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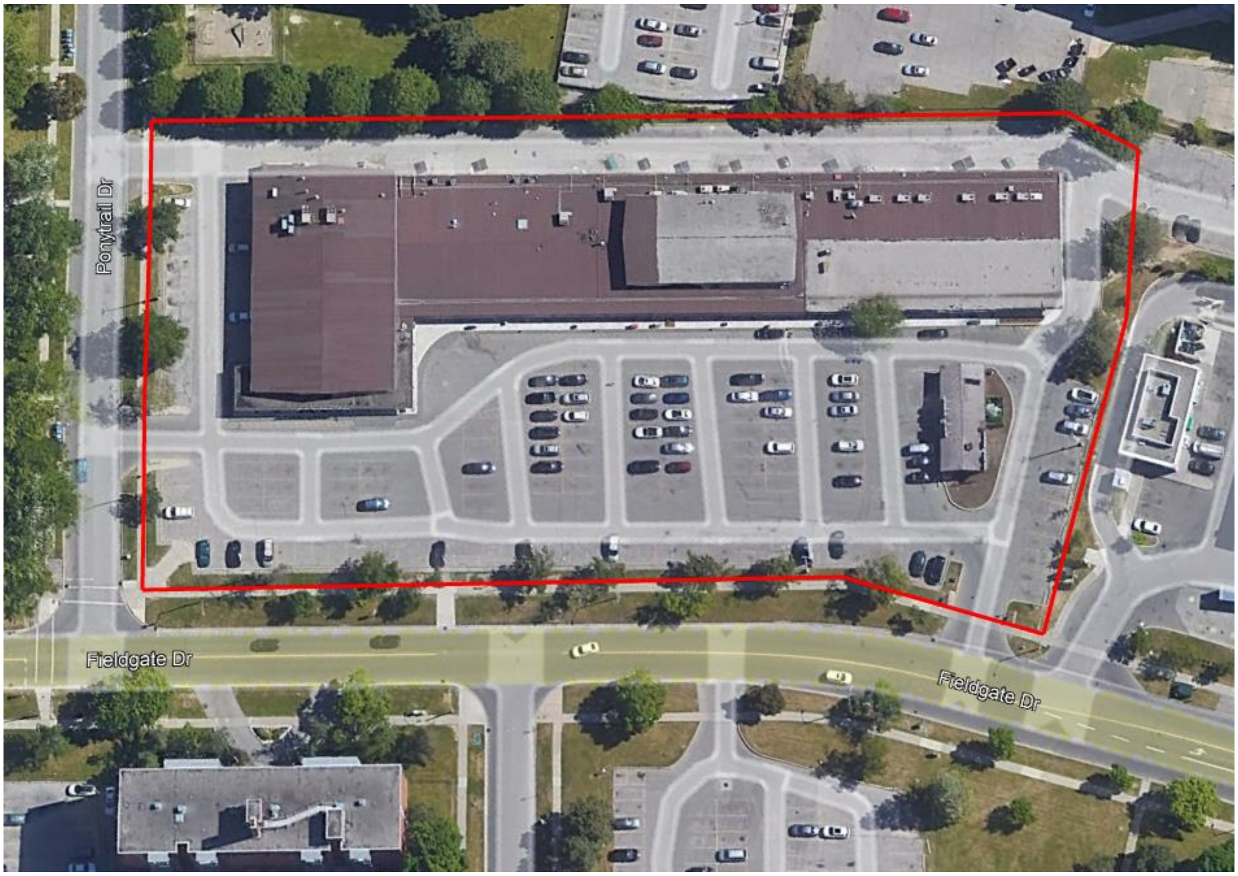
Geotechnical Investigation
Proposed Mixed-Use Development



3403-3445 Fieldgate Drive, Mississauga, Ontario
G2S24018-C

Sajecki Planning Inc.
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Toronto, Ontario
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Attention: Morgan Dundas

July 2024

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Appendices

1. Introduction

G2S Consulting Inc. (G2S) was retained by Sajecki Planning Inc. (the Client) to complete a Geotechnical Investigation for the Proposed Mixed-Use Development located at 3403-3445 Fieldgate Drive, Mississauga, Ontario, hereinafter referred to as the 'Site'.

The Site is located approximately 80 m northwest of the intersection of Bloor Street and Fieldgate Drive, Mississauga, Ontario, and covers an approximate plan area of 15,840 m2.

The general location of the Site is shown on the Site Key Plan included in Drawing 1 in Appendix A. This Geotechnical Investigation was carried out as outlined in G2S' Proposal No. G2S24018, dated January 18, 2024.

2. Site and Project Overview

2.1 Site Description

The Site is an irregularly shaped property located to the north side of Fieldgate Drive, Mississauga, Ontario, and covers an approximate plan area of 15,840 m^2. The Site is also surrounded by residential developments to the north, commercial development to the east, and Ponytrail Drive followed by a residential development to the west. At the time of this investigation, the Site was occupied by the existing commercial plaza, along with its access reads and parking areas.

2.2 Proposed Development

It is understood that the proposed development plan includes the demolition of the existing commercial plaza and the construction of a residential development which will comprise three towers (13, 18, and 22 storeys) and two levels of underground parking.

The purpose of this geotechnical investigation was to determine the subsurface conditions at Ten (10) borehole locations and to interpret these findings 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, it has not been addressed in this document.

3. Investigation Methodology

A total of 10 sampled boreholes were advanced at the locations illustrated in the attached Drawing 1, Borehole and Monitoring Well Location Plan, in Appendix A. The borings were put down uncased using continuous flight hollow stem auger and rock coring equipment. The drilling and sampling operations were carried out under the direction and supervision of a G2S staff member. The boreholes were advanced to depths of between approximately 4.0 to 15.2 metres below the existing grade (mbeg). Upon 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 spoon sampling equipment driven in accordance with the requirements of Standard Penetration Resistance Testing. 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 borings.

Rock coring was carried out in boreholes BH102, BH106, and BH109 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 BH110, inclusive, 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 titled “Surveyor’s Real Property Report and Topography of Block J Registered Plan 719, City of Mississauga, Regional Municipality of Peel”, dated April 26, 2023, by Genesis Land Surveying Inc. provided to G2S by the Client on April 29, 2024. 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 twelve 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 the boreholes.

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

4.1 Pavement Structure

In BH101 and BH103 to BH110, a surficial asphaltic concrete layer with a thickness ranging between approximately 90 to 150 mm was encountered over approximately 80 to 175 mm thick granular material. A surficial granular layer with a thickness of approximately 150 mm was encountered at BH102 location.

4.2 Fill

In all investigated boreholes, fill material was encountered below the pavement structure/granular. The fill consisted generally of clayey silt or silty sand/sand and gravel. Organic material was indicated within the fill layer at the locations of BHBH101, BH104, 106 to BH108, and BH110. The fill material extended to depths ranging between 0.8 and 1.5 metres below the existing grade (mbeg). The moisture content for the fill ranged between 7 and 37%, indicating moist to wet conditions.

4.3 Silt

Silt material was encountered beneath the fill in BH102 and extended to a depth of approximately 2.3 mbeg. The SPT “N” values of this silt deposit ranged from 11 blows per 300 millimetres of penetration, indicating compact condition. The moisture content for the silt was in the order of 17%, indicating moist conditions.

4.4 Sand/Silty Sand/Sandy Silt

Sand/silty sand/sandy silt material was encountered beneath the fill in BH103 to BH105 and BH107 to BH110 and extended to depths ranging from approximately 3.8 and 6.1 mbeg. The SPT “N” values of this sand/silty sand/sandy silt deposit ranged between 10 and 30 blows per 300 millimetres of penetration, indicating compact to dense compactness. The moisture content of the silty sand/sandy silt till ranged between 3% and 21%, indicating moist to wet conditions.

4.5 Silty Sand/Sandy Silt Till

Silty sand/sandy silt till material was encountered beneath the fill in BH101 and BH106, beneath the silt in BH102, and beneath the sand/silty sand/sandy silt in BH103, BH104, BH109, and BH110, and extended to depths ranging from approximately 3.0 and 9.1 mbeg.

The SPT “N” values of this silty sand/sandy silt till deposit ranged between 12 to in excess of 50 blows per 300 millimetres of penetration, indicating compact to very dense compactness. The moisture content for the silty sand/sandy silt till ranged between 7% and 22%, indicating moist to wet conditions.

4.6 Clayey Silt Till

Clayey silt till was encountered beneath the sand/silty sand/sandy silt in BH104, BH105, BH107 and BH108, beneath the silty sand/sandy silt till in the remaining investigated boreholes. The clayey silt till deposit extended to the depths ranged between approximately 4.0 and 12.2 mbeg. This clayey silt till. Boreholes BH101, BH103, and BH104 were terminated in this deposit. With "N" values ranging from 20 to in excess of 50 blows per 300 millimetres of penetration indicating the clayey sandy silt till deposit was classified as very stiff to hard in consistency. The moisture content for the clayey silt till ranged between 7% and 16%, indicating moist conditions. Boreholes BH101, and BH103 to BH104 were terminated in this deposit. Based on three (3) grain size analyses, the clayey silt till contained between 40 to 54% gravel, 17 to 19% sand, 21 to 34% silt, and 89 to 10% clay sized particles. Based on the laboratory results for two (2) selected samples of this deposit, the liquid limit ranged between 26% and 27%, and the plastic limit ranged between 18% and 19%, indicating low to moderate plasticity. Results of the grain size analyses, and the Atterberg Limits are included in Appendix B.

4.7 Shale Bedrock

Weathered to unweathered shale bedrock was encountered and/or inferred by the auger/sampler refusal at BH103 to BH105, BH107 to BH108, and BH110, at and coring in BH102, BH106 and BH109 at depths ranging between approximately 4.3 and 10.7 mbeg. The approximate depth and elevation of the shale bedrock surface/probable shale bedrock surface at the borehole locations are presented in Table No. 1 below:

Table 1: Approximate Depth and Elevation of Shale Bedrock Surface

Borehole ID	Depth of Shale Bedrock Surface Below Existing Grade (m)	Approximate Elevation of Bedrock Surface (m)	Remarks
BH102	6.1	129.0	Proven by coring from ~Elev. 129.0 to 122.7 m
BH105	9.1	126.2	Inferred by auger and sampler refusal
BH106	5.0	130.0	Proven by coring from ~Elev. 129.8 to 124.2 m
BH107	4.8	130.2	Inferred by auger and sampler refusal
BH108	9.1	125.4	Inferred by auger and sampler refusal
BH109	10.7	123.8	Proven by coring from ~Elev. 123.6 to 119.2 m
BH110	12.2	121.9	Inferred by auger and sampler refusal

Bedrock was proven by coring in Borehole BH102, BH106, and BH109, between 10.7 and 15.2 mbeg (~Elev. 129.8 and 119.2 m). Due to the method of drilling and sampling, the surface elevation of the bedrock can be different than indicated on the borehole logs. Typically, the till overlying the shale contains slabs of limestone that may give a false indication of the bedrock level. Based on our experience and the available published information, the upper portion of the bedrock is typically weathered and becomes more sound with depth.

The shale was typically grey of the Georgian Bay formation and contained increasing limestone/siltstone content. Based on the rock core samples, which were obtained from Boreholes BH102, BH106, and BH109, the shale bedrock was generally highly weathered to unweathered with completely weathered zones in BH102 between 6.2 – 7.0, between 7.0 - 7.4, between 9.0 – 9.1, between 10.0 –10.3, at 11.6, and at 12.0 mbeg. In BH106 the completely to highly weathered zones were noted at 5.0, between 5.2 – 5.5, between 5.6 – 6.0, at 6.2, between 7.0 – 7.4, at 9.3, at 10.0, between 10.1 – 10.2, and at 10.7 mbeg. The completely to highly weathered zones were noted between 11.0 – 11.4, between 12.2– 12.6, between 13.7 – 13.91, between 14.3 – 14.9, and at 15.0 mbeg in BH109. Details of the completely to highly weathered zones are shown on the rock core log sheets attached in Appendix B. The shale bedrock contained strong limestone interbedding. The thickness of these Limestone layers ranged from approximately 10 to 160 mm. Details of the limestone/siltstone interbedding layers are shown on the rock core log sheets attached in Appendix B.

The Total Core Recovery (TCR) ranged from 12 to 100 percent. The recorded Rock Quality Designation (RQD) value ranged from 0 to 70 percent, indicating fair to good quality. The discontinuities observed in the rock core were typically horizontal bedding planes with flat orientation, with occasional fault joints with vertical orientation. The spacing of the discontinuities ranged from close to very close, and joint fillings were generally Tight, non-softening to sandy and silty minor clay. Laboratory Unconfined Compressive Strength (UCS) tests were performed on selected samples of the rock cores. The compressive strength values measured on samples of the shale bedrock ranged from 26.5 to 53.4 MPa, indicating medium to high strength. The UCS laboratory test results report along with photographs for the retrieved core samples are included in Appendix C.

Based on the Ministry of Northern Development and Mines Map 2544, *Bedrock Geology of Ontario, Southern Sheet*, the bedrock in the Site vicinity consists of Georgian Bay Shale of the Upper Ordovician period. Sandstone, shale, dolostone, and siltstone (Lockport formation) lenses may also be encountered within the shale.

4.8 Groundwater Observations

Groundwater level observations in the open boreholes were recorded during the drilling operation. Upon the completion of the drilling operation, cave-in material was measured in BH103, BH104, BH105, BH108, and BH110 at the depths of 6.7, 2.1, 7.8, 4.3, and 4.5 mbeg, respectively. Free water was recorded in BH103, BH107, BH108 and BH110 at the depths of 5.5, 4.8, 4.3, 4.3 mbeg, respectively. All other boreholes were open and dry. Further, groundwater monitoring wells were installed in boreholes BH101 to BH102, BH106. Results of our groundwater monitoring to date are presented below:

Table 2: Groundwater Observations

BH/MW ID	Well Depth (m)	April 12, 2024		May 13, 2024		June 11, 2024	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
BH/MW 101	3.8	1.8	134.4	1.6	134.6	1.9	134.3
BH/MW 102	12.2	**	-	**	-	**	-
BH/MW 106	10.7	2.9	132.1	2.8	132.2	2.7	132.3

** The monitoring well was not accessible at the time of the measurements.

It is noted that the static groundwater level fluctuates based on seasonal conditions experienced during the wet and dry periods of the year. Therefore, we would recommend that additional monitoring of these wells be conducted prior to construction. Refer to Appendix B for the list of abbreviations and borehole logs.

