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Geotechnical Investigation
Proposed Commercial Development
2505 and 2707 Solandt Road

Ottawa, Ontario

GEMTEC Project: 104638.001

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Submitted to:

Silk Development Group Limited
40 Shields Court
Markham, Ontario
L3R 0M5

Geotechnical Investigation Proposed Commercial Development 2505 and 2707 Solandt Road

Ottawa, Ontario

January 19, 2026
GEMTEC Project: 104638.001

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1.0 INTRODUCTION

GEMTEC Consulting Engineers and Scientists Limited (GEMTEC) was retained by NOVATECH Engineers, Planners & Landscape Architects (NOVATECH) on behalf of Silk Development Group Limited (Silk) to provide geotechnical engineering services in support of the proposed commercial development to be located at 2505 and 2707 Solandt Road in Ottawa, Ontario.

The purpose of the investigation was to identify the general subsurface and groundwater conditions at the site by means of a limited number of boreholes and monitoring wells, and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

This report is subject to the Conditions and Limitations of This Report, which follows the text of the report, and which are considered an integral part of the report.

2.0 BACKGROUND

2.1 Project Description

Plans are being prepared for a proposed commercial development to be located at 2505 and 2707 Solandt Road, in Ottawa, Ontario. The following is known about the site and project:

- The overall site is approximately rectangular in shape with plan dimensions of about 320 by 130 metres;
- The site at 2707 Solandt Road is currently an undeveloped lot that is treed and 2505 Solandt Road is an asphaltic concrete surfaced parking lot; and,
- The development will consist of a main wellness spa building with several smaller buildings across the site.
    - The wellness spa building will be three storeys in height with a slab on grade construction (i.e., no basement level). The main entrance will have a ramp up to the second floor, and the first floor at the rear of the building will be a “walk-out” at ground level.
    - Based on information provided by Cunliffe & Associates (Cunliffe), NOVATECH Engineers, Planners & Landscape Architects (NOVATECH), and Simmonds Architecture (Simmonds), it is understood that the main wellness spa building will have an underside of footing elevation of about 76.6 metres with a grade raise of up to about 82.5 metres (at the main entrance at the front of the building).
    - It is understood that a retaining wall will be located on the south side of the main entrance ramp. Little is known about the retaining wall, however, it is understood that the underside of footing elevation will match (and be tied into) the footings of the wellness spa building and the retaining wall will be of cast-in-place concrete construction.

  - It is also understood that the foundation options considered for the main spa building are shallow foundations, raft slab foundation, shallow foundations on ground improvement and deep foundations.
  - There are 12 smaller buildings on site, numbered B1 to B12. Little is known about the smaller buildings, however, it is understood that they will be one storey in height and of slab on grade construction (i.e., no basement level), with the exception of building B2, which will have one basement level.

2.2 Previous Geotechnical Investigation

A previous geotechnical investigation was carried out at 2707 Solandt Road by Golder Associates (Golder). The results were provided in the following report:

- Report to KRP Properties, titled “Geotechnical Investigation, Proposed Commercial Development, 2707 Solandt Road, Ottawa, Ontario” dated September 2019 (Report No. 18111016).

As part of that investigation, six boreholes were advanced on the site to depths of about 3.7 to 9.1 metres below the existing ground surface. The subsurface conditions encountered in the boreholes generally consists of silty sand to sand over silty clay and glacial till. Auger refusal and the bedrock surface was encountered at depths of about 3.7 to 7.5 metres below the existing ground surface.

2.3 Review of Available Information and Geology Maps

Based on a review of surficial geology maps, the subsurface conditions at the site are expected to consist of organic deposits (2707 Solandt Road) and older alluvial deposits of silt and clay (2505 Solandt Road), with shallow bedrock to the northeast. Bedrock geology maps indicate the Site is underlain by dolostone of the Oxford formation. Drift thickness mapping indicates that the bedrock surface is expected at depths ranging from about 15 to 25 metres below the existing ground surface, sloping down to the southwest.

3.0 METHODOLOGY

The fieldwork for this investigation was carried out on September 18, 19, and 30, 2025. On those days, five boreholes (numbered 25-01 to 25-05, inclusive) were advanced at the approximate locations shown on the Site Plan, Figure 1 following the text of this report.

The boreholes were advanced using a truck mounted hollow stem drill rig supplied and operated by Limitless Drilling Limited of Renfrew, Ontario. The boreholes were advanced to depths ranging from about 4.6 to 7.2 metres below the existing ground surface. Shallow auger refusal was encountered in boreholes 25-01, 25-03, and 25-05 and the boreholes were advanced adjacent to the original location. Borehole 25-03 was advanced using wash boring drilling techniques.

Standard penetration tests were carried out in the boreholes at regular intervals of depth and samples of the soils encountered were recovered using a 50-millimetre diameter split barrel sampler. In situ vane testing was carried out in the boreholes to measure the undrained shear strength of the silty clay deposit.

A monitoring well was installed in each of boreholes 25-02 and 25-04 for subsequent measurement of the groundwater levels.

The fieldwork was supervised throughout by a member of our engineering staff who directed the drilling operations, observed the in-situ sampling, and logged the soil stratigraphy. The borehole locations were selected by GEMTEC personnel and positioned at the site relative to existing site features. The locations and ground surface elevations at the borehole locations were determined using a precision GPS survey instrument. The coordinates of the boreholes are referenced to NAD83 (CSRS) Epoch 2010, vertical network CGVD28.

Following the fieldwork, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content, Atterberg Limit, and grain size distribution testing. In addition, one sample of soil recovered from borehole 25-05 was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

4.0 SUBSURFACE CONDITIONS

4.1 General

Descriptions of the subsurface conditions logged in the current boreholes are provided on the Record of Borehole Sheets in Appendix A. The results of the laboratory classification testing are provided on the Record of Borehole Sheets and in Appendix B. The results of the chemical analysis (corrosivity) are provided in Appendix C. The borehole logs and laboratory testing from the previous investigation are provided in Appendix D.

The following sections provide a description of the subsurface conditions encountered in the geotechnical boreholes for the current investigation, unless noted otherwise.

4.2 Pavement Structure

The boreholes were advanced through the asphaltic concrete surface of the existing parking lot. The thickness of the asphaltic concrete surface ranges from about 40 to 80 millimetres.

Base material was encountered below the asphaltic concrete in boreholes 25-01, 25-02, 25-03, and 25-05. The base material was composed of sandy gravel, with some non-plastic fines, with thicknesses ranging from about 160 to 200 millimetres. A subbase material was encountered below the base material in boreholes 25-01, 25-02, 25-03, and 25-05, composed of gravel and sand, with some non-plastic fines. The subbase material has a thickness ranging from about

380 to 520 millimetres. In borehole 25-04, a base/subbase layer was encountered below the asphaltic concrete. The base/subbase material was composed of gravel and sand, with some non-plastic fines, with a thickness of about 450 millimetres.

Grain size distribution tests were carried out on two samples of the base and subbase material. The results are summarized in Table 4.1, below. The measured water contents of three samples of base and subbase material ranges from about 3 to 6 percent.

Table 4.1 – Summary of Grain Size Distribution Test (Base and Subbase Material)

Borehole ID	Sample Number	Gravel (%)	Sand (%)	Silt and Clay (%)
25-01	2	54	36	10
25-04	1	50	39	11

4.3 Fill Material

A layer of fill material was encountered below the pavement structure in the boreholes and extends to depths ranging from about 2.4 to 3.8 metres below the existing ground surface. The fill material is highly variable, ranging in composition from fine to coarse grained soils. The fill material also contains organics, cobbles, and boulders. Refer to the borehole logs for further details.

Shallow auger refusal was encountered in boreholes 25-01, 25-03, and 25-05, with multiple attempts made to advance the boreholes. A summary of the auger refusal depths that were encountered in the fill material are provided in Table 4.2, below.

Table 4.2 – Summary of Shallow Auger Refusal Depths and Elevations

Borehole ID	Ground Surface Elevation (metres)	Refusal Depth (metres)	Refusal Elevation (metres)
25-01	77.5	1.2 to 2.6 (4 attempts)	76.3 to 74.9
25-03	77.2	1.5	75.7
25-05	77.6	1.1	76.5

Standard penetration tests carried out in the fill material gave N values ranging from 2 to greater than 50 blows per less than 0.3 metres of penetration, which reflects a very loose to very dense relative density. The higher N values may also be caused by the presence of cobbles, boulders or other hard material within the fill.

Grain size distribution tests were carried out on one sample of the fill material. The results are summarized in Table 4.3, below. The measured water contents of 12 samples of fill material ranges from about 3 to 41 percent.

Table 4.3 – Summary of Grain Size Distribution Test (Fill Material)

Borehole ID	Sample Number	Gravel (%)	Sand (%)	Silt and Clay (%)
25-03	2	33	31	37

4.4 Clay to Clayey Silt

A native deposit of silty clay to clayey silt, with organics was encountered below the fill material in boreholes 25-01, 25-03, and 25-04. The silty clay to clayey silt with organics has a thickness ranging from about 0.3 to 0.8 metres and extends to depths ranging from about 3.4 to 4.1 metres below the existing ground surface.

The measured water contents of three samples of the silty clay with organics ranges from about 25 to 42 percent.

4.5 Clay to Silty Clay

A native deposit of clay to silty clay exists below the fill material and/or silty clay to clayey silt with organics. In borehole 25-02, a probable layer of silty clay was encountered below the fill material. The clay to silty clay extends to depths ranging from about 4.6 to 7.2 metres below the existing ground surface.

The upper portion of the clay in borehole 25-05 has been weathered to a grey brown crust. The weathered crust has a thickness of about 1.9 metres and extends to a depth of about 4.3 metres below the existing ground surface.

Standard penetration tests carried out in the weathered clay gave N values ranging from 2 to 8 blows per 0.3 metres of penetration, which based on our experience in the Eastern Ontario region, reflects a stiff to very stiff consistency.

Atterberg limit testing was carried out on one sample of the weathered clay crust. The results are summarized in Table 4.4. The measured water contents of three samples of the weathered clay ranges from about 35 to 52 percent.

Table 4.4 – Summary of Atterberg Limit Test (Weathered Crust)

Borehole / Sample No.	Water Content (%)	Liquid Limits (%)	Plastic Limits (%)	Plasticity Index
25-05 / 6	35	45	23	22

The clay below the depth of weathering in borehole 25-05 and the full depth of the clay to silty clay in boreholes 25-01, 25-03, and 25-04 is unweathered and grey in colour. The grey clayey soils extend to depths ranging from about 5.6 to 7.2 metres below the existing ground surface.

Standard penetration tests carried out in the unweathered clay to silty clay gave N values ranging from weight of hammer (WH) to 2 blows per 0.3 metres of penetration. In situ shear vane testing gave undrained shear strengths ranging from about 45 to greater than 100 kilopascals, which reflects a firm to very stiff consistency.

Atterberg limit testing was carried out on four samples of the grey silty clay. The results are summarized in Table 4.5. The measured water content of 12 samples of the silty clay ranges from about 11 to 59 percent.

Table 4.5 – Summary of Atterberg Limit Test (Clay)

Borehole / Sample No.	Water Content (%)	Liquid Limits (%)	Plastic Limits (%)	Plasticity Index
25-01 / 8	48	39	21	18
25-03 / 6	60	41	20	21
25-03 / 7	44	34	17	16
25-04 / 6	47	43	18	25

The results of shrinkage limit testing on one sample of the silty clay from borehole 25-03 is about 15 percent.

4.6 Glacial Till

A native deposit of glacial till was encountered below silty clay in boreholes 25-03, 25-04, and 25-05. The glacial till extends to depths ranging from about 5.9 to 7.1 metres below the existing ground surface. The glacial till is a heterogeneous mixture of all grain sizes, which at this site, can be described as a gravelly silty sand with trace clay, and clayey silt with trace gravel and sand.

The glacial till deposit is known to contain cobbles and boulders.

Standard penetration tests carried out in the glacial till gave N values of 2 and greater than 50 blows per less than 0.3 metres of penetration, which reflects a very loose to very dense relative density. The higher N values may also be caused by the presence of cobbles or boulders within the glacial till, or the bedrock surface.

Grain size distribution testing was carried out on one sample of the glacial till. The results are summarized in Table 4.6, below. The measured water content of three samples of the glacial till ranges from about 13 to 24 percent.

Table 4.6 – Summary of Grain Size Distribution Test (Glacial Till)

Borehole / Sample No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
24-03 / 7	13	56	23	8

4.7 Auger and Casing Refusal

Refusal to auger or casing advancement occurred in all the boreholes at depths ranging from about 4.6 to 7.2 metres below the existing ground surface.

Practical auger refusal was encountered in boreholes advanced during the previous investigation at depths ranging from about 3.7 to 7.5 metres below the existing ground surface. The bedrock surface was proven in boreholes 18-102, 18-103, and 18-104 at depths ranging from about 4.9 to 7.5 metres below the existing ground surface.

Table 4.7 summarizes the depth of refusal and corresponding elevations at the borehole locations.

Table 4.7 – Refusal and Bedrock Surface Summary

Borehole ID	Ground Surface Elevation (metres)	Depth to Auger Refusal (metres)	Depth to Bedrock (metres)	Auger Refusal / Bedrock Surface Elevation (metres)
25-01	77.5	7.2	n/a	70.3

Borehole ID	Ground Surface Elevation (metres)	Depth to Auger Refusal (metres)	Depth to Bedrock (metres)	Auger Refusal / Bedrock Surface Elevation (metres)
25-02	77.6	4.6	n/a	73.0
25-03	77.2	6.9	n/a	70.3
25-04	77.5	5.9	n/a	71.6
25-05	77.6	7.1	n/a	70.5
18-101	77.4	4.9	n/a	72.5
18-102	77.2	6.2	6.2	71.0
18-103	77.2	7.5	7.5	69.7
18-104	77.4	4.9	4.9	72.5
18-105	77.0	3.7	n/a	73.3
18-106	77.2	5.8	n/a	71.4

4.8 Bedrock

Bedrock coring was carried out in the previous investigation where sandstone bedrock was proven in boreholes 18-102, 18-103, 18-104, at depths ranging from about 4.9 to 7.5 metres (i.e., elevations ranging from about 72.5 to 69.7 metres) below the existing ground surface. The sandstone bedrock was cored to depths ranging from about 6.5 to 9.1 metres below the existing ground surface.

The recovered bedrock core samples had rock quality designation (RQD) values ranging from about 97 to 100 percent. Based on these values, in accordance with the classification system set out in the Canadian Foundation Engineering Manual (5th Edition) the bedrock can be classified as Excellent Quality.

The result of unconfined compressive strength testing carried out on one sample of recovered bedrock core from borehole 18-103 is about 183 megapascals, resulting in a rock strength classification of very strong. The rock strength classification in the Canadian Foundation Engineering Manual (5th Edition) has been applied.

4.9 Groundwater

The groundwater level in the monitoring wells in boreholes 25-02 and 25-04 were measured on October 21, 2025, and are presented in Table 4.5. The groundwater levels measured during the previous investigation are also summarized in Table 4.8, below.

The groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

Table 4.8 – Groundwater Level Depths and Elevations

Borehole ID	Ground Surface Elevation (metres)	Groundwater Depth (metres)	Groundwater Elevation (metres)	Date of Reading
25-01	77.6	2.8	74.8	October 21, 2025
25-04	77.5	2.9	74.6	October 21, 2025
18-102	77.2	1.6	75.7	November 16, 2018
18-105	77.0	1.8	75.3	November 16, 2018
18-106	77.2	2.2	75.0	November 16, 2018

4.10 Soil Chemistry Relating to Corrosion

The results of chemical testing on a soil sample recovered from borehole 25-05 are provided in Appendix D and are summarized in Table 4.9, below.

Table 4.9 – Summary of Corrosion Testing

Parameter	Borehole 25-05 Sample No. 6
Chloride Content (µg/g)	48
Resistivity (Ohm.m)	27.4
pH	7.13
Sulphate Content (µg/g)	170

5.0 GUIDELINES AND RECOMMENDATIONS

5.1 General

At the time of preparing this report, only limited, conceptual information was available to GEMTEC. The recommendations provided in the following sections may require review as the design of the project progresses and further details are made available to GEMTEC.

5.2 Site Grade Raise and Foundations Options (2505 Solandt Road)

5.2.1 General

This site is underlain by up to 4 metres of fill material over a deposit of silty clay, which has a reduced capacity to support loads imposed by additional grade raise fill material and foundations for the buildings.

The foundation loading and placement of fill material at the proposed main wellness spa building and smaller buildings must therefore be carefully planned and controlled so that the stress imposed by the fill material and foundations do not result in excessive consolidation of the silty clay deposit.

Concrete slabs, granular fill materials, overall grade raise, and pavement structures are considered grade raise filling. Groundwater lowering also results in a stress increase on the underlying silty clay deposit.

5.2.2 Wellness Spa

The following foundation, grade raise and finished floor elevations are understood for the proposed main wellness spa building, based on information provided by NOVATECH and Cunliffe:

- Approximate underside of footing elevation: 76.6 metres;
- Lowest finished floor elevation: 78.3 metres;
- Existing ground surface elevation range at borehole locations: 77.2 to 77.6 metres; and,
- Maximum finished grade elevation at ramp: 82.5 metres.

