GEOTECHNICAL SITE INVESTIGATION

PROPOSED 32-STOREY RESIDENTIAL TOWER DEVELOPMENT (PHASE 2)
3 SELKIRK STREET, OTTAWA, ONTARIO

Riverain Developments Inc.

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FINAL

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Project No.: 2605215

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EXECUTIVE SUMMARY

GeoTerracs Inc. (GeoTerra) carried out a geotechnical site investigation for the proposed Phase 2 development at 3 Selkirk Street, Ottawa, Ontario, comprising a 32-storey residential tower ("Tower B") with one level of underground parking, on behalf of Riverain Developments Inc. This report presents final geotechnical design recommendations based on seven boreholes (BH26-1 to BH26-7), advanced within the tower footprint and along the proposed shoring perimeter, together with laboratory testing and information available for the Project at the time of writing.

The subsurface profile generally consists of fill to 1.5 to 2.9 m depth, underlain by glacial till to 2.9 to 4.6 m depth, weathered shale bedrock, and sound bedrock (limestone interbedded with shale) encountered as shallow as 6.1 mbgs and confirmed by rock coring in five of the seven boreholes. Groundwater was measured at 6.20 to 6.60 mbgs (El. approximately 50.1 to 50.2 m) in two monitoring wells, providing limited clearance (approximately 0.8 to 1.1 m) below the anticipated tower mat founding elevation.

A mat foundation extending across the full tower footprint is recommended for Tower B, bearing on weathered shale bedrock at an assumed underside elevation of approximately 5.5 mbgs, with a net bearing resistance of 1,000 kPa, subject to confirmation and approval by a GeoTerra geotechnical engineer in the field at the time of construction. Conventional pad and strip footings, bearing on till or weathered shale bedrock at approximately 4 mbgs, are recommended for the podium and underground parking areas. Uplift and sliding resistance for the tower mat should be confirmed by the structural engineer once factored loads are available; sliding and uplift are not expected to govern the design, but the limited groundwater clearance beneath the mat warrants explicit confirmation.

Excavation for the underground parking level is expected to encounter fill and till at the general excavation level, transitioning to weathered shale bedrock locally beneath the tower mat. A cantilevered soldier pile and lagging shoring system, embedded into weathered shale bedrock, is expected to be geotechnically feasible for the perimeter excavation; anchor design parameters bonded in till or weathered shale are also provided should supplementary lateral capacity be required. Given the reduced groundwater clearance at the tower mat excavation, dewatering measures should be planned as a design requirement rather than a contingency item, and weathered shale bearing surfaces should be protected from prolonged exposure to air due to the potential for pyrite-related volumetric expansion.

The Site is classified as Site Class C (Very Dense Soil and Soft Rock) in accordance with the Ontario Building Code, and the underlying soils and rock are not considered susceptible to liquefaction. The water-soluble sulphate content of the soil corresponds to a Class of Exposure S-3 (Moderate) under CSA A23.1; general use (GU) cement is not considered suitable for concrete in contact with the ground at this Site.

Given the limited groundwater clearance beneath the tower mat and the elevator pit, particular attention to permanent drainage and waterproofing is required at these locations. Field review and construction monitoring, including confirmation of bearing surfaces, shoring installation, and groundwater control, will be required throughout construction, as outlined in Section 8.0.

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1.0 INTRODUCTION

GeoTerracs Inc. (GeoTerra) has been retained by Riverain Developments Inc. (the "Client") to conduct a geotechnical investigation for the proposed Phase 2 development located at 3 Selkirk Street, Ottawa, Ontario (the "Site"). This report presents the findings of the field investigation, laboratory testing program, and geotechnical recommendations for the proposed 32-storey residential tower ("Tower B") and associated one-level underground parking garage.

This report is specifically and exclusively for the Client and consultants collaborating on the Project for the purposes of detailed design and construction. Use of this report, in whole or in part, by any other party or for any other purpose is not authorized. This report is subject to the limitations presented in Section 9.0.

1.1 Project Understanding

Our understanding of the Project is based on the Client's request and the following documents:

- Site Plan and Master Plan drawings, Project No. 2511, prepared by rla/architecture, dated February 13, 2026 (Site Plan Control Application submission).
- GeoTerra Proposal No. P2606145, Geotechnical Site Investigation, Excess Soil Planning and Reporting, and Imported Soil Sampling Program, dated June 16, 2026.

It is understood that the proposed development consists of a 32-storey residential tower (Tower B) with one level of underground parking accessed from Selkirk Street, forming Phase 2 of the wider Riverain Development.

For the purposes of this study, the below-grade structure is assumed to consist of a single level of underground parking, with founding levels consistent with the current design information available at the time of reporting.

1.2 Scope of Work

The scope of work included drilling seven (7) boreholes (BH26-1 to BH26-7): three boreholes (BH26-1 to BH26-3) within the Phase 2 tower footprint, and four additional boreholes (BH26-4 to BH26-7) along the proposed shoring perimeter, following the excavation boundary along North River Road and Selkirk Street, in general accordance with GeoTerra Proposal No. P2606145 (Option B).

This report also draws on prior geotechnical and environmental information completed by others for the wider Riverain Development, as summarized in Section 2.

The scope of work also included the preparation of this geotechnical report to:

(i) describe the subsurface and groundwater conditions at the Site based on the borehole and laboratory information;

(ii) present the borehole location plan, records of the borehole logs, and laboratory test results;

(iii) provide an assessment of the anticipated geotechnical conditions influencing the design and construction of the proposed tower foundation, underground parking excavation, and temporary shoring; and,

(iv) present geotechnical design parameters and recommendations for foundation design, excavation and shoring, dewatering, seismic site classification, and other items required to advance the detailed design and construction of the Project.

2.0 SITE DESCRIPTION AND GEOLOGICAL FORMATION

2.1 Summary of Site Description

The Site is located at 3 Selkirk Street, Ottawa, Ontario, and forms Phase 2 of the Riverain Development, bordered generally by Selkirk Street to the south, North River Road to the west, Phase 1 of the Riverain Development to the north, and Phase 3 of the Riverain Development to the east. At the time of the field investigation, the Site was occupied by a fenced outdoor storage and salvage yard with scattered stockpiled materials and equipment. The ground surface across the Site is relatively flat, at an approximate geodetic elevation of 56.4 to 56.7 m. The Site location and layout are shown on the Site Location Plan and Site Plan (Figures 1 and 2) in Appendix A.

2.2 Geological Formation

Based on published geological mapping, the Site is located within the Ottawa–St. Lawrence Lowland, part of the Great Lakes–St. Lawrence physiographic region. The area is underlain by relatively flat-lying Paleozoic sedimentary bedrock, consisting primarily of shale with interbedded limestone, consistent with the Ottawa Group, which unconformably overlies Precambrian crystalline rocks of the Canadian Shield. Over this bedrock, the Ottawa basin is typically mantled by Quaternary deposits comprising glacial till and, in low-lying areas, post-glacial marine and glaciolacustrine sediments of the Champlain Sea.

In the vicinity of the Site, however, no marine or glaciolacustrine clay was encountered; the surficial deposits consist of fill directly overlying glacial till, in turn overlying weathered and sound Paleozoic bedrock (limestone interbedded with shale) encountered as shallow as 6.1 mbgs.

3.0 FIELD INVESTIGATION AND TESTING

3.1 Underground Utility Clearance

Utility clearance requisitions were submitted to Ontario One Call (ON1Call) to obtain public utility locates, and private utility clearances were completed by a sub-contractor assigned by GeoTerra. All utility clearance documents were obtained before the commencement of drilling.

3.2 Site Layout and Surveying Operations

A geodetic survey for the boreholes was completed using an Emlid Reach RS2+ Multi-band RTK GNSS receiver, which was operated in Real-Time Kinematic (RTK) mode using Networked Transport of RTCM via Internet Protocol (NTRIP) to receive real-time correction data and enable centimeter-level accuracy.

The coordinates and elevations of the boreholes are referenced to the Canadian Spatial Reference System (CSRS) UTM NAD 1983 Zone 18 North coordinate system.

The location, coordinates and elevation of all boreholes are presented on Table 1 and Figure 2 in Appendix A; the borehole logs are provided in Appendix B.

Table 1: Borehole Information Summary

BOREHOLE	DRILLING DATE	COORDINATES (NAD83(CSRS)/UTM ZONE 18N)			BOREHOLE INFORMATION	
		NORTHING	EASTING	GEODETIC ELEVATION (m)	DEPTH (m)	EL. (masl)
BH26-1	Jul. 21, 2026	5,031,203.6	447,628.2	56.71	12.60	44.11
BH26-2	Jul. 21, 2026	5,031,176.2	447,635.4	56.56	7.98	48.58
BH26-3	Jul. 22, 2026	5,031,142.5	447,648.9	56.40	11.07	45.33
BH26-4	Jul. 22, 2026	5,031,144.3	447,681.5	56.38	11.13	45.25
BH26-5	Jul. 23, 2026	5,031,166.8	447,651.4	56.43	8.86	47.57
BH26-6	Jul. 23, 2026	5,031,149.9	447,661.9	56.69	7.62	49.07
BH26-7	Jul. 23, 2026	5,031,189.5	447,641.4	56.40	7.62	48.78

3.3 Fieldwork

The field investigation was carried out between July 21 and July 23, 2026. It comprised drilling seven (7) boreholes (BH26-1 to BH26-7), two of which (BH26-6 and BH26-7) were instrumented with groundwater monitoring wells.

Drilling was completed using a CME 75 truck-mounted drill rig equipped with hollow-stem augers and NQ wireline diamond coring equipment. The drilling equipment was owned and operated by Forage Grenville Drilling, Grenville, Quebec.

Soil samples were generally obtained at regular depth intervals using a split-spoon sampler in general accordance with the Standard Penetration Test (SPT), ASTM D1586. Boreholes BH26-1 through BH26-5 were advanced through the overburden and cored a minimum of 3 m into bedrock; BH26-6 and BH26-7 were both terminated at practical refusal on inferred weathered shale bedrock without coring. As the subsurface at the Site consists of fill, till, and bedrock rather than clay, Field Vane Shear (FVS) testing and undisturbed Shelby tube sampling were not required for this program.

Two (2) 51 mm diameter flush-mount groundwater monitoring wells were installed, in Boreholes BH26-6 and BH26-7. The BH26-6 installation consisted of a 3.05 m long well screen (4.57 to 7.62 mbgs) attached to 51 mm diameter PVC riser pipe; the BH26-7 installation consisted of a 3.20 m long well screen (4.42 to 7.62 mbgs) attached to 51 mm diameter PVC riser pipe. Both wells terminate in a flush-mount protective cover at ground surface. The wells were developed following installation, and groundwater level measurements were subsequently obtained (Subsection 5.2).

Drilling and sampling were supervised on a full-time basis by GeoTerra personnel. All boreholes were logged during drilling, and all samples were labelled sequentially as recovered. Disturbed samples were sealed in zip-lock plastic bags to minimize moisture loss and transported to the GeoTerra office for examination and testing. Following completion of drilling and well installation, the boreholes were backfilled with cuttings or bentonite-sand hole plug materials to ground surface, and the monitoring wells were backfilled with clean sand within the screened interval and bentonite-sand hole plug materials above the screen to ground surface. The field logs are presented in Appendix B.

4.0 LABORATORY WORK DESCRIPTION

The soil and rock core samples collected during fieldwork were transported to GeoTerra's office for the purposes of sample and log reviews, analysis, identification, and classification. All samples underwent a thorough visual examination by a geotechnical engineer. Geotechnical laboratory testing was performed by an accredited laboratory on representative soil and rock core samples, and included:

- Grain size (sieve and hydrometer) analysis, including natural moisture content, on three (3) representative soil samples
- Unconfined compressive strength (UCS) testing on three (3) representative rock core samples

Eurofins Environment Testing in Ottawa carried out chemical tests on two (2) representative soil samples including pH, chloride, sulphate, resistivity, and electrical conductivity.