5. Geotechnical Considerations

5.1 Site Preparation

At the time of the investigation, the final grading plan for the Site was not yet available to G2S. However, based on the available information including the site plan Drawing No. A-040, dated February 07, 2024, by Onespace Unlimited Inc., limited engineered fill may be utilized to develop the Site. 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 is recommended to be maintained within ± 2 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 is 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 Shallow Foundations

The shallow foundations are to be designed applying the Limit State Design (LSD) methodology described in Chapter 8 of the most recent edition of the 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 ($\Phi\ R_n$) was computed by multiplying R_n with a reduction factor $\Phi=0.5$, in accordance with NBCC (2015) and the most recent edition of the CFEM.

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 $e_L=M_L/V$.

The CFEM 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 the most recent edition of the CFEM.

The serviceability limit state (SLS) bearing pressure is considered by calculating the settlements due to foundation load (immediate, consolidation and total). 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.

5.2.1.1 Strip and Spread Footings

Based on preliminary information, the lowest basement slab at a depth of approximately 6.0 mbeg (~Elev. 130.2 to 128.1 m) assuming the grade level is between (~Elev. 136.2 and 134.1 m). As such, the founding level for the proposed structure can be supported on conventional spread and strip footings with a maximum width of 2.0 x 2.0 m and 2.0 m, respectively. The footings to be founded between 7.0 and 8.0 mbeg (~Elev. 129.2 to 126.1 m) on the silty sand till, clayey silt till or weathered to sound shale bedrock. Foundations constructed on the silty sand till and clayey silt till can be designed for a bearing resistance of 600 KPa at Ultimate Limit States (ULS) and 300 KPa at Serviceability Limit State (SLS). Foundation constructed on the competent bedrock below the completely weathered shale approximately 0.5 m below the weathered shale surface can be designed for 3000 KPa at (ULS), and 1500 KPa at (SLS). The geotechnical resistance of a sustained load at Serviceability Limit State (SLS) should be within the normally tolerated limits of 25 millimetres of 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. Foundation constructed on the sound shale material; at approximately 2.0 m below the weathered shale surface can be designed with a geotechnical resistance of 5,000 kPa at (ULS). The settlement of foundations placed on sound bedrock is expected to be negligible. As such, the bearing resistance SLS is not provided.

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

Table 3: Bearing Resistance for Conventional Spread/Strip Footing

Borehole ID	Material	Bearing Resistance (kPa)	Recommended Founding Depth (m)	Approximate Founding Elevation (m)
BH102	Weathered Shale	1500 SLS/3000 ULS	7.0	128.1
BH103	Clayey Silt Till	300 SLS/600 ULS	7.0	128.0
BH105	Clayey Silt Till	300 SLS/600 ULS	7.0	128.2
BH106	Shale Bedrock	5000 ULS	7.1	127.9
BH108	Clayey Silt Till	300 SLS/600 ULS	7.0	127.5
BH109	Silty Sand Till	300 SLS/600 ULS	7.0	127.5
BH110	Silty Sand Till	300 SLS/600 ULS	7.0	127.1

5.2.2 Deep Foundations

5.2.2.1 Methodology

Pile design was carried out using the limit state design methodology described in Chapter 8 of the most recent edition of the CFEM.

The ultimate pile geotechnical resistance of a single pile in compression (Q_c) and tension (Q_t) for the Site are calculated using the following equations:

$$Q_c = \sum f_{c,z} \cdot A_s + q_p A_p \quad \text{and} \quad Q_t = \sum f_{t,z} \cdot A_s + W$$

Where;

$f_{c,z}$ = unit shaft friction in compression at depth z (kPa)
$f_{t,,z}$ = unit shaft friction in tension at depth z (kPa)
A_s = outside surface area of pile shaft at depth z (m^2)
q_p = unit end bearing resistance (kPa)
A_p = gross area of pile toe (m^2)
W = buoyant pile self-weight (kN)

5.2.2.2 Drilled Caissons

Based on the borehole information, the shale bedrock surface was found dipping down towards the east and southeast sections of the site with depths ranging between 4.3 m (~Elev. 131.2 m) and 12.2 m (~Elev. 121.9 m) below the grade in BH104 and BH110, respectively. Given that the founding level for the proposed structure is expected to be between 7.0 and 8.0 mbeg (~Elev. 129.2 to 126.1 m), portions of the proposed building foundation are anticipated to be constructed partially on silty sand till and clayey silt till deposits, weathered shale, or sound shale bedrock, with design bearing resistance widely ranging from 300 KPa (SLS)/600 kPa (ULS) on the silty sand till, clayey silt till to 5000 KPa (ULS) on the sound shale bedrock.

As such, in the areas where the bedrock surface is dipping, and/or higher bearing resistance be required due to the anticipated loading of the proposed buildings, the proposed structures may be supported on a minimum of 0.76 m diameter drilled cast-in-place concrete caissons founded into the competent shale bedrock below any highly weathered and loose rock at an approximate depth of 1.0 m below the top of weathered shale bedrock. For caissons founded at this depth on the weathered shale bedrock, a bearing resistance of 1,500 kPa at SLS and 3,000 kPa at ULS can be utilized for the caisson design with the anticipated total settlement in the order of 19 mm.

For caissons founded on sound shale bedrock at an approximate depth of 2.5 m below the top of the shale a factored bearing resistance of 5,000 kPa at ULS can be used for caisson design. The geotechnical resistance of a sustained load at Serviceability Limit State (SLS) should be within the normally tolerated limits of 25 mm of settlement. The settlement of foundations placed on sound bedrock is expected to be negligible. As such, the bearing resistance at Serviceability Limit State (SLS) is not provided.

All caisson bases should be auger cleaned by mixing the loose materials with a small amount of concrete at the base of each caisson. The mixture should then be completely removed down to the sound shale bedrock. Downhole camera inspection may be employed if necessary and contractor’s cooperation will be required to facilitate the inspection process. The caisson installation must be monitored on a full-time basis by geotechnical personnel to verify that the bottom of each caisson is free of loose or soft material.

During the installation of caissons, a temporary steel casing will have to be installed to prevent the caving of the drilled hole and to seal off any water which may be perched into the permeable seams in the native soils. A 150 mm slump concrete is recommended for use to prevent the concrete from having a honeycombed structure and to avoid bridging in the liner upon its withdrawal. If there is a significant amount of water in the caisson hole base that cannot be bailed out, the concrete should be placed using the tremie method.

Concrete to be placed using the tremie concrete method, should be carried out by feeding the concrete from the bottom of the drilled shaft by pumping and filling from the bottom up or using the free fall method or another method approved by the structural engineer. If the free fall method is used, the concrete must be poured through a centering chute, so that it falls down the center of the hole and does not hit the reinforcing steel on the side of the shaft.

After drilling, whole concrete should be placed as soon as possible to reduce risk of groundwater seepage and/or sloughing soil. If a group of caissons are required to achieve the required structural capacity, the minimum center-to-center caisson spacing should be 3 times the caisson diameter. The efficiency of a friction pile group will be determined by the layout, diameter, and number of caissons. The contractor should be prepared to deal with cobbles and boulders that may exist within the till deposit during the drilling.

The caisson design and installation procedure should be reviewed by G2S prior to the commencement of construction. The installation operation must be monitored on a full-time basis by geotechnical personnel from G2S to verify the allowable bearing pressure, foundation elevations and alignment.

5.2.2.3 Micropiles

The proposed structure at areas where the shale bedrock is dipping may also be supported by grouted micropiles, which should be drilled into the shale bedrock below any weathered zone. Micropiles typically consist of augured holes with a diameter of 275 millimetres, reinforced with DYWIDAG bars or similar and grouted according to standard practice and the product specifications. The installation should be carried out in accordance with the general procedure for installation of micropile as per the Micropile Design and Construction Manual (FHWA 2000). It is recommended that a field load test be carried out in order to determine the piles' actual load capacity and confirm the theoretical design foundation.

At minimum, one test pile should be loaded to twice the design load. The load test should be evaluated by the contractor and engineer to assume compliance with job performance requirements. The test shall be performed in accordance with ASTM D 1143-81, Testing of Plies under Axial Compressive Load.

5.3 Foundation Construction

The footing beds in the silty sand till/weathered shale/shale bedrock will be prone to disturbance from construction, foot traffic and precipitation. It would be prudent to consider the placement of a 50-millimetre 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 existing structures should 'match' the level of the existing foundations.

All footings exposed to the environment must be provided with a minimum of 1.2 metres 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.

With foundations designed as outlined above and as required by the current Ontario Building Code, and with careful attention paid to construction detail, total and differential settlements should be well within normally tolerated limits of 25 and 20 mm, respectively. However, as is typical in most institutional construction, 'cosmetic' cracking of plasterboard, foundation walls, etc., may occur within the first year of construction due to shrinkage, minor settlement, etc. Subsequent to repair, additional cracking should be minimal.

It is imperative that a soil 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 this report's design concepts and recommendations 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 Ontario Building Code, Ontario Regulation 332/12. Based on the subsurface soil conditions encountered in this investigation and assuming that the proposed structures will be founded on the weathered shale, the applicable Site Classification for the seismic design is Site Class C – very dense soils and soft rock, based on the average soil characteristics for the Site. Conducting site-specific shear wave velocity testing may be considered to confirm or upgrade the site class. The seismic data as per the 2020 National Building Code interpolated seismic hazard values, in accordance with Article 4.1.8.23. of the NBC 2020, are as follows:

$S_a[0.2]$	$S_a[0.5]$	$S_a[1.0]$	$S_a[2.0]$	$S_a[5.0]$	$S_a[10.0]$	PGA	PGV
0.319	0.194	0.102	0.0467	0.0121	0.00408	0.174	0.126

It should be noted that the values above were based on the 2%-in-50-year seismic hazard values 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.

5.5 Floor Slab Considerations

The slab-on-grade for the proposed building is expected to be constructed on shale bedrock. The exposed subgrade should be prepared by carefully examining the surface for any loose or unsuitable material (i.e., weak, disturbed, fractured rock, etc.). All unsuitable and loose material should be removed and replaced with approved suitable material and be compacted to at least 98 percent of its Standard Proctor Maximum Dry Density (SPMDD), in the presence of a G2S representative. Oversized particles in excess of 4 inches (100 mm) in diameter should not be permitted in the backfilling operations. 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. Based on the conditions encountered in the boreholes, backfill recommendations, and the floor slab considerations as included in our report, a modulus of sub-grade reaction, k_s, of 40 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. 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.

The 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 essential that excess free water is not added to the concrete during its placement as this could increase the potential for shrinkage, cracking and curling of the slab.

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 type of backfill material, which is expected to be used in construction.

Table 4: Soil Properties for Design of Earth Retaining Structures

Material	φ	γ	K_a	$K_◦$	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	32	21.0	0.31	0.47	3.25
Till – Stiff to very stiff	30	19.5	0.33	0.50	3.00
Till – Dense /compact	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, $K_◦$ = At-rest Earth Pressure Coefficient, K_P = Active Earth Pressure Coefficient

The following equation can be used to calculate the 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

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 consist 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. The perimeter 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 operated 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. Even a small break in the filtering materials could result in loss of 'fines' into the drains with attendant performance difficulties, including settlements of the ground surface. 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 City of Mississauga 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.

The suggested perimeter drainage against a soldier pile and timber lagging, if it was utilized for temporary shoring system, is shown on Figure 1, and that against caisson wall is shown on Figure 2 in Appendix D.

The hydraulic uplift pressure below the waterproofed structure can be calculated using the following formula:

$$P = k\,[\gamma_w\, h_w]$$

Where;

P = hydraulic pressure acting on the base of the structure (kPa)
γ_w = unit weight of water (10 kN/m^3)
h_w = Depth of the base of the structure below the highest GWL (m)

For construction where foundations are placed directly on shale 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 = friction between the footing(s) and the bedrock
N = normal load acting on the bedrock
$\tan \varphi$ = friction resistance of the bedrock
μ = factor of safety for the ultimate limits states design (ULS) for sliding (0.8)

5.7 Excavations and Groundwater Control

It is anticipated that the excavations for the proposed foundations, sewers and other underground services may extend to depths of up to 8.0 mbeg through the fill and into the silt, sand/silty sand/sandy silt, silty sand/sandy silt till, clayey silt/silty clay till and shale bedrock.

The excavation must be completed in accordance with the current OHSA regulations. For guidance, soft and very loose soils and soils below the groundwater level are classified as Type 4. The fill material, the native compact silt/sandy silt/silty sand, and sandy silt/silty sand till are classified as Type 3 soil. The dense sandy silt/silty sand till, very stiff to hard clayey silt till can be classified as Type 2 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, G2S, 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. It should be noted that the glacial tills are non-sorted sediments and therefore may contain boulders. Provisions must be made in the excavation/drilling contract for the removal of possible boulders within the overburden soils.

The excavation of the upper layers of shale can be made with conventional hydraulic backhoes equipped with ripping teeth. However, increasing resistance to excavation should be expected with depth due to the increasing bedrock quality especially where limestone layers of considerable thickness are encountered. At these depths, excavations may require the use of a ripper and a hydraulic hammer. Provisions should be made in the excavation contract to include the use of these equipment for excavation in bedrock. It is not uncommon for natural gas pockets to be encountered in shale excavation. Therefore, gas monitoring should be carried out during construction for any excavation into the shale.

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

Typically, in deep excavation into the shale material in Southern Ontario, the shale may begin to experience pressure relief once it is excavated. This pressure relief/deformation is time dependent. In addition, the shale may also exhibit the potential for swelling. In the Georgian Bay formations, the time dependent deformation can take up to 4 months. As such, consideration should be given to accommodate this long-term deformation property of the bedrock in the project schedule. Alternatively, a layer of compressible material (i.e., minimum of 50 mm of compressible insulation or at least 600 mm of granular backfill) should be placed behind the portions of the structure that will be constructed against the rock face.

If a continuous caisson wall is installed around the perimeter, it should be possible to dewater the caisson wall enclosed area using a series of pumping wells installed inside the excavation. The quality of water and disposal method should be taken into consideration during tendering. The dewatering system should be designed and installed by an experienced specialist contractor who can provide an estimate of time needed to dewater the Site and the amount of groundwater anticipated. G2S will be pleased to review and comment on the contractor’s proposed dewatering system. 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.

The base of the excavations in the till material and shale bedrock encountered in the boreholes should remain firm and stable. Therefore, standard pipe bedding, as typically specified by the City of Mississauga, should suffice. The bedding material should be uniformly compact to at least 95 percent SPMDD, with special attention paid to compaction under the pipe haunches.

