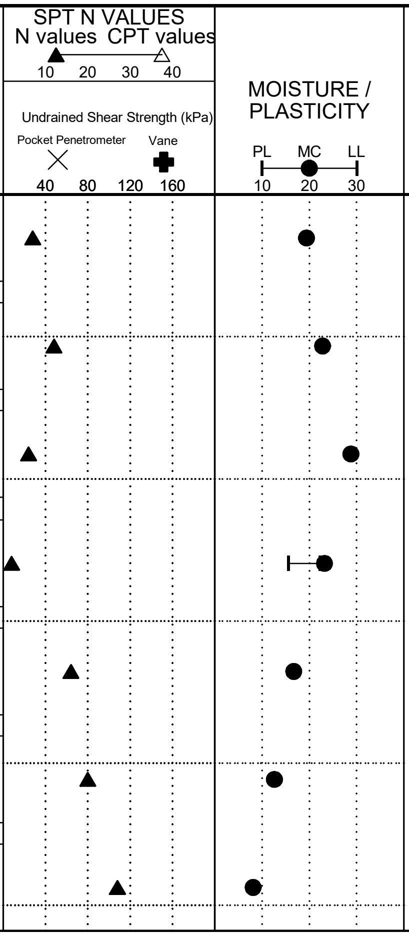
BOREHOLE NUMBER 103

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-22 **COMPLETED** 21-10-22
GROUND ELEVATION 194.39 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.13	TOPSOIL: ~125 mm	194.27				
1.00	FILL: Clayey silt, brown to grey, some sand, moist	193.39	SS1	SPT	7	
1.5	CLAYEY SILT: Brown, trace sand, rootlets, moist, stiff, reworked appearance at the upper section	192.89	SS2	SPT	12	
4.0	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist to wet, very loose to compact	190.39	SS3	SPT	6	
			SS4	SPT	2	2 6 68 24
			SS5	SPT	16	
5.2	SANDY SILT TILL: Grey, some gravel, moist, compact	189.21	SS6	SPT	20	
			SS7	SPT	27	



Borehole terminated at 5.2 m.

Upon completion of augering
Cave at 4.3m
Free water at 4.1 m

BOREHOLE NUMBER 104

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CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-10-22 **COMPLETED** 21-10-22

GROUND ELEVATION 194.40 m

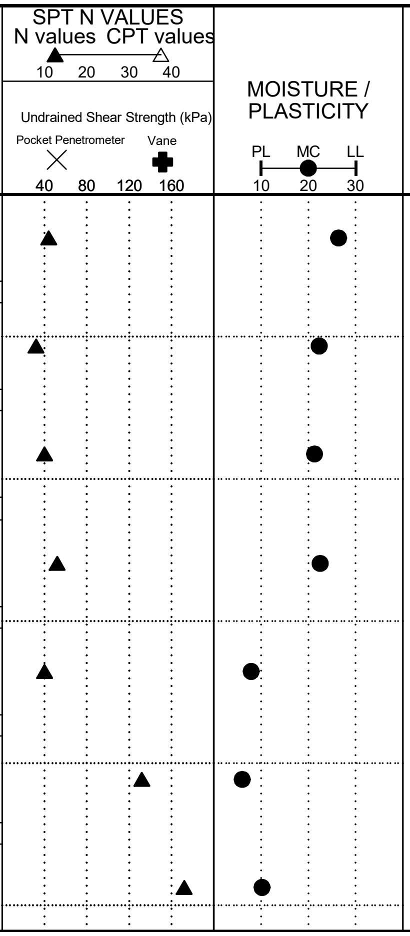
DRILLING CONTRACTOR LST

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Diedrich D50 Track

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.10	TOPSOIL: ~100 mm	194.30						
0.78	FILL: Clayey silt, brown and grey mottled, some sand, moist	193.62	SS1	SPT	11	0/0		
1.5	CLAYEY SILT: Brown to grey, some sand, reworked appearance at top, moist, stiff	192.90	SS2	SPT	8	0/0		
	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist to wet, compact		SS3	SPT	10	0/0		
			SS4	SPT	13	0/0		
3.8		190.59	SS5	SPT	10	0/0		
	SANDY SILT TILL: Grey, some gravel, moist, dense		SS6	SPT	33	0/0		
5.2		189.22	SS7	SPT	43	0/0		



Borehole terminated at 5.2 m.

Upon completion of augering
No cave
Free water at 4.4 m

BOREHOLE NUMBER 105

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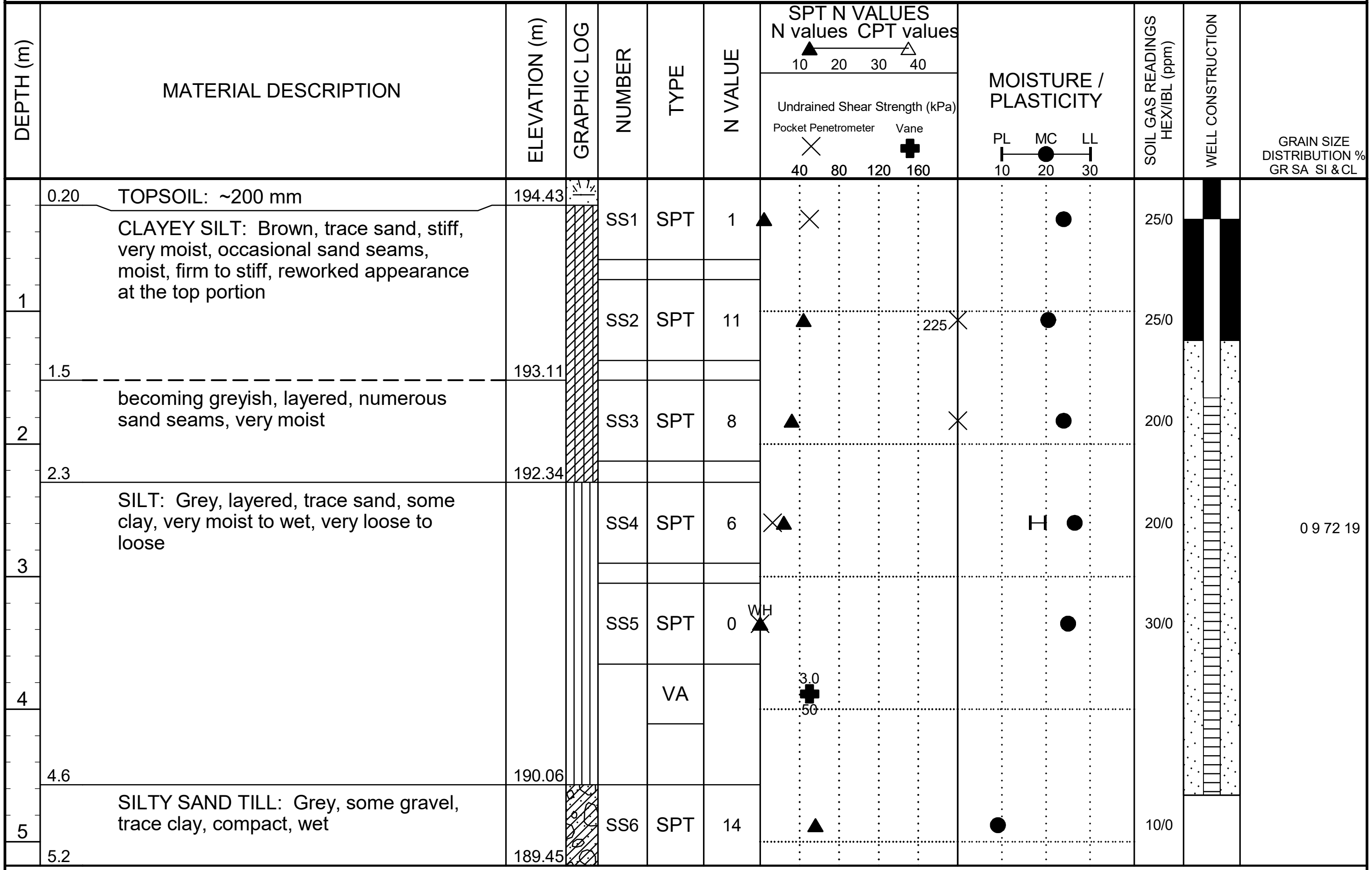
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-1 **COMPLETED** 21-10-1
GROUND ELEVATION 194.63 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES



DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	Undrained Shear Strength (kPa)	SOIL GAS READINGS HEX/IBL (ppm)	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.20	TOPSOIL: ~200 mm	194.43						
	CLAYEY SILT: Brown, trace sand, stiff, very moist, occasional sand seams, moist, firm to stiff, reworked appearance at the top portion		SS1	SPT	1		25/0	
			SS2	SPT	11	225	25/0	
1.5	becoming greyish, layered, numerous sand seams, very moist	193.11	SS3	SPT	8		20/0	
2.3	SILT: Grey, layered, trace sand, some clay, very moist to wet, very loose to loose	192.34	SS4	SPT	6		20/0	0 9 72 19
			SS5	SPT	0		30/0	
				VA		3.0 50		
4.6	SILTY SAND TILL: Grey, some gravel, trace clay, compact, wet	190.06	SS6	SPT	14		10/0	
5.2		189.45						

Borehole terminated at 5.2 m.

Refer to report for groundwater elevation data

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BOREHOLE NUMBER 106

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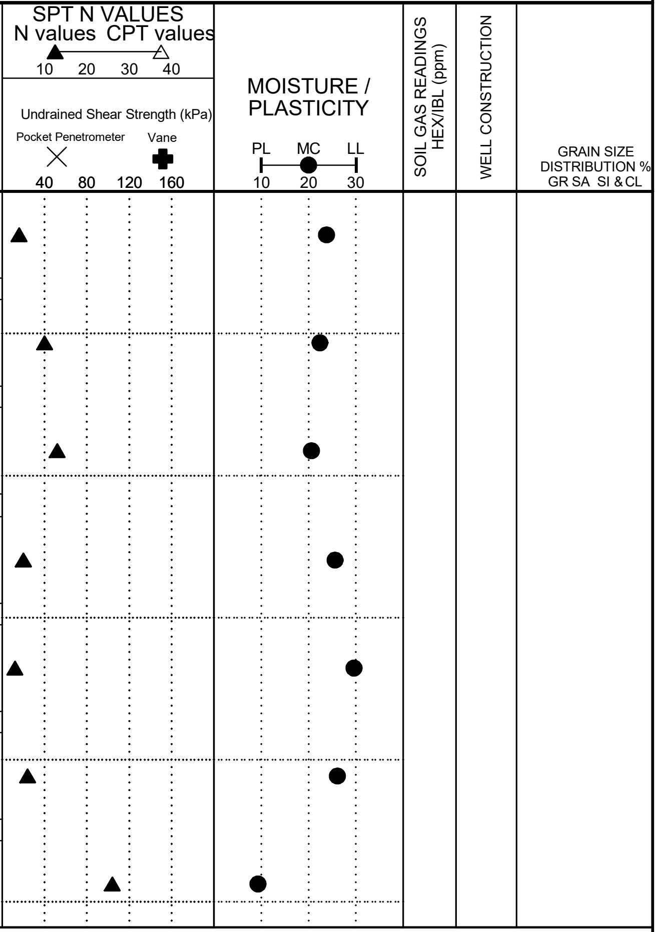
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-22 **COMPLETED** 21-10-22
GROUND ELEVATION 194.54 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE
0.08	TOPSOIL: ~75 mm	194.47			
0.76	FILL: Clayey silt, brown and grey mottled, some sand, very moist, reworked native	193.78	SS1	SPT	4
2.3	CLAYEY SILT: Brown, occasional sand seams, moist, stiff	192.25	SS2 SS3	SPT SPT	10 13
3.1	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist to wet, very loose to loose	191.49	SS4	SPT	5
4.6	trace gravel, occasional sand seams	189.97	SS5 SS6	SPT SPT	3 6
5.2	SILTY SAND TILL: Grey, some gravel, very moist, compact	189.36	SS7	SPT	26



Borehole terminated at 5.2 m.