Laboratory test results are included in Appendix C.

All samples taken from the boreholes that were not subjected to laboratory testing will be stored for a period of three (3) months from the date of the final report. Afterward, they will be disposed-off unless a written notice regarding their disposition is provided to GeoTerra in the meantime.

5.0 SOIL STRATIGRAPHY AND PROPERTIES

A summary of the soil stratigraphy encountered in the boreholes is presented in Table 2 below and described in detail in the following subsections. Detailed borehole logs are presented in Appendix B.

Table 2: Summary of the Stratigraphy in the Boreholes

BOREHOLE	GROUND EL. (masl)	FILL DEPTH (m) [EL. (masl)]	TILL DEPTH (m) [EL. (masl)]	WEATHERED BEDROCK DEPTH (m) [EL. (masl)]	SOUND BEDROCK DEPTH (m) [EL. (masl)]
BH26-1	56.71	0.00–1.52 [56.71–55.19]	1.52–3.05 [55.19–53.66]	3.05–11.07 [53.66–45.64]	11.07–12.60* [45.64–44.11]
BH26-2	56.56	0.00–2.74 [56.56–53.83]	2.74–3.81 [53.83–52.76]	3.81–6.10 [52.76–50.47]	6.10–7.98* [50.47–48.59]
BH26-3	56.40	0.00–2.90 [56.40–53.50]	2.90–4.42 [53.50–51.98]	4.42–9.55 [51.98–46.85]	9.55–11.07* [46.85–45.33]
BH26-4	56.38	0.00–2.90 [56.38–53.48]	2.90–4.11 [53.48–52.27]	4.11–9.60 [52.27–46.78]	9.60–11.13* [46.78–45.25]
BH26-5	56.43	0.00–1.68 [56.43–54.75]	1.68–3.05 [54.75–53.38]	3.05–7.34 [53.38–49.09]	7.34–8.86* [49.09–47.57]
BH26-6	56.69	0.00–2.90 [56.69–53.79]	2.90–4.42 [53.79–52.27]	4.42–7.62* [52.27–49.07]	—
BH26-7	56.40	0.00–2.29 [56.40–54.11]	2.29–4.42 [54.11–51.98]	4.42–7.62* [51.98–48.78]	—

* Denotes the termination depth of boreholes

5.1 Soil Stratigraphy

5.1.1 Fill

Fill was encountered at ground surface in all seven boreholes (BH26-1 to BH26-7), extending to depths ranging from 1.52 mbgs (El. 55.19 m) in Borehole BH26-1 to 2.90 mbgs (El. 53.48 to 53.79 m) in Boreholes BH26-3, BH26-4, and BH26-6.

The fill generally consisted of silty sand, some to trace gravel, trace clay, grey to dark brown or greyish brown, dry to moist, and ranging from very loose to very dense in consistency.

Recorded SPT N-values within the fill ranged from 2 blows/300 mm (BH26-3) to 72 blows/300 mm (BH26-6). The lower N-values were generally recorded in the upper 1 to 2 m of the fill at Boreholes BH26-3, BH26-4, and BH26-6, consistent with a variably compacted, uncontrolled fill associated with the Site's prior use.

Grain size analysis was carried out on one representative fill sample, BH26-6 [SS3], summarized in Table 3. The sample classifies as a silty, gravelly sand, with gravel content of 17.2%, sand content of 49.1%, silt content of 23.7%, and clay content of 10.0%. Natural moisture content was 11.4%.

5.1.2 Till

A glacial till deposit was encountered beneath the fill in all seven boreholes, extending to depths ranging from 2.90 mbgs (El. 53.50 m) in BH26-3 to 4.57 mbgs (El. 51.86 m) in BH26-5.

The till generally consisted of silty sand, some to trace gravel, with weathered shale fragments and trace clay noted locally, brownish grey to dark brown or black, damp to moist, and ranging from compact to very dense in consistency.

Recorded SPT N-values within the till ranged from 9 blows/300 mm (BH26-6) to 71 blows/300 mm (BH26-3), indicating a predominantly dense to very dense deposit. The value of 9 blows/300 mm recorded in BH26-6 is anomalous relative to the remainder of the till dataset.

Grain size analysis was carried out on two representative till samples, BH26-5 [SS4] and BH26-7 [SS5], summarized in Table 3. The till classifies as a silty, gravelly sand, with gravel content of 14.8 to 18.2%, sand content of 44.1 to 54.5%, silt content of 20.3 to 35.1%, and clay content of 6.0 to 7.0%. Natural moisture content ranged from 4.9 to 8.6%.

Table 3: Grain Size Distribution Summary

Borehole [Sample ID]	DEPTH (M)	w (%)	GRAIN SIZE DISTRIBUTION (%)			
			GRAVEL	SAND	SILT	CLAY
BH26-5 [SS4]	2.29–2.90	8.6	14.8	44.1	35.1	6.0
BH26-6 [SS3]	1.52–2.13	11.4	17.2	49.1	23.7	10.0
BH26-7 [SS5]	3.05–3.66	4.9	18.2	54.5	20.3	7.0

Note: G = Gravel, S = Sand, M = Silt, C = Clay (percent by mass); w = natural moisture content.

5.1.3 Weathered Bedrock

A weathered bedrock layer was encountered beneath the till in all seven boreholes, at depths ranging from 3.05 mbgs (El. 53.66 m) in BH26-1 to 4.57 mbgs (El. 51.86 m) in BH26-5. The layer generally consisted of highly weathered shale, some to trace silty sand, grey to dark brown or black, moist to wet, and dense to very dense based on SPT N-values, which ranged from 22 to 99 blows/300 mm and included several intervals of practical refusal (recorded as 50/15 or similar).

In Boreholes BH26-3, BH26-4, BH26-5, and BH26-6, the lower portion of the weathered bedrock unit was advanced by augering to refusal on inferred sound bedrock, without SPT sampling or rock coring; this interval is described on the borehole logs as "inferred weathered shale bedrock."

GeoTerra notes that no rock core recovery or Rock Quality Designation (RQD) data are available within the inferred weathered bedrock interval.

5.1.4 Sound Bedrock

Sound bedrock, confirmed by NQ wireline coring, was encountered in five of the seven boreholes (BH26-1 through BH26-5), at depths ranging from 6.10 mbgs (El. 50.47 m) in BH26-2 to 11.07 mbgs (El. 45.64 m) in BH26-1. Boreholes BH26-6 and BH26-7 were terminated within the inferred weathered bedrock interval, at 7.62 mbgs, without reaching or coring sound bedrock.

The cored bedrock generally consisted of limestone interbedded with shale, grey to black, of fair to excellent quality based on Rock Quality Designation (RQD) values ranging from 52 to 96 percent, Total Core Recovery (TCR) from 83 to 100 percent, and Solid Core Recovery (SCR) from 58 to 98 percent.

Unconfined compressive strength (UCS) testing was carried out on three representative rock core samples, summarized in Table 4.

Table 4: Rock Core Unconfined Compressive Strength Summary

BOREHOLE [SAMPLE ID]	DEPTH (m)	UCS (MPa)	DESCRIPTION OF BREAK
BH26-1 [RC1]	11.30–11.54	193.5	Columnar vertical cracking through both ends.
BH26-3 [RC1]	10.40–10.60	82.8	Vertical cracking through both ends.
BH26-4 [RC1]	10.80–10.94	100.6	Vertical cracking through both ends, small cone formed on one end.

Note: Unconfined compressive strength (UCS) testing carried out in general accordance with ASTM D7012, Method C.

5.2 Groundwater

GeoTerra instrumented Boreholes BH26-6 and BH26-7 with 51 mm diameter flush-mount groundwater monitoring wells. Water levels measured in both wells are summarized in Table 5.

Table 5: Groundwater Observation and Monitoring Well Summary

BOREHOLE	SCREEN INTERVAL (mbgs)	WATER LEVEL OBSERVATION			REMARKS
		INSTALLATION DATE	MEASUREMENT DATE	DEPTH (mbgs) [EL. (masl)]	
BH26-6	4.57 – 7.62	July 23, 2026	July 31, 2026	6.60 [50.09]	51 mm diameter flush-mount well
BH26-7	4.42 – 7.62	July 23, 2026	July 31, 2026	6.20 [50.20]	51 mm diameter flush-mount well

Groundwater conditions at the Site should be interpreted with caution, as the observed levels reflect conditions only at the specific monitoring locations and dates noted in this report. Groundwater levels may vary seasonally and annually in response to precipitation, snowmelt, and changes in site or adjacent-site conditions.

5.3 Chemical Analysis

Chemical testing was carried out by Eurofins Environment Testing Canada Inc. in Ottawa on two representative soil samples, BH26-1 (Sample SS3, sampled July 21, 2026) and BH26-4 (Sample SS4, sampled July 22, 2026), for sulphate, water-soluble chloride, pH, electrical conductivity, and resistivity. Results are summarized in Table 6.

Table 6: Chemical Test Results Summary

BOREHOLE [SAMPLE ID]	DEPTH (m)	PH	SULPHATE (%)	CHLORIDE (%)	RESISTIVITY (ohm-cm)	ELECTRICAL CONDUCTIVITY (mS/cm)
BH26-1 [SS3]	1.52 – 2.13	8.17	0.06	<0.002	2040	0.49
BH26-4 [SS4]	2.29 – 2.90	7.78	0.12	0.008	1120	0.89

The implications of these results for the sulphate exposure classification of buried concrete and the corrosion potential of buried metallic elements are discussed in Section 6.0

6.0 DISCUSSIONS AND GEOTECHNICAL RECOMMENDATIONS

6.1 General

From a geotechnical perspective, the Site is considered suitable for the proposed development, subject to the recommendations presented in this report.

The following discussion is provided to assist the Client and Design Team with the selection and design of the foundation, excavation, and temporary shoring systems for the Project.

At this stage, detailed structural loading information has not yet been provided by the structural engineer; accordingly, the foundation recommendations presented herein are based on GeoTerra's estimate of typical loading for a development of this type and should be confirmed once factored structural loads are available (Subsection 6.4).

The proposed development comprises a 32-storey residential tower (Tower B) with one level of underground parking, to be supported on a mat foundation across the tower footprint (Subsection 6.4.3) and conventional pad and strip footings in the podium and parking areas (Subsections 6.4.1 and 6.4.2), bearing on native till or weathered shale bedrock. Particular attention is required for bearing capacity and settlement of the tower mat and the podium and parking footings on till and weathered shale bedrock (Subsection 6.4).

The construction-related comments in this section are intended to highlight geotechnical issues that may affect the Contractor's means and methods. The Contractor remains responsible for construction safety and site-specific means and methods, in accordance with the Occupational Health and Safety Act and O. Reg. 213/91 (Construction Projects).

6.2 Site Preparation

Site preparation should include the removal of existing surface materials, stockpiled materials, and debris associated with the Site's current use as an outdoor storage yard, followed by stripping of topsoil and any unsuitable fill from within the proposed building and pavement footprints. Excavation should be carried out in stages and in conjunction with an appropriate temporary protection system to protect adjacent buildings, municipal services, and to maintain worker safety.

Any unknown or abandoned underground tanks, pits, or utilities encountered during demolition and excavation should be made safe, removed or appropriately decommissioned in consultation with the Owner and relevant authorities.

6.2.1 Buried Services

Public and private utility owners should be notified prior to the commencement of any construction activities at the Site. Existing underground utilities in the vicinity of the proposed excavation and shoring pile locations should be located and reviewed before undertaking work to identify potential conflicts. Any utilities exposed or intersected will need to be supported, protected, temporarily diverted or permanently relocated, as appropriate.

6.2.2 Excavation Impact on Adjacent Structures

The Designer and Contractor should account for the influence of the proposed excavation and shoring installation on adjacent structures, roadways, and buried municipal services along North River Road and Selkirk Street.