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 for in-construction and post-construction dewatering needs will be issued under a separate cover.

5.8 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 given 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, clayey silt till, weathered shale, and shale bedrock are capable of supporting the proposed raker footings. The raker footings supported on the dewatered, native sandy silt till and clayey silt till at 45 degree inclination can be designed for bearing resistance of 600 kPa at ULS, and the weathered shale and shale bedrock at 45 degree inclination can be designed for bearing resistance of 1000 kPa at ULS.

Caisson and solider pile toes will be made in sound shale bedrock. The horizontal resistance of the solider 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 sandy silt till deposit may be designed based on skin frictions ranging between 60 kPa. Anchors extended into the clayey silt till may be designed based on skin friction ranging between 50 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.

5.9 Backfill Considerations

The majority of the excavated material will consist of the native sand/silty sand/sandy silt, silty sand/sandy silt till, and clayey silt till. The existing fill apart from organics debris, or otherwise any deleterious material are 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. Dusting could be a problem in the 'dry' summer months.

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 soil 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.

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.

5.10 Pavement Considerations

5.10.1 Subgrade Preparation

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 with 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 sub-excavated and replaced with suitable backfill material. Alternatively, the soft areas may be repaired by placing coarse aggregate, such as 50 mm clear crushed stone. The need for sub-excavations 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. The fill placed over shallow utilities that cut into or across the paved areas must also be compacted to 100 percent of its SPMDD.

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.

5.10.2 Pavement Structure

The suggested pavement structures, for the pavement areas, outlined in Table 5 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 ten-year 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 5: 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	65 millimetres	40 millimetres
Binder Course OPSS HL 8	Min 92.0 % *MRD	--	65 millimetres
Base Course OPSS Granular A	100% **SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular B Type II	100% **SPMDD	200 millimetres	350 millimetres

Notes: * MRD denotes maximum relative density, MTO LS-264
** SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

Depending on the arrangement of light duty and heavy-duty pavement sections, the transition between sections may present some difficulty for contractors. In this regard, consideration might be given to a slightly increased light duty pavement structure consisting of 50 mm of HL8 binder course and 40 mm of HL3 surface course asphaltic concrete. This structure will provide for a continuous depth of surface course asphalt allowing for ease of construction.

As well, such a structure would have an improved performance over an increased design life. Such an arrangement of asphalt layers would also allow for future rehabilitation with a ‘mill and pave’ type operation.

5.11 Construction Review and Testing

Site review should be carried out during construction to confirm that the conditions exposed are consistent with the conditions encountered during the borehole investigation. In-situ testing should be carried out to verify compliance with the relevant local and provincial specification, as well as the project contract documents.

6. 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 Section 4.2.2.2 of the Ontario Building Code (2012). 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.

7. 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 (Forest Glen Shopping Centre 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 City of Mississauga, in its 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 expressed and implied.

A 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 groundwater chemical quality and the suitability of excess soil for off-site re-use.

8. 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.

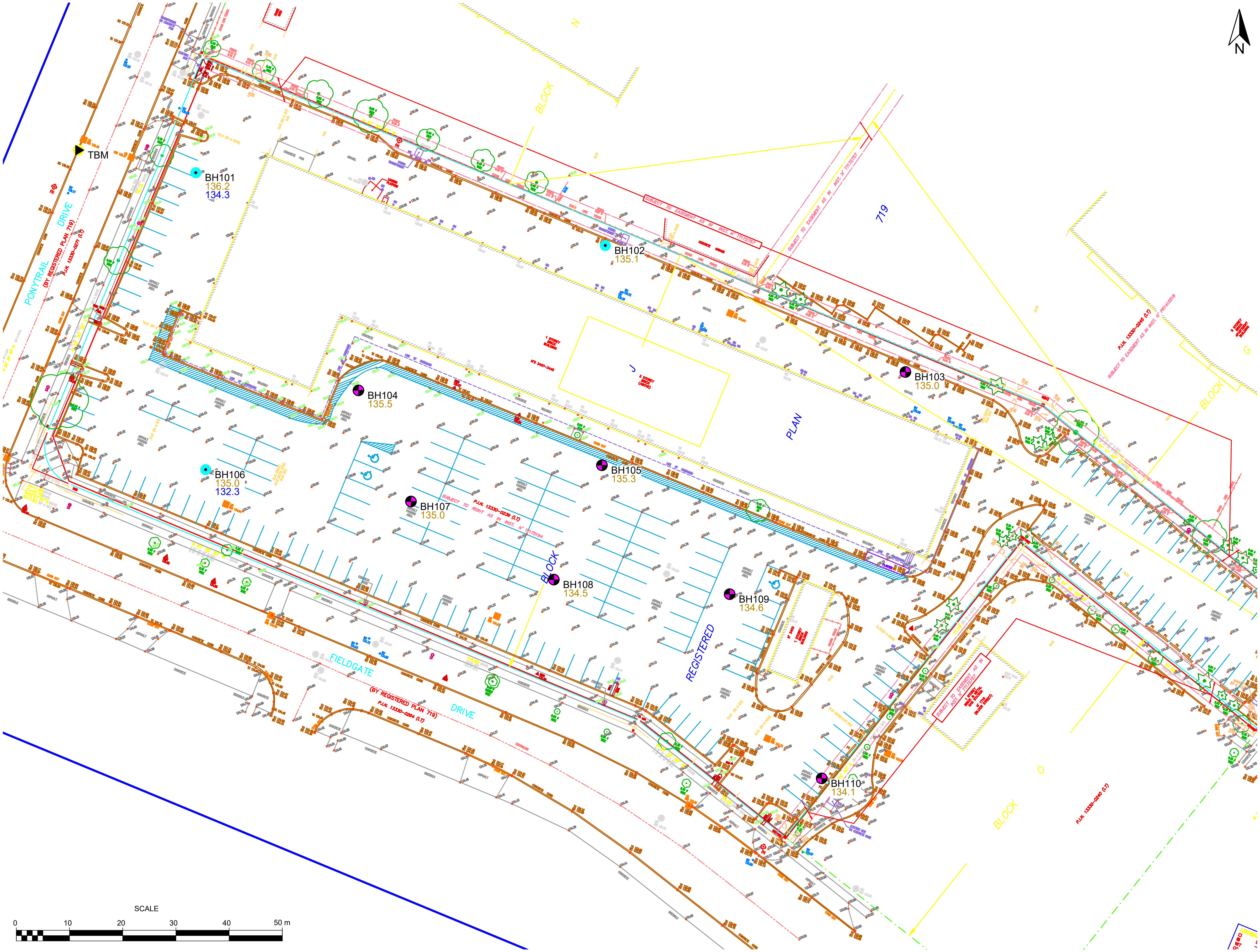
LICENSED PROFESSIONAL ENGINEER
A. ABASS
100071816
JUL 19/2024
PROVINCE OF ONTARIO

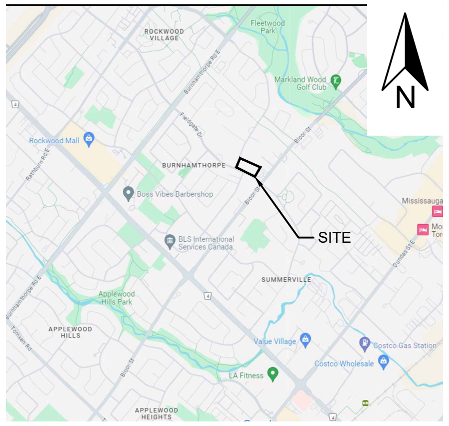
Ashraf Abass, P.Eng.
Senior Geotechnical Engineer/Project Manager

Steve Campbell, P.Geo
Principal

Appendix A:

Drawing No. 1: Borehole Location Plan





LEGEND

●	BOREHOLE LOCATION
⊙	MONITORING WELL LOCATION
▶	TEMPORARY BENCHMARK
135.0	GROUND SURFACE ELEVATION (m)
134.3	GROUNDWATER ELEVATION (2024-06-11)

REFERENCE:

DRAWING REPRODUCED USING TOPOGRAPHIC SURVEY BY GENESIS LAND SURVEYING INC. DATED APRIL 2024

TITLE:

BOREHOLE AND MONITORING WELL LOCATION PLAN

CLIENT:

FOREST GLEN SHOPPING CENTRE LTD.

LOCATION:

PROPOSED MIXED USE DEVELOPMENT
3403-3445 FIELDGATE DR, MISSISSAUGA, ONT.

PROJECT NO.: G2S24018

DRAWING: 1
SCALE: AS SHOWN
DATE: JULY 2024
DRAWN BY: DB
FILE NAME: G2S24018.dwg

Appendix B:

Borehole Logs
Grain Size Analyses
Plasticity Chart

LIST OF ABBREVIATIONS

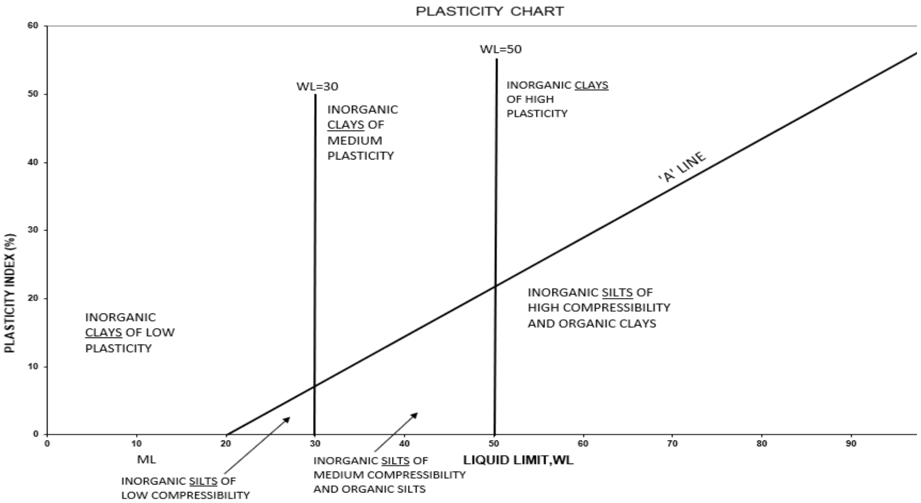
Description of Soil

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



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.

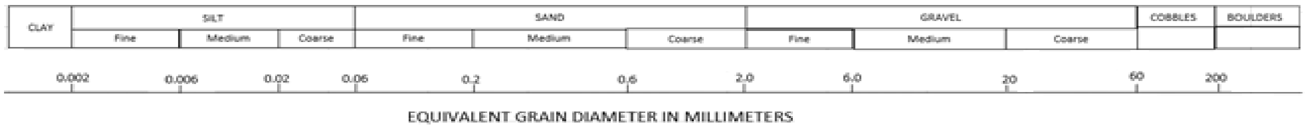
Soil Classification Dynamic Penetration Resistance: The number of blows required to advance a 51 mm, 60-degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow. Soils descriptions are made in accordance with the Canadian Foundations Engineering Manual (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



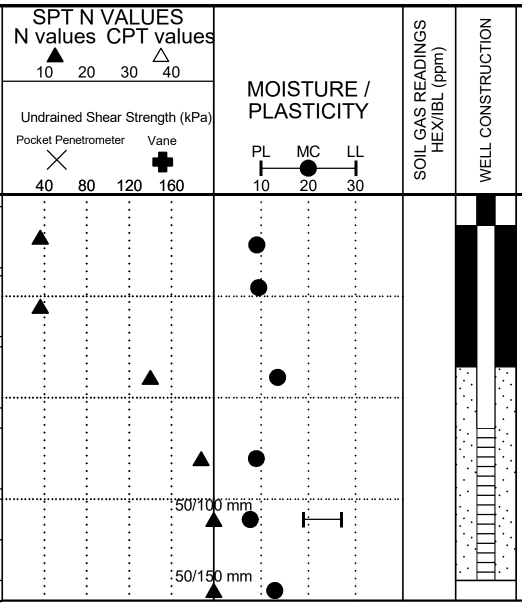
G2S Consulting Inc.

BH/MW NUMBER 101

PAGE 1 OF 1

CLIENT Sajecki Planning Inc.
PROJECT NAME Proposed Mixed Use Development
PROJECT NUMBER G2S24018
PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON
DATE STARTED 24-3-15 **COMPLETED** 24-3-15
GROUND ELEVATION 136.2 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Continuous Flight Hollow Stem Auger
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.12	ASPHALT: ~120 mm	136.08	S1A	SPT	9	
0.27	GRANULAR: ~150 mm	135.93	S1B			
1.5	FILL: Silty sand, brown, organics, moist	134.70	S2A S2B	SPT	9	
3.0	SILTY SAND TILL: Brown, some gravel, moist, dense	133.20	S3 S4	SPT SPT	35 47	
4.0	CLAYEY SILT TILL: Grey, some sand, some gravel, moist, hard	132.20	S5 S6	SPT SPT	50 50	48 19 23 10



Flushmount protective casing set in concrete

Bentonite seal

Filter sand

Slotted screen

Borehole terminated at 4.0 m.

Water Level Readings:

Date	Depth (m)	Elev. (m)
2024-04-12	1.80	134.40
2024-05-13	1.60	134.60
2024-06-11	1.90	134.30

BOREHOLE NUMBER 102

PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Sajecki Planning Inc.

PROJECT NAME Proposed Mixed Use Development

PROJECT NUMBER G2S24018

PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON

DATE STARTED 24-3-15 **COMPLETED** 24-3-15

GROUND ELEVATION 135.1 m

DRILLING CONTRACTOR Davis Drilling Ltd.

