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September 6, 2024 Reference No. G2S21366D

Attention: Mr. Steve Assaff, President
Charis Developments Ltd.
186 Hurontario Street, Suite 204
Collingwood, Ontario
L9Y 4T4

Technical Memorandum
Re: Additional Geotechnical Investigation
839, 853 and 869 Hurontario Street & 7564 Poplar Sideroad
Collingwood, Ontario

1. Introduction & Background

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

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

Based on the updated Site Plan provided by the client and dated May 15, 2024, the property at 853 Hurontario Street was added to the proposed development and consequently the proposed initial scope of work was altered to include construction of a commercial plaza including the following phases:

- Phase 1
    - Three (3) slab on grade retail buildings; Starbucks Café, Dollar Store , and McDonald Restaurant, with building footprint of 225.5, 929.0, and 410.9 m^2, respectively.
    - Associated site servicing and above ground parking
- Phase 2
    - Two (2) slab on grade commercial buildings; Sobey’s grocery store and a retail building with building footprint of 4,405.8 and 1,384.8 m^2, respectively.
    - Associated site servicing and above ground parking
- Phase 3
    - Twelve (12) storey mixed-use building consisting of commercial on ground floor and residential above, and 2 underground parking levels
    - Retail building with offices above (3 storey), with building footprint of 599.8 m^2
    - Associated site servicing

The purpose of this geotechnical investigation was to determine the subsurface conditions at two (2) borehole locations and to interpret these findings with respect to the design and construction of the proposed structures (Phase 3) including underground services, foundations and related earthworks for this project from a geotechnical point-of-view. In addition, the current geotechnical investigation will use information from the previous investigation to address the issues resulting from the alteration of the initial scope of work and design concept.

Any geotechnical recommendations arising from this investigation have been addressed in the previously submitted report which was utilized in the preparation of this technical memorandum, that is considered an integral part of the investigation/study. This technical memorandum should be read in conjunction with the previous geotechnical report.

The Site is rectangular shaped, comprising an approximate plan area of 3.9 hectares (9.6 acres) in size, and is located approximately 2.7 km south of Georgian Bay, in an area consisting of residential, commercial, and vacant land uses. The Site is currently vacant, undeveloped land, apart from the northwest corner of the Site, which constitutes the property of 853 Hurontario Street and is currently occupied by a single family dwelling. It is understood that the current structures will be demolished to make way for a new twelve-storey mixed use building with up to two levels of underground parking.

2. Investigation Methodology

2.1 Phase Nos. 1 & 2

The updated Site Plan dated, May 15, 2024, included the following changes from the initial conceptual site plan at the area of Phase 1 & 2.

- Building 01 (BH121), former Building B, the building footprint was reduced, and the building orientation was altered.
- Building 02 (BH105 & BH118), former Building C, the building footprint was increased.
- Building 05, Sobey’s (BH106, BH107, and BH117), former Building D, the building footprint was increased.
- Building 06, (BH113, BH114, and BH115), former Building E, the building footprint was extended adjoining the south side of Sobey’s.
- Building 07 (BH112 & BH123) former Building A (convenience store) was slightly moved southward, the gas pumps and gas tanks were removed. .

Based on the above information, the changes from the initial conceptual plan were found insignificant and the recommendations, which were included in our previous Report No. G2S21366C, and were based on the above mentioned borehole information are considered valid.

2.2 Phase No. Phase 3

A total of two (2) investigated boreholes were advanced and monitoring wells installed at the locations illustrated in the attached Drawing No. 1 (Building 03) , Borehole and Monitoring Well Location Plan in Appendix A. The borings were put down uncased using continuous flight auger equipment. The drilling and sampling operations were carried out under the direction and supervision of a G2S staff member. The investigated boreholes were advanced to depths of between approximately 6.9 and 11.1 metres below the existing grade (mbeg). On completion of drilling, the boreholes were backfilled in general accordance with Ontario Regulation 903. Information from BH103 and BH104 from our previous investigation (Repot No. .G2S21366C) will be utilized to provide relevant recommendations pertinent to the design and construction of the proposed Building 06.

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

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

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

3. Subsurface Conditions

The subsurface soil conditions have been evaluated in the 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:

3.1 Topsoil/Granular

Surficial layer of topsoil with an approximate thickness of 150 mm was encountered at the surface in BH202. A surficial layer of granular fill was encountered at the top of BH201 with an approximate thickness of 50 mm.

3.2 Fill

Below the granular in BH201 a layer of fill material was encountered. This layer extended to a depth of approximately 1.5 mbeg. The fill material consisted mainly of greyish brown clayey silt, containing some sand, some gravel, and trace organics. The moisture content for the fill layer ranged between 15% and 19% indicating moist condition.

3.3 Clayey Silt

Clayey silt deposit was encountered beneath the fill in BH201 and beneath the surficial topsoil in BH202. The clayey silt extended to a depth of approximately 4.6 mbeg. The clayey silt deposit was generally grey and brown in color, containing trace to some sand, and having a reworked appearance at the top portion. With Standard Penetration Testing 'N' values ranging from 9 to 15 blows per 300 mm of penetration, this deposit was classified as stiff in consistency. The moisture content for the clayey silt was in the range of 16 to 22% indicating moist condition.

Based on the grain size analyses completed for three (3) representative samples of this deposit, the clayey silt contained from 0 to 3% gravel, 3 to 5% sand, 71 to 72% silt, and 20 to 26% clay-sized particles. The liquid limit for the clayey silt deposit ranged from 20 to 27% and the plastic limit for this deposit ranged from 14 to 20%, indicating low plasticity. The results of the grain size analysis, and the Atterberg Limits testing are indicated where applicable, on the borehole logs, grain size distribution graphs, and plasticity index chart, which are included in Appendix B.

3.4 Sandy Silt Till

Sandy silt till was encountered beneath the clayey silt in both boreholes. The sandy silt till extended to depths ranging from approximately 6.1 to 6.9 mbeg. The sandy silt till deposit was generally grey in color, containing some gravel to gravelly, trace clay, and occasional rock fragments. With Standard Penetration Testing 'N' values ranging from 20 to in excess of 50 blows per 300 mm of penetration, this deposit was classified as compact to very dense in compactness. The moisture content for the sandy silt till was in the range of 5 to 7% indicating moist condition.

3.5 Sandy Gravel Till

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

3.6 Limestone/Dolostone Bedrock

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

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

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

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

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

The Total Core Recovery (TCR) was 100% with a recorded Rock Quality Designation (RQD) for each core run ranging between 26% and 89%, indicating poor to good quality. The discontinuities observed in the rock cores were typically bedding planes with flat orientation. Vertical or dipping discontinuities were noted at depths of 7.6, 8.3, 8.4, 8.9, 9.5, and 9.9. The spacing of the discontinuities ranged from very close to moderate, and joints filling was generally classified as slightly altered, sandy and silty minor clay, or oxidation.

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

3.7 Groundwater Observations

Groundwater level observations in the open boreholes were recorded during the drilling operations. No free water and no cave-in were reported in the boreholes during the course of the investigation. Groundwater monitoring wells were installed in Boreholes BH201 and BH202. The results of our groundwater monitoring to date are presented below:

Table 2: Groundwater Observation

BH/MW ID	Full Depth of MW (mbeg)	June 20, 2024		June 26, 2024		July 19, 2024	
		Depth (m)	Relative Elevation (m)	Depth (m)	Relative Elevation (m)	Depth (m)	Relative Elevation (m)
BH/MW201	7.6	1.1	194.2	0.62	194.7	0.52	194.8
BH/MW202	6.9	0.69	194.3	0.42	194.6	0.36	194.6

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

4. Geotechnical Considerations

At the time of this report, the final grading plan for Phase 3 of the proposed development was not yet available to G2S. However, based on the updated Site Plan and the proposed development plan as outlined in Section 1 above, the lowest basement slab for the proposed mixed use building is anticipated to be at a depth of 6.0 mbeg. The proposed 3 storey retail/office building is anticipated to be constructed as a slab on grade structure with the associated surface parking.