A pre-construction condition survey of adjacent structures, roadways, and municipal services is recommended prior to commencement of excavation, together with a vibration and settlement monitoring program during excavation and shoring installation.

6.3 Excavation

6.3.1 Overburden Excavation

Shoring piles, together with any anchors, are expected to be installed through the overburden (fill and till) and embedded into the underlying weathered shale bedrock, in accordance with the recommendations of Subsection 6.3.2.

Groundwater measured in BH26-6 and BH26-7 at 6.60 mbgs and 6.20 mbgs, respectively (El. approximately 50.1 to 50.2 m; Subsection 5.2), is below the anticipated general excavation level of approximately 4 mbgs (El. approximately 52.4 to 52.7 m), and below the anticipated founding level of the tower mat foundation of approximately 5.5 mbgs (El. approximately 50.9 to 51.2 m; Subsection 6.4.3), though with limited clearance at the mat founding level.

For the purposes of the Occupational Health and Safety Act and O. Reg. 213/91 (Construction Projects), the overburden soils, including fill and till, should be considered Type 3 soil. Weathered shale should be considered Type 2 soil. Where excavation walls are not supported by an engineered shoring system, the unsupported portion of the excavation face exceeding 1.2 m in height should be sloped no steeper than 1 horizontal to 1 vertical, in accordance with the requirements of the Regulation.

The final shoring approach for the perimeter of the excavation should be confirmed by a shoring engineer, incorporating the geotechnical parameters presented in Subsection 6.3.2.

6.3.2 Temporary Support System

A Temporary Protection System (TPS) or engineered shoring system will be required along the perimeter of the excavation for the underground parking level, particularly along the property lines and adjacent to North River Road and Selkirk Street.

Given the general excavation depth of approximately 4 m at the perimeter, and the groundwater level below the base of excavation (Subsection 6.3.1), a cantilevered soldier pile and lagging system is expected to be geotechnically feasible for the perimeter shoring, in accordance with the Canadian Foundation

Engineering Manual, 5th Edition (Canadian Geotechnical Society, 2023), Subsection 20.8.1.1, which identifies soldier piles and lagging as a suitable cantilevered wall system for excavations up to approximately 5 m in height.

GeoTerra recommends that the shoring piles be embedded into weathered shale bedrock, rather than terminated within the overlying till, given the higher passive resistance and stiffer toe-fixity condition this stratum provides (SPT N-values of 22 to 99, including several intervals of practical refusal), its role in controlling top-of-wall deflection near the buried municipal services on Selkirk Street, and its reliable presence across the full perimeter (onset ranging from 3.05 to 4.57 mbgs in all seven boreholes).

Lateral earth pressure coefficients, unit weights, and passive resistance for the fill, till, and weathered shale bedrock are presented in Table 8 (Subsection 6.7). A uniform traffic and construction surcharge of 12 kPa should be applied along the North River Road and Selkirk Street frontages, in accordance with typical municipal design practice. Toe embedment design should use the passive coefficient (Kp) for weathered shale from Table 8, with a factor of safety of at least 1.5 applied to the calculated passive resistance.

Table 7: Anchor Design Parameters

BOND STRATUM	ULTIMATE LOAD TRANSFER	BASIS	FACTOR OF SAFETY
Till	130 kN/m of bond length	CFEM 5th Ed. (2023), Table 20.10	3, pending anchor pull-out testing
Weathered shale bedrock	145 kN/m of bond length	CFEM 5th Ed. (2023), Table 20.11	3, pending anchor pull-out testing

The following additional recommendations apply to the shoring design:

- Toe embedment should be designed using a free-earth support (or equivalent limit-equilibrium) method, with penetration specified as a minimum depth into weathered shale rather than to a fixed general excavation elevation, given the variable onset depth of this unit across the perimeter.
- The shoring should be designed to limit horizontal movement at the top of the wall to a level acceptable to the Designer, considering the proximity of North River Road, Selkirk Street, and the buried municipal services (200 mm sanitary sewer and 150 mm watermain) shown on the Site Plan along Selkirk Street. Deflection tolerance, rather than geotechnical bearing or passive capacity, is expected to govern the shoring design in this area.
- Toe advancement into weathered shale should be confirmed in the field, by auger resistance, drilling rate change, or equivalent means, with review by GeoTerra field personnel prior to backfilling or grouting each pile.
- Where the toe embedment and passive resistance in weathered shale do not achieve the required deflection performance, anchors (Table 7) should be used as the supplementary lateral capacity measure, with capacity confirmed by proof and performance testing on the first several production anchors (CFEM Section 20.8.6.2).

- Given the groundwater level below the base of the perimeter excavation (Subsection 6.3.1), significant seepage at the shored perimeter is not anticipated under the design condition; groundwater control for the deeper interior mat excavation is addressed separately in Subsection 6.3.4.

The deeper excavation required locally for the tower mat foundation (Subsection 6.4.3), where it is set back from the shored perimeter, is expected to be carried out as an open, sloped excavation within the OHSA slope limits presented in Subsection 6.3.1, rather than requiring engineered shoring, provided sufficient lateral working space is available within the excavation footprint. This should be confirmed once the mat footprint and its setback from the perimeter shoring line are finalized.

All excavation is to be carried out within an engineered TPS designed and stamped by a Professional Engineer licensed in the Province of Ontario.

6.3.3 Subgrade Preparation

Subgrade preparation for the proposed pad and strip footings, at the general 4 m excavation level, and for the tower mat foundation, at approximately 5.5 mbgs (Subsection 6.4.3), should extend below all fill to an undisturbed, compact to very dense native till or weathered shale bearing surface, in accordance with Subsection 6.4.

The bearing surface should be proof-rolled and reviewed by GeoTerra field personnel prior to placing concrete. Where the till bearing surface is confirmed to be competent and stable, footings and the mat may be poured directly on the exposed till. Where the till bearing surface is found to be soft or disturbed, it should be sub-excavated and replaced with a minimum 300 mm thick granular pad, extending a minimum of 150 mm beyond the footing footprint, compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD).

Where weathered shale bedrock forms the bearing surface, whether beneath the pad and strip footings or the tower mat, it should be protected from prolonged exposure to air, as regional shale can contain pyrite that oxidizes upon exposure to oxygen following a lowering of the groundwater table, causing volumetric expansion. GeoTerra recommends placing a minimum 75 mm thick mud slab (25 MPa concrete) over the exposed weathered shale bearing surface as soon as practical following approval of the bearing surface, in accordance with Section 15.8 of the Canadian Foundation Engineering Manual, 5th Edition (2023). Given the reduced groundwater clearance beneath the tower mat excavation (approximately 0.8 to 1.1 m at 5.5 mbgs; Subsection 6.3.4), this precaution is particularly important at that location, and subgrade preparation there should be coordinated with the dewatering measures described in Subsection 6.3.4 to maintain a dry, undisturbed bearing surface at the time of inspection and mud slab placement.

The mud slab, granular pad, or footing/mat concrete, as applicable, should be placed on the same day the bearing surface is approved, to the extent practical, to limit exposure of the till or weathered shale to air, precipitation, groundwater seepage, and construction traffic.

6.3.4 Temporary Construction Dewatering

Groundwater measured in BH26-6 and BH26-7 at El. approximately 50.1 to 50.2 m (Subsection 5.2) is below the anticipated general excavation level of approximately 4 mbgs (El. approximately 52.4 to 52.7 m), providing approximately 2.3 to 2.5 m of clearance, but only approximately 0.8 to 1.1 m below the anticipated tower mat founding level of approximately 5.5 mbgs (El. approximately 50.9 to 51.2 m; Subsection 6.4.3).

Given this limited clearance beneath the tower mat, localized groundwater inflow or seepage should be anticipated at the base of the deeper tower mat excavation, and dewatering measures should be planned accordingly rather than treated as a contingency-only item. Significant inflow is not anticipated at the general 4 mbgs perimeter excavation level.

Groundwater control should be planned using open sumps and perimeter ditches directed to a sump pit; the pumping capacity required should be confirmed once the excavation geometry and founding elevations are finalized, and should specifically account for the reduced clearance at the tower mat.

Water discharged from dewatering operations should be managed in accordance with applicable municipal and provincial requirements, including sediment control prior to discharge to the storm sewer or an approved outlet.

Given the reduced groundwater clearance at the tower mat founding level, GeoTerra recommends the dewatering requirement be assessed against the Ministry of the Environment, Conservation and Parks (MECP) Permit to Take Water (PTTW) and Environmental Activity and Sector Registry (EASR) thresholds at the design stage, rather than only in the event of unanticipated conditions during construction.

6.4 Foundation

Three foundation options were evaluated for the proposed Tower B and associated underground parking structure: conventional pad footings, strip footings, and a mat foundation extending across the full tower footprint.

6.4.1 Pad Footings (Podium and Parking Columns)

For conventional pad footings, approximately 2 m in width, founded at the general excavation level of approximately 4 mbgs:

- Bearing on till: SLS 700 kPa (25 mm settlement), ULS 1,050 kPa ($\phi = 0.5$), modulus of subgrade reaction (ks) = 25 MPa/m
- Bearing on weathered shale bedrock: 1,000 kPa (Subsection 6.4.3)

Total settlement of footings bearing on till is expected to be within a tolerance limit of 25 mm, with differential settlement between adjacent footings within a tolerance limit of 20 mm. Settlement of footings bearing on weathered shale bedrock is expected to be small and is not anticipated to govern the design.

These values are specific to a footing width of approximately 2 m. Bearing surface preparation, in accordance with Subsection 6.3.3, is required prior to placing concrete.

6.4.2 Strip Footings (Podium and Parking Walls)

For strip footings, approximately 3 m in width, founded at the general excavation level of approximately 4 mbgs:

- Bearing on till: SLS 640 kPa (25 mm settlement), ULS 960 kPa ($\phi = 0.5$), ks = 25 MPa/m
- Bearing on weathered shale bedrock: 1,000 kPa (Subsection 6.4.3)

Total settlement of footings bearing on till is expected to be within a tolerance limit of 25 mm, with differential settlement between adjacent footings within a tolerance limit of 20 mm. Settlement of footings bearing on weathered shale bedrock is expected to be small and is not anticipated to govern the design.

These values are specific to a footing width of approximately 3 m. Bearing surface preparation, in accordance with Subsection 6.3.3, is required prior to placing concrete.

6.4.3 Mat Foundation, Full Tower Footprint (Recommended)

For a mat foundation extending across the full Tower B footprint (approximately 907 m^2), founded at approximately 5.5 mbgs on weathered shale bedrock:

- Net bearing resistance: 1,000 kPa
- Modulus of subgrade reaction: 25 MPa/m

This value is subject to confirmation and approval by a GeoTerra geotechnical engineer in the field at the time of construction. Settlement of the mat is expected to be small and is not anticipated to govern the design. The underside of the mat elevation is assumed, for the purposes of this report, at approximately 5.5 mbgs, pending confirmation of the excavation depth and mat thickness by the structural engineer.

Differential settlement between the mat and the surrounding pad and strip footings, where those elements bear on till within a tolerance limit of 25 mm total and 20 mm differential settlement (Subsections 6.4.1 and 6.4.2), should be evaluated by the structural engineer once factored loads are confirmed, and a settlement-compatibility detail provided at the interface between these foundation elements.

The modulus of subgrade reaction presented above is a preliminary value. Optimization of the subgrade reaction can be carried out in collaboration with the structural engineer once factored structural loads and a preliminary mat geometry are available.

