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Geotechnical Investigation
Proposed Residential Development
1600 James Naismith Drive

Ottawa Ontario

GEMTEC Project: 103752.002

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Submitted to:

Moreton Properties Ltd.
29 Connell Ct., Unit 6
Toronto, Ontario
M8Z 5T7

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December 12, 2025
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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation carried out for the proposed residential development at 1600 James Naismith Drive in Ottawa, Ontario (referred to further as the Site).

The purpose of the geotechnical investigation is to identify the general subsurface conditions at the Site by means of a limited number of boreholes and, based on the information obtained, to provide engineering guidelines and recommendations on the geotechnical aspects that might affect the planning of the of the project, including design and construction considerations.

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 Site Condition and Development

Currently, a multistory commercial structure is present at the Site. The surrounding areas have been paved for at grade / surficial parking and access roadways, with some landscaped areas. The proposed development will include the following components:

- A series of 3 to 3.5 storey residential buildings (townhouse blocks) with below ground basements. The approximate depth of basement floor slabs are about 1.1 to 2.6 metres below existing ground surface. The approximate depth of footings below existing ground surface will be about 2.7 to 4.2 metres;
- A series of 2 to 6 storey apartment buildings with one continuous level of below grade parking below them. The maximum anticipated depth of excavation for the parking garage is 3 metres below existing ground surface;
- Asphaltic concrete paved laneways and at-grade parking areas; and
- A storage tank, for Low Impact Development (LID) stormwater management will be installed with an underside depth at about 2.7 metres below ground surface.

The development will be serviced by sanitary and storm sewers, and watermains. It is understood that the maximum depth of the service installations (being sanitary) is about 4.1 metres.

2.2 Site Geology

Surficial geology maps of the Ottawa area indicate that the Site is underlain by deposits of silty clay over glacial till. Bedrock geology maps indicate the presence of shale of the Billings Formation below the soil cover. Fill material associated with the existing structures on the Site is also anticipated.

2.3 Previous Investigations by Others

In 2022 a geotechnical investigation was carried out at the Site by Pinchin which included five boreholes advanced on the eastern side of the property to depths ranging from about 2.6 to 3.5 metres.

The boreholes encountered surficial layers of topsoil and asphaltic concrete over fill material and native deposits of glacial till. Bedrock coring was not carried out, and bedrock level was inferred from increased resistance to insitu testing / augering. Standpipe piezometers (monitoring wells) were not installed for that investigation. The boreholes were observed to be dry upon completion.

The record of borehole logs from the Pinchin (2022) investigation are provided in Appendix A. The positions of the boreholes are shown on the Site Plan – Figure 1, following the text of this report.

3.0 METHODOLOGY

3.1 Geotechnical Investigation

The fieldwork for the GEMTEC geotechnical investigation was carried out on November 19, 2025. On that day, five boreholes identified as 25-01 to 25-05 inclusive, were advanced at the approximate locations shown on the Site Plan, Figure 1.

The boreholes were advanced using a truck mounted hollow stem auger drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec.

The boreholes were initially advanced using a hollow stem rotary auger to about the level of the bedrock. Below this level the boreholes were advanced using HQ sized triple tube rotary core drilling equipment. The boreholes were advanced to depths ranging from about 3.4 to 5.9 metres inclusive of bedrock coring.

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.

A single monitoring well was installed in the overburden in boreholes 25-01 and 25-05 to measure groundwater levels.

The fieldwork was supervised throughout by a member of our engineering staff who directed the drilling operations, logged the samples and carried out the in-situ testing. Following the completion of the drilling, the soil samples of soil and bedrock core samples were returned to our laboratory for examination by a geotechnical engineer and for laboratory testing. Selected samples of the soil were tested for water content and grain size distribution testing. Unconfined compressive strength testing was attempted on samples of the shale bedrock, however, these could not be completed due to the fractured and weak condition of the core. In addition, two soil samples were sent to an accredited analytical laboratory for sulphate and chloride testing to assess the corrosive potential of the soils to exposed concrete and steel.

The borehole locations were selected by GEMTEC and positioned on site relative to existing features. The ground surface elevations at the borehole locations were determined using precision GPS survey equipment. The elevations are referenced to geodetic datum NAD83 (CSRS) Epoch 2010, vertical network CGVD1928.

4.0 SUBSURFACE CONDITIONS

Descriptions of the subsurface conditions logged in the GEMTEC boreholes are provided on the Record of Borehole Sheets in Appendix B. The results of the laboratory classification testing are provided on the Record of Borehole Sheets and in Appendix C. The results of the chemical analysis on the selected soil samples are provided in Appendix D.

The sections below provide a description of the subsurface conditions encountered in the boreholes advanced as part of this investigation. The subsurface conditions are, in general, similar to those identified in Pinchin (2022).

4.1 Pavement Materials / Layers

Asphaltic concrete is present at ground surface at all of the borehole locations. The thickness of the asphaltic concrete ranges from about 70 to 100 millimetres.

Below the asphaltic concrete pavement layers, pavement base and subbase materials are present. These layers are generally composed of sand with varying amounts of gravel and trace silt. The combined thickness of the base and subbase layers range from about 340 to 530 millimetres.

The results of two grain size distribution tests carried out on sample of the pavement base / subbase layers are summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Test, Pavement Layers

Borehole ID	Sample Number	Gravel (%)	Sand (%)	Silt & Clay (%)
25-04	1	52.4	37.2	10.4
25-04	2	56.6	36.6	6.8

4.2 Fill Material

Fill material was encountered in boreholes 25-01, 25-03, and 25-04. The fill material extends to a depth of about 0.7 to 1.7 metres. The fill material is variable in composition but can generally be described as silty sand with variable amounts of gravel and clay. Occasional cobbles were also observed in the fill material.

Standard penetration tests (SPT) carried out in the fill material gave N values ranging from 19 to greater than 50 blows per 0.3 metres of penetration, which indicates a compact to very dense relative density. The higher N values may also be due to the presence of cobbles, boulders, or other hard material within the fill material.

The water content of one sample of the fill material is about 10 percent.

Although not identified in the boreholes, zones of deeper fill material are also likely present at the site, for instance as backfill to the existing building foundation walls and site services, which could not be investigated directly.

4.3 Glacial Till

A native deposit of glacial till was encountered below the pavement and fill materials in all of the boreholes. The glacial till is a heterogeneous mixture of all grain sizes, which at this Site, can be described as grey silty sand with some gravel. The glacial till deposits in this area are known to contain cobbles and boulders. The glacial till is present to depths of about 1.7 to 2.6 metres below surface grade, although as discussed in Section 4.4, the transition to the underlying shale bedrock is difficult to identify with certainty.

SPT N values in the glacial till range from 19 to greater than 50 blows per 0.3 metres of penetration, which indicates a compact to very dense relative density. The higher SPT values may also be caused by the presence of cobbles or boulders within the glacial till.

The results of one grain size distribution test carried out on a sample of the glacial till are summarized in Table 4.2. The water content of samples of the glacial till ranges from about 7 to 11 percent.

Table 4.2 – Summary of Grain Size Distribution Test, Glacial Till

Borehole ID	Sample Number	Gravel (%)	Sand (%)	Silt & Clay (%)
25-05	5	26.4	51.4	22.3

4.4 Shale Bedrock

Shale bedrock of the Billings Formation is present below the glacial till. The bedrock was encountered in the GEMTEC boreholes at depths ranging from 1.7 to 2.6 metres below surface grade (elevations ranging from 70.0 to 71.5 metres). However, it should be noted that the transition between the glacial till and the shale bedrock is likely not distinct, and an upper zone of highly fractured rock / cobbles and boulders is likely present at the base of the glacial till / top of rock level.

For comparison in the Pinchin (2022) investigation, borehole refusal which is likely to have occurred close to bedrock (but not necessarily at bedrock), occurred in the eastern portion of the Site at depths ranging from 2.6 to 3.5 metres. A local temporary benchmark was applied, and therefore accurate description of the borehole refusal level is not possible.

The bedrock was cored for 1.7 to 3.7 metres below the rock surface level using triple tube diamond drilling equipment. Within the bedrock core, the total core recovery (TCR) ranged from 12 to 100 percent, and the rock quality designation (RQD) ranged from 0 to 71 percent. According to the classification system in the Canadian Foundation Engineering Manual, 5th Edition, the bedrock at the borehole locations can be classified as very poor to fair quality. Photographs of the recovered core are provided in Appendix B with the associated borehole logs.

4.5 Groundwater Level

A monitoring well was installed in boreholes 25-01 and 25-05. The groundwater levels in the monitoring wells was measured on November 27, 2025, and are summarized in Table 4.3. The groundwater levels are only accurate for the date and time of measurement and may vary seasonally, in response to precipitation events, and due to nearby construction activities.

Table 4.3 – Summary of Groundwater Depth and Elevation

Borehole ID	Groundwater Depth Below Existing Ground Surface (metres)	Groundwater Elevation (metres, geodetic datum)	Date of Measurement	Response Zone
25-01	1.1	71.5	November 27, 2025	Glacial Till & Bedrock
25-05	1.8	71.7	November 27, 2025	Glacial Till & Bedrock

4.6 Chemistry Relating to Corrosion

The results of chemical testing on two soil samples recovered during the investigation are summarized in Table 4.4.

Table 4.4 – Soil Chemistry Related to Corrosion

Parameter	BH25-02 Sample 3	BH25-05 Sample 4
Resistivity (ohm.m)	31.6	14.1
pH	7.54	8.17
Chloride Content (ug/g)	<10	18
Sulphate Content (ug/g)	158	128

5.0 RECOMMENDATIONS

5.1 General

At the time of preparing this report, the final details for the development were not 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 Restriction

Based on the borehole information, there is no grade raise restrictions at the Site, from a geotechnical perspective. The settlement due to compression of the native soils as a result of fill placement should be relatively small and should occur during or shortly after the fill placement. Notwithstanding, should the proposed grade raise at the Site exceed three metres, more detailed assessment should be carried out by GEMTEC.

