Preliminary Geotechnical Investigation

12612 and 12586 Woodbine Avenue, Stouffville, Ontario

Kingsdale Development

95 Royal Crest Ct, Unit 21-22, Markham, ON L3R 9X5

Prepared by:

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SLR Project No.: 209.065495.00001

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Revision: A

Making Sustainability Happen

Revision Record

Revision	Date	Prepared By	Checked By	Authorized By
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July 9, 2026

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Re: **Preliminary Geotechnical Investigation – 12612 and 12586 Woodbine Avenue, Stouffville, Ontario**
Project #: **243.024905.00001**

SLR is pleased to submit the attached report describing the results of our preliminary geotechnical investigation for the project at the subject site ("the Site") located at 12612 and 12586 Woodbine Avenue, in the Town of Stouffville, Ontario.

The report provides site information from our site investigation, laboratory testing, records reviews, and our interpretations/recommendations for your consideration.

Thank you for the opportunity to be of service on this project. We trust that this report will be satisfactory for your current needs. If you have any questions or require further information, please contact our office at your convenience. This report is subject to the Statement of Limitations provided at the end of this report.

Yours truly,

SLR

Steven Lu

Steven Lu, P.Eng.
Geotechnical Engineer

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1.0 Introduction

SLR was retained by Kingsdale Development (the “Client”) to undertake a Preliminary Geotechnical Investigation in support of a development located at 12612 and 12586 Woodbine Avenue, in the Town of Stouffville, Ontario.

It is understood that the proposed development includes a mausoleum, banquet hall, two crossings of Berczy Creek, reception area and crematorium, parking lot and underground parking areas. The objective of this investigation was to determine the subsurface conditions in the area of the proposed development by means of six (6) exploratory boreholes and to make engineering recommendations for the project based on the findings from the boreholes.

Borehole BH26-1 was drilled and advanced within the footprint of mausoleum with eight floors and two underground parking. Borehole BH26-2 was drilled and advanced within the banquet hall with three (3) floors. Borehole BH26-3 and BH26-5 were drilled and advanced within the footprint of the warehouse and reception and crematorium with three (3) floors and one (1) underground parking. Boreholes BH26-4 and BH26-6 were drilled and advanced within the two crossings of Berczy Creek on the east side of crematorium.

The report is provided on the basis of the terms of reference presented above, and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the changes. It may then be necessary to carry out additional borings and reporting before the recommendations of this report can be relied upon.

The site investigation and recommendations follow generally accepted practices for geotechnical consultants in Ontario. The format and contents are guided by client-specific needs and economics and do not conform to generalized standards for services. Laboratory testing for the most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report deals with geotechnical issues only. The hydrogeological assessment and Fluvial Geomorphic Studies are provided in a separate SLR report.

This report has been prepared for the Client and their designers. Use of this report by a third party without SLR’s consent is prohibited. The limitations of the report presented within form an integral part of the document and they must be considered in conjunction with this report.

2.0 Site and Regional Geology

The site is located at the civic address of 12612 and 12586 Woodbine Avenue, in the Town of Stouffville, Ontario. The project area is situated within the South Slope Physiographic region of Southern Ontario (Chapman and Putman, 1984). The topography for this region typically consists of drumlinized Till Plains.

A review of available Ontario surficial geology mapping indicated that the overburden materials in the area of the site is comprised of clay to silt-textured till derived from glaciolacustrine deposits or shale. Bedrock geology mapping indicated that the site is underlain by bedrock comprised shale, limestone, dolostone, and siltstone of Georgian Bay Formation (Ontario Geological Survey, 2011).

3.0 Field and Laboratory Work

The field work for the preliminary geotechnical investigation was carried out on May 21 and 22, 2026, by drilling specialists subcontracted to SLR, during which time six (6) boreholes (BH26-1 to BH26-6) were advanced to depths ranging from about 6.7 to 10.9 m below the existing ground surface (Elevation 266.3 to 259.5 m). All boreholes were drilled within the footprints of the proposed site development. The locations of six (6) boreholes are shown on the Borehole/Monitoring Well Location Plan **Drawing 1**.

The boreholes were advanced with a power auger drilling machine, where soil stratigraphy was recorded by observing the quality and changes of augured materials which were retrieved from the boreholes, and by sampling the soils at regular intervals of depth using a 50 mm O.D. split spoon sampler, in accordance with the Standard Penetration Test (SPT) method (ASTM D 1586). This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 300 mm depth into the soil (SPT ‘N’ values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT ‘N’ values are indicated on the borehole logs (Refer to **Appendix A**). In addition to SPT testing, dynamic cone penetration tests (ASTM D6951) and pocket penetrometer tests were also conducted on the soil samples during the field work. The fieldwork for this investigation was supervised by SLR engineering staff, who also logged the boreholes and cared for the recovered samples.

Four (4) monitoring wells were installed in Boreholes BH26-1 to BH26-4. The stabilized groundwater levels were measured on May 27 and June 4, 2026. The monitoring well installation details and the measured groundwater levels are summarized in **Table 1** and shown on the individual borehole logs.

All soil samples obtained during this investigation were brought to our laboratory for further examination. These soil samples will be stored for a period of three (3) months after the day of issuing the draft report, after which time they will be discarded unless SLR is advised otherwise in writing. In addition to visual examination in the laboratory, all soil samples from geotechnical boreholes were tested for moisture content. Grain size analyses of four (4) selected soil samples and Atterberg limits tests of four (4) selected soil samples were conducted. The results are presented in the borehole logs and in **Appendix B**.

The approximate elevations at the as-drilled borehole locations were surveyed using a differential GPS unit. The elevations at the as-drilled borehole locations were not provided by a professional surveyor and should be considered as approximate. Contractors performing the work should confirm the elevations prior to construction. The borehole locations plotted in **Drawing 1** should be considered as approximate.

4.0 Subsurface Conditions

The borehole locations are shown in **Drawing 1**. General notes on soil description are presented on the "Explanation of Terms Used in the Record of Borehole" sheet in **Appendix A**. The subsurface conditions in the boreholes are presented in the individual borehole logs (**Enclosures 1** to **6** inclusive, **Appendix A**). The subsurface conditions in the boreholes are summarized in the following paragraphs.

4.1 Soil Conditions

Topsoil

A 50 to 150 mm thick layer of surficial topsoil was encountered at the surface of all boreholes. It should be noted that the thickness of the topsoil contacted at the borehole locations may not be representative of the site and should not be relied on to calculate the amount of topsoil at the site.

Disturbed Native Soils and Fill Materials

A thin veneer of disturbed native fill soils (herein referred to as "fill") or Fill materials consisting of clayey silt / silty clay textures were encountered below topsoil in all boreholes and extended to depths ranging from about 1.2 to 1.7 m below the existing ground surface (Elev. 271.8 to 264.5). For cohesive disturbed silty clay / clayey silt and fill materials, the standard penetration SPT 'N' value ranging from 4 to 26 blows per 300 mm penetration indicated a firm to very stiff consistency. The in-situ moisture contents measured in the disturbed native and fill samples ranged from approximately 11% to 23%.

Silty Clay Till / Clayey Silt Till

Silty clay till / clayey silt till deposits were encountered beneath the disturbed native and fill materials in all boreholes, and extended to depths ranging from 6.7 to 10.9 m below the existing ground surface (Elev. 266.3 to 259.5 m). All boreholes were terminated in these deposits. the standard penetration SPT ‘N’ value ranging from 5 to greater than 50 blows per 300 mm penetration indicated a firm to hard consistency. The natural moisture contents measured in the soil samples ranged from approximately 9% to 20%.

Grain size analyses were conducted on four (4) samples (BH26-1/SS8, BH26-2/SS6, BH26-3/SS7, and BH26-4/SS6) from the clayey silt till / silty clay till deposits. The results are presented on individual borehole logs and in **Appendix B**, with the following fractions:

Gravel:	3 to 8%
Sand:	29 to 34%
Silt:	41 to 46%
Clay:	18 to 24%

Consistency (Atterberg) limits tests on four (4) samples (BH26-1/SS8, BH26-2/SS6, BH26-3/SS7, and BH26-4/SS6) of the fines content of the soil matrix component of the samples indicate liquid limits ranging from 17 to 22, plastic limits ranging from 11 to 13, and plasticity induces ranging from 6 to 9 (see **Appendix B**). According to the modified Unified Soil Classification System, BH26-1/SS8, BH26-2/SS6, BH26-3/SS7 were classified inorganic low plasticity clays and low compressibility silts (CL-ML), and BH26-4/SS6 was classified as inorganic clays of medium plasticity (CL).

4.2 Groundwater Conditions

Four (4) monitoring wells (50 mm dia.) were installed to monitor stabilized groundwater levels. The stabilized groundwater levels were measured on May 27 and June 4, 2026. The monitoring well installation details and the measured groundwater levels are summarized in **Table 1** and shown in the individual borehole logs.

Table 1: Monitoring Well Details and Water Levels

Monitoring Well ID	Screen Interval (mbgs)	Groundwater Level Depth (mbgs) / Groundwater Elevation (masl)	
		May 27, 2026	June 4, 2026
BH26-1	7.6 ~ 10.7	-0.7 / 271.5	-0.5 / 271.3
BH26-2	4.6 ~ 6.1	1.9 / 271.1	2.1 / 270.9
BH26-3	4.6 ~ 6.1	1.1 / 267.2	0.3 / 268.0
BH26-4	4.6 ~ 6.1	4.8 / 264.3	1.0 / 268.1

Notes: mbgs = meter below ground surface
masl = metres above sea level

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations. In addition, groundwater levels may respond to individual weather events and higher groundwater levels could have occurred between monitoring events.

5.0 Geotechnical Parameters

Based on the results of the field investigation and experience with similar soils, estimated geotechnical soil parameters are provided in **Table 2**.

Table 2: Estimated Geotechnical Soil Parameters

Soil Type	New Granular Fill		Existing Fill	Cohesive Native Soil – Clayey Silt (Till) / Silty Clay (Till)				
SPT 'N'	**'A'**	**'B'**	**4-50**	**3-9**	**10-14**	**15-29**	**30-50**	**>50**
Unit weight (kN/m^3)	22	21	19	19	20.5	21	21.5	22.5
Effective angle of internal friction (°), ϕ'	35	32	24	26	28	30	32	34
Effective cohesion, c' (kPa)	-	-	-	0	2	5	10	10
Undrained shear strength (kPa)	-	-	-	40	70	100	200	300

Soil Type	**New Granular Fill**		**Existing Fill**	**Cohesive Native Soil – Clayey Silt (Till) / Silty Clay (Till)**				
SPT ‘N’	**‘A’**	**‘B’**	**4-50**	**3-9**	**10-14**	**15-29**	**30-50**	**>50**
Coefficient of lateral earth pressure								
Active, K_a	0.27	0.31	0.42	0.39	0.36	0.33	0.31	0.28
At rest, K_o	0.43	0.47	0.59	0.56	0.53	0.50	0.60	1.00
Passive, K_p	3.69	3.25	2.37	2.56	2.77	3.00	3.25	3.54
Elastic modulus (MPa)	-	-	-	4	8	15	30	50
Poisson’s ratio	-	-	-	0.3	0.3	0.3	0.3	0.3
Modulus of subgrade reaction, k (MN/m^3) (*)	-	-	-	4/B	8/B	15/B	30/B	50/B
Lateral modulus of subgrade reaction, K_h (MN/m^3) (*)	-	-	-	4/B	8/B	15/B	30/B	50/B

Note: * B is the width of footing or width of wall/pile in metres.

6.0 Discussion and Recommendations

It is understood that the proposed development includes a mausoleum, banquet hall, two crossings of Berczy Creek, reception area and crematorium, parking lot, and underground parking areas. Borehole BH26-1 was drilled and advanced within the footprint of mausoleum with eight floors and two underground parking. Borehole BH26-2 was drilled and advanced within the banquet hall with three (3) floors. Borehole BH26-3 and BH26-5 were drilled and advanced within the footprint of the warehouse and reception and crematorium with three (3) floors and one (1) underground parking. Boreholes BH26-4 and BH26-6 were drilled and advanced within the two crossings of Berczy Creek on the east side of crematorium.

