GEOTECHNICAL INVESTIGATION REPORT

Proposed Residential Development
50 Stephanie Street
Toronto, Ontario

Project No.: 25-0019

Date: October 24, 2025

Prepared For:
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1 INTRODUCTION

GeoCore Engineering Ltd. (GeoCore) has been retained by Davad Investments Inc. to revise the geotechnical investigation report prepared for the proposed residential building at the existing residential development with the municipal address of 50 Stephanie Street, Toronto, Ontario (the Site). Authorization to proceed with this work was given by Mr. Ben Hung on behalf of Davad Investments Inc.

The original investigation report which was prepared by Groundwater Environmental Management Services Inc. in August 2025 was provided to GeoCore by Davad Investments Inc. The report was authored by the two authors of this revised report. The geotechnical investigation was carried out in conjunction with a Hydrogeological Assessment undertaken by GEMS.

The Site is located at the northeast corner of Stephanie Street and Phoebe Street in Toronto. It has a rectangular shape with approximate dimensions of 68 m by 92 m. A 24-storey residential apartment building is present in the south section of the Site, constructed over two underground parking garage levels. The underground levels extend to the boundaries of the property. The roof of the underground section north of the 24-storey building is landscaped with grass and some trees.

The architectural drawings for the Zoning By-law Amendment (ZBA) prepared by Quadrangle Architects Limited dated September 25, 2025 reveal that it is proposed to develop the north section of the Site with an 11-storey apartment building constructed over one level of underground parking garage. The P1 floor slab is situated 3.97 m below grade; corresponding with Elevation 89.13 m. It is anticipated that the exterior foundation walls of the existing underground structure surrounding the proposed building will be left in place and comprise part of the excavation shoring system.

The geotechnical investigation was undertaken to characterize the underlying soil and groundwater conditions at the site, to determine the relevant geotechnical properties of encountered soils and to provide recommendations for the proposed development (foundation type and design, temporary shoring, basement slab construction, seismic site classification, etc.).

This report presents the results of the investigation performed in accordance with the general terms of reference outlined above and is intended for the guidance of the owner and the design architects or engineers only. It is assumed that the design will be in accordance with the applicable building codes and standards.

2 FIELDWORK

The fieldwork for this study was carried out on April 28 and July 24, 2025. It consisted of two boreholes, advanced by a drilling contractor commissioned by GEMS. The boreholes are designated as BH2 and MW3; advanced to depths of 16.8 and 15.7 m below ground surface (mbg) respectively. Borehole BH2 was advanced using mud rotary drilling, and MW3 was advanced with hollow stem augers.

It is noted that GEMS originally planned for 3 boreholes. A watermain was struck by the auger at the location of Borehole BH1. A hydrovac was used in the vicinity of this borehole to daylight utilities, and due to the presence of several utilities, BH1 was cancelled.

A cluster of two monitoring wells (shallow and deep) were installed in MW3, extended to depths of 9 and 15.3 mbg. The wells were installed for long-term monitoring of the groundwater table necessary for the Hydrogeological assessment.

The locations of the boreholes and monitoring wells are shown on Figure 1 ‘Borehole Location Plan’ in Appendix B. The borehole log sheets are enclosed in Appendix C of this report.

Standard penetration tests (SPT) were carried out while advancing the boreholes through the overburden soils to take representative soil samples and to measure penetration index values (N-values) to characterize the condition of the various soil materials. The number of blows of the striking hammer required to drive the split spoon sampler through 300 mm depth increments was recorded and these are presented on the logs as penetration index values.

Groundwater level observations were made in the boreholes during their advancement, and subsequently in the monitoring wells on August 12, 2025.

The ground surface elevations at the locations of the boreholes and monitoring wells were established utilizing a TopCon HiPer V GNSS Receiver.

The fieldwork for this project was carried out under the supervision of an experienced technician who laid out the positions of the boreholes in the field; arranged locates of buried services; effected the drilling, sampling and in situ testing; observed groundwater conditions; and prepared field borehole log sheets.

3 LABORATORY TESTS

The soil samples recovered from the split spoon sampler were properly sealed, labelled and taken to the laboratory. They were visually classified and water content tests were conducted on all samples retained from Borehole BH2. The results of the classification, water contents, and Standard Penetration tests are presented on the borehole log sheets in Appendix C.

Grain-size analyses were carried out on two (2) native soil samples (Borehole BH2 Sample 2, and BH2 Sample 6). These samples were also subjected to Atterberg Limits tests. The results of these tests are enclosed in Appendix D.

In addition, one soil sample, BH2 Sample 3, was submitted to Bureau Veritas for determination of pH and soluble sulphate content and its potential for sulphate attack on buried concrete. The results of these tests are enclosed in Appendix E; discussed in Section 5.10 of this report.

4 SITE AND SUBSURFACE CONDITIONS

Full details of the subsurface soil and groundwater conditions at the site are given on the Borehole Log Sheets attached in Appendix C of this report.

The following paragraphs present a description of the site and a commentary on the engineering properties of the various soil materials contacted in the boreholes.

It should be noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design, and therefore, should not be construed as exact planes of geological change.

4.1 SITE DESCRIPTION

The Site is located at the northeast corner of Stephanie Street and Phoebe Street. It has a rectangular shape with approximate dimensions of 68 m by 92 m. A 24-storey residential apartment building is present in the south section of the Site, constructed over two underground parking garage levels. The underground levels extend to the boundaries of the Site. The roof of the underground section north of the building is landscaped with grass and some trees.

The Site is bound by the Grange Park to the north, and a playground followed by commercial and institutional buildings to the east.

4.2 FILL MATERIAL

The uppermost stratum of the soil profile in Borehole MW3 consists of fill material. It extends to an approximate depth of 2.2 m and consists of a thin: 150 mm thick layer of topsoil underlain by clayey silt with some sand, and traces of gravel and organics.

The fill is brown and dark brown in colour and moist in appearance. SPT in the fill provided N-values ranging from 10 to 13 blows for 300 mm of penetration, indicating a stiff in situ consistency.

4.3 NATIVE SOIL

The native soils below the fill material consist of layers of clayey silty till, followed by silty clay, underlain by shale bedrock at an approximate depth of 15.7 mbg.

It is noted that wet sandy soils are known to be present near the Site, and that these layers may be present locally at the Site. Wet sand soils were not present in the two boreholes advanced at the Site.

4.3.1 Clayey Silt Till

Clayey silt till with some sand to sandy and trace gravel is present below the fill. It extends to a depth of 8.5 mbg in Borehole BH2 and 5.5 m in MW3. The clayey silt till is a glacial deposit consisting of a random mixture of soil particles ranging from clay to gravel, with clay and silt being the predominant fractions.

It is brown in color in MW3 and grey in BH2. The water content of samples obtained from Borehole BH2 range from 12 to 22% by weight, moist in appearance.

SPT carried out in the clayey silt till provided N-values ranging from 10 to 28 blows indicating a stiff to very stiff consistency, more typically being very stiff.

Grain size analysis was carried out on a representative sample of the clayey silt till. The test result is enclosed in Appendix D and summarized in the following table.

Borehole Number	Sample No. and Depth	Sample Description	Gravel %	Sand %	Silt %	Clay %
BH2	Sample 2; 6.3 m	SANDY CLAYEY SILT trace gravel	9	22	42	27

Atterberg Limits tests conducted on the sample revealed that the clayey silt till has a Liquid Limit of 28 and Plasticity Index of 9, indicating that the soil has a low plasticity. The test result is enclosed in Appendix D.

Based on the results of the grain size analysis, the Coefficient of Permeability (k) of the clayey silt till is estimated to be less than 10^{-7} cm/sec, corresponding to very low relative permeability.

4.3.2 Silty Clay

Silty clay with trace sand and trace of gravel is present below the clayey silt till in both boreholes. The silty clay extends to bedrock situated at an approximate depth of 15.5 mbg.

The silty clay is grey in colour and the water content of the samples obtained from Borehole BH2 range from 18% to 23% by weight, moist in appearance.

SPT carried out in the silty clay provided N-values ranging from 5 to 58 blows indicating a soft to hard consistency, typically being very stiff to hard.

Grain size analysis was carried out on one (1) representative sample of silty clay. The test result is enclosed in Appendix D and summarized in the following table.