5.3 Billings Shale Considerations

The Site is within a region where Billings Formation shale is mapped as the bedrock type underlying the soils, and likely Billings Formation shale was encountered in the boreholes. In the Ottawa area, problems associated with swelling bedrock are mainly associated with the Billings Formation which contains pyrite which is known to breakdown to sulfides when exposed to air (oxygen) under some combinations of conditions. This can result in relatively rapid deterioration of the shale and heaving of the rock in the longer term which has been known to cause damage to structures.

Swelling bedrock can present a significant risk to the works at the Site and additional measures will be required during design and construction stages to reduce the risk of swelling from effecting the proposed structures. As preliminary general guidance;

- Where possible exposure of the bedrock to oxygen should be avoided. Any exposed bedrock should be covered by a layer of cementitious material (lean mix for example) as soon as practical upon exposure (but not later than within 24 hours of first exposure). Spray coating of exposed vertical excavation faces should be carried out within a similar timeframe to the top of the rock level;
- Dewatering can also cause swelling of the shale to occur. Further it should be considered that radius of influence of dewatering activities can extend beyond the perimeter of the Site and could trigger swelling of shale on adjacent properties;
- Heave from Billing Shale can cause basement floor cracking to occur. The risk can be reduced by not draining the groundwater level below the level of the shale bedrock and this should be considered in the selection of the levels of the basement floor slabs, and the design of the basement foundation walls (drained or watertight), and the type of floor slab, i.e. ground bearing or suspended.

- Where possible, the base level of structures should be planned to avoid groundwater level lowering in the rock (i.e. with floor slabs above bedrock) or the structures should be designed to reduce the risk of groundwater lowering (water tight basements), or the structure should be designed to reduce impact from heaving should it occur;
- Re-use of shale bedrock should not be considered due to the potential for the shale to swell once exposed to air. The swelling could cause damage to overlying structures including floor slabs, foundations, utilities and roadways in areas where shale is re-used as backfill;
- The oxidation reaction of pyrite in expansive shale can produce sulphuric acid. The sulphites can corrode unprotected buried steel elements, including, for example rock anchors. Any concrete in contact with the shale may also require use of sulphate resistant cement.

It should be noted that the above measures are intended to reduce not eliminate the risk associated with swelling bedrock conditions. Further discussion on risks and mitigation measures associated with Billings Formation is provided for specific aspects of the development in the subsections below, and review of the design is recommended as it is progressed.

5.4 Seismic Design of Proposed Structures

Based on the results of the current geotechnical investigation, Seismic Site Designation X_c / Site Class C as per Table 4.1.8.4.-B in the National Building Code of Canada (NBCC 2020) is applicable to the structures at the Site. GEMTEC considers it likely that a more favourable

Seismic Site Designation / Site Class could be applicable for the Site although additional testing using our MASW equipment would be required to justify this. GEMTEC can perform the required testing upon request. It is likely that augmenting the Site Class could result in considerable savings in the design and construction of the building. Consultation with a structural engineer is recommended to verify the likely benefits that could be obtained from a more favourable Seismic Site Designation / Site Class value.

There is no potential for liquefaction of the overburden supporting the structures, provided the recommendations provided in this report are followed.

5.5 Excavation

The excavations for the proposed development will be carried out through topsoil, pavement layers, fill material, glacial till and shale bedrock.

The soils are anticipated to be readily excavatable using conventional hydraulic excavation equipment, noting that large boulders may be encountered in the glacial till, and that the fractured bedrock may be encountered primarily as boulder sized fragments of rock. The presence of reinforced concrete and other hard materials / construction debris associated with the existing structures on the Site may also slow excavation.

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 2 and 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, excavation slopes for soils above the groundwater level. Excavation in the fractured bedrock should be considered in a similar manner to the soils, i.e. with battered side slopes.

Bedrock excavation should be anticipated. The upper portion of the shale bedrock (which is of poorer quality) can likely be carried out using larger hydraulic excavating equipment. Excavation into higher quality, stronger bedrock zones will likely require and hoe-ramming and possibly drill and blast techniques to be efficient. Where blasting is to be carried out precondition surveys of nearby existing structures is recommended, and if necessary, measurement of peak particle velocities at nearby structures (including services) could be carried out. Blast induced damage may require significant bedrock reinforcement to be implemented, unless blasting is carefully controlled, for instance by using line drilling on close centres along the excavation perimeter or other approaches.

Near vertical excavation batters will likely be feasible in the sound shale bedrock, noting that in general the bedrock core recovery is frequently fractured, as indicated by the low RQD values, and the above comments on the effects of bedrock blasting and therefore shallower batters may be required in general.

5.6 Groundwater Management

5.6.1 Temporary Excavation Dewatering

For temporary excavations to shallow depth in the fill material and glacial till, groundwater should be managed using typical construction methods by pumping within the excavations from filtered sumps. Fill material can be variable in composition, and increased groundwater management effort will be required should more granular / coarse grained soils be encountered. These may be present, for example, within zones of foundation wall backfill, bedding and surround to existing below ground services, and also pavement layers. Zones of more granular / coarse grained glacial till may also be encountered close to the rock level, and the upper surface of the rock is likely to be fractured and readily permit significant waterflow to excavations.

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 shallow excavation is the overburden will have a significant effect on nearby structures. However, dewatering of the Billings Formation shale can cause swelling of the shale to occur. Further it should be considered that radius of influence of dewatering activities can extend beyond the perimeter of the site could trigger swelling of shale on adjacent sites within the zone of influence of the dewatering operations. Therefore temporary groundwater level lowering in the shale should be avoided / minimised. If temporary drawing down of the water table is required, the exposed shale should be protected from exposure to air as described in Section 5.3 of this report.

The design and implementation of the temporary groundwater control systems is the responsibility of the contractor and the contractor should be required to submit a detailed work plan for review which specifically addresses the issues relating to the presence of potentially expansive rock.

5.6.2 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.

5.7 Foundation Design

5.7.1 Spread Footing Foundations

The topsoil and fill material layers are not suitable for support of structures and should be removed from below any foundations (and floor slab) support areas. The existing foundation elements and fill material associated with the existing structures on site will also need to be removed.

The proposed buildings can be founded on shallow spread footings bearing on or within the native glacial till, and upon the shale bedrock.. During construction the risk associated with exposure of Billings Shale to oxygen should be mitigated as described in Section 5.3.

After the removal of the existing fill material, where the subsequent subgrade surface is below the proposed founding level, the grade could be raised with compacted granular material or lean mix concrete (to maintain higher bearing for foundations over bedrock). The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 98 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.

Spread footing foundations should be sized using the preliminary net geotechnical reactions at Serviceability Limit States (SLS) and the factored net geotechnical resistances at Ultimate Limit States (ULS) provided in Table 5.1. Zones of weathered / highly fractured shale should be excavated to expose fresh shale which should be sealed as soon as practical as described in Section 5.3.

Table 5.1 – Summary of SLS and ULS Resistance Values

Subgrade Material	Net Geotechnical Reaction at Serviceability Limit States (SLS) [1]	Factored Net Geotechnical Resistance at Ultimate Limit States (ULS)
Glacial Till (compact or better) or compacted engineered fill over Glacial Till	150	300
Fresh Shale Bedrock or Lean Mix Concrete over Bedrock	n/a	1000

The post construction total and differential settlement of the footings on soil at SLS should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces. The post construction settlement of footing on shale bedrock should be less than 15 millimetres, provided any highly fractured bedrock zones containing significant soil seams are cleared from below the footings.

To reduce the potential for larger differential settlement / cracking, foundations supported on a mix of soil and bedrock should be avoided. If the foundations are required to span different materials the walls should be reinforced for a distance of 3 metres on both sides of the transition areas or as recommended by the structural engineer, in combination with other structural measures to reduce the risk of damage occurring.

5.8 Foundation Wall Backfill and Drainage

For the purposes of this report, it is assumed that a conventional drained basement construction will be implemented.

5.8.1 Foundation Backfill

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), 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. 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. A gradual transition (frost tapers) 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 should be sloped at 1 horizontal to 3 vertical, from 1.8 metres below finished grade to the underside of the granular subbase material for the hard surfaced areas.

In landscaped areas, if some settlement of the backfill is acceptable, frost susceptible native soils could be considered for foundation wall backfill purposes, 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. In these areas the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

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.

As previously indicated the use of excavated shale bedrock as backfill is not recommended, due to its potential to swell over time.

5.8.2 Drainage

Drainage of the foundation wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in a geotextile, which leads by gravity drainage to an adjacent storm sewer, or a sump from which it can be pumped.

The presence of Billings Shale should be considered in the design and construction of foundation drainage and sumps to avoid groundwater level lowering in the bedrock. Sumps for long term management of water should be made watertight to reduce the risk of local groundwater lowering occurring in the long term.

5.8.3 Rock Anchors

Depending on the building heights / loading conditions grouted rock anchors may be required for the proposed structure to resist uplift / lateral loads. It should be noted that some movement of the anchors is required to generate resistance unless prestressing of the anchors is carried out. The design, construction, and testing of anchors should be carried out in accordance with OPSS 942. 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; and,
- Failure of the steel tendon or top anchorage.

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

In accordance with *Recommendations for Prestressed Rock and Soil Anchors*, PTI document, a minimum bonded length of 3.0 metres should be provided. Long bonded anchor lengths should be avoided (i.e., maximum 8 metres). SLS movement in the anchor can be determined from the elastic elongation of the unbonded portion of the tendon under design load. Double corrosion protection of anchors should be specified.