6.1 Foundation Design Considerations

6.1.1 Shallow Foundations

6.1.1.1 Spread and Strip Foundations

Based on the borehole information, the proposed mausoleum, the low-rise banquet hall, warehouse, two bridge crossings of Berczy Creek, and reception and crematorium can be supported by spread and strip footings founded on the undisturbed native soils for a bearing capacity of 150 to 300 kPa at SLS (Serviceability limit states), and for a factored geotechnical resistance of 225 to 450 kPa at ULS (Ultimate limit states). The bearing values and the corresponding founding depths at borehole locations are summarized on **Table 3**.

Table 3. Bearing Values and Founding Levels of Spread and Strip Footings

Borehole No.	BH Location	Anticipated Founding Soil	Bearing Capacity at SLS (kPa)	Factored Geotechnical Resistance at ULS (kPa)	Anticipated Founding Depth (mBGS)/ Elevation (m)
BH26-1	Mausoleum	Silty Clay Till	150 100	225 150	2.4 / 268.4 6.1 / 264.7
BH26-2	Banquet Hall	Silty Clay Till	200	300	1.6 / 271.4
BH26-3	Warehouse and reception& crematorium	Silty Clay Till	200	300	1.6 / 266.7
BH26-4	Bridge / Culvert	Silty Clay Till	150 100	225 150	1.8 / 267.3 4.5 / 264.6
BH26-5	Warehouse and Reception & Crematorium	Silty Clay Till	200 150	300 225	2.0 / 265.4 4.5 / 262.9
BH26-6	Bridge / Culvert	Silty Clay Till	150 80	225 120	2.0 / 264.2 4.2 / 262.0

Notes: mbgs = meter below ground surface

(*) B is the width of footing or width of wall/pile in metres

All footing bases must be inspected by qualified geotechnical engineering personnel prior to pouring concrete. The excavated footing bases can be covered with 50 mm thick lean concrete slab immediately after inspection and cleaning in order to avoid disturbance of the founding soil due to water, construction activity and weathering/drying.

Foundations designed to the specified bearing capacity at the serviceability limit states (SLS) are expected to settle less than 25 mm total and 19 mm differential, if designed as per **Table 3**.

All foundations exposed to seasonal freezing conditions must have at least 1.6 metres of soil cover for frost protection.

In the vicinity of the existing buried utilities, all footings must be lowered to undisturbed native soils, or alternatively the services must be structurally bridged. Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

It should be noted that the recommended bearing resistances have been calculated by SLR from the borehole information for the preliminary design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections to validate the information for use during the construction stage.

6.1.1.2 Engineered Fill

Engineered fill may need to be placed to raise the grade to the design elevation. Prior to the placement of the engineered fill, all of the existing fill and weak native soils and disturbed materials must be removed, and the exposed surface proof rolled. Any soft spots revealed during proof rolling must be sub-excavated and re-engineered. The engineered fill consisting of approved inorganic material must be compacted to 100% Standard Proctor Maximum Dry Density throughout. To reduce the risk of improperly placed engineered compacted fill, full-time supervision of the contractor is essential. Despite full-time supervision, it has been found that contractors frequently bulldoze loose fill into areas and compact only the surface. This potential problem must be recognized and discussed at a pre-construction meeting. Procedures can then be instigated to reduce the risk of settlement resulting from un-compacted fill. The detailed requirements for engineered fill are given in **Appendix C**.

Foundations designed to the specified bearing resistance values at the serviceability limits states (SLS) are expected to settle less than 25 mm total and 19 mm differential.

6.1.2 Deep Foundations

Where structural loading conditions exceed the values discussed above, or where unsuitable soils are encountered during construction, alternative foundation solutions such as deep foundation systems should be considered. All foundation design and construction should be carried out in accordance with the recommendations provided herein and verified through field inspection.

It is noted that the subsurface conditions interpreted herein are based on a limited number of boreholes and may not fully represent site-wide variability. As such, additional subsurface investigation and geotechnical assessment would be required to properly evaluate and design deep foundation options. Deep foundation systems, including helical piles and Oliver Displacement piles, are presented herein for preliminary discussion purposes only.

6.1.2.1 Helical Piles

The eight-storey mausoleum may be supported by helical piles founded in the firm to very stiff native soils, providing the required pile torque rating is capable of installing piles into the deep founding levels. Possible large obstructions such as buried concrete slabs or existing foundations within the fill materials or boulders within the tills may be encountered during the installation of the helical piles and may prevent the piles from reaching proposed founding depths.

Helical piles are proprietary products designed, supplied and installed by specialist contractors. Bearing capacity and other design details regarding helical piles can be discussed with the specialized contractor. Geotechnical comments concerning installation and design capacities are provided in the following paragraphs. It must be noted that they are considered to be preliminary values suitable for preliminary design only. The actual design and installation of helical piles should be undertaken by contractors in Ontario that are approved by the manufacturer.

The designer should define the depth and type of helical piles according to the soil conditions and the required design loads. The designer should also consider the buckling resistance in weak soils and their lateral capacity. The contractor should also be responsible for the design capacity of the foundation units. In this regard, it is recommended that compression and tension tests be conducted to verify helical pile capacities prior to final design.

Table 4 below shows the capacities of various Chance helical piles that may be appropriate for this site. These values are taken from EBS Geostructural (specialty geotechnical contractor) and have been confirmed through field capacity testing.

Table 4. Chance Helical Pile Factored Geotechnical Resistances

"N" value Cohesive Soil	Pile Type (size)	Compression Capacity		Tension Capacity	
		SLS (kN)	ULS (kN)	SLS (kN)	ULS (kN)
25-35	SS5 (38 mm)	200	270	60	80
35-45	SS175 (44 mm)	370	500	115	150
50-60	SS200 (51 mm)	500	670	205	270

Detailed design services are available through a number of specialty helical pile contractors, and shop drawings should be provided to SLR for review. In general, helical piles should be advanced into the competent native soils at a minimum of 5.0 m below existing ground surface until the design torque is achieved.

The provided capacities are preliminary and must be confirmed with a specialist helical pile contractor and verified with field load tests on surficial piles per structure in accordance to ASTM D1143.

It should be noted that there is a possibility that the firm to hard stratum may not extend uniformly throughout the areas and the need for deeper helical piles in some areas must not be overlooked.

If site grades were to be raised, consideration should be given to the effects of negative skin friction on the helical piles.

It is recommended that SLR be retained to monitor and document helical pile installation to verify that the recommended capacity is achieved.

6.1.2.2 Oliver Displacement Piles

Oliver Displacement Piles are a deep foundation type that is a proprietary product, supplied and installed by specialty contractors for the mausoleum building. Installation of this pile type results in a cast in place concrete rigid inclusion. This pile is installed by first augering down to a specified suitable depth with a hallow auger, the installation of a rebar cage structure for reinforcement, and then pouring of structural concrete with the simultaneous removal of the auger to fill the resulting cavity. The top of the pile is then finished with either a tie into a grade-beam or pile cap as specified by the foundation design. These piles can also be excavated and cropped.

This pile type is ideal for sites where the soil required for similar capacities for traditional or helical piles are found below 20 m, the site has a high groundwater table and there are limited obstructions found in the soils. Capacities for this pile type range from 500 to 3,000 kN, final capacities and other design details can be discussed with the specialized contractor. Geotechnical comments concerning installation and design capacities are provided in the following paragraphs. It must be noted that they are considered to be preliminary values suitable for preliminary design only. The actual design and installation of helical piles should be undertaken by contractors in Ontario that are approved by the manufacturer.

Oliver Displacement Pile capacities are based upon both the shaft resistance and the end bearing resistance in compression while capacity in tension is based on the shaft resistance only. The designer will consider an installation depth that will satisfy capacity requirements. An example capacity for an Oliver Displacement Pile on this site installed in the native clayey silt till / silty clay till soils found below elevation 262.1 m would be 461 kN for total capacity in compression and 303 kN in tension in ultimate limit states (ULS). Once detailed design services by the specialty contractor are completed, shop drawings should be provided to SLR for review.

Design capacities for Oliver Displacement Piles must be confirmed and verified with field load tests on a sacrificial pile. A minimum of one (1) pile test in both compression and tension should be conducted per structure on the site.

It is recommended that SLR be retained to monitor and document piles installation to verify that the provided capacity is achieved.

It is noted that the preliminary assessment of the Oliver displacement pile system is based solely on subsurface conditions encountered at Borehole BH26-1. Additional subsurface investigation is required to adequately assess the site-wide feasibility and design parameters for the proposed Oliver displacement pile system.

6.1.3 Lateral Earth Pressures on Two Bridge Crossings

Earth retaining structures can be used at the end of the bridge crossing abutments of the Berczy Creek. The following recommendations are made concerning the design of the retaining wall assuming that the backfill for the retaining wall consists of free-draining granular fill meeting the requirements of OPSS 1010 Granular “A” or Granular “B”. This fill should be compacted in loose lifts, not greater than 200 mm in thickness, to 98 percent of the material's Standard Proctor Maximum Dry Density (SPMDD) in accordance with OPSS 501. Backfill adjacent to the abutments should be placed in general accordance with Ontario Provincial Standard Drawing (OPSD) 803.010, OPSS 422 and Section 7.0 of the Canadian Highway Bridge Design Code (CHBDC) CAN/CSA-S6-14.

Earth retaining structures should be structurally designed to resist the unbalanced horizontal earth and hydrostatic pressure imposed by the backfill adjacent to the wall. The lateral earth and water pressure, p, should be computed using the equivalent fluid pressures presented in Section 6.12 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA S6 14, December 2014, or employing the following equation assuming a triangular pressure distribution:

$$\mathbf{P = K\,[\gamma\,(h\text{-}h_i) + \gamma' h_i + q] + \gamma_w h_i + Cp + Cs}$$

where P = Total lateral pressure acting at depth “h” (kPa)

K = Earth pressure coefficient for vertical walls
and horizontal backfill for permanent construction

h = Depth below finished grade of the point of interest (m)

h_i = Depth below ground water at depth "h" (m)

γ = Unit weight of soil/backfill (kN/m^3)

γ' = Buoyant Unit weight of soil/backfill = $\gamma - \gamma_w$ (kN/m^3)

γ_w = Unit weight of water = 9.8 kN/m^3

q = Equivalent value of surcharge adjacent to walls (kPa)

Cp = compaction pressure (refer to clause 6.12.3 of CHBDC)

Cs = earth pressure induced by seismic events (refer to clause 4.6.4 of CHBDC)

For design purposes, the ground water level at this Site should be assumed to be 1 m higher than the highest measured groundwater level.

A weeping tile system and/or weep holes should be installed in the backfill to minimize the build up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter of geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

For design purposes, the following soil parameters can be assumed for backfill materials.

Table 5. Recommended Soil Parameters for Backfill Materials

Parameter	Materials Conforming to OPSS 1010		
	Granular A	Granular B Type I	Granular B Type II
Effective Angle of Internal Friction, ϕ (°)	35	32	35
Unit Weight, γ (kN/m^3)	22	21	22
Coefficient of Active Earth Pressure, k_a	0.27	0.31	0.27
Coefficient of Earth Pressure At Rest, k_o	0.43	0.47	0.43
Coefficient of Passive Earth Pressure, k_p	3.69	3.25	3.69

The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).

Any topsoil, existing fill materials and existing native soils should not be reused as backfill material under structurally loaded areas. A geotextile/fabric should be placed between retaining wall granular backfill materials and existing soils to mitigate the migration of fines subject to the retaining wall type, geotextile/fabric should be placed at any place where the potential of losing fines in the backfill material would occur.

Sliding resistance between the proposed retaining wall base and subgrade should be calculated in accordance with applicable manuals or guidelines. For design purposes, the coefficient of friction of 0.4 may be assumed between the retaining wall base and the subgrade soils. This value is unfactored and a sufficient factor of safety should be applied in calculating the horizontal resistance.

The final retaining wall design must be reviewed by a qualified engineer. The external modes of failure, such as overturning and global instability (circular failures), should be checked in conjunction with sliding and bearing capacity modes of failures by the engineer/designer. Global stability analysis should be carried out by a geotechnical engineer once the detailed retaining wall design is available. The required minimum factor of safety for sliding and overturning should be 1.5 and 2.0 respectively. The required minimum factor of safety for global stability should be 1.5.