6.4.4 Floor Slab Design

The underground parking floor slab is anticipated to be constructed at a uniform elevation of approximately 4 mbgs across the entire underground parking level, including above both the tower mat foundation (Subsection 6.4.3, founded at approximately 5.5 mbgs) and the podium/parking pad and strip footings (Subsections 6.4.1 and 6.4.2). The same slab-on-grade assembly applies throughout, consisting of, from the subgrade or mat surface upward:

- a minimum 300 mm layer of engineered fill, compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD);
- a minimum 200 mm layer of clear stone, connected to the perimeter and sub-slab drainage system (Subsection 6.8);
- a vapour barrier, in accordance with CAN/CGSB-51.34 or ASTM E1745; and
- the concrete slab-on-grade, with control joints to be detailed by the structural engineer.

This assembly requires a minimum 500 mm clearance between the top of the tower mat and the underside of the floor slab; this should be confirmed once the structural engineer finalizes the mat thickness (Subsection 6.4.3), as a thicker mat may reduce or eliminate this clearance.

For a lightly loaded slab-on-grade, without heavy racking, process equipment, or concentrated storage loads, a maximum applied pressure of 25 kPa and a modulus of subgrade reaction of approximately 25 MPa/m, based on the slab bearing on the 300 mm engineered fill layer described above, can be used. This should be confirmed with the structural engineer; where loading exceeds 25 kPa, or involves concentrated or moving loads, GeoTerra should be consulted for review.

6.4.5 Uplift Resistance

Given the limited clearance between groundwater and the founding elevations of the tower mat and the podium/parking slab-on-grade (approximately 0.8 to 1.1 m under the design condition; Subsection 6.3.4), the potential for hydrostatic uplift on these elements should be assessed by the structural engineer.

For the tower mat foundation (Subsection 6.4.3), the dead load of the tower is expected to substantially exceed any plausible hydrostatic uplift force; this should nonetheless be confirmed by the structural engineer with an explicit flotation check, using the groundwater levels presented in Subsection 5.2 as the design water level, and an appropriate factor of safety against flotation.

For the podium and parking slab-on-grade (Subsection 6.4.4), the available dead load is comparatively limited, and the potential for hydrostatic uplift should be specifically evaluated. Where the slab dead load alone is insufficient to resist uplift under the design groundwater level, additional measures, such as slab thickening, anchorage, or a permanent underslab drainage and pressure relief system, may be required, and should be confirmed by the structural engineer in consultation with GeoTerra.

6.4.6 Sliding Resistance

Resistance to lateral sliding of the tower mat and the podium/parking footings can be provided by friction at the base of the foundation. A coefficient of friction of 0.5 for concrete cast against till or weathered shale bedrock can be used, together with a sliding resistance factor of 0.8, in accordance with the

Canadian Foundation Engineering Manual, 5th Edition (2023).

Given the substantial dead load and bearing area of the tower mat relative to the anticipated lateral demand, sliding is not expected to govern the design of the mat; this should be confirmed by the structural engineer using factored lateral loads once available.

Where additional sliding resistance is required, passive resistance from soil or backfill against the embedded edge of the foundation may be considered, using the passive earth pressure coefficients presented in Table 8 (Subsection 6.7), scaled in accordance with CFEM guidance to account for the mobilization compatibility between sliding and passive resistance.

6.5 Frost Protection

The proposed pad and strip footings (Subsections 6.4.1 and 6.4.2), founded at approximately 4 mbgs, and the tower mat foundation (Subsection 6.4.3), founded at approximately 5.5 mbgs, are both well below the depth required for frost protection and do not require additional measures against frost action.

For perimeter footings of heated portions of the structure founded above this depth, a minimum of 1.5 m of soil cover (or equivalent insulation) should be provided.

For exterior, unheated elements, such as isolated exterior piers or access ramp wall footings, which are more susceptible to frost-related movement, a minimum of 2.1 m of soil cover should be provided, or a reduced soil cover of 0.6 m in conjunction with rigid foundation insulation.

Should construction take place during winter, exposed foundation and engineered fill surfaces should be protected by the Contractor against freezing for the duration of construction, until adequate soil cover is in place. Backfill should not be placed in a frozen condition or on frozen subgrade.

Entrance slabs should be founded on frost walls, or alternatively incorporate an insulation detail, to prevent frost heaving at building entrances. Where building backfill underlies pavement, sidewalks, or other hard landscaping, the excavation should incorporate a frost taper to prevent differential heaving adjacent to the building.

6.6 Seismic Site Classification

Seismic site classification for this Project has been assessed in accordance with the Ontario Building Code, Division B, Article 4.1.8.4, "Site Properties," which specifies that Site Class be determined based on average shear wave velocity, energy-corrected average Standard Penetration Resistance (N_{60}), or average undrained shear strength in the upper 30 m of the profile.

Within the upper 30 m at the Site, the profile consists of fill to approximately 1.5 to 2.9 m below grade, underlain by glacial till to approximately 2.9 to 4.6 m below grade (Subsection 5.1.2), with SPT N-values generally in the range of 9 to 71 blows/300 mm. Weathered bedrock underlies the till (Subsection 5.1.3), with SPT N-values ranging from 22 to 99 blows/300 mm, including several intervals of practical refusal, transitioning to sound bedrock (limestone interbedded with shale) encountered as shallow as 6.1 m below grade and confirmed by rock coring at five of the seven boreholes (Subsection 5.1.4). Based on the predominance of N_{60} values exceeding 50 blows/300 mm within the fill, till, and weathered bedrock, and the presence of sound bedrock at shallow depth, the Site is classified as Site Class C (Very Dense Soil and Soft Rock), in accordance with Table 4.1.8.4.A of the Ontario Building Code.

Liquefaction susceptibility has been assessed at a screening level, in accordance with Section 18.6.3.1 of the Canadian Foundation Engineering Manual, 5th Edition (2023), based on soil type, deposit age, relative density, particle size, and depth to groundwater. The fill and till at the Site are predominantly situated above the measured groundwater table (Subsection 5.2) and are therefore unsaturated. Where saturated, the underlying material consists of a well-graded, dense to very dense glacial till containing significant fines and gravel content (Table 3), or weathered and sound bedrock, neither of which meets the soil type, gradation, or age criteria associated with liquefaction susceptibility. On this basis, the soils and rock underlying the Site are not considered susceptible to liquefaction. A quantitative liquefaction triggering analysis (CFEM Section 18.6.3.2) was not considered warranted, given the Site fails the screening criteria for liquefaction-susceptible material.

6.7 Lateral Earth Pressures

The following lateral earth pressure parameters are provided to assist Contractors and Designers with the design of permanent below-grade walls for the Project. These parameters supplement the temporary shoring parameters presented in Subsection 6.3.2.

The lateral earth pressure coefficients and unit weights presented in Table 8 are based on the stratigraphy and SPT data described in Section 5.0.

Table 8: Lateral Earth Pressure Design Parameters for Backfill and Native Soil

Material	Parameter					
	ϕ' (°)	c' (kPa)	γ (kN/m³)	K_o	K_a	K_p
Granular A	34	0	22	0.44	0.28	3.54
Granular B II	32	0	22	0.47	0.31	3.25
Fill	30	0	19	0.50	0.33	3.00
Till	35	0	21	0.43	0.27	3.69
Weathered Shale	38	0	21	0.38	0.24	4.20
Note: ϕ': effective friction angle – c': effective cohesion – γ: unit weight – K_o: coefficient of lateral earth pressure at rest – K_a: coefficient of active earth pressure – K_p: coefficient of passive earth pressure (for a vertical slope and a horizontal surface)						

The shoring and retaining wall designers are responsible for selecting the appropriate set of lateral pressure conditions (at rest, active, passive) based on wall type, deformation tolerances and construction sequence. Where wall movements are intentionally limited (e.g., to protect adjacent structures or services), at-rest earth pressure conditions should be assumed unless project-specific analyses demonstrate that lower pressures are appropriate.

Static lateral earth pressure can be calculated using the following equation:

$$\sigma_h = K \times (\gamma h + q)$$

where K is the lateral earth pressure coefficient. For yielding retaining walls, the active earth pressure coefficient, K_a, is recommended for design. For non-yielding permanent walls, such as foundation walls, the at-rest, K_o, is recommended for design. The resultant of the applicable static or at-rest force is assumed to act at h = 1/3H above the base of the wall where H is the Height of the wall. The unit weight of the retained soil "γ" is given in Table 8, and "q" is the value of any applied surcharge.

Hydrostatic pressure, based on the groundwater levels presented in Subsection 5.2, should be added to the static active earth pressure for the design of permanent below-grade walls. A seismic increment should also be added to the static active pressure, in accordance with Section 20.4 of the Canadian Foundation Engineering Manual, 5th Edition (2023).

The above lateral pressure coefficients are calculated assuming the wall back is vertical and the retained ground surface is level; these should be adjusted by the wall designer where sloped backfill or a battered wall face is used.

6.8 Permanent Drainage and Waterproofing

Groundwater at the Site was measured at 6.20 to 6.60 mbgs across the two monitoring wells (El. approximately 50.1 to 50.2 m), providing limited clearance below the tower mat foundation (Subsection 6.4.3 and Subsection 5.2). Given this limited clearance, and the presence of till and weathered shale bedrock, which are less free-draining than a clean granular soil, the following measures are recommended:

- A conventional waterproofing membrane system should be applied to the exterior face of all below-grade walls, designed and installed in accordance with the membrane manufacturer's specifications and the Ontario Building Code.
- A composite drainage system should be installed between the waterproofing membrane and the foundation wall, extending from the exterior finished grade to the founding elevation, to direct any water infiltration resulting from a breach of the waterproofing membrane toward the perimeter drainage system.
- A perimeter drain, consisting of a perforated drain pipe embedded in clear stone and wrapped in a non-woven geotextile filter fabric, should be installed at the base of the below-grade walls, connected to a sump and pump system at a low point within the structure.
- Below-slab drainage should be provided by the 200 mm clear stone layer described in Subsection 6.4.4, connected to the perimeter drain system or to slab drains discharging to the sump, to manage any incidental seepage beneath the slab.
- Exterior grades adjacent to the below-grade walls should be sloped away from the structure to limit surface water infiltration, with roof and surface drainage directed away from the foundation walls.

6.8.1 Elevator Pit Waterproofing

Given the limited groundwater clearance beneath the tower mat, particular attention should be given to waterproofing the elevator pit, which is typically the lowest point of the below-grade structure and most susceptible to water infiltration. A waterproofing membrane should be applied to the exterior of the elevator shaft foundation walls, extending over the vertical portion of the mat and down to its underside, in accordance with the manufacturer's specifications. A continuous PVC waterstop should be installed at the interface between the elevator shaft foundation walls and the mat. A 150 mm diameter perforated corrugated underfloor drainage pipe should be placed along the perimeter of the elevator pit sidewalls, with a gravity connection to a sump pump basin.

The perimeter and below-slab drainage systems should be connected to a sump with adequate pumping capacity and maintained accessible for inspection and cleanout. The mechanical engineer should design the sump and pump system, including appropriate redundancy for a below-grade parking structure.

6.9 Backfill

6.9.1 Engineered Fill

Any over excavation shall be leveled with lean concrete or a concrete mix of the same strength as the foundation system. A 75 mm thick concrete mud slab should be placed over the undisturbed excavation bottom, as approved by the Geotechnical Engineer.

All new fill soils that underlie floor slab, or other structural applications are considered as engineered fill. Engineered fill must meet the strict requirements as shown below:

- Engineered fill shall not be placed in frozen condition or placed on a frozen subgrade.
- Typically, a crushed well-graded material such as an OPSS 1010 Granular A or Granular B Type II is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions.
- Before placing any engineered fill, all debris, and unsuitable fill must be removed, and the subgrade proof rolled and approved by a geotechnical engineer. Any deficient areas should be repaired before Engineered Fill placement.
- Engineered fill shall be placed in maximum loose lifts of 300 mm and adequately compacted to achieve 98% of its SPMDD. Engineered fill must have full-time compaction testing by geotechnical personnel.