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

5.9 Lateral Earth Pressures

Foundation should be designed to resist “at rest” earth pressures. Heavy construction traffic should not be allowed to operate adjacent to foundation walls for the proposed building (within about 2 metres horizontal) during construction, without the approval of the designers.

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 walls that are backfilled with granular material such as that meeting OPSS Granular B Type I or II requirements the soil parameters provided in Table 5.2 can be used to calculate the at rest thrust components acting on the wall.

Table 5.2 – 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)	22	22

Parameter	OPSS Granular B Type I	OPSS Granular B Type II
Internal Friction Angle (degrees)	36	38
"At Rest" Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.41	0.38
"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	0.37[1]	0.34[1]

Notes:

1) According to the 2020 National Building Code of Canada, the peak ground acceleration (PGA) with a 2% probability of exceedance in 50 years for the Site is 0.365 (for Seismic Site Designation X_c / 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 0.183 (taken as the corrected PGA and assuming flexible wall conditions) and assuming that the vertical seismic coefficient, k_v, is zero.

5.10 Basement Floor Slabs

For the purposes of this report, it is assumed that a conventional (ground bearing/supported) floor slabs will be implemented in combination with conventional drained basement construction.

5.10.1 Conventional Basement Floor Slabs

For predictable performance of the slabs, all surficial topsoil, fill material, and any soft, wet, disturbed or deleterious materials should be removed from the subgrade surface, and the subgrade surface should be proof rolled with a large steel drum roller under dry conditions. The base of the floor slab should consist of at least 300 millimetres of 19 millimetre clear crushed stone. This may require shallow bedrock excavation in some zones – but is recommended to obtain a more uniform support to the slab, and also for drainage purposes.

Some of the proposed structures will be partially or fully located within the footprint of the existing structure, and as such, fill material should be anticipated to depths which exceed those identified in the borehole logs. After the removal of the existing fill material, where the subsequent subgrade surface is below the required level, the grade could be raised with either 19 millimetre clear crushed stone or OPSS Granular B Type II.

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 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. The use of site won crushed shale should not be permitted.

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 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 slab.

5.10.2 Drainage

Provided clear crushed stone is used below the basement floor slab areas, 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.

Groundwater level lowering in the shale bedrock should be avoided in the long term. Alternatives which may be considered to address this include selection of the levels of the basement floor slabs to avoid groundwater lowering in the shale bedrock, design of the basement as a watertight structure, or design of the floor slab as suspended (to allow some swelling to occur without damage).

5.11 Frost Protection

5.11.1 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.

5.11.2 Floor Slabs

The native soils at the Site are susceptible to heaving during freezing conditions. Concrete floor slabs in buildings which are to remain unheated during the winter period, and any exterior slab on grade slabs, will require thermal protection to avoid cracking and distortion of the slab (assuming that excavation and replacement below the slab is not carried out). Further details on the insulation requirements could be provided, if necessary. This would not be required in heated buildings.

5.12 Site 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.

5.12.1 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.

Bedrock excavation should be anticipated, based on the anticipated lowest invert depth identified.

5.12.2 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.

5.12.3 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. This will reduce the potential for differential frost heaving between the area over the trench and the adjacent roadway. 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.

5.12.4 Material Reuse

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Topsoil, fill material, or other organic material should be wasted from the trench. Depending on the weather conditions at the time of construction, some wetting of the excavated native materials could occur. Where used in such a condition, some settlement should be expected. Alternatively, consideration could be given to implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to placement and compaction;
- Reuse any wet materials in the lower part of the trenches and make provision to defer installation of hard surfacing for 3 months, or longer, to allow the trench backfill settlement to occur followed by additional compaction and levelling, thereby improving the final performance of the hard surfaced area.

The reuse of excavated shale bedrock within the trench excavations should not be permitted.

5.12.5 Seepage Barriers

The use of seepage barriers is not considered necessary at this Site for geotechnical purposes, but may may serve to reduce the risk of groundwater level lowering and associated impacts to the shale bedrock.

If implemented, the seepage barriers should begin at the base of the excavation and extend vertically through granular pipe bedding and granular surround, to the upper surface of the native clay exposed along the sides of the excavation, and horizontally across the full width of the service trench excavation.

The seepage barriers could consist of 1.5 metre wide dykes of compacted native clay. The clay 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 locations of the seepage barriers could be provided as the design progresses.

Potential locations for seepage barriers can be identified once further information on the alignment and levels of the services / utilities are known.

5.12.6 LID Storage Tank

No details of the LID storage tank were available to GEMTEC at the time of preparing this report. Refer to Section 5.5 and 5.6 for general commentary on excavation and groundwater management. Further commentary can be provided as the design progresses.

5.13 Access Roadway/Parking Lot Areas

Information on the proposed access roadway and parking area configuration was not available at the time of preparing this report. It is anticipated that a mixture of at grade / surficial parking and basement parking areas will be provided in combination with a series of access roadways. More tailored guidelines can be provided as further information becomes available.

5.13.1 Subgrade Preparation – Surficial Parking

In preparation for access roadway/parking lot construction at the Site, all surficial topsoil, and any soft, wet or deleterious materials should be removed from the proposed roadway areas.

Prior to placing granular material for the surficial roads and parking lots, the exposed subgrade should be heavily proof rolled under suitable (dry) conditions and inspected and approved by geotechnical personnel. Any soft areas should be subexcavated and replaced with suitable (dry) Earth Borrow, or Select Subgrade Material that is frost compatible with the materials exposed on the sides of the area of subexcavation.

Where it will be necessary to raise the roadway/parking lot grades at this Site, material which meets OPSS specifications for Select Subgrade Material, Earth Borrow, may be used. This may include materials currently on site associated with the existing pavements – subject to further detailed inspection and testing. The reuse of excavated shale bedrock within the roadway and parking areas should not be permitted.

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 standard Proctor maximum dry density value using vibratory compaction equipment. Rock fill should be placed in maximum 500 millimetre thick lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both. The subgrade should be shaped and crowned to promote drainage of the roadway granular materials.

Truck traffic should be avoided on the native soil subgrade or the trench backfill within the roadways/parking lot areas especially under wet conditions.

5.13.2 Pavement Structure

5.13.2.1 Light Duty Pavement

The following minimum pavement structure is suggested for local roadways and parking lots that will be used by light traffic:

- 80 millimetre thick layer of asphaltic concrete, comprising
    - Two layers of Superpave 12.5 Traffic Level B with PG 58-34 asphalt cement each 40 millimetres in thickness
- 150 millimetre thick layer of base (OPSS Granular A); over
- 300 millimetre thick layer of subbase (OPSS Granular B Type II);

5.13.2.2 Heavy Duty Pavement

For roadways which will be subject to routine truck or bus traffic, or which will be used as fire access routes, the suggested minimum pavement structure is:

- 100 millimetres of hot mix asphaltic concrete (40 millimetres of Superpave 12.5 (Traffic Level B) over 60 millimetres of Superpave 19.0 (Traffic Level B), over
- 150 millimetres of OPSS Granular A base over
- 450 millimetres of OPSS Granular B, Type II subbase

It is recommended that the above pavement structure also be utilized for any access ramps to below ground parking areas.

5.13.2.3 Below Ground / Basement Pavement

The basement area may be used for below ground parking utilising a concrete slab over a drainage layer as described previously. GEMTEC can review the composition of these components as the design progresses and as information on traffic loading is available.

5.13.3 Granular Material Placement

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

5.13.4 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.

5.14 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the samples of soil range from 128 to 158 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. Notwithstanding, concrete in contact with the shale, or below the bedrock surface should be batched using sulphate resistance Portland cement i.e. MS or MSb 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 non-aggressive to 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.

However, it should be noted that the oxidation reaction of pyrite in expansive shale can produce sulphuric acid. The sulphites can corrode unprotected buried steel elements, including, for example rock anchors. Adequate protection of rock anchor and any other steel elements should therefore be provided.

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 due to excavation and compaction is expected to be much less than that required to cause damage to the nearby structures or services. Refer to Section 5.5 if bedrock blasting will be carried out at the Site.

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 Well Abandonment

The monitoring wells installed in boreholes 25-01 and 25-03 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.

6.4 Frost Tapers

In addition to the locations identified in this report frost tapers should also be considered where existing non-frost susceptible material associated with existing structures at the site abut soil which are prone to frost heaving, should it be preferable to avoid differential ground movement at these locations. The frost tapers should be sloped at 3 horizontal to 1 vertical, or flatter, where possible.

6.5 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.

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.

Daire Cummins

Daire Cummins M.Sc.
Geotechnical Analyst

Brent Wiebe, P.Eng.
Principal Geotechnical Engineer

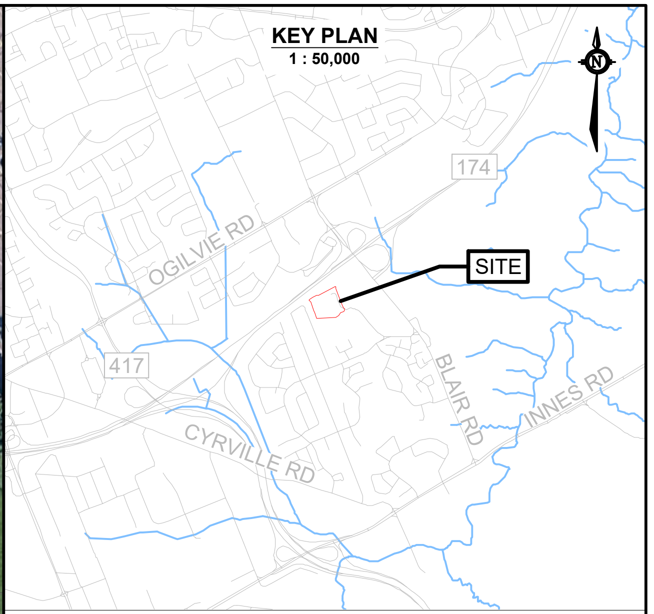
LICENSED PROFESSIONAL ENGINEER
B.D. WIEBE
100060438
Dec 12/25
PROVINCE OF ONTARIO

DC/BW

Enclosures
N:\Projects\103700\103752.002\05_Technical Work\Reporting\103752.002_RPT_GEOT_1600 James Naismith Dr_2025-12-12_rev0.docx

KEY PLAN

1 : 50,000



LEGEND

BH # ← BOREHOLE ID

XX.XX ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

◒ BOREHOLE LOCATION (current investigation by GEMTEC)

◒ BOREHOLE LOCATION (previous investigation by PINCHIN)

— APPROXIMATE SITE BOUNDARY

DATA SOURCES AND REFERENCES

1. Coordinate system: CSRS.MTM-9
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

1:1,000

0 20 40 60m

DRAWING

SITE PLAN

CLIENT

MORETON PROPERTIES LTD.