Borehole Number	Sample No. and Depth	Sample Description	Gravel %	Sand %	Silt %	Clay %
BH2	Sample 6; 12.5 m	SILTY CLAY, trace sand	0	2	60	38

Atterberg Limits test conducted on this sample revealed that the silty clay has Liquid Limit of 26 and a Plasticity Index of 9, indicating that the soil has a low plasticity. The test result is enclosed in Appendix D.

Based on the results of the grain size analyses, the Coefficient of Permeability (k) of the clayey silt is estimated to be less than 10^{-8} cm/sec, corresponding to very low relative permeability.

4.3.3 Shale Bedrock

The native soils are underlain by bedrock.

The bedrock was not investigated / cored. Typically, in Downtown Toronto, the bedrock consists of grey shale of the Georgian Bay Formation with occasional thin layers of limestone interbeds.

The condition and strength of the bedrock typically improves with depth, from being highly weathered, soft and fractured within the uppermost 1 to 2 m depth becoming intact and strong.

4.4 GROUNDWATER

The groundwater levels measured on August 12, 2025 in the monitoring wells installed in MW3 are shown on the borehole log and are summarized in the following table.

Borehole Number	Ground Surface Elevation (m)	Groundwater Depth (mbg)	Groundwater Elevation (m)
MW3S	93.04	8.39	84.65
MW3D	93.04	9.24	83.80

It should be noted that groundwater levels are subject to seasonal fluctuations. A higher groundwater level condition may also develop following significant rainfall events.

5 DISCUSSION AND RECOMMENDATIONS

The following discussions and recommendations are based on the factual data obtained from the boreholes advanced at the site and are intended for use by the client and design architects and engineers only.

We understand that it is proposed to construct an 11-storey apartment building in the north section of the Site. The building will be constructed over one level of underground parking garage extending 3.97 m below grade, at Elevation 89.13 m.

Two levels of underground parking garage for the existing building are present within the footprint of the proposed building. It is anticipated that the exterior foundation walls of the existing underground structure surrounding the proposed building will be left in place and comprise part of the shoring system for the proposed excavation.

Given that the basement floor slab for the new building is situated at a higher elevation than the existing basement floor slab, it will be necessary to backfill the existing basement with engineered fill to support the new floor slab. Given that deep foundations will be required to support the building, it will be necessary to remove the concrete floor of the existing slab-on-grade of P2 and building foundations to facilitate installation of the new foundations.

The construction methods described in this report are not specifications or recommendations to the contractors or as the only suitable methods. The collected data and the interpretation presented in this report may not be sufficient to assess all the factors that may influence the construction. Contractors bidding on this project or conducting work associated with this project should make their own interpretation of the factual data and/or carry out their own investigations as they might deem necessary. The contractor should also select the method of construction, equipment and sequence based on their previous experience on similar projects.

5.1 EXCAVATION

Based on the field results, excavations of the existing foundations and for proposed foundations are not expected to pose any unusual difficulty. Excavation of the soils at this site can be carried out with hydraulic excavators.

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). With respect to the OHSA, the native clayey silt till and silty clay soils are expected to conform to Type 2 soils. Sandy soils situated below the groundwater table are considered Type 4 soil.

Temporary excavation sidewalls in Type 2 soils may be cut vertically to a maximum height of 1.2 m from the excavation bottom, then sloped at a maximum inclination of 1.0 horizontal to 1.0 vertical.

Side slopes of excavations extended into Type 4 soil should not be any steeper than 3.0 horizontal to 1.0 vertical.

In the event very loose and/or soft soils are encountered at shallow depths or within zones of persistent seepage, it will be necessary to flatten the side slopes to achieve stable conditions.

Where workers must enter excavations extending deeper than 1.2 m below grade, the excavation sidewalls must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects.

It is anticipated that the existing underground parking garage walls incorporating tiebacks or walers will be utilized as the shoring system. Shoring recommendations are provided in Section 5.7 of this report.

5.2 GROUNDWATER CONTROL

The subsoil at the site encountered in the boreholes consist of clayey silt fill, clayey silt till, and native silty clay. Groundwater yield from the fill materials is not expected to be significant. The groundwater yield from the native clayey silt till and silty clay units is expected to be very small. Provided that wet sand layers are not encountered in the excavation, it should be possible to control groundwater seepage into the excavation using sump pumps placed locally in the excavation as deemed necessary.

Surface water should be directed away from open excavations.

The Hydrogeological Report prepared by GEMS should be referred to for recommendations for estimated dewatering volumes during construction and during the service life of the building, and requirements for an Environmental Activity and Sector Registry (EASR) or Permit to Take Water (PTTW).

5.3 REUSE OF ON-SITE EXCAVATED SOIL

On-site excavated inorganic soils, and soils free of construction debris and other deleterious materials are considered suitable for reuse as backfill provided their water content is within 2% of their optimum water contents (OWC) as determined by Standard Proctor test, and the materials are effectively compacted with a heavy sheepsfoot compactor.

While the quality of the on-site soils is considered suitable for backfilling; the moisture content of the soils and the lift thickness for compaction must be properly controlled during backfilling. Measured water content within the fill and native soils within the presumed excavation depth generally range from approximately 12 to 20%; typically, being above the optimum water content of the soils.

On-site soils that are wetter than their OWC should be dried sufficiently prior to use as backfill to achieve the specified degree of compaction.

5.4 FOUNDATION DESIGN

We understand that it is proposed to construct an 11-storey building over one level of underground parking garage extending 3.97 m below grade, at Elevation 89.13 m. As there are two underground levels present at the Site, it will be necessary to fill the area under the proposed building to the underside of the new P1 floor slab.

It is anticipated that the new P1 level will be above the water table at the Site and it will not be necessary to waterproof the substructure and design the basement as a tank. On this basis, a subfloor and perimeter drainage system can be installed at the site to provide drainage of stormwater, connected to the drainage system of the existing building at the site.

The compacted fill material used to fill to the new P1 floor slab will not be capable of supporting building foundations for an 11-storey building. It will be necessary to utilize caisson foundations extended into hard silty clay or shale bedrock to support the building.

Caisson foundations extended to elevation 81 m, about 12 m below existing grade founded on the hard silty clay can be used, dimensioned on the basis of a bearing resistance at Serviceability Limit States (SLS) of 1000 kPa and factored geotechnical bearing resistance at Ultimate Limit States (ULS) of 1500 kPa, for vertical and centric loads.

It is anticipated that caissons founded a minimum of 1.5 m into sound shale bedrock, can be designed based on end bearing resistance at Ultimate Limit States (ULS) of 7 to 10 MPa. The caissons should be extended through the weathered zone and founded on the intact zone. Accordingly, it will be necessary to advance additional boreholes at the Site to determine the thickness of the weathered portion of the bedrock, and depth to intact zone.

A factored shaft resistance of 200 kPa may be used to determine the axial capacity of the caissons due to shaft skin friction within the shale bedrock. The uplift resistance of the piles would be 75% of the piles shaft resistance.

The centre to centre spacing between adjoining caissons should not be less than 2 X the largest diameter (B) of the caissons. The following reduction factors for pile group effects should be applied.

Center to Center Pile Spacing	Axial Capacity Efficiency Reduction Factor
2B to 3B	0.7
3B to 6B	Linear interpolation between 0.7 to 1.0

The caisson contractor must take into consideration the excavation method used through wet fine sand and/or silt and the concreting technique for installing caissons in accordance with good construction practice.

The foundation construction must be closely monitored and inspected by qualified geotechnical personnel to ensure that the founding rock is consistent with the findings of the geotechnical investigation, and the bottom of the caisson hole has been sufficiently cleaned prior to concrete pour. Concrete should be placed to a minimum thickness of 600 mm in the caisson hole and stirred with the auger. The concrete should then be extracted from the caisson hole and disposed. Concrete placement for the caisson foundation may then proceed.

If more than 150 mm of water is present in the base of the hole, it will be necessary to place concrete using the tremie method to ensure proper placement of the concrete in water.