6.9.2 General Backfill and Grade Raise

The backfill placed against exterior foundations shall be free draining granular material meeting the grading requirements of an OPSS 1010 Granular B Type I, or Selected Subgrade Material (SSM), or equivalent granular material.

The exterior backfill should be placed and compacted as outlined below:

- Backfill should not be placed in frozen condition, or placed on a frozen subgrade;
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 300 mm. Each lift should be uniformly compacted to achieve 95% of its SPMDD;
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98% of its SPMDD;
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95% of its SPMDD;
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall;
- Entrance slabs should be founded on frost walls or alternatively have insulation details developed to prevent frost heaving at the building entrances; and
- In areas where the building backfill underlies pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

6.10 Underground Utilities

At the subject Site, the burial depth of water-bearing utility lines is typically 1.8 m below the ground surface or as dictated by local applicable codes. If this depth is not achievable, equivalent thermal insulation should be provided. The Contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

For the purposes of the Occupational Health and Safety Act and O. Reg. 213/91, utility trench excavations at this Site should be classified consistently with Subsection 6.3.1: fill and till as Type 3 soil, and weathered shale bedrock as Type 2 soil, at the applicable depth. Unsupported trench side slopes should be excavated no steeper than 1 horizontal to 1 vertical in Type 3 soil, consistent with Subsection 6.3.1, or lateral support such as a trench box should be used.

The engineer designing utilities should ensure the proposed utility pipes can tolerate compaction loads. The recommendations within this section are intended to supplement, and not replace, the most recent local municipal requirements.

6.10.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS 1010 "Granular A" material and be placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed. All utility pipes and high amps electrical conduits shall receive a minimum of 150 mm bedding.

- The use of clear stone is not recommended for use as pipe bedding. The voids in the stone may result in a low gradient water flow and infiltration of fines from the surrounding soils and cover materials, causing settlement and loss of support to pipes and structures.
- The cover material should be a service sand material or an OPSS 1010 "Granular A". The dimensions should comply with the pertinent specification section.
- The bedding, spring line, and cover should be compacted to at least 98% of its SPMDD.
- All covers are to be compacted to 100% SPMDD if they are intersecting structural elements.
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.10.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 95% of its SPMDD.
- The backfill placed in the upper 300 mm below the pavement subgrade elevation should be compacted to a minimum of 98% of its SPMDD.
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe; and
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction are as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

6.11 Cement Type and Corrosion Potential

Two (2) soil samples, from Boreholes BH26-1 and BH26-4, were submitted to Eurofins Environment Testing Canada Inc. in Ottawa for testing of chemical properties relevant to the corrosion potential of buried concrete and metallic infrastructure, including water-soluble sulphate, water-soluble chloride, pH, electrical resistivity, and electrical conductivity. Results are summarized in Table 6 (Subsection 5.3).

Electrical resistivity, pH value, and chloride concentration can provide an indication of the corrosion potential to buried ferrous or metallic infrastructure. The resistivity values obtained (1,120 and 2,040 ohm-cm) and pH values (7.78 and 8.17) indicate a mildly corrosive to moderately corrosive soil condition. The chloride concentrations obtained (0.008% and less than 0.002%) are low.

Concrete mix design and cover requirements should be confirmed by the Structural Engineer and materials supplier, taking into account the sulphate exposure classification discussed below and any additional project-specific requirements.

Corrosion protection should be provided for ferrous elements in direct contact with the native soil, including buried steel piping, tie rods, and reinforcement in contact with the soil, in accordance with the recommendations of a corrosion specialist, given the resistivity values obtained.

Table 9: Additional Requirements for Concrete Subjected to Sulphate Attack (CSA A23.1-24, Table 3)

CLASS OF EXPOSURE	DEGREE OF EXPOSURE	WATER SOLUBLE SULPHATE IN SOIL SAMPLE (%)	CEMENTING MATERIAL TO BE USED
S-1	Very Severe	>2.0	HS, HSL, HSb, HSLb, or HSe*
S-2	Severe	0.2 – 2.0	HS, HSL, HSb, HSLb, or HSe*
S-3	Moderate	0.1 – 0.2	MS, MSL, MSb, MSe*, MSLb, HS, HSL, HSb, HSLb, or HSe*

*Combinations of supplementary cementitious materials and portland, portland-limestone, or blended hydraulic cements may be used instead, provided they meet equivalent performance requirements against sulphate exposure (designated MSe or HSe).

The water-soluble sulphate content of the two soil samples tested was 0.06% (BH26-1) and 0.12% (BH26-4). Based on Table 9, and in accordance with CSA A23.1-24, the governing (maximum) value of 0.12% corresponds to a Class of Exposure S-3 (Moderate). General use (GU) cement is not considered suitable for concrete in contact with the ground at this Site; MS, MSL, MSb, MSLb, HS, HSL, HSb, HSLb, or an equivalent-performance MSe/HSe blended cementing material should be used instead, to be confirmed by the Structural Engineer.

7.0 PAVEMENT STRUCTURE

The pavement structure shall be placed on properly compacted engineered fill above the underground parking structure, or on a properly prepared native subgrade surface where pavement areas fall outside the underground parking footprint. Where native subgrade applies, preparation involves the removal of the fill and organic matter to expose competent native till or weathered shale. In either case, the subgrade should be proof-rolled and all loose and soft spots should be sub-excavated and replaced with OPSS 1010 Granular B Type II compacted to 95% SPMDD under the direction of a Geotechnical Engineer. The upper 0.3 m of backfill placed below the pavement subbase level should be compacted to a minimum of 98% of its SPMDD.

The pavement structure proposed in this design considers traffic ranging from light-duty passenger vehicles in surface parking areas to heavy-duty fire truck access along fire routes and drive aisles. The recommended pavement structure is summarized in Table 10.

Table 10: Pavement Structure

MATERIAL		THICKNESS (mm)	
		LIGHT DUTY	HEAVY DUTY
Surface Course	Superpave 12.5 mm, PG 58-34 or HL-3	50	40
Binder Course	Superpave 19 mm, PG 58-34 or HL-8	--	50
Base	OPSS 1010 Granular A	150	150
Subbase	OPSS 1010 Granular B Type II	300	300

Light duty applies to surface parking areas; heavy duty applies to fire routes and drive aisles.

The proposed pavement structure is designed for a proof-rolled subgrade. The base and subbase materials, Granular A for base and Granular B Type II for subbase, shall conform to OPSS 1010. Both base and subbase should be compacted to 100% SPMDD. Asphalt layers should be compacted to comply with OPSS 310.

8.0 CONSTRUCTION CONSIDERATIONS

This report presents final geotechnical design recommendations for the Project, based on the subsurface information obtained from the current field investigation and laboratory testing program (Sections 3 and 4). Notwithstanding the final nature of this report, an appropriate level of geotechnical field review and construction monitoring will be required during construction, as outlined below. The scope and frequency of this monitoring program should be confirmed with GeoTerra once the shoring design and construction sequencing are finalized.

GeoTerra recommends the following field review and construction monitoring items be incorporated into the Project's construction program:

- review and approval of the tower mat, pad footing, and strip footing bearing surfaces by a GeoTerra geotechnical engineer prior to concrete placement, including confirmation of the founding material (till or weathered shale bedrock) against the recommendations of Subsection 6.4;
- protection of exposed weathered shale bearing surfaces from prolonged exposure to air, in accordance with Subsection 6.3.3, given the potential for pyrite-related volumetric expansion described in Section 15.8 of the Canadian Foundation Engineering Manual, 5th Edition (2023);
- review of the temporary shoring installation, including confirmation of pile toe embedment into weathered shale bedrock and, where tied-back anchors are used, proof and performance testing of the first several production anchors, in accordance with Subsection 6.3.2;
- a pre-construction condition survey and a vibration and settlement monitoring program for adjacent structures, roadways, and buried municipal services during excavation, shoring installation, and dewatering, in accordance with Subsection 6.2.2;
- monitoring of groundwater control measures during excavation, particularly at the deeper tower mat excavation given the limited groundwater clearance identified in Subsection 6.3.4, with contingency provisions in the event that groundwater inflow exceeds the anticipated volumes;
- proof-rolling and review of pavement subgrade, whether native till/weathered shale or engineered fill placed above the underground parking structure, in accordance with Section 7.0;
- compaction testing of engineered fill, general backfill, and utility trench backfill materials, in accordance with Subsections 6.9 and 6.10; and
- confirmation that subsurface conditions encountered during construction are consistent with those described in this report.

Where subsurface conditions encountered during construction differ from those described in this report, GeoTerra should be consulted to confirm the continued applicability of the recommendations presented herein.

9.0 SCOPE OF THE REPORT AND LIMITATION OF LIABILITY

The characteristics of the soil and rock described in this report are based on boreholes and laboratory testing conducted at a specific period and depict the nature of the Site precisely where these boreholes were carried out. The characteristics between sampling points can vary significantly from the conditions encountered at the exact location where the samples were taken.

Furthermore, it should be noted that soil and rock formations may differ across the Site, and the boundaries between the various formations presented in this report, including those presented in Table 2, should not be considered fixed. GeoTerracs Inc. cannot guarantee the accuracy of these boundaries, which depend on factors such as the number of boreholes or the sampling method.

GeoTerra notes that the bearing resistance adopted for the tower mat foundation (Subsection 6.4.3) is subject to field confirmation, given the absence of rock core recovery or Rock Quality Designation (RQD) testing within the weathered bedrock interval at this horizon.

Additionally, the properties of the soil and rock can be significantly altered after construction activities are carried out on the Site or on adjacent sites. They can also be indirectly affected by exposure to freezing or weather conditions.

The groundwater conditions presented in this report apply solely to the Site. The groundwater levels indicated correspond only to the levels observed during the specified works, on the specified dates and locations (Subsection 5.2). It should be noted that these conditions may vary depending on precipitation, snowmelt, or season. Moreover, construction activities or modifications to the physical conditions of the Site or adjacent sites can also alter groundwater conditions.

In this report, the descriptions of the sampled materials were conducted using commonly recognized methods of identification and classification in geotechnical engineering. These methods may involve judgment and interpretation. In practice, these descriptions are presumed to be accurate and correct.