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1600 JAMES NAISMITH DRIVE
OTTAWA, ONTARIO

DRAWN BY SL	CHECKED BY MR
PROJECT NO. 103752.002	REVISION NO. 0
DATE DECEMBER 2025	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

N:\PROJECTS\103700\103752.002\06_CIVIL DRAFTING\GSP R0\103752.002_GSP_R0_2025_12.DWG

GEOTECHNICAL REPORT CONDITIONS & LIMITATIONS

STANDARD OF CARE: GEMTEC has prepared this report in a manner consistent with generally accepted engineering or environmental consulting practice in the jurisdiction in which the services are provided at the time of the report. No other warranty, expressed or implied is made.

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COMPLETE REPORT: This report is of a summary nature and is not intended to stand alone without reference to the instructions given to GEMTEC by the Client, communications between GEMTEC and the Client and to any other reports prepared by GEMTEC for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. GEMTEC can not be responsible for use of portions of the report without reference to the entire report.

BASIS OF REPORT: This Report has been prepared for the specific site, development, design objectives and purposes that were described to GEMTEC by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this report expressly addresses the proposed development, design objectives and purposes. Any change of site conditions, purpose or development plans may alter the validity of the report and GEMTEC cannot be responsible for use of this report, or portions thereof, unless GEMTEC is requested to review any changes and, if necessary, revise the report.

TIME DEPENDENCE: If the proposed project is not undertaken by the Client within 18 months following the issuance of this report, or within the timeframe understood by GEMTEC to be contemplated by the Client, the guidance and recommendations within the report should not be considered valid unless reviewed and amended or validated by GEMTEC in writing.

USE OF THIS REPORT: The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without GEMTEC's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, GEMTEC may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

NO LEGAL REPRESENTATIONS: GEMTEC makes no representations whatsoever concerning the legal significance of its findings, or as to other legal matters touched on in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and change. Such interpretations and regulatory changes should be reviewed with legal counsel.

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.

APPENDIX A

Record of Borehole Logs Pinchin (2022)

Log of Borehole: BH1

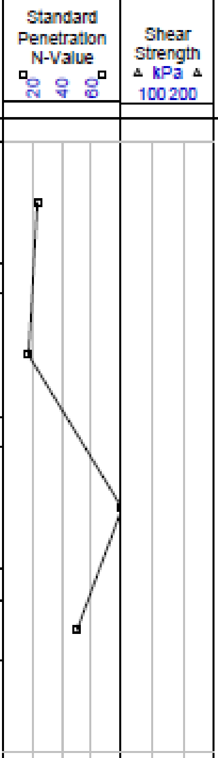
Project #: 306147 **Logged By:** MK

Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 10, 2022 **Project Manager:** WT



SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength kPa	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	99.21	No Monitoring Well Installed										
		Asphalt ~ 150 mm												
		Fill Sand and gravel, some silt, brown, damp, compact	98.45		SS	1	60	23						GS
1		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, compact	97.69		SS	2	80	17						
2		Trace to some shale bedrock fragments, very dense	96.62		SS	3	100	81						
					SS	4	100	50						
3		End of Borehole Borehole was terminated at 2.59 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												
4														

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.21 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH2

Project #: 306147 **Logged By:** MK

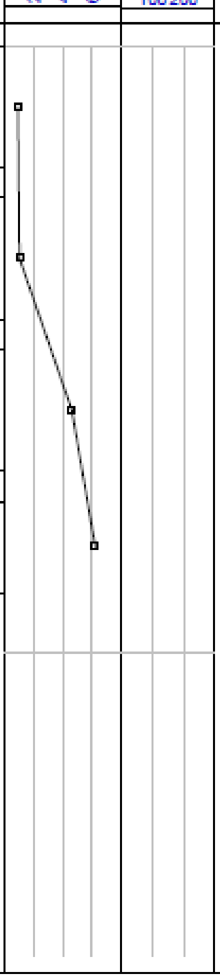
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 10, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength kPa	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.77											
0		**Organics** ~ 300 mm	99.47	No Monitoring Well Installed										
		Glacial Till Silty gravelly sand, trace clay, brown, damp, loose to compact			SS	1	30	9						
1					SS	2	70	10						
		Trace to some shale bedrock fragments, dense to very dense	98.09		SS	3	80	46						
2			97.03		SS	4	100	62						
		End of Borehole Borehole was terminated at 2.74 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												
3														
4														



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.77 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH3

Project #: 306147　　*Logged By:* MK

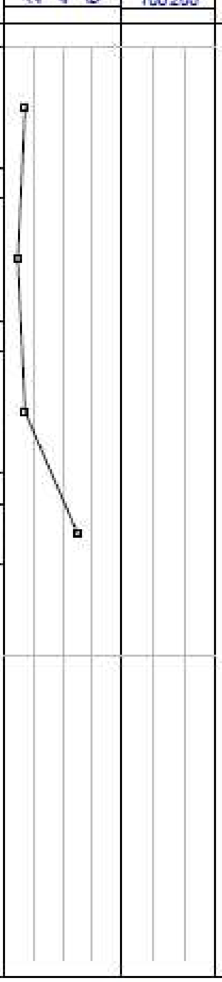
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 10, 2022　　*Project Manager:* WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength ▲ kPa ▲ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.31											
0		**Asphalt** ~ 150 mm		No Monitoring Well Installed										
		Fill Sand and gravel, some silt, brown, damp, compact	98.45		SS	1	60	14						
1		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, loose to compact			SS	2	80	9						
2			96.92		SS	3	100	14			6.6			Hyd. MC
		Trace to some shale bedrock fragments, very dense	96.72		SS	4	100	50						
3		End of Borehole												
4		Borehole was terminated at 2.59 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												



Contractor: Canadian Environmental Drilling & Contractors Inc.　　*Grade Elevation:* 99.21 m

Drilling Method: Hollow Stem Auger / Split Spoon　　*Top of Casing Elevation:* N/A

Well Casing Size: N/A　　*Sheet:* 1 of 1

Log of Borehole: BH4

Project #: 306147 **Logged By:** MK

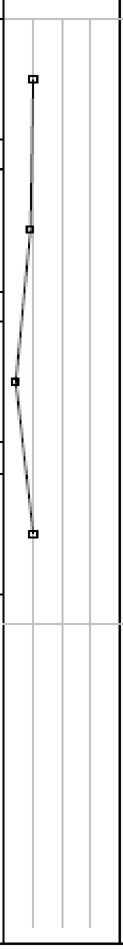
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 11, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	100.31	No Monitoring Well Installed										
		Asphalt ~ 100 mm												
		Fill Sand and gravel, trace silt, brown, damp, compact			SS	1	40	20						GS
1		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, loose to compact	99.24		SS	2	60	18						
2					SS	3	40	8						
					SS	4	75	20						
3		End of Borehole Borehole was terminated at 2.90 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.	97.41											
4														



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 100.31 m

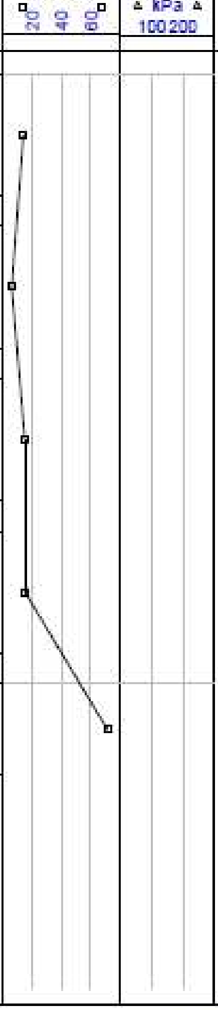
Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH5

Project #: 306147 **Logged By:** MK
Project: Geotechnical Investigation
Client: 1600 James Naismith LP
Location: 1600 James Naismith Drive, Ottawa, ON
Drill Date: May 11, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength ▲ kPa ▲ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	99.99	No Monitoring Well Installed										
		Organics ~ 300 mm	99.68											
1		**Fill** Sand and gravel, trace silt, trace organics, brown, damp, loose to compact	98.77		SS SS	1 2	30 50	14 6						
2 3		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, compact to very dense	96.48		SS SS SS	3 4 5	80 80 60	15 15 71						
4		End of Borehole Borehole was terminated at 3.51 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.99 m
Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A
Well Casing Size: N/A **Sheet:** 1 of 1

APPENDIX B

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

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

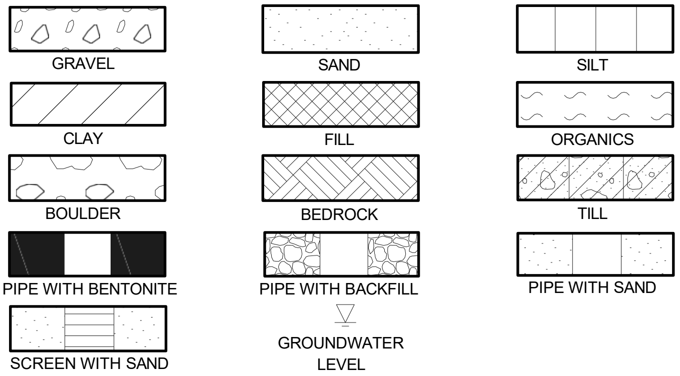
PENETRATION RESISTANCE

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.