5.5 BASEMENT FLOOR SLAB

The following recommendations regarding filling the existing P2 level to the proposed underside of the single level basement should be adhered to during construction:

- All demolition rubble, former building foundations, and disturbed and weathered soil must be removed, and the exposed subgrade soils proof-rolled in conjunction with an inspection by the Geotechnical Engineer prior to any fill placement.
- Fill operations should be monitored, and compaction tests should be performed on a full-time basis by a qualified engineering technician supervised by the project engineer.
- Soils used as fill should be free of organic and/or other unsuitable material. The engineered fill must be placed in lifts not exceeding 200 mm in thickness and compacted to 98% Standard Proctor Maximum Dry Density (SPMDD).
- The fill operation should take place in favorable climatic conditions. If the work is carried out in months when freezing temperatures may occur, all frost affected material must be removed prior to the placement of frost-free fill.

It is recommended that a combined moisture barrier and a levelling course, having a minimum thickness of 200 mm and comprised of free draining material using 19 mm clear stone be provided as a base for the slab-on-grade. The base material should be compacted to a dense state.

Prior to placement of the 19 mm clear stone, the subgrade must be properly prepared by the removal of any wet, loose and disturbed soils. After removal of all unsuitable materials, the subgrade should be inspected and adjudged as satisfactory before preparing the granular base course. Any loose or unsuitable subgrade areas should be sub-excavated and replaced with suitable approved compacted backfill; placed in maximum lifts of 200 mm thickness and compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD). Provided the subgrade, under-floor fill and granular base are prepared in accordance with the above recommendations, the Modulus of Subgrade Reactions (Ks) for floor slab design will be 25,000 kPa/m.

A single line of sub-floor weeping pipe 100 mm in diameter must be placed under the slab-on-grade along the length of the building . The weeping tile must be wrapped with filter fabric and covered with a minimum of 150 mm of clear stone. It should be placed a minimum of 0.5 m below the basement floor slab, above the founding level of the footings.

The basement of the proposed building must also be provided with perimeter drainage. Prefabricated drainage sheets (Terradrain 600 or equivalent) must be placed continuously against the shoring wall. These should drain through drainage ports in the building walls into a perimeter solid pipe and channel all the water into the sump pit in the building. The maximum spacing of the drainage ports must not exceed 6 m, subject to confirmation at the time of construction.

The perimeter and sub-floor drains must be connected to a positive frost-free outlet from which the water can be removed or connected to a sump located in the basement. The water from the sump must be pumped out to a suitable discharge point.

The installation of the perimeter drains as well as the outlet must conform to the applicable plumbing code requirements.

The soils at the site are susceptible to frost effects which would have the potential to deform hard landscaping adjacent to the building. At locations where the new structures are expected to have flush entrances, care must be taken in detailing the exterior slabs/sidewalks, providing insulation/drainage/ non-frost susceptible backfill to maintain the flush threshold during freezing weather conditions.

5.6 LATERAL EARTH PRESSURE

Parameters used in the determination of earth pressure acting on structures subject to unbalanced pressures are defined below.

Parameter	Definition	Units
Φ’	angle of internal friction	degrees
γ	bulk unit weight of soil	kN/m^3
Ka	active earth pressure coefficient (Rankine)	dimensionless
Ko	at-rest earth pressure coefficient (Rankine)	dimensionless
Kp	passive earth pressure coefficient (Rankine)	dimensionless

The appropriate un-factored values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Soil	Parameters				
	Φ'	γ	Ka	Ko	Kp
Fill Material	28°	20.5	0.36	2.77	0.53
Native Soil	32°	21.0	0.31	3.25	0.47

Walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following formula:

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

where **P** = lateral pressure in kPa acting at a depth h (m) below ground surface

K = applicable lateral earth pressure coefficient (Use Ko for basement wall design)

γ = bulk unit weight of backfill (kN/m^3)

h = height at any point along the interface (m)

q = the complete surcharge loading (kPa)

This equation assumes that free-draining backfill and positive drainage is provided behind the basement walls.

Subsurface walls that are subject to unbalanced earth and hydrostatic pressures must be designed to resist a pressure that can be calculated based on the following formula:

$$\mathbf{P = K\ [\gamma\ (h - h_w) + \gamma' h_w + q] + \gamma_w h_w}$$

where **P** = lateral pressure in kPa acting at a depth h (m) below ground surface

K = applicable lateral earth pressure coefficient

H = height at any point along the interface (m)

h_w = depth below the groundwater level at point of interest (m)

γ = bulk unit weight of backfill (kN/m^3)

γ' = the submerged unit weight (kN/m^3) of exterior soil (□' = □ - □w)

γ_w = unit weight of water, assume a value of 9.8 kN/m^3

q = the complete surcharge loading (kPa)

Resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction (R) depends on the normal load on the soil contact (N) and the frictional resistance of the soil (tan Φ’) expressed as: **R = N tan Φ’**. This is an ultimate resistance value and does not contain a factor of safety.

5.7 SHORING DESIGN

As stated earlier, it is anticipated that the exterior foundation walls of the existing underground structure surrounding the proposed building will be left in place and comprise part of the excavation shoring system.

The design of temporary shoring for the support of the excavation walls must account for the presence of structures and buried services on the adjacent properties, and the existing subsurface conditions at the site.

The lateral restraining force for the shoring system may be provided by employing either rakers or tieback anchors. The latter is favorable because they do not protrude into the excavations as is the case with rakers. The use of tieback anchors will depend on whether permission is obtained to extend the anchors to the required distance on to the neighboring properties.

The shoring design should be based on the procedure detailed in the latest edition of the Canadian Foundation Engineering Manual.

The earth pressure coefficients applicable for the design of the shoring system are:

Ko the ‘at rest’ earth pressure coefficient, applicable where no movement in the retained soil can be permitted, such as the presence of buried services or foundations close to the wall, and the railway corridor to the east,

Ka the active pressure coefficient,

= 0.3 - where adjacent building footings or buried services fall outside an envelope formed by a 60o line drawn from the base of the excavation wall to the ground surface

= 0.25 - where adjacent building footings or buried services are outside an envelope formed by a 45o line drawn from the base of the excavation wall to the ground surface

Raker footings should be designed in accordance with the design principals for shallow foundations subject to inclined loading. All raker footings should be located outside the zone of influence of the buried portion of soldier piles, and at no less than 1.5D from the piles, where D = Depth of penetration of the piles below the base of the excavation. No excavation should be made within two footing widths of the raker footings, on the side opposite the rakers.

Anchors extended into the very stiff to hard silty clay may be designed based on soil/grout bond value of 70 kPa. This value depends on the anchor installation method and grouting procedures.

Gravity poured concrete can result in low bond values, while pressure grouted anchors will give higher values and produce a more satisfactory anchor.

It will be necessary to perform load tests on the tiebacks to confirm the bond stresses assumed in the design of anchors.

Movement of the shoring system is inevitable. Vertical movements will result from the vertical loads on the soldier piles resulting from the inclined tiebacks and inward horizontal movement will result from the earth and water pressures. The magnitude of this movement can be controlled by sound construction practices. The lateral and vertical movement of the shoring system must be monitored especially at locations in which settlement sensitive structures are present, to ensure that movements are kept within an acceptable range.

5.8 PAVEMENT DESIGN

It is anticipated that most of the pavement at the site will be situated on the parking garage roof slab. In this regard, the pavement may be comprised of a minimum of 75 mm thick layer of Granular ‘A’ topped with asphaltic concrete having a minimum thickness of 80 mm (40 mm HL8 and 40 mm HL3).

Pavement which will be supported by soil subgrade should comprise a minimum 300 mm compacted depth of OPSS Granular B Type I sub-base, followed by a minimum 150 mm compacted depth of Granular A base material, 50 mm of HL8 asphaltic concrete base course, and 40 mm of HL3 asphaltic concrete surface course.

The critical section of pavement will be at the transition between the pavement on subgrade and the pavement above the garage roof slab. To alleviate the detrimental effects of dynamic loading / settlement / pavement depression in the backfill to the rigid garage roof structure, it is recommended that an approach type slab be constructed at the entrance/exit points, by extending the granular sub-base to greater depths along the exterior garage wall.

5.9 EARTHQUAKE DESIGN PARAMETERS

The Ontario Building Code (2024) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.18.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of the Site Classification for Seismic Site Response are set out in Table 4.1.8.4.B of the Ontario Building Code. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (vs) measurements have been taken. In the absence of such measurements, the classification is estimated based on empirical analysis of undrained shear strength or penetration resistance. The applicable penetration resistance is that which has been corrected to a rod energy efficiency of 60% of the theoretical maximum or the (N60) value.