The results of the investigation and laboratory testing, in conjunction with empirical calculations correlating the undrained shear strength of the silty clay to the preconsolidation pressure, indicate that the loading from the proposed grade raise fill and the design foundation loading will exceed the capacity of the silty clay. This would result in higher than tolerable settlements for shallow foundations and the concrete slab on grade.

Therefore, the use of a raft slab foundation and/or shallow strip and pad footings founded on a pad of engineered fill on the native silty clay deposits are not considered feasible and will not be discussed further.

The following options may be considered for the design and construction of the proposed wellness spa foundations:

- Intrusive ground improvement (e.g., controlled modulus columns); or,
- Deep foundations (e.g., drilled micro-piles with rock anchors) to support the foundations.

Intrusive ground improvement installed by a specialty contractor, is feasible for this site and the intrusive elements (e.g., controlled modulus columns) could provide adequate support for shallow foundations, the concrete slab on grade floor and the grade raise fill. A key advantage of this type of ground improvement for site preparation is that the work is carried out in advance of the construction on the site and no special techniques or treatments are required for the remainder of the construction.

An additional advantage of ground improvement would be the existing fill material can remain in place, however, with the presence of obstructions in the existing fill material, pre-drilling through the fill material will likely be required.

Deep foundations for support of the building may also be considered. Based on discussions with Cunliffe, the deep foundations would likely consist of small diameter, steel pipe piles drilled into the underlying bedrock with rock anchors grouted into the bedrock. For the deep foundations option, the existing fill material will need to be removed in order to support a slab on grade. Alternatively, a structural floor slab can be considered, however, it is understood that a structural floor slab would likely not be feasible due to the high cost.

5.2.2.1 Shallow Foundations on Improved Ground

Ground improvement using Controlled Modulus Columns (CMC’s), or similar intrusive elements, is the stabilization of soils to increase their bearing capacity, provide stability, control seepage, and provide liquefaction resistance.

Shallow foundations may be supported on improved ground after installation of CMC’s, or similar. Generally, the building area after improvement consists of a granular load distribution pad which has been placed over the installed CMC’s. Shallow strip or spread foundations may be placed directly on the granular load distribution pad. The achievable bearing resistances should be confirmed by the selected specialty contractor carrying out the ground improvement but bearing resistances at serviceability limit states (SLS) and ultimate limit states (ULS) of 200 kilopascals and 350 kilopascals, respectively, should be achievable.

5.2.2.2 Deep Foundations

Deep foundations (e.g., drilled micro-piles) may be used to transfer the foundation loads to the rock at depth below the compressible silty clay.

If deep foundations are considered, the existing fill material will need to be removed and replaced with compacted engineered fill, as discussed in Section 5.8.

Based on the rock type expected at this site, the geotechnical resistance of piles will exceed the structural capacity of the piles and the structural capacity will therefore govern. However, if PDA testing of the piles is considered, and it would be advisable on a selected number of piles, the geotechnical capacity of the pile that can be verified with PDA testing is limited to 0.5 of the structural capacity of the pile.

It should be noted that the top of bedrock was not confirmed at all borehole locations and therefore pile refusal depths may vary somewhat across the site.

Further guidance can be provided if this option is preferred and the pile types and sizes are provided based on further design.

5.2.2.3 Rock Anchors

The following provides preliminary guidelines on grouted rock anchors.

The design of the rock anchors should consider the following failure modes:

- Failure within the rock mass, or rock cone pull-out;
- Failure of the rock/grout bond;
- Failure of the grout/tendon bond;
- Failure of the steel tendon or top anchorage.

Of the failure modes identified above, failure of the tendon and grout bond, and failure of the tendon or top anchorage should be checked by a structural engineer.

Anchor resistance, (Qr) for a single anchor against failure within the rock mass can be determined from the equation for the volume of a cone, according to a 60 degree cone apex angle, with apex located at the mid-point of the fixed length section. The equation for anchor resistance for failure within the rock mass is provided below, neglecting shear resistance generated along the cone surface:

$$Q_r = Ø*0.33*\pi*\gamma' D^3 Tan^2\theta$$

Where:

γ' = Buoyant unit weight of rock: 16 kilonewtons per cubic metre (conservative value)
Ø = Resistance factor to be applied
D = cone height (anchor midpoint)
θ = Half the value of the apex angle.

Where loads are off vertical the capacity of the anchor should be modified according to the angle of application.

Group effects should be considered in assessing anchor capacity where overlapping occurs between adjacent cones. For this case, the volume of a truncated trapezoidal failure zone should be considered. However, for preliminary design purposes we suggest anchors be no closely spaced that about 1.5 metres to reduce the potential for drillholes to intersect and avoid overstressed areas of bedrock.

For failure of the grout/rock bond the unfactored ULS bond strength at concrete to rock interface pull out use a value of 1,000 kilopascals (assuming a resistance factor of 0.4 is applied). This value assumes that the fixed anchor length is in sound rock. To achieve the bond strength the surface of the rock bores should be rough and all debris and rock flour should be cleared from the bore or the anchor capacity shall be reduced as a result. The required bonded length should be determined according to the factored tensile resistance to be carried.

Long bonded anchor lengths should be avoided (i.e., max 8 metres). SLS movement in the anchor can be determined from the elastic elongation of the unbonded portion of the tendon under design load.

The use of a specialist rock anchor contractor is recommended for installation of the anchors. The installation and testing of rock anchors shall be observed by a suitably qualified and experienced geotechnical practitioner.

Further details can be provided as the design progresses and the positioning of anchors (if required) are established.

5.2.3 Buildings B1, B4, and B12

The following is understood about buildings B1, B4, and B12:

- The buildings will be one storey in height;
- The buildings will be of slab on grade construction (i.e., no basement level);
- The grade raise around the buildings will be up to about 0.5 metres above existing (i.e., to an elevation of about 78.0 metres); and,
- The finished floor elevation for the buildings will be about 78.0 metres, with an assumed underside of footing elevation of no less than about 76.5 metres (i.e., about 1.5 metres below finished grade).

The following options may be considered for the design and construction of the proposed smaller building foundations:

- Intrusive ground improvement (e.g., controlled modulus columns); or,

- Shallow foundations on engineered fill to the native silty clay deposit (i.e., removal and replacement of the existing fill material).

Recommendations for intrusive ground improvement can be taken as per Section 5.2.2.1.

For shallow spread footing foundations, the existing fill material should be removed to expose the native, undisturbed silty clay. The grade can then be raised with compacted granular material (engineered fill) with a Class II non-woven geotextile placed on the subgrade. The engineered fill should consist of granular material meeting OPSS requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material's standard Proctor maximum dry density using suitably sized vibratory compaction equipment. To provide adequate spread of load beneath the footings, the engineered fill should extend horizontally at least 0.5 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter. The excavations for these buildings should be sized to accommodate the placement of the engineered fill.

For design purposes, footings bearing on the native, undisturbed native soils, or on a pad of engineered fill above native, undisturbed native soils should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 200 kilopascals.

The post construction total and differential settlement of the footings at SLS should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

5.2.4 Entrance Ramp Grade Filling and Retaining Wall

5.2.4.1 Grade Filling

Based on the results of the subsurface investigation, the maximum thickness of any grade raise filling at this site should be limited to about 3.0 metres above the existing ground surface (i.e., to a maximum elevation of about 80.5 metres). The grade raise restriction for the entrance ramp has been calculated in order to limit the total settlement of the ground to about 25 millimetres in the long term.

For any area where the final grade is above the 3.0 metres (80.5 metres elevation), the thickness of filling above that limit may be achieved with lightweight fill consisting of expanded polystyrene (EPS) blocks, or supporting the full grade raise fill with ground improvement (as per Section 5.2.2.1).

5.2.4.2 Retaining wall

It is understood that a retaining wall will be constructed along the south side of the main entrance ramp for the wellness spa. The following is known about the retaining wall:

- The underside of footing elevation will be at about 76.6 metres, and will be tied into the proposed footing of the wellness spa;
- The retaining wall will extend up to an elevation of about 82.5 metres; and,
- The retaining wall will be of cast-in-place concrete construction.

Since the retaining wall will be tied into the foundations of the main spa building, the proposed retaining wall foundation support should match the foundation support of the building (as per Section 5.2.2).

The retaining wall can therefore be supported on a pad of engineered fill on intrusive ground improvement (e.g., CMC's), however, this assumes that the ground improvement design can accommodate the additional grade raise fill loading along the foundation. This should be coordinated with the ground improvement designers.

Alternatively, the cast in place retaining wall, can be supported on pile foundations if that is the preferred option for the main building.

For any area where the final grade is above 3.0 metres (about 80.5 metres elevation), additional measures will be required to manage the imposed stress and limit settlement. To limit the ground improvement or deep foundation requirements, the thickness of filling above that limit may be achieved with lightweight fill consisting of expanded polystyrene (EPS) blocks. Alternatively, cellular concrete, with a unit weight of about 10 kilonewtons per cubic metre could be considered for backfill between/along the retaining wall. Further guidance, based on the design details, will be required if lightweight fills are preferred.

5.3 Site Grade Raise and Foundation Design (2707 Solandt Road)

The subsurface conditions at this site consist of a deposit of silty sand and weathered silty clay crust over stiff silty clay, which has a reduced capacity to support loads imposed by additional grade raise fill material and foundations for the buildings.

The foundation loading and placement of fill material at the proposed buildings must therefore be carefully planned and controlled so that the stress imposed by the fill material and foundations do not result in excessive consolidation of the silty clay deposit.

Concrete slabs, granular fill materials, overall grade raise, and pavement structures are considered grade raise filling. Groundwater lowering also results in a stress increase on the underlying silty clay deposit.

The following is understood about the smaller buildings at this site (B2, B3, and B5 to B11):

- The buildings will be one storey in height;

- The buildings will be of slab on grade construction (i.e., no basement level), except for building B2, which will have one basement level;
- The grade raise around the buildings will be up to about 1.0 metre above existing (i.e., to elevation of about 78.0 metres); and,
- The finished floor elevations will be about 77.7 to 77.8 metres, with assumed underside of footing elevations at no less than about 75.2 metres (i.e., about 2.5 metres below finished grade).

Based on the results of the previous investigation, the proposed buildings can be founded on footings bearing on or within the native undisturbed weathered silty clay crust deposits. The topsoil and fill material, if encountered, are considered to be highly compressible and should be removed from below any foundations and slabs on grade.

Where the subsequent subgrade surface is below the proposed founding level, the grade could be raised with compacted granular material (engineered fill) with a Class II non-woven geotextile placed on the subgrade. The engineered fill should consist of granular material meeting OPSS requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. To provide adequate spread of load beneath the footings, the engineered fill should extend horizontally at least 0.5 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter.

For design purposes, footings bearing on the native, undisturbed native soils, or on a pad of engineered fill above native, undisturbed native soils should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 200 kilopascals.

The post construction total and differential settlement of the footings at SLS should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

To reduce the potential for cracking in the footings, foundation walls, and concrete slab on grade where the footings transition between different subgrade materials, the foundation walls should be reinforced for a distance of 3 metres on both sides of the transition areas or as recommended by the structural engineer.

5.4 Excavation

5.4.1 Overburden Excavations

The excavations for the proposed buildings will be carried out through the existing fill material and silty sand, where encountered, and into the native silty clay deposit. The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the

Occupational Health and Safety Act. According to the Act, the overburden soils at this site can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, above the groundwater level. An allowance should be made for excavation side slopes of 3 horizontal to 1 vertical, or flatter, below the groundwater level.

Cobbles, boulders, rockfill and possibly construction debris, should be anticipated in the fill material, which may lead to increased excavation effort and slower progress. As such, an allowance should be made for removal of boulder sized particles from the fill material during excavation which may require the use of larger excavation plant.

The silty clay deposit is sensitive to disturbance from ponded water, vibration, and construction traffic. As such, it is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket. Allowance should be made to remove and replace any disturbed silty clay with compacted sand and gravel, such as that meeting Ontario Provincial Standards Specification (OPSS) Granular A or Granular B Type II, where required.

5.4.2 Temporary Shoring

Where open cut excavations are not feasible, the sides of the excavations could be supported vertically using temporary shoring.

The shoring should be designed, installed, and monitored in accordance with Ontario Provincial Standard Specification (OPSS) 539. The following comments are provided on the selection and design of the shoring system:

- Different shoring methods will have differing stiffness and ability to resist ground movements. Also, some forms of shoring can be incorporated into the permanent works component of the structure (for instance secant piling or diaphragm walls).
- The selection of the type of temporary shoring system, and the method of lateral restraint, should be entirely the choice/responsibility of the contractor; however, it is expected that shoring can be achieved using conventional techniques (e.g., sheet piling, pile and lagging walls etc.). Some form of lateral support to the shoring will be required. Interior whalers and struts are likely the most practical options; however, during excavation for the proposed wellness spa building, the shoring required to support the existing ground surface may require tie-backs (e.g., bedrock anchors).
- The shoring system should be designed to resist lateral earth pressures imposed on the shoring from the weight of the retained soil and any other surcharge loads. The design should also consider soil stratigraphy, the groundwater conditions, the methods adopted to manage the groundwater, the permissible ground movements associated with the excavation and construction of the shoring system, and potential impacts on adjacent structures and utilities.
- The lateral earth pressures acting on the shoring system will depend on the type of shoring system used and on the type of lateral support. The selection of the lateral earth pressures

should be the responsibility of the contractor, who will also be responsible for the overall shoring design. The contactor should be required to submit the shoring system design prior to the start of shoring construction, including details on the design lateral earth pressures, expected movements, and a monitoring plan, for review prior to the start of shoring construction.

In areas where vertical support of the excavation for the proposed wellness spa is required, it is recommended that the temporary shoring should be constructed using driven interlocking steel sheet piling or other suitable systems compatible with the permissible levels of ground movement. Sheet piling is preferred given that it will reduce the amount of groundwater inflow from the overburden into the excavations.

It should be noted that the fill material and glacial till contains cobble and boulder size obstructions which could affect the shoring installation. Therefore, should sheet pile systems be used hard driving conditions should be expected and some of the sheet piles will likely terminate within the overburden on cobbles and boulder obstructions. For deeper excavations or if it is required that the shoring system should extend to the surface of the bedrock sheet piles may not be suitable and other systems such as pile and lagging walls may have to be considered in combination with predrilling.

5.5 Groundwater Management

5.5.1 Temporary Excavation Dewatering

Groundwater inflow into the excavations from the fill material and native silty clay deposits can likely be managed using typical construction dewatering techniques. Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

It is not expected that short term pumping during excavation will have a significant effect on nearby structures.

5.5.2 Water Taking Permitting and Approvals

The type of water taking permit that is required is dependant on the anticipated groundwater inflow volumes during construction.

As part of the recent changes to *Ontario Regulation 387/04* under the *Ontario Water Resources Act* (effective July 1, 2025) all groundwater takings over 50,000 litres per day for construction dewatering will be subject to Environmental Activity and Sector Registry (EASR). A Category 3 Permit To Take Water (PTTW) is no longer required for groundwater takings over 400,000 litres per day for construction dewatering.

A precautionary EASR registration is recommended to avoid potential delays during construction, if groundwater inflows exceed 50,000 litres per day. EASR registration must be supported by a Water Taking and Discharge Plan report prepared by a Qualified Professional. A more accurate assessment of potential groundwater inflows could be carried out by GEMTEC, upon request, as the design progresses.

5.6 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

If the foundation and/or slab on grade are insulated in a manner that will reduce heat flow to the surrounding soil, the foundation depth shall conform to that required for foundations for an unheated space.

5.7 Seismic Design of Proposed Structures

Based on the results of the investigation, it is anticipated that the proposed foundations will be supported on a deposit of stiff to very stiff weathered silty clay crust or a pad of engineered fill constructed on the weathered crust. As such, in our opinion, the proposed commercial development should be designed for seismic Site Class C (Site Designation X_C).

There is no potential for liquefaction of the overburden deposits at this site.

5.8 Foundation Wall Backfill

The existing fill material and native deposits at this site are frost susceptible and should not be used as backfill against foundations. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting the requirements of OPSS Granular A, or Granular B Type I or II.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Light walk behind compaction equipment should be used next to the foundation walls to avoid excessive compaction induced stress on the foundation walls.

Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value. Where areas of hard surfacing (concrete, sidewalks, pavement, etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible fill material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.5 metres below finished grade to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

The frost susceptible native soils could be considered for foundation wall backfill purposes in landscaped areas provided that a suitable bond break is applied to the surface of the foundations to prevent frost jacking. A suitable bond break could consist of at least 2 layers of 6 MIL polyethylene sheeting or a proprietary plastic drainage medium. It is also pointed out that the native soils at this site can be impacted by changes in moisture content and this could affect the ability to compact this material to the required density.

5.9 Foundation Drainage

For the main wellness spa building and building B2 (i.e., the buildings with a below ground floor slab), a perforated plastic foundation drain with a surround of clear crushed stone should be installed along the exterior of the foundation walls. A nonwoven geotextile should be placed between the top of the clear stone and any sandy foundation wall backfill material to avoid loss of sand backfill into the voids in the clear stone (and possible post construction settlement of the ground around the building). The top of the drain should be located below the bottom of the floor slab. The drain should outlet to a sump from which the water is pumped or should drain by gravity to a storm sewer or other suitable outlet.

Perimeter foundation drainage is not considered necessary for slab on grade buildings, with no basement level (buildings B1 and B3 to B12), provided that the finished floor level is above the finished exterior ground surface level.