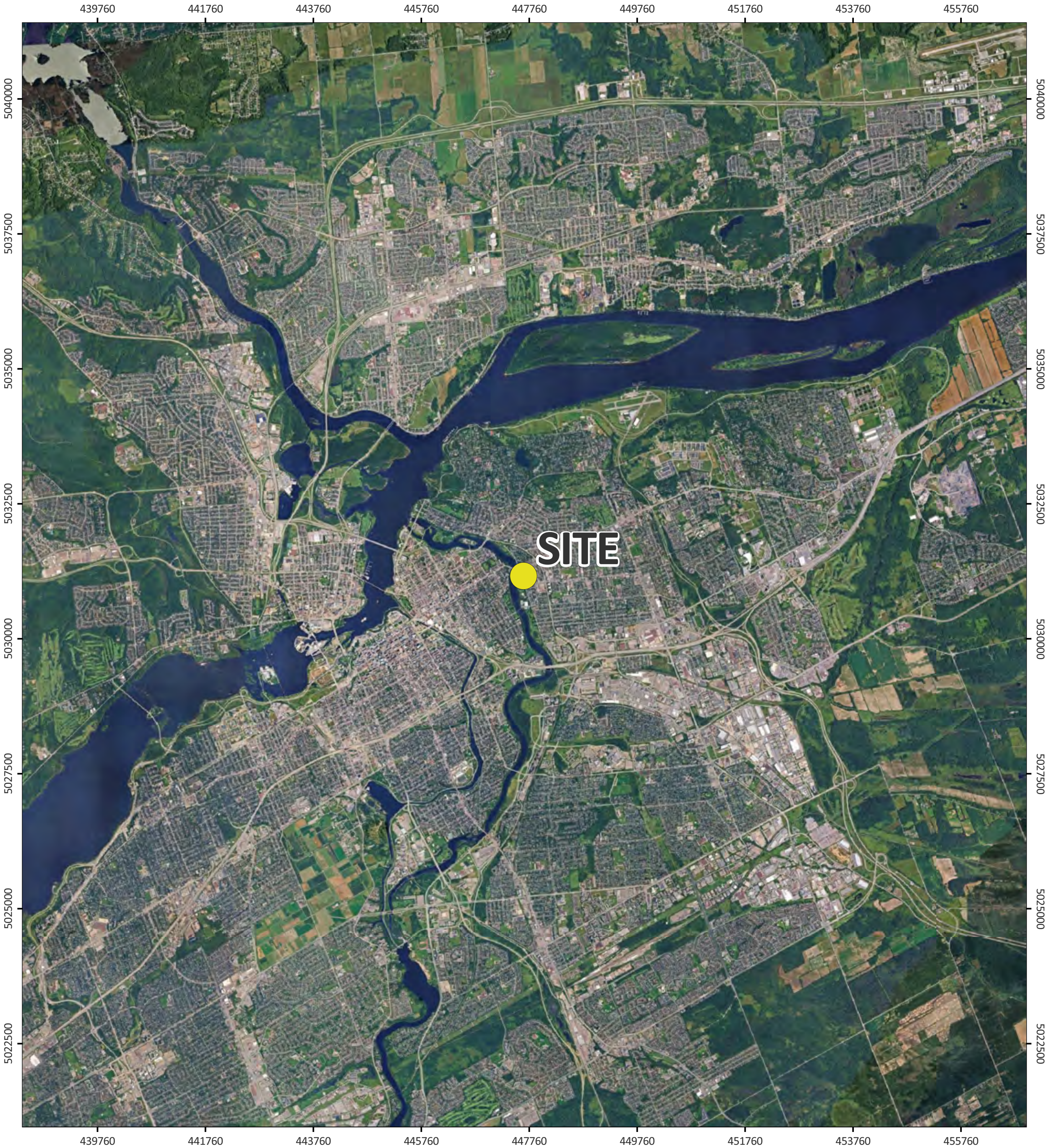
The results of tests and analyses are valid only for the samples described in this report. The interpretation of field and laboratory results, as well as the recommendations provided, is applicable only to the Site and the information available for the Project at the time of writing this report. They do not apply to any other project or site.

The recommendations given in this report are primarily intended for the Project design team. The number of boreholes needed to determine all subsurface conditions may exceed the number of boreholes conducted for design purposes. If the Project design is modified, GeoTerracs Inc. should be consulted to ensure that the recommendations in this report are still valid. In the event of modifications to the recommendations, additional field or laboratory work may be necessary.

It is recommended that site visits be conducted by GeoTerracs Inc. as the work progresses to confirm, and if necessary, modify the interpretations or recommendations provided in this report, in accordance with Section 8.0. If such verifications are not possible, GeoTerracs Inc. will assume no responsibility for the geotechnical interpretation that third parties may make of this report, especially if the design is altered or if Site conditions differ from those described in this report.

This report should not be reproduced, in whole or in part, without the permission of GeoTerracs Inc. This report does not constitute a legal opinion and should not be interpreted as such.

APPENDIX A – FIGURES



GeoTerra	Location: 3 Selkirk Street, Ottawa, ON	Project: Riverain - Phase 2
	Client: Riverain Developments Inc.	Title: Site Location Plan
0 2,000 4,000 m N	Drawn by: A. Senghani · Ref.: 2605215 · Approved by: A. Lamrani · Date: September 2026 · Version: 1 · Scale: 1:100,000	Figure: FIG-01

GeoTerra

Legend

- Site Boundary
- Borehole Location
- Monitoring Well Location

Title:	Site Plan
Project:	Riverain - Phase 2
Location:	3 Selkirk St, Ottawa, Ontario
Client:	Riverain Developments Inc.

Drawn by:	A. Senghani	Ref.:	2605215
Approved by:	A. Lamrani	Date:	September 2026
Version:	1	Scale:	1:400
Figure:	FIG-02		

0 11 22 m

N

APPENDIX B – BOREHOLE LOGS

BOREHOLE LOG: BH26-1

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,628.201
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,203.603
CLIENT:	Riverain Development	**DRILLING DATE:**	July 21, 2026	**GROUND ELEV.:**	56.71 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	12.6 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Ground Surface EL 56.71 m

Depth (M)	Material Description	Sample No.	Sample Type	RQD (%)	TCR (%)	SCR (%)	N Value	Additional Observations
0 – 1.52	FILL Silty Sand, some to trace gravel, trace clay, greyish brown to brownish grey, damp, compact 1.52 m / EL 55.19 m	1	SS				29	
		2	SS				12	
1.52 – 3.05	Till Silty Sand, some to trace gravel, brownish grey to grey, damp to moist, dense to very dense 3.05 m / EL 53.66 m	3	SS				30	
		4	SS				55	
3.05 –	Weathered Bedrock Highly weathered Shale bedrock, some to trace silty sand, grey to black, moist to wet, very dense per SPT "N" Values	5	SS				70	
		6	SS				50	
							50	
		7	SS					
							50/15	
		8	SS					
		9	SS				2	

Field Tests: Su_rs (45, 90, 135, 180); Su_pk (45, 90, 135, 180); NValue (20, 40, 60, 80)

Depth scale: FT 0–32; M 0–10

Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

BOREHOLE LOG: BH26-1

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,628.201
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,203.603
CLIENT:	Riverain Development	**DRILLING DATE:**	July 21, 2026	**GROUND ELEV.:**	56.71 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	12.6 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Depth FT	Depth M	Material Description	Stratigraphy	Sample No.	Sample Type	Rock Quality RQD (%)	Rock Quality TCR (%)	Rock Quality SCR (%)	N Value	Field Tests (+ Su_rs 45 90 135 180; × Su_pk 45 90 135 180; ■ NValue 20 40 60 80)	Additional Observations
		(continued)									
33–36	10–11	Highly weathered Shale bedrock, some to trace silty sand, grey to black, moist to wet, very dense per SPT "N" Values									
		11.07 m / EL 45.64 m									
36–41	11–12.6	Bedrock Limestone interbedded with Shale, good quality based on RQD value.		1	RC	77	83	80			
		12.60 m / EL 44.11 m									
42–65	12.6–20	End of Hole (12.6 m)									

Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

BOREHOLE LOG: BH26-2

GeoTerra

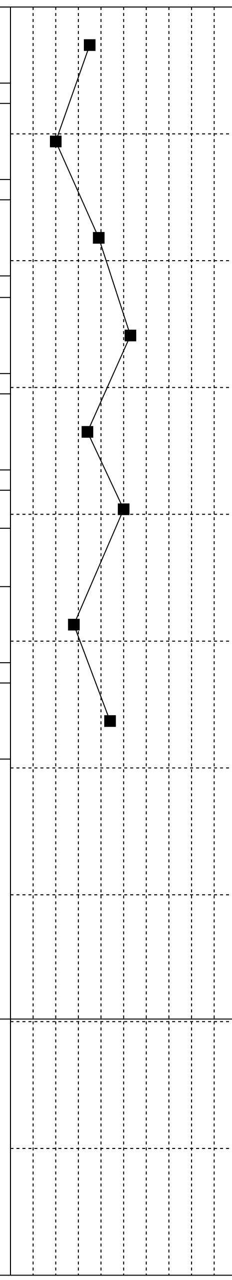
PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,635.356
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,176.212
CLIENT:	Riverain Development	**DRILLING DATE:**	July 21, 2026	**GROUND ELEV.:**	56.56 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	7.98 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Ground Surface EL 56.57 m

Depth (M)	Material Description	Sample No.	Sample Type	RQD (%)	TCR (%)	SCR (%)	N Value	Additional Observations
0 – 2.74 m	Fill Silty Sand, some to trace gravel, trace clay, dark brown to greyish brown, dry to damp, compact to very dense 2.74 m EL 53.83 m	1	SS				35	
		2	SS				20	
		3	SS				39	
		4	SS				53	
2.74 – 3.81 m	Till Silty Sand, some to trace gravel, some weathered shale, grey to dark brown or black, damp to moist, compact to very dense 3.81 m EL 52.76 m	5	SS				34	
3.81 – 6.10 m	Weathered Bedrock Weathered Shale, trace of sand and silt, dark brown to black, compact to very dense based on SPT "N" Value 6.10 m EL 50.47 m	6	SS				50	
		7	SS				28	
		8	SS				44	
6.10 – 7.98 m	Bedrock Limestone interbedded with Shale, fair quality based on RQD Values 7.98 m EL 48.59 m	1	RC	58	100	88		
		2	RC	52	90	58		
7.98 m	End of Hole (7.98 m)							

Field Tests: Su_rs (45, 90, 135, 180); Su_pk (45, 90, 135, 180); NValue (20, 40, 60, 80)



Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

BOREHOLE LOG: BH26-3

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,648.974
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,142.535
CLIENT:	Riverain Development	**DRILLING DATE:**	July 22, 2026	**GROUND ELEV.:**	56.4 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	11.07 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Ground Surface EL 56.40 m

Depth (M)	Material Description	Sample No.	Sample Type	RQD (%)	TCR (%)	SCR (%)	N Value	Additional Observations
0.00	Fill Silty Sand, some to trace gravel, trace clay, grey to dark brown, dry to moist, very loose to compact	1	SS				8	
		2	SS				2	
		3	SS				3	
		4	SS				13	
2.90 m EL 53.50 m	Till Silty Sand, some gravel (weathered shale), dark brown, moist, very dense	5	SS				71	
		6	SS				55	
4.42 m EL 51.98 m	Weathered Bedrock Highly weathered Shale, dark brown to black, dry, very dense	7	SS				50	
		8	SS				65	
6.10 m EL 50.30 m	Inferred weathered shale bedrock (auger through to refusal on sound bedrock)							
9.55 m EL 46.85 m	Bedrock	1	RC	85	100	97		

(continued)

Field Tests: Su_rs (45, 90, 135, 180); Su_pk (45, 90, 135, 180); NValue (20, 40, 60, 80)

BOREHOLE LOG: BH26-3

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,648.974
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,142.535
CLIENT:	Riverain Development	**DRILLING DATE:**	July 22, 2026	**GROUND ELEV.:**	56.4 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	11.07 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Depth FT	Depth M	Material Description	Stratigraphy	Sample No.	Sample Type	Rock Quality RQD (%)	Rock Quality TCR (%)	Rock Quality SCR (%)	N Value	Field Tests (Su_rs 45 90 135 180; Su_pk 45 90 135 180; NValue 20 40 60 80)	Additional Observations
		(continued)									
33–36	10–11.07	Limestone interbedded with Shale, good quality based on RQD value		1	RC	85	100	97			
	11.07	11.07 m EL 45.33 m									
37–65	11.07–20	End of Hole (11.07 m)									

BOREHOLE LOG: BH26-4

GeoTerra

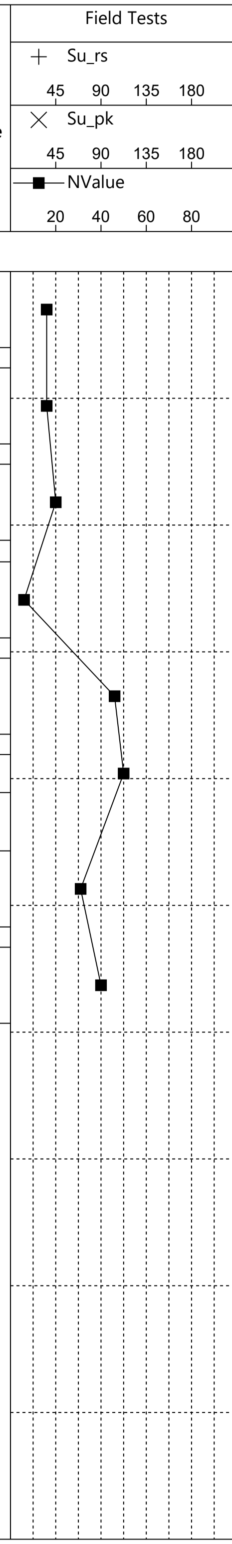
PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,681.596
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,144.311
CLIENT:	Riverain Development	**DRILLING DATE:**	July 22, 2026	**GROUND ELEV.:**	56.38 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	11.13 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Ground Surface EL 56.38 m