The proposed building will either be founded on the hard silty clay soil or on bedrock. Accordingly, the site designation for seismic analysis is Class C.

The site specific 5% damped spectral acceleration coefficients, and the peak ground acceleration factors are provided in the 2024 Ontario Building Code - Supplementary Standard SB-1, location Toronto, Ontario.

5.10 CHEMICAL CHARACTERIZATION OF SUBSURFACE SOIL

A native soil samples obtained from Borehole BH2-SS3 (from approximate depth of 7.6 mbg) was submitted to Bureau Veritas for pH index test and water-soluble sulphate content to determine the potential of attacking the subsurface concrete. The Certificate of Analysis provided by the analytical chemical testing laboratory is contained in Appendix E of this report.

The test results revealed that the pH index of the soil sample is 7.63, indicating a slight alkalinity.

The water-soluble sulphate content of the tested sample is 0.012%. The concentration of water-soluble sulphate content of the tested samples is below the CSA Standard of 0.1% water-soluble sulphate (Table 12 of CSA A23.1, Requirements for Concrete Subjected to Sulphate Attack). Special concrete mixes against sulphate attack are therefore not required for the sub-surface concrete of the proposed building.

6 LIMITATIONS OF REPORT

The Limitations of Report, as quoted in Appendix ‘A’, are an integral part of this report.

7 CLOSURE

We trust that this report meets your requirements. If you have any questions or require clarification, please do not hesitate to contact the undersigned.

Sincerely,

GeoCore Engineering Ltd.

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Appendix A

Limitations of Report

LIMITATIONS OF REPORT

The conclusions and recommendations in this report are based on information determined at the inspection locations. Soil and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation.

The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known to us, in our analysis certain assumptions had to be made as set out in this report. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

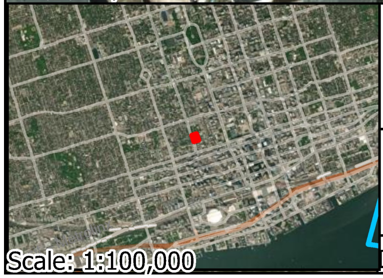
This report was prepared for Davad Investments Inc. by GeoCore. The material in it reflects GeoCore's judgement in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions which the Third Party may make based on it, are the sole responsibility of such Third Parties.

We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis. We recommend also that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the test holes. In cases where these recommendations are not followed, the company's responsibility is limited to accurately interpreting the conditions encountered at the test holes, only.

The comments given in this report on potential construction problems and possible methods are intended for the guidance of the design engineers and architects, only. The number of inspection locations may not be sufficient to determine all the factors that may affect construction methods and costs. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

Appendix B

Borehole Location Plan



Borehole Layout Plan
50 Stephanie Street

Client: Gateway Properties	Project # 25-0049	Created by: N. Colangelo

Date: 2025-09-02

Figure 1 of 1

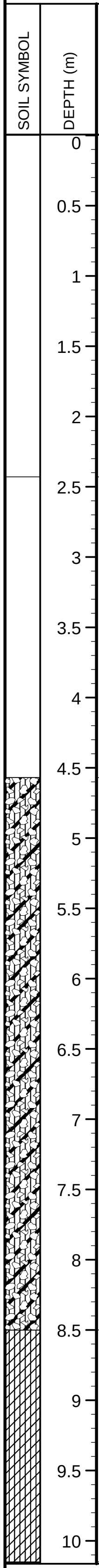
0 21.47 42.95 Meters

Reference:City of Toronto, ON, Maxar, Microsoft, Earthstar Geographics, Esri Canada, Maxar

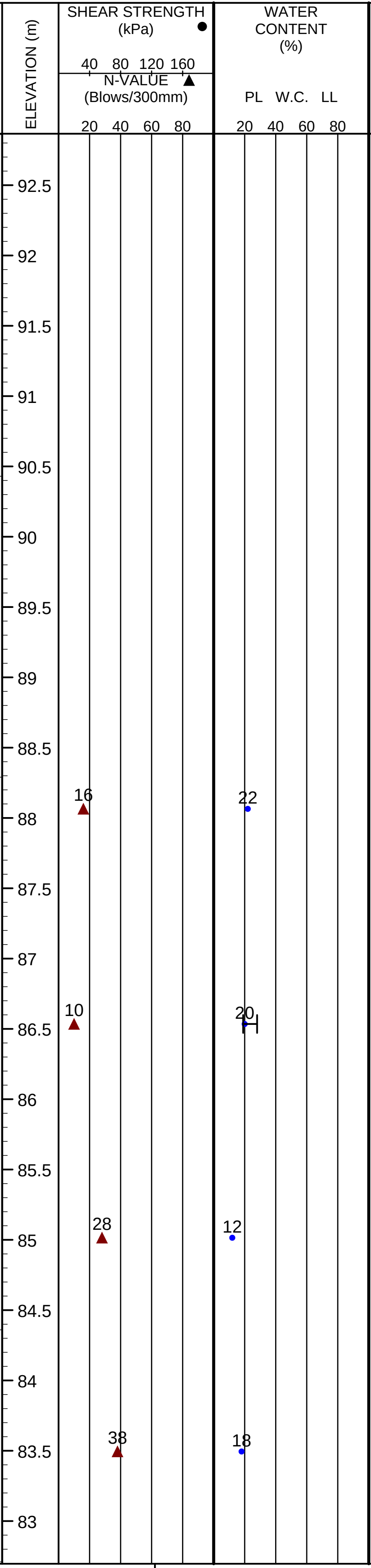
Appendix C

Borehole Log Sheets

CLIENT: Davad Investments INC	PROJECT NO.: 25-0049	**BOREHOLE NO. : BH2**
PROJECT: 50 Stephanie Street		
LOCATION: Toronto, Ontario	NORTHING (m):	EASTING (m):
		ELEV. (m) 92.86
DRILLING CONTRACTOR: Drilltech Drilling ltd.	BOREHOLE DIAMETER (cm): 15	WELL DIAMETER (cm):
DRILLING METHOD: Augering & Split Spoon Sampling		TOTAL DEPTH OF BOREHOLE (m): 16.8