4.1 Foundation Recommendations

4.1.1 Shallow Foundation - Mixed use Building (BH201 & BH202)

The foundation level for the proposed Mixed use building would be set at approximately 7 to 7.5 mbeg (~Elev. 188.2 – 187.7 m). As such, the proposed structure can be founded on the competent bedrock below any highly weathered or fractured zones below approximately 7.8 mbeg (~Elev. 187.7 m) and designed for bearing resistance of 2000 kPa at Ultimate Limit State (ULS). The geotechnical resistance of a sustained load at SLS should be within the normally tolerated limits of 25 millimetres of settlement. The settlement of foundations placed on competent limestone bedrock is expected to be negligible. As such, the bearing resistance SLS is not provided.

4.1.2 Shallow Foundation Retail/Office Building (BH103 & BH104)

The proposed structures can be supported on conventional spread and strip footings founded on the native silt deposit below any fill or disturbed soils. Bearing resistance ranging between 50 to 150 kPa Serviceability Limit State (SLS) and 100 to 300 kPa at Ultimate Limit State (ULS). could be utilized for the foundation design. The geotechnical resistance of a sustained load should be within the normally tolerated limits of total and differential settlement of 25 and 19mm, respectively, and a maximum footing size of 1.5 m. Should a different footing size being used, G2S should be contacted to provide an update for the bearing resistance and the associated settlement.

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

Table 2: Bearing Resistance for Conventional Spread Footing

Building Location	Borehole ID	Material	Bearing Resistance (kPa)	Recommended Founding Depth (m)	Approximate Founding Elevation (m)
Building (4) Retail/ Office Structure	BH103	Silt	50 SLS/100 ULS	1.5	192.9
			150 SLS/300 ULS	3.1	191.3
	BH104	Silt	80 SLS/160 ULS	1.5	194.2

			150 SLS/300 ULS	2.3	192.3

Based on our communication with the Client, the finished Floor Elevation (FFE) for the proposed retail/office building is expected to be at approximately 196.25 m. Given that the average existing ground surface elevation at the proposed building location is approximately 195.2 m, and the thickness of the existing fill encountered in the borehole (approximately 0.9 m), engineered fill maybe utilized for the construction of the proposed building pad. The proposed structure can be supported on conventional spread and strip footings founded on engineered fill, utilizing a design bearing resistance of 100 kPa at SLS and 150 kPa at ULS. For the anticipated settlement within the weak silt zoon and measures to mitigate the risk of excessive settlement, such as preloading, refer to Section 5.2 of the previous geotechnical report.

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

4.2 Seismic Design Parameters

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

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

4.2.1 Mixed use Building (BH201 & BH202)

Based on the subsurface soil conditions encountered in the current investigation, the applicable Site Classification for the seismic design is Site Class B – bedrock, based on the average soil characteristics for the Site. The seismic data as per the 2020 National Building Code interpolated seismic hazard values, in accordance with Article 4.1.8.23. of the NBC 2020, are as follows:

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

4.2.2 Retail/office Structure (BH103 & BH104)

Based on the subsurface soil conditions encountered in the previous investigation, the applicable Site Classification for the seismic design is Site Class D – stiff soil, based on the average soil characteristics for the Site. The seismic data as per the 2020 National Building Code interpolated seismic hazard values, in accordance with Article 4.1.8.23. of the NBC 2020, are as follows:

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

4.3 Lateral Earth Pressure and Perimeter Drainage (Mixed Use Building)

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

Table 3: Soil Properties for Design of Earth Retaining Structures

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

Φ = Angle of Internal Friction (degrees), γ = Bulk Unit Weight of Soil (kN/m^3), K_a = Active Earth Pressure Coefficient, $K_{\circ}$ = 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. Alternatively, vertical drainage system can be attached to the exterior basement walls such as Terradrain 600 or equivalent, which should cover the entire basement wall. The drainage system should outlet to a gravity storm sewer connection, fitted with a suitable back-flow prevention valve. In the event that a sump pump system is required, it should be constructed with an 'oversized' reservoir to limit pumping intervals and include an alarm in the event that the system fails to operate as per design, with a municipal water 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. The exterior grade around the structure should be sloped away from the structure to prevent the ponding of water against the foundation walls. Additional well graded granular material should be placed and compacted in exterior sidewalk and accessibility ramp areas to reduce the effects of frost heaving. Alternatively, insulation could be placed in these areas, or a structural 'frost' slab should be constructed at the doorways.

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

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

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

Where,

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

N = The normal load acting on the bedrock

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

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

4.4 Excavations and Groundwater Control (Mixed Use Building)

Based on the investigation findings, excavation for underground garages, foundations, and site services will be carried out through the fill, native sand/sand and gravel, and into the limestone bedrock. The excavation must be completed in accordance with the current OHSA regulations. For guidance, soft soils and soils below the groundwater level are classified as Type 4. The fill material, loose to compact material could be classified as Type 3 soil. The very dense sand/sand and gravel would be classified as Type 2 soil. The limestone/dolostone would be classified as Type 1 soil. If the excavation contains more than one type of soil, the soil shall be classified as the type with the highest number. Excavation slopes steeper than those required in the Safety

Act must be supported or a trench box must be provided, and a senior Geotechnical Engineer from this office should supervise the work. We note that the rate of excavation may be slowed when existing buried services and foundations, and floor slabs of the existing structures are encountered by the contractor.

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

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

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

The base of the excavations in the sand/sand and gravel material and limestone/dolostone bedrock encountered in the boreholes should remain stable. Therefore, standard pipe bedding, as typically specified by the Town of Collingwood, 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 a hydrogeological investigation should be completed for the Site to provide recommendations pertinent to the type and extent of the groundwater control for in-construction and post-construction dewatering needs. G2S would be pleased to assist in this regard.

4.5 Temporary Shoring (Mixed Use Building)

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

Caisson and solider pile toes will be made in limestone/dolostone 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 clayey silt till may be designed based on skin friction ranging between 50 kPa. Anchors extended into the sandy silt till deposit and limestone bedrock may be designed based on skin frictions ranging between 60 kPa and 1000 kPa. These values depend on the anchor installation method and grouting procedures. Gravity poured concrete can result in low bond values, while pressure-grouted anchors will give higher values and produce a more satisfactory anchor.

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

5. Closing Remarks

Should you have any questions regarding the information contained in this memorandum, please do not hesitate to contact this office.

Yours truly,

G2S Consulting Inc.