5.10 Lateral Earth Pressures

The following earth pressure parameters could be used for rigid foundation walls and retaining walls.

The static at rest thrust (P_o) acting on the wall should be calculated using the following formula:

$P_o = 0.5\ K_o\ \gamma\ H^2$

where;

- P_o: Static at rest thrust component (kilonewtons per metre);
- γ: Moist material unit weight (kilonewtons per cubic metre);
- K_o: “At Rest” earth pressure coefficient;
- H: Wall height (metres).

The total “At Rest” thrust acting on the walls (P_{oe}) during a seismic event is composed of a static component (P_o) and a dynamic component (P_e), that is:

$P_{oe} = P_o + P_e$

The dynamic thrust component (P_e), which acts only during seismic loading conditions, should be calculated using the following formula:

$P_e = 0.5 (K_{ae} – K_a) \gamma H^2$

where;

- P_e: Dynamic thrust (kilonewtons per metre)
- γ: Moist material unit weight (kilonewtons per cubic metre)
- K_a “Active” Earth Pressure Coefficient
- K_{ae}: Dynamic earth pressure coefficient
- H: Wall height (metres)

The static thrust component (P_o) acts at a point located H/3 above the base of the wall. During seismic shaking, the dynamic at rest thrust component (P_e) acts at a point located about 0.6H above the base of the wall.

For design purposes, the soil parameters provided in Table 5.1 can be used to calculate the at rest thrust components acting on the wall.

Table 5.1 – Summary of Soil Parameters for At Rest Wall

Parameter	OPSS Granular B Type I	OPSS Granular B Type II
Material Unit Weight, γ (kN/m^3)	21	22
Internal Friction Angle (degrees)	34	38
“At Rest” Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.44[1]	0.38[1]
Active Earth Pressure Coefficient, K_a, assuming horizontal backfill behind the structure	0.28	0.24
Dynamic Earth Pressure Coefficient, K_{ae}, assuming horizontal backfill behind the structure	0.52[1]	0.45[1]

Notes:

1) According to the 2020 National Building Code, the peak ground acceleration (PGA) for the site is 0.33 g for firm ground conditions (i.e., for Site Class C). The dynamic at rest earth pressure coefficient was calculated

using the method suggested by Mononobe and Okabe, assuming a horizontal seismic coefficient, k_h, of 0.33 (taken as the PGA for Site Class C) and assuming that the vertical seismic coefficient, k_v, is 0.

5.11 Slab on Grade Support

As discussed above, the proposed main wellness spa building will be located within the footprint of the existing fill material within the existing parking lot.

The fill material is not considered suitable for support of the slab on grade. To prevent long term settlement of the floor slab, all fill material should be removed from below the proposed slab to expose the native silty clay deposits, unless additional support is provided with ground improvement.

The grade within the proposed building could be raised, where necessary, with material meeting OPSS requirements for Granular A and Granular B Type I or II. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A. To provide adequate spread of load beneath the slab on grade, the engineered fill should extend horizontally at least 0.5 metres beyond the building footprint and then down and out from this point at 1 horizontal to 1 vertical, or flatter.

OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slab be composed of virgin material only, for environmental reasons.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value.

Underfloor drainage is not considered necessary provided that the floor slab levels are above the finished exterior ground surface level. If any areas of the buildings are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

The floor slabs should be wet cured to minimize shrinkage cracking and slab curling. The slab should be saw cut to about 1/3 the thickness of the slab as soon as curing of the concrete permits, in order to minimize shrinkage cracks.

Proper moisture protection with a vapour retarder should be used for floor slabs where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The “Guide for Concrete Floor and Slab Construction”, ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slabs.

5.12 Basement Floor Slab

The base of the basement floor slab should consist of at least 200 millimetres of 19 millimetre clear crushed stone.

To provide predictable settlement performance of the basement slab, all fill material, loose soil, or deleterious material should be removed from the slab area. Any necessary grade raise fill should consist of either 19 millimetre clear crushed stone or OPSS Granular B Type II. Where the subsequent subgrade surface is below the proposed underside of slab level, the grade could be raised with compacted granular material (engineered fill) with a Class II non-woven geotextile placed on the subgrade.

The clear crushed stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor. The Granular B Type II should be compacted in maximum 150 millimetre thick lifts to at least 95 percent of the material's standard Proctor maximum dry density value using suitable vibratory compaction equipment.

OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular B Type II material. Since the source of recycled material cannot be determined or controlled, it is suggested that any imported Granular B Type II materials be composed of 100 percent crushed rock only.

Underfloor drainage should be provided below the basement floor slab. If well graded granular material (such as OPSS Granular B Type II) is used below the basement floor slab, we suggest that drainage be provided by means of plastic perforated pipes spaced at about 6 metres horizontally or as required to link any hydraulically isolated areas in the basement. If clear crushed stone is used below the basement floor slab, drains are not considered essential provided that the clear stone can outlet to the sump and drains are installed to link any hydraulically isolated areas in the basement. The drains should outlet by gravity to a storm sewer.

The floor slab should be wet cured to minimize shrinkage cracking and slab curling. The slab should be saw cut to about 1/3 the thickness of the slab as soon as curing of the concrete permits, in order to minimize shrinkage cracks.

Proper moisture protection with a vapour retarder should be used for any slab on grade where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The "Guide for Concrete Floor and Slab Construction", ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slab.

5.13 Proposed Services

Information on the proposed services/underground utilities were not available at the time of preparing this report. As such, relatively generic guidelines are provided. More tailored guidelines can be provided as further information becomes available.

5.13.1 Excavation

Refer to Section 5.3 for general commentary on excavation. As an alternative or where space constraints dictate, the service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

5.13.2 Pipe Bedding and Cover

The bedding for service pipes should consist of at least 150 millimetres of crushed stone meeting OPSS requirements for Granular A. Cover material, from spring line to at least 300 millimetres above the tops of the pipes, should consist of granular material, such as that meeting OPSS Granular A.

In areas where the subsoil is disturbed, or where unsuitable material exists below the pipe subgrade level, the disturbed or unsuitable material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular B Type II.

The subbedding, bedding, and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density value using suitable vibratory compaction equipment.

5.13.3 Trench Backfill

The backfill materials within the zone of seasonal frost penetration (i.e., 1.8 metres below finished grade) should match the materials exposed on the trench walls below external paved areas. This will reduce the potential for differential frost heaving between the area over the trench and the pavement. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material, imported granular material conforming to OPSS Granular B Type I or II, or imported OPSS Select Subgrade Material.

To minimize future settlement of the backfill and achieve an acceptable subgrade for any roadways, curbs, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density value using suitable vibratory compaction equipment. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures, provided that some settlement above the trench is acceptable.

5.14 Sensitive Marine Clay – Effects on Trees

The site is underlain by silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations, or hard surfaced areas. Therefore, deciduous tree planting should be carried in accordance with the guidelines identified in the City of Ottawa document titled: “Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines”.

The City of Ottawa Tree Planting Guidelines indicates that sensitive marine clay soils with a modified plasticity index of less than 40 percent are considered to have a low/medium potential for soil volume change. Clay soils with a modified plasticity index that exceeds 40 percent are considered to have a high potential for soil volume change.

The modified plasticity index of nine samples tested ranges from about 16 to 38 percent (assuming 100 percent of the silty clay passes the 425 micrometre sieve). As such, the potential for soil volume change, as defined by the City of Ottawa, is low/medium in areas where clay soils were encountered at this site.

It should be noted that the City of Ottawa tree planting guidelines references setback restrictions for trees planted on the City of Ottawa road right of ways in residential developments, and not for privately owned trees on commercial properties. In GEMTEC’s opinion, the weathered silty clay crust at this site, which the footings are expected to be founded on, has a low potential for volume change, and therefore, the tree planting guidelines do not apply to this site.

However, in accordance with the City of Ottawa Tree Planting Guidelines, consideration should be given to planting small to medium sized trees (up to 14 metres in height) at least 4.5 metres from foundations, and large trees (greater than 14 metres in height) should be planted at least the height of the tree from foundations. Refer to the City of Ottawa document titled: “Tree Planting in Sensitive Marine Soils - 2017 Guidelines” for additional guidelines.

5.15 Pavement Design

Information on the pavement layout, zone of bulk excavation, and traffic loading levels are not available at the time of preparing this report. It is understood that the traffic loading will consist of typical commercial building traffic (i.e., light vehicles, delivery vehicles, fire trucks, garbage trucks, etc.).

5.15.1 Subgrade Preparation

In preparation for the construction of roadways at this site, all surficial topsoil, and any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface.

This need not include removal of the existing fill material provided that some post construction settlement of the roadways can be tolerated.

Any subexcavated areas could be filled with compacted earth borrow. Similarly, should it be necessary to raise the roadway grades at this site, material which meets OPSS specifications for Select Subgrade Material or Earth Borrow may be used. The select subgrade material or earth borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the material’s standard Proctor maximum dry density value using vibratory compaction equipment. Prior to placing granular material for the roadways, the exposed subgrade should be heavily proof rolled under suitable (dry) conditions and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling should be subexcavated and replaced with suitable earth borrow approved by the geotechnical engineer.

The subgrade should be shaped and crowned to promote drainage of the roadway granular materials.

5.15.2 Pavement Structure

The following minimum pavement structure is suggested for exterior roadways and parking areas that will be for light traffic only (i.e., no heavy truck traffic):

- 80 millimetre thick layer of asphaltic concrete (2 lifts of 40 millimetres of Superpave 12.5 Traffic Level B); over
- 150 millimetre thick layer of base (OPSS Granular A); over
- 300 millimetre thick layer of subbase (OPSS Granular B Type II);

The following minimum pavement structure is suggested for exterior roadways for heavy traffic (i.e., garbage and fire trucks):

- 100 millimetre thick layer of asphaltic concrete (40 millimetres of Superpave 12.5 Traffic Level D over 60 millimetres of Superpave 19.0 Traffic Level D); over
- 150 millimetre thick layer of base (OPSS Granular A); over
- 450 millimetre thick layer of subbase (OPSS Granular B Type II);

The above pavement structures assumes that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thickness given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction. In our experience, a geotextile will likely be required in most cases where the subgrade consists of overburden, if the roadway construction is planned during the wet period of the year (such as the spring or fall).

Similarly, if the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the Granular B Type II, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.15.3 Granular Material Compaction

The pavement granular materials should be compacted in maximum 300-millimetre-thick lifts to at least 99 percent of material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.15.4 Asphaltic Cement

Performance graded PG 58-34 asphaltic cement is recommended for the roadways and parking areas.

5.15.5 Transition Treatments

In areas where the new pavement structure will abut existing pavements, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

5.15.6 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long-term performance of the pavement at this site. It is suggested that storm sewer catch basins be equipped with 3 metre stub drains extending in at least 2 directions. The stub drains should be installed at the subgrade level.

Further details on pavement drainage can be provided as the design progresses.

5.16 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the sample of soil recovered from borehole 25-05 was 170 micrograms per gram. According to Canadian Standards Association (CSA) "Concrete Materials and Methods of Concrete Construction", the concentration of sulphate can be classified as low. Therefore, any concrete in contact with the native soil could be batched with General Use (GU) cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) use on the roadway should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

Based on the resistivity and pH of the sample, the soil in this area can be classified as slightly aggressive towards unprotected steel. It should be noted that the corrosivity of the soil or groundwater could vary throughout the year due to the application sodium chloride for de-icing.

6.0 ADDITIONAL CONSIDERATIONS

6.1 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction and excavation) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. However, the magnitude of the vibrations is expected to be much less than that required to cause damage to the nearby structures or services.

6.2 Winter Construction

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In the event that construction is required during freezing temperatures, the soil below the footings and floor slabs should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

6.3 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

6.4 Well Abandonment

The monitoring wells installed as part of this investigation should be decommissioned by a licensed well technician. The well abandonment could be carried out in advance of, or during the construction.

6.5 Design Review and Construction Observation

The final details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the design drawings be reviewed by the geotechnical engineer as the design progresses to ensure that the guidelines provided in this report have been interpreted as intended.

In accordance with Section 4.2.2.2 of the Ontario Building Code (2024), the engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the proposed structures, access roadways, and parking areas should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

7.0 CLOSURE

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

W. Alex Meacoe

Alex Meacoe, P.Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
W. A. MEACOE
100162115
Jan 19, 2026
PROVINCE OF ONTARIO

William (Bill) Cavers
Principal Geotechnical Engineer

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DECREASE IN PROPERTY VALUE: GEMTEC shall not be responsible for any decrease, real or perceived, of the property or site's value or failure to complete a transaction, as a consequence of the information contained in this report.

RELIANCE ON PROVIDED INFORMATION: The evaluation and conclusions contained in this report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations. information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in this report as a result of misstatements, omissions, misrepresentations. or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

INVESTIGATION LIMITATIONS: Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions but even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions.

The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. Accordingly, GEMTEC does not warrant or guarantee the exactness of the subsurface descriptions.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination-or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

In addition, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

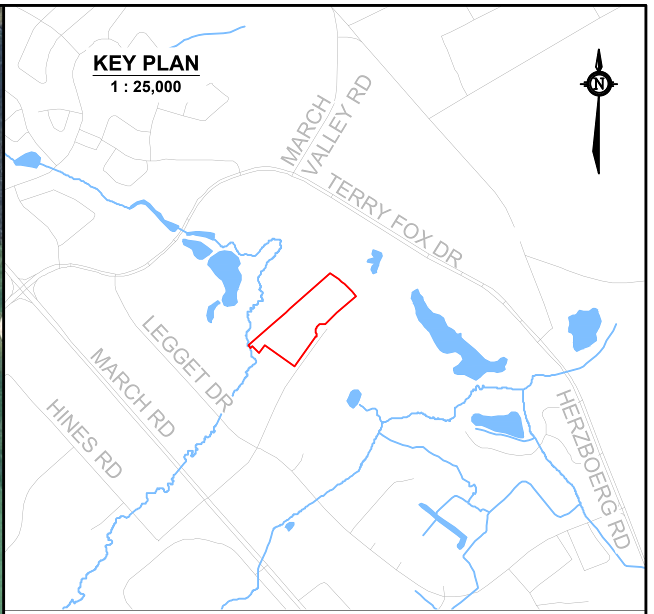
SAMPLE DISPOSAL: GEMTEC will dispose of all uncontaminated soil and/or rock samples 60 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

FOLLOW-UP AND CONSTRUCTION SERVICES: All details of the design were not known at the time of submission of GEMTEC's report. GEMTEC should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of GEMTEC's report.

During construction, GEMTEC should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of GEMTEC's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in GEMTEC's report. Adequate field review, observation and testing during construction are necessary for GEMTEC to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, GEMTEC's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

CHANGED CONDITIONS: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that GEMTEC be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that GEMTEC be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

DRAINAGE: Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. GEMTEC takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



KEY PLAN
1 : 25,000

LEGEND

BH # ← BOREHOLE ID
XX.XX ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM
⊕ BOREHOLE LOCATION (current investigation by GEMTEC)
⊕ BOREHOLE LOCATION (previous investigation by Golder)
🟥 APPROXIMATE SITE BOUNDARY

DATA SOURCES AND REFERENCES

1. Coordinate system: CSRS.UTM-18N
2. Distances, elevations, and coordinates are shown in metres unless denoted otherwise
3. This drawing is a schematic representation and should not be taken as a substitute for a legal survey.
4. Image @2025 Google Maps, CNES / Airbus, First Base Solutions, Maxar Technologies
5. Contains information licensed under the Open Government Licence – Ontario
6. Geographic dataset source: Ontario GeoHub

SCALE
1:1,500
0 30 60 90m

DRAWING
SITE PLAN

CLIENT
SILK DEVELOPMENT GROUP LIMITED

PROJECT
GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
2505 AND 2507 SOLANDT ROAD
OTTAWA, ONTARIO

DRAWN BY SL
CHECKED BY WAM/BC
PROJECT NO. 104638.001
REVISION NO. 0
DATE JANUARY 2026
FIGURE NO. **FIGURE 1**

GEMTEC
Consulting Engineers and Scientists

32 Steacie Drive
Ottawa, ON, K2K 2A9
Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

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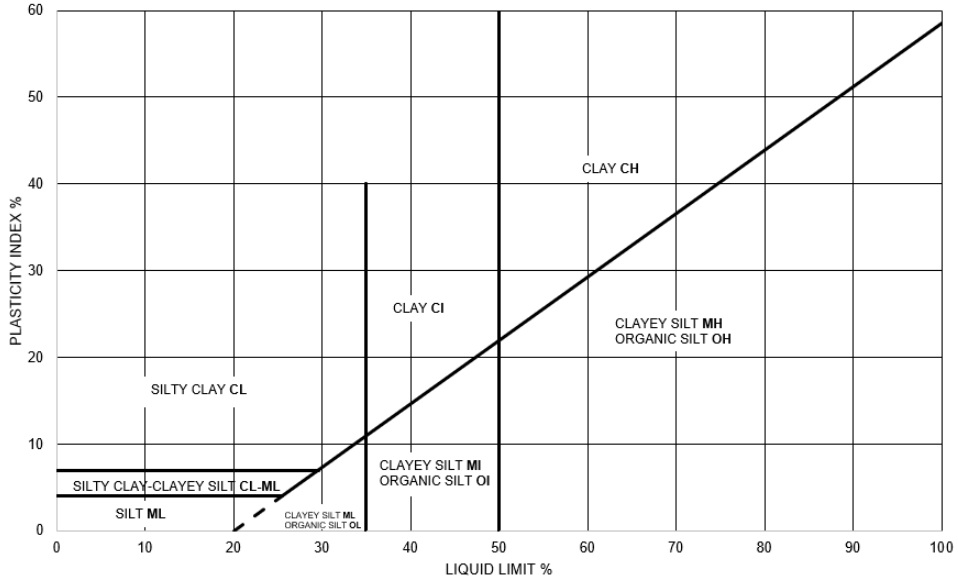
APPENDIX A