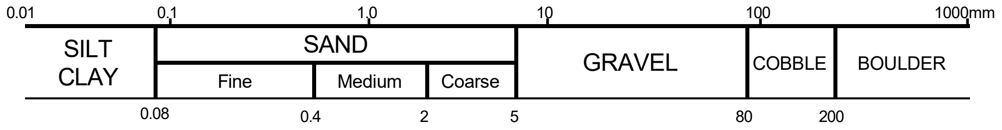
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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.

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE	
Fresh	No visible sign of rock material weathering
Faintly weathered	Weathering limited to the surface of major discontinuities
Slightly weathered	Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material
Moderately weathered	Weathering extends throughout the rock mass but the rock material is not friable
Completely weathered	Rock is wholly decomposed and in a friable condition but the rock and structure are preserved

CORE CONDITION
Total Core Recovery (TCR) The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run
Solid Core Recovery (SCR) The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.
Rock Quality Designation (RQD) The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completed broken core to 100% for core in solid segments.

BEDDING THICKNESS	
Description	**Thickness**
Thinly laminated	< 6 mm
Laminated	6 - 20 mm
Very thinly bedded	20 - 60 mm
Thinly bedded	60 - 200 mm
Medium bedded	200 - 600 mm
Thickly bedded	600 - 2000 mm
Very thickly bedded	2000 - 6000 mm

DISCONTINUITY SPACING	
Description	**Spacing**
Very close	20 - 60 mm
Close	60 - 200 mm
Moderate	200 - 600 mm
Wide	600 -2000 mm
Very wide	2000 - 6000 mm

ROCK QUALITY	
RQD	**Overall Quality**
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

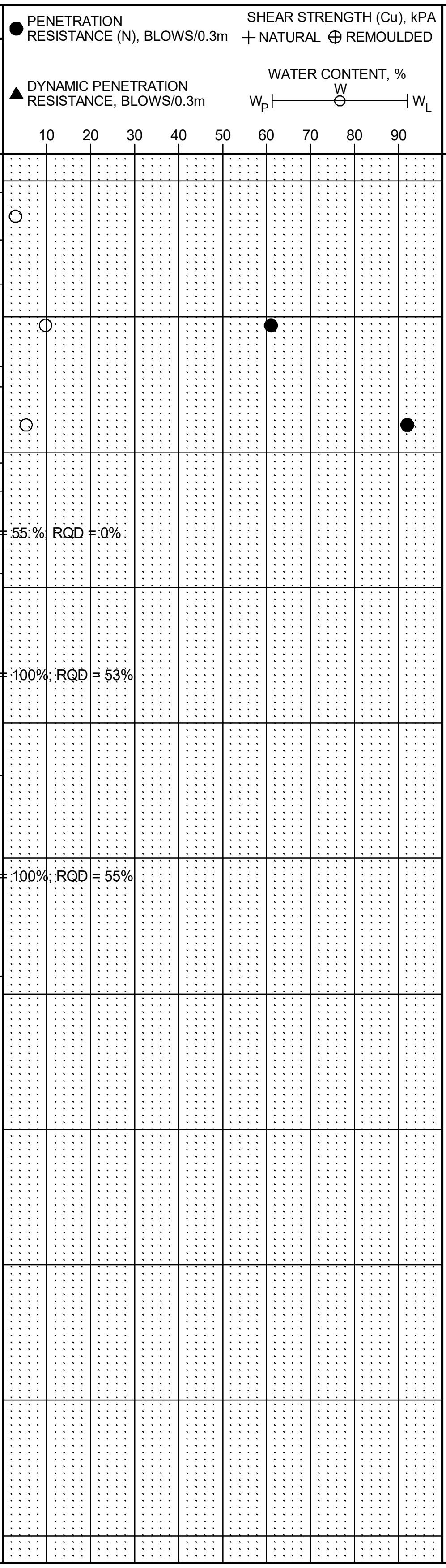
ROCK COMPRESSIVE STRENGTH	
Comp. Strength, MPa	**Description**
1 - 5	Very weak
5 - 25	Weak
25 - 50	Moderate
50 - 100	Strong
100 - 250	Very strong

RECORD OF BOREHOLE 25-01

CLIENT: Moreton Properties Ltd.
PROJECT: Geotechnical Investigation - Proposed Residential Development
JOB#: 103752.002
LOCATION: See Borehole Location Plan, Figure 1

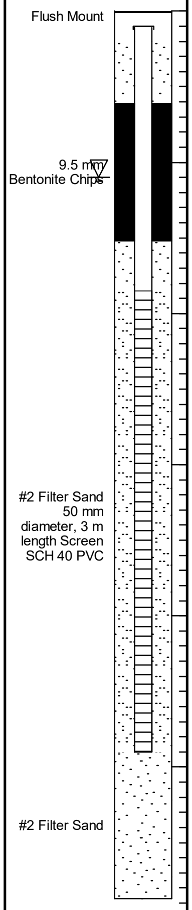
SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 19 2025

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
	Ground Surface	72.64				
Power Auger 200 mm O.D. Hollow Stem	Asphaltic concrete	72.56 0.09				
	BASE - (SP-GP) sand and gravel, trace silt, grey; non-cohesive, moist	72.21 0.43	1	GS		
	FILL - (SM) silty sand, trace clay, some gravel, grey brown and reddish brown; non-cohesive, moist	70.94 1.70	2	SS	230	61
114 mm O.D. HW Casing	(SM) SILTY SAND, some gravel, grey to grey brown & dark grey, (GLACIAL TILL); non-cohesive, moist, very dense. Containing cobbles and boulders.	70.04 2.60	3	SS	560	92
Diamond Rotary 96 mm O.D. HQ3 Triple Tube	Black, fresh, laminated to thinly bedded, SHALE bedrock		4	RC		TCR = 55 %, RQD = 0%
			5	RC		TCR = 100%; RQD = 53%
		66.77 5.87	6	RC		TCR = 100%; RQD = 55%
	End of borehole					



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/11/27	1.10 ▽	71.5

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LOGGED: A.N.

CHECKED:

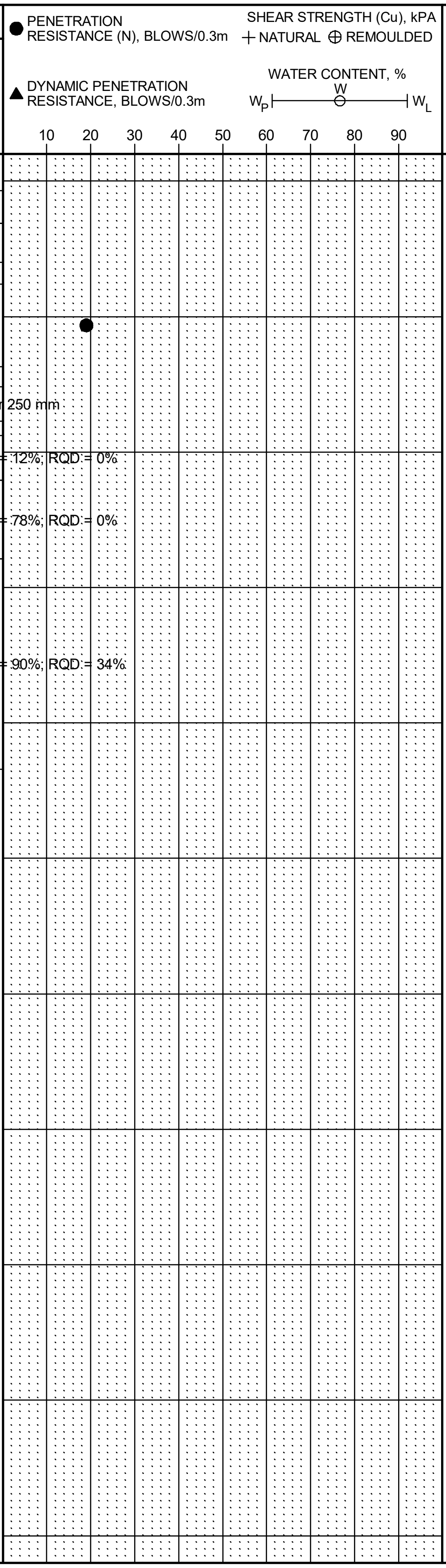
RECORD OF BOREHOLE 25-02

CLIENT: Moreton Properties Ltd.
PROJECT: Geotechnical Investigation - Proposed Residential Development
JOB#: 103752.002
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 29 2025

BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)
	Ground Surface	72.88
Power Auger 200 mm O.D. Hollow Stem	Asphaltic concrete	72.81 0.07
	BASE - (SP-GP) sand and gravel, trace silt, grey; non-cohesive, moist	72.57 0.31
	SUBBASE - (SP-SM) gravelly sand, trace silt, grey; non-cohesive, moist	72.28 0.60
	(SM) SILTY SAND, some gravel, grey brown & dark grey, (GLACIAL TILL); non-cohesive, moist, compact to dense. Containing cobbles and boulders.	70.67 2.21
Diamond Rotary 96 mm O.D. HQ3 Triple Tube	Black, fresh, laminated to thinly bedded, SHALE bedrock	68.54 4.34
	End of borehole	

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	GS		
2	GS		
3	SS	310	19
4	SS	0	72 for 250 mm
5	RC		TCR = 12%; RQD = 0%
6	RC		TCR = 78%; RQD = 0%
7	RC		TCR = 90%; RQD = 34%



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Asphaltic Cold Patch
Auger Cuttings
9.5 mm Bentonite Chips