6.1.4 Ground Improvement

An alternative to deep foundation methods, ground improvement techniques such as rammed aggregate piers (RAPs) or controlled modulus columns (CMCs) may be considered for this Site. Utilizing RAPs or CMCs would have several potential benefits for this Site, including higher bearing capacity, increased Seismic Site Class, as well as potentially reducing excess soil and earthworks compared to other deep foundation methods.

A ground improvement will be installed through a granular working platform. The design would include diameter, spacing and depths to provide an improved subgrade designed to support the required footing and floor slab loads and control settlement within tolerable limits.

CMCs are vertical semi-rigid inclusions, i.e. injected columns, with concrete or mortar, installed with a hollow full displacement auger. During the auger extraction process, the column is developed by grouting under controlled limited pressure through the stem of the auger. Column diameter varies between 250 and 450 mm. A load transfer platform is designed between the top of the columns and the structure (slab or foundation) to transfer the load to the CMC. The solution is to provide increased stiffness to the soil mass to safely control both total and differential settlements. The structure's loads are transferred between the soil and the CMC elements.

A specialty contractor must be retained to design and install ground improvement. The designer of specialty contractors should design the depth and methods of installation according to soil conditions and the required design loads. Further recommendations regarding bearing capacity and other design details regarding RAPs or CMCs can be discussed with the specialty contractor. Detailed design services of ground improvement are available through a number of specialty design-build contractors, and stamped shop drawings should be provided to SLR for review.

6.1.5 Additional Comments

Foundations designed to the specified bearing capacity at the serviceability limit states (SLS) are expected to settle less than 25 mm total and 19 mm differential, if designed as per **Table 3**.

All foundations exposed to seasonal freezing conditions must have at least 1.6 metres of soil cover for frost protection.

In the vicinity of the existing buried utilities, all footings must be lowered to undisturbed native soils, or alternatively the services must be structurally bridged. Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

It should be noted that the recommended bearing resistances have been calculated by SLR from the borehole information for the preliminary design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections to validate the information for use during the construction stage.

6.2 Floor Slab and Permanent Drainage

All unsuitable fill materials, weak or disturbed soils or other objectionable material must be removed and stripped prior to supporting the floor slab. Suitable fill materials free of topsoil and organics may be repurposed for floor slab support only if approved after inspection by a geotechnical professional, followed by proof-rolling of the exposed subgrade with a heavy roller to ensure uniform adequate support. Any soft spots revealed during proof rolling must be sub-excavated and backfilled with a well compacted approved material. The backfill required to raise the grade can consist of inorganic soil, placed in shallow lifts (200 mm) and compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD).

The foundation may be designed as a water-tight structure, assuming the hydrostatic water pressure at the ground surface. However, a moisture barrier consisting of at least 300 mm of 19 mm clear crushed stone should be installed under the floor slab, in case minor water seepage may migrate through the waterproof system.

Special care should be taken to ensure compaction around columns and adjacent to foundation walls. Unless the foundations are designed to account for the floor slab loads, the floor slabs should be structurally separated from the foundation walls and columns. Sawcut control joints should be provided at regular intervals and along column lines to minimize shrinkage cracking and to allow for differential settlement of the floor slabs.

Where the backfill against the exterior walls is to support settlement sensitive structures, such as concrete slabs, pavements or walkways, it should be uniformly compacted to at least 98% of SPMDD.

6.3 Excavations and Backfilling

Excavations can be carried out with a heavy hydraulic backhoe. Possible large obstructions such as buried concrete pieces and existing foundations may also be encountered at the site and in the fill materials. Provisions must be made in the excavation contract for the removal of possible boulders in the till or obstructions in the fill material.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill materials and disturbed silty clay / clayey silt would be classified as Type 3 Soils above the groundwater table and Type 4 soils below the groundwater table. The firm to hard soils cohesive silty clay till / clayey silt till would be classified as Type 2 Soil above the groundwater table and Type 4 Soils below the groundwater table. Within any excavation with multiple soil types, the lowest type soil will govern the classification of the entire excavation area.

Provided adequate groundwater control is achieved, it is anticipated that the majority of the engineered fill pad excavations at the Site consist of temporary open cuts with side slopes of 1 horizontal to 1 vertical (1H: 1V) to the base of the excavation. However, depending on the construction procedures adopted by the contractor and weather conditions at the time of construction, some local flattening of the slopes might be required. Where side slopes of excavations are to be steepened, then a positive excavation support system should be considered.

The existing fill in the boreholes is generally not suitable for re-use as backfill. The native soils free from topsoil and organics can be used as general construction backfill. Loose lifts of soil, which are to be compacted, should not exceed 200 mm. Depending on the time of construction and weather, some excavated material may be too wet to compact and will require aeration prior to its use. The excavated soils are not considered to be free draining.

Under floor fill should be compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD). The excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill such as OPSS Granular “B” should be used. Imported granular fill, which can be compacted with handheld equipment, should be used in confined areas.

It should be noted that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for adequate compaction. Stockpiles should be compacted at the surface or be covered with tarpaulins to minimize moisture uptake.

It is expected that any seepage above the groundwater table can be removed by pumping from sumps in the building development area. Significant seepage should be expected once the excavations extend below the prevailing groundwater tables in the cohesionless sandy/silty soils at the Site. Depending upon the actual thickness and extent of these soils, the prevailing groundwater level at the time of construction, “active, advance” dewatering measure using well points/eductors may be required to maintain the stability of the base and side slopes of the excavations in these areas. These ‘active dewatering’ measures would have to be installed and then operated for one to two weeks in advance of excavation work progressing to these areas. A contractor specializing in dewatering should be retained to design the active dewatering systems.

Under MECP requirements, registration on the Environmental Activity Sector Registry (EASR) is required when dewatering is greater than 50,000 L/day. It is noted that recent changes to Ontario Regulation (O. Reg.) 63/16 under the Environmental Protection Act effective July 1, 2025, removed the maximum daily rate where previously dewatering greater than 400,000 L/day triggered the need for a Permit to Take Water (PTTW). A separate Hydrogeological Investigation by SLR provides discussions regarding the dewatering requirements for any excavations below the groundwater table.

6.4 Basal Heave Risk Assessment

Based on the proposed site plan provided by the Client, one (1) monitoring well (BH26-1) was installed within the footprint of mausoleum. The groundwater monitoring data show that artesian pressure head up to 0.7 m above ground surface is observed in this monitoring well. The remaining three (3) monitoring wells (BH26-2 to BH26-4) indicate relatively high groundwater levels, ranging from approximately 0.3 m to 2.1 m below the existing ground surface.

Depressurizing the aquifer within the excavation area may be required to reduce the risk of potential basal heave or raising founding levels.

It is noted that the assessment of safe excavation depths is preliminary and based on limited subsurface information. More detailed evaluation of allowable excavation depths can be undertaken following the completion of additional deeper boreholes and groundwater monitoring to better characterize subsurface and hydrogeological conditions.

A ground water monitoring program is recommended to review groundwater conditions and establish final founding depths for all structures.

6.5 Temporary Shoring and Shoring Movement Monitoring

In view of the anticipated underground parking structure of the subjected buildings and excavation, and the fact that the proposed structures will extend close to the property lines, it is anticipated that the proposed excavations will be supported by a temporary shoring system. Within the anticipated excavation depth, the Site soils consist of clayey silt till to silty clay till deposits encountered in all boreholes extending to the approximate elevations of 267.3 to 260.6 m. Contiguous cut-off caisson wall may be considered to reduce the groundwater seepage during the temporary dewatering stage. Alternatively, soldier piles with wood lagging could be considered, provided that adequate groundwater control is achieved and the contractor is prepared to minimize the expected caving and groundwater seepage.

The contiguous concrete caisson walls could be supported by tie-back anchors or struts. For post tensioned pressure-grouted soil anchors installed into very stiff to hard soils, the allowable (SLS) bond stress between grout and soil can be assumed to be 150 kPa, but in no case should the bonded length be less than 6m.

The top anchor must not be placed lower than 3.0 meters below the top of the ground surface. Casings will be required for the anchors when penetrating through sandy/silty deposits to prevent from soil caving. Bond values are suggested but these values are arbitrary since the contractor's installation procedures will determine the actual soil to concrete bond value. Hence, the contractor must decide on a capacity and confirm its availability. The actual capacity (bond resistance) of the anchors should be established by full scale pull-out tests ("performance test") at each anchor level in accordance with CFEM (5th edition), testing to 200% of working load. Each installed anchor must be proof loaded to 1.33 times the design working load, in accordance with Post-Tensioning Institute (PTI) guidelines. The ground anchors should be double- corrosion protected (i.e. PTI Class I). Adhesion on the behind the shoring system must be neglected when designing this shoring system.

An SLS bearing capacity can be taken as 300 kPa for pile/caissons founded in the hard clayey silt till / silty clay till.

Lagging walls are assumed to permit drainage of perched water between the lagging boards. No water pressure has been assumed to act against the shoring for lagging walls which are assumed to permit drainage of perched water between the lagging boards. Construction caisson walls need to be designed for hydrostatic water pressure, taken as 1m higher than the highest highest measured from nearby monitoring wells. The top of the shaft lining should be raised up above the grade in consideration of the risk of shaft flooding. Surcharge load due to construction machinery and traffic must be considered.

If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding. The soldier piles/caissons should be installed in pre-augered holes drilled into the piles. No loss of ground should be permitted during augering for soldier piles/caissons and the drilling contractor should be warned of the potential for encountering overburden obstructions within the fill and obstructions with the till such as cobbles and/or boulders. The use of liners may be required to keep the augered holes open before concrete placement. In the event that loss of ground occurs during lagging installation, the depth of excavated lifts must be reduced. In critical areas, the lost soils must be replaced by grouting. The concrete strength must be specified by the shoring designer.

For contiguous concrete caisson shoring, the rate of groundwater seepage is expected to be slow to moderate and can be handled by gravity drainage and pumping from filtered sumps established at the base of the excavation.

All shoring designs should be in accordance with the 5th Edition of the Canadian Foundation Engineering Manual and must be reviewed by a qualified geotechnical engineer. The soil parameters estimated to be applicable for this design are provided in **Table 2**.

Monitoring of shored wall deflections by means of survey targets is recommended in areas where settlements could damage existing utilities, infrastructures, or buildings.

Movement of the shoring system is inevitable. Vertical movement will result from the vertical load on the shoring system resulting from the inclined tiebacks and inward horizontal movement results from earth and water pressures. The magnitude of this movement can be controlled by sound construction practices, and it is anticipated that the horizontal movement will be in the range of 0.1% to 0.25%H.

To ensure that movements of the shoring are within an acceptable range, a monitoring program must be carried out. Vertical and horizontal targets on the soldier piles must be located and surveyed before excavation begins. Weekly readings during excavation should show that the movements will be within those predicted; if not, the monitoring results will enable directions to be given to improve the shoring. The movement should be monitored throughout the construction period.

In order to install the shoring system and carrying out the excavation, dewatering may be required at the Site. Concrete plug may need to be installed at the bottom of the excavation in order to prevent basal heave and/or excessive groundwater seepage through the bottom of the excavation. Temporary dewatering could be an issue with regards to surrounding ground settlements due to the sandy/silty soils which exist within the zone of influence. If required, the impact of temporary dewatering will be discussed in the hydrogeological report.

6.6 Preconstruction Condition Survey

It is recommended that a preconstruction survey of the neighbouring buildings and other nearby utilities and structures be carried out prior to commencing excavation. In addition, the types and conditions of all adjacent structures and underground services should be reviewed by the structural and geotechnical design engineers. Each utility owner should be contacted to establish deformation limits. The deformation should be monitored throughout the construction period.

6.7 Earth Pressures

The lateral earth pressures acting at any depth on foundation walls may be calculated from the following expression:

$$\mathbf{P_h = K\,(\gamma\, h + q)}$$

where P_h = Lateral earth pressure acting at depth “h” (kPa)

K = Earth pressure coefficient, as shown in **Table 2**

and horizontal backfill for permanent construction

γ = Unit weight of backfill, as shown in **Table 2**

h = Depth below finished grade of the point of interest (m)

q = Equivalent value of surcharge on the ground surface (kPa)

The hydrostatic water pressure must also be added to the foregoing earth pressure. The unit weight of the soil below the design groundwater table elevation should be taken as the submerged unit weight in the earth pressure calculations.