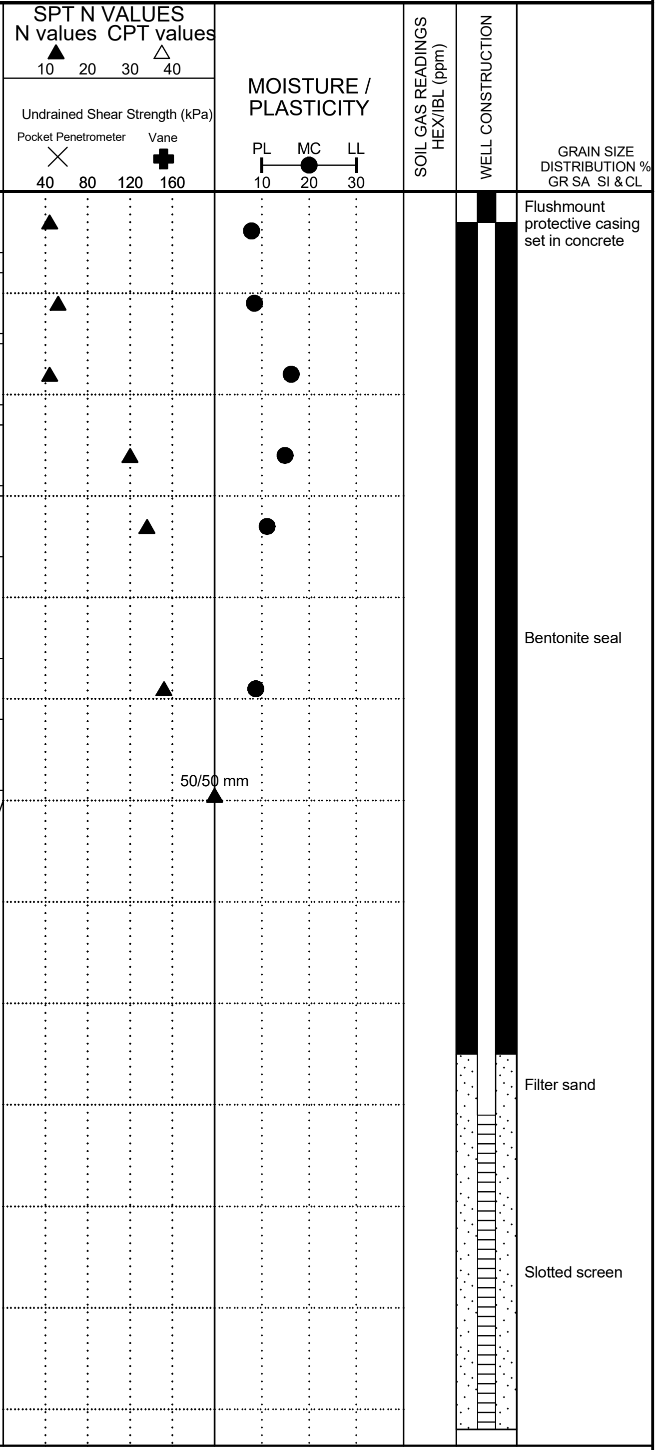
LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Continuous Flight Hollow Stem Auger+ Rock Coring

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	WELL CONSTRUCTION
0.18	GRANULAR: ~175 mm	134.93	S1A			Flushmount protective casing set in concrete
	FILL: Sand, brown, some silt, moist		S1B	SPT	11	
			S2	SPT	13	
1.5	SILT: Brown, some sand, moist, compact	133.60	S3	SPT	11	
2.3	SANDY SILT TILL: Grey, some clay, some gravel, moist,, dense	132.80	S4	SPT	30	
3.0	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, hard	132.10	S5	SPT	34	
			S6	SPT	38	Bentonite seal
6.1		129.00	S7	SPT	50	
6.2	Weathered Shale: Georgian Bay Formation, Grey	128.95	S8	RC		
6.6	SHALE BEDROCK OF THE GEORGIAN BAY FORMATION: REFER TO LOG OF ROCK CORE FOR DETAILS OF BEDROCK CORING RUN 1: Total Recovery 73% - RQD 0% Very Poor Quality Very Low to Low Strength	128.46	S9	RC		
7.8	RUN 2: Total Recovery 100% - RQD 15% Very Poor Quality Very Low to Medium Strength	127.29	S10	RC		Filter sand
9.3	RUN 3: Total Recovery 100% - RQD 21% Very Poor Quality Low to Medium Strength	125.77	S11	RC		Slotted screen
10.9	RUN 4: Total Recovery 100% - RQD 58% Fair Quality Very Low to Medium Strength	124.25	S12	RC		
12.4	RUN 5: Total Recovery 100% - RQD 49% Poor Quality Very Low to Medium Strength	122.74				

Borehole terminated at 12.4 m.



BOREHOLE NUMBER 103

PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Sajecki Planning Inc.

PROJECT NAME Proposed Mixed Use Development

PROJECT NUMBER G2S24018

PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON

DATE STARTED 24-3-14 **COMPLETED** 24-3-14

GROUND ELEVATION 135.0 m

DRILLING CONTRACTOR Davis Drilling Ltd.

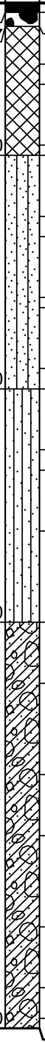
LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Continuous Flight Hollow Stem Auger

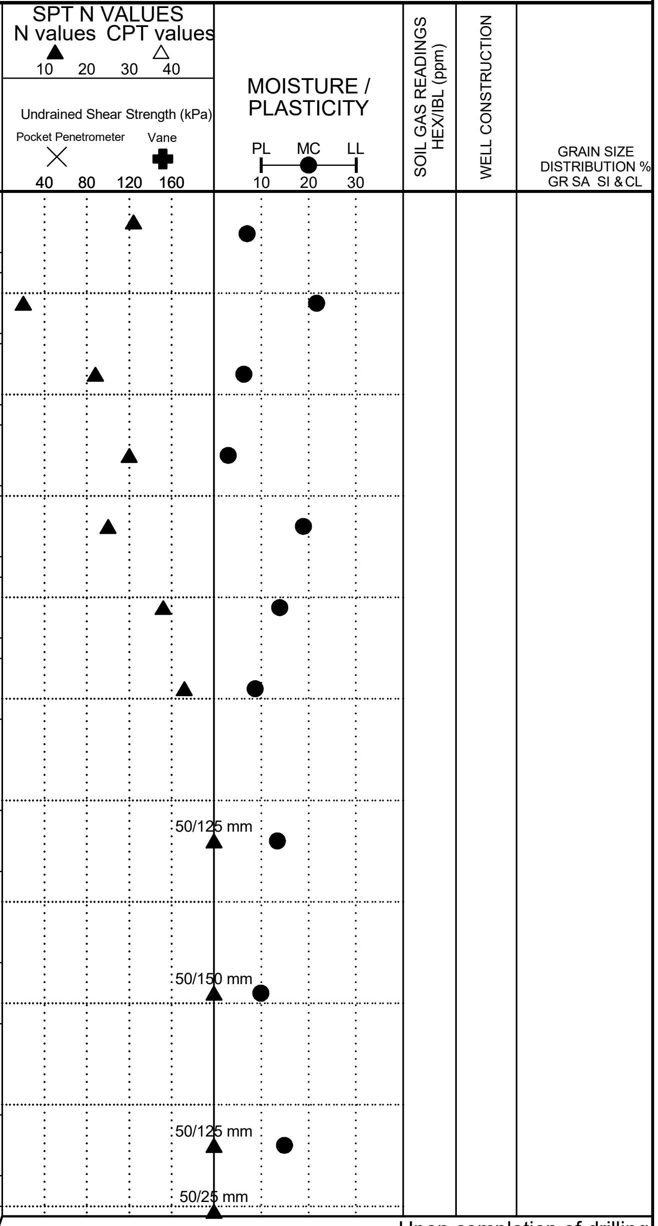
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)
0.09	ASPHALT: ~90 mm	134.91
0.23	GRANULAR: ~140 mm	134.77
1.5	FILL: Silty sand, dark brown, moist to very moist	133.50
3.8	SAND: Light brown, some silt, occasional silt zones, moist, compact	131.20
6.1	SILTY SAND TILL: Grey, some gravel to gravelly, moist, compact to dense	128.90
10.1	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, hard	124.90

GRAPHIC LOG



NUMBER	TYPE	N VALUE
S1A S1B	SPT	31
S2	SPT	5
S3	SPT	22
S4	SPT	30
S5	SPT	25
S6	SPT	38
S7	SPT	43
S8	SPT	50
S9	SPT	50
S10	SPT	50
S11	SPT	50



Borehole terminated at 10.1 m.

Upon completion of drilling
Cave at 6.7 m
Free water at 5.5 m

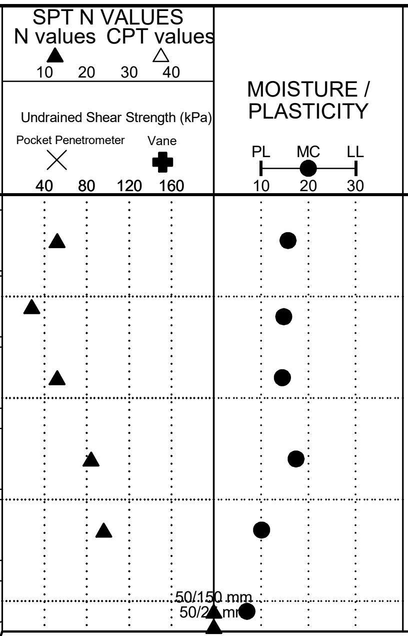
G2S Consulting Inc.

BOREHOLE NUMBER 104

PAGE 1 OF 1

CLIENT Sajecki Planning Inc.
PROJECT NAME Proposed Mixed Use Development
PROJECT NUMBER G2S24018
PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON
DATE STARTED 24-3-12 **COMPLETED** 24-3-12
GROUND ELEVATION 135.5 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Continuous Flight Hollow Stem Auger
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	ASPHALT: ~150 mm	135.35							
0.26	GRANULAR: ~110 mm	135.24		S1	SPT	13			
1.00	FILL: Silty sand, brown to dark brown, trace gravel, wood pieces, organics, moist	134.50		S2A S2B	SPT	7			
1.5	becoming brown, no organics, moist	134.00		S3	SPT	13			
	SILTY SAND/SANDY SILT: Brownish grey, trace gravel, moist, compact			S4	SPT	21			
				S5	SPT	24			
3.8	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, hard	131.70		S6	SPT	50			
4.3	No further progress due to auger and sampler refusal on possible weathered shale bedrock Borehole terminated at 4.3 m.	131.20		S7	SPT	50			



Upon completion of drilling
Cave at 2.1 m
No free water

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BOREHOLE NUMBER 105

PAGE 1 OF 1

CLIENT Sajecki Planning Inc.

PROJECT NAME Proposed Mixed Use Development

PROJECT NUMBER G2S24018

PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON

DATE STARTED 24-3-13 **COMPLETED** 24-3-13

GROUND ELEVATION 135.3 m

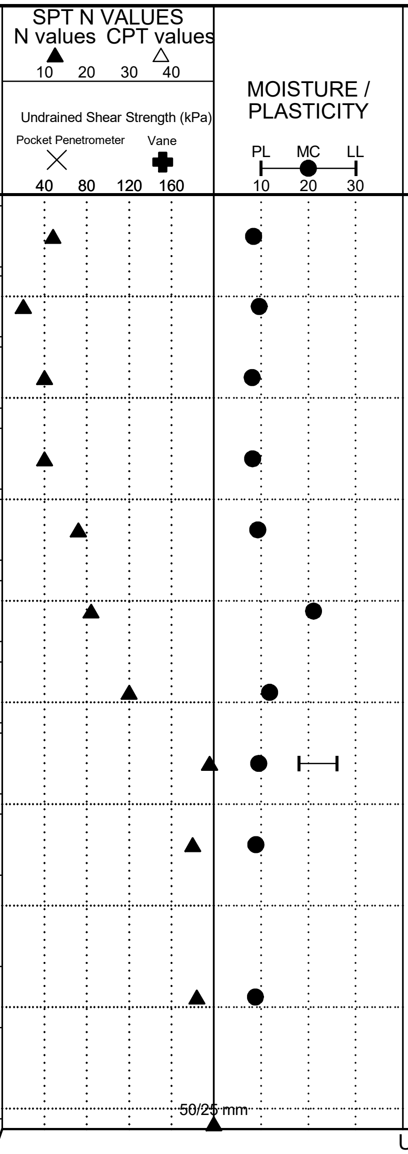
DRILLING CONTRACTOR Davis Drilling Ltd.

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Continuous Flight Hollow Stem Auger

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.11	ASPHALT: ~110 mm	135.19				
0.26	GRANULAR: ~150 mm	135.04	S1	SPT	12	
	FILL: Sand, brown, some silt, moist to very moist		S2	SPT	5	
1.5		133.80				
	SANDY SILT: Brown, very moist, compact		S3	SPT	10	
			S4	SPT	10	
			S5	SPT	18	
3.8		131.50				
	SAND: Brown, some silt, moist, compact		S6	SPT	21	
4.6		130.70				
	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, hard		S7	SPT	30	
			S8	SPT	49	40 17 34 9
			S9	SPT	45	
			S10	SPT	46	
9.1		126.20				
9.2	Weathered Shale: Georgian Bay Formation, Grey	126.10	S11	SPT	50	



SOIL GAS READINGS HEX/IBL (ppm)

WELL CONSTRUCTION

No further progress due to auger and sampler refusal on possible bedrock
Borehole terminated at 9.2 m.