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

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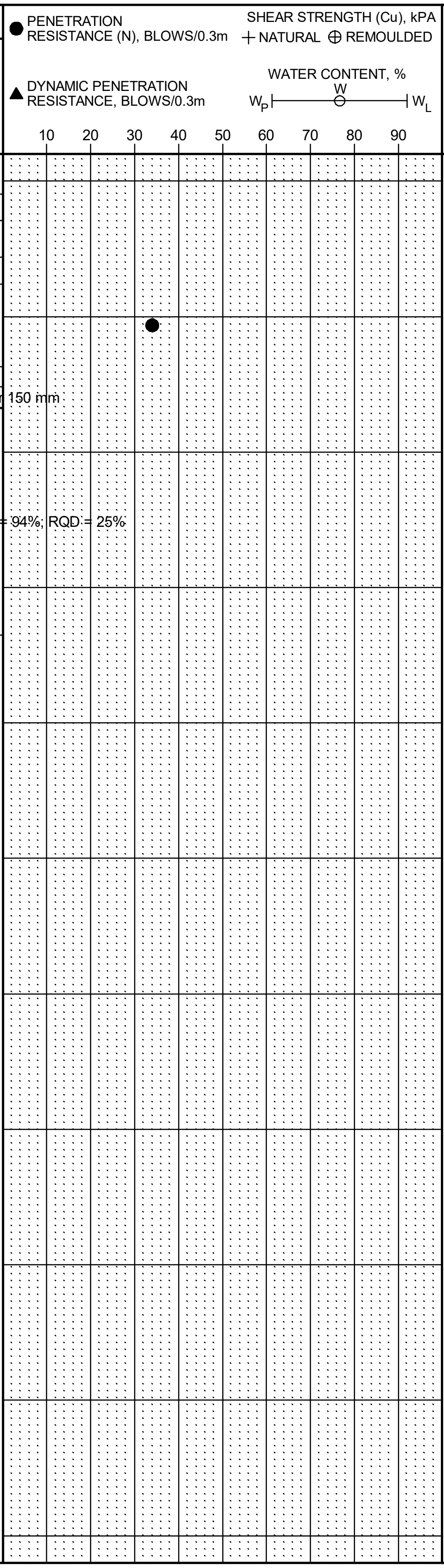
RECORD OF BOREHOLE 25-03

CLIENT: Moreton Properties Ltd.
PROJECT: Geotechnical Investigation - Proposed Residential Development
JOB#: 103752.002
LOCATION: See Borehole Location Plan, Figure 1

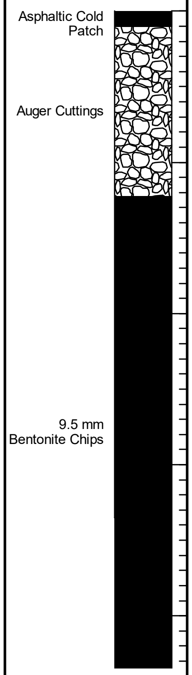
SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 20 2025

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
		Ground Surface	73.06				
0	Power Auger 200 mm O.D. Hollow Stem	Asphaltic concrete	72.97 0.10				
		BASE - (SP-GP) sand and gravel, trace silt, grey; non-cohesive, moist	72.77 0.29	1	GS		
		SUBBASE - (SP-GP) sand and gravel, trace silt, grey; non-cohesive, moist	72.50 0.56	2	GS		
		FILL - (SM) silty sand, some gravel, occasional cobble, brown; non-cohesive, moist, dense	72.33 0.73				
1		(SM) SILTY SAND, some gravel, grey brown & dark grey, (GLACIAL TILL); non-cohesive, moist, dense. Containing cobbles and boulders.		3	SS	230	34
			71.38 1.68	4	SS	0	50 for 150 mm
2	Diamond Rotary 96 mm O.D. HQ3 Triple Tube	Black, fresh, laminated to thinly bedded, SHALE bedrock		5	RC		TCR = 94%; RQD = 25%
3			69.71 3.35				
		End of borehole					
4							
5							
6							
7							
8							
9							
10							



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



GEO - BOREHOLE LOG 103752.002_BH LOGS_2025-11-24.GPJ GEMTEC 2018.GDT 12/10/25

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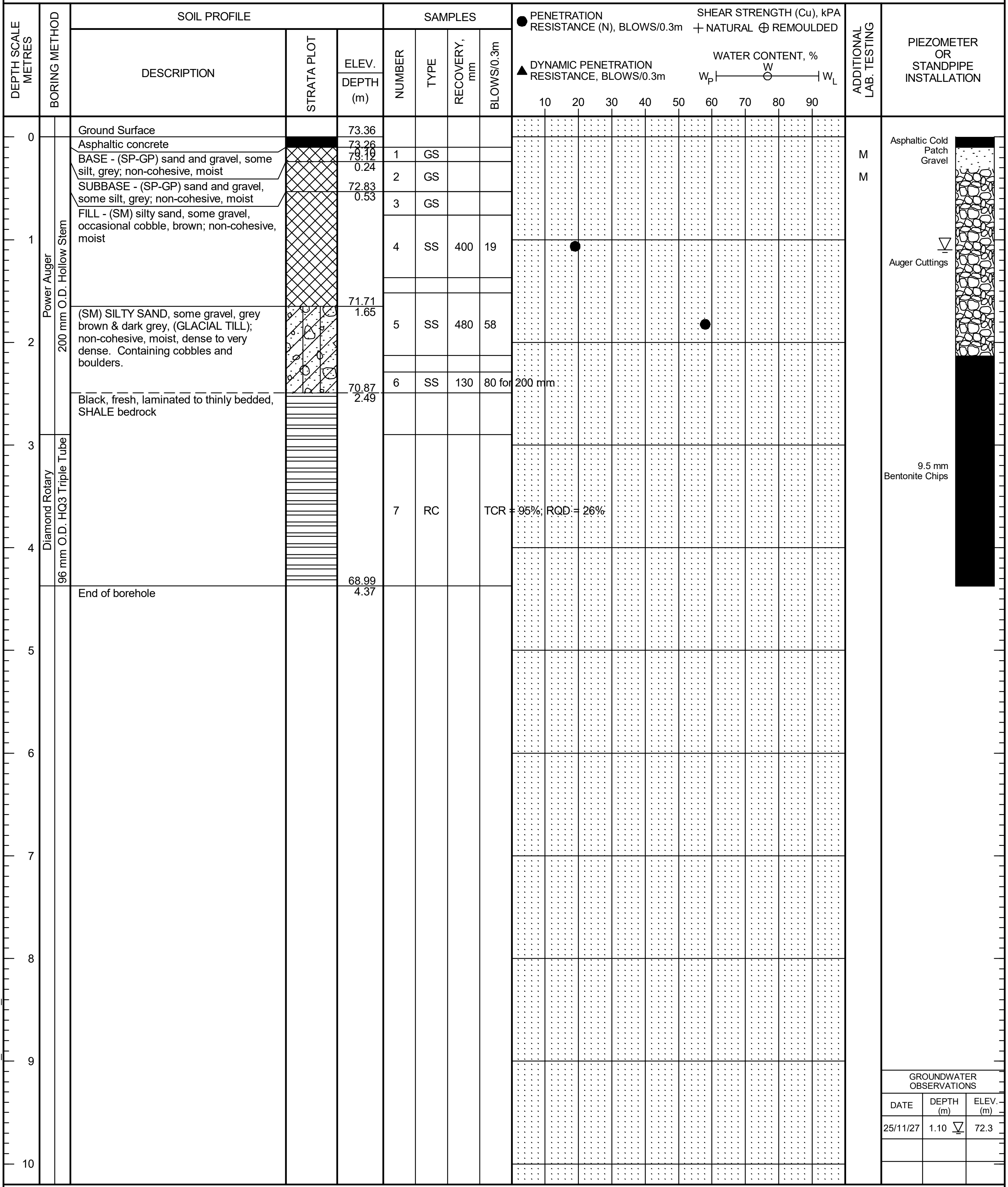
RECORD OF BOREHOLE 25-04

CLIENT: Moreton Properties Ltd.
PROJECT: Geotechnical Investigation - Proposed Residential Development
JOB#: 103752.002
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 19 2025



DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	73.36
Asphaltic concrete	73.26 0.10
BASE - (SP-GP) sand and gravel, some silt, grey; non-cohesive, moist	73.12 0.24
SUBBASE - (SP-GP) sand and gravel, some silt, grey; non-cohesive, moist	72.83 0.53
FILL - (SM) silty sand, some gravel, occasional cobble, brown; non-cohesive, moist	71.71 1.65
(SM) SILTY SAND, some gravel, grey brown & dark grey, (GLACIAL TILL); non-cohesive, moist, dense to very dense. Containing cobbles and boulders.	70.87 2.49
Black, fresh, laminated to thinly bedded, SHALE bedrock	68.99 4.37
End of borehole	

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	GS			M
2	GS			M
3	GS			
4	SS	400	19	
5	SS	480	58	
6	SS	130	80 for 200 mm	
7	RC		TCR = 95%; RQD = 26%	

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/11/27	1.10 ∇	72.3

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CHECKED:

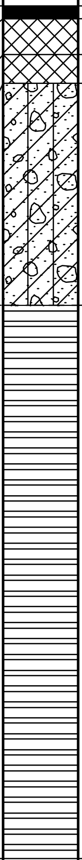
RECORD OF BOREHOLE 25-05

CLIENT: Moreton Properties Ltd.
PROJECT: Geotechnical Investigation - Proposed Residential Development
JOB#: 103752.002
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 20 2025