Upon completion of augering
Cave at 4.3 m
Free water at 2.1 m

BOREHOLE NUMBER 107

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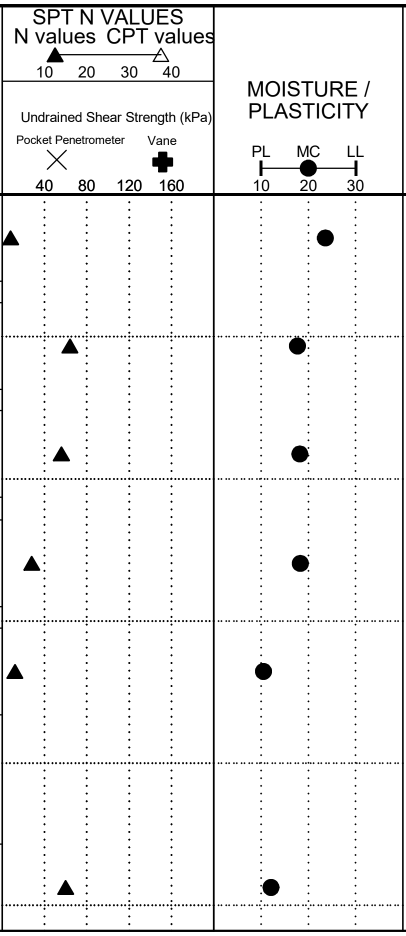
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-21 **COMPLETED** 21-10-21
GROUND ELEVATION 194.78 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.25	TOPSOIL: ~250 mm	194.53				
0.76	FILL: Sand, brown, some silt, very moist	194.02	SS1	SPT	2	
2.3	CLAYEY SILT: Brown, some sand, moist, very stiff	192.49	SS2	SPT	16	
			SS3	SPT	14	
5.2	SILTY SAND TILL/SANDY SILT TILL: Grey, some gravel, some clay, moist to very moist, very loose to compact	189.60	SS4	SPT	7	
			SS5	SPT	3	10 36 35 19
			SS6	SPT	15	



SOIL GAS READINGS HEX/IBL (ppm)
WELL CONSTRUCTION

Borehole terminated at 5.2 m.

Upon completion of augering
No cave
Free water at 3.0 m
Free water at 0.3 m after 24 hours

BOREHOLE NUMBER 108

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CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-10-21 **COMPLETED** 21-10-21

GROUND ELEVATION 195.37 m

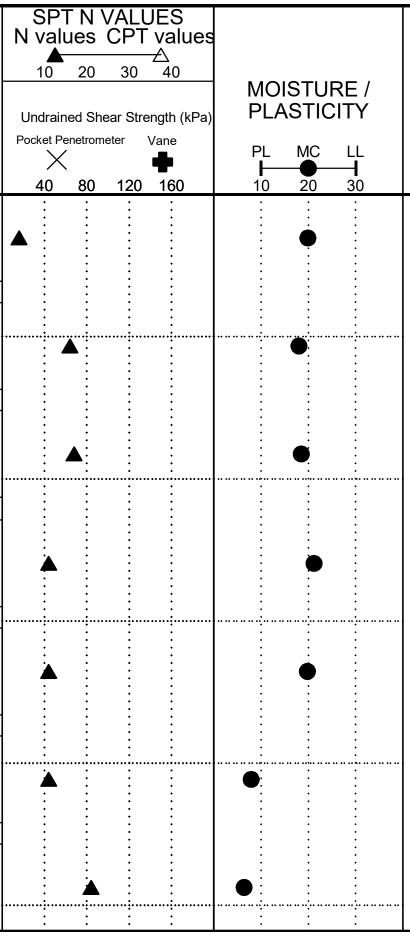
DRILLING CONTRACTOR LST

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Diedrich D50 Track

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.10	TOPSOIL: ~100 mm	195.27						
	FILL: Clayey silt, brown to dark brown, some sand, very moist		SS1	SPT	4	10/0		
0.76	CLAYEY SILT: Brown and grey mottled, some sand seams, moist, very stiff	194.61	SS2	SPT	16	0/0		
1.5	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist to very moist, compact	193.87	SS3	SPT	17	0/0		
			SS4	SPT	11	10/0		
			SS5	SPT	11	5/0		
3.8	SANDY SILT TILL: Grey, some gravel, moist, compact,	191.56	SS6	SPT	11	0/0		
			SS7	SPT	21	0/0		
5.2		190.19						



Borehole terminated at 5.2 m.

Upon completion of augering
Cave at 4.4 m
Free water at 3.4 m

BOREHOLE NUMBER 109

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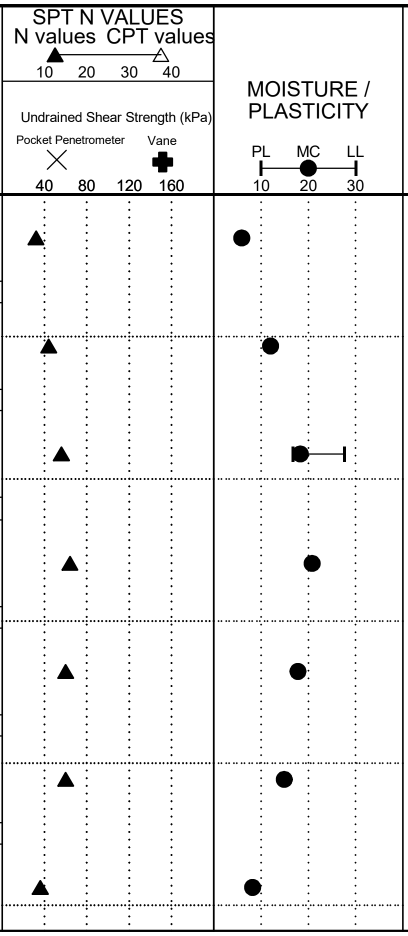
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-21 **COMPLETED** 21-10-21
GROUND ELEVATION 195.81 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.95	FILL: Sand and gravel, brown, some silt to silty, possible cobble and boulder size material at the surface, moist	194.86	SS1	SPT	8	25/0		
1.5	becoming clayey silt, brown and grey, some sand, some gravel, moist	194.29	SS2	SPT	11	25/0		
	CLAYEY SILT: Brown to grey mottled, trace sand, moist, stiff to very stiff		SS3	SPT	14	30/0		0 2 64 34
3.1		192.76	SS4	SPT	16	35/0		
3.8	becoming layered with trace gravel	192.00	SS5	SPT	15	20/0		
4.6	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist, compact	191.24	SS6	SPT	15	15/0		
5.2	SANDY SILT TILL: Grey, some gravel, wet, loose	190.63	SS7	SPT	9	15/0		



Borehole terminated at 5.2 m.

Upon completion of augering
No cave
No free water

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BH/MW NUMBER 110

CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-10-1 **COMPLETED** 21-10-1

GROUND ELEVATION 195.87 m

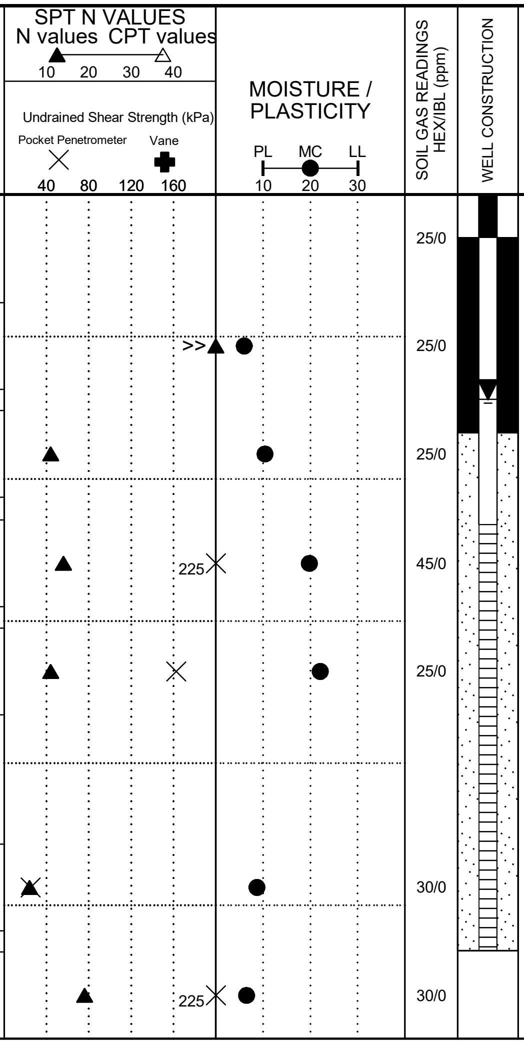
DRILLING CONTRACTOR Davis

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD CME 45 Track

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
1.6	FILL: Sand and gravel, brown, trace to some silt, possible cobble and boulder size material at the surface, moist	194.24	SS1 SS2	AU SPT	 52	25/0 25/0	
2.3	SILT: Brown to grey, trace sand, some gravel, very moist, compact, reworked appearance at top portion	193.58	SS3	SPT	11	25/0	
4.6	CLAYEY SILT: Brown to grey mottled, trace sand, trace gravel, moist, stiff	191.30	SS4 SS5	SPT SPT	14 11	45/0 25/0	
5.3	SILT: Grey, occasional sand pockets, some clay, moist, loose	190.54	SS6	SPT	6	30/0	
5.9	SANDY SILT TILL: Grey, some gravel, trace clay, very moist, compact	189.93	SS7	SPT	19	30/0	



Borehole terminated at 5.9 m.

Water Level Readings:

Date	Depth (m)	Elev. (m)
2021-11-03	0.90	194.97
2024-06-05	1.30	194.57
2024-06-26	1.30	194.57
2024-07-19	1.30	194.57
2022-04-05	1.33	194.54
2022-01-21	1.44	194.43
2024-06-20	1.50	194.37
2021-10-13	2.00	193.87
2025-05-16	1.30	194.57

BOREHOLE NUMBER 111

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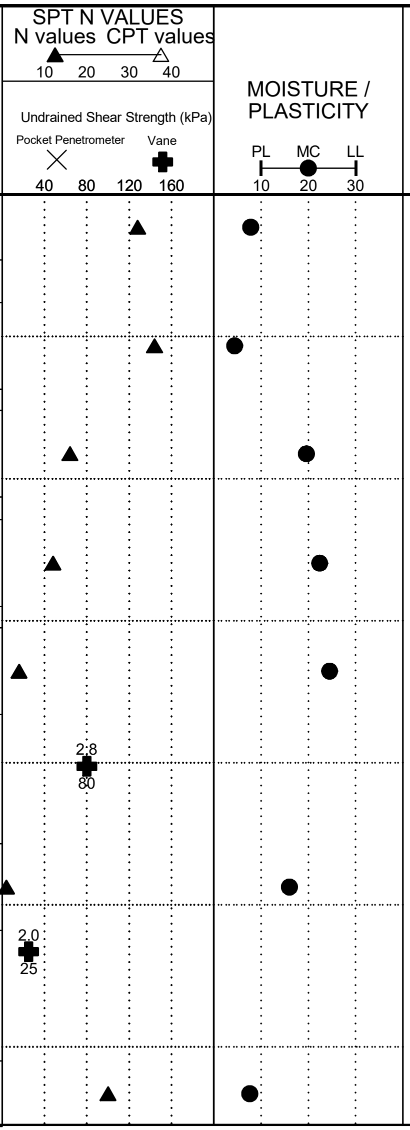
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-9-30 **COMPLETED** 21-9-30
GROUND ELEVATION 195.72 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	Undrained Shear Strength (kPa) Vane
	FILL: Sand and gravel, brown, some silt, cobble and boulder size material at the surface, moist		SS1	SPT	32	
			SS2	SPT	36	
1.5	CLAYEY SILT: Brown to grey, trace sand, some gravel, moist to very moist, stiff to very stiff, reworked appearance at top portion	194.22	SS3	SPT	16	
			SS4	SPT	12	
3.1	SILT: Grey, layered, trace sand, trace gravel, some clay, wet, very loose	192.67	SS5	SPT	4	
				VANE		2.8 80
			SS6	SPT	1	
				VANE		2.0 25
6.1	SILTY SAND TILL: Grey, some gravel, trace clay, compact, wet	189.62	SS7	SPT	25	
6.6		189.17				



SOIL GAS READINGS HEX/IBL (ppm)

WELL CONSTRUCTION

GRAIN SIZE DISTRIBUTION % GR SA SI & CL

Borehole terminated at 6.6 m.