Record of Borehole Logs
List of Abbreviations and Symbols
Boreholes 25-01 to 25-05

Method of Soil Classification

GEMTEC's Soil Classification is based on the MTC Soil Classification Manual (January 1980)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10}\ X\ D_{60}}$	USCS Group Symbol	Group Name
Inorganic (Organic Content less than 30%)	Coarse Grained Soils (>50% is larger than 0.075 mm)	Gravel (>50% of coarse fraction is > 4.75 mm)	Gravel with ≤12% fines	Poorly Graded	<4	≤1 or ≥3	GP	Gravel
				Well Graded	≥4	1 to 3	GW	Gravel
			Gravel with >12% fines	Below A Line	N/A		GM	Silty Gravel
				Above A Line	N/A		GC	Clayey Gravel
		Sand (≥50% coarse fraction is > 4.75 mm)	Sand with ≤12% fines	Poorly Graded	<6	≤1 or ≥3	SP	Sand
				Well Graded	≥6	1 to 3	SW	Sand
			Sand with >12% fines	Below A Line	N/A		SM	Silty Sand
				Above A Line	N/A		SC	Clayey Sand
	Soil Group	**Type of Soil**	**Liquid Limit**	**Field Tests**			**USCS Group Symbol**	**Group Name**
				Dilatancy	**Thread Diameter**	**Toughness**		
	Fine Grained Soils (≥50% is smaller than 0.075 mm)	Silts (Non-Plastic or PI and LL plot below A-Line)	<50	Rapid	>6 mm	N/A	ML	Silt
				Slow	3 to 6 mm	None to low	ML	Clayey Silt
				Slow to V. Slow	3 to 6 mm	Low	OL	Organic Silt
			≥50	Slow to V. Slow	3 to 6 mm	Low to Medium	MH	Clayey Silt
				None	1 to 3 mm	Medium to High	OH	Organic Silt
		Clays (PI and LL plot above A-Line)	Liquid Limit <35	None	~3 mm	Low to Medium	CL	Silty Clay
			Liquid Limit 35 to 50	None	1 to 3 mm	Medium	CI	Silty Clay
			Liquid Limit >50	None	<1 mm	High	CH	Clay
Highly Organic (> 30%)	Peat (Amorphous or Fibrous)						PT	Peat



Dual Symbol – Is used to indicate when soils are transitional. For coarse grained soils, it is used when the soil has between 5 and 12% fines (e.g., SP-SC, Sand to Silty Sand). For fine-grained soils it is used when the plasticity index and liquid limit values plot in the area shown in the plasticity chart on this page.

Borderline Symbol – Is used to indicate soils that are not clearly in one soil type but have similar behaviour and properties as similar materials (e.g., CL/CI or GM/SM).

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	Specific gravity
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
UC	Unconfined compression test
γ	Unit weight

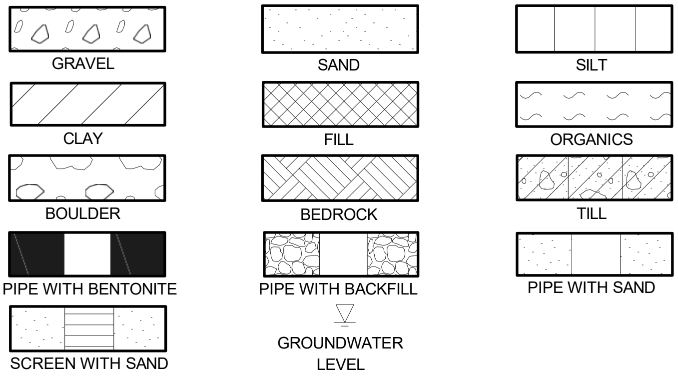
PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

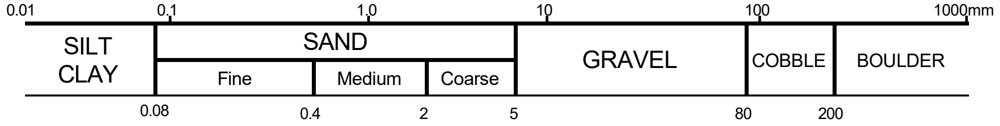
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

WH	Sampler advanced by static weight of hammer and drill rods
WR	Sampler advanced by static weight of drill rods
PH	Sampler advanced by hydraulic pressure from drill rig
PM	Sampler advanced by manual pressure

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	**Description**	**Cu, kPa**	**Description**
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE



DESCRIPTIVE TERMINOLOGY

0	5	12	30
TRACE	SOME	ADJECTIVE	noun > 30% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

RECORD OF BOREHOLE 25-01

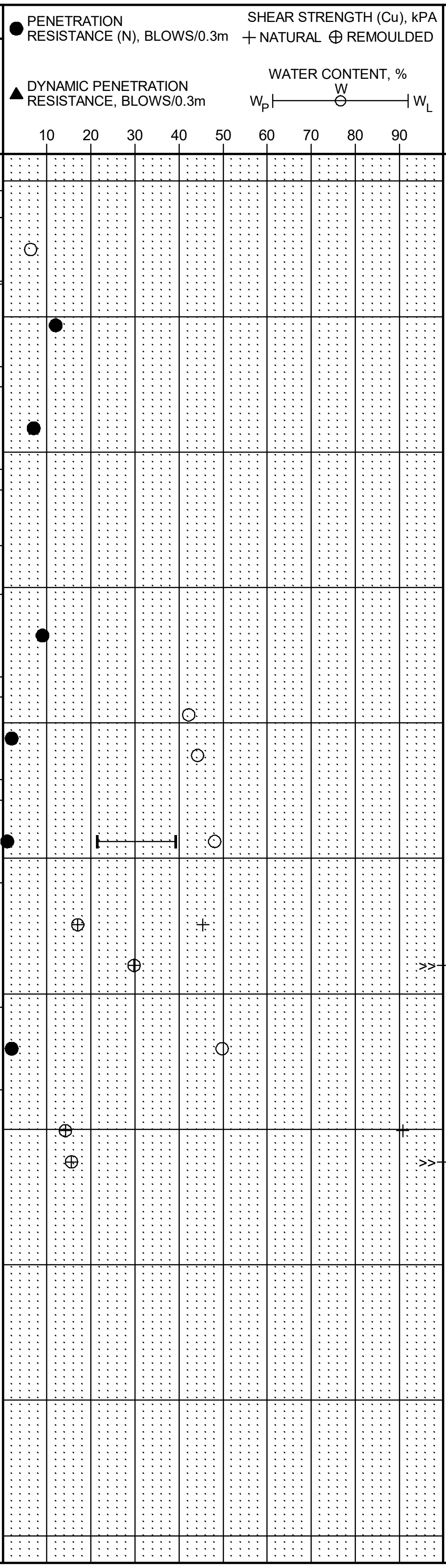
CLIENT: Silk Development Group Limited
PROJECT: Geotechnical Investigation, Proposed Commercial Develoment, 2505 Solandt Road, Ottawa, Ontario
JOB#: 104638.001
LOCATION: See Site Plan, Figure 1

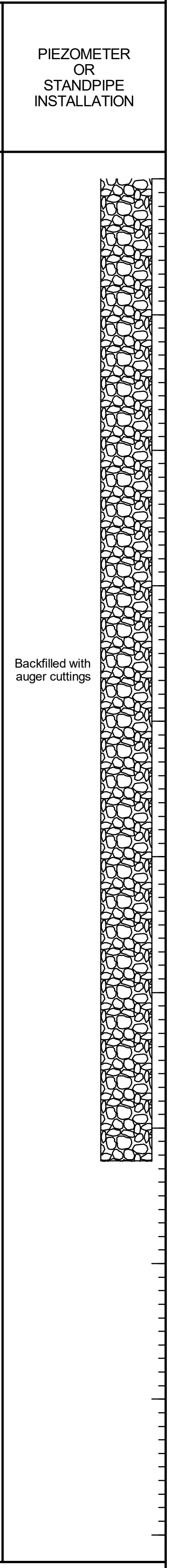
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Sep 18 2025

BORING METHOD: Power Auger, Hollow Stem Auger (114mm OD)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	77.54
ASPHALTIC CONCRETE	0.07
BASE MATERIAL - (GP-GM) sandy gravel, some non-plastic fines; grey; non-cohesive, moist	0.27
SUBBASE MATERIAL - (GP-SP) gravel and sand, some non-plastic fines; brown, with cobbles; non-cohesive, moist	76.80 0.74
FILL - (ML) gravelly sandy silt; grey brown, with cobbles and boulders; non-cohesive, moist, compact to loose	
FILL - (GP) gravel, some sand, trace non-plastic fines; grey, with cobbles and bolders; non-cohesive, moist, loose	74.49 3.05
(CL-ML) SILTY CLAY to CLAYEY SILT; brown, with organics; cohesive, w>PL	73.73 3.81
(CL) CLAY; grey; cohesive, w>PL, firm to very stiff	73.48 4.06
End of borehole Auger refusal Note: Auger refusal encountered at about 2.6 metres depth. the borehole was continued adjacent to the original location	70.30 7.24

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	GS		-	
2	GS		-	M
3	SS	255	12	
4	SS	230	7	
5	SS	150	>50	
6	SS	100	9	
7	SS	355	2	
8	SS	610	1	
9	SS	610	2	





LOGGED: A.N.
CHECKED: C.C.

RECORD OF BOREHOLE 25-02

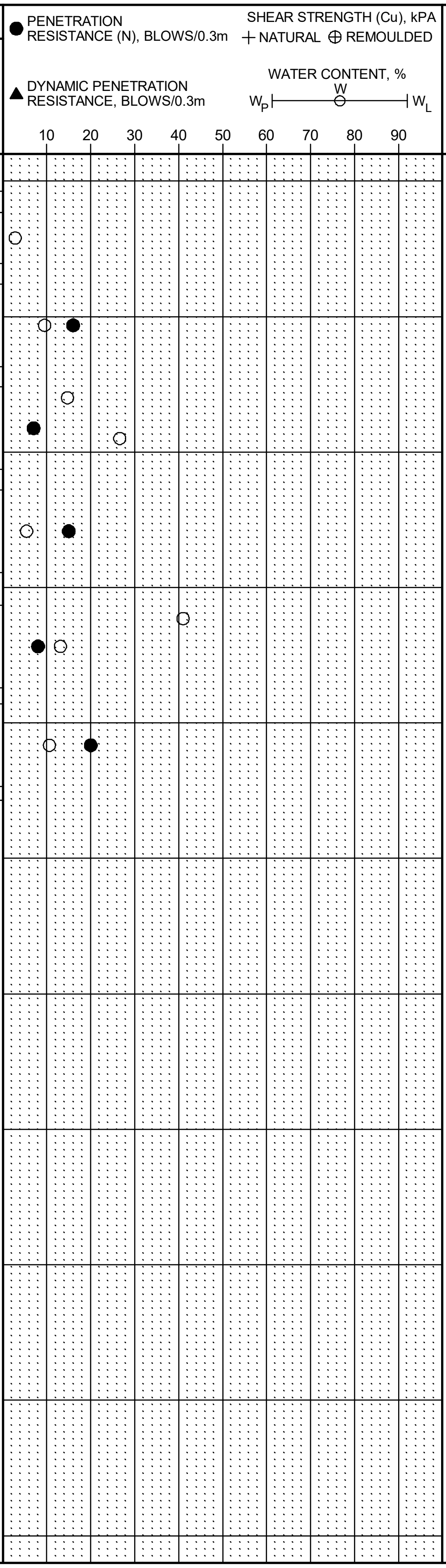
CLIENT: Silk Development Group Limited
PROJECT: Geotechnical Investigation, Proposed Commercial Develoment, 2505 Solandt Road, Ottawa, Ontario
JOB#: 104638.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Sep 19 2025

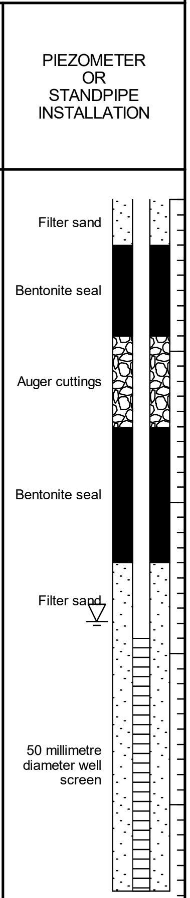
BORING METHOD: Power Auger
Solid Stem Auger (114mm OD)

SOIL PROFILE DESCRIPTION	ELEV.	DEPTH (m)
Ground Surface	77.55	
ASPHALTIC CONCRETE		0.08
BASE MATERIAL - (GP-GM) sandy gravel, some non-plastic fines; grey; non-cohesive, moist		0.23
SUBBASE MATERIAL - (GP-SP) gravel and sand, some non-plastic fines; brown, with cobbles; non-cohesive, moist	76.94	0.61
FILL - (CL) gravelly silty clay, some sand; grey brown, with cobbles and boulders; cohesive, w>PL		
FILL - (SM) silty sand, some gravel; grey, with cobbles and boulders; non-cohesive, moist to wet, loose to compact	75.82	1.73
FILL - (CL-ML) silty clay to clayey silt; brown, with organics; cohesive, w>PL	74.35	3.25
FILL - (GP-GM) gravel, some sand, some non-plastic fines; grey brown, with cobbles and boulders; non-cohesive, wet, loose		
PROBABLE (CL) SILTY CLAY; brown, with organics; cohesive, w>PL, stiff	73.64	3.91
End of borehole Auger refusal	72.98	4.57

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	GS		-
2	GS		-
3	SS	205	16
4	SS	430	7
5	SS	150	15
6	SS	205	8
7	SS	150	20
8	SS	0	>50



ADDITIONAL LAB. TESTING



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/10/21	2.79 ▽	74.8

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LOGGED: A.N.
CHECKED: C.C.

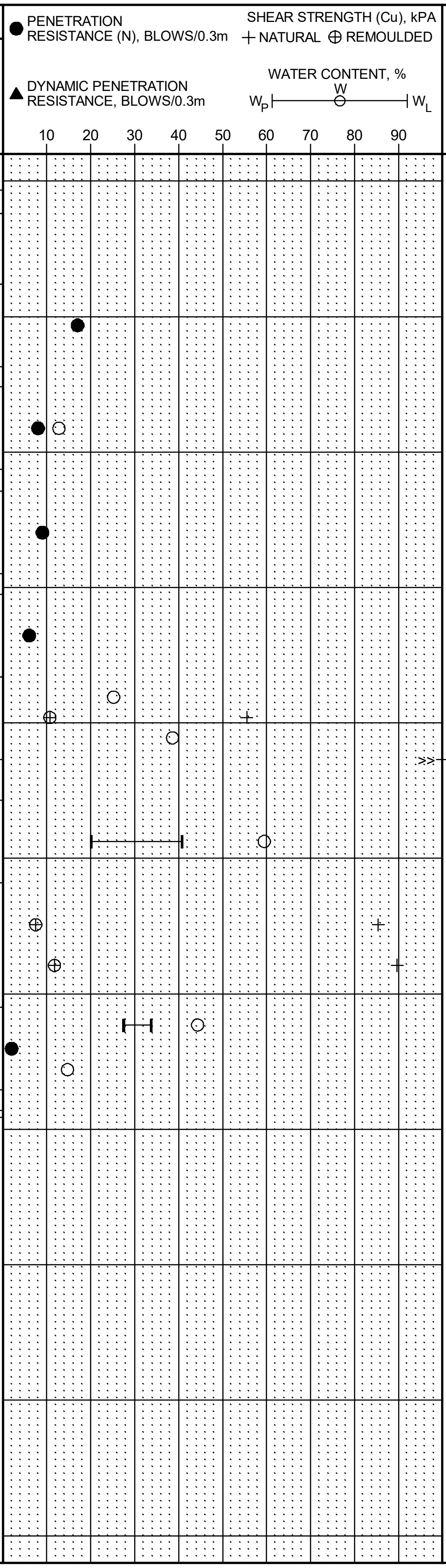
RECORD OF BOREHOLE 25-03

CLIENT: Silk Development Group Limited
PROJECT: Geotechnical Investigation, Proposed Commercial Develoment, 2505 Solandt Road, Ottawa, Ontario
JOB#: 104638.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Sep 30 2025

BORING METHOD: Diamond Rotary Core Advance; NW Casing Advance

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	77.18					
ASPHALTIC CONCRETE	0.07	A	GS		-	
BASE MATERIAL - (GP-GM) sandy gravel, some non-plastic fines; grey to grey brown; non-cohesive, moist	0.24	B	GS		-	
SUBBASE MATERIAL - (GP-SP) gravel and sand, some non-plastic fines; grey, with cobbles and boulders, non-cohesive, moist	76.42 0.76					
FILL - (ML) gravelly sandy silt; grey; with cobbles and boulders, non-cohesive, moist, compact to loose		1	SS	380	17	
		2	SS	305	8	M
		3	SS	205	9	
	73.52 3.66	4	SS	0	6	
(CL) SILTY CLAY, trace sand, trace gravel; dark brown, with organics; non-cohesive, w>PL	73.22 3.96	5	SS	230	-	
(CL/CI) CLAY to SILTY CLAY, trace sand; grey; cohesive, w>PL, stiff to very stiff	70.78 6.40	6	SS	610	WH	
(SM) gravelly SILTY SAND; dark brown, with cobbles and boulders (GLACIAL TILL); non-cohesive, very loose, wet	70.27 6.91	7	SS	300	2	MH
		8	SS	0	>50	
End of borehole Casing refusal Note: Auger refusal encountered at about 1.5 metres depth. The borehole was then advanced using washed boring adjacent to the original location						



PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with auger cuttings

LOGGED: A.N.
CHECKED: C.C.