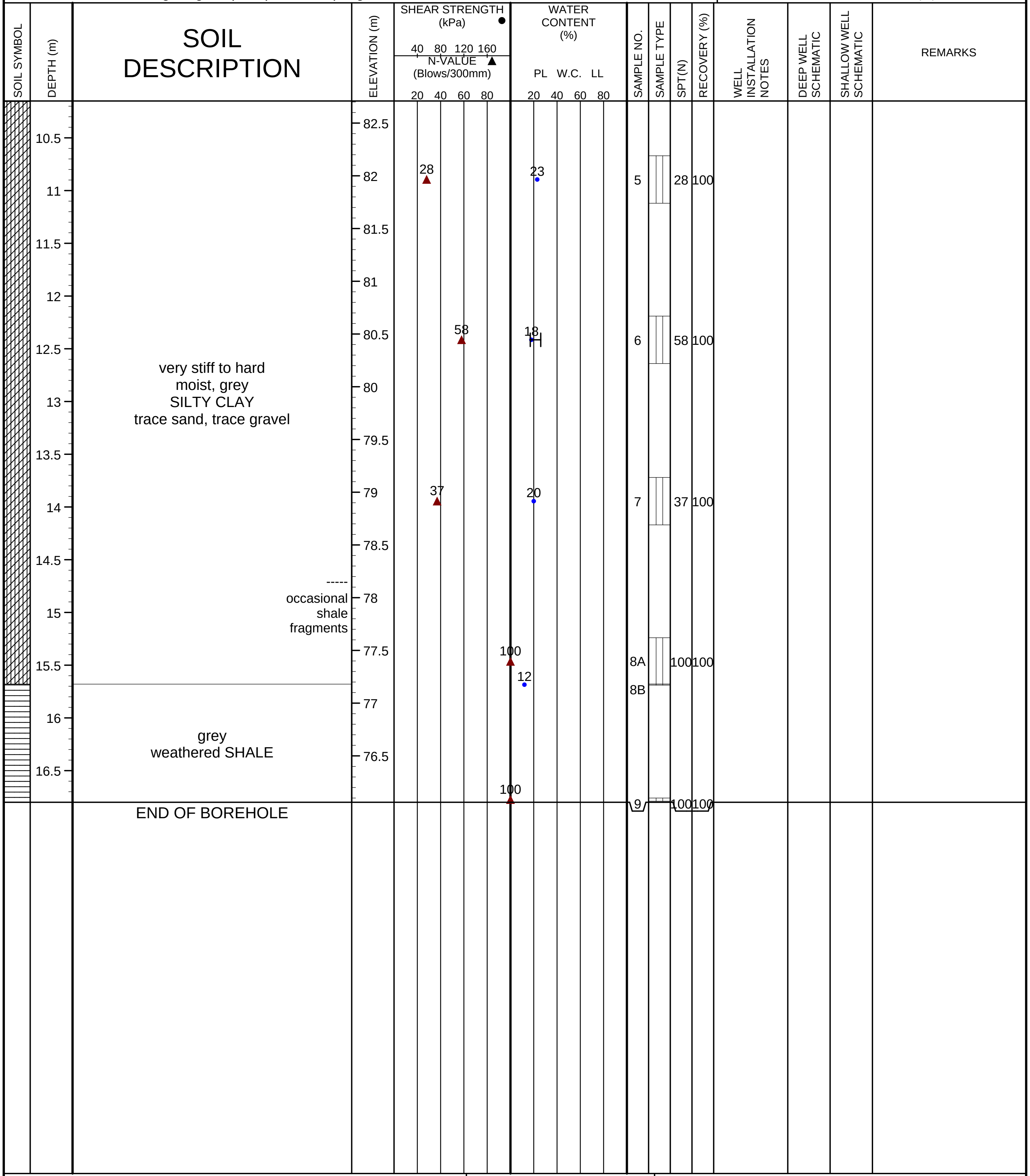


SOIL DESCRIPTION	SAMPLE NO.	SAMPLE TYPE	SPT(N)	RECOVERY (%)	WELL INSTALLATION NOTES	DEEP WELL SCHEMATIC	SHALLOW WELL SCHEMATIC	REMARKS
Excavated with Hydro Vacuum to 2.4 m depth								Borehole advanced with mud rotary drilling from 4.6 m depth.
Straight Auger to 4.5 m depth								
stiff to very stiff, grey, moist CLAYEY SILT some sand to sandy trace gravel (TILL) occasional wet sand seams	1		16	30				
	2		10	50				
	3		28	60				
hard, moist, grey SILTY CLAY trace sand, trace gravel frequent seams of moist sand and silt	4		38	80				



GEMS	LOGGED BY: AC
	DRILLING DATE: July 24, 2025
	REVIEWED BY: KC
	PAGE 1 OF 2

CLIENT: Davad Investments INC	PROJECT NO.: 25-0049	**BOREHOLE NO. : BH2**
PROJECT: 50 Stephanie Street		
LOCATION: Toronto, Ontario	NORTHING (m):	EASTING (m): ELEV. (m) 92.86
DRILLING CONTRACTOR: Drilltech Drilling ltd.	BOREHOLE DIAMETER (cm): 15	WELL DIAMETER (cm):
DRILLING METHOD: Augering & Split Spoon Sampling		TOTAL DEPTH OF BOREHOLE (m): 16.8



SOIL DESCRIPTION

very stiff to hard
moist, grey
SILTY CLAY
trace sand, trace gravel

-----
occasional
shale
fragments

grey
weathered SHALE

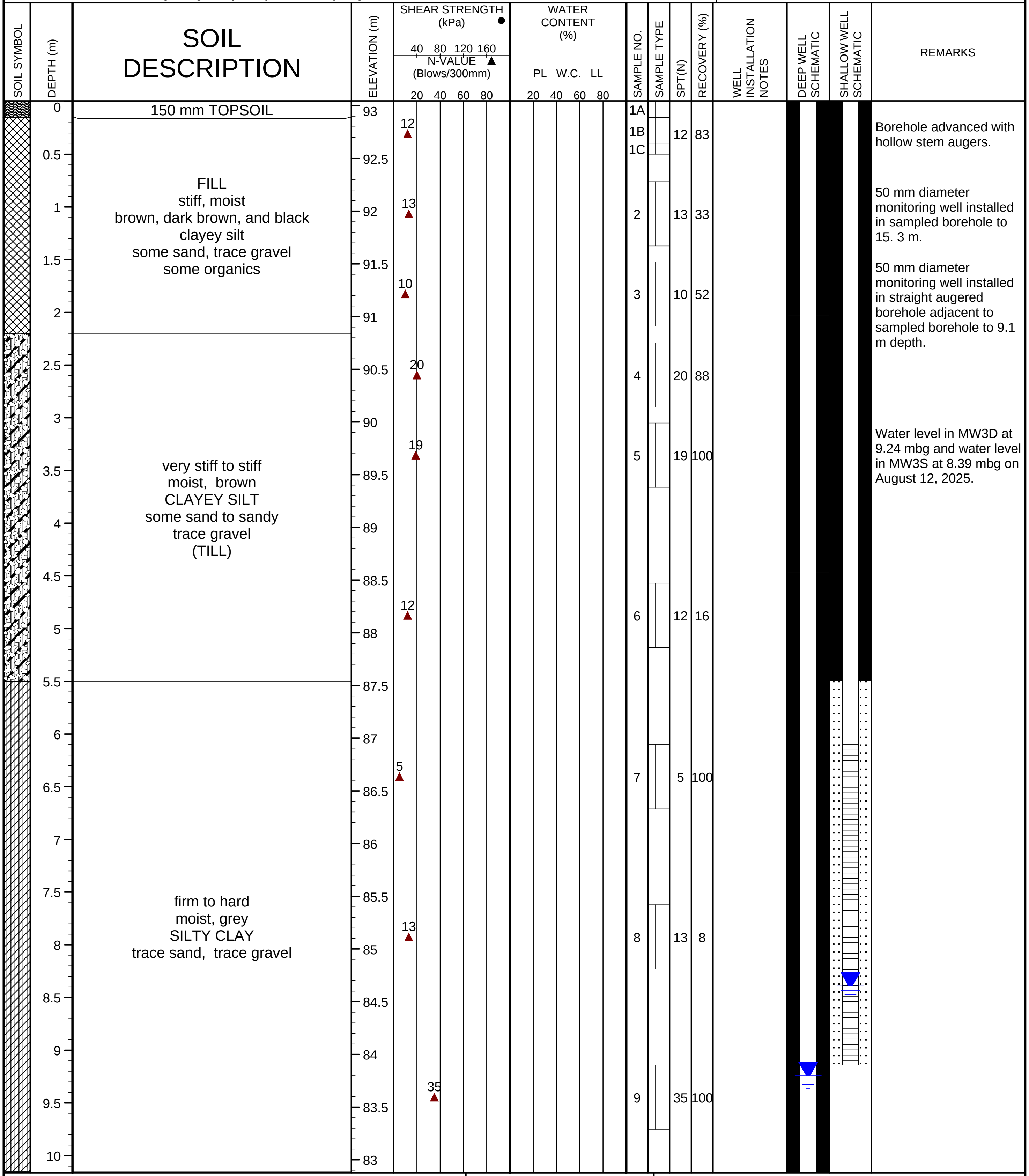
END OF BOREHOLE

SAMPLE NO.	SPT(N)	RECOVERY (%)
5	28	100
6	58	100
7	37	100
8A	100	100
8B		
9	100	100

GEMS

LOGGED BY: AC	DRILLING DATE: July 24, 2025
REVIEWED BY: KC	PAGE 2 OF 2

CLIENT: Davad Investments INC
PROJECT NO.: 25-0049
BOREHOLE NO. : MW3D/S
PROJECT: 50 Stephanie Street
LOCATION: Toronto, Ontario
NORTHING (m): 4834478.109
EASTING (m): 629669.04
ELEV. (m) 93.04
DRILLING CONTRACTOR: Drilltech Drilling ltd.
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm): 5
DRILLING METHOD: Augering & Split Spoon Sampling
TOTAL DEPTH OF BOREHOLE (m): 15.7