Upon completion of drilling
Cave at 7.8 m
No free water

2021 G2S GEOTECH BOREHOLE LOG G2S24018 BOREHOLE LOGS (100 SERIES).GPJ G2S 2021 BH DATA TEMPLATE.GDT 24-7-19

BH/MW NUMBER 106

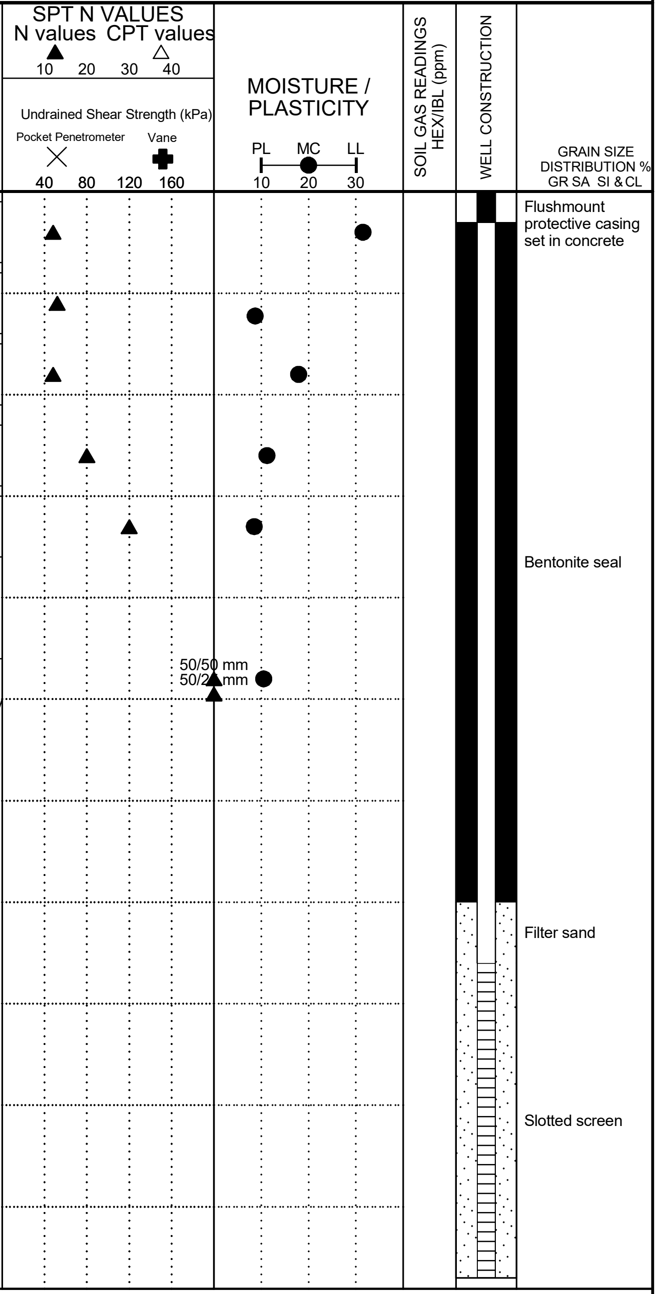
PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Sajecki Planning Inc.
PROJECT NAME Proposed Mixed Use Development
PROJECT NUMBER G2S24018
PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON
DATE STARTED 24-3-12 **COMPLETED** 24-3-12
GROUND ELEVATION 135.0 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Continuous Flight Hollow Stem Auger+ Rock Coring
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE
0.16	ASPHALT: ~160 mm	134.84			
0.24	GRANULAR: ~80 mm	134.76	S1	SPT	12
1.5	FILL: Clayey silt, dark grey, organics, trace sand, moist	133.50	S2A S2B	SPT	13
3.0	SILTY SAND/SANDY SILT TILL: Grey, some clay, some gravel, moist, compact	132.00	S3 S4	SPT SPT	12 20
5.0	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, hard	130.02	S5 S6	SPT SPT	30 50
5.2		129.79	S7	SPT	50
6.3	SHALE BEDROCK OF THE GEORGIAN BAY FORMATION: REFER TO LOG OF ROCK CORE FOR DETAILS OF BEDROCK CORING RUN 1: Total Recovery 100% - RQD 0% Very Poor Quality Very Low to Very High Strength	128.75	S8 S9	RC RC	
7.8	RUN 2: Total Recovery 92% - RQD 0% Very Poor Quality Very Low to Medium Strength	127.23	S10	RC	
9.3	RUN 3: Total Recovery 100% - RQD 39% Poor Quality Low to Medium Strength	125.71	S11	RC	
10.8	RUN 4: Total Recovery 100% - RQD 51% Fair Quality Very Low to Medium Strength RUN 5: Total Recovery 100% - RQD 70% Fair Quality Very Low to Medium Strength	124.19	S12	RC	

Borehole terminated at 10.8 m.



Water Level Readings:

Date	Depth (m)	Elev. (m)
2024-04-12	2.90	132.10
2024-05-13	2.80	132.20
2024-06-11	2.70	132.30

G2S Consulting Inc.

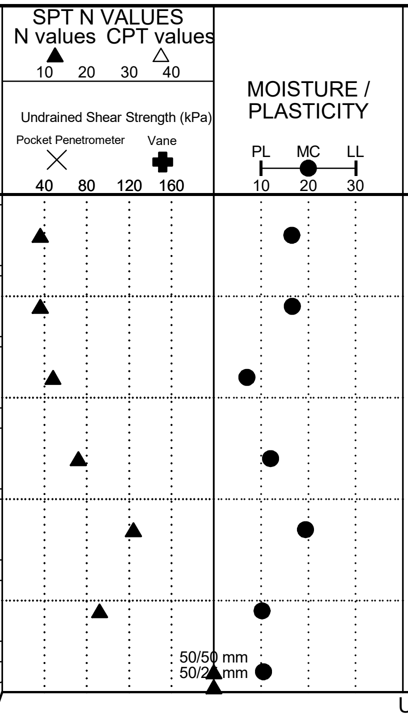
BOREHOLE NUMBER 107

PAGE 1 OF 1

CLIENT Sajecki Planning Inc.
PROJECT NAME Proposed Mixed Use Development
PROJECT NUMBER G2S24018
PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON
DATE STARTED 24-3-12 **COMPLETED** 24-3-12
GROUND ELEVATION 135.0 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Continuous Flight Hollow Stem Auger
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	NUMBER	TYPE	N VALUE
0.10	ASPHALT: ~100 mm	134.90			
0.21	GRANULAR: ~110 mm	134.79	S1	SPT	9
0.80	FILL: Sand, brown, trace silt, very moist becoming clayey silt, dark grey to grey, some sand, trace organics, very moist	134.20	S2	SPT	9
1.5	SILTY SAND/SANDY SILT: Grey, trace gravel, moist, compact to dense	133.50	S3	SPT	12
			S4	SPT	18
			S5	SPT	31
3.8	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, very stiff to hard	131.20	S6	SPT	23
4.8		130.20	S7	SPT	50
4.9	Weathered Shale: Georgian Bay Formation, Grey	130.10	S8	SPT	50

No further progress due to auger and sampler refusal on possible bedrock
Borehole terminated at 4.9 m.



SOIL GAS READINGS HEX/IBL (ppm)

WELL CONSTRUCTION

GRAIN SIZE DISTRIBUTION %
GR SA SI & CL

Upon completion of drilling
No cave
Free water at 4.8 m

BOREHOLE NUMBER 108

PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Sajecki Planning Inc.

PROJECT NAME Proposed Mixed Use Development

PROJECT NUMBER G2S24018

PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON

DATE STARTED 24-3-13 **COMPLETED** 24-3-13

GROUND ELEVATION 134.5 m

DRILLING CONTRACTOR Davis Drilling Ltd.

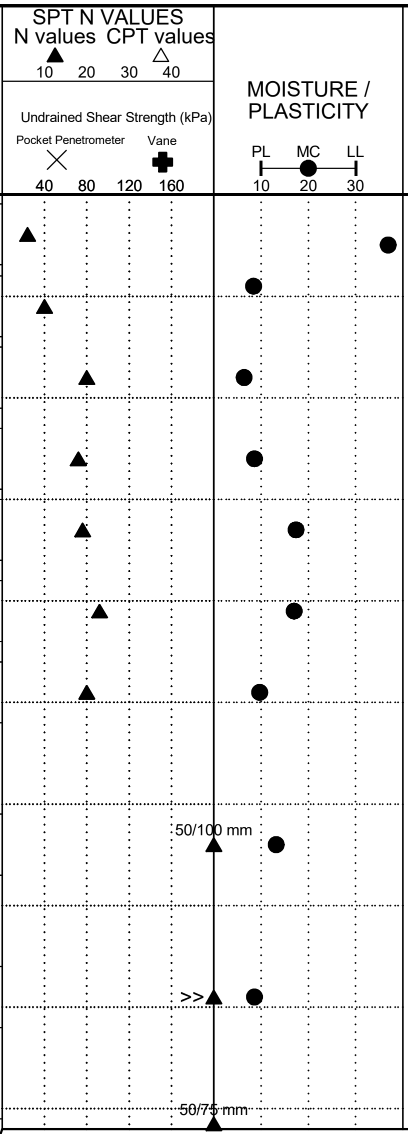
LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD Continuous Flight Hollow Stem Auger

NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)
0.09	ASPHALT: ~90 mm	134.41
0.20	GRANULAR: ~110 mm	134.30
0.30	FILL: Silty sand, brown, moist becoming silt, dark grey, organics, some sand, moist	134.20
1.5	SILTY SAND: Brown, trace gravel, moist, compact	133.00
4.9	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, very stiff to hard	129.60
9.1	Weathered Shale: Georgian Bay Formation, Grey	125.40
9.2	No further progress due to auger and sampler refusal on possible bedrock Borehole terminated at 9.2 m.	125.30

NUMBER	TYPE	N VALUE
S1A S1B	SPT	6
S2A S2B	SPT	10
S3	SPT	20
S4	SPT	18
S5	SPT	19
S6	SPT	23
S7A S7B	SPT	20
S8	SPT	50
S9	SPT	74
S10	SPT	50



SOIL GAS READINGS HEX/IBL (ppm)

WELL CONSTRUCTION

GRAIN SIZE DISTRIBUTION % GR SA SI & CL

Upon completion of drilling
Wet cave at 4.3 m

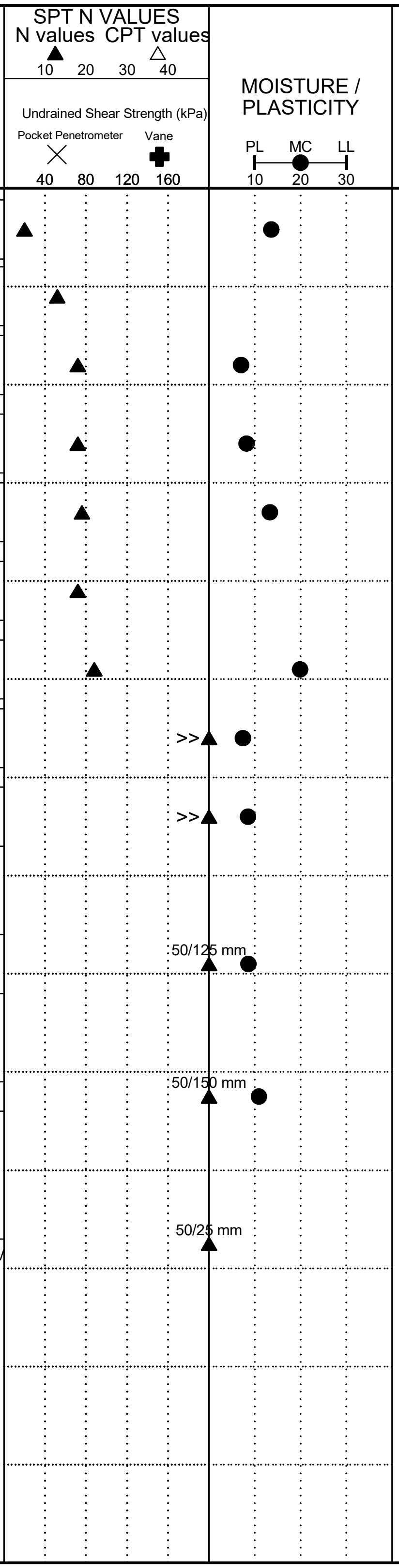
BOREHOLE NUMBER 109

G2S Consulting Inc.

CLIENT Sajecki Planning Inc.
PROJECT NAME Proposed Mixed Use Development
PROJECT NUMBER G2S24018
PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON
DATE STARTED 24-3-14 **COMPLETED** 24-3-14
GROUND ELEVATION 134.5 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Continuous Flight Hollow Stem Auger+ Rock Coring
NOTES

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)
0.12	ASPHALT: ~120 mm	134.38
0.24	GRANULAR: ~120 mm	134.26
0.80	FILL: Silty sand, dark brown to brown, trace gravel, moist	133.70
5.3	SILTY SAND: Brown, trace gravel, reworked appearance at top portion, moist, compact	129.20
7.6	SILTY SAND TILL: Grey, some gravel, moist, very dense	126.90
10.7	SILTY CLAY TILL: Grey, some sand, some gravel, shale fragments, moist, hard	123.80
11.0	Weathered Shale: Georgian Bay Formation, Grey	123.55
12.3	SHALE BEDROCK OF THE GEORGIAN BAY FORMATION: REFER TO LOG OF ROCK CORE FOR DETAILS OF BEDROCK CORING RUN 1: Total Recovery 100% - RQD 35% Poor Quality Very Low to High Strength	122.19
13.8	RUN 2: Total Recovery 93% - RQD 30% Poor Quality	120.67

NUMBER	TYPE	N VALUE	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
S1	SPT	5	
S2	SPT	13	
S3	SPT	18	
S4	SPT	18	
S5	SPT	19	
S6	SPT	18	
S7	SPT	22	
S8	SPT	58	
S9	SPT	54	
S10	SPT	50	54 17 21 8
S11	SPT	50	
S12	SPT	50	
S13	RC		
S14	RC		



SOIL GAS READINGS HEX/IBL (ppm)

WELL CONSTRUCTION

(Continued Next Page)

Consulting Inc.

BOREHOLE NUMBER 109

CLIENT Sajecki Planning Inc.

PROJECT NAME Proposed Mixed Use Development

PROJECT NUMBER G2S24018

PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON

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
15 15.4	Low to High Strength RUN 3: Total Recovery 100% - RQD 45% Poor Quality Very Low to Medium Strength *(continued)*	119.15	▨	S15	RC						

Borehole terminated at 15.4 m.

Cave-in material and free water were not measured due to drilling/coring method

G2S Consulting Inc.