Upon completion of augering
Cave at 5.8 m
Free water at 1.2 m

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BOREHOLE NUMBER 112

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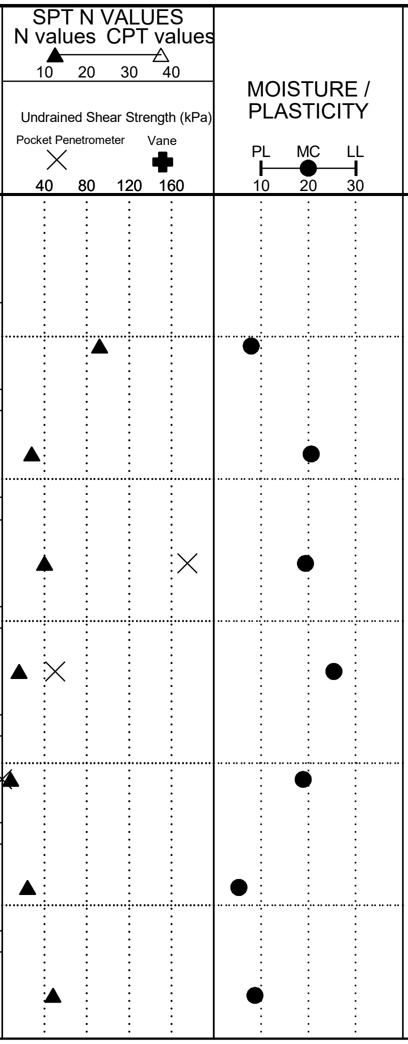
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-9-30 **COMPLETED** 21-10-1
GROUND ELEVATION 195.87 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
	FILL: Sand and gravel, brown, some silt, possible cobble and boulder sized material at surface, moist			SS1	AU		15/0		
1				SS2	SPT	23	20/0		
1.5		194.37							
2	SILT: Greyish brown, layered, trace sand, trace gravel, some clay moist to wet, very loose to loose			SS3	SPT	7	10/0		
3				SS4	SPT	10	20/0		
				SS5	SPT	4	15/0		
4				SS6	SPT	2	20/0		
5				SS7	SPT	6	20/0		
5.3		190.54							
	SANDY SILT TILL: Grey, some gravel, very moist, compact			SS8	SPT	12	15/0		
5.9		189.93							



Borehole terminated at 5.9 m.

Upon completion of augering
Cave at 3.0 m
Free water at 3.1 m

G2S Consulting Inc.

BOREHOLE NUMBER 113

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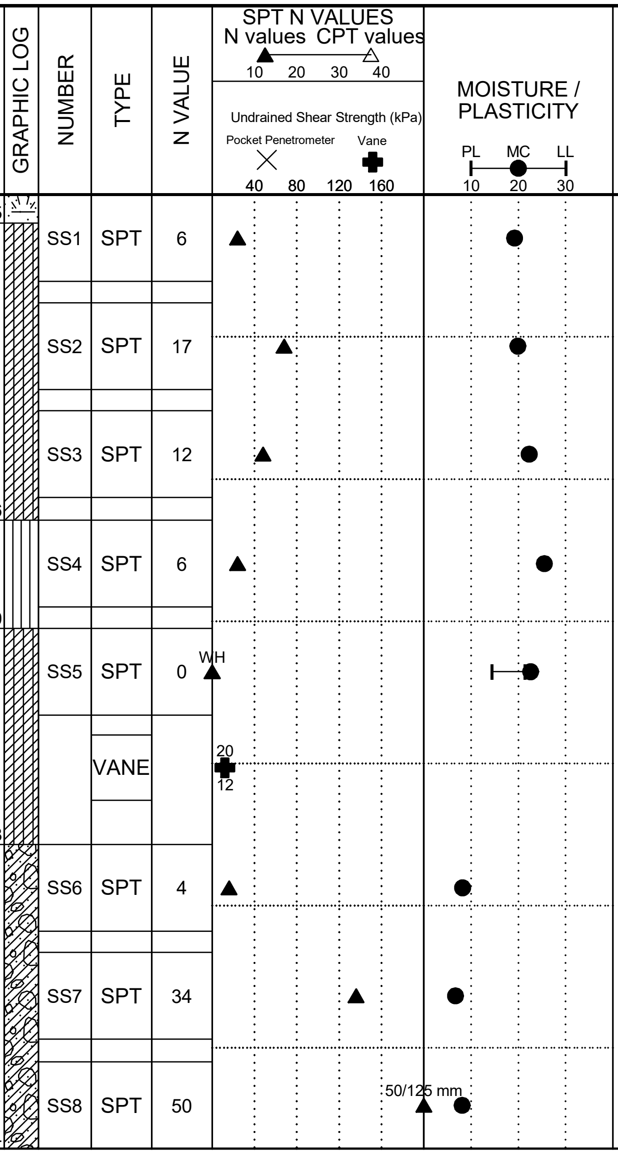
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-9-30 **COMPLETED** 21-9-30
GROUND ELEVATION 194.85 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.20	TOPSOIL: ~200 mm	194.65				
	CLAYEY SILT: Brown to grey, trace sand, trace organics, moist, firm to very stiff, reworked appearance at the top portion		SS1	SPT	6	
1			SS2	SPT	17	
2			SS3	SPT	12	
2.3		192.56				
	SILT: Greyish brown, layered, trace sand, trace gravel, some clay moist to wet, very loose to loose		SS4	SPT	6	
3						
3.1		191.80				
	CLAYEY SILT: Brown to grey, trace sand, trace gravel, moist, very soft		SS5	SPT	0	1 3 68 28
4				VANE		
4.6		190.28				
5	SILTY SAND TILL/SANDY SILT TILL: Grey, some gravel, some clay, very moist, loose to very dense		SS6	SPT	4	16 38 28 18
6			SS7	SPT	34	
			SS8	SPT	50	
6.7		188.14				



SOIL GAS READINGS HEX/IBL (ppm)

WELL CONSTRUCTION

Borehole terminated at 6.7 m.

Upon completion of augering
No cave
Free water at 2.3 m

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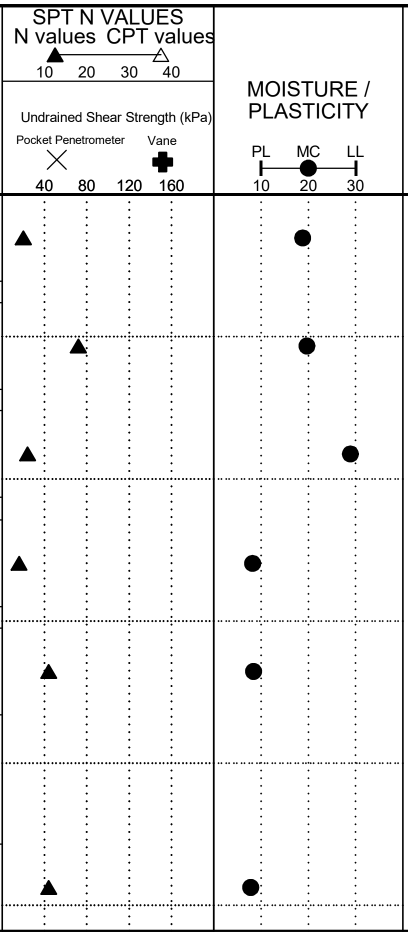
BOREHOLE NUMBER 114

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-9-30 **COMPLETED** 21-9-30
GROUND ELEVATION 195.01 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.10	TOPSOIL: ~100 mm	194.91						
	CLAYEY SILT: Brown to grey, some sand, trace organics, occasional sand seams, moist, firm to very stiff, reworked appearance at the top portion		SS1	SPT	5			
			SS2	SPT	18			
			SS3	SPT	6			
2.3		192.72						
	SILT: Grey, layered, trace sand, trace gravel, some clay, moist, soft		SS4	SPT	4			
3.1		191.96						
	SANDY SILT TILL: Grey, trace to some gravel, trace clay, compact, wet		SS5	SPT	11			
			SS6	SPT	11			
5.2		189.83						



Borehole terminated at 5.2 m.

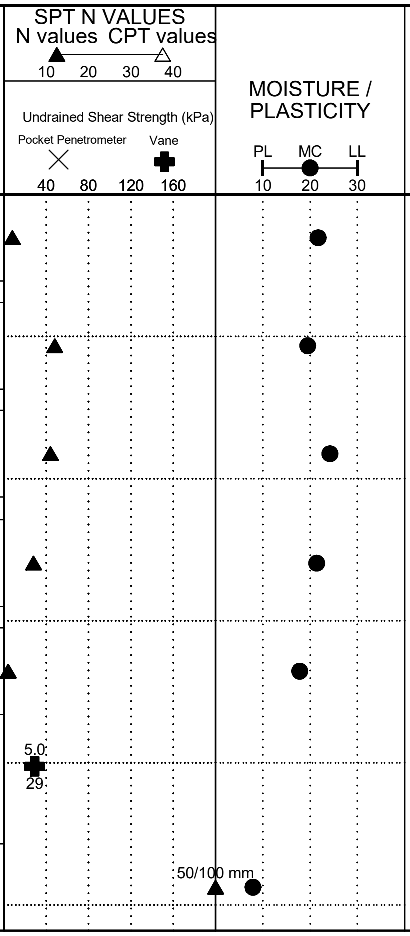
Upon completion of augering
No cave
Free water at 3.7 m

BOREHOLE NUMBER 115

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-1 **COMPLETED** 21-10-1
GROUND ELEVATION 195.28 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.25	TOPSOIL: ~250 mm	195.03							
0.76	FILL: Clayey silt, brown, some sand, trace gravel, moist	194.52		SS1	SPT	2			
2.3	CLAYEY SILT: Brown to grey, trace sand, trace organics, very moist, stiff, reworked appearance at the top portion	192.99		SS2 SS3	SPT SPT	12 11			
4.6	SILT: Greyish brown, layered, trace sand, trace gravel, some clay, very moist to wet, very loose to loose	190.71		SS4 SS5 	SPT SPT VANE	7 1 			
5.2	SANDY SILT TILL: Grey, some gravel, trace clay, very dense, wet	190.10		SS6	SPT	50			



Borehole terminated at 5.2 m.

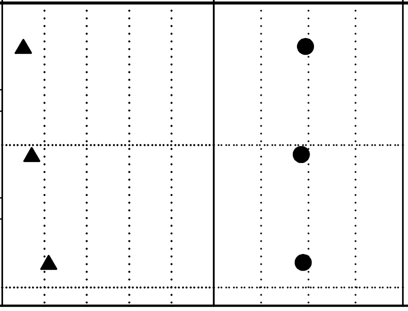
Upon completion of augering
Cave at 3.9 m
Free water at 4.0 m

BOREHOLE NUMBER 116

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-22 **COMPLETED** 21-10-22
GROUND ELEVATION 195.22 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SPT N VALUES (N values ▲, CPT values △) 10 20 30 40 / Undrained Shear Strength (kPa) Pocket Penetrometer × Vane ✚ 40 80 120 160	MOISTURE / PLASTICITY PL MC LL 10 20 30	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
	FILL: Sandy silt, dark brown to brown, trace clay, trace gravel, trace organics, moist		▨	SS1	SPT	5	▲	●			
1 1.1		194.15	▨	SS2	SPT	7	▲	●			
	CLAYEY SILT: Brown, some sand, reworked appearance at top portion, moist, stiff		▥								
2 2.1		193.09	▥	SS3	SPT	11	▲	●			



Borehole terminated at 2.1 m.