Depth (M)	Material Description	Stratigraphy	Sample No.	Sample Type	RQD (%)	TCR (%)	SCR (%)	N Value	Additional Observations
0	Fill Silty Sand, some to trace gravel, grey to dark brown, damp to moist, loose to compact		1	SS				16	
			2	SS				16	
			3	SS				20	
	2.90 m EL 53.48 m		4	SS				6	
2.90	Till Silty Sand, some gravel, trace clay, grey to dark brown, moist, dense to very dense		5	SS				46	
	4.11 m EL 52.27 m		6	SS				50	
4.11	Weathered Bedrock Highly weathered Shale, dark brown to black, moist to wet, dense to very dense based on SPT "N" Value		7	SS				31	
			8	SS				40	
	6.71 m EL 49.67 m		9	SS					
6.71	Inferred weathered shale bedrock (auger to refusal on sound bedrock) 9.60 m EL 46.78 m								
9.60	Bedrock *(continued)*		1	RC	96	99	98		

Field Tests



Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

BOREHOLE LOG: BH26-4

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,681.596
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,144.311
CLIENT:	Riverain Development	**DRILLING DATE:**	July 22, 2026	**GROUND ELEV.:**	56.38 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	11.13 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Depth FT	Depth M	Material Description	Stratigraphy	Sample No.	Sample Type	Rock Quality RQD (%)	Rock Quality TCR (%)	Rock Quality SCR (%)	N Value	Field Tests (Su_rs 45 90 135 180; Su_pk 45 90 135 180; NValue 20 40 60 80)	Additional Observations
		(continued)									
33–36	10–11	Limestone interbedded with Shale, excellent quality based on RQD value		1	RC	96	99	98			
		11.13 m EL 45.25 m									
37–65	11.13–20	End of Hole (11.13 m)									

Depth scale: FT 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65; M 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

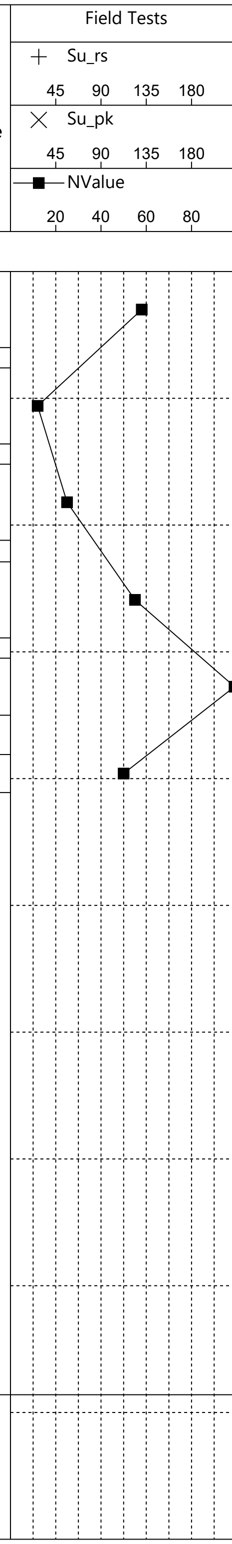
BOREHOLE LOG: BH26-5

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,651.476
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,166.824
CLIENT:	Riverain Development	**DRILLING DATE:**	July 23, 2026	**GROUND ELEV.:**	56.43 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	8.86 m	**COORDINATE SYSTEM:**	UTM

NOTES:

Ground Surface EL 56.43 m

Depth (m)	Material Description	Sample No.	Sample Type	RQD (%)	TCR (%)	SCR (%)	N Value	Additional Observations
0 – 1.68 m (EL 54.75 m)	Fill Silty Sand, some to trace gravel, trace clay, greyish brown to dark brown, dry to moist, compact to very dense	1	SS				58	
		2	SS				12	
1.68 – 3.05 m (EL 53.38 m)	Till Silty Sand, some to trace gravel, greyish brown to dark brown, moist, compact to very dense	3	SS				25	
		4	SS				55	
3.05 – 4.57 m (EL 51.86 m)	Weathered Bedrock Highly weathered Shale, some to trace of sand and silt, dark brown to black, moist, very dense	5	SS				99	
		6	SS				50	
4.57 – 7.34 m (EL 49.09 m)	Inferred weathered shale (auger to refusal on sound bedrock)							
7.34 – 8.86 m (EL 47.57 m)	Bedrock Limestone interbedded with Shale, fair quality based on RQD value	1	RC	67	100	83		
8.86 m	End of Hole (8.86 m)							



Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

BOREHOLE LOG: BH26-6

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,661.93
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,149.954
CLIENT:	Riverain Development	**DRILLING DATE:**	July 23, 2026	**GROUND ELEV.:**	56.69 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	7.62 m	**COORDINATE SYSTEM:**	UTM

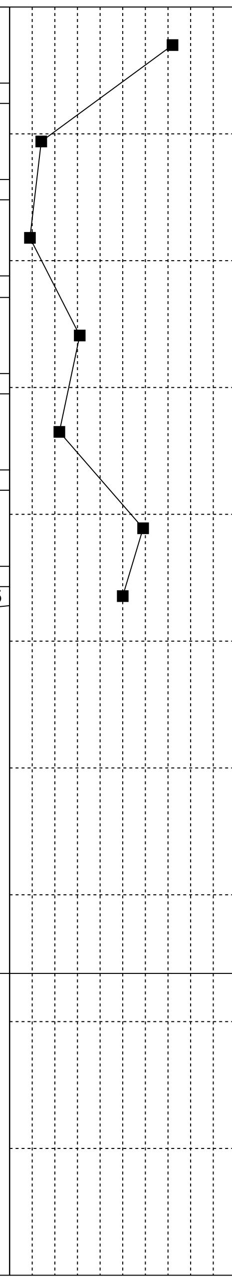
NOTES:

Columns: Depth (FT, M) | Material Description | Stratigraphy | Sample No. | Sample Type | N Value | Field Tests (Su_rs: 45, 90, 135, 180; Su_pk: 45, 90, 135, 180; NValue: 20, 40, 60, 80) | Laboratory Tests (Plastic Index: 20, 40, 60, 80; Liquid Limit: 20, 40, 60; Moisture: 20, 40, 60) | Additional Observations | Piezometer / Well

Ground Surface EL 56.69 m

Depth (m)	Material Description
0 – 2.90 m (EL 53.79 m)	**Fill** Silty Sand, some to trace gravel, trace clay, grey to dark brown, dry to moist, loose to very dense
2.90 – 4.42 m (EL 52.27 m)	**Till** Silty Sand, some to trace gravel, brown to dark brown, moist, compact to very dense
4.42 – 7.62 m (EL 49.07 m)	**Weathered Bedrock** Inferred weathered shale bedrock (auger to termination depth)
7.62 m	End of Hole (7.62 m)

Sample No.	Sample Type	N Value
1	SS	72
2	SS	14
3	SS	9
4	SS	31
5	SS	22
6	SS	59
7	SS	50/15



Piezometer / Well: #1 MW (JP)

Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

BOREHOLE LOG: BH26-7

GeoTerra

PROJECT NAME:	Riverain	**LOGGED BY:**	Alpesh Senghani	**EASTING:**	447,641.437
PROJECT NO.:	2605215	**REVIEWED BY:**	A. Lamrani	**NORTHING:**	5,031,189.52
CLIENT:	Riverain Development	**DRILLING DATE:**	July 23, 2026	**GROUND ELEV.:**	56.4 m
ADDRESS:	3 Selkirk Street	**TOTAL DEPTH:**	7.62 m	**COORDINATE SYSTEM:**	UTM

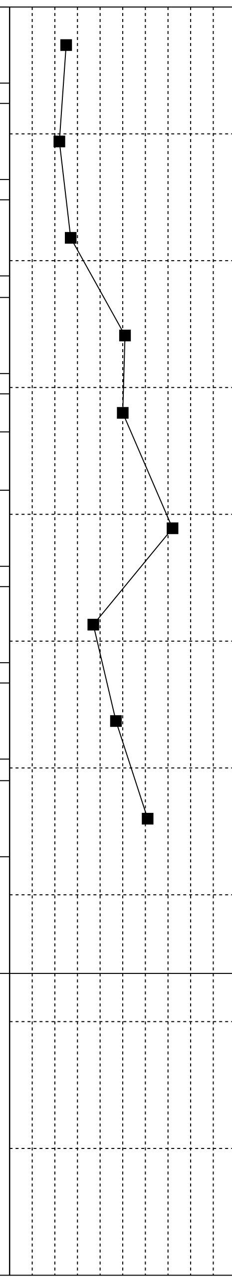
NOTES:

Ground Surface EL 56.40 m

Depth (M)	Material Description	Sample No.	Sample Type	N Value	Additional Observations	Piezometer / Well
0 – 2.29 m (EL 54.11 m)	Fill Silty Sand, some to trace gravel, grey to dark brown, dry to moist, compact	1	SS	25		#1 MW (JP)
		2	SS	22		
		3	SS	27		
2.29 – 4.42 m (EL 51.98 m)	Till Silty Sand, some to trace gravel, trace clay, greyish brown to dark brown, moist, very dense	4	SS	51		
		5	SS	50		
		6	SS	72		
4.42 – 7.62 m (EL 48.78 m)	Weathered Bedrock Highly weathered shale bedrock, moist to wet, dense to very dense based on SPT "N" Value	7	SS	37		
		8	SS	47		
		9	SS	61		
7.62 m	End of Hole (7.62 m)					

Field Tests: + Su_rs (45, 90, 135, 180); × Su_pk (45, 90, 135, 180); ■ NValue (20, 40, 60, 80)