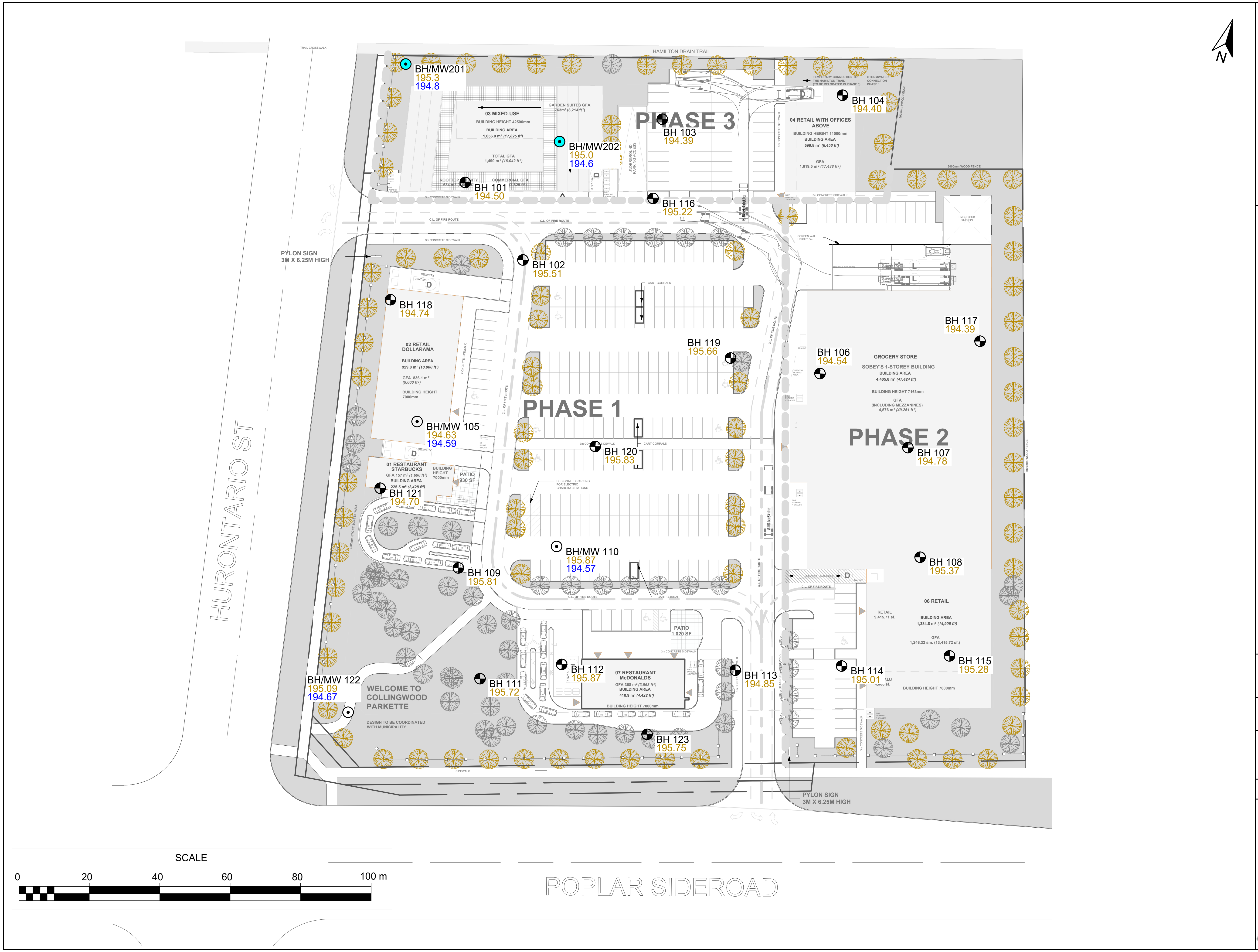
LICENSED PROFESSIONAL ENGINEER
A. ABASS
100071816
Sept 13/2024
PROVINCE OF ONTARIO

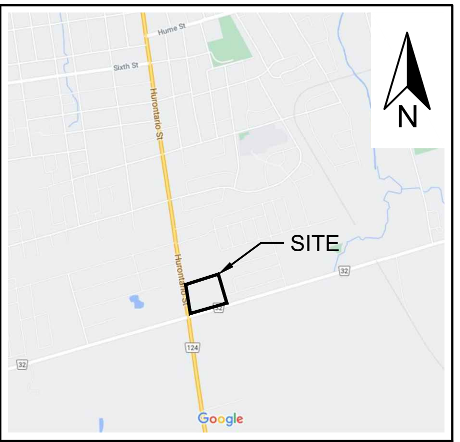
Navid Shafiee, P. Eng.
Geotechnical Engineer

Ashraf Abass, P. Eng.
Senior Geotechnical Engineer

Appendix A:

Site and Boreholes Location Plan





LEGEND

◉ MONITORING WELL / BOREHOLE LOCATION (CURRENT INVESTIGATION BY G2S COMPLETED IN JUNE 2024)

⊕ BOREHOLE LOCATION (PREVIOUS INVESTIGATION BY G2S COMPLETED IN OCTOBER 2021)

⊙ MONITORING WELL LOCATION (PREVIOUS INVESTIGATION BY G2S COMPLETED IN OCTOBER 2021)

195.0 GROUND SURFACE ELEVATION (m)

194.6 GROUNDWATER ELEVATION (JULY 19, 2024)

REFERENCE:
PLAN REPRODUCED FROM TOPOGRAPHIC SURVEY COMPLETED BY J.D.BARNES, DATED 2021-09-22, AND PLOTTED 2023-09-13

TITLE:
BOREHOLE AND MONITORING WELL LOCATION PLAN

CLIENT:
CHARIS DEVELOPMENTS LTD.

LOCATION:
853 HURONTARIO STREET
COLLINGWOOD, ONTARIO

PROJECT NO.: G2S21366

DRAWING: 1
SCALE: AS SHOWN
DATE: SEPTEMBER 2024
DRAWN BY: DB/SN
FILE NAME: G2S21366.dwg

Appendix B:

Borehole Logs & Boreholes Stratigraphy Plot

LIST OF ABBREVIATIONS

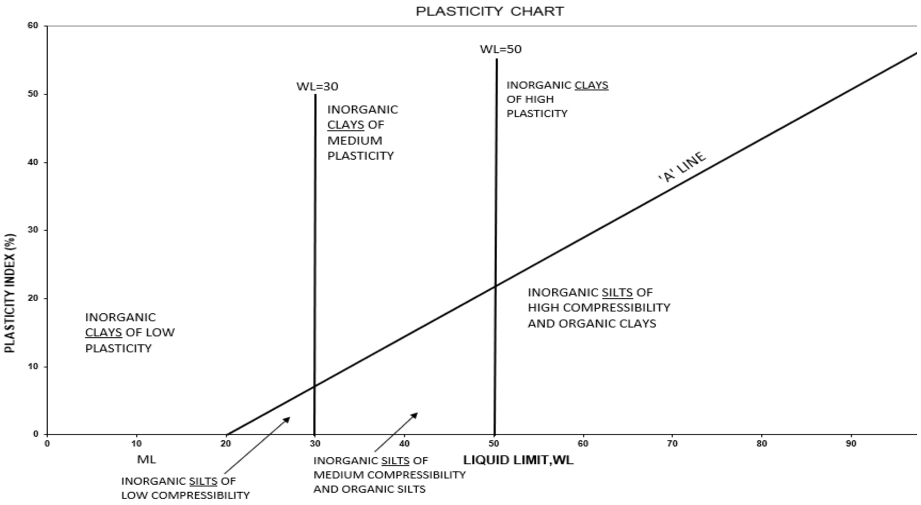
Description of Soil

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

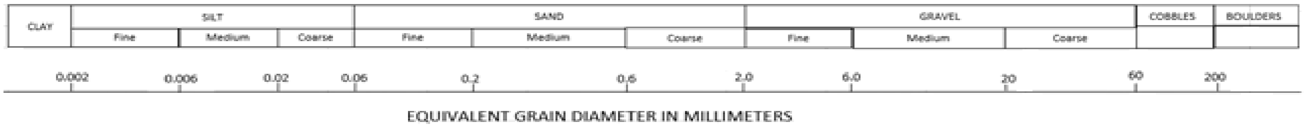
Standard Penetration Resistance N: The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m. The values reported are as noted in the field without corrections.