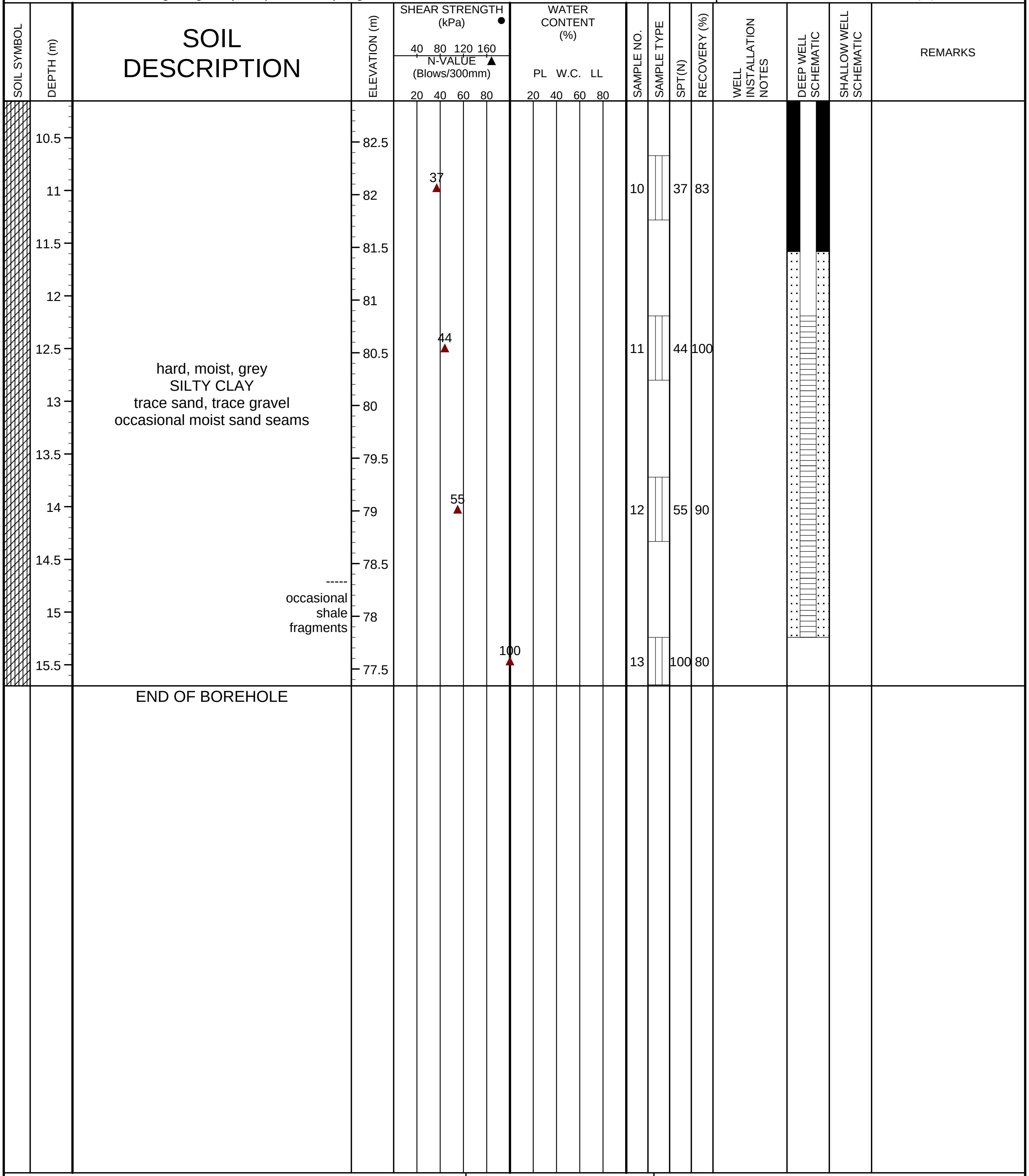


DEPTH (m)	SOIL DESCRIPTION	ELEVATION (m)	SAMPLE NO.	SPT(N)	RECOVERY (%)	REMARKS
0	150 mm TOPSOIL	93	1A 1B 1C	12	83	Borehole advanced with hollow stem augers.
0.15	FILL stiff, moist brown, dark brown, and black clayey silt some sand, trace gravel some organics		2	13	33	50 mm diameter monitoring well installed in sampled borehole to 15. 3 m.
			3	10	52	50 mm diameter monitoring well installed in straight augered borehole adjacent to sampled borehole to 9.1 m depth.
2.2	very stiff to stiff moist, brown CLAYEY SILT some sand to sandy trace gravel (TILL)		4	20	88	
			5	19	100	Water level in MW3D at 9.24 mbg and water level in MW3S at 8.39 mbg on August 12, 2025.
			6	12	16	
5.5	firm to hard moist, grey SILTY CLAY trace sand, trace gravel	87.5	7	5	100	
			8	13	8	
			9	35	100	

GEMS

LOGGED BY: AC
DRILLING DATE: April 28, 2025
REVIEWED BY: KC

CLIENT: Davad Investments INC	PROJECT NO.: 25-0049	**BOREHOLE NO. : MW3D/S**
PROJECT: 50 Stephanie Street		
LOCATION: Toronto, Ontario	NORTHING (m): 4834478.109	EASTING (m): 629669.04 ELEV. (m) 93.04
DRILLING CONTRACTOR: Drilltech Drilling ltd.	BOREHOLE DIAMETER (cm): 15	WELL DIAMETER (cm): 5
DRILLING METHOD: Augering & Split Spoon Sampling		TOTAL DEPTH OF BOREHOLE (m): 15.7



SOIL DESCRIPTION

hard, moist, grey
SILTY CLAY
trace sand, trace gravel
occasional moist sand seams

-----
occasional
shale
fragments

END OF BOREHOLE

SAMPLE NO.	SPT(N)	RECOVERY (%)
10	37	83
11	44	100
12	55	90
13	100	80

GEMS

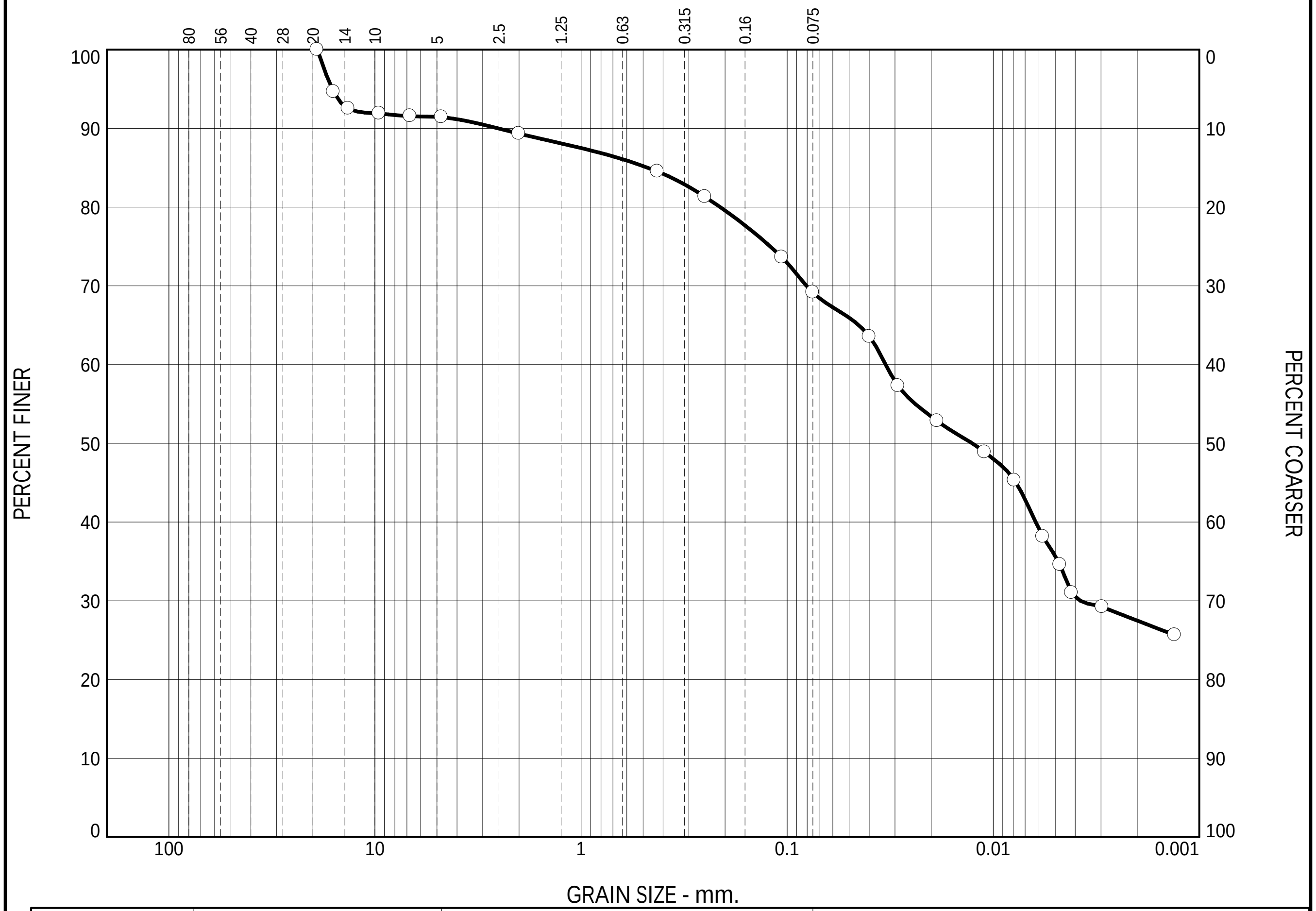
LOGGED BY: AC	DRILLING DATE: April 28, 2025
REVIEWED BY: KC	PAGE 2 OF 2