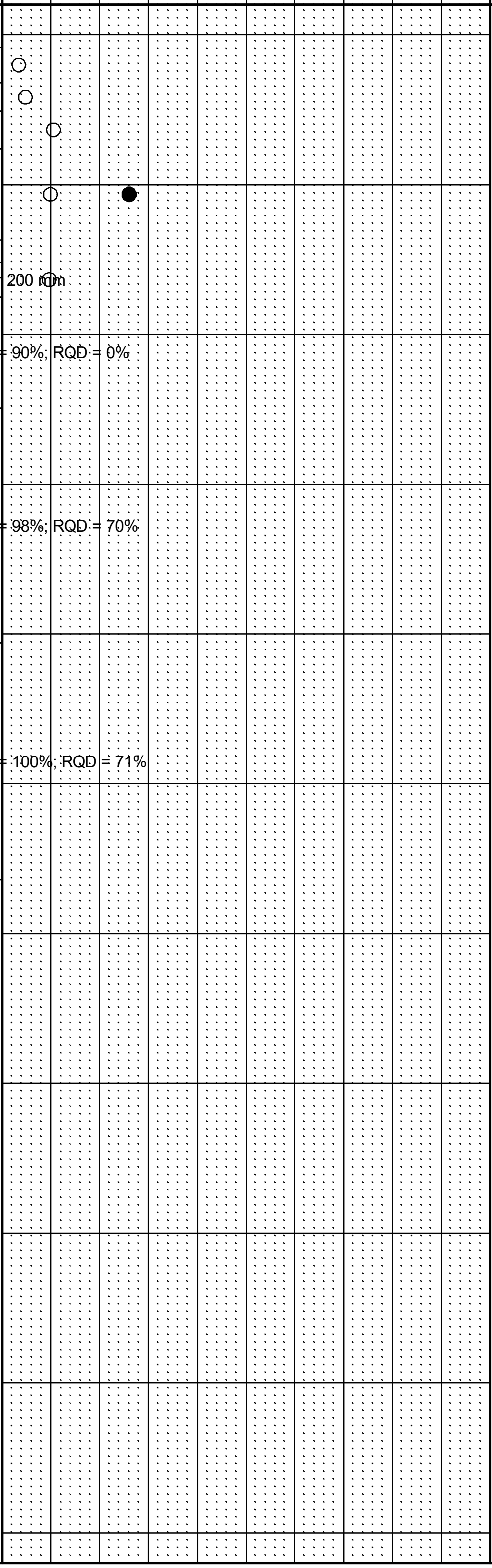
BORING METHOD: Power Auger, 200 mm O.D. Hollow Stem (0 to 1.64 m); 114 mm O.D. HW Casing (1.64 to 1.98 m); Diamond Rotary, 96 mm O.D. HQ3 Triple Tube (1.98 to 5.64 m)

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	73.53
Asphaltic concrete	73.45 0.09
BASE - (SP-GP) sand and gravel, trace silt, grey; non-cohesive, moist	73.21 0.32
SUBBASE - (SP-GP) sand and gravel, trace silt, grey; non-cohesive, moist	73.02 0.51
(SM) SILTY GRAVELLY SAND, grey brown & dark grey, (GLACIAL TILL); non-cohesive, moist, compact to dense. Containing cobbles and boulders.	71.55 1.98
Black, fresh, laminated to thinly bedded, SHALE bedrock	67.89 5.64
End of borehole	

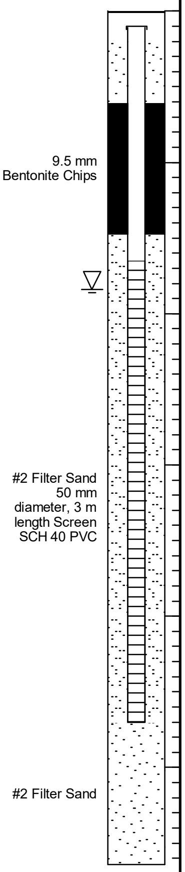


NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	GS			
2	GS			
3	GS			
4	SS	250	26	
5	SS	350	81 for 200 mm	M
6	RC		TCR = 90%; RQD = 0%	
7	RC		TCR = 98%; RQD = 70%	
8	RC		TCR = 100%; RQD = 71%	

● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P ⊢—○—⊣ W_L



PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/11/27	1.84 ▽	71.7

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BOREHOLE: BH25-01 (DRY)
BORING DATE: NOVEMBER 19, 2025
DEPTH: 2.29 TO 5.87 METRES BELOW GROUND SURFACE

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Drawing
ROCKCORE PHOTOGRAPH
BOREHOLE BH25-01 (DRY)

FIGURE B1

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103752.002	SL	DC	0	DECEMBER 2025

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BOREHOLE: BH25-01 (WET)
BORING DATE: NOVEMBER 19, 2025
DEPTH: 2.29 TO 5.87 METRES BELOW GROUND SURFACE

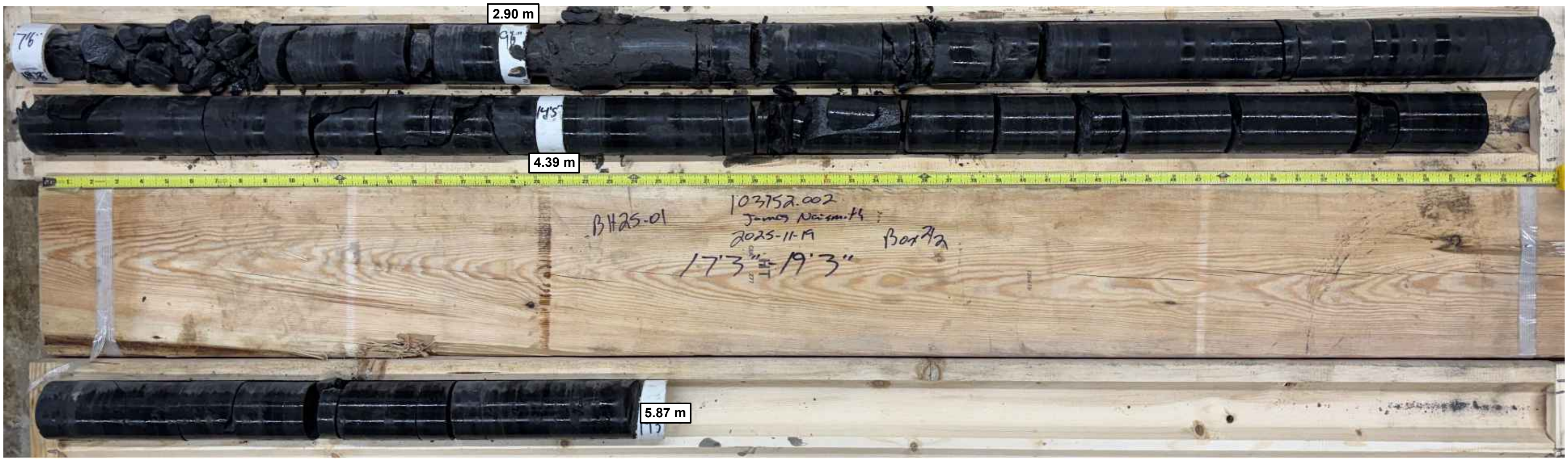

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FIGURE B2

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BOREHOLE: BH25-02 (DRY)
BORING DATE: NOVEMBER 19, 2025
DEPTH: 1.88 TO 4.34 METRES BELOW GROUND SURFACE

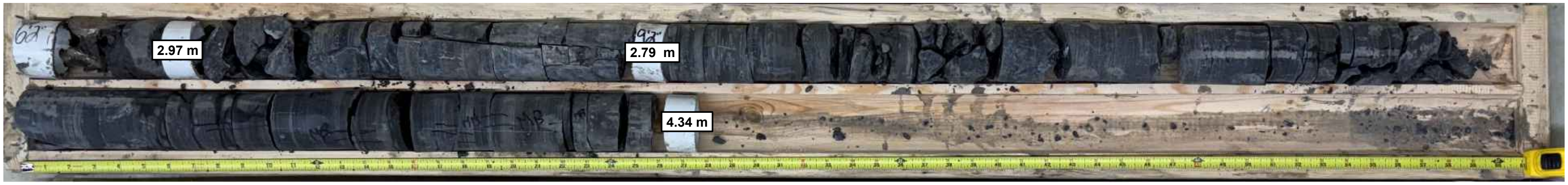

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FIGURE B3

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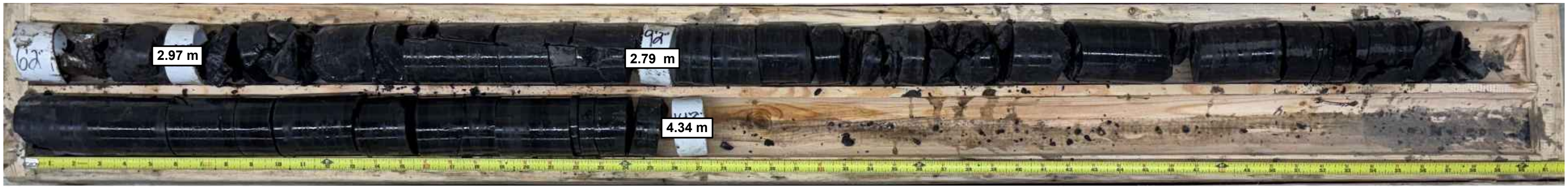
BOREHOLE: BH25-02 (WET)
BORING DATE: NOVEMBER 19, 2025
DEPTH: 1.88 TO 4.34 METRES BELOW GROUND SURFACE



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BOREHOLE BH25-02 (WET)

FIGURE B4

Project No.	Drwn By	Chkd By	Rev No.	Date
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BOREHOLE: BH25-03 (DRY)
BORING DATE: NOVEMBER 20, 2025
DEPTH: 1.68 TO 3.35 METRES BELOW GROUND SURFACE