6.8 Seismic Considerations

The 2012 Ontario Building Code (OBC 2012) came into effect on January 1, 2014 and contains updated seismic analysis and design methodology. The seismic site classification methodology outlined in the code is based on the subsurface conditions within the upper 30 m below existing grade.

The conservative site classification is based on physical borehole information obtained at depths of less than 30 m and based on general knowledge of the local geology and physiography. In this regard, SLR's drilling program included boreholes drilled to depths up to 6.7 m below the existing ground surface. Based on the borehole information and our local experience, a Site Class D may be used for the building design.

Should optimization of the site class be recommended by the structural engineer, in situ geophysical testing or a deep borehole extending to 30 m may be considered.

6.9 Pavement Recommendations

The recommended pavement structures provided in **Table 6** are based upon the borehole information obtained in this investigation. The values may need to be adjusted based on the municipality/regional standards. Consequently, the recommended pavement structures should be considered for reference purposes only. A functional design life of eight to ten years has been used to establish the pavement recommendations. This represents the number of years to the first rehabilitation, assuming regular maintenance is carried out. If required, a more refined pavement structure design can be performed by a specialized pavement engineer based on specific traffic data and design life requirements and will involve specific laboratory tests to determine frost susceptibility and strength characteristics of the subgrade soils, as well as specific data input from the client.

Table 6. Recommended Pavement Structure Thickness

Pavement Layer	Compaction Requirements	Light Duty Pavement (Parking for Cars)	Heavy Duty Pavement (Fire Routes, Parking for Delivery Trucks)
Asphaltic Concrete	92%	40 mm HL 3	40 mm HL 3
	Maximum Relative Density (MRD)	50 mm HL 8	80 mm HL 8
OPSS Granular 'A' Base (or 20mm Crusher Run Limestone)	100% SPMDD*	150 mm	150 mm

Pavement Layer	Compaction Requirements	Light Duty Pavement (Parking for Cars)	Heavy Duty Pavement (Fire Routes, Parking for Delivery Trucks)
OPSS Granular ‘B’ (or 50mm Crusher Run Limestone)	100% SPMDD*	250 mm	350 mm

* Denotes Standard Proctor Maximum Dry Density, ASTM-D698

The subgrade must be compacted to 98% SPMDD for at least the upper 500 mm unless accepted by SLR.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over-emphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. Subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening. This is particularly important in heavy-duty pavement areas.

Additional comments on the construction of parking areas and access roadways are as follows:

1) As part of the subgrade preparation, proposed parking areas and roadways should be stripped of topsoil and other obvious objectionable material. Fill required to raise the grades to design elevations should conform to backfill requirements outlined in previous sections of this report. The subgrade should be properly shaped, crowned then proof-rolled in the full-time presence of a qualified engineering personnel. Soft or spongy subgrade areas should be sub-excavated and properly replaced with suitable approved backfill compacted to 98% SPMDD.

2) The locations and extent of sub-drainage required within the paved areas should be reviewed by a pavement engineer in conjunction with the proposed lot grading. Assuming that satisfactory crossfalls in the order of two percent have been provided, subdrains extending from and between catch basins may be satisfactory. In the event that shallower crossfalls are considered, a more extensive system of sub-drainage may be necessary and should be reviewed by a specialized pavement engineer.

3) The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted access

lanes, half-loads during paving, etc., may be required, especially if construction is carried out during unfavourable weather.

4) It is recommended that a specialized pavement engineer be retained to design and review the final pavement structure and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report and applicable municipal/regional standards.

6.10 Geotechnical Quality of Excavated Soils

Reference to the borehole logs suggests that the excavated materials with respect to their compaction characteristics can be divided into three groups:

- **Group 1** comprises the native cohesive clayey silt till / silty clay till and have moisture content very close to or above its optimum water content. This material will excavate in clods and would thus require a heavy pad footed compactor or hoe pack to break it down and adequately compact it. Given the water content of the clayey silt till, it may not be possible to obtain a degree of compaction of this material much above 95% of SPMDD. This degree of compaction might be acceptable within landscaped areas above which pavements or infrastructure are not expected to be built in the future.

- **Group 2** soils consist of unsuitable materials because of their high moisture, organic inclusions, or deleterious inclusions, including the existing fill materials and disturbed native soils with organics and topsoil. These soils should be either disposed off-site or should be used only in “soft” landscaping areas where they can be placed with nominal compaction, and where surface settlements are tolerable.

- **Group 3** soils consist of existing fill materials and disturbed native soils with low organic content. These soils may be reused for backfill in areas of development that are not within proposed building / foundation footprint, provided these fill materials are carefully segregated from the topsoil / organic fill.

As a general requirement, all backfill material should be placed in 200 to 300mm thick loose lifts and compacted to at least 96% of SPMDD, at a placement moisture content within +/-2% of the optimum. Below future pavements, the backfill must be Granular “A” or “B” material, and the top 1.5m of subgrade backfill below the underside of the pavement structure should be compacted to 98% of SPMDD. Where a free-draining backfill is needed or where the backfill is needed for structural support of overlying structures, the site soils will not be suitable and OPSS Granular “A” or “B” sand and gravel will be required. Similarly, during work in the autumn, winter and spring months, re-use of the excavated soils as compacted fill may not be practical and imported OPSS Granular “B” should be used.

7.0 Certification

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

This report was prepared, reviewed and approved by the undersigned:

Regards,

SLR Consulting (Canada) Ltd.

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8.0 References

ASTM International. 2018. ASTM D1586 / D1586M-18, Standard test method for standard penetration test (SPT) and split-barrel sampling of soils.

Canadian Geotechnical Society. 2023. Canadian Foundation Engineering Manual, 5th Edition.

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Ontario Geological Survey 2010. Surficial geology of southern Ontario; Ontario Geological Survey, Miscellaneous Release— Data 128 – Revised.

Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1.

Ontario Ministry of Natural Resources, Water Resources Section, 2002. Technical Guide, River & Stream Systems: Erosion Hazard Limit.

General Comments and Limitations of Report

SLR should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, SLR will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes and test pits required to determine the localized underground conditions between boreholes and test pits affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole and test pit results, so that they may draw their own conclusions as to how the subsurface conditions may affect them. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to SLR at the time of preparation. Unless otherwise agreed in writing by SLR, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SLR accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

Drawings

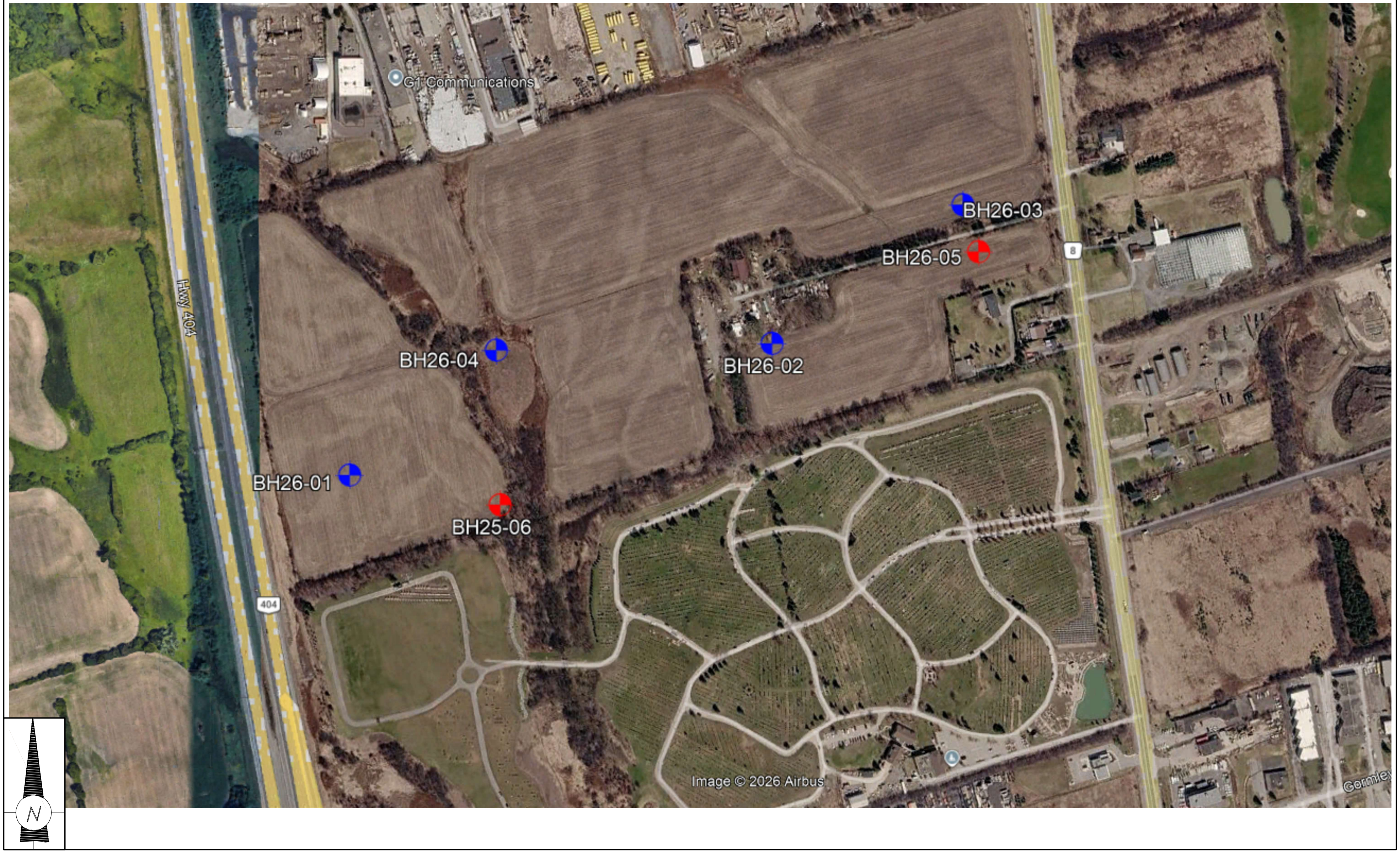
Preliminary Geotechnical Investigation

12612 and 12586 Woodbine Avenue, Stouffville, Ontario

Kingsdale Development

SLR Project No.: 209.065495.00001

July 9, 2026



LEGEND:

🔵 Borehole/Monitoring Well Location

🔴 Borehole Location

Client: **Kingsdale Developement**

Project No.: **243.024905.00001**

Drawing No.: **1**

Drawn: **SL**

Approved: **DRAFT**

Title: **Borehole / Monitoring Well Location Plan**

Date: **June, 2026**

Scale: **As Shown**

Project: **Geotechnical Investigation -**
12612 and 12586 Woodbine Avenue, Stouffville, Ontario

Original Size: **Letter**

Rev: **1**

SLR

871 Equestrian Court, Unit 1
Oakville, ON
L6L 6L7

Appendix A Borehole Records

Preliminary Geotechnical Investigation

12612 and 12586 Woodbine Avenue, Stouffville, Ontario

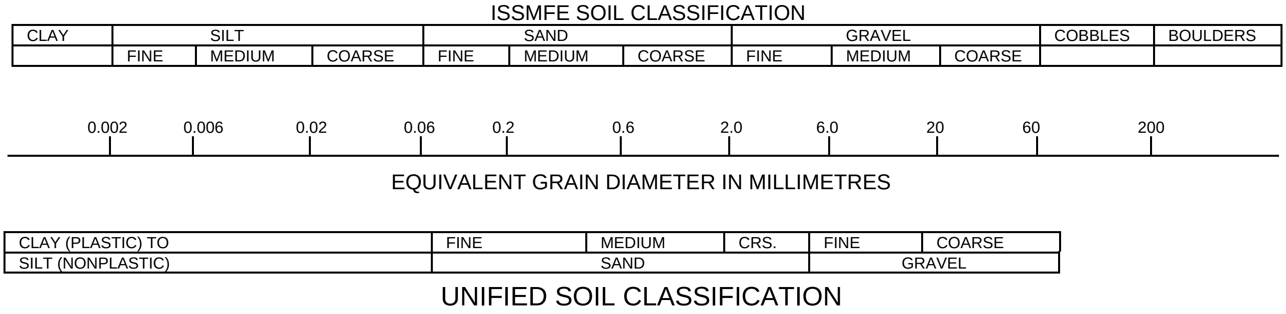
Kingsdale Development

SLR Project No.: 209.065495.00001

July 9, 2026

Notes On Sample Descriptions

1. All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by PECG also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Explanation of Terms Used in the Record of Borehole