Appendix D

Geotechnical Laboratory Testing

Grain Size Distribution Report

ASTM D422



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	9	2	4	16	42	27

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	28	19	0.4743	0.0333	0.0126	0.0038				

Material Description	Test Date	USCS	NM
○ SANDY CLAYEY SILT, trace gravel	July 29,	CL	

Project No. 25-0049 Client: Davad Investments INC

Project: 50 Stephanie Street

○Depth: 6.1 m Sample Number: BH2, Sample 2

Remarks:

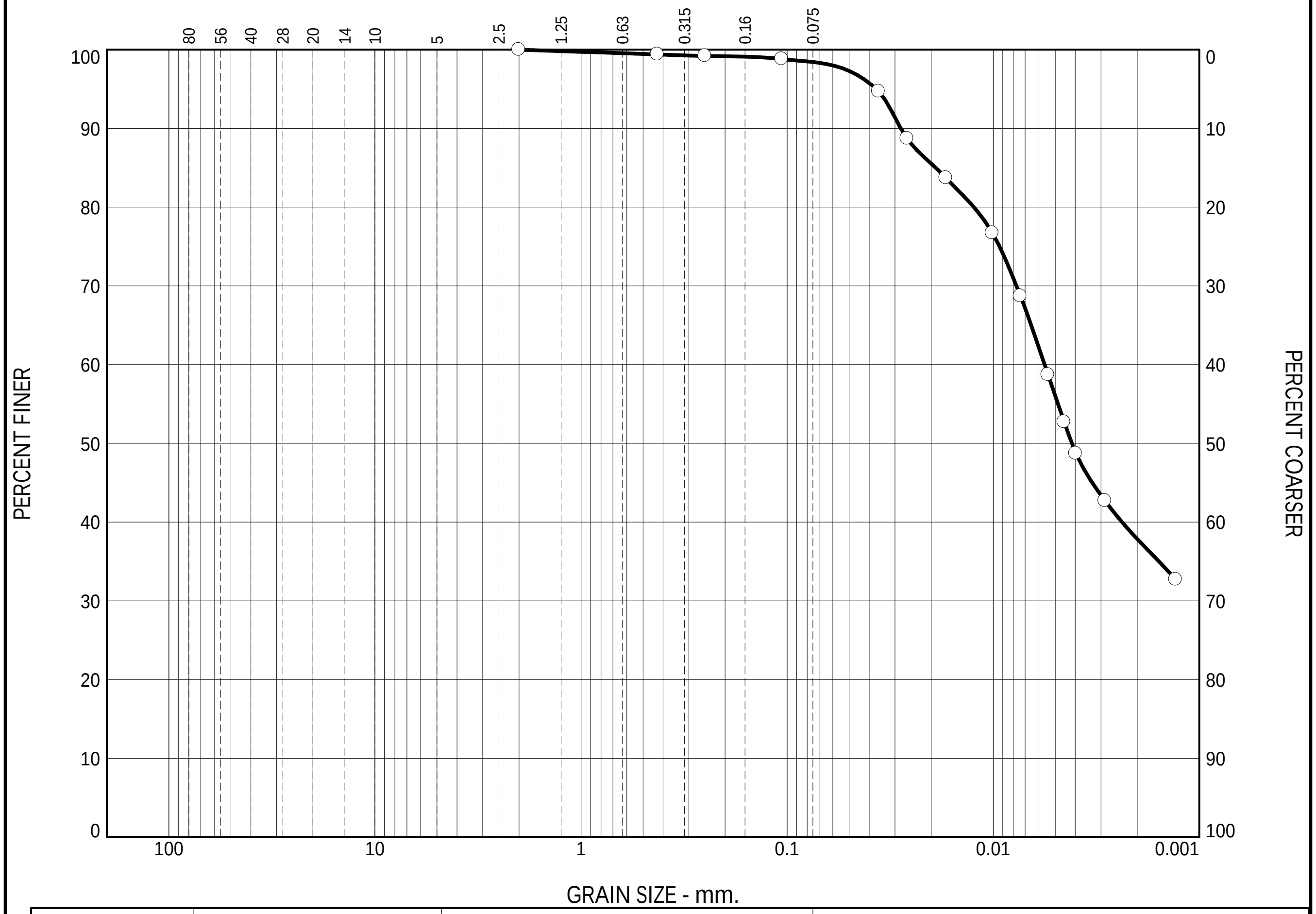
Groundwater Environmental Management Services Inc.