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Project	MORETON PROPERTIES LTD. GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL DEVELOPMENT 1600 JAMES NAISMITH DRIVE OTTAWA, ONTARIO
Drawing	ROCKCORE PHOTOGRAPH BOREHOLE BH25-03 (DRY)
Figure	**FIGURE B5**

Project No.	Drwn By	Chkd By	Rev No.	Date
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BOREHOLE: BH25-03 (WET)
BORING DATE: NOVEMBER 20, 2025
DEPTH: 1.68 TO 3.35 METRES BELOW GROUND SURFACE

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Drawing
ROCKCORE PHOTOGRAPH
BOREHOLE BH25-03 (WET)

FIGURE B6

Project No.	Drwn By	Chkd By	Rev No.	Date
103752.002	SL	DC	0	DECEMBER 2025

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BOREHOLE: BH25-04 (DRY)
BORING DATE: NOVEMBER 19, 2025
DEPTH: 2.90 TO 4.37 METRES BELOW GROUND SURFACE

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ROCKCORE PHOTOGRAPH
BOREHOLE BH25-04 (DRY)

FIGURE B7

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BOREHOLE: BH25-04 (WET)
BORING DATE: NOVEMBER 19, 2025
DEPTH: 2.90 TO 4.37 METRES BELOW GROUND SURFACE

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		Project No. 103752.002	Drwn By SL	Chkd By DC	Rev No. 0
					Date DECEMBER 2025

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BOREHOLE: BH25-05 (WET)
BORING DATE: NOVEMBER 20, 2025
DEPTH: 1.70 TO 5.49 METRES BELOW GROUND SURFACE

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BOREHOLE BH25-05 (WET)

FIGURE B9

Project No.	Drwn By	Chkd By	Rev No.	Date
103752.002	SL	DC	0	DECEMBER 2025

BOREHOLE: BH25-05 (DRY)
BORING DATE: NOVEMBER 20, 2025
DEPTH: 1.70 TO 5.49 METRES BELOW GROUND SURFACE

N:\PROJECTS\103700\103752.002\06_CIVIL DRAFTING\RC R0\103752.002_RC_R0_2025_12.DWG

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
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Project	Drawing				
MORETON PROPERTIES LTD. GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL DEVELOPMENT 1600 JAMES NAISMITH DRIVE OTTAWA, ONTARIO	ROCKCORE PHOTOGRAPH BOREHOLE BH25-05 (DRY)				**FIGURE B10**
	Project No.	Drwn By	Chkd By	Rev No.	Date
	103752.002	SL	DC	0	DECEMBER 2025

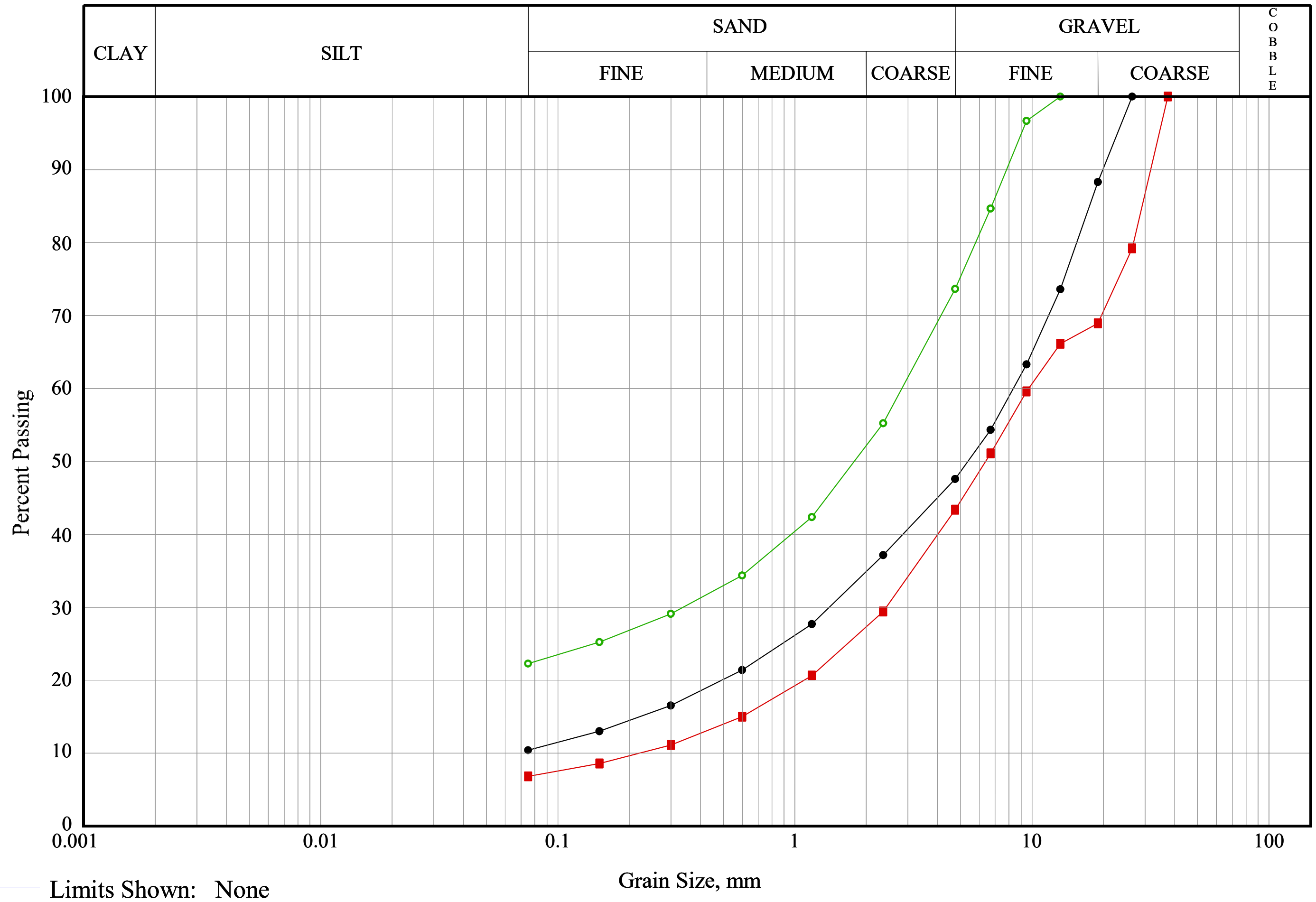
APPENDIX C

Laboratory Test Results

GEMTEC
Consulting Engineers and Scientists

Client: Moreton Properties Ltd.
Project: Geotechnical Investigation, Proposed Residential Develo
Project #: 103752002

Soils Grading Chart



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●		25-04	01		52.4	37.2	10.4	
■		25-04	02		56.6	36.6	6.8	
○		25-05	05		26.4	51.4	22.3	

Line Symbol	USCS Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75μm
●		N/A	---	0.223	1.40	5.38	8.36	17.52	---
■		N/A	0.222	0.601	2.43	6.39	9.71	29.20	---
○		N/A	---	---	0.34	1.78	2.83	6.77	---

Note: More information available upon request

APPENDIX D

Chemical Analysis of Soil Samples
Samples Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2548448)

Report to: Moreton Properties Ltd.
GEMTEC Project: 103752.002 (December 12, 2025)

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Certificate of Analysis

GEMTEC Consulting Engineers and Scientists Limited
32 Steacie Drive
Kanata, ON K2K 2A9
Attn: Matt Rainville

Report Date: 3-Dec-2025
Order Date: 27-Nov-2025

Client PO:
Project: 103752.002
Custody:

Order #: 2548448

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2548448-01	BH25-02 SA3
2548448-02	BH25-05 SA4

Approved By: A. Tirca

Adriana Tirca, B.Eng (Chem)

Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	1-Dec-25	1-Dec-25
Conductivity	MOE E3138 - probe @25 °C, water ext	2-Dec-25	3-Dec-25
pH, soil	MOE E3137 - probe @25 °C, CaCl2 ext	1-Dec-25	1-Dec-25
Resistivity	EPA 120.1 - probe, water extraction	2-Dec-25	3-Dec-25
Solids, %	CWS Tier 1 - Gravimetric	1-Dec-25	2-Dec-25

Certificate of Analysis

Report Date: 03-Dec-2025

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

Order Date: 27-Nov-2025

Client PO:

Project Description: 103752.002

Client ID:		BH25-02 SA3	BH25-05 SA4	-	-		
Sample Date:		19-Nov-25 12:00	20-Nov-25 09:00	-	-	-	-
Sample ID:		2548448-01	2548448-02	-	-		
Matrix:		Soil	Soil	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	90.1	90.7	-	-	-	-
General Inorganics							
Conductivity	5 uS/cm	316	709	-	-	-	-
pH	0.05 pH Units	7.54	8.17	-	-	-	-
Resistivity	0.1 Ohm.m	31.6	14.1	-	-	-	-
Anions							
Chloride	10 ug/g	<10	18	-	-	-	-
Sulphate	10 ug/g	158	128	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Chloride	ND	10	ug/g					
Sulphate	ND	10	ug/g					
General Inorganics								
Conductivity	ND	5	uS/cm					
Resistivity	ND	0.1	Ohm.m					

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	57.0	10	ug/g	56.9			0.2	35	
Sulphate	337	10	ug/g	531			44.7	35	QR-04
General Inorganics									
Conductivity	310	5	uS/cm	306			1.3	5	
pH	8.12	0.05	pH Units	8.08			0.5	2.3	
Resistivity	32.2	0.1	Ohm.m	32.7			1.3	20	
Physical Characteristics									
% Solids	73.3	0.1	% by Wt.	74.3			1.4	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	149	10	ug/g	56.9	91.8	82-118			
Sulphate	624	10	ug/g	531	93.6	80-120			

Qualifier Notes:

QC Qualifiers:

QR-04 Duplicate results exceeds RPD limits due to non-homogeneous matrix.

Sample Data Revisions:

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

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

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