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



BH/MW NUMBER 201

PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Charis Developments Ltd.

PROJECT NAME Geotechnical Investigation for The Gateway Centre

PROJECT NUMBER G2S21366D

PROJECT LOCATION 853 Hurontario St, Collingwood, ON

DATE STARTED 24-6-4 **COMPLETED** 24-6-5

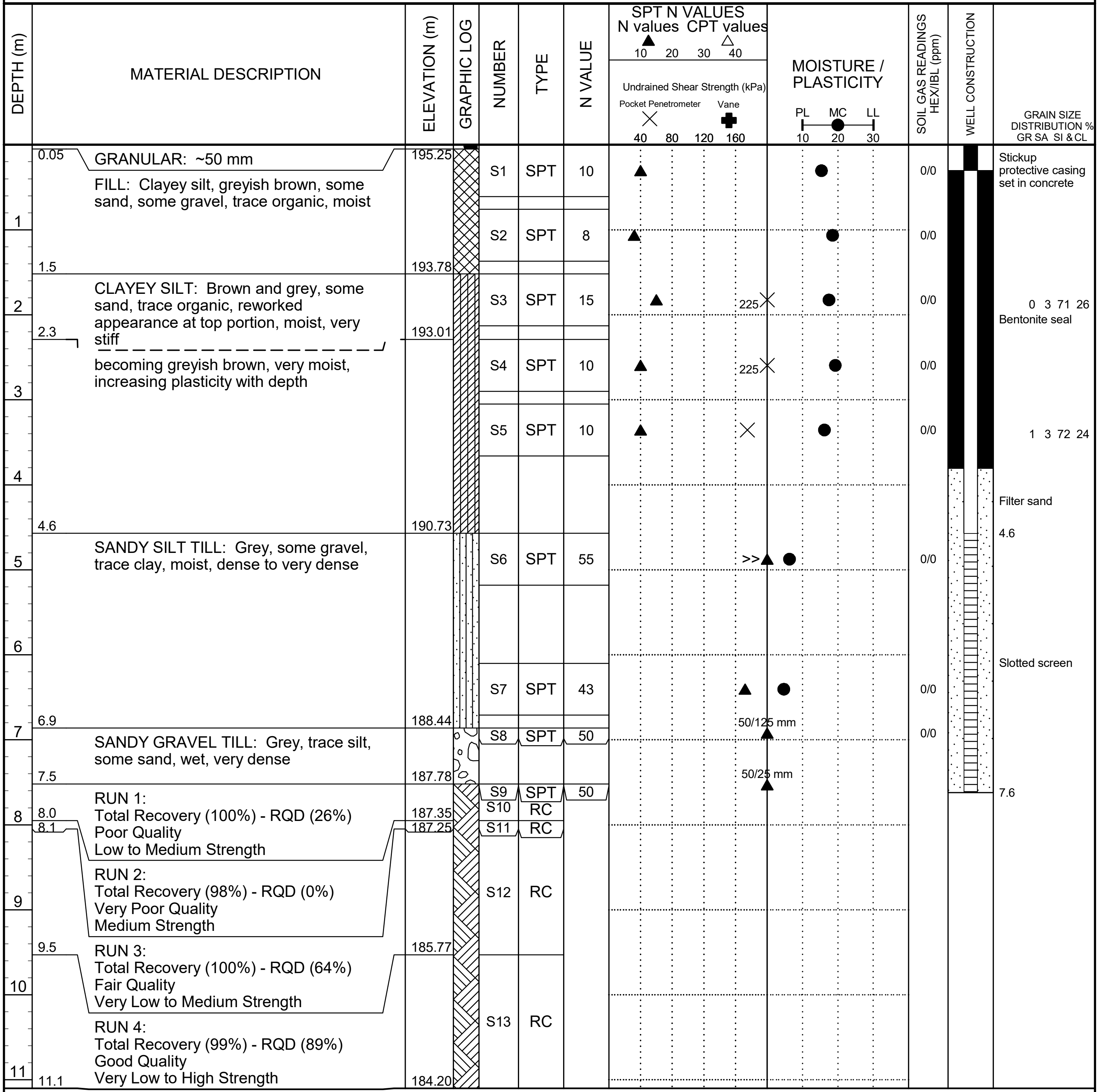
GROUND ELEVATION 195.3 m

DRILLING CONTRACTOR Davis Drilling Ltd.

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD CME 55 Track

NOTES



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

NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)
S1	SPT	10	0/0
S2	SPT	8	0/0
S3	SPT	15	0/0
S4	SPT	10	0/0
S5	SPT	10	0/0
S6	SPT	55	0/0
S7	SPT	43	0/0
S8	SPT	50	0/0
S9	SPT	50	
S10	RC		
S11	RC		
S12	RC		
S13	RC		

WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
Stickup protective casing set in concrete	
Bentonite seal	0 3 71 26
	1 3 72 24
Filter sand	
4.6	
Slotted screen	
7.6	

Borehole terminated at 11.1 m.

Water Level Readings:

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

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BH/MW NUMBER 202

PAGE 1 OF 1

CLIENT Charis Developments Ltd.