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60^{o} cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	>300 mm
Cobbles	75 mm-300 mm
Gravel (Gr)	4.75 mm-75 mm
Sand (Sa)	0.075 mm-4.75 mm
Silt (Si)	0.002 mm-0.075 mm
Clay (Cl)	<0.002 mm

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	>35%

Soil Description

a) Cohesive Soils

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

b) Cohesionless Soils

Density Index (Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

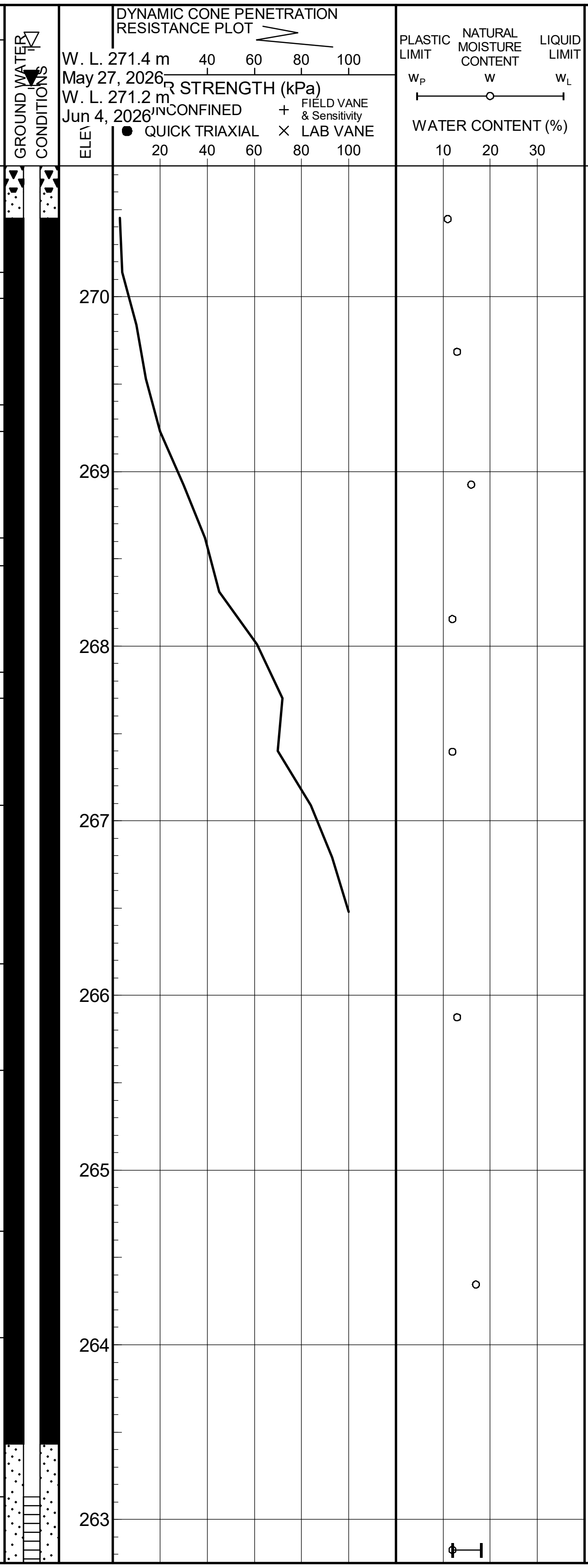
LOG OF BOREHOLE BH26-01

PROJECT: Geotechnical Investigation - 12612 Woodbine Ave
CLIENT: 12612 Woodbine (KD) Corp.
PROJECT LOCATION: Gormley, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan N 4867373.54 E 629183.86

Method: Hollow Stem Augers
Diameter: 200 mm
Date: May 22, 2026

REF. NO.: 243.024905.00001
ENCL NO.: 1

SOIL PROFILE		SAMPLES			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (fines)
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m			
270.8	Ground Surface						
0.0	**TOPSOIL:** 150mm						
270.6 0.2	**CLAYEY SILT (DISTURBED):** sandy, trace gravel, trace organics, brown, moist, firm to very stiff.	1	SS	8			
		2	SS	16			
269.1 1.7	**SILTY CLAY TILL:** some sand to sandy, trace gravel, oxidised, brown to grey, moist to wet, stiff to very stiff.	3	SS	17			
	grey below 2.3m	4	SS	16			
		5	SS	14	125		
	wet below 4.6m	6	SS	15	75		
		7	SS	11			
	contains silt seams/layers	8	SS	28			7 29 46 18



Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH26-01

PROJECT: Geotechnical Investigation - 12612 Woodbine Ave

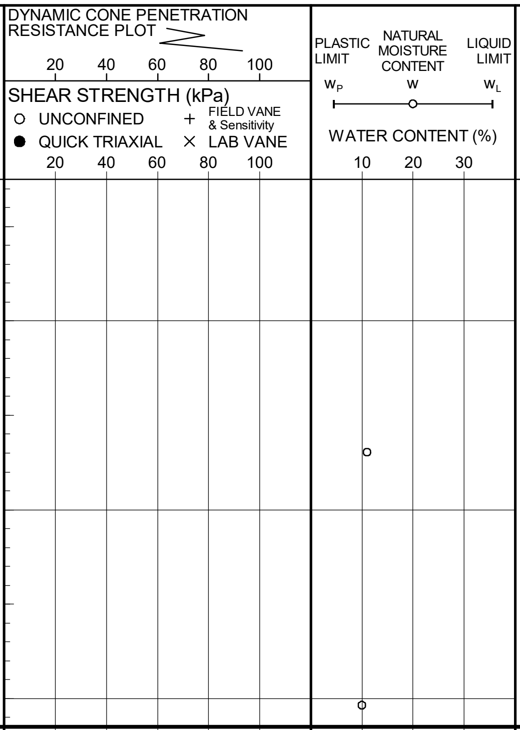
CLIENT: 12612 Woodbine (KD) Corp.
Method: Hollow Stem Augers

PROJECT LOCATION: Gormley, ON
Diameter: 200 mm
REF. NO.: 243.024905.00001

DATUM: Geodetic
Date: May 22, 2026
ENCL NO.: 1

BH LOCATION: See Borehole Location Plan N 4867373.54 E 629183.86

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION
	Continued				
	SILTY CLAY TILL: some sand to sandy, trace gravel, oxidised, brown to grey, moist to wet, stiff to very stiff.(Continued)				
262.1 8.7	**SILTY CLAY TILL:** some sand, trace gravel, contains dilatant wet sandy silt layers, grey, moist to wet, hard.				262
		9	SS	32	
					261
	contains wet sand and gravel layers	10	SS	50/ 75mm	260
259.9 10.9	**END OF BOREHOLE**				



POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (kN/m³) | REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (fines)

Depth scale: 9, 10

Notes:
1) A 50mm diameter monitoring well was installed upon completion of drilling.

2) Water Level Readings:

Date	W. L. Depth (mBGS)
2026-05-27	-0.68
2026-06-04	-0.46

(negative means above ground surface)

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

SLR

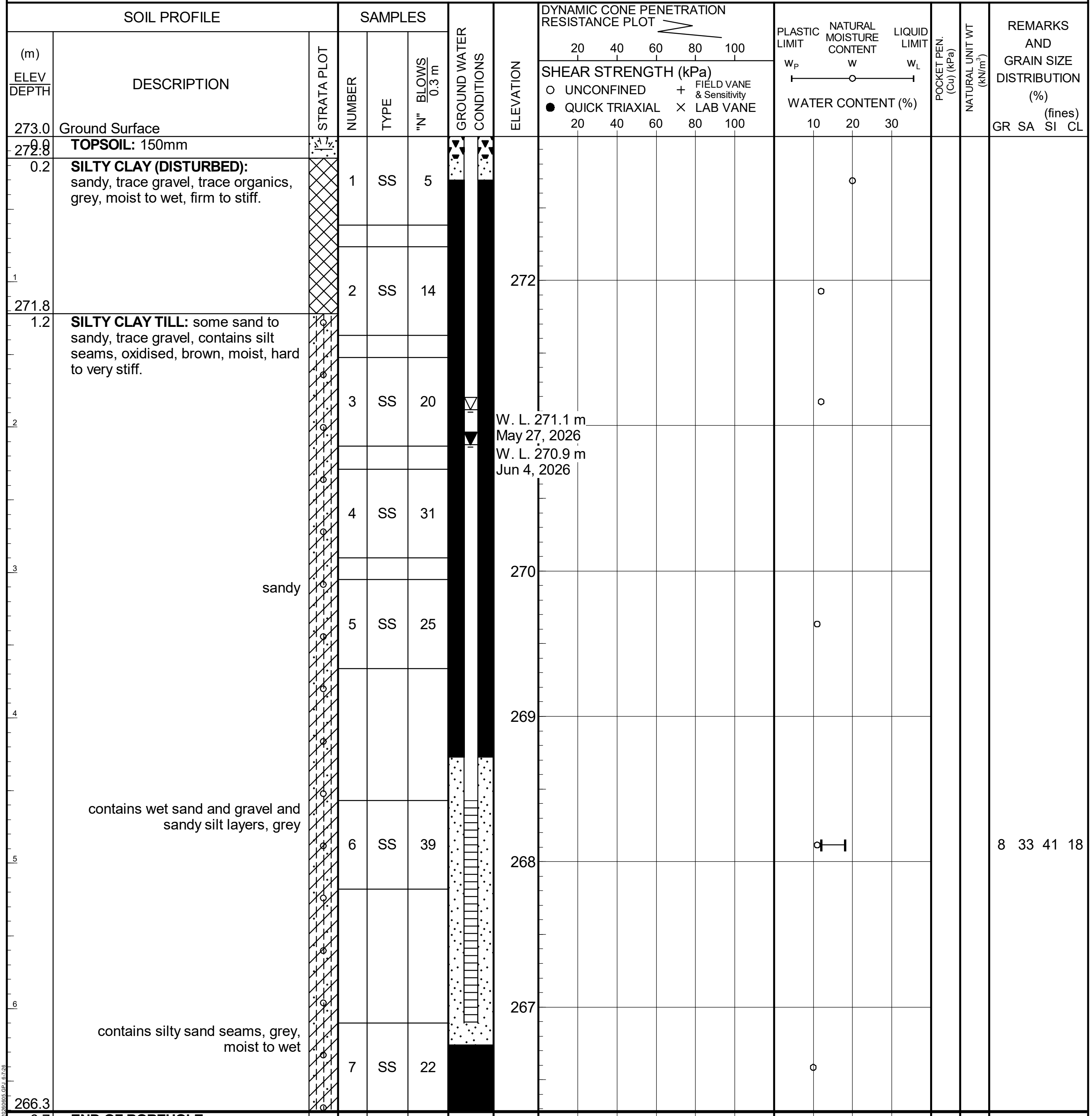
LOG OF BOREHOLE BH26-02

1 OF 1

PROJECT: Geotechnical Investigation - 12612 Woodbine Ave
CLIENT: 12612 Woodbine (KD) Corp.
PROJECT LOCATION: Gormley, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan N 4867523.03 E 629637.17

Method: Hollow Stem Augers
Diameter: 200 mm
Date: May 21, 2026

REF. NO.: 243.024905.00001
ENCL NO.: 2



(m) ELEV DEPTH	DESCRIPTION
273.0	Ground Surface
272.8 0.0	**TOPSOIL:** 150mm
0.2	**SILTY CLAY (DISTURBED):** sandy, trace gravel, trace organics, grey, moist to wet, firm to stiff.
271.8 1.2	**SILTY CLAY TILL:** some sand to sandy, trace gravel, contains silt seams, oxidised, brown, moist, hard to very stiff. sandy contains wet sand and gravel and sandy silt layers, grey contains silty sand seams, grey, moist to wet
266.3 6.7	**END OF BOREHOLE**

NUMBER	TYPE	"N" BLOWS / 0.3 m	GR	SA	SI	CL (fines)
1	SS	5				
2	SS	14				
3	SS	20				
4	SS	31				
5	SS	25				
6	SS	39	8	33	41	18
7	SS	22				

REMARKS AND GRAIN SIZE DISTRIBUTION (%)

Notes:
1) A 50mm diameter monitoring well was installed upon completion of drilling.