Concord, Ontario

Figure C-1

Grain Size Distribution Report

ASTM D422



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	0	0	1	1	60	38

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	26	17	0.0190	0.0056	0.0042					

Material Description	Test Date	USCS	NM
○ SILT and CLAY, trace sand	July 29,	CL	

Project No. 25-0049 Client: Davad Investments INC
Project: 50 Stephanie Street

○Depth: 12.2 m Sample Number: BH2, Sample 6

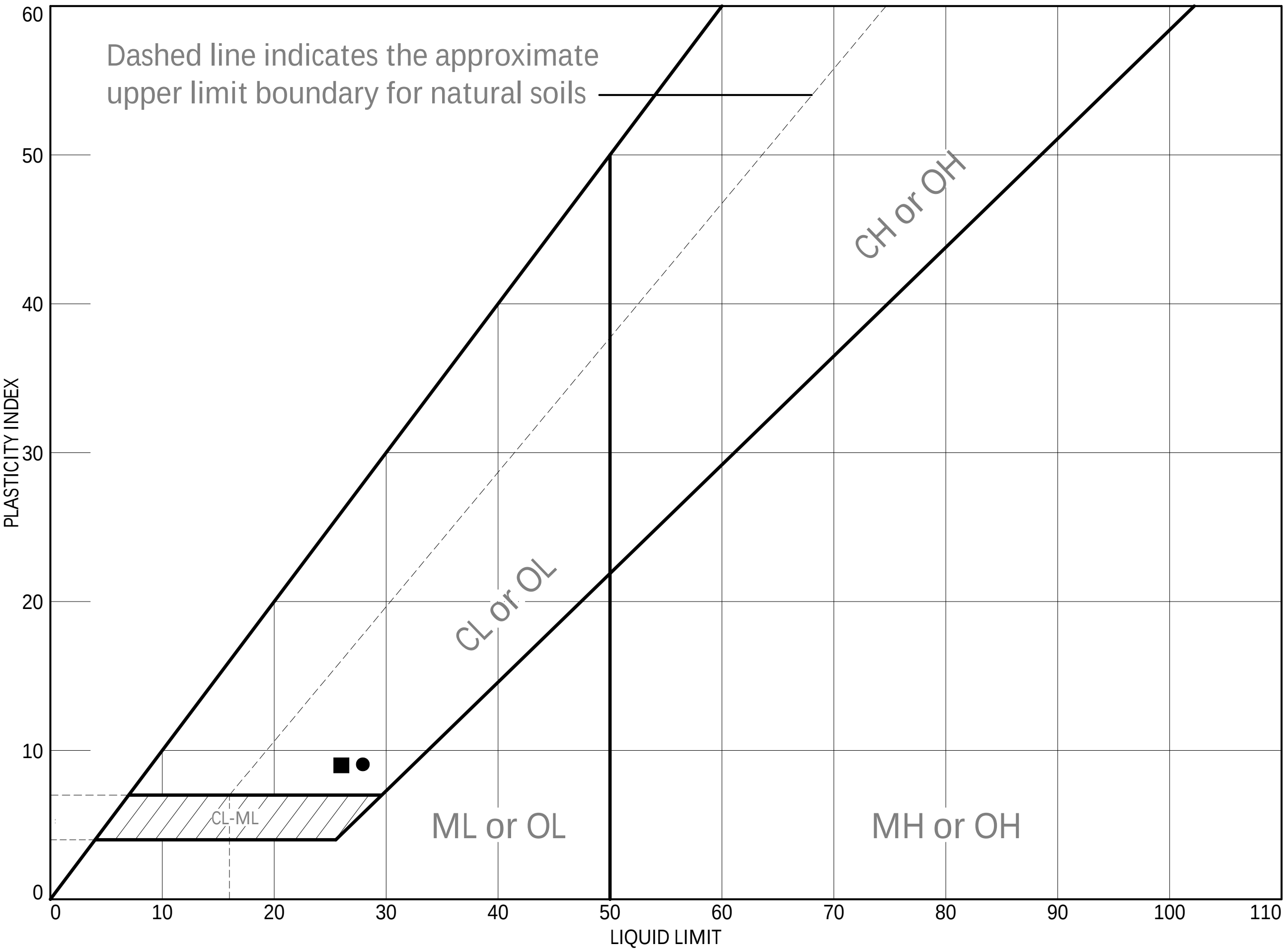
Remarks:

Groundwater Environmental Management Services Inc.

Concord, Ontario

Figure C-2

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SANDY CLAYEY SILT, trace gravel	28	19	9	85	69	CL
■	SILT and CLAY, trace sand	26	17	9	99	98	CL

Project No. 25-0049 Client: Davad Investments INC

Project: 50 Stephanie Street

●Depth: 6.1 m Sample Number: BH2, Sample 2

■Depth: 12.2 m Sample Number: BH2, Sample 6

Groundwater Environmental Management Services Inc.

Concord, Ontario

Remarks:

Figure C-3

Tested By: AC

Checked By: KC

Appendix E

Certificate of Chemical Analysis

Your Project #: 25-0049
Your C.O.C. #: 1044731-01-01

Attention: Kellen Campbell

Groundwater Environmental Management Services Inc.
150 Rivermede Rd
Unit # 9
Concord, ON
CANADA L4K 3M8

Report Date: 2025/08/07
Report #: R8589324
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C593026
Received: 2025/07/30, 15:07

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
pH CaCl2 EXTRACT	1	2025/08/06	2025/08/06	CAM SOP-00413	EPA 9045 D m
Sulphate (20:1 Extract)	1	2025/08/06	2025/08/07	CAM SOP-00464	MOE E3013 m

Remarks:

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, EPA, APHA or the Quebec Ministry of Environment.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Your Project #: 25-0049
Your C.O.C. #: 1044731-01-01

Attention: Kellen Campbell

Groundwater Environmental Management Services Inc.
150 Rivermede Rd
Unit # 9
Concord, ON
CANADA L4K 3M8

Report Date: 2025/08/07
Report #: R8589324
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C593026
Received: 2025/07/30, 15:07

Encryption Key

Please direct all questions regarding this Certificate of Analysis to:
Jolanta Goralczyk, Project Manager
Email: Jolanta.Goralczyk@bureauveritas.com
Phone# (905)817-5751
==

This report has been generated and distributed using a secure automated process.
Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by Rodney Major, General Manager responsible for Ontario Environmental laboratory operations.

RESULTS OF ANALYSES OF SOIL

Bureau Veritas ID		ATPF62			
Sampling Date		2025/07/24 11:00			
COC Number		1044731-01-01			
	UNITS	**BH2 SS3**	**RDL**	**MDL**	**QC Batch**
Inorganics					
Available (CaCl2) pH	pH	7.63			9983470
Soluble (20:1) Sulphate (SO4)	ug/g	120	20	10	9984069

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

TEST SUMMARY

Bureau Veritas ID: ATPF62
Sample ID: BH2 SS3
Matrix: Soil

Collected: 2025/07/24
Shipped:
Received: 2025/07/30

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	9983470	2025/08/06	2025/08/06	Nachiketa Gohil
Sulphate (20:1 Extract)	SKAL/EC	9984069	2025/08/06	2025/08/07	Alina Dobreanu

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	5.0°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
9983470	NGI	Spiked Blank	Available (CaCl2) pH	2025/08/06		100	%	97 - 103
9983470	NGI	RPD	Available (CaCl2) pH	2025/08/06	4.4		%	N/A
9984069	ADB	Matrix Spike	Soluble (20:1) Sulphate (SO4)	2025/08/07		NC	%	70 - 130
9984069	ADB	Spiked Blank	Soluble (20:1) Sulphate (SO4)	2025/08/07		89	%	70 - 130
9984069	ADB	Method Blank	Soluble (20:1) Sulphate (SO4)	2025/08/07	<20		ug/g	
9984069	ADB	RPD	Soluble (20:1) Sulphate (SO4)	2025/08/07	16		%	35

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Cristina Carriere

Cristina Carriere, Senior Scientific Specialist

Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by Rodney Major, General Manager responsible for Ontario Environmental laboratory operations.

C593026
2025/07/30 15:07

BUREAU VERITAS

Bureau Veritas
6740 Campobello Road, Mississauga, Ontario Canada L5N 2L8 Tel:(905) 817-5700 Toll-free:800-563-6266 Fax:(905) 817-5777 www.bvna.com

NONT-2025-07-6419

Page 1 of 1

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:	
Company Name:	#24874 Groundwater Environmental Management Ser	Company Name:		Quotation #:	~~C45815~~
Attention:	Accounts Payable	Attention:		P.O. #:	
Address:	150 Rivermede Rd Unit # 9 Concord ON L4K 3M8	Address:		Project:	25-0049
				Project Name:	
Tel:	(905) 907-3077 Fax: (905) 907-6617	Tel:	Fax:	Site #:	
Email:	Accounts.Payable@gemservicesinc.com	Email:	Keller.Campbell@gemservicesinc.com	Sampled By:	AC

Bottle Order #: 1044731

COC #: C#1044731-01-01

Project Manager: Jolanta Goralczyk

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011)
☐ Table 1 ☐ Res/Park ☐ Medium/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other ☐ **For RSC**
☐ Table ___

Other Regulations
☐ CCME ☐ Sanitary Sewer Bylaw
☐ Reg 558: ☐ Storm Sewer Bylaw
☐ MISA Municipality ______
☐ PWQO ☐ Reg 406 Table ______
☐ Other ______

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ______

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☑
(will be applied if Rush TAT is not specified):
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

Job Specific Rush TAT (if applies to entire submission)
Date Required: ______ Time Required: ______ ☐
Rush Confirmation Number: ______ *(call lab for #)*

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Toronto Sanitary&Storm Sewer (100-2016)	SO4+pH									# of Bottles	Comments
1		BH2 SS3	25/07/24	11:00	Soil												1	
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
Tom Alston	25/07/29	14:00	AC	25/07/30	1500	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	5/5/5	Present	✓	
		Intact	✓	

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BUREAU VERITAS'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVNA.COM/ENVIRONMENTAL-LABORATORIES/RESOURCES/COC-TERMS-AND-CONDITIONS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVNA.COM/ENVIRONMENTAL-LABORATORIES/RESOURCES/CHAIN-CUSTODY-FORMS-COCS.

#116a25

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

White: Bureau Veritas Yellow: Client

Bureau Veritas Canada (2019) Inc.