PROJECT NAME Geotechnical Investigation for The Gateway Centre

PROJECT NUMBER G2S21366D

PROJECT LOCATION 853 Hurontario St, Collingwood, ON

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

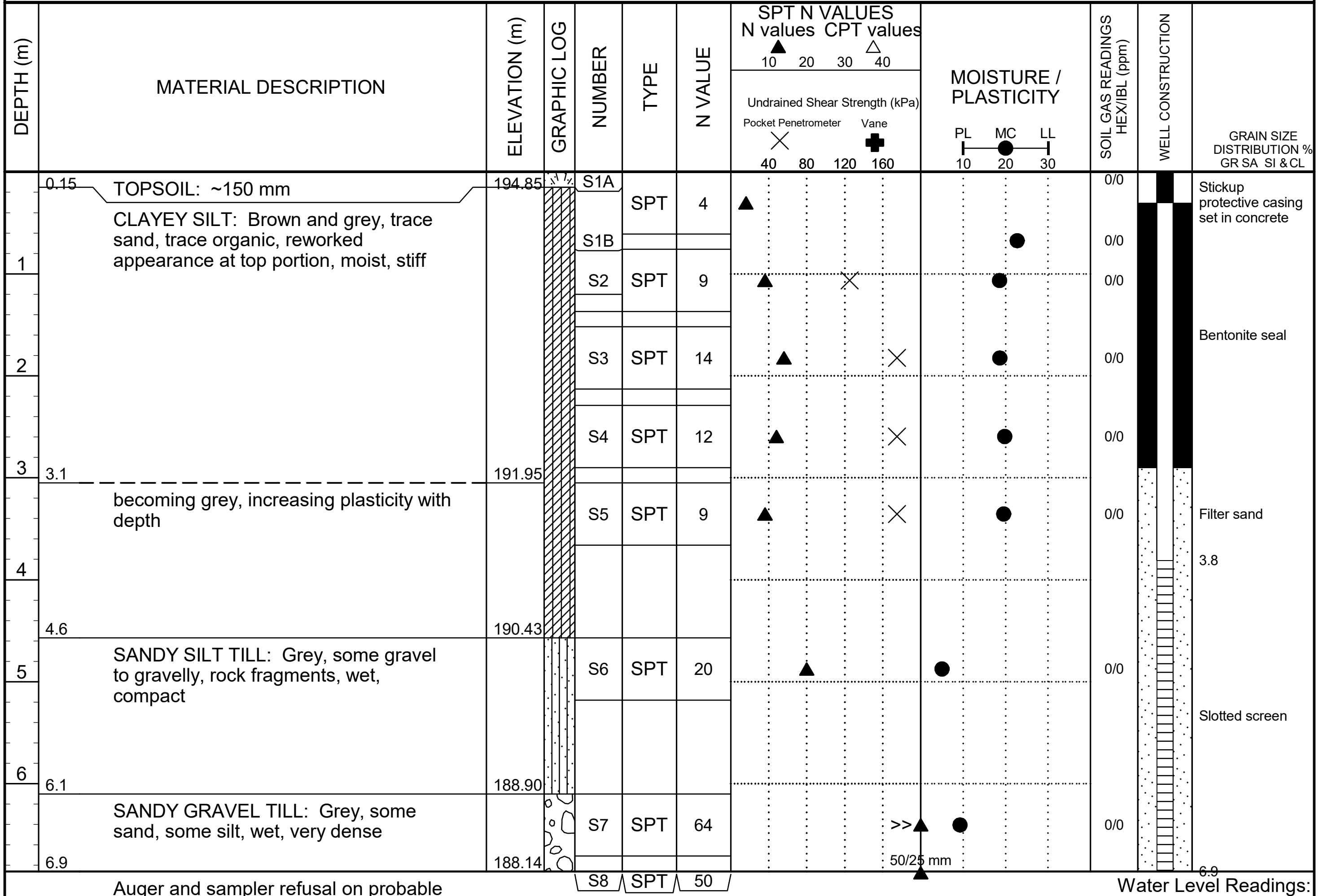
GROUND ELEVATION 195.0 m

DRILLING CONTRACTOR Davis Drilling Ltd.

LOGGED BY DB **CHECKED BY** AA

DRILLING METHOD CME 55 Track

NOTES



Depth (m)	MATERIAL DESCRIPTION	ELEVATION (m)
0.15	TOPSOIL: ~150 mm	194.85
	CLAYEY SILT: Brown and grey, trace sand, trace organic, reworked appearance at top portion, moist, stiff	
3.1	becoming grey, increasing plasticity with depth	191.95
4.6	SANDY SILT TILL: Grey, some gravel to gravelly, rock fragments, wet, compact	190.43
6.1	SANDY GRAVEL TILL: Grey, some sand, some silt, wet, very dense	188.90
6.9		188.14

NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)
S1A	SPT	4	0/0
S1B			0/0
S2	SPT	9	0/0
S3	SPT	14	0/0
S4	SPT	12	0/0
S5	SPT	9	0/0
S6	SPT	20	0/0
S7	SPT	64	0/0
S8	SPT	50	

Auger and sampler refusal on probable bedrock
Borehole terminated at 6.9 m.

Water Level Readings:

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

BOREHOLE NUMBER 103

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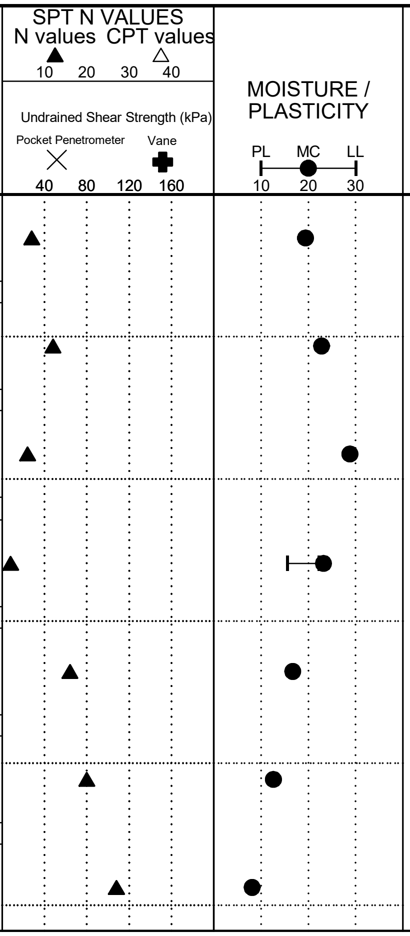
CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-22 **COMPLETED** 21-10-22
GROUND ELEVATION 194.39 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich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

DEPTH (m)	MATERIAL DESCRIPTION	ELEVATION (m)	GRAPHIC LOG	NUMBER	TYPE	N VALUE	SOIL GAS READINGS HEX/IBL (ppm)	WELL CONSTRUCTION	GRAIN SIZE DISTRIBUTION % GR SA SI & CL
0.13	TOPSOIL: ~125 mm	194.27							
	FILL: Clayey silt, brown to grey, some sand, moist			SS1	SPT	7			
1 1.00		193.39							
	CLAYEY SILT: Brown, trace sand, rootlets, moist, stiff, reworked appearance at the upper section			SS2	SPT	12			
1.5		192.89							
2	SILT: Grey, layered, trace sand, trace gravel, some clay, very moist to wet, very loose to compact			SS3	SPT	6			
				SS4	SPT	2			2 6 68 24
3				SS5	SPT	16			
4 4.0		190.39							
	SANDY SILT TILL: Grey, some gravel, moist, compact			SS6	SPT	20			
5				SS7	SPT	27			
5.2		189.21							



Borehole terminated at 5.2 m.

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

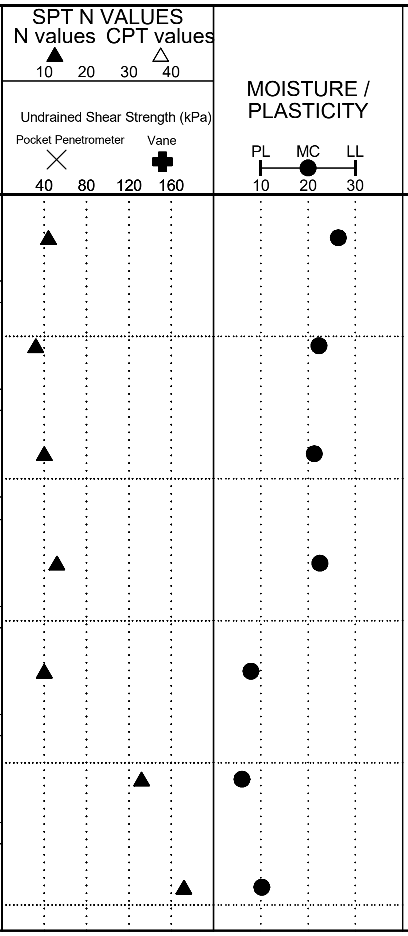
BOREHOLE NUMBER 104

PAGE 1 OF 1

G2S Consulting Inc.

CLIENT Charis Developments Ltd.
PROJECT NAME 839 & 869 Hurontario St & 7564 Poplar Side Rd
PROJECT NUMBER G2S21366B
PROJECT LOCATION Collingwood, Ontario
DATE STARTED 21-10-22 **COMPLETED** 21-10-22
GROUND ELEVATION 194.40 m
DRILLING CONTRACTOR LST
LOGGED BY DB **CHECKED BY** AA
DRILLING METHOD Diedrich D50 Track
NOTES

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



Borehole terminated at 5.2 m.