2) Water Level Readings:

Date	W. L. Depth (mBGS)
2026-05-27	1.88
2026-06-04	2.12

SLR_0_SOIL-ROCK-FEB-2025_PM_ROCK_HYDROG FORM_SLR_LOGO 1.GLB
SLR SOIL - 2025 1DIG 243.024905.00001-12612 WOODBINE GINT LOGS 20260605.GPJ 6-7-26

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 3=3% Strain at Failure

SLR

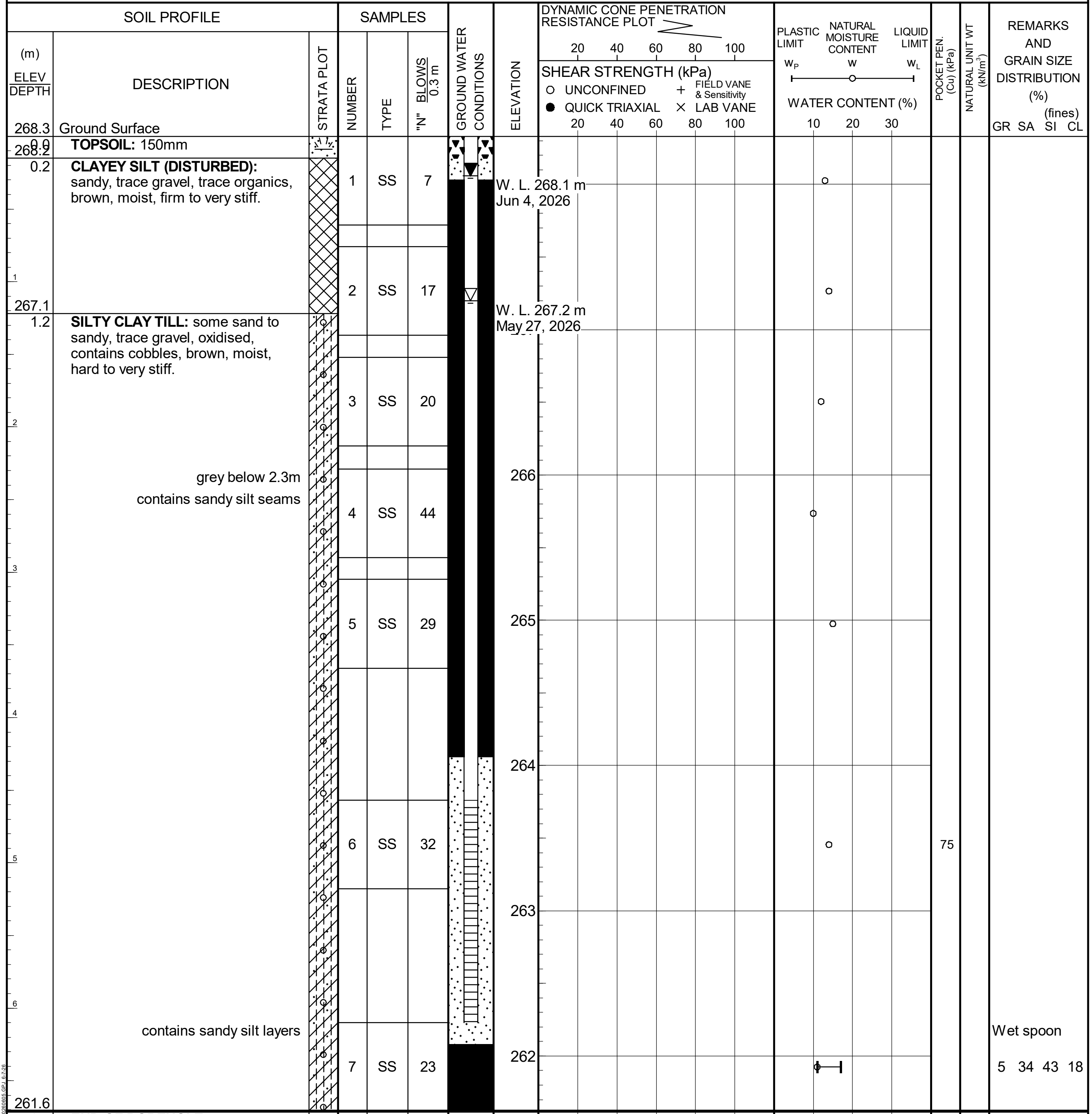
LOG OF BOREHOLE BH26-03

1 OF 1

PROJECT: Geotechnical Investigation - 12612 Woodbine Ave
CLIENT: 12612 Woodbine (KD) Corp.
PROJECT LOCATION: Gormley, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan N 4867676.94 E 629838.87

Method: Hollow Stem Augers
Diameter: 200 mm
Date: May 21, 2026

REF. NO.: 243.024905.00001
ENCL NO.: 3



ELEV / DEPTH (m)	DESCRIPTION
268.3	Ground Surface
0.0	**TOPSOIL:** 150mm
268.2 0.2	**CLAYEY SILT (DISTURBED):** sandy, trace gravel, trace organics, brown, moist, firm to very stiff.
267.1 1.2	**SILTY CLAY TILL:** some sand to sandy, trace gravel, oxidised, contains cobbles, brown, moist, hard to very stiff. grey below 2.3m contains sandy silt seams contains sandy silt layers
261.6 6.7	**END OF BOREHOLE**

NUMBER	TYPE	"N" BLOWS 0.3 m	POCKET PEN. (Cu) (kPa)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7		
2	SS	17		
3	SS	20		
4	SS	44		
5	SS	29		
6	SS	32	75	
7	SS	23		Wet spoon 5 34 43 18

Notes:
1) A 50mm diameter monitoring well was installed upon completion of drilling.

2) Water Level Readings:

Date	W. L. Depth (mBGS)
2026-05-27	1.13
2026-06-04	0.27

SLR_0_SOIL-ROCK-FEB-2025_PM_ROCK_HYDROG FORM_SLR_LOGO 1.GLB
SLR SOIL - 2025 1DIG 243.024905.00001-12612 WOODBINE GINT LOGS 20260605.GPJ 6-7-26

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity
○ 8=3% Strain at Failure

LOG OF BOREHOLE BH26-04

PROJECT: Geotechnical Investigation - 12612 Woodbine Ave

CLIENT: 12612 Woodbine (KD) Corp.

PROJECT LOCATION: Gormley, ON

DATUM: Geodetic

BH LOCATION: See Borehole Location Plan N 4867508.5 E 629341.64

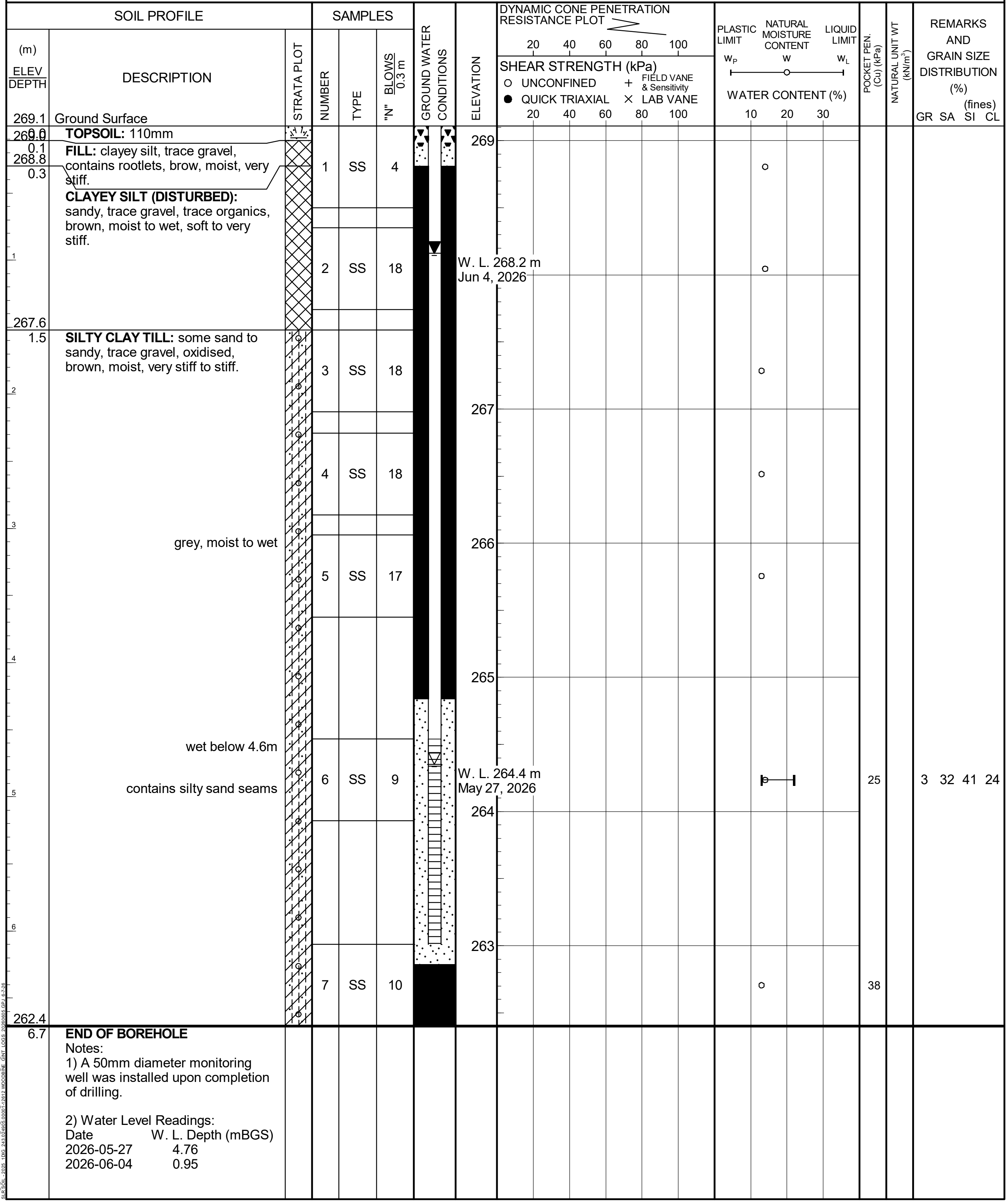
Method: Hollow Stem Augers

Diameter: 200 mm

Date: May 22, 2026

REF. NO.: 243.024905.00001

ENCL NO.: 4



ELEV DEPTH (m)	DESCRIPTION
269.1	Ground Surface
269.0 0.0	**TOPSOIL:** 110mm
0.1	**FILL:** clayey silt, trace gravel, contains rootlets, brow, moist, very stiff.
268.8 0.3	**CLAYEY SILT (DISTURBED):** sandy, trace gravel, trace organics, brown, moist to wet, soft to very stiff.
267.6 1.5	**SILTY CLAY TILL:** some sand to sandy, trace gravel, oxidised, brown, moist, very stiff to stiff. grey, moist to wet wet below 4.6m contains silty sand seams
262.4 6.7	**END OF BOREHOLE**

NUMBER	TYPE	"N" BLOWS/0.3 m	POCKET PEN. (Cu) (kPa)	GR	SA	SI	CL
1	SS	4					
2	SS	18					
3	SS	18					
4	SS	18					
5	SS	17					
6	SS	9	25	3	32	41	24
7	SS	10	38				

W. L. 268.2 m
Jun 4, 2026

W. L. 264.4 m
May 27, 2026

Notes:
1) A 50mm diameter monitoring well was installed upon completion of drilling.