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

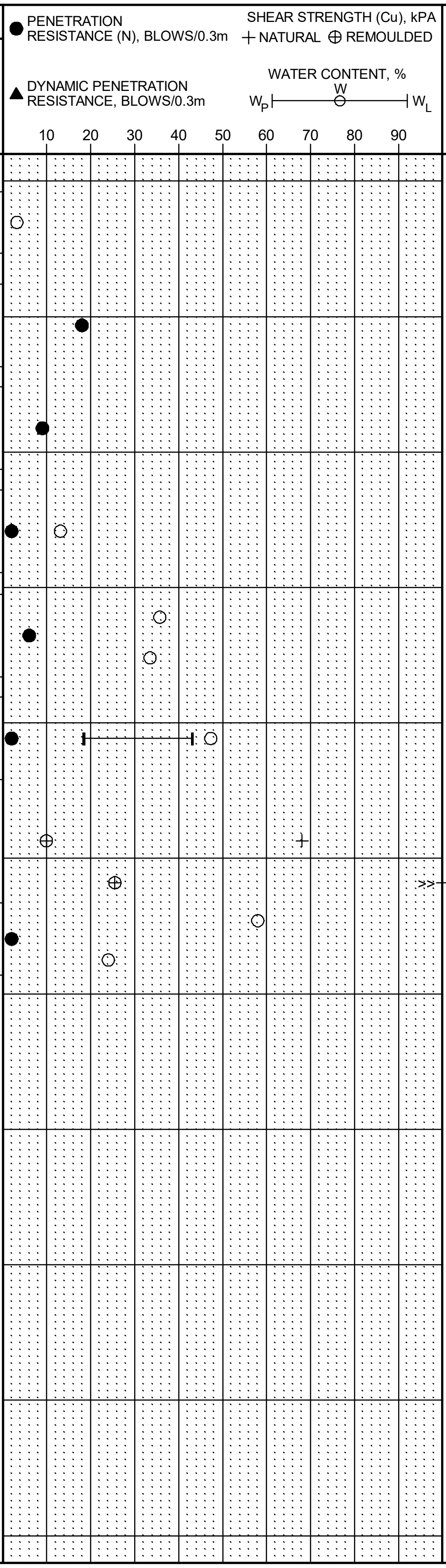
RECORD OF BOREHOLE 25-04

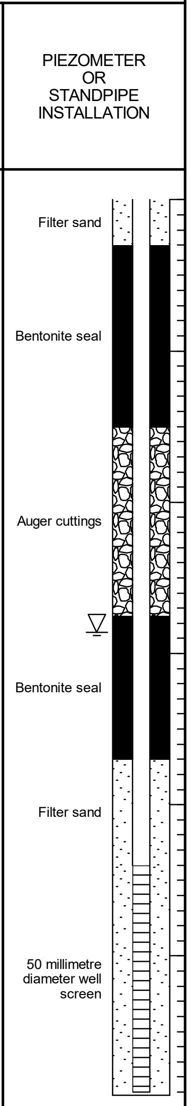
CLIENT: Silk Development Group Limited
PROJECT: Geotechnical Investigation, Proposed Commercial Develoment, 2505 Solandt Road, Ottawa, Ontario
JOB#: 104638.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Sep 19 2025

BORING METHOD: Power Auger, Solid Stem Auger (114mm OD)

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	77.47					
ASPHALTIC CONCRETE	0.08					
BASE/SUBBASE - (GP-SP) gravel and sand, some non-plastic fines; grey; non-cohesive, moist	0.53	1	GS		-	M
FILL - (SM) silty sand and gravel, grey brown, with cobbles and boulders; non-cohesive, moist, compact to very loose	74.42 3.05	2	SS	355	18	
		3	SS	150	9	
		4	SS	355	2	
(CI) SILTY CLAY; brown, with organics; cohesive, w>PL	74.09 3.38	5	SS	485	6	
(CI) CLAY; grey; cohesive, w>PL, stiff to very stiff	71.89 5.58	6	SS	610	2	
(ML) gravelly sandy SILT, grey with cobbles and boulders (GLACIAL TILL); non-cohesive, wet, loose	71.55 5.92	7	SS	485	2	
End of borehole Auger refusal						





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/10/21	2.86 ▽	74.6

LOGGED: A.N.
CHECKED: C.C.

RECORD OF BOREHOLE 25-05

CLIENT: Silk Development Group Limited
PROJECT: Geotechnical Investigation, Proposed Commercial Develoment, 2505 Solandt Road, Ottawa, Ontario
JOB#: 104638.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Sep 18 2025

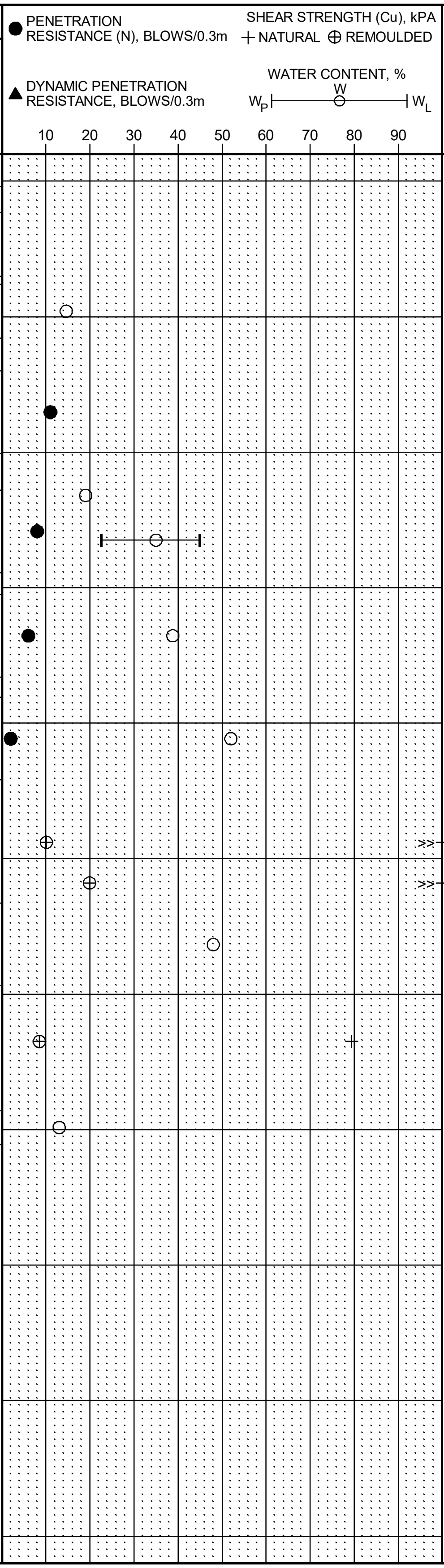
BORING METHOD: Power Auger, Solid Stem Auger (114mm OD)

SOIL PROFILE

DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	77.61
ASPHALTIC CONCRETE	0.04
BASE - (GP-GM) sandy gravel, some non-plastic fines; grey; non-cohesive, moist	0.23
SUBBASE - (GP-GM) gravel and sand, some non-plastic fines; brown, with cobbles; non-cohesive, moist	76.91 0.70
FILL - (CI) silty clay, some gravel, some sand; grey brown, with cobbles and boulders; non-cohesive, moist, very dense to compact	75.22 2.39
(CI) CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, stiff to very stiff	73.34 4.27
(CI) CLAY; grey; cohesive, w>PL, stiff to very stiff	71.16 6.45
(ML) sandy SILT, some gravel; grey, with cobbles and boulders (GLACIAL TILL); non-cohesive, wet, very dense	70.50 7.11
End of borehole Auger refusal Note: Auger refusal encountered at 1.1 metres depth. the borehole was continued adjacent to the original location	

SAMPLES

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	GS		-
2	GS		-
3	GS		-
4	SS	205	>50
5	SS	50	11
6	SS	585	8
7	SS	610	6
8	SS	610	2
9	SS	610	WH
10	SS	180	>50



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with auger cuttings

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Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: C.C.

APPENDIX B

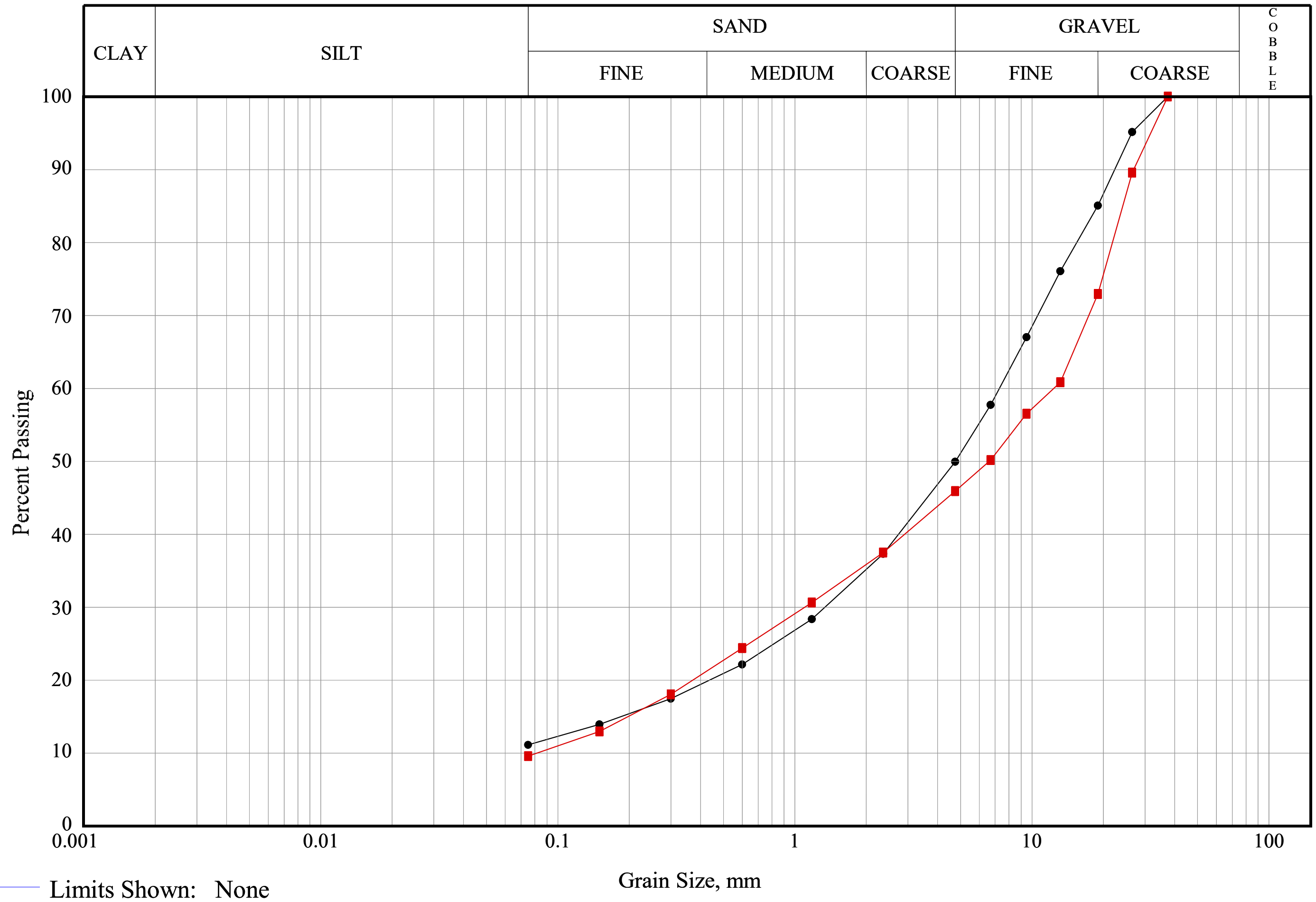
Laboratory Testing

Report to: Silk Development Group Limited
GEMTEC Project: 104638.001 (January 19, 2026)

GEMTEC
Consulting Engineers
and Scientists

Client:	Silk Development Group Limited
Project:	Silk/Solandt Road/ottawa
Project #:	104638001

Soils Grading Chart



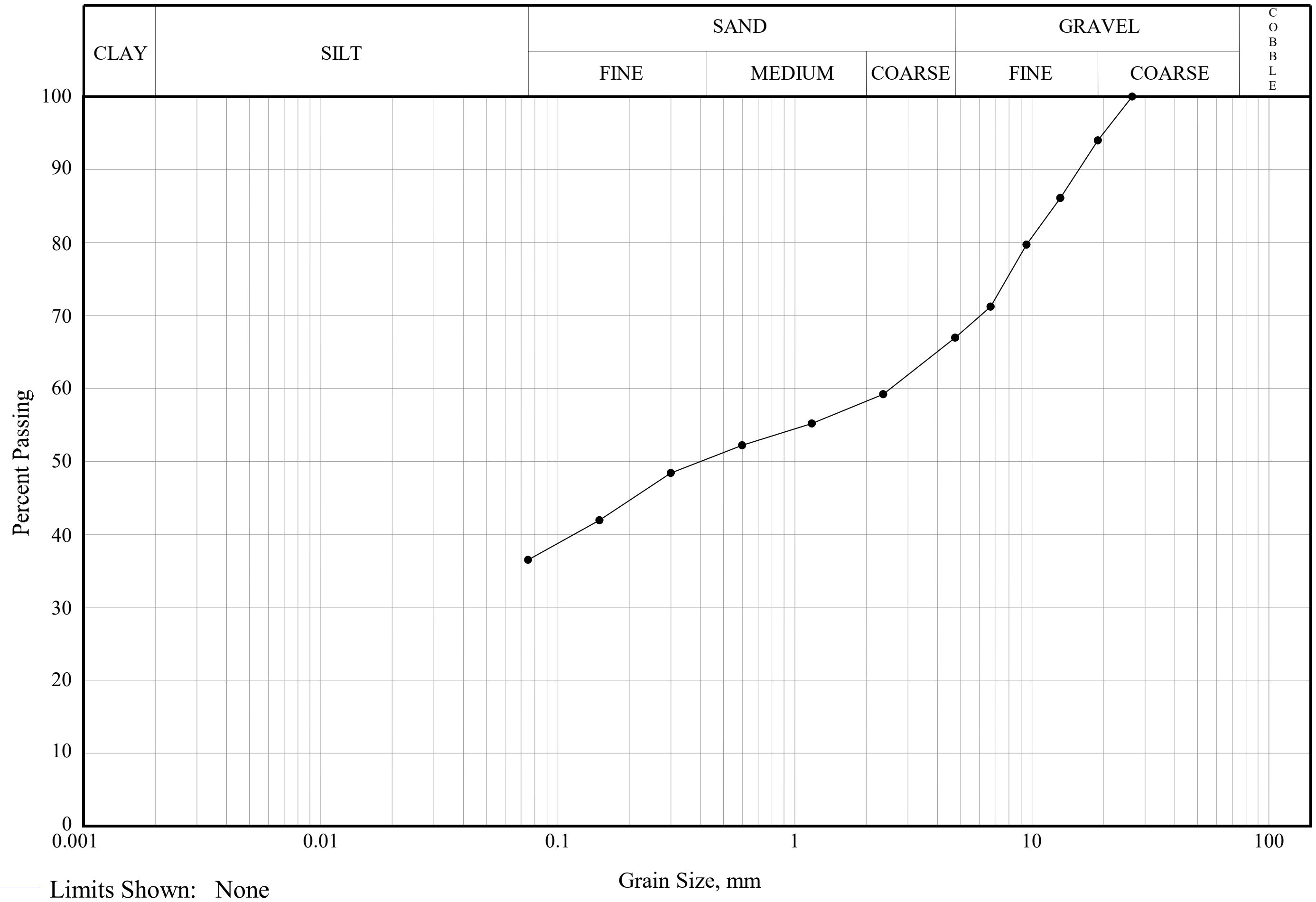
Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	BASE/SUBBASE MATERIAL	25-04	SA 01	0.08-0.53	50.1	38.8	11.1	
—■—	SUBBASE MATERIAL	25-01	SA 02	0.27-0.61	54.1	36.3	9.6	

Line Symbol	USCS Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—		N/A	---	0.185	1.34	4.77	7.30	18.96	---
—■—		N/A	0.082	0.198	1.10	6.61	12.38	24.19	---

Note: More information available upon request

GEMTEC Consulting Engineers and Scientists	Client: Silk Development Group Limited	# Soils Grading Chart
	Project: Silk/Solandt Road/ottawa	
	Project #: 104638001	