Upon completion of augering
No cave
No free water

BOREHOLE NUMBER 117

PAGE 1 OF 1

CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-10-22 **COMPLETED** 21-10-22

GROUND ELEVATION 194.39 m

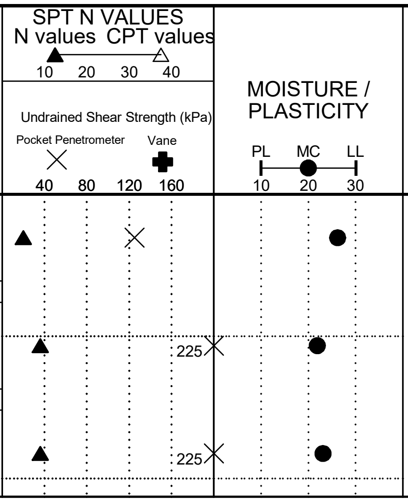
DRILLING CONTRACTOR LST

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Diedrich D50 Track

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.08	TOPSOIL: ~75 mm	194.32							
	CLAYEY SILT: Brown to grey, occasional sand seams, moist to very moist, firm to stiff, reworked appearance at top portion			SS1	SPT	5	0/0		
1				SS2	SPT	9	0/0		
2				SS3	SPT	9	0/0		
2.1		192.26							



Borehole terminated at 2.1 m.

Upon completion of augering
No cave
No free water

BOREHOLE NUMBER 118

PAGE 1 OF 1

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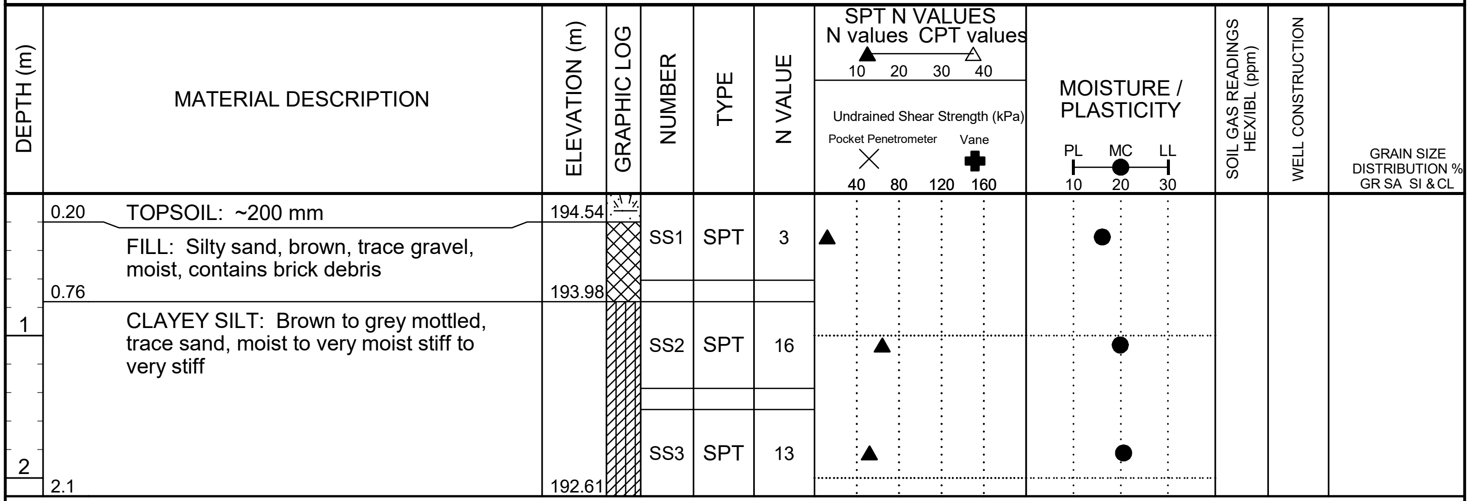
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-1 **COMPLETED** 21-10-1
GROUND ELEVATION 194.74 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES



DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE
0.20	TOPSOIL: ~200 mm	194.54			
	FILL: Silty sand, brown, trace gravel, moist, contains brick debris		SS1	SPT	3
0.76		193.98			
1	CLAYEY SILT: Brown to grey mottled, trace sand, moist to very moist stiff to very stiff		SS2	SPT	16
2			SS3	SPT	13
2.1		192.61			

Borehole terminated at 2.1 m.

Upon completion of augering
No cave
No free water

BOREHOLE NUMBER 119

PAGE 1 OF 1

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CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-10-21 **COMPLETED** 21-10-21

GROUND ELEVATION 195.66 m

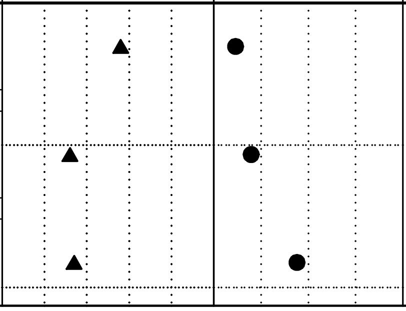
DRILLING CONTRACTOR LST

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Diedrich D50 Track

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SPT N VALUES N values ▲ CPT values △ 10 20 30 40 Undrained Shear Strength (kPa) Pocket Penetrometer × Vane ✚ 40 80 120 160	MOISTURE / PLASTICITY PL MC LL 10 20 30	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
1 1.2	FILL: Sand and gravel, brown, some silt, cobble and boulder size material on the surface, moist	194.46	▨	SS1 SS2	SPT SPT	28 16					
2 2.1	CLAYEY SILT: Brown to grey mottled, trace sand, moist to very moist, very stiff	193.53	▨	SS3.	SPT	17					



Borehole terminated at 2.1 m.

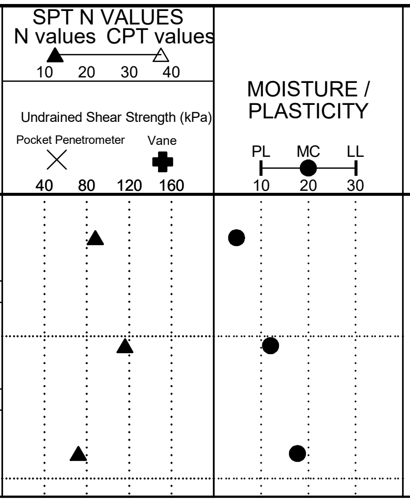
Upon completion of augering
No cave
No free water

BOREHOLE NUMBER 120

G2S Consulting Inc.

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-21 **COMPLETED** 21-10-21
GROUND ELEVATION 195.83 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
	FILL: Sand and gravel, brown, some silt, cobble and boulder size material on the surface, moist			SS1	SPT	22			
1				SS2	SPT	29			
1.4		194.48							
	CLAYEY SILT: Brown to grey mottled, trace sand, moist to very moist, very stiff			SS3	SPT	18			
2 2.1		193.70							



Borehole terminated at 2.1 m.

Upon completion of augering
No cave
No free water

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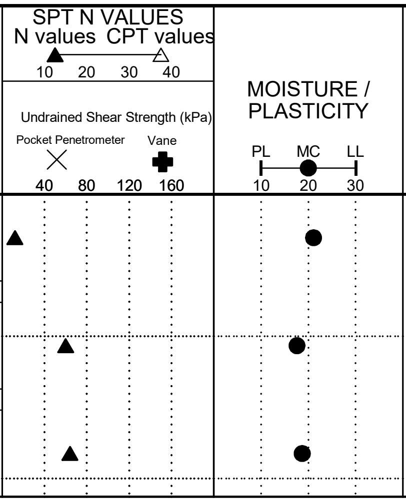
BOREHOLE NUMBER 121

PAGE 1 OF 1

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-1 **COMPLETED** 21-10-1
GROUND ELEVATION 194.70 m
DRILLING CONTRACTOR Davis
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD CME 45 Track
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.15	TOPSOIL: ~150 mm	194.55							
	CLAYEY SILT: Brown to grey mottled, trace sand, moist to very moist, stiff to very stiff			SS1	SPT	3			
1				SS2	SPT	15			
2				SS3	SPT	16			
2.1		192.57							



Borehole terminated at 2.1 m.

Upon completion of augering
No cave
No free water

BOREHOLE NUMBER 122

PAGE 1 OF 1

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CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-9-30 **COMPLETED** 21-9-30

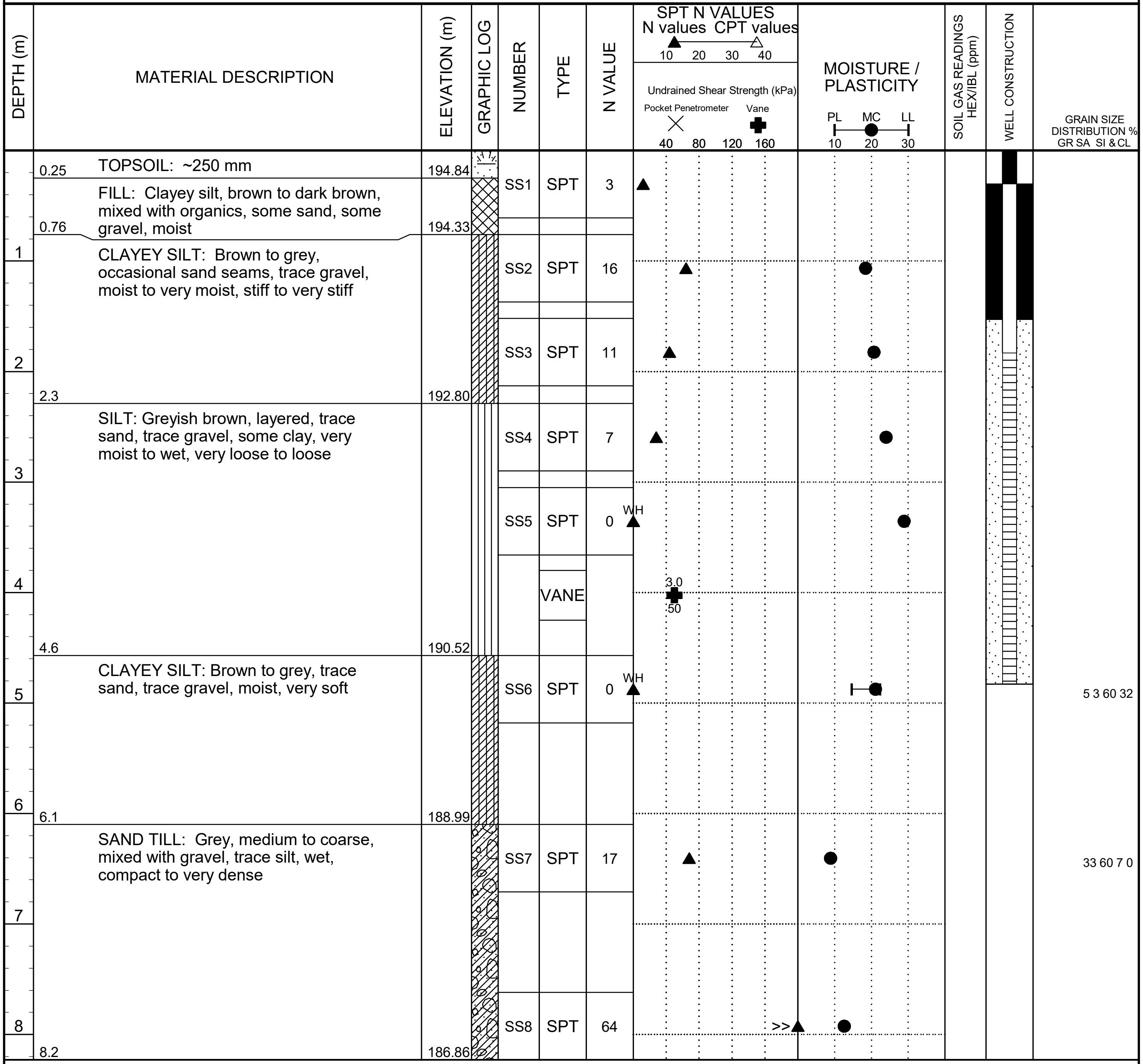
GROUND ELEVATION 195.09 m

DRILLING CONTRACTOR Davis

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD CME 45 Track

NOTES



DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.25	TOPSOIL: ~250 mm	194.84				
0.76	FILL: Clayey silt, brown to dark brown, mixed with organics, some sand, some gravel, moist	194.33	SS1	SPT	3	
	CLAYEY SILT: Brown to grey, occasional sand seams, trace gravel, moist to very moist, stiff to very stiff		SS2	SPT	16	
2.3		192.80	SS3	SPT	11	
	SILT: Greyish brown, layered, trace sand, trace gravel, some clay, very moist to wet, very loose to loose		SS4	SPT	7	
			SS5	SPT	0	
4.6		190.52		VANE		
	CLAYEY SILT: Brown to grey, trace sand, trace gravel, moist, very soft		SS6	SPT	0	5 3 60 32
6.1		188.99				
	SAND TILL: Grey, medium to coarse, mixed with gravel, trace silt, wet, compact to very dense		SS7	SPT	17	33 60 7 0
8.2		186.86	SS8	SPT	64	

Borehole terminated at 8.2 m.