2) Water Level Readings:

Date	W. L. Depth (mBGS)
2026-05-27	4.76
2026-06-04	0.95

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH26-05

1 OF 1

PROJECT: Geotechnical Investigation - 12612 Woodbine Ave

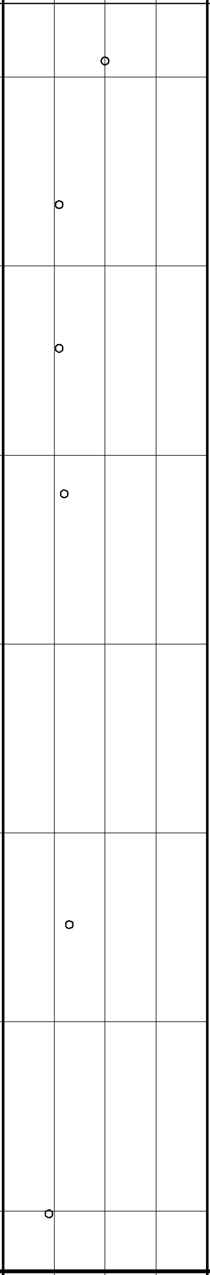
CLIENT: 12612 Woodbine (KD) Corp.
Method: Hollow Stem Augers

PROJECT LOCATION: Gormley, ON
Diameter: 200 mm
REF. NO.: 243.024905.00001

DATUM: Geodetic
Date: May 21, 2026
ENCL NO.: 5

BH LOCATION: See Borehole Location Plan N 4867626.47 E 629855.77

SOIL PROFILE			SAMPLES									
(m) ELEV/DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (fines)
267.4	Ground Surface											
0.0 267.3	**TOPSOIL:** 120mm											
0.1	**SILTY CLAY (DISTURBED):** sandy, trace gravel, trace organics, brown, moist, firm to very stiff.		1	SS	6		267					
			2	SS	18		266					
265.7 1.7	**SILTY CLAY TILL:** some sand to sandy, contains organics, oxidised, brown, moist, hard to stiff.		3	SS	19							
			4	SS	57		265					Spoon bouncing Auger grinding
	contains rock fragments, grey below 3.05m		5	SS	23		264					
							263					
			6	SS	13		262					
	contains silty sand seams/pockets, moist to wet		7	SS	32		261					
260.7 6.7	**END OF BOREHOLE**											



GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 8=3% Strain at Failure

LOG OF BOREHOLE BH26-06

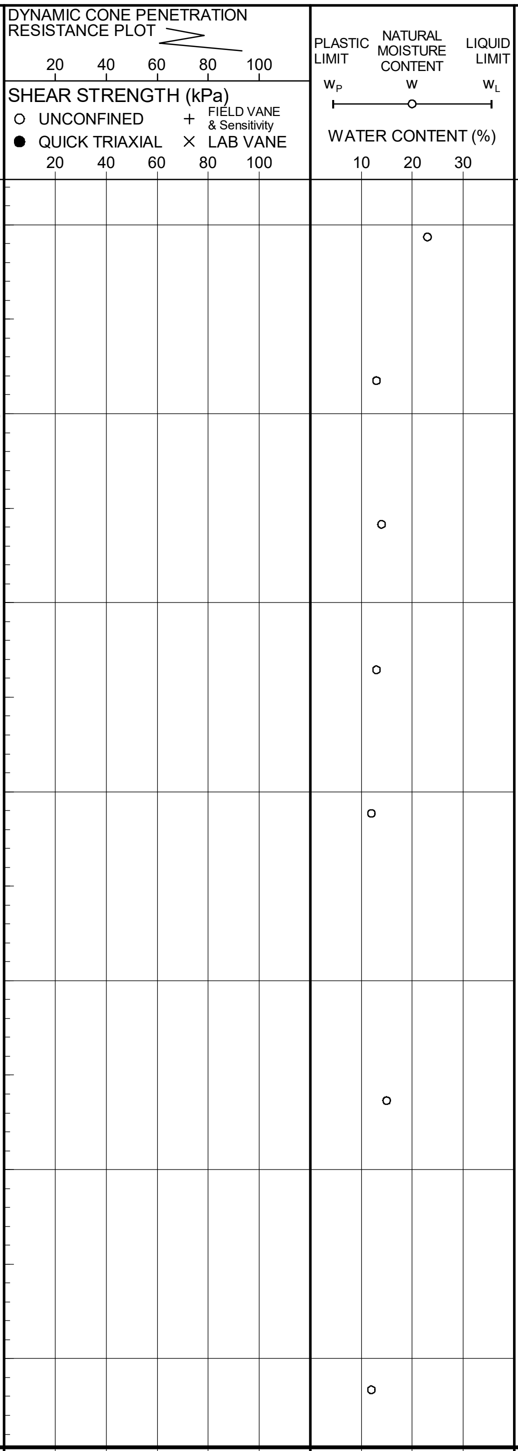
PROJECT: Geotechnical Investigation - 12612 Woodbine Ave
CLIENT: 12612 Woodbine (KD) Corp.
PROJECT LOCATION: Gormley, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan N 4867343.93 E 629345.34

Method: Hollow Stem Augers
Diameter: 200 mm
Date: May 22, 2026

REF. NO.: 243.024905.00001
ENCL NO.: 6

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION
266.2	Ground Surface
0.0 266.1 0.1	**TOPSOIL:** 50mm **SILTY CLAY (DISTURBED):** some sand, trace gravel, trace organics, brown, moist, firm to very stiff.
264.6 1.7	**CLAYEY SILT TILL:** sandy, trace gravel, oxidised, brown, moist to wet, very stiff.
264.0 2.2	**SILTY CLAY TILL:** some sand, trace gravel, brownish grey, moist, very stiff to firm. some sand to sandy, grey, wet
260.6 5.6	**CLAYEY SILT TILL:** sandy, trace gravel, grey, wet, stiff.
259.5 6.7	**END OF BOREHOLE** Note: 1) Borehole was open with no groundwater accumulated.

SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	POCKET PEN. (Cu) (kPa)
1	SS	6	
2	SS	8	
3	SS	26	
4	SS	16	
5	SS	17	
6	SS	6	25
7	SS	15	



GROUND WATER CONDITIONS | NATURAL UNIT WT (kN/m³) | REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (fines)

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity
○ 3 =3% Strain at Failure

Appendix B Laboratory Test Results

Preliminary Geotechnical Investigation

12612 and 12586 Woodbine Avenue, Stouffville, Ontario

Kingsdale Development

SLR Project No.: 209.065495.00001

July 9, 2026

SLR Consulting (Canada) Ltd.

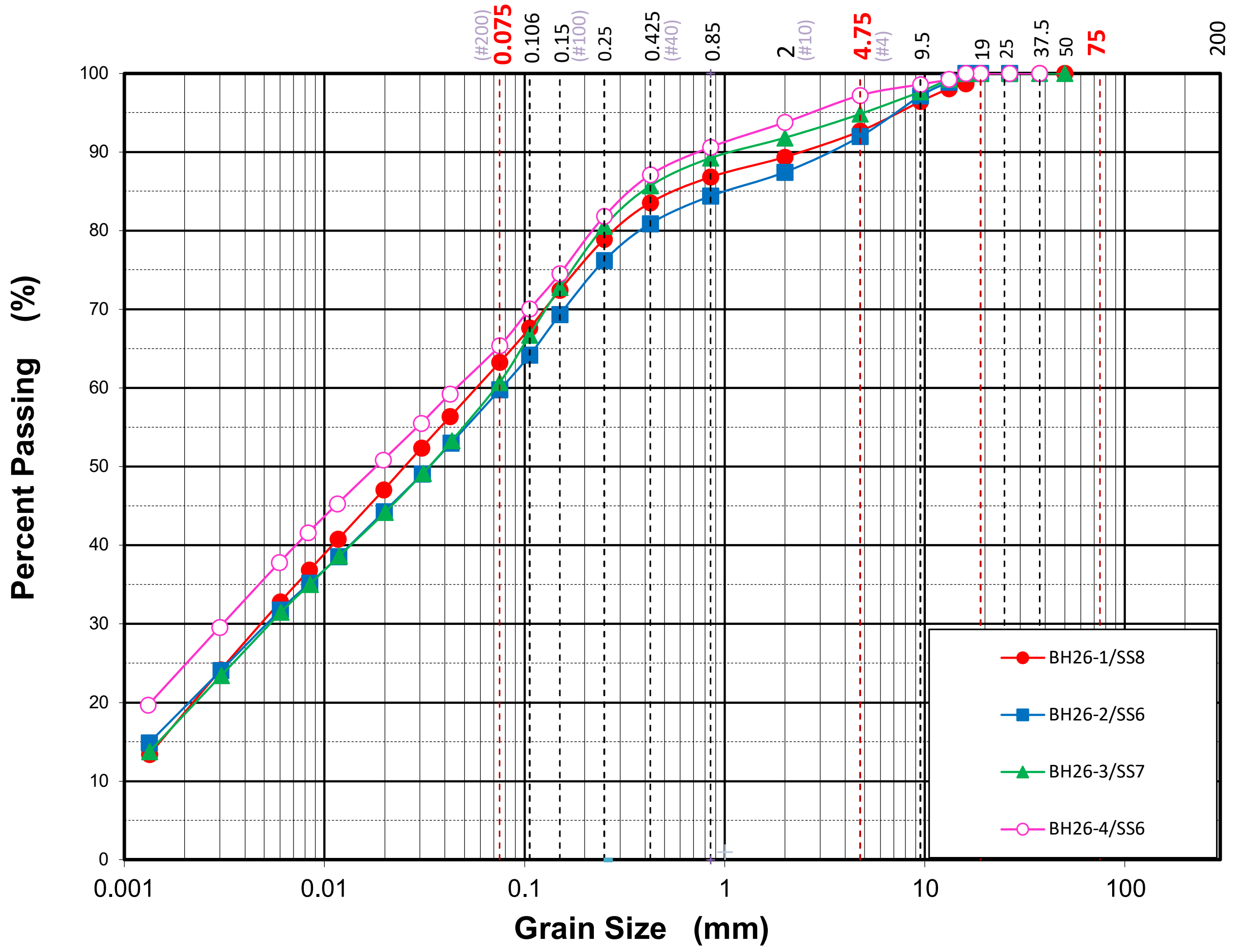
871 Equestrain Ct, Unit 1
Oakville, ON L6L 6L7

Particle Size Distribution Report
(ASTM D421/422)

Project No.:	243.024905.00001	Lab No.:	R26-022
Project Name:	12612 Woodbine Ave	Tested By:	BW
Client:	12612 Woodbine (KD) Corp.	Checked By:	TO
Location:	Gromley, Ontario	Date:	5/26/2026

Test Results

Test No.	Sample No.	Clay	Silt	Sand			Gravel		Cobble+	Remarks
				Fine	Medium	Coarse	Fine	Coarse		
1	BH26-1/SS8	18	46	20	6	3	7			
2	BH26-2/SS6	18	41	21	7	5	8			
3	BH26-3/SS7	18	43	25	6	3	5			
4	BH26-4/SS6	24	41	22	7	3	3			
5										
6										
7										
8										