Upon completion of augering
No cave
Free water at 4.4 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

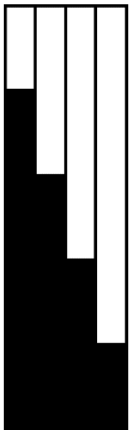
RU = Rough Undulating
RP = Routh Planar
SU = Smooth Undulating
SP = Smooth Planar
LU = Slickensided Undulating
LP = Slickensided Planar

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

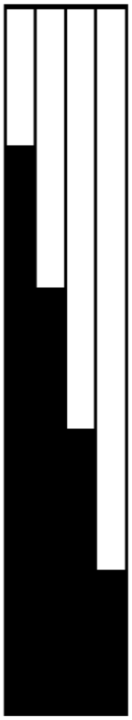
Slightly weathered = partial discoloration: fractures (joints) typically oxidized

Moderately weathered = total discoloration

Highly weathered = total discoloration: typically, friable & pitted

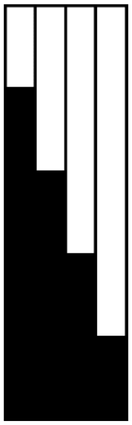
Completely weathered = resembles soil: rock structure usually preserved

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

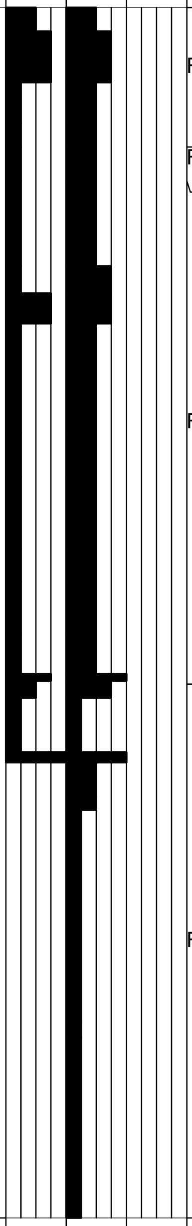
19 Water Colour: The colour of the water returning out the casing

ROCK CORE LOG

BH NO. 201

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

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



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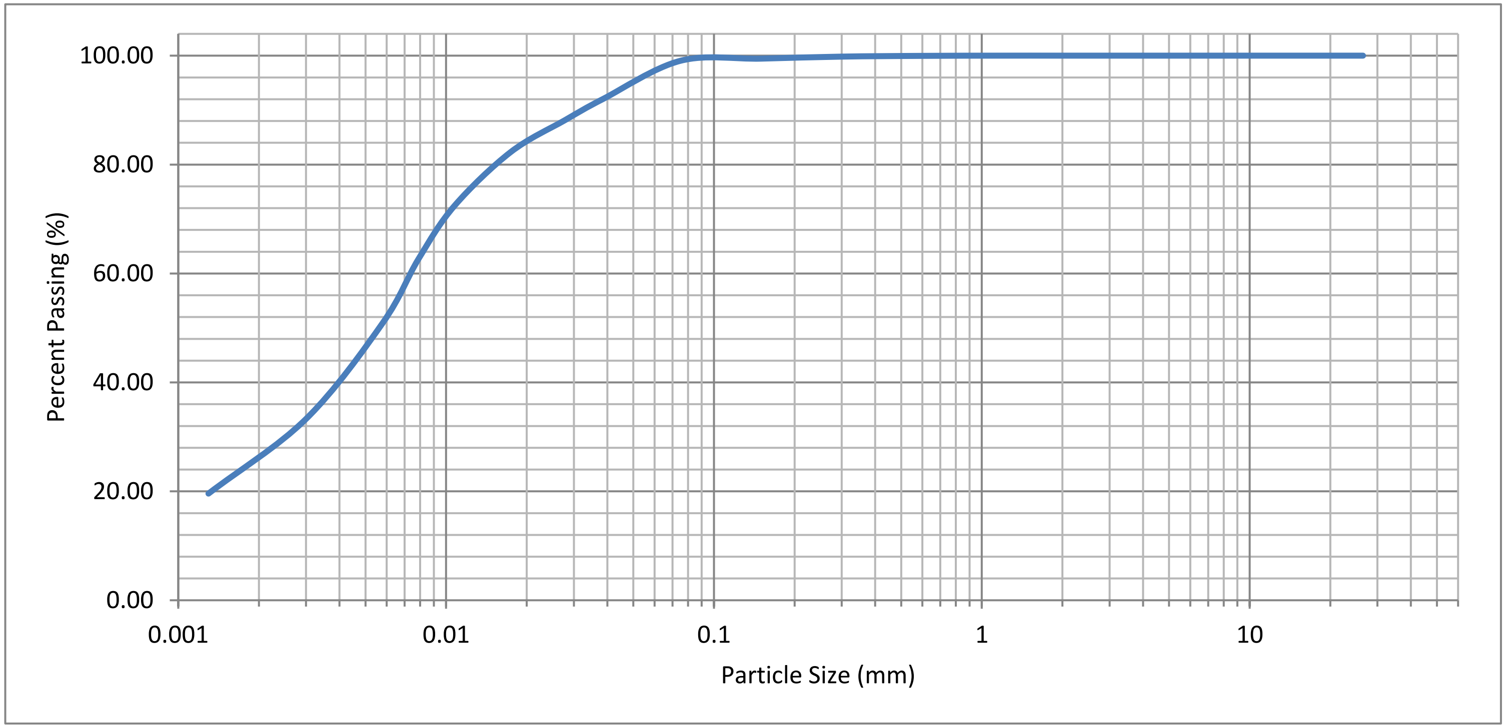
Project No.:	G2S21366D	**Lab No.:**	24132A
Project Name:	839, 853, 869 Hurontario St. & 7564 Poplar Sideroad, Collingwood	**Borehole/Sample No.:**	BH201-S3