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	FILL MATERIAL	25-03	SA 2	1.52-2.13	33.1	30.5	36.5	

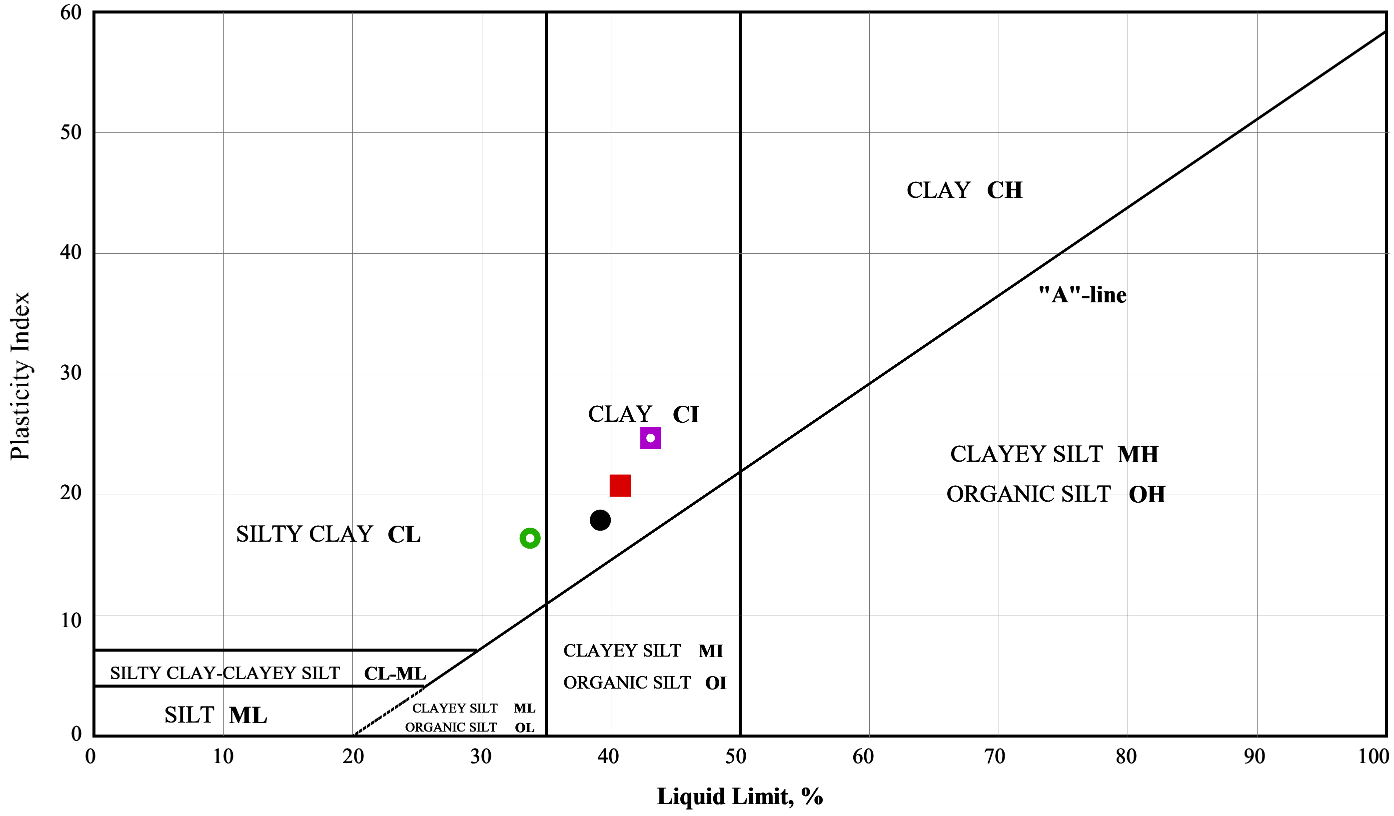
Line Symbol	USCS Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●		N/A	---	---	---	0.40	2.54	12.47	---

Note: More information available upon request

GEMTEC
Consulting Engineers
and Scientists
Ottawa, ON

Client: Silk Development Group Limited
Project: Silk/Solandt Road/ottawa
Project #: 104638001

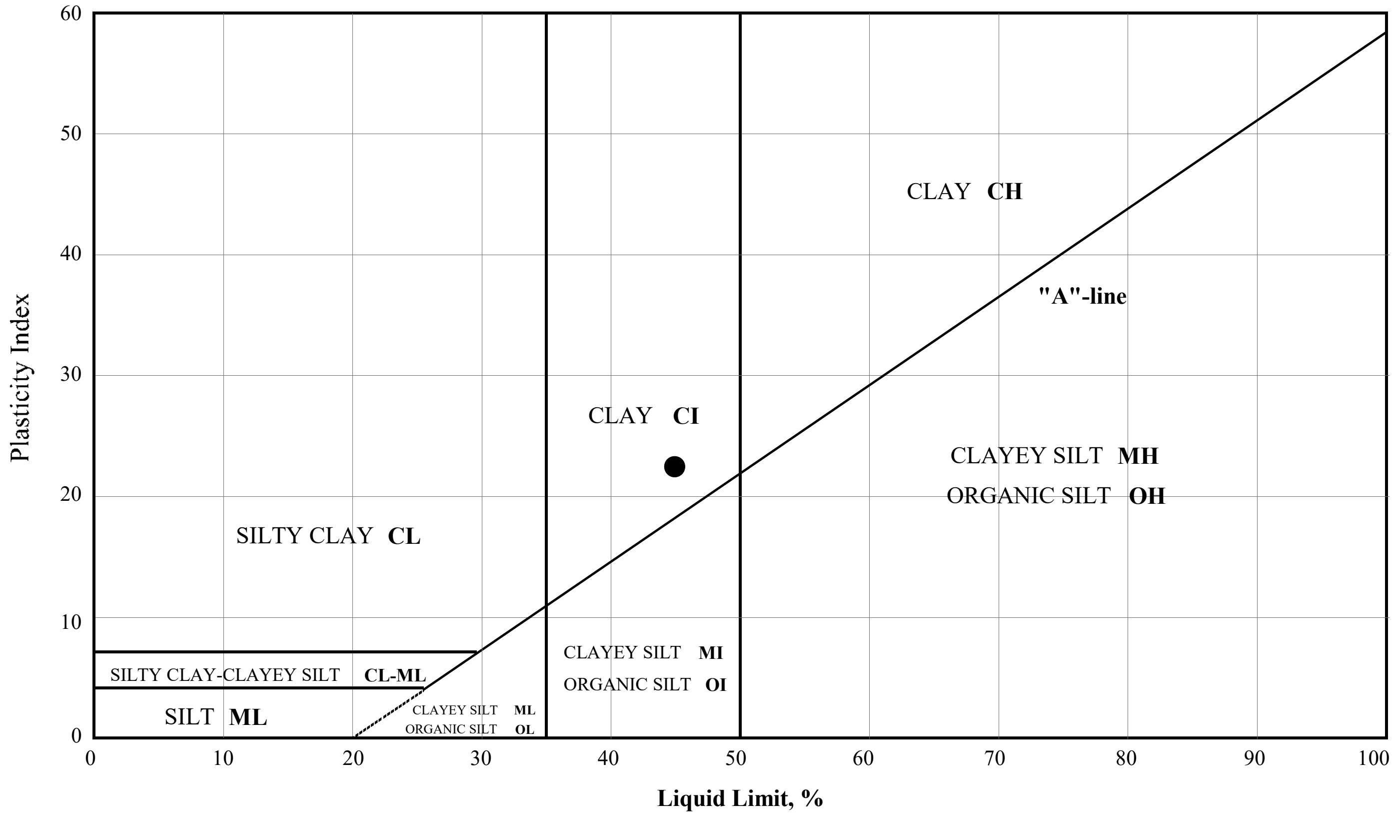
Plasticity Chart
(LS-7034/ASTM D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	25-01	SA 08	4.57-5.18	39.2	21.3	18	N/A	48.1
🟥	25-03	SA 6	4.57-5.18	40.7	20.0	21	N/A	59.5
○	25-03	SA 7	6.09-6.4	33.7	17.4	16	N/A	44.3
▣	25-04	SA 06	3.81-4.42	43.1	18.4	25	N/A	47.3

Note: More information available upon request

GEMTEC Consulting Engineers and Scientists Ottawa, ON	Client: Silk Development Group Limited Project: Silk/Solandt Road/ottawa Project #: 104638001	Plasticity Chart (LS-7034/ASTM D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	25-05	SA 06	2.28-2.89	44.9	22.5	22	N/A	35.0
							N/A	
							N/A	
							N/A	

Note: More information available upon request

GEMTEC
www.gemtec.ca

Shrinkage Limit

ASTM
D4943

Volume of Shrinkage Dish

Mass of Glass Plate (g):	**37.33**
Mass of Shrinkage Dish (g) (m):	**20.70**
Mass of Shrinkage Dish, Plate, Grease and Water (g):	**75.40**
Mass of Water (g):	**17.37**
Volume of Shrinkage Dish:	**17.0**

Test Specimen

Specimen No:	**1**
Mass of Shrinakge Dish, m (g):	**20.82**
Mass of Shrinkage Dish and Wet Soil, m_w (g):	**51.43**
Mass of Shrinkage Dish and Dry Soil, m_d (g):	**41.9**
Mass of Wax-Coated Soil in Air, m_{sxa} (g):	**21.63**
Mass of Wax-Coated Soil in Water, m_{sxw} (g):	**10.5**

Calculated Shrinkage Limit

Specimen No:	**1**
Mass of Dry Soil, m_s (g):	**21.08**
Water Content of Soil when Placed in Dish, w (%):	**45.21**
Mass of Water Displaced by Wax-Coated Soil, m_{wsx} (g):	**11.13**
Volume of Dry Soil and Wax, V_{dx} (cm^3):	**11.13**
Mass of Wax, m_x (g):	**0.55**
Volume of Wax, V_x (cm^3):	**0.61**
Volume of Dry Soil, V_d (cm^3):	**10.52**
Shrinkage Limit, SL	**14.46**

Specific Gravity of Wax = 0.908 at15.5°C

Specific Gravity of Wax = 0.900 at 20°C

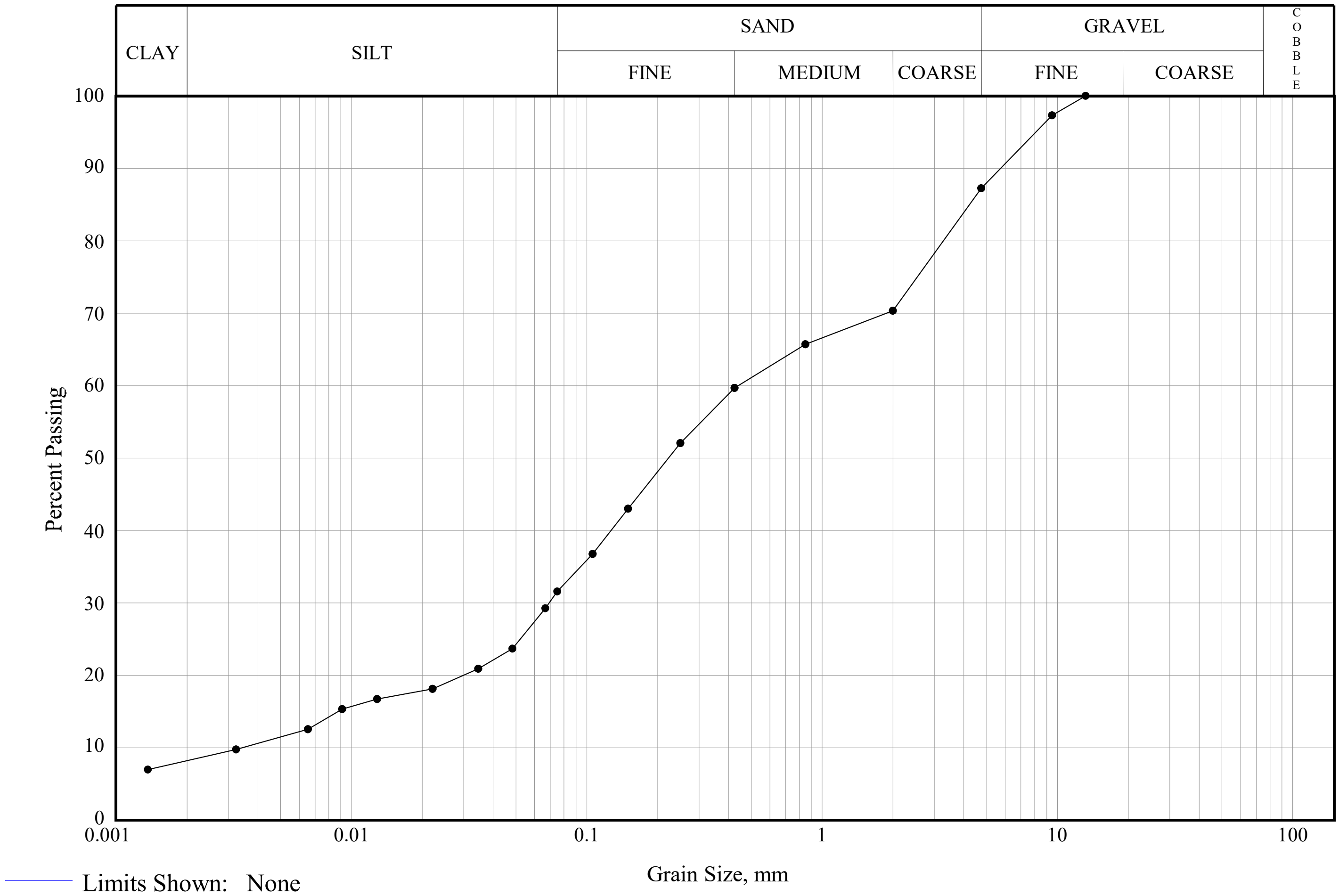
Density of Water (g/cm^3) = 1.000 (g/cm^3)

Project No: 104638.001	**Tested By: K.Neil**
Project Name: 2505 Solandt Road	**Checked By: K.Smith**
Date Tested: Oct 20, 2025	**Sample No: 25-03B SA 6**
Sample Date: October 7, 2025	**Source:**
Remarks:	**Depth: 4.57-5.18**

GEMTEC
Consulting Engineers and Scientists

Client: Silk Development Group Limited
Project: Silk/Solandt Road/ottawa
Project #: 104638001

Soils Grading Chart (LS-702)



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	GLACIAL TILL	25-03B	SA 7B	6.4-6.9	12.8	55.7	23.4	8.2

Line Symbol	USCS Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—		N/A	0.003	0.009	0.07	0.22	0.44	4.24	23.4

Note: More information available upon request

APPENDIX C

Chemical Analysis of Soil Samples
Sample Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2547575)

PARACEL
LABORATORIES LTD.