Refer to report for groundwater elevation data

BOREHOLE NUMBER 123

PAGE 1 OF 1

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CLIENT Charis Developments Ltd.

PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd

PROJECT NUMBER G2S21366B

PROJECT LOCATION Collingwood, Ontario

DATE STARTED 21-10-21 **COMPLETED** 21-10-21

GROUND ELEVATION 195.75 m

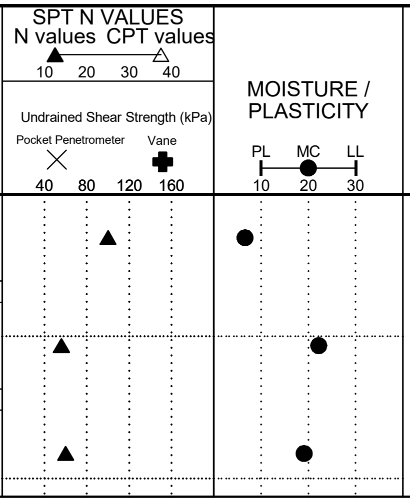
DRILLING CONTRACTOR LST

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Diedrich D50 Track

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SPT N VALUES / Undrained Shear Strength (kPa)	MOISTURE / PLASTICITY	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.93	FILL: Sand and gravel, brown, some silt, cobble and boulder size material on the surface, moist	194.83		SS1	SPT	25					
2.1	CLAYEY SILT: Greyish brown, mottled, trace sand, trace gravel, moist, stiff to very stiff	193.62		SS2 SS3	SPT SPT	14 15					



Borehole terminated at 2.1 m.

Upon completion of augering
No cave
No free water

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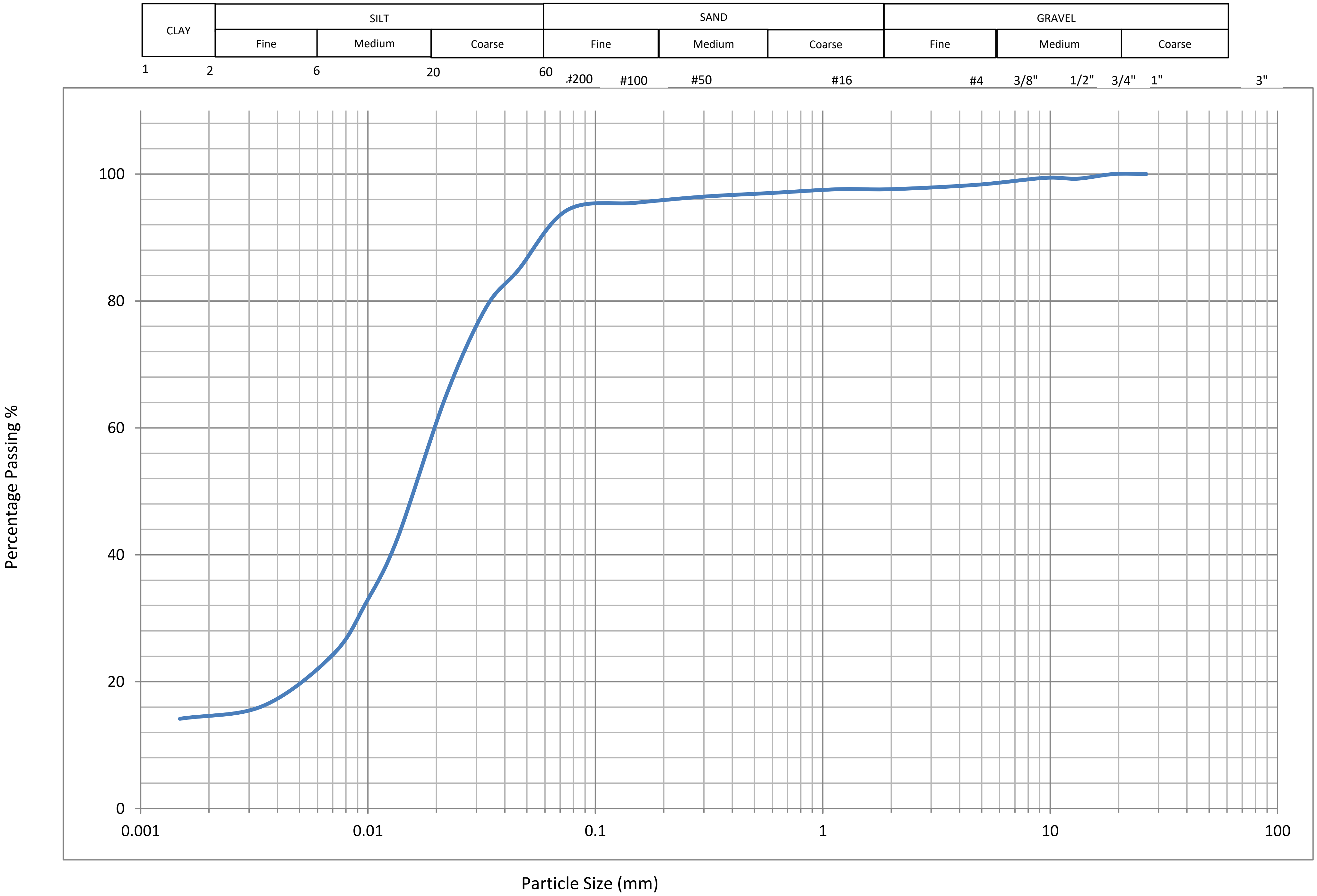
Project No.: G2S21366B

Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031A

Borehole/Sample No.: BH102-SS5

ISSMFE SOIL CLASSIFICATION



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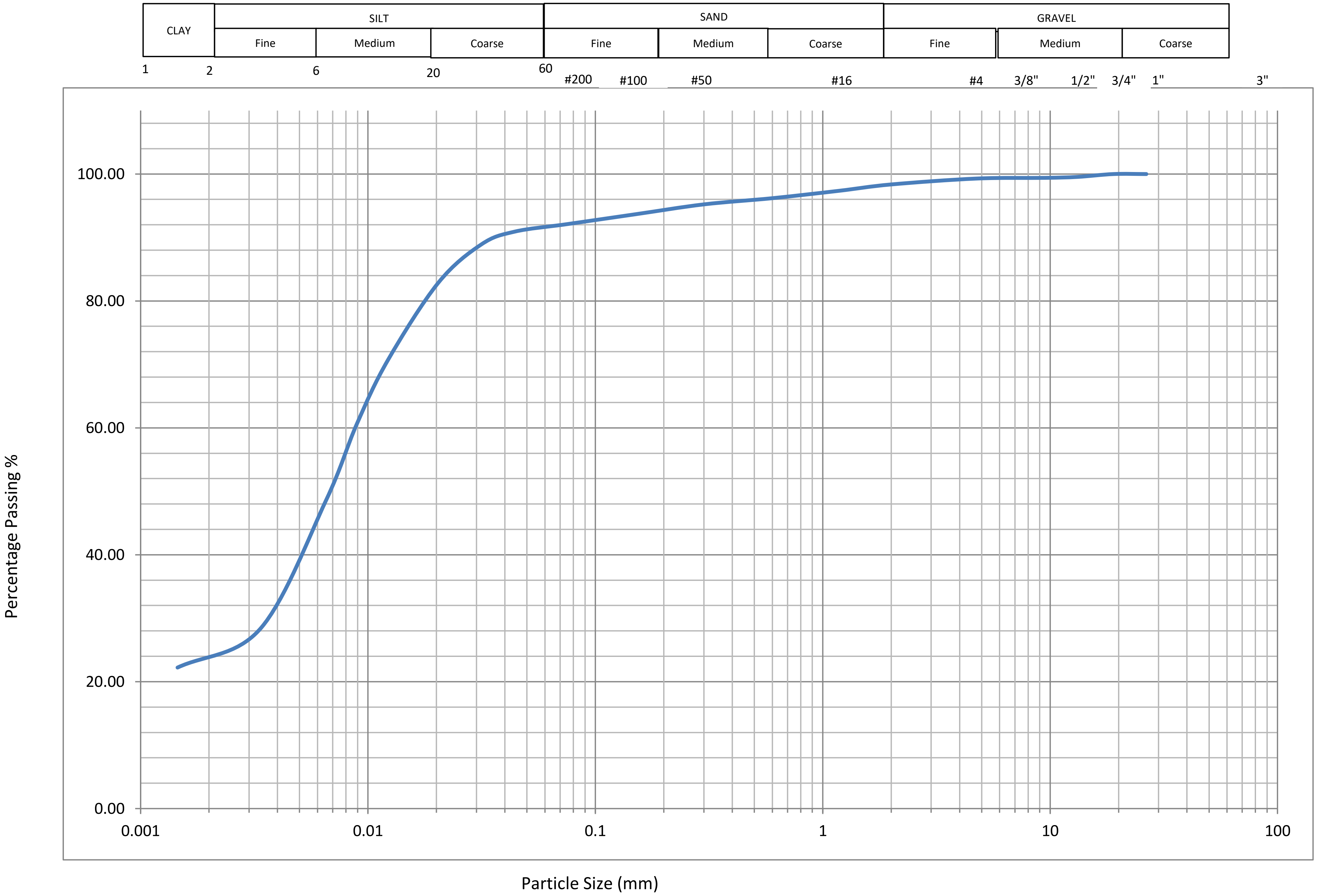
Project No.: G2S21366B

Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031B

Borehole/Sample No.: BH103-SS4

ISSMFE SOIL CLASSIFICATION



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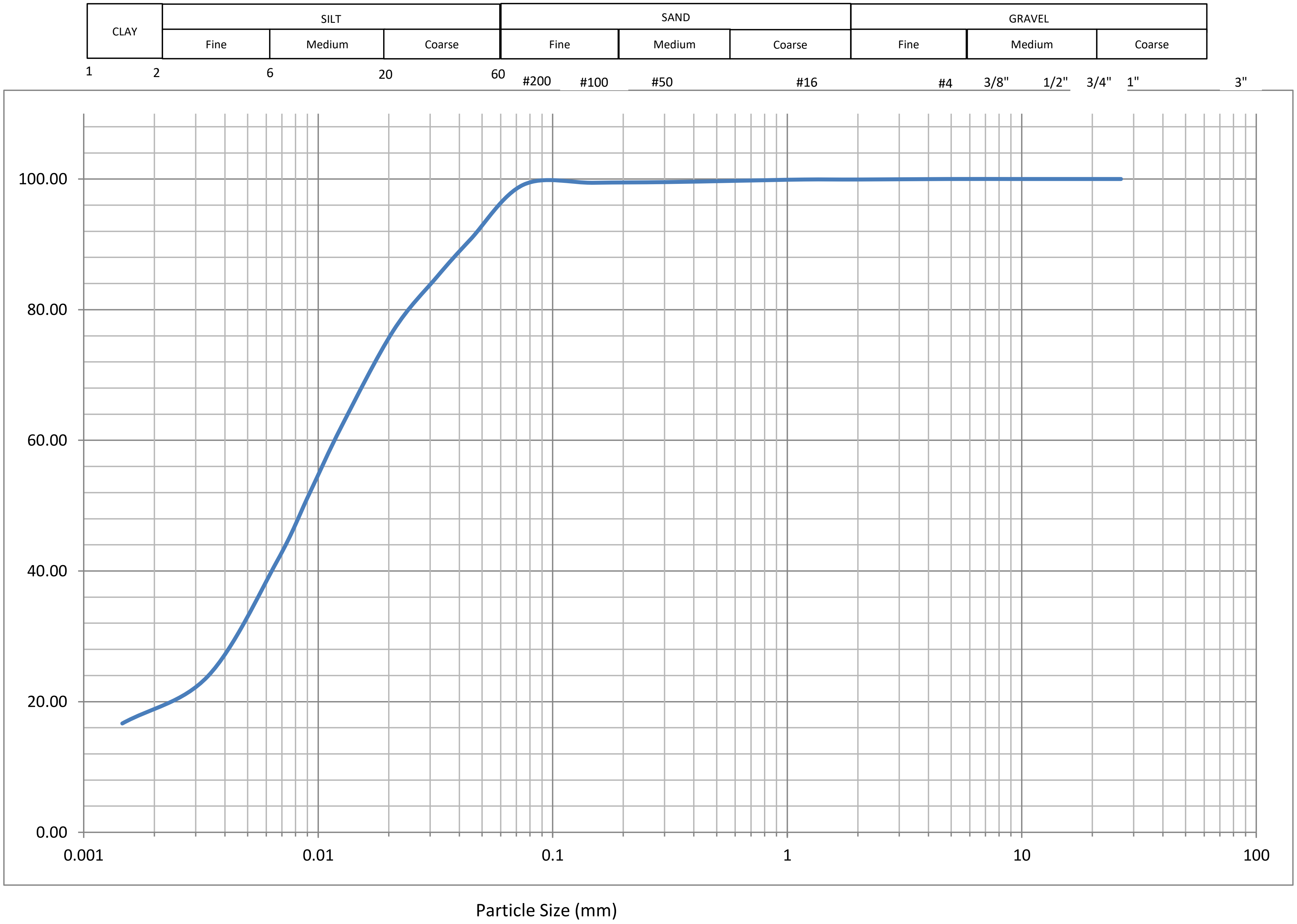
Project No.: G2S21366B

Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031C

Borehole/Sample No.: BH105-SS4

ISSMFE SOIL CLASSIFICATION



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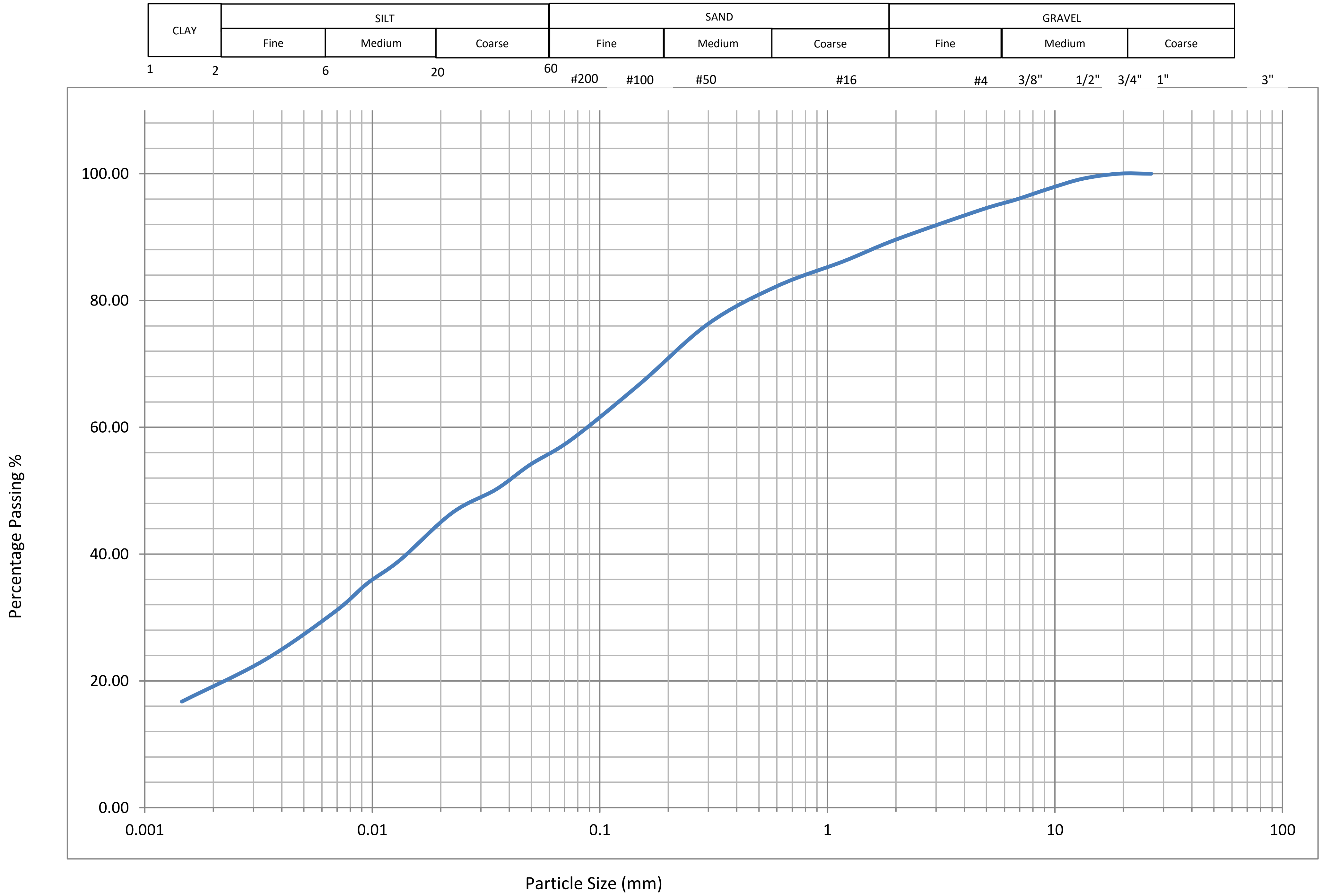
Project No.: G2S21366B

Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031D

Borehole/Sample No.: BH107-SS5

ISSMFE SOIL CLASSIFICATION



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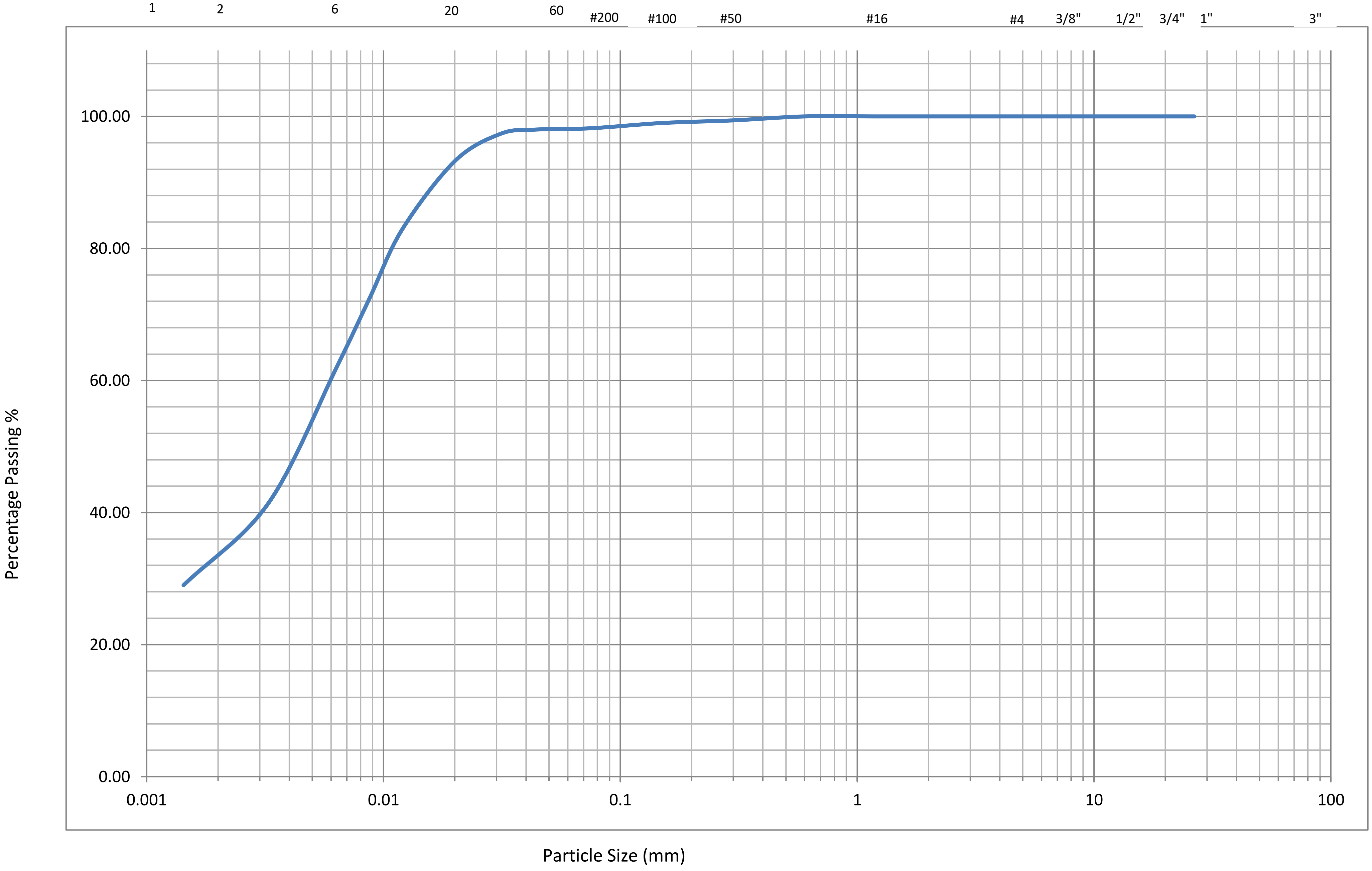
Project No.: G2S21366B
Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031E
Borehole/Sample No.: BH109-SS3