ISSMGE SOIL CLASSIFICATION

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

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

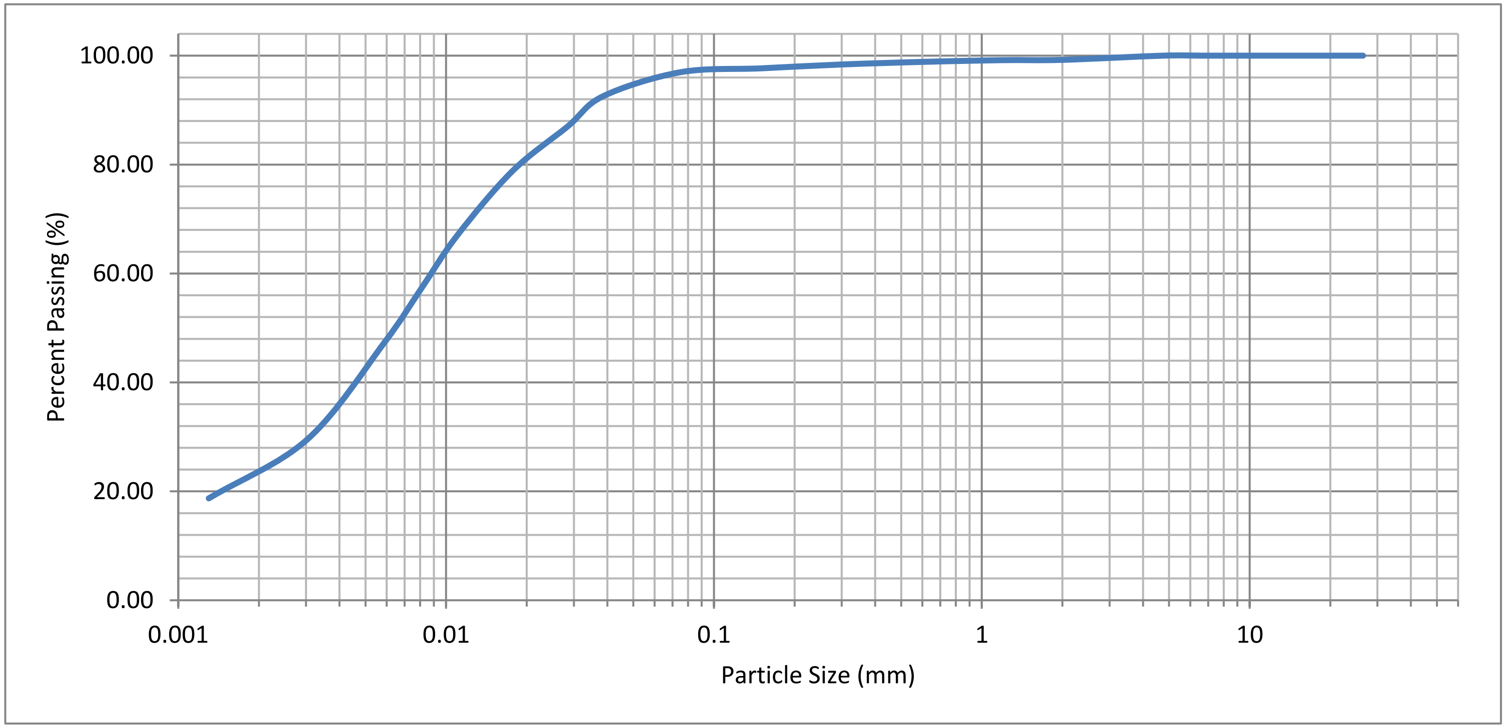


Project No.:	G2S21366D	**Lab No.:**	24132B
Project Name:	839, 853, 869 Hurontario St. & 7564 Poplar Sideroad, Collingwood	**Borehole/Sample No.:**	BH201-S5

ISSMGE SOIL CLASSIFICATION

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

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



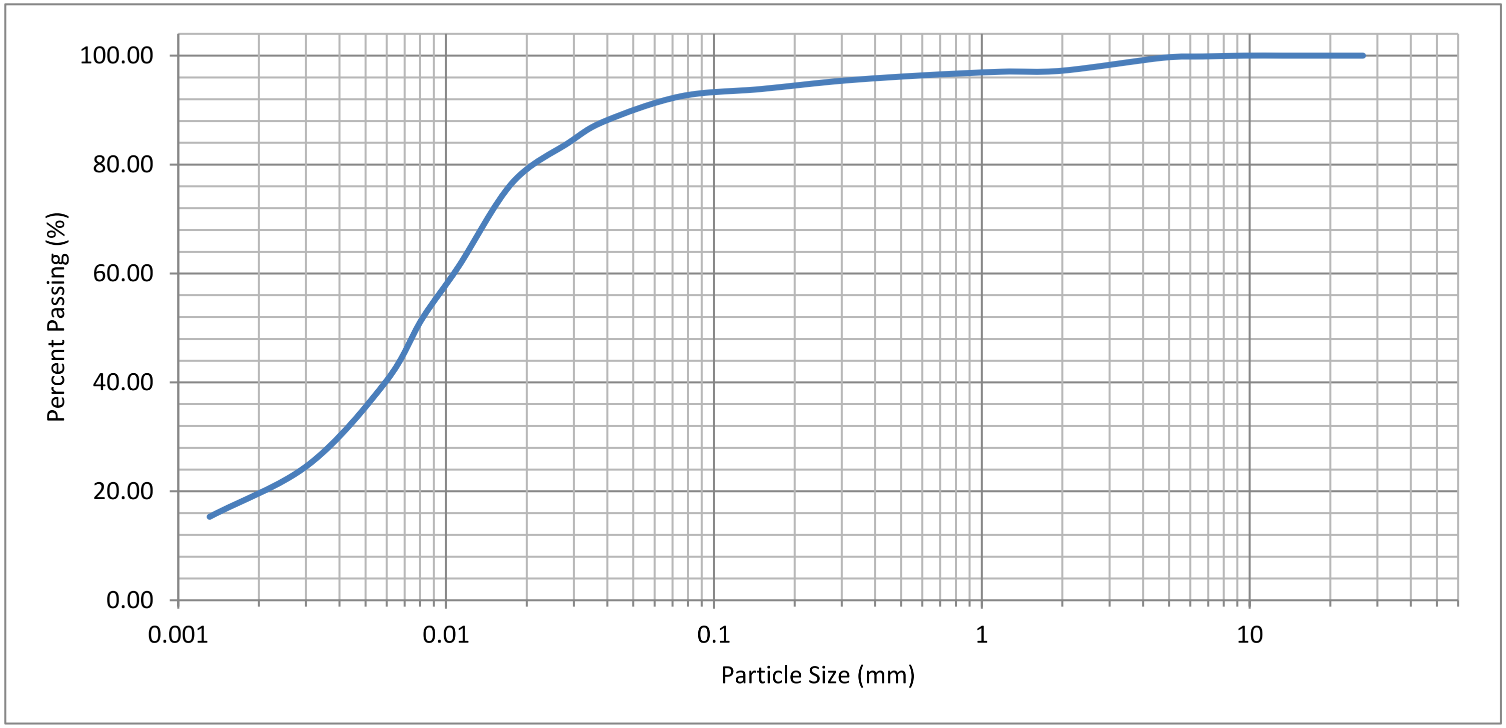
G2S Consulting Inc.
4361 Harvester Rd., Unit 12, Burlington, ON L7L 5M4
P 905.331.3735 F 905.642.5999
G2Senvironmental.com

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

ISSMGE SOIL CLASSIFICATION

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

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



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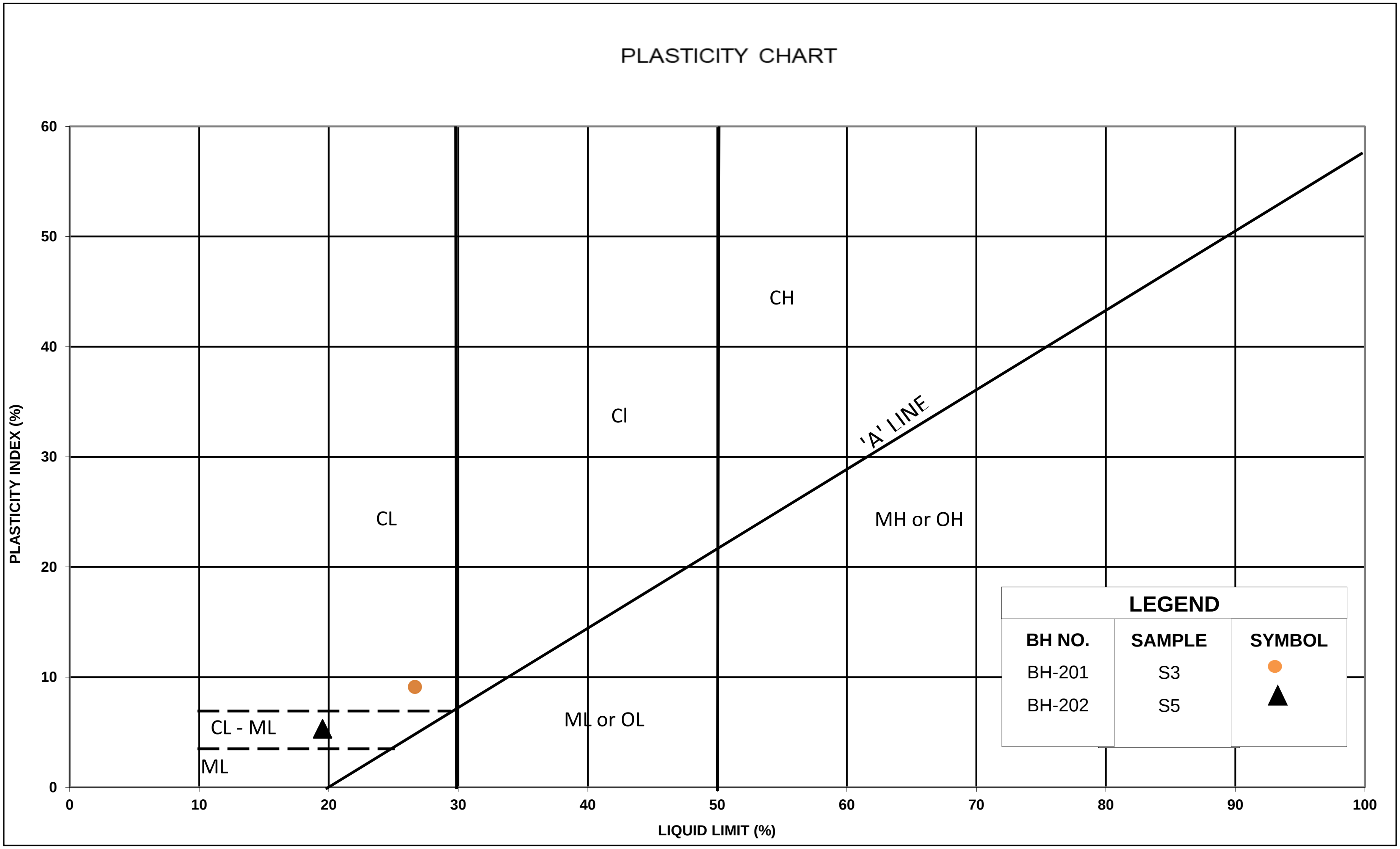
G2Sconsulting.com