SLR Consulting (Canada) Ltd.
871 Equestrain Ct, Unit 1
Oakville, ON L6L 6L7

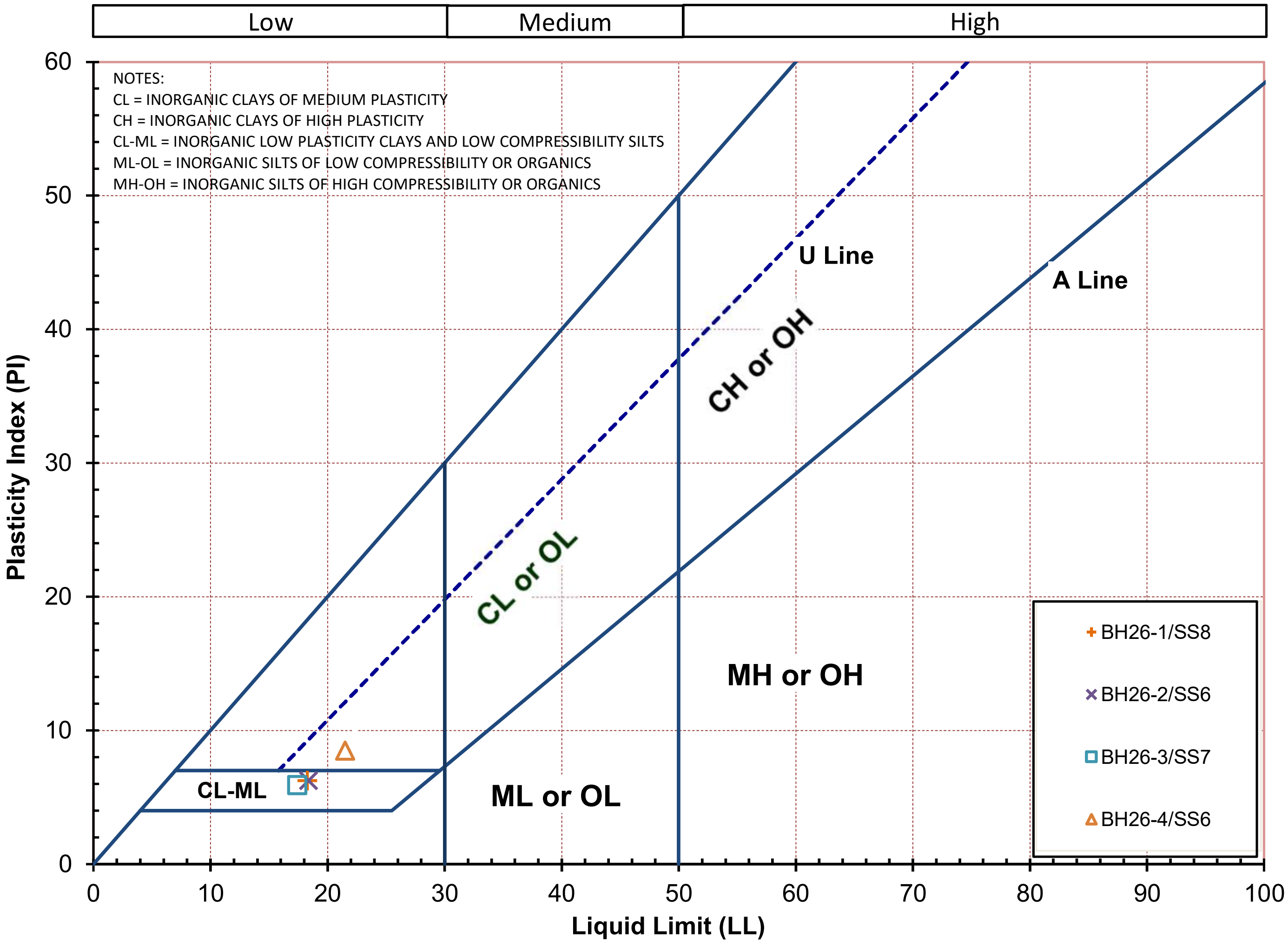
Plasticity Chart

(ASTM D4318-17)

Project No.:	243.024905.00001	Lab No.:	R26-022
Project Name:	12612 Woodbine Ave	Tested By:	BW
Client:	12612 Woodbine (KD) Corp.	Checked By:	OT
Location:	Gromley, Ontario	Date:	5/26/2026

Test Results

Test No	Sample No	LL	PL	PI	Fines	W%	Description	USCS
1	BH26-1/SS8	18	12	6				CL-ML
2	BH26-2/SS6	18	12	6				CL-ML
3	BH26-3/SS7	17	11	6				CL-ML
4	BH26-4/SS6	22	13	9				CL
5								
6								
7								
8								



Appendix C General Requirements For Engineered Fill

Preliminary Geotechnical Investigation

12612 and 12586 Woodbine Avenue, Stouffville, Ontario

Kingsdale Development

SLR Project No.: 209.065495.00001

July 9, 2026

GENERAL REQUIREMENTS FOR ENGINEERED FILL

Compacted imported soil that meets specific engineering requirements and is free of organics and debris and that has been continually monitored on a full-time basis by a qualified geotechnical representative is classified as engineered fill. Engineered fill that meets these requirements and is bearing on suitable native subsoil can be used for the support of foundations.

Imported soil used as engineered fill can be removed from other portions of a site or can be brought in from other sites. In general, most of Ontario soils are too wet to achieve the 100% Standard Proctor Maximum Dry Density (SPMDD) and will require drying and careful site management if they are to be considered for engineered fill. Imported non-cohesive granular soil is preferred for all engineered fill. For engineered fill, we recommend use of OPSS Granular 'B' sand and gravel fill material.

Adverse weather conditions such as rain make the placement of engineered fill to the required degree of density difficult or impossible; engineered fill cannot be placed during freezing conditions, i.e. normally not between December 15 and April 1 of each year.

The location of the foundations on the engineered fill pad is critical and certification by a qualified surveyor that the foundations are within the stipulated boundaries is mandatory. Since layout stakes are often damaged or removed during fill placement, offset stakes must be installed and maintained by the surveyors during the course of fill placement so that the contractor and engineering staff are continually aware of where the engineered fill limits lie. Excavations within the engineered fill pad must be backfilled with the same conditions and quality control as the original pad.

To perform satisfactorily, engineered fill requires the cooperation of the designers, engineers, contractors and all parties must be aware of the requirements. The minimum requirements are as follows, however, the geotechnical report must be reviewed for specific information and requirements.

1. Prior to site work involving engineered fill, a site meeting to discuss all aspects must be convened. The surveyor, contractor, design engineer and geotechnical engineer must attend the meeting. At this meeting, the limits of the engineered fill will be defined. The contractor must make known where all fill material will be obtained from and samples must be provided to the geotechnical engineer for review, and approval before filling begins.

2. Detailed drawings indicating the lower boundaries as well as the upper boundaries of the engineered fill must be available at the site meeting and be approved by the geotechnical engineer.

3. The building footprint and base of the pad, including basements, garages, etc. must be defined by offset stakes that remain in place until the footings and service connections are all constructed. Confirmation that the footings are within the pad, service lines are in place, and that the grade conforms to drawings, must be obtained by the owner in writing from the surveyor and Palmer. Without this confirmation no responsibility for the performance of the structure can be accepted by Palmer. Survey drawing of the pre and post fill location and elevations will also be required.

4. The area must be stripped of all topsoil and fill materials. Subgrade must be proof-rolled. Soft spots must be dug out. The stripped native subgrade must be examined and approved by an engineer prior to placement of fill.

5. The approved engineered fill material must be compacted to 100% Standard Proctor Maximum Dry Density throughout. Engineered fill should not be placed during the winter months. Engineered fill compacted to 100% SPMDD will settle under its own weight approximately 0.5% of the fill height and the structural engineer must be aware of this settlement. In addition to the settlement of the fill, additional settlement due to consolidation of the underlying soils from the structural and fill loads will occur and should be evaluated prior to placing the fill.

6. Full-time geotechnical inspection by approved geotechnical engineering personnel during placement of engineered fill is required. Work cannot commence or continue without the presence of the geotechnical engineering representative.

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

8. A bearing capacity of 100 kPa at SLS (150 kPa at ULS) can be used provided that all conditions outlined above are adhered to. A minimum footing width of 500 mm (20 inches) is suggested and footings must be provided with nominal steel reinforcement.

9. All excavations must be done in accordance with the Occupational Health and Safety Regulations of Ontario.

10. After completion of the engineered fill pad a second contractor may be selected to install footings. The prepared footing bases must be evaluated by engineering staff from geotechnical consultant prior to footing concrete placements. All excavations must be backfilled under full time supervision by approved geotechnical engineering personnel to the same degree as the engineered fill pad. Surface water cannot be allowed to pond in excavations or to be trapped in clear stone backfill. Clear stone backfill can only be used with the approval of geotechnical engineer.

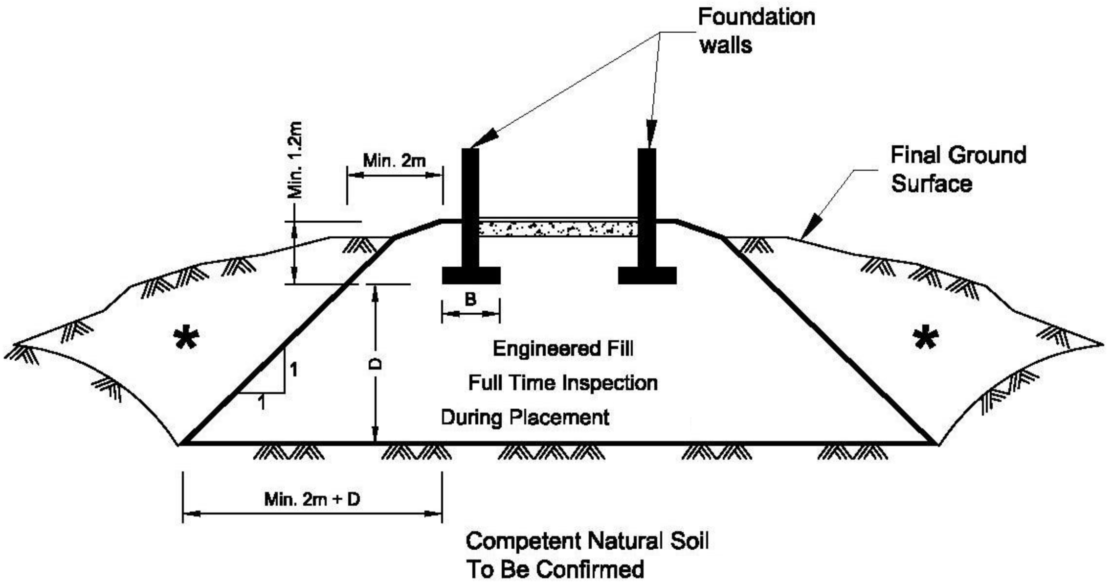
11. After completion of compaction, the surface of the engineered fill pad must be protected from disturbance from traffic, rain and frost. During the course of fill placement, the engineered fill must be smooth-graded, proof-rolled and sloped/crowned at the end of each day, prior to weekends and any stoppage in work in order to promote rapid runoff of rainwater and to avoid any ponding surface water. Any stockpiles of fill intended for use as engineered fill must also be smooth-bladed to promote runoff and/or protected from excessive moisture take up.

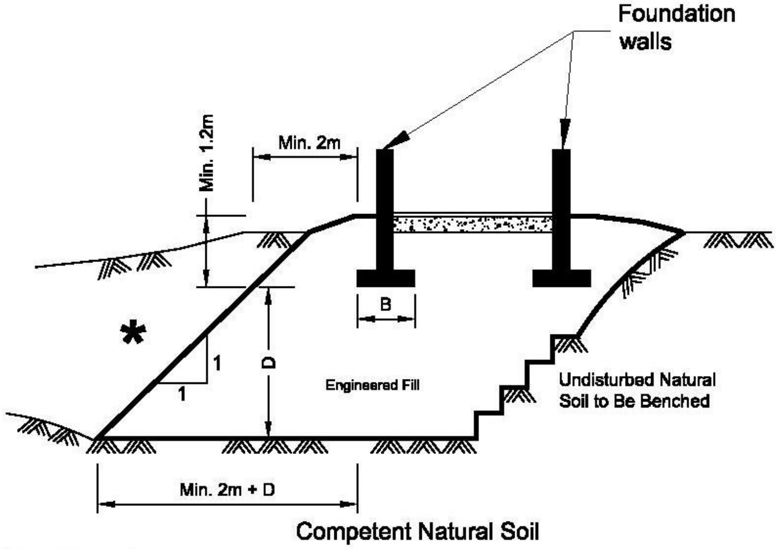
12. If there is a delay in construction, the engineered fill pad must be inspected and accepted by the geotechnical engineer. The location of the structure must be reconfirmed that it remains within the pad.

13. The geometry of the engineered fill as illustrated in these General Requirements is general in nature. Each project will have its own unique requirements. For example, if perimeter

sidewalks are to be constructed around the building, then the projection of the engineered fill beyond the foundation wall may need to be greater.

14. These guidelines are to be read in conjunction with Palmer report attached.





* Backfill in this area to be as per the report.

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