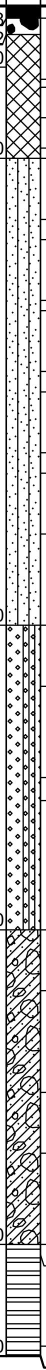
BOREHOLE NUMBER 110

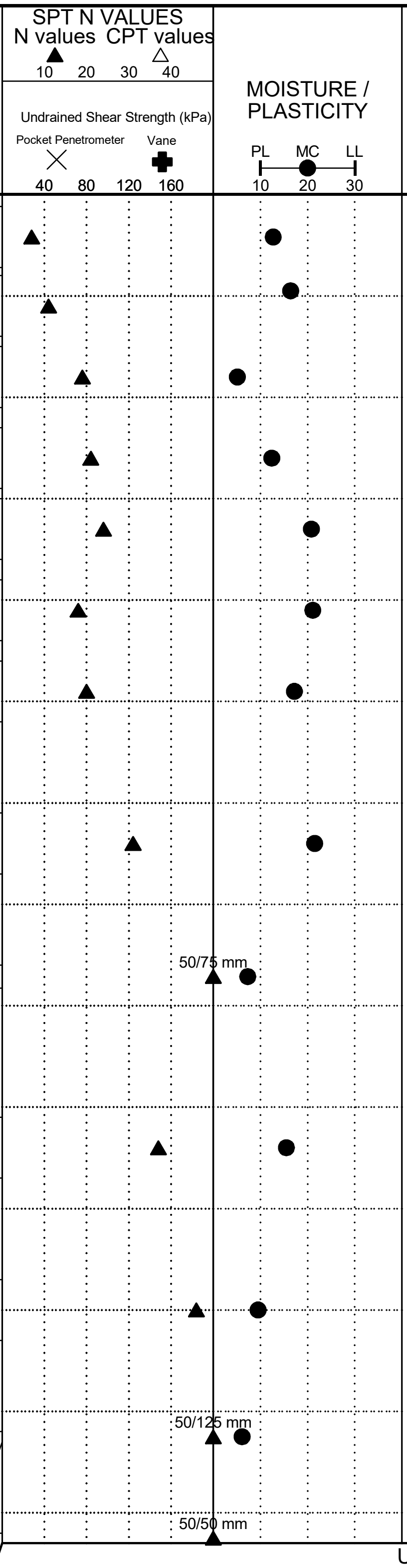
PAGE 1 OF 1

CLIENT Sajecki Planning Inc.
PROJECT NAME Proposed Mixed Use Development
PROJECT NUMBER G2S24018
PROJECT LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON
DATE STARTED 24-3-13 **COMPLETED** 24-3-13
GROUND ELEVATION 134.1 m
DRILLING CONTRACTOR Davis Drilling Ltd.
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Continuous Flight Hollow Stem Auger
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
0.12	ASPHALT: ~120 mm	133.98						
0.28	GRANULAR: ~160 mm	133.82						
0.60	FILL: Sand and gravel, brown, some silt, moist	133.50	S1	SPT	7			
			S2A S2B	SPT	11			
1.5	becoming sandy silt, dark grey, organics, moist	132.60						
	SILTY SAND: Brown to grey, trace gravel, moist to wet, compact		S3	SPT	19			
			S4	SPT	21			
			S5	SPT	24			
			S6	SPT	18			
			S7	SPT	20			
6.1	SILTY SAND TILL: Grey, some gravel, moist, dense to very dense	128.00	S8	SPT	31			
			S9	SPT	50			
9.1	CLAYEY SILT TILL: Grey, some sand, some gravel, shale fragments, moist, hard	125.00	S10	SPT	37			
			S11	SPT	46			
12.2	WEATHERED SHALE: Grey, very dense	121.90	S12	SPT	50			
13.3		120.80	S13	SPT	50			

GRAPHIC LOG





No further progress due to auger and sampler refusal on possible bedrock

Borehole terminated at 13.3 m.

Upon completion of drilling
Cave a 4.5 m
Free water at 4.3 m

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

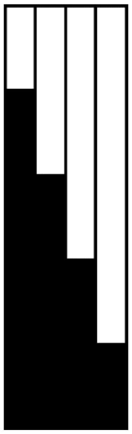
RU = Rough Undulating
RP = Routh Planar
SU = Smooth Undulating
SP = Smooth Planar
LU = Slickensided Undulating
LP = Slickensided Planar

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

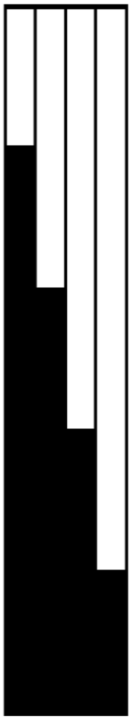
Slightly weathered = partial discoloration: fractures (joints) typically oxidized

Moderately weathered = total discoloration

Highly weathered = total discoloration: typically, friable & pitted

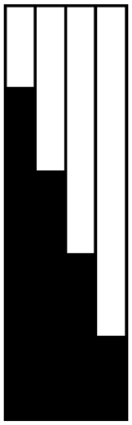
Completely weathered = resembles soil: rock structure usually preserved

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. 102

PROJECT Proposed Mixed Use Development	**ORIENTATION** Vertical	**ELEVATION (m)** 135.1	**DATUM**	**PROJECT NUMBER** G2S24018
LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON	**DATE STARTED** 03-15-24	**COMPLETED** 03-15-24	**LOGGED BY** DB	**DRAWING NUMBER**
CLIENT Sajecki Planning Inc.	**DRILLER** Davis Drilling Ltd.	**DRILL TYPE** CME 75 Truck	**CORE BARREL** HQ	**SHEET** of

				JOINT CHARACTERISTICS														
ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	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
129.0																		
					B	F	VC	RU	SA									
								RP										
								RU										
														S8	73	0	100	Grey
					F	V		RP	Si									
					B	F		RU	SA									
128.6							C	RP										
								RU										
128.5					F	V												
					B	F	VC		Si									
									SA									
									Si									
							C											
							VC											
	7																	
128.1																		
							C											
127.9																		
					C	D	VC	RP										
					B	F		RU	SA					S9	100	15	100	Grey
							C	RP										
127.8							VC	RU										
127.8									Si									
									SA									
							C		Si									
									SA									
								SU										
							VC	RP										
								SU										
								RP										
								RU										
	8						C	SP	SA									
							VC	SU										
								RU										
							C	RP										
								SP										
							VC	RU										
							C	SP										
								RU						S10	100	21	100	Grey
									Si									
								RP										
								SP	NC									
	9							RP	Si									
							VC											
								RU	SA									
							C	SP										
								RP										

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ROCK CORE LOG

BH NO. 102

PROJECT	**ORIENTATION**	**ELEVATION (m)**	**DATUM**	**PROJECT NUMBER**
Proposed Mixed Use Development	Vertical	135.1		G2S24018
LOCATION	**DATE STARTED**	**COMPLETED**	**LOGGED BY**	**DRAWING NUMBER**
3403-3445 Fieldgate Dr, Mississauga, ON	03-15-24	03-15-24	DB	
CLIENT	**DRILLER**	**DRILL TYPE**	**CORE BARREL**	**SHEET**
Sajecki Planning Inc.	Davis Drilling Ltd.	CME 75 Truck	HQ	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
							VC	RU										
								RP										
								SP										
							C	RU										
								RP										
							VC	SP										
							C											
	10							SU										
							VC	RU										
							C							S11	100	58	100	Grey
							VC											
							C											
							VC	RP										
							C	RU	Si									
								SP	SA									
								SU										
							VC	RU										
	11						C	SU										
							VC											
124.0								SP										
124.0								RU										
							C	SU										
							VC	RU										
							C											
							VC	SU										
								SP						S12	100	49	100	Grey
								RU										
								SP										
							C	RU										
							VC											
	12								Si									
							C	SP	SA									
								RP	Si									
122.7																		
								RU	SA									

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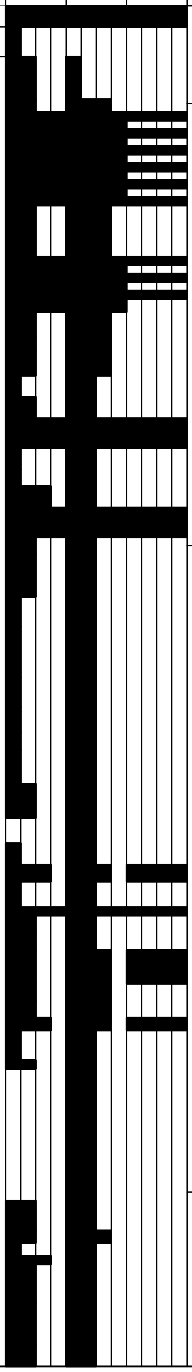
ROCK CORE LOG

BH NO. 106

PROJECT	ORIENTATION	ELEVATION (m)	DATUM	PROJECT NUMBER
Proposed Mixed Use Development	Vertical	135.0		G2S24018
LOCATION	**DATE STARTED**	**COMPLETED**	**LOGGED BY**	**DRAWING NUMBER**
3403-3445 Fieldgate Dr, Mississauga, ON	03-12-24	03-12-24	DB	
CLIENT	**DRILLER**	**DRILL TYPE**	**CORE BARREL**	**SHEET**
Sajecki Planning Inc.	Davis Drilling Ltd.	CME 75 Truck	HQ	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
130.0	5																	
130.0																		
					B	F	VC	RP	Si									
129.9														S8	100	0	100	Grey
							C	RU										
							VC	SP										
								RP										
								RU						S9	92	0	100	Grey
							C	RP										
							VC											
							C	SP	SA									
							VC	RU	Si									
							C	SP										
	6						VC	SU	SA									
								RU	Si									
								SP	SA									
							C	RU										
							VC	RP	Si									
									SA									
							C	SP										
								SU										
								SP										
							VC	SU										
								SP										
							C	SU	Si									
								RU	SA									
	7						VC	RP	Si					S10	100	39	100	Grey
									SA									
							C	RU	Si									
							VC	SP	SA									
								RU	Si									
							C											
							VC											
							C	RP										
							VC	RU										
								RP	SA									
							C	SP										
								RP	Si									
							VC	RU	SA									
								RP										
	8						C	SP										
								SU										

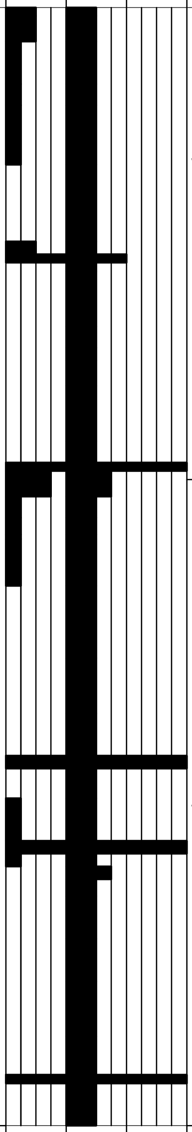


ROCK CORE LOG

BH NO. 106

PROJECT Proposed Mixed Use Development	**ORIENTATION** Vertical	**ELEVATION (m)** 135.0	**DATUM**	**PROJECT NUMBER** G2S24018
LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON	**DATE STARTED** 03-12-24	**COMPLETED** 03-12-24	**LOGGED BY** DB	**DRAWING NUMBER**
CLIENT Sajecki Planning Inc.	**DRILLER** Davis Drilling Ltd.	**DRILL TYPE** CME 75 Truck	**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
								RU	Si									
								RP	SA									
							VC		Si									
														S11	100	51	100	Grey
							C	SP	SA									
							VC	RP RU	Si									
								SP	SA									
	9						C VC C	RP	Si SA									
							VC	SU RU	Si									
							C VC	SP RP SP RP	SA									
							C											
								SP										
								RP										
							VC	RU	Si									
	10						C							S12	100	70	100	Grey
							VC	RP SU RU	SA Si									
							C	RP SU	SA									
								RP										
								SP	Si									
							VC	SU RU										
124.2							C	RP	SA									
	11																	



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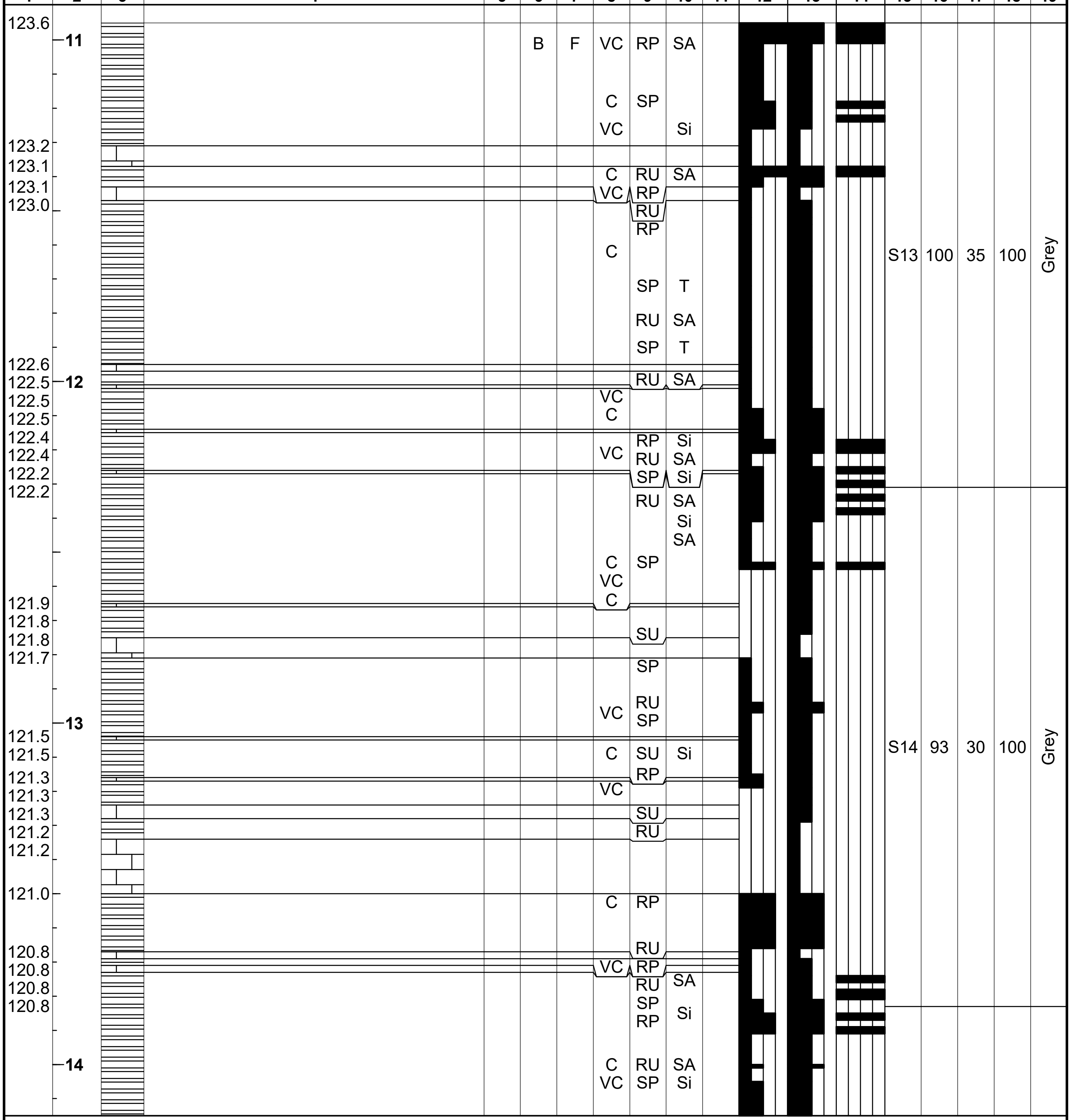
ROCK CORE LOG