Order #: 2547575

Certificate of Analysis

Report Date: 27-Nov-2025

Client: **GEMTEC Consulting Engineers and Scientists Limited**

Order Date: 21-Nov-2025

Client PO:

Project Description: 104638.001

	Client ID:	BH25-05 SA6B 7'6"-9'6"	-	-	-		
	Sample Date:	21-Nov-25 09:00	-	-	-	-	-
	Sample ID:	2547575-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	77.5	-	-	-	-	-
General Inorganics							
Conductivity	5 uS/cm	365	-	-	-	-	-
pH	0.05 pH Units	7.13	-	-	-	-	-
Resistivity	0.1 Ohm.m	27.4	-	-	-	-	-
Anions							
Chloride	10 ug/g	48	-	-	-	-	-
Sulphate	10 ug/g	170	-	-	-	-	-

APPENDIX D

Borehole Records from Previous Investigation
Boreholes 18-101 to 18-105

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-101

SHEET 1 OF 1

LOCATION: N 5023240.0 ;E 350737.9

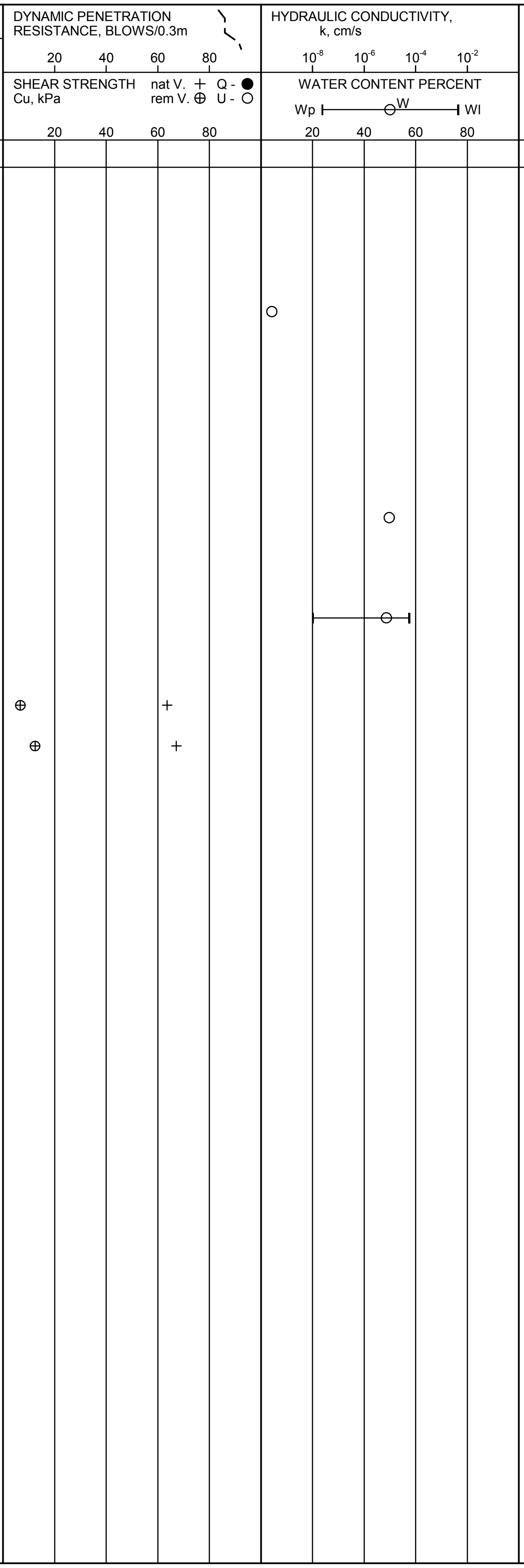
BORING DATE: November 5, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		77.35					
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; brown		0.00					
		(SM) SILTY SAND; red brown; non-cohesive, moist, very loose to compact		77.10 0.25	1	SS	3		
1					2	SS	13	MH	
		(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff		75.83 1.52	3	SS	6		
2		(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff		75.07 2.28	4	SS	2		
3					5	SS	WH		
4									
		(SM/ML) SILTY SAND to sandy SILT, some gravel; grey, contains clayey silt seams (GLACIAL TILL); non-cohesive, wet, very dense		72.78 4.57	6	SS	>50		
5		End of Borehole Auger Refusal		72.48 4.87					
6									
7									
8									
9									
10									



DEPTH SCALE
1 : 50

GOLDER

LOGGED: DJG/RK
CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-102

SHEET 1 OF 2

LOCATION: N 5023196.9 ;E 350745.2

BORING DATE: November 7, 2018

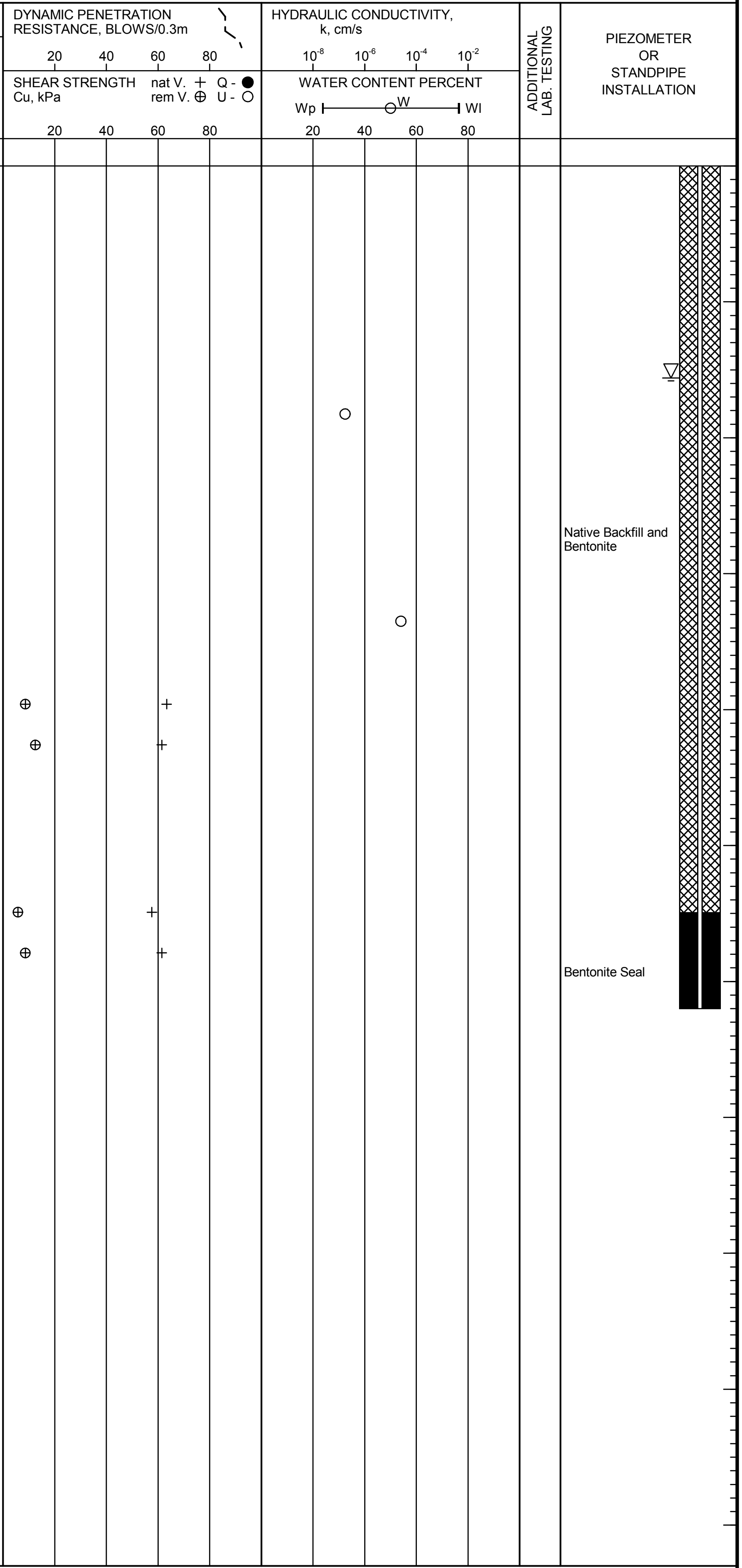
DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
		GROUND SURFACE		77.23
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL- (SM) SILTY SAND; brown		0.00 77.03
		(SM) SILTY SAND; red brown to brown; non-cohesive, moist, very loose to loose		0.20 75.96
		(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); w>PL, very stiff to stiff		1.27 74.34
		(CI/CH) SILTY CLAY to CLAY; grey, contains silt seams; cohesive, w>PL, stiff		2.89 71.44
		Probable (SM) SILTY SAND, some gravel; grey, contains cobbles (GLACIAL TILL); non-cohesive		5.79 71.03
		Borehole continued on RECORD OF DRILLHOLE 18-102		6.20

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	SS	2
2	SS	6
3	SS	4
4	SS	3
5	SS	WH
6	SS	WH



DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF DRILLHOLE: 18-102

SHEET 2 OF 2

LOCATION: N 5023196.9 ;E 350745.2

DRILLING DATE: November 7, 2018

DATUM: CGVD28

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: CCC Drilling

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

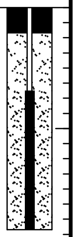
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-8} 10^{-6} 10^{-4} 10^{-2}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		71.03															
7	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, grey, fine to medium grained, non-porous, very strong SANDSTONE		6.20 69.56	1														Bentonite Seal Silica Sand Standpipe
		End of Drillhole		7.67															
8																			WL in Standpipe at Elev. 75.67 m on Nov. 16, 2018
9																			
10																			
11																			
12																			
13																			
14																			
15																			
16																			



DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-103

SHEET 1 OF 2

LOCATION: N 5023202.5 ;E 350791.8

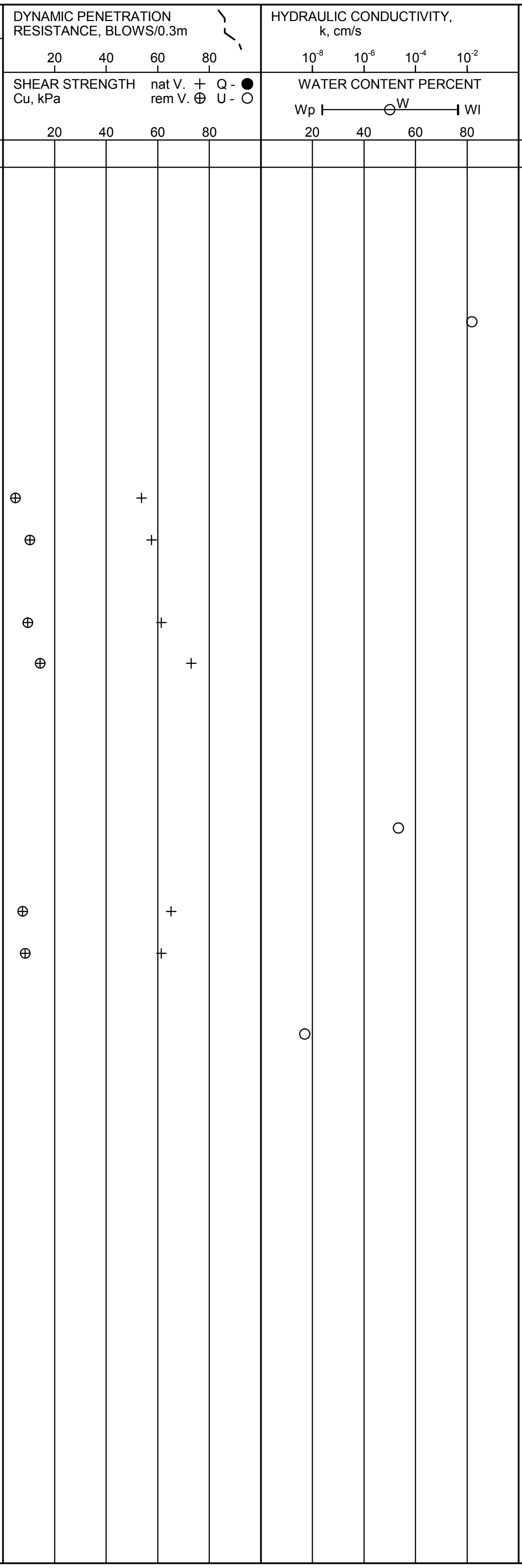
BORING DATE: November 7, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	77.18					
Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; brown	0.00					
	(SM) SILTY SAND; brown; non-cohesive, moist, loose	0.15	1	SS	7		
	(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	76.27 0.91	2	SS	7		
			3	SS	3		
	(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff	75.05 2.13	4	SS	WH		
			5	SS	WH		
	(SM/ML) SILTY SAND to sandy SILT, some gravel to gravelly; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, very loose to very dense	71.09 6.09	6	SS	2	MH	
			7	SS	>50		
	Borehole continued on RECORD OF DRILLHOLE 18-103	69.65 7.53					



DEPTH SCALE 0 1 2 3 4 5 6 7 8 9 10

PROJECT: 18111016-1000

RECORD OF DRILLHOLE: 18-103

SHEET 2 OF 2

LOCATION: N 5023202.5 ;E 350791.8

DRILLING DATE: November 7, 2018

DATUM: CGVD28

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: CCC Drilling

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH COLOUR / % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.25 m (5 10 15 20)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-8} 10^{-6} 10^{-4} 10^{-2})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.
		BEDROCK SURFACE		69.65														
8 9	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, grey, fine to medium grained, non-porous, very strong SANDSTONE		7.53 68.13	1	100												
		End of Drillhole		9.05														

10

11

12

13

14

15

16

17

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-104

SHEET 1 OF 2

LOCATION: N 5023266.5 ;E 350723.5

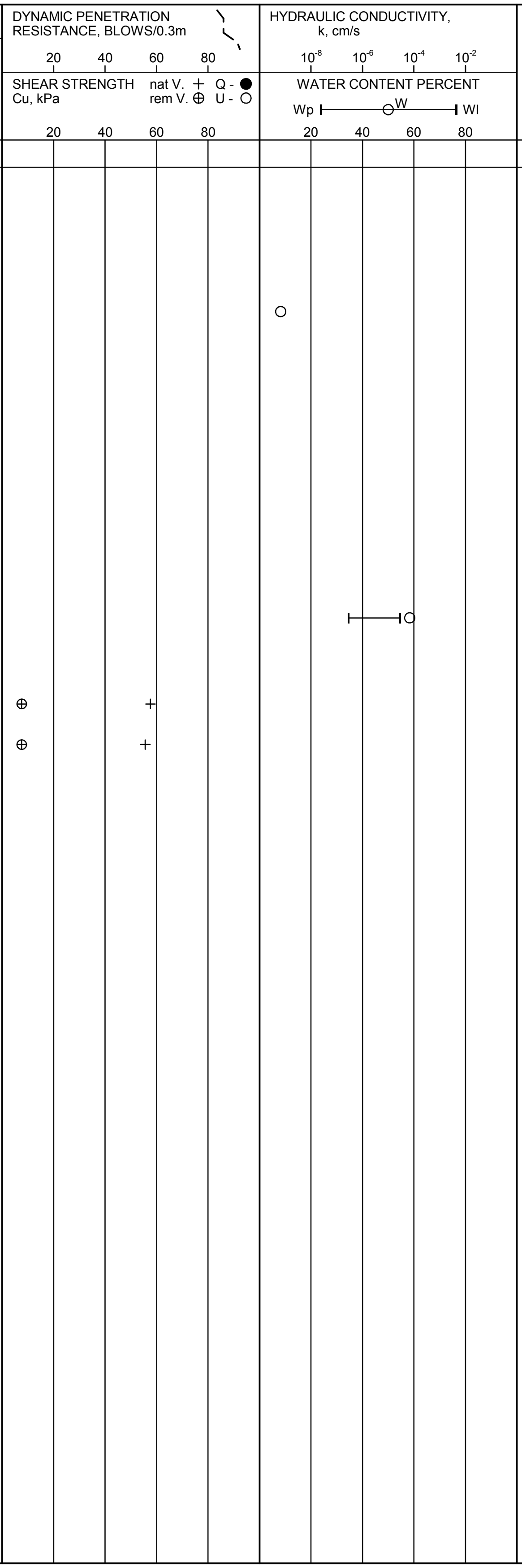
BORING DATE: November 6, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		77.36					
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; dark brown		0.00					
		(SM) SILTY SAND; grey brown; non-cohesive, moist, loose		77.11 0.25	1	SS	7		
1					2	SS	9	MH	
2		(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff		75.69 1.67	3	SS	7		
					4	SS	3		
3		(CI/CH) SILTY CLAY to CLAY; grey, contains silt seams; cohesive, w>PL, stiff		74.47 2.89	5	SS	2		
4					6	SS	WH		
5		Borehole continued on RECORD OF DRILLHOLE 18-104		72.46 4.90					
6									
7									
8									
9									
10									



DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF DRILLHOLE: 18-104

SHEET 2 OF 2

LOCATION: N 5023266.5 ;E 350723.5

DRILLING DATE: November 6, 2018

DATUM: CGVD28

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: CCC Drilling

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-8} 10^{-6} 10^{-4} 10^{-2}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		72.46														
5 6	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, grey fine to medium grained, non-porous, very strong SANDSTONE, with thinly interbedded shale		4.90 70.91	1	100												
7 8 9 10 11 12 13 14		End of Drillhole		6.45														

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-105

SHEET 1 OF 1

LOCATION: N 5023242.5 ;E 350674.9

BORING DATE: November 5, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
GROUND SURFACE	77.03			
TOPSOIL - (SM) SILTY SAND; dark brown	0.00			
(SM-SP) SILTY SAND to SAND, some low-plasticity fines; grey brown; non-cohesive, moist, very loose to compact	76.73 0.30	1	SS	3
		2	SS	15
(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	75.81 1.22			
(CI/CH) SILTY CLAY to CLAY; grey, contains silt seams; cohesive, w>PL, stiff	75.51 1.52	3	SS	6
		4	SS	4
(SM/ML) SILTY SAND to sandy SILT, some gravel to gravelly; grey, contains clayey silt seams and cobbles (GLACIAL TILL); non-cohesive, wet, compact	73.68 3.35	5	SS	13
End of Borehole Auger Refusal	73.33 3.70			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

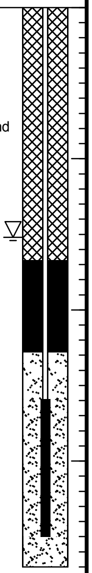
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-8} 10^{-6} 10^{-4} 10^{-2}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Native Backfill and Bentonite