Project No.: G2S21366D
Lab No.: 24132A&C
Project Name 839, 853, 869 Hurontario St. & 7564 Poplar Sideroad
Borehole/Sample No.: BH201-S3/BH202-S5



Appendix C:

UCS Test Results & Retrieved Rock Core Samples Photographs

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

Rock Core Compressive Strength Test Report

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

Project No.: G2S21366D
Lab No.: 24169

Compressive Strength Test Results

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

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

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

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

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

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

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

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

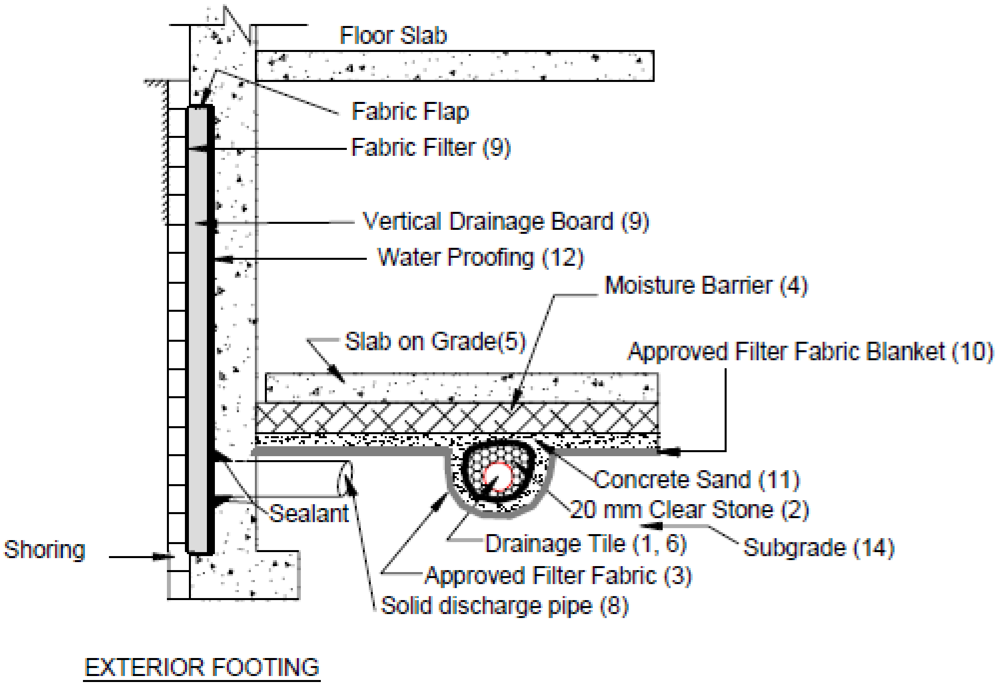
Photo No. 7: BH201 – Box 2 (Photo 1 of 3) – Run No. 4

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

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

Appendix D:

Drainage and Underpinning Recommendation Figures

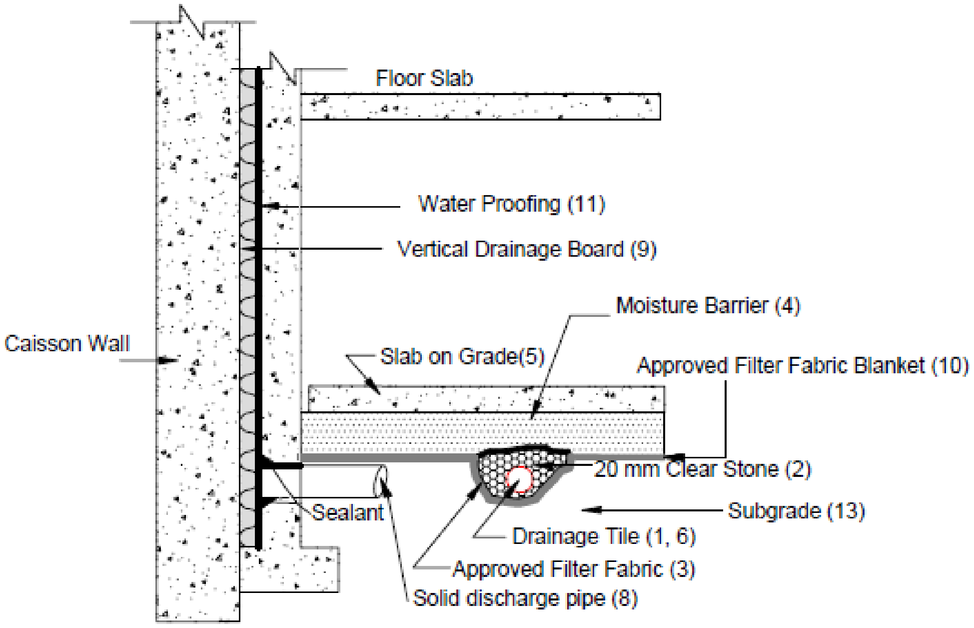


Notes

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

DRAINAGE RECOMMENDATIONS
Shored Basement Wall with Underfloor Drainage System

(NOT TO SCALE)



EXTERIOR FOOTING

Notes

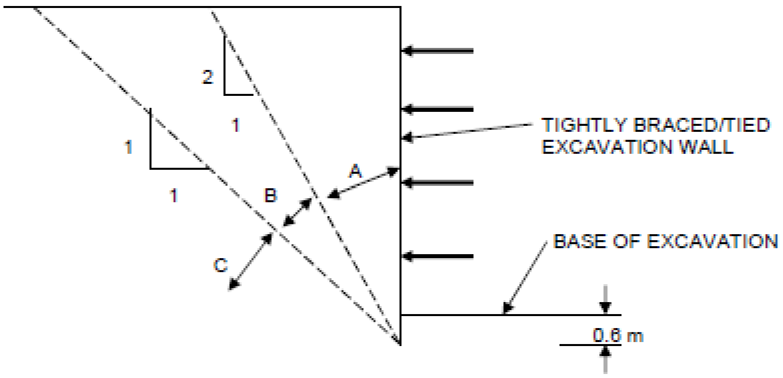
1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain.
3. Wrap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

DRAINAGE RECOMMENDATIONS
Shored Basement Wall with Underfloor Drainage System

(NOT TO SCALE)

Guidelines for Underpinning in Soil and Excavation Support

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



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

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

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