ISSMFE SOIL CLASSIFICATION

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



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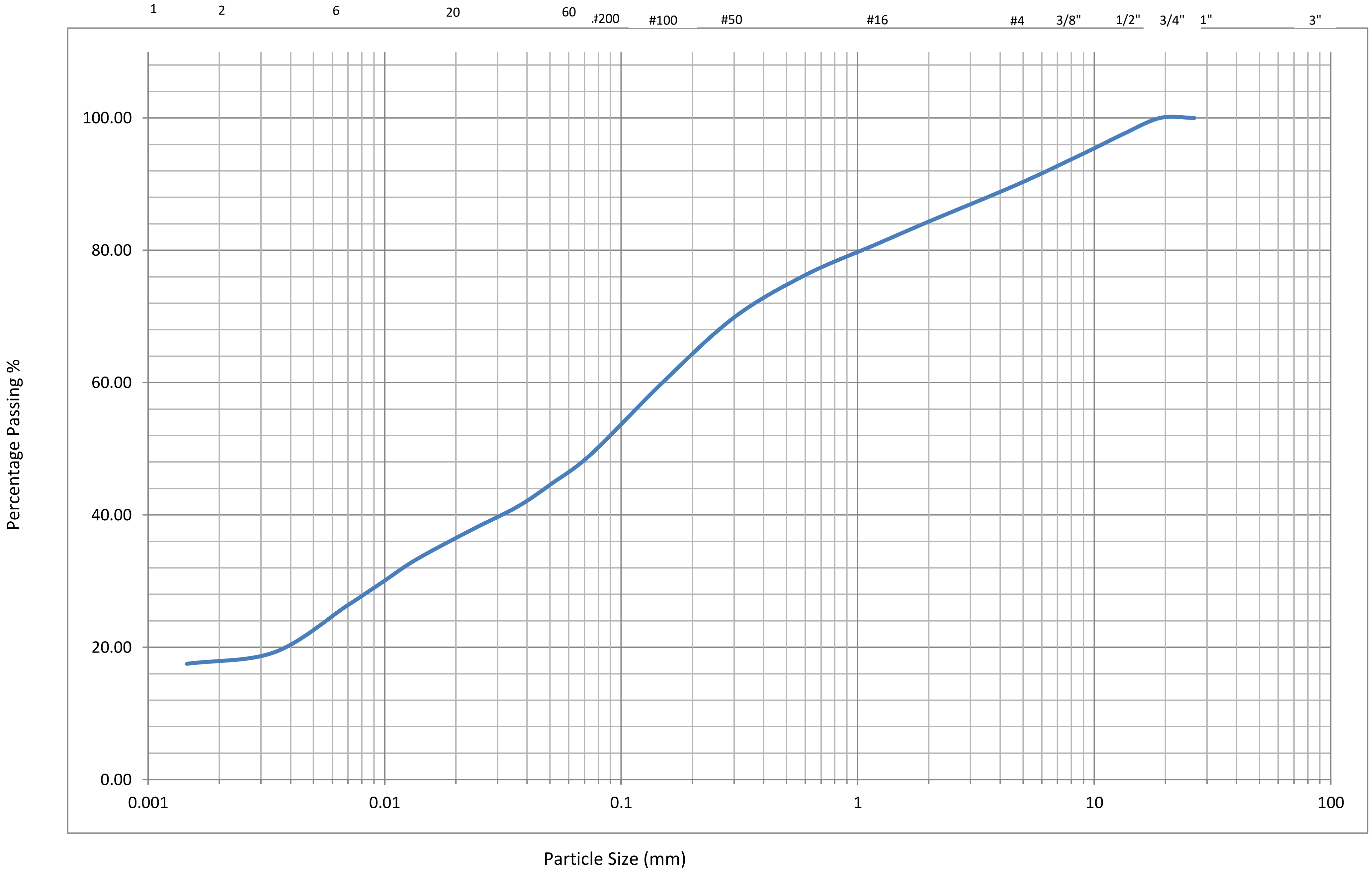
Project No.: G2S21366B
Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031F
Borehole/Sample No.: BH113-SS6

ISSMFE SOIL CLASSIFICATION

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



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Project No.: G2S21366B

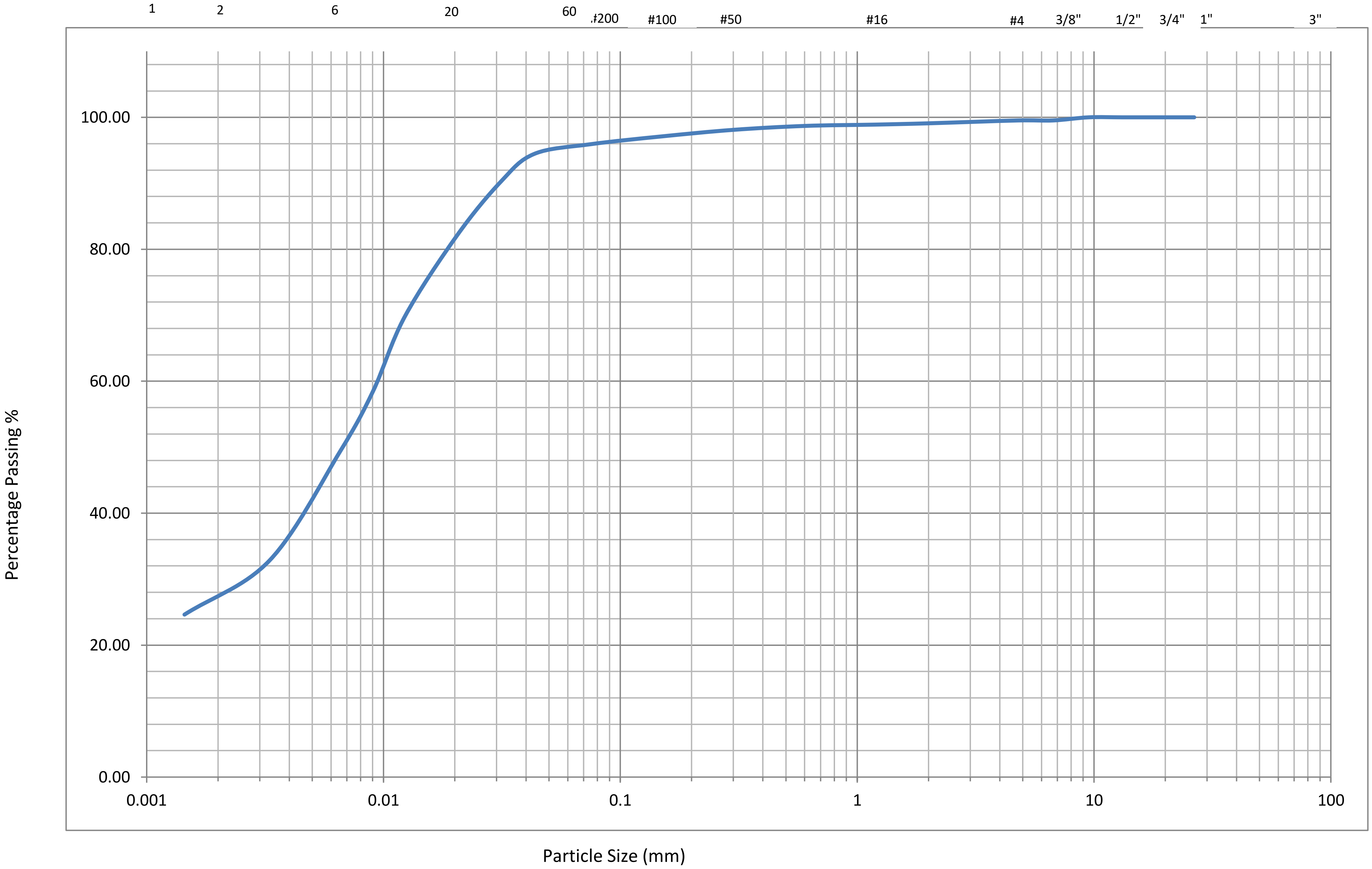
Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031G

Borehole/Sample No.: BH113-SS5

ISSMFE SOIL CLASSIFICATION

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



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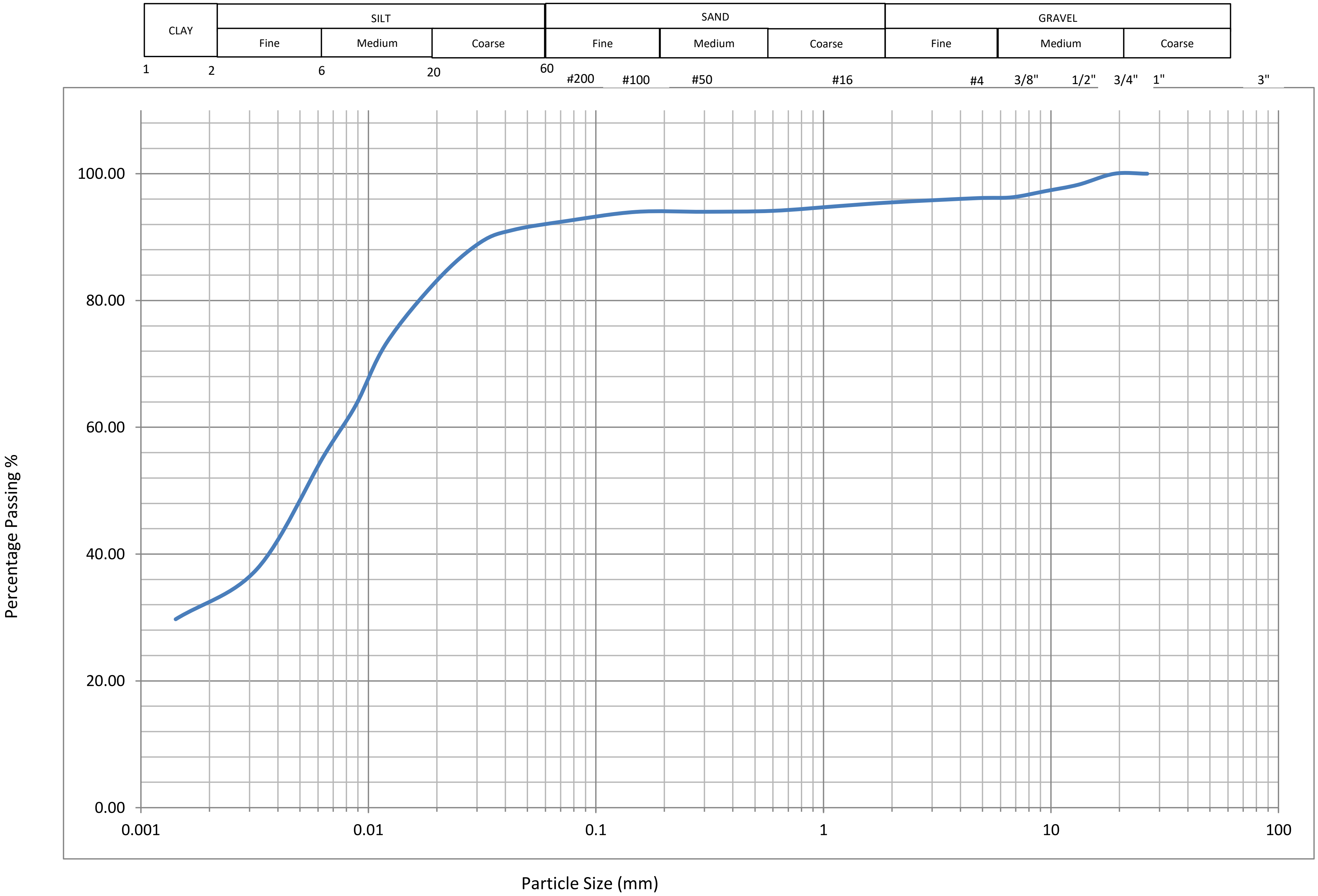
Project No.: G2S21366B

Project Name: 839 & 869 Hurontario St.&7564 Poplar St.