Bentonite Seal

Silica Sand

Standpipe

Silica Sand

WL in Standpipe at Elev. 75.25 m on Nov. 16, 2018

DEPTH SCALE
1 : 50

GOLDER

LOGGED: DJG/RK
CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-106

SHEET 1 OF 1

LOCATION: N 5023250.2 ;E 350784.3

BORING DATE: November 5, 2018

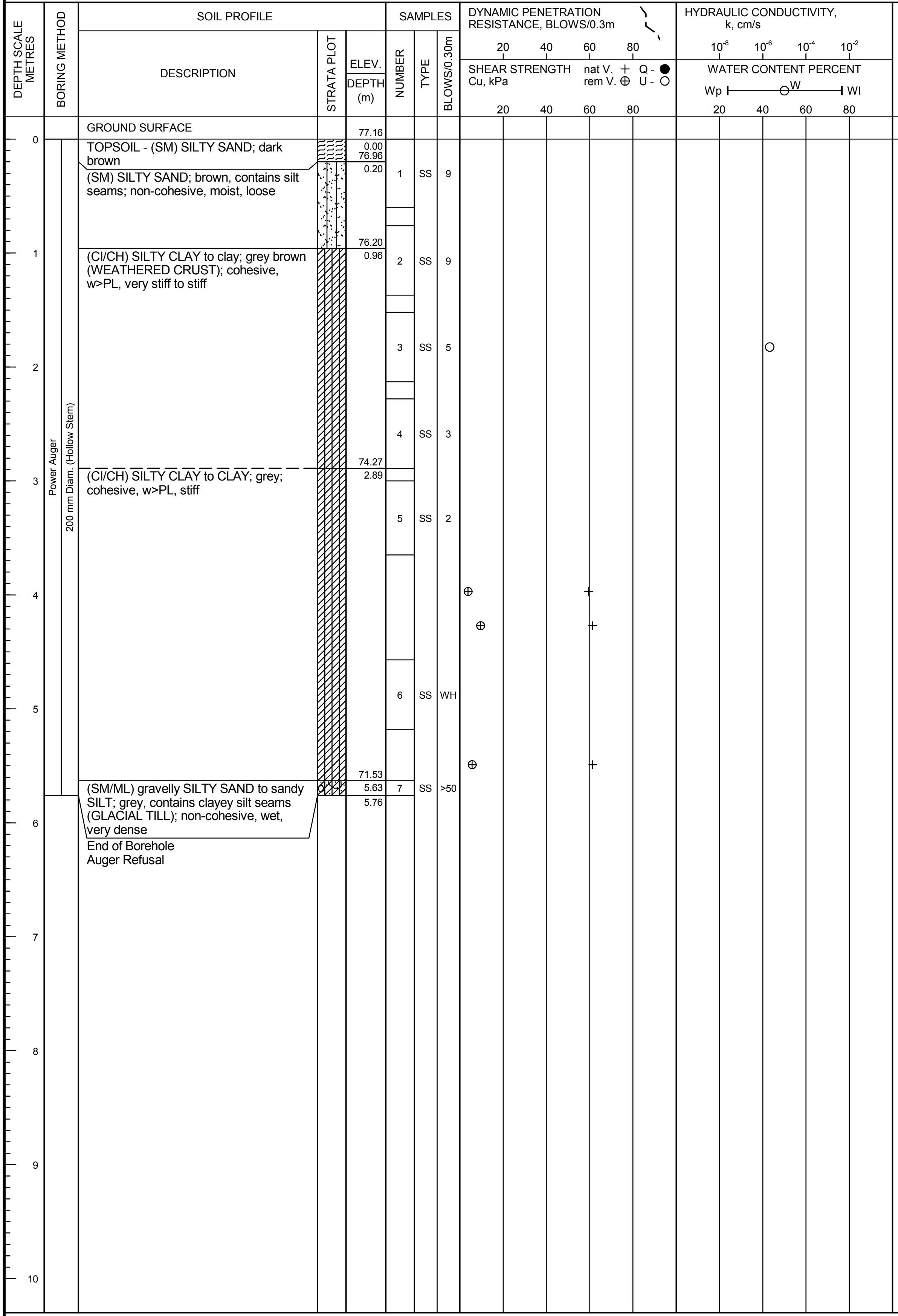
DATUM: CGVD28

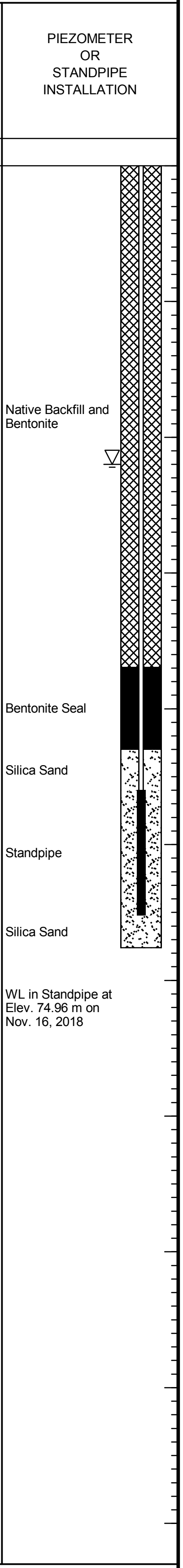
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
GROUND SURFACE	77.16			
TOPSOIL - (SM) SILTY SAND; dark brown	0.00			
(SM) SILTY SAND; brown, contains silt seams; non-cohesive, moist, loose	76.96 0.20	1	SS	9
(CI/CH) SILTY CLAY to clay; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	76.20 0.96	2	SS	9
		3	SS	5
		4	SS	3
(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff	74.27 2.89	5	SS	2
		6	SS	WH
(SM/ML) gravelly SILTY SAND to sandy SILT; grey, contains clayey silt seams (GLACIAL TILL); non-cohesive, wet, very dense	71.53 5.63	7	SS	>50
End of Borehole Auger Refusal	5.76			





DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

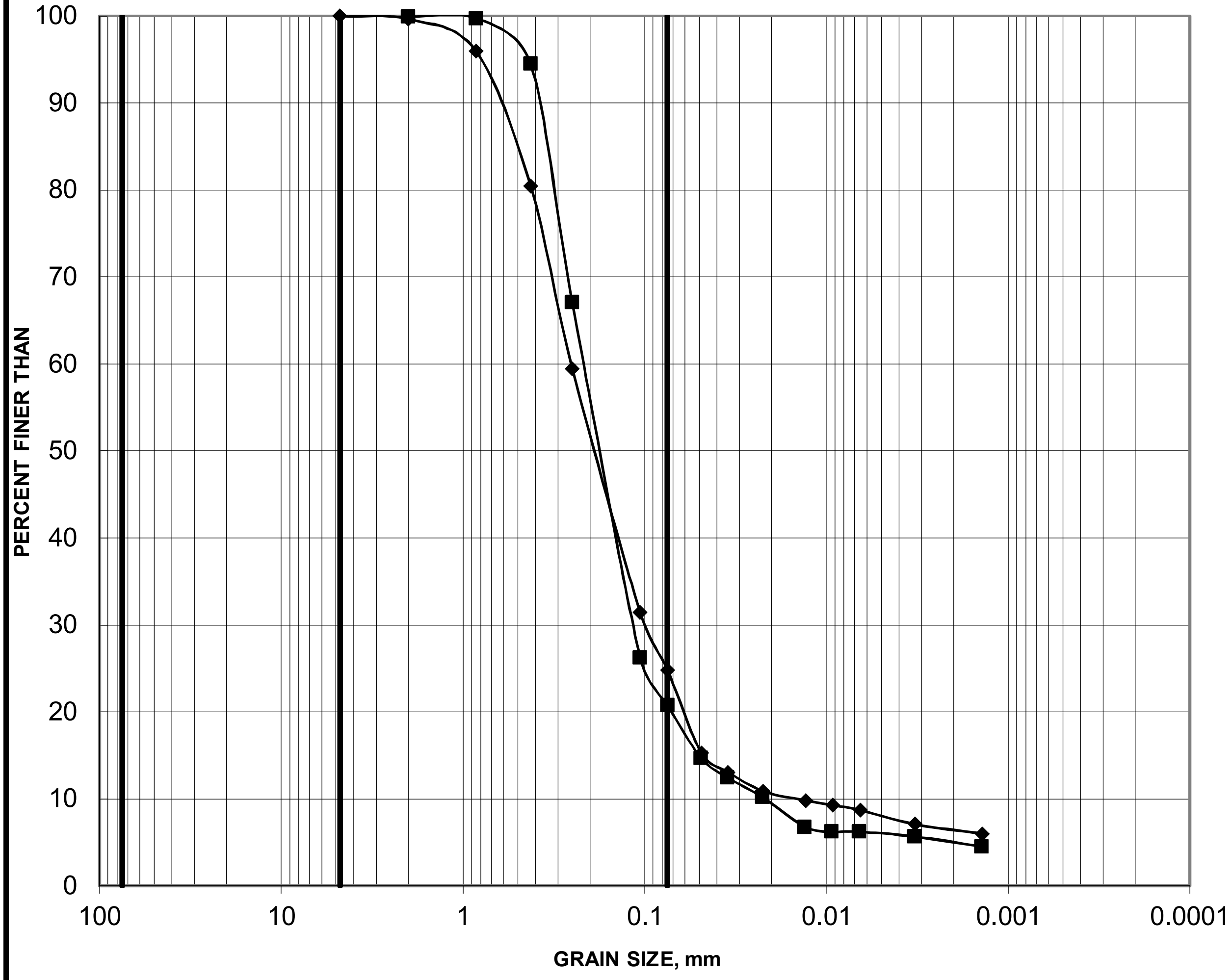
CHECKED: WAM

MIS-BHS 001 18111016.GPJ GAL-MIS.GDT 14/1/19 ZS

GRAIN SIZE DISTRIBUTION

FIGURE 2

(SM) SILTY SAND



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

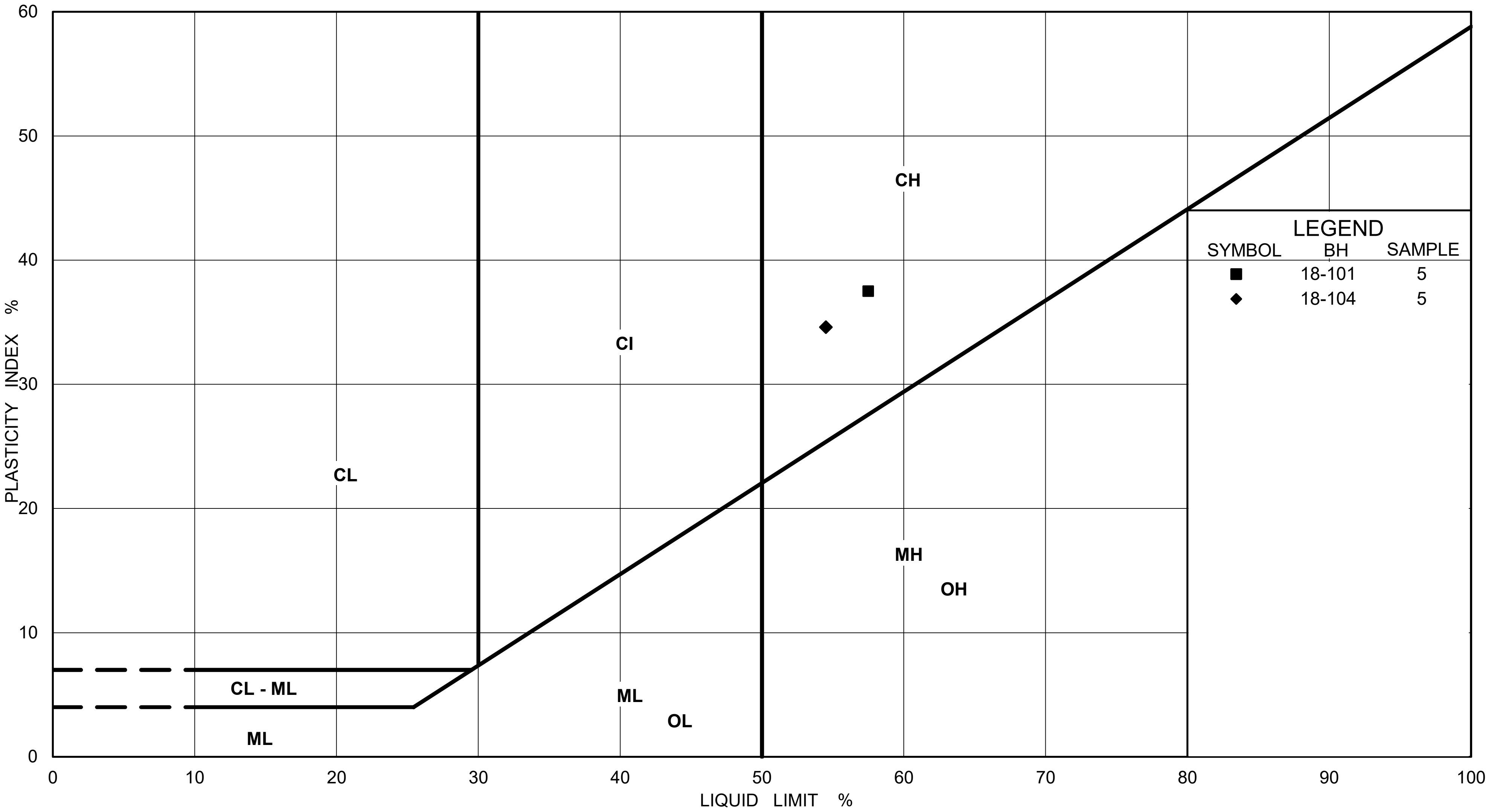
Borehole	Sample	Depth (m)
■ 18-101	2	0.76-1.37
◆ 18-104	2	0.76-1.37

Project: 18111016

Golder Associates

Created by: MI

Checked by: CW



GOLDER

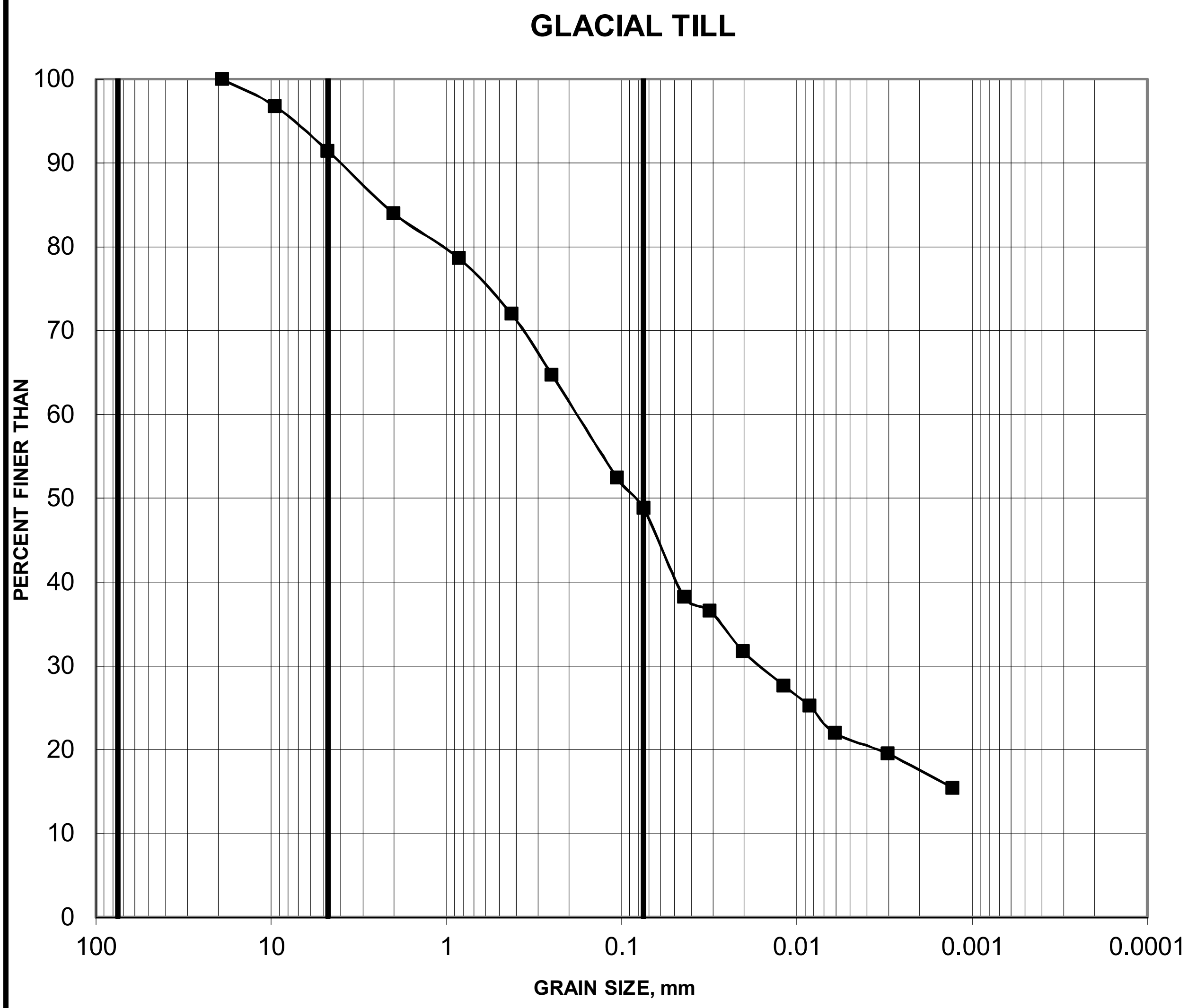
PLASTICITY CHART
SILTY CLAY TO CLAY

Figure : 3

Project No.: 18111016

Compiled By : MI Checked By : CW

GRAIN SIZE DISTRIBUTION	FIGURE 4



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
—■— 18-103	6	6.10-6.71

Project: 18111016

Golder Associates

Created by: MI

Checked by: CW

Golder Associates Ltd.
1931 Robertson Road
Ottawa, Ontario
K2H 5B7

GOLDER

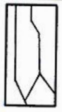
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: KRP Properties Kinaxis Building

Project No.: 18111016

Date: December 4, 2018

Location(s): See Table Below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)	Failure Mode
18-103	8.29-8.43	Nov 29/18	HQ	60.5	2667	182.9	

REMARKS :
- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Specimen ends prepared with high-strength plaster, but un-restrained.
- L/D ratio's between 2.0:1 and 2.5:1
- Time to failure > 2 and < 15 minutes.
- This report constitutes a testing service only. Interpretation of results will be provided on request only.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED: Melanie Ireland

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnement
structural	structures
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

expérience • connaissance • intégrité