BH NO. 109

PROJECT	ORIENTATION	ELEVATION (m)	DATUM	PROJECT NUMBER
Proposed Mixed Use Development	Vertical	134.5		G2S24018
LOCATION	**DATE STARTED**	**COMPLETED**	**LOGGED BY**	**DRAWING NUMBER**
3403-3445 Fieldgate Dr, Mississauga, ON	03-14-24	03-14-24	DB	
CLIENT	**DRILLER**	**DRILL TYPE**	**CORE BARREL**	**SHEET**
Sajecki Planning Inc.	Davis Drilling Ltd.	CME 75 Truck	HQ	of



				JOINT CHARACTERISTICS														
ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	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
123.6																		
	11				B	F	VC	RP	SA									
							C	SP										
							VC		Si									
123.2																		
123.1																		
							C	RU	SA									
123.1							VC	RP										
123.0								RU										
								RP										
							C							S13	100	35	100	Grey
								SP	T									
								RU	SA									
								SP	T									
122.6																		
122.5	12							RU	SA									
122.5							VC											
122.5							C											
122.4								RP	Si									
122.4							VC	RU	SA									
122.2								SP	Si									
122.2								RU	SA									
									Si									
									SA									
							C	SP										
							VC											
121.9							C											
121.8																		
121.8								SU										
121.7																		
								SP										
								RU										
							VC											
	13							SP										
121.5														S14	93	30	100	Grey
121.5							C	SU	Si									
121.3								RP										
121.3							VC											
121.3								SU										
121.2								RU										
121.2																		
121.0							C	RP										
120.8								RU										
120.8							VC	RP										
120.8								RU	SA									
120.8								SP										
									Si									
								RP										
	14						C	RU	SA									
							VC	SP	Si									

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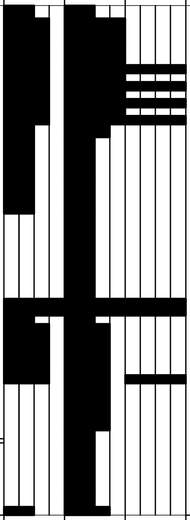
ROCK CORE LOG

BH NO. 109

PROJECT Proposed Mixed Use Development	**ORIENTATION** Vertical	**ELEVATION (m)** 134.5	**DATUM**	**PROJECT NUMBER** G2S24018
LOCATION 3403-3445 Fieldgate Dr, Mississauga, ON	**DATE STARTED** 03-14-24	**COMPLETED** 03-14-24	**LOGGED BY** DB	**DRAWING NUMBER**
CLIENT Sajecki Planning Inc.	**DRILLER** Davis Drilling Ltd.	**DRILL TYPE** CME 75 Truck	**CORE BARREL** HQ	**SHEET** of

				JOINT CHARACTERISTICS														
ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	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
							C VC	RP	SA Si									
					F	V		RU	SA									
							C	SP	T					S15	100	45	100	Grey
					B	F		RU										
							VC	RP	Si SA									
	15						C VC	RU	Si									
119.3							C		SA									
119.3							VC	RP	T									
							C	RU	Si									
119.2							VC	RP SP	SA									
	16																	
	17																	



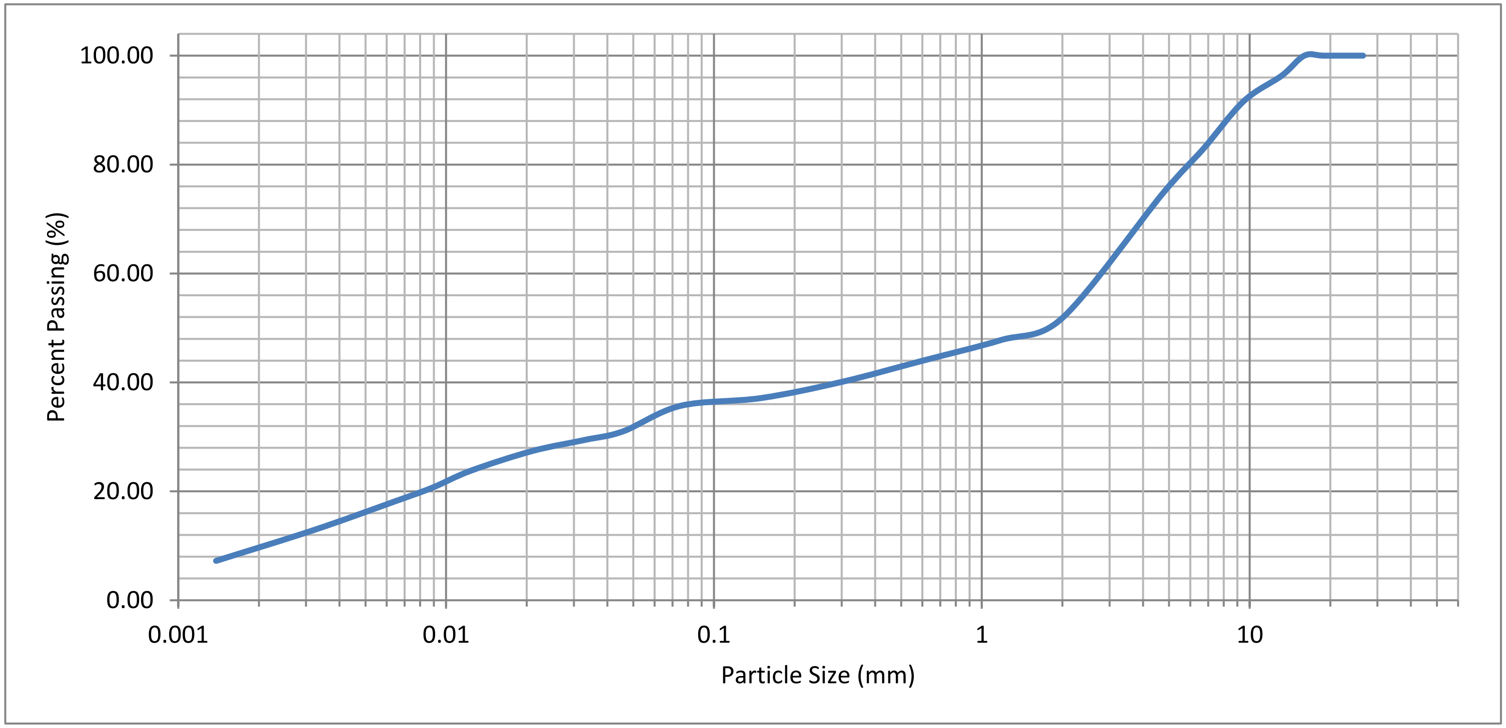
G2S Consulting Inc.

Project No.:	G2S24018C	**Lab No.:**	24027A
Project Name:	Proposed Mixed Use Development - Fieldgate Dr., Mississuaga	**Borehole/Sample No.:**	BH101-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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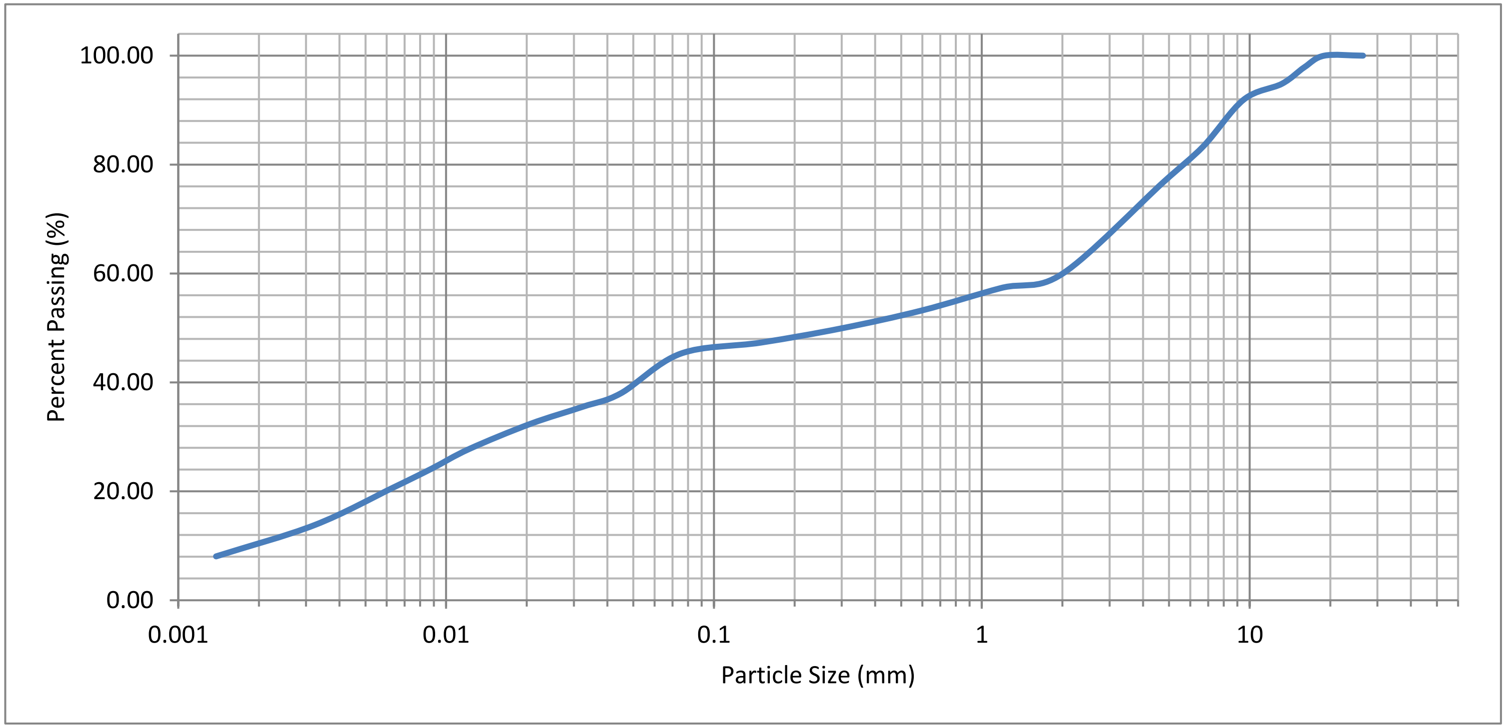
4361 Harvester Rd., Unit 12, Burlington, ON L7L 5M4
P 905.331.3735 F 905.642.5999
G2Senvironmental.com

Project No.:	G2S24018C	**Lab No.:**	24027B
Project Name:	Proposed Mixed Use Development - Fieldgate Dr., Mississuaga	**Borehole/Sample No.:**	BH105-S8

ISSMGE SOIL CLASSIFICATION

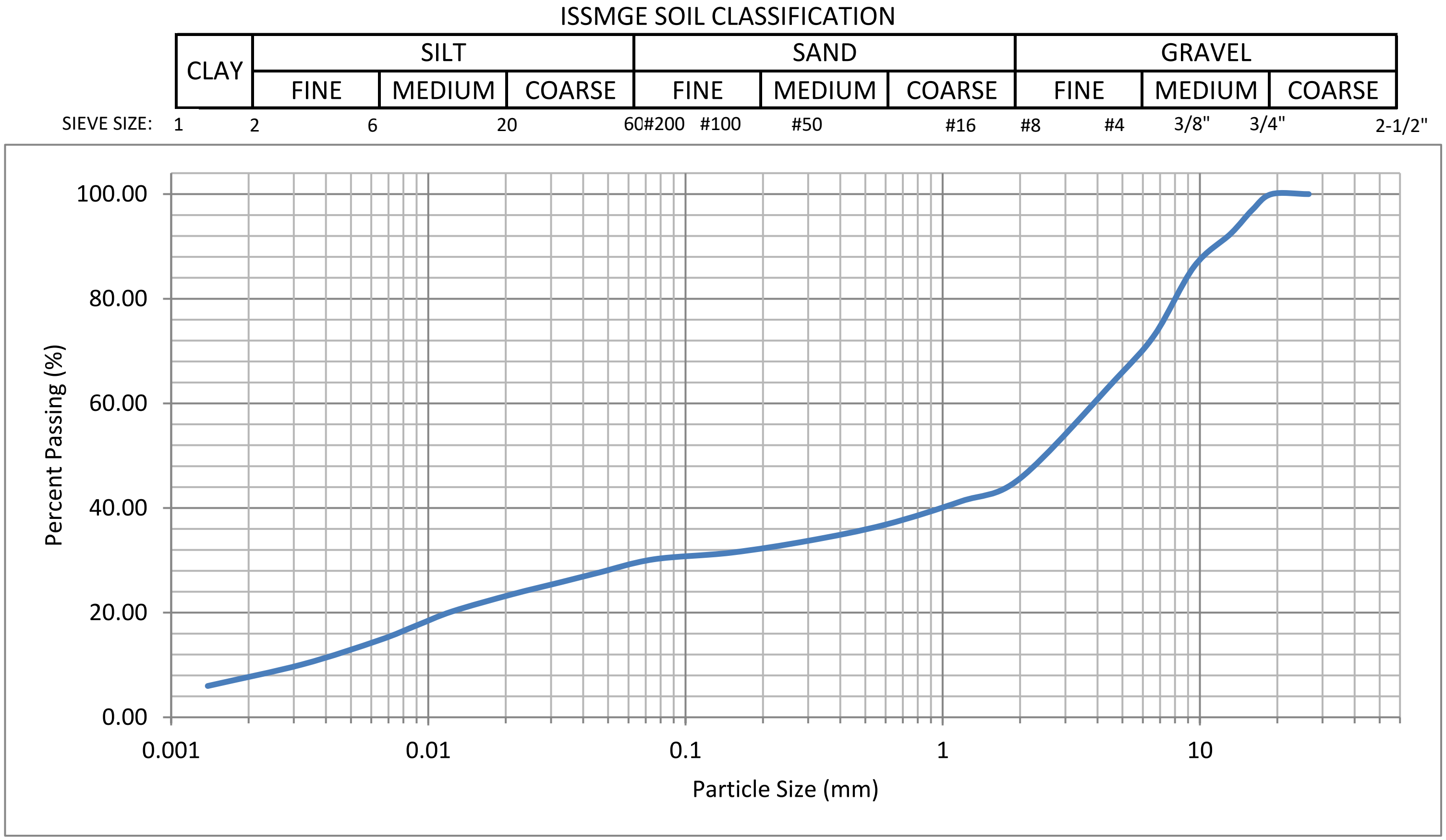
CLAY	SILT			SAND			GRAVEL		
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE

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



G2S Consulting Inc.
4361 Harvester Rd., Unit 12, Burlington, ON L7L 5M4
P 905.331.3735 F 905.642.5999
G2Senvironmental.com

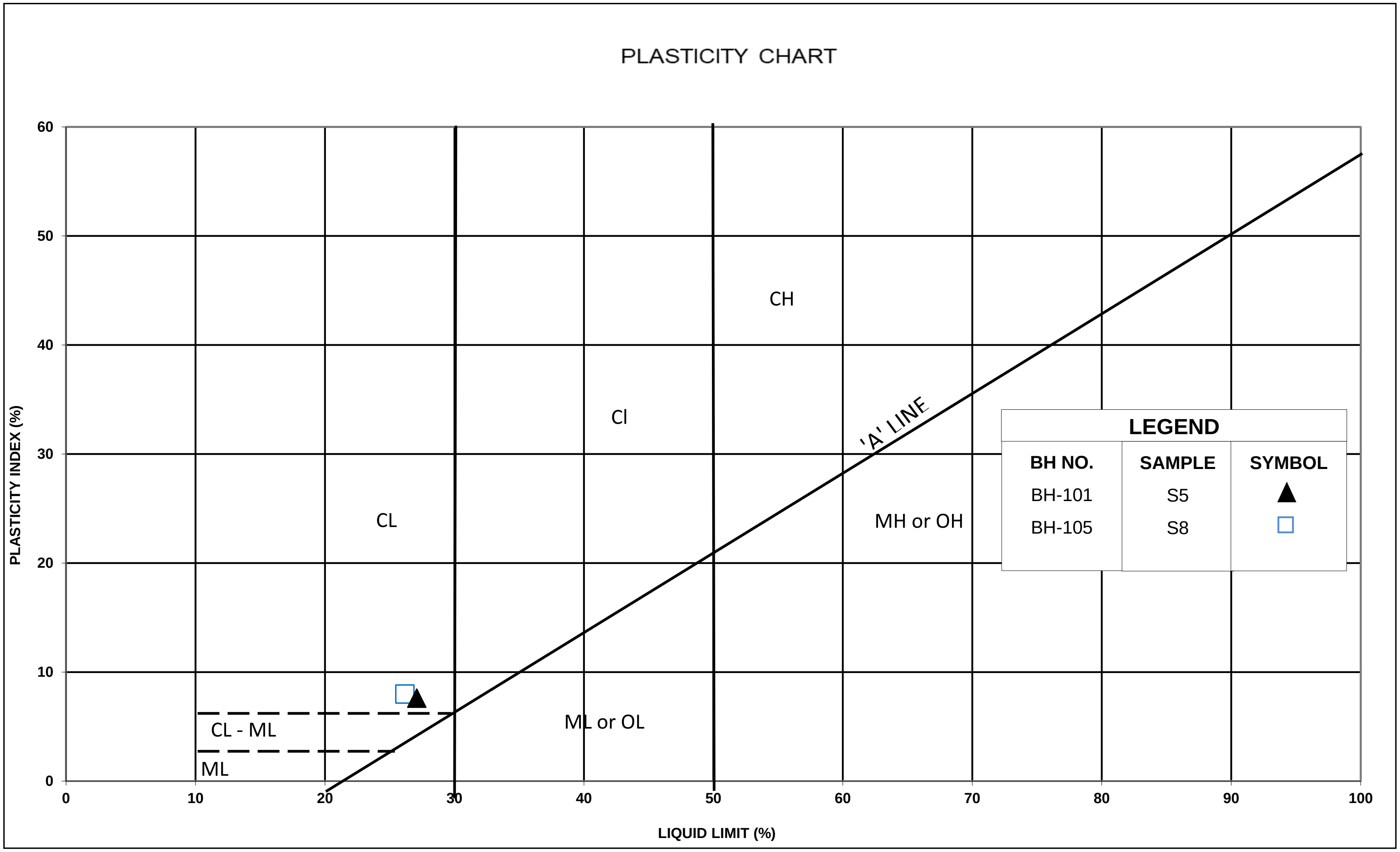
Project No.:	G2S24018C	**Lab No.:**	24027C
Project Name:	Proposed Mixed Use Development - Fieldgate Dr., Mississuaga	**Borehole/Sample No.:**	BH109-S10



G2S
Consulting Inc.
4361 Harvester Rd., Unit 12, Burlington, ON L7L 5M4
P 905.331.3735 F 905.642.5999
G2Sconsulting.com

Project No.: G2S24018C **Lab No.:** 24027 A&B

Project Name: Proposed Development - Fieldgate Dr., Mississuaga **Borehole/Sample No.:** BH101, BH105



Appendix C:

Unconfined Compression Strength (UCS) Laboratory Test Results
Retrieved Core Sample Photographs

G2S
Consulting Inc.
4361 Harvester Rd., Unit 12, Burlington, ON L7L 5M4
P 905.331.3735 F 905.642.5999
G2Sconsulting.com

Rock Core Compressive Strength Test Report

Project: 3403-3445 Fieldgate Drive, Mississauga, Ontario

Project No.: G2S24018C
Lab No.: 24133

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)
24133-A	BH102 - Run 3 8.5 - 8.58	11-Jun-24	613.0	62	75	2707	1.21	101.0	33.5	0.92	30.8
24133-B	BH102 - Run 4 9.7 - 9.81	11-Jun-24	794.9	62	97	2714	1.56	83.2	27.6	0.96	26.5
24133-C	BH106 - Run 3 7.29 - 7.42	11-Jun-24	916.4	62	115	2639	1.85	85.8	28.4	0.98	27.9
24133-D	BH106 - Run 5 7.93 - 8.05	11-Jun-24	1015.0	62	124	2711	2.00	163.5	54.2	0.98	53.1
24133-E	BH109 - Run 2 12.4 - 12.5	11-Jun-24	747.3	62	91	2720	1.47	128.2	42.5	0.94	39.9

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

Photo No. 1: BH102 – Run Nos. 1 & 2 – Box 1 (1 of 1)

Photo No. 2: BH102 – Run Nos. 1 & 2 – Box 1 (1 of 3)

Photo No. 3: BH102 – Run Nos. 1 & 2– Box 1 (2 of 3)

Photo No. 4: BH102 – Run Nos. 1 & 2– Box 1 (3 of 3)

Photo No. 5: BH102 – Run Nos. 3 & 4 – Box 2 (1 of 1)

Photo No. 6: BH102 – Run Nos. 3 & 4 – Box 1 (1 of 3)

Photo No. 7: BH102 – Run Nos. 3 & 4 – Box 2 (2 of 3)

Photo No. 8: BH102 – Run Nos. 3 & 4 – Box 2 (3 of 3)

Photo No. 9: BH106 – Run Nos. 1, 2, 3 & 4 – Box 1 (1 of 1)

Photo No. 10: BH106 – Run Nos. 1, 2, 3 & 4 – Box 1 (1 of 3)

Photo No. 11: BH106 – Run Nos. 1, 2, 3 & 4 – Box 1 (2 of 3)

Photo No. 12: BH106 – Run Nos. 1, 2, 3 & 4 – Box 1 (3 of 3)

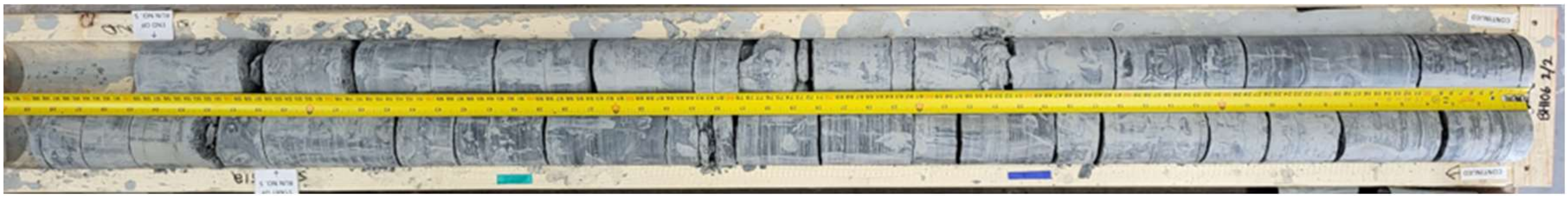


Photo No. 13: BH106 – Run Nos. 4 & 5 – Box 2 (1 of 1)

Photo No. 14: BH106 – Run Nos. 4 & 5 – Box 2 (1 of 3)

Photo No. 15: BH106 – Run Nos. 4 & 5 – Box 2 (2 of 3)

Photo No. 17: BH106 – Run Nos. 4 & 5 – Box 2 (3 of 3)

Photo No. 18: BH109 – Run Nos. 1, 2 & 3 – Box 1 (1 of 1)

Photo No. 19: BH109 – Run Nos. 1, 2 & 3 – Box 1 (1 of 3)

Photo No. 20: BH109 – Run Nos. 1, 2 & 3 – Box 1 (2 of 3)

Photo No. 21: BH109 – Run Nos. 1, 2 & 3 – Box 1 (3 of 3)

Photo No. 22: BH109 – Run No. 3 – Box 2 (1 of 1)

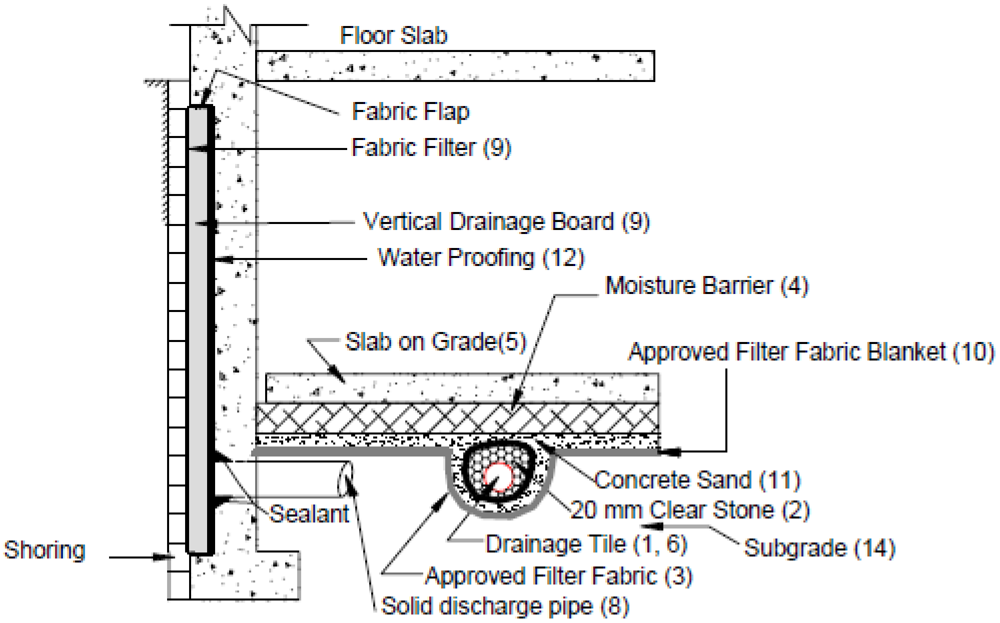
Photo No. 23: BH109 – Run No. 3 – Box 2 (1 of 3)

Photo No. 24: BH109 – Run No. 3 – Box 2 (2 of 3)

Photo No. 25: BH109 – Run No. 3 – Box 2 (3 of 3)

Appendix D:

Drainage and Underpinning Recommendation Figures



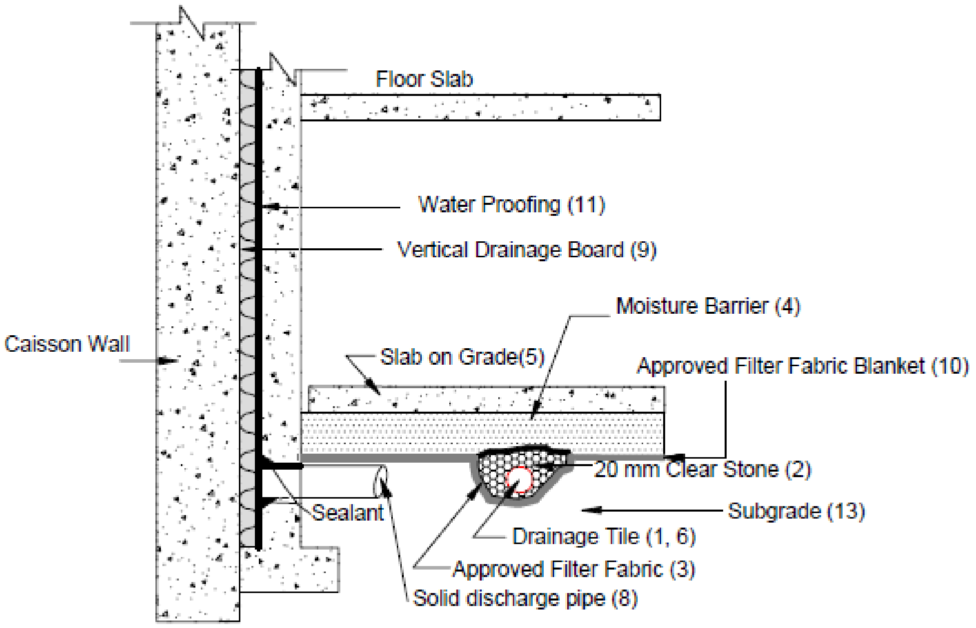
EXTERIOR FOOTING

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 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

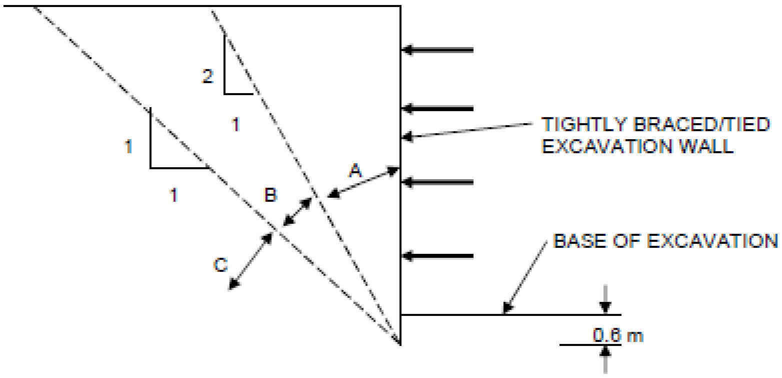
1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain.
3. Wrap 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.