Lab No.: # 21031H

Borehole/Sample No.: BH122-SS6

ISSMFE SOIL CLASSIFICATION



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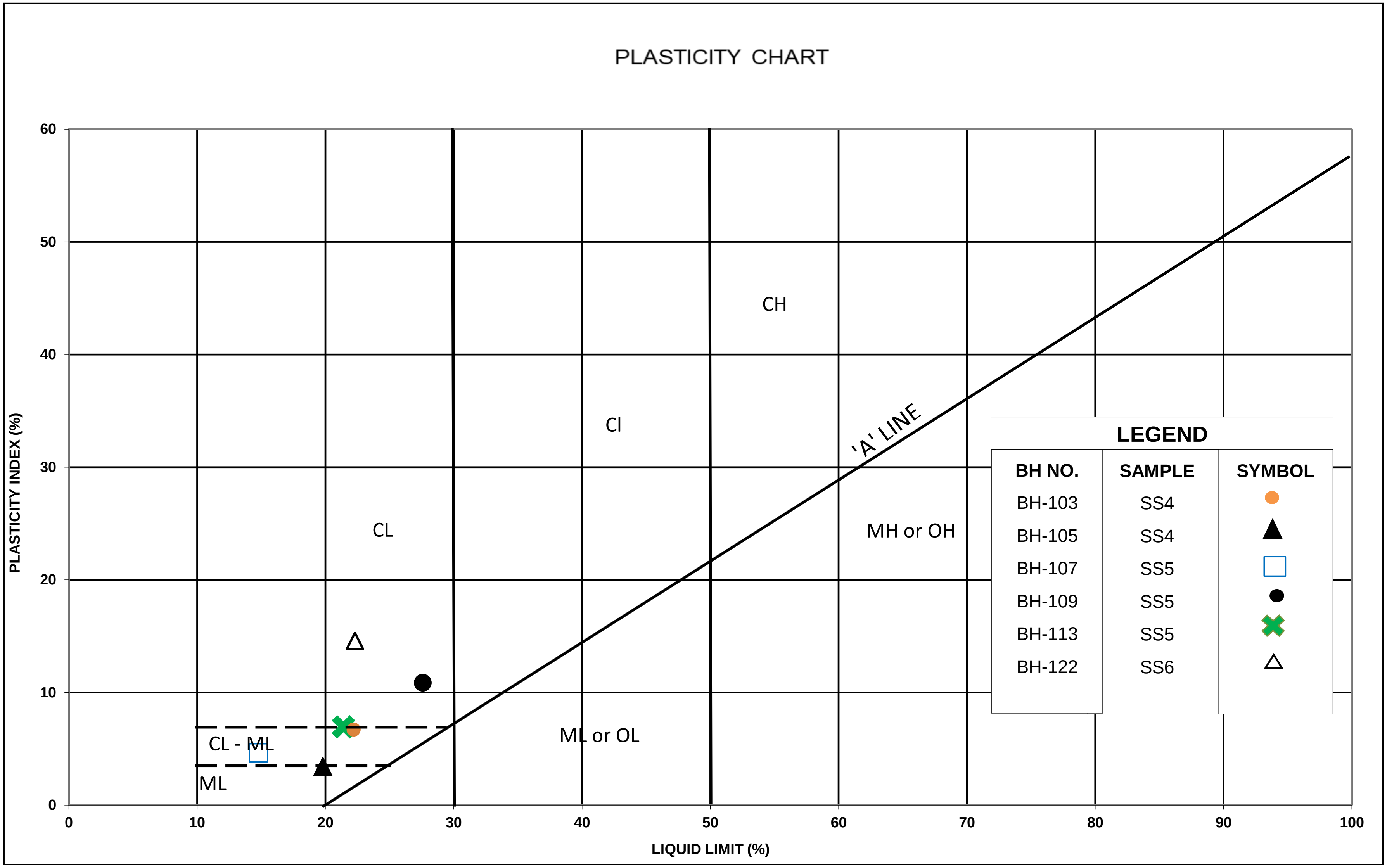
Project No.: G2S21366

Project Name 839 & 869 Hurontario St. & 7564 Poplar Side Rd.

Lab No.: 21031

Borehole/Sample No.:



Appendix C:
UCS Test Results & Retrieved Rock Core Samples Photographs

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Rock Core Compressive Strength Test Report

Project: 869 Hurontario St & 7564 Poplar Side Road, Collingwood

Project No.: G2S21366
Lab No.: 24169

Compressive Strength Test Results

Core No	Location/ Depth BGL (m)	Date Tested	Weight (g)	Diameter (mm)	Length (mm)	Unit Mass (kg/m3)	L/D	Test Load (kN)	Compressive Strength (MPa)	Correction Factor	Corrected Compressive Strength (MPa)
24169-A	BH201 - Run 3 8.05 - 8.15	23-Jul-24	1077.7	62	128	2789	2.06	113.9	37.7	1.00	37.7
24169-B	BH201 - Run 3 8.40 - 8.55	23-Jul-24	1088.0	62	132	2730	2.13	137.8	45.6	1.00	45.6
24169-C	BH201 - Run 4 8.70 - 8.85	23-Jul-24	1101.7	62	131	2786	2.11	222.3	73.6	1.00	73.6

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

Photo No. 1: BH201 – Boxes 1 & 2 of 2 – Run Nos. 1 - 4

Photo No. 2: BH201 – Box 1 – Run Nos. 1 - 4

Photo No. 3: BH201 – Box 1 (Photo 1 of 3) – Run Nos. 1 - 4

Photo No. 4: BH201 – Box 1 (Photo 2 of 3) – Run Nos. 1 - 4

Photo No. 5: BH201 – Box 1 (Photo 3 of 3) – Run Nos. 1 - 4

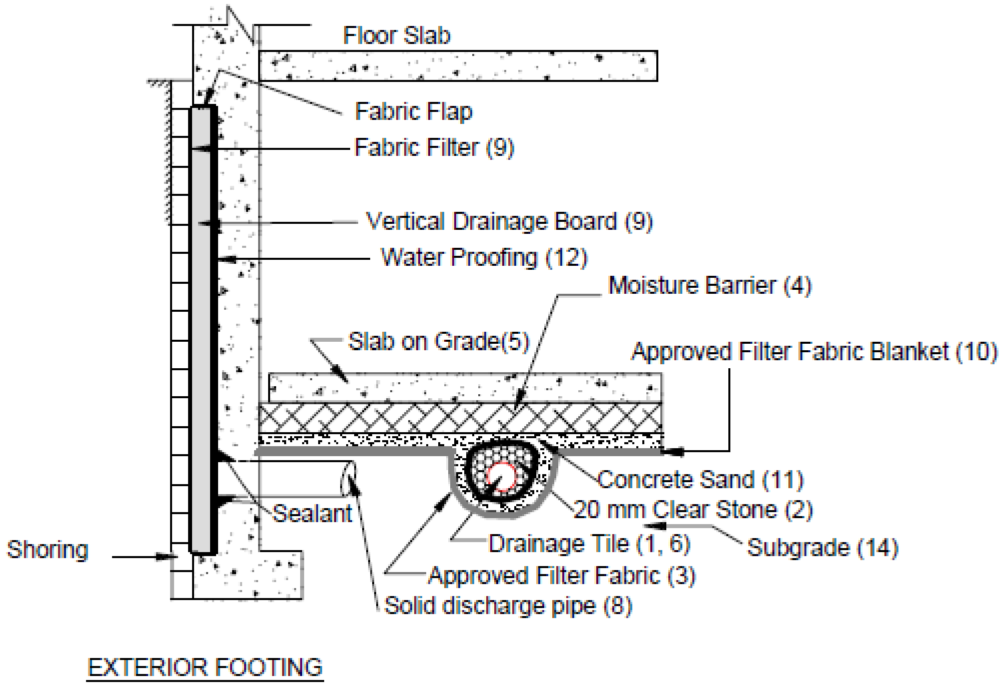
Photo No. 6: BH201 – Box 2 – Run No. 4

Photo No. 7: BH201 – Box 2 (Photo 1 of 3) – Run No. 4

Photo No. 8: BH201 – Box 2 (Photo 2 of 3) – Run No. 4

Photo No. 9: BH201 – Box 2 (Photo 3 of 3) – Run No. 4

Appendix D:
Drainage and Underpinning Recommendation Figures

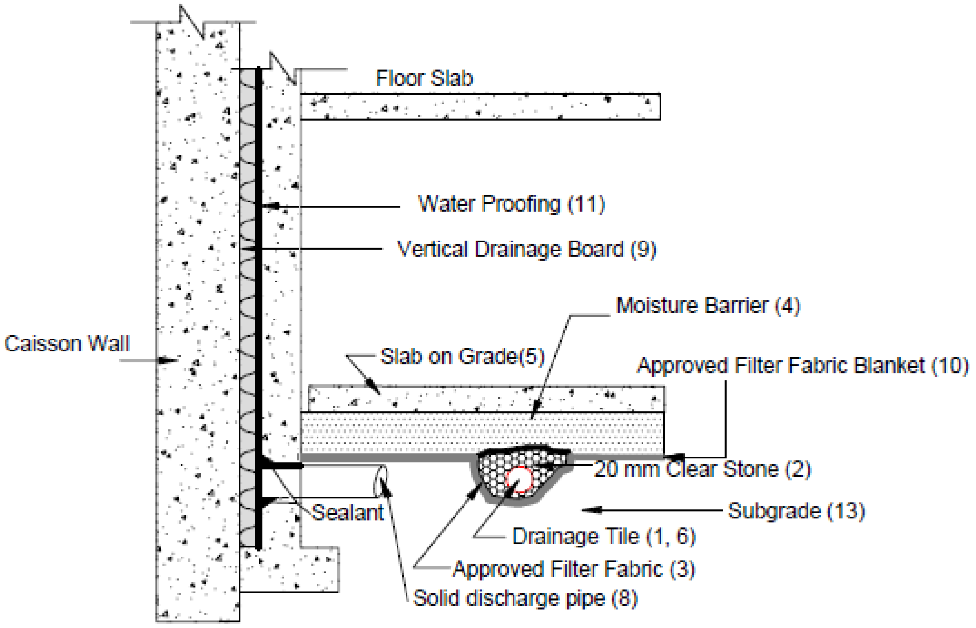


Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain.
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
5. Slab on grade should not be structurally connected to the wall or footing.
6. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
7. Do not connect the underfloor drains to perimeter drains.
8. Solid discharge pipe located at the middle of each bay between the solider piles, approximate spacing 2.5 m, outletting into a solid pipe leading to a sump.
9. Vertical drainage board with filter cloth should be kept a minimum of 1.2 m below exterior finished grade.
10. The entire subgrade to be sealed with approved filter fabric (Terrafix 270R or equivalent) if non-cohesive (sandy) soils below ground water table encountered.
11. Above the filter fabric, we recommend 60 mm thick concrete sand be placed to prevent loss of fines through filter fabric.
12. The basement walls should be water proofed using bentonite or equivalent water-proofing system.
13. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.
14. Subgrade must be inspected and approved by geotechnical personal. If soft/ loose is encountered, the soft/ loose soil must replaced with compacted granular material or the recommendations in the Geotechnical report must be followed.

DRAINAGE RECOMMENDATIONS
Shored Basement Wall with Underfloor Drainage System

(NOT TO SCALE)



EXTERIOR FOOTING

Notes

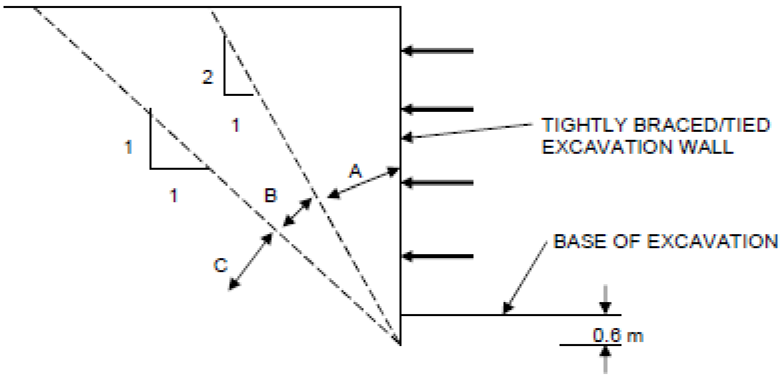
1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain.
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
5. Slab on grade should not be structurally connected to the wall or footing.
6. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
7. Do not connect the underfloor drains to perimeter drains.
8. Solid discharge pipe located at the middle of each bay between the solider piles, approximate spacing 2.5 m, outletting into a solid pipe leading to a sump.
9. Vertical drainage board mira-drain 6000 or equivalent with filter cloth should be continuous from bottom to 1.2 m below exterior finished grade.
10. The entire subgrade to be sealed with approved filter fabric (Terrafix 270R or equivalent) if non-cohesive (sandy) soils below ground water table encountered.
11. The basement walls must be water proofed using bentonite or equivalent water-proofing system.
12. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.
13. Subgrade must be inspected and approved by geotechnical personal. If soft/ loose is encountered, the soft/ loose soil must be replaced with compacted granular material or the recommendations in the Geotechnical report must be followed.

DRAINAGE RECOMMENDATIONS
Shored Basement Wall with Underfloor Drainage System

(NOT TO SCALE)

Guidelines for Underpinning in Soil and Excavation Support

Existing foundations located within Zone A normally require underpinning, especially for heavy structures. For some foundations in Zone A, it may be possible to eliminate underpinning and control foundation movement by tightly braced excavation walls, such as caisson walls.



Zone A Foundations located within this zone normally require underpinning. Horizontal and vertical pressures on the excavation wall of non underpinned foundations must be considered.

Zone B Foundations located within this zone normally do not require underpinning. Horizontal and vertical pressures on the excavation wall of non underpinned foundations must be considered.

Zone C Underpinning to structures is normally founded in this zone. Lateral pressure from underpinning is not normally considered.