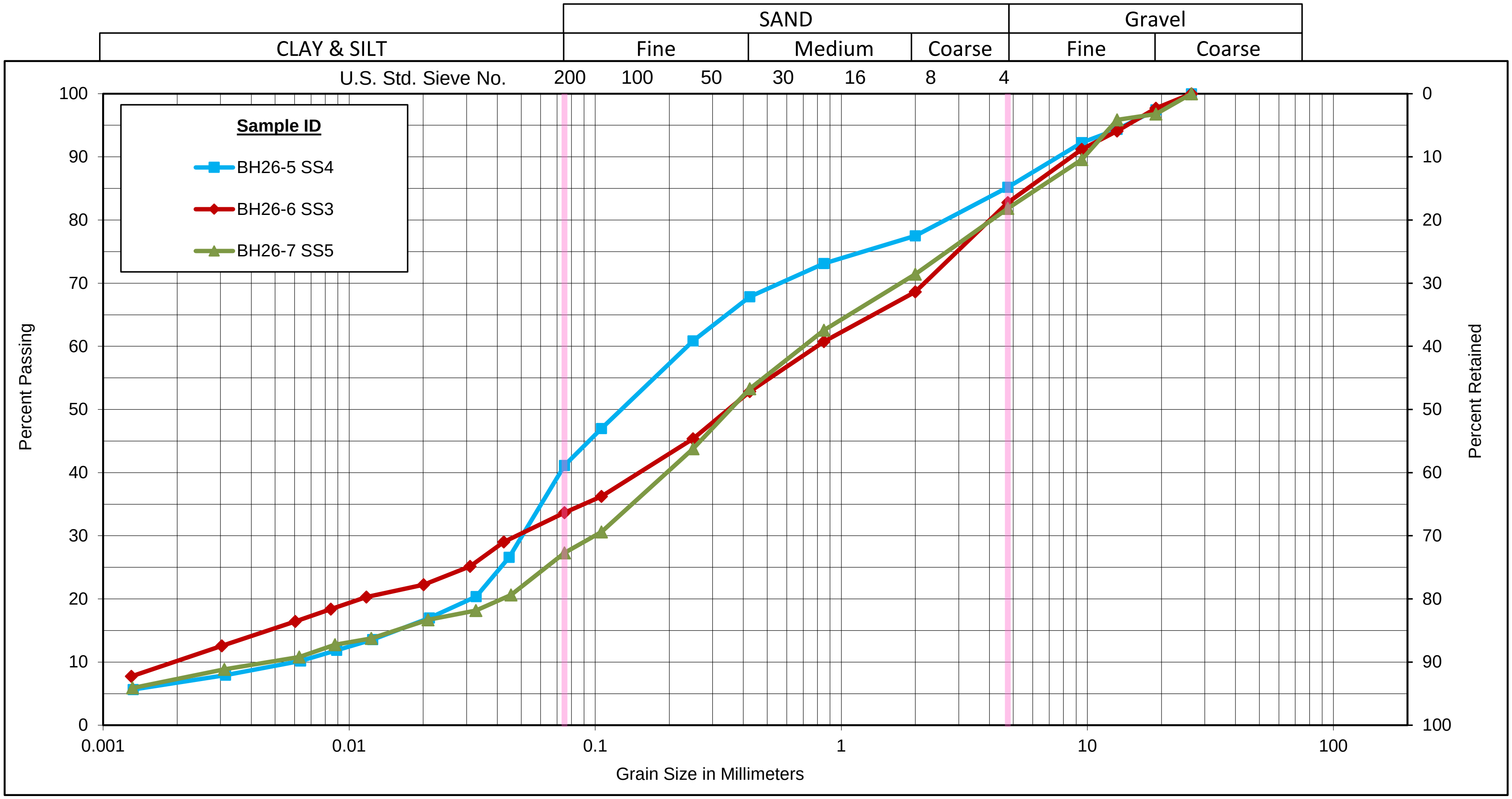
Laboratory Tests: ○ Plastic Index (20, 40, 60, 80); ● Liquid Limit (20, 40, 60); □ Moisture (20, 40, 60)



Disclaimer: The information provided herein is based on field observation and intended for geotechnical purposes only.

APPENDIX C – LABORATORY TESTS

Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH26-5 SS4	2.29-2.90 m	14.8	44.1	35.1	6.0
BH26-6 SS3	1.52-2.13 m	17.2	49.1	23.7	10.0
BH26-7 SS5	3.05-3.66 m	18.2	54.5	20.3	7.0

Stantec

GRAIN SIZE DISTRIBUTION

GeoTerra, File No. 2606145

Riverain Developments Inc., 3 Selkirk St., Ottawa

Figure No.

Project No. 121626390

Particle-Size Analysis of Soils

LS702

AASHTO T88

PROJECT DETAILS

Client:	**GeoTerra, File No. 2606145**	Project No.:	**121626390**
Project:	**Riverain Developments Inc., 3 Selkirk St., Ottawa**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**GeoTerra**
Source:	**BH26-5**	Date Sampled:	**July 23, 2026**
Sample No.:	**SS4**	Tested By:	**Brian Prevost**
Sample Depth	**2.29-2.90 m**	Date Tested:	**August 2, 2026**

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	67.03
Sample Weight after Hydrometer and Wash (g)	32.36
Percent Passing No. 200 Sieve (%)	**51.7**
Percent Passing Corrected (%)	**40.09**

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	**1029.20**
Sample Weight After Sieve (g)	**1028.10**
Percent Loss in Sieve (%)	**0.11**

SOIL INFORMATION

Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.75	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	153.21
Air Dried Mass (W_a), (g)	154.30
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9929
Air Dried Mass in Analysis (M_a), (g)	67.51
Oven Dried Mass in Analysis (M_o), (g)	67.03
Percent Passing 2.0 mm Sieve (P_{10}), (%)	77.51
Sample Represented (W), (g)	86.49

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5	0.0	100.0
19.0	26.2	97.5
13.2	58.1	94.4
9.5	79.7	92.3
4.75	152.4	85.2
2.00	231.5	77.5
Total (C + F)[1]	1028.10	0.1
0.850	3.80	73.11
0.425	8.36	67.84
0.250	14.39	60.87
0.106	26.41	46.97
0.075	31.45	41.14
PAN	31.68	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS

Volume of Bulb (V_B), (cm^3)	63.3
Length of Bulb (L_2), (cm)	14.2
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.3
Scale Dimension (h_s), (cm/Div)	0.17
Cross-Sectional Area of Cylinder (A), (cm^2)	27.25
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:35 AM **Sedimentation Cylinder No:** 11

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
02-Aug-26	10:36 AM	1	25.5	2.0	21.5	23.5	26.58	11.73298	9.73081	0.013047	0.04469
02-Aug-26	10:37 AM	2	20.0	2.0	21.5	18.0	20.36	12.66798	9.73081	0.013047	0.03284
02-Aug-26	10:40 AM	5	17.0	2.0	21.5	15.0	16.97	13.17798	9.73081	0.013047	0.02118
02-Aug-26	10:50 AM	15	14.0	2.0	21.5	12.0	13.57	13.68798	9.73081	0.013047	0.01246
02-Aug-26	11:05 AM	30	12.5	2.0	21.5	10.5	11.88	13.94298	9.73081	0.013047	0.00889
02-Aug-26	11:35 AM	60	11.0	2.0	21.5	9.0	10.18	14.19798	9.73081	0.013047	0.00635
02-Aug-26	2:45 PM	250	9.0	2.0	21.5	7.0	7.92	14.53798	9.73081	0.013047	0.00315
03-Aug-26	10:35 AM	1440	7.0	2.0	21.5	5.0	5.66	14.87798	9.73081	0.013047	0.00133

Remarks: Specific Gravity assumed. As received moisture content, 8.6%.

Reviewed By: Daniel Boateng

Date: August 4, 2026

V:\01216\active\laboratory_standing_offers\2026-Laboratory Standing Offer\121626390 Geo Terra\July 23, Three_Hyd_MC_UCS_Geoterra #260145\Hydrometer-Standing Offers_Traceability.xlsx

Stantec

Particle-Size Analysis of Soils

LS702

AASHTO T88

PROJECT DETAILS

Client:	**GeoTerra, File No. 2606145**	Project No.:	**121626390**
Project:	**Riverain Developments Inc., 3 Selkirk St., Ottawa**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**GeoTerra**
Source:	**BH26-6**	Date Sampled:	**July 23, 2026**
Sample No.:	**SS3**	Tested By:	**Brian Prevost**
Sample Depth	**1.52-2.13 m**	Date Tested:	**August 2, 2026**

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	69.44
Sample Weight after Hydrometer and Wash (g)	35.85
Percent Passing No. 200 Sieve (%)	**48.4**
Percent Passing Corrected (%)	**33.20**

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	**432.60**
Sample Weight After Sieve (g)	**431.70**
Percent Loss in Sieve (%)	**0.21**

SOIL INFORMATION

Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.75	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	150.92
Air Dried Mass (W_a), (g)	152.90
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9871
Air Dried Mass in Analysis (M_a), (g)	70.35
Oven Dried Mass in Analysis (M_o), (g)	69.44
Percent Passing 2.0 mm Sieve (P_{10}), (%)	68.63
Sample Represented (W), (g)	101.18

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5	0.0	100.0
19.0	10.0	97.7
13.2	25.5	94.1
9.5	38.1	91.2
4.75	74.6	82.8
2.00	135.7	68.6
Total (C + F)[1]	431.70	0.2
0.850	7.96	60.76
0.425	15.98	52.84
0.250	23.53	45.38
0.106	32.79	36.22
0.075	35.39	33.65
PAN	35.48	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS

Volume of Bulb (V_B), (cm^3)	63.3
Length of Bulb (L_2), (cm)	14.2
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.3
Scale Dimension (h_s), (cm/Div)	0.17
Cross-Sectional Area of Cylinder (A), (cm^2)	27.25
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:27 AM **Sedimentation Cylinder No:** 10

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
2-Aug-26	10:28 AM	1	32.0	2.0	21.5	30.0	29.01	10.62798	9.73081	0.013047	0.04253
2-Aug-26	10:29 AM	2	28.0	2.0	21.5	26.0	25.14	11.30798	9.73081	0.013047	0.03102
2-Aug-26	10:32 AM	5	25.0	2.0	21.5	23.0	22.24	11.81798	9.73081	0.013047	0.02006
2-Aug-26	10:42 AM	15	23.0	2.0	21.5	21.0	20.31	12.15798	9.73081	0.013047	0.01175
2-Aug-26	10:57 AM	30	21.0	2.0	21.5	19.0	18.37	12.49798	9.73081	0.013047	0.00842
2-Aug-26	11:27 AM	60	19.0	2.0	21.5	17.0	16.44	12.83798	9.73081	0.013047	0.00604
2-Aug-26	2:37 PM	250	15.0	2.0	21.5	13.0	12.57	13.51798	9.73081	0.013047	0.00303
3-Aug-26	10:27 AM	1440	10.0	2.0	21.5	8.0	7.74	14.36798	9.73081	0.013047	0.00130

Remarks: Specific Gravity assumed. As received moisture content, 11.4%.

Reviewed By: Daniel Boateng

Date: August 4, 2026

Stantec

Particle-Size Analysis of Soils
LS702
AASHTO T88

PROJECT DETAILS			
Client:	**GeoTerra, File No. 2606145**	Project No.:	**121626390**
Project:	**Riverain Developments Inc., 3 Selkirk St., Ottawa**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**GeoTerra**
Source:	**BH26-7**	Date Sampled:	**July 23, 2026**
Sample No.:	**SS5**	Tested By:	**Brian Prevost**
Sample Depth	**3.05-3.66 m**	Date Tested:	**August 3, 2026**

WASH TEST DATA	
Oven Dry Mass In Hydrometer Analysis (g)	71.15
Sample Weight after Hydrometer and Wash (g)	44.48
Percent Passing No. 200 Sieve (%)	**37.5**
Percent Passing Corrected (%)	**26.77**

PERCENT LOSS IN SIEVE	
Sample Weight Before Sieve (g)	**477.10**
Sample Weight After Sieve (g)	**476.40**
Percent Loss in Sieve (%)	**0.15**

SOIL INFORMATION		
Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.75	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS	
Oven Dried Mass (W_o), (g)	153.02
Air Dried Mass (W_a), (g)	154.14
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9927
Air Dried Mass in Analysis (M_a), (g)	71.67
Oven Dried Mass in Analysis (M_o), (g)	71.15
Percent Passing 2.0 mm Sieve (P_{10}), (%)	71.41
Sample Represented (W), (g)	99.63

SIEVE ANALYSIS		
Sieve Size mm	**Cum. Wt. Retained**	**Percent Passing**
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5	0.0	100.0
19.0	15.4	96.8
13.2	19.7	95.9
9.5	49.8	89.6
4.75	86.6	81.8
2.00	136.4	71.4
Total (C + F)[1]	476.40	0.1
0.850	8.83	62.55
0.425	18.07	53.27
0.250	27.53	43.78
0.106	40.67	30.59
0.075	43.97	27.28
PAN	44.33	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS	
Volume of Bulb (V_B), (cm^3)	63.3
Length of Bulb (L_2), (cm)	14.2
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.3
Scale Dimension (h_s), (cm/Div)	0.17
Cross-Sectional Area of Cylinder (A), (cm^2)	27.25
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:44 AM **Sedimentation Cylinder No:** 14

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
3-Aug-26	10:45 AM	1	23.0	2.0	21.5	21.0	20.62	12.15798	9.73081	0.013047	0.04549
3-Aug-26	10:46 AM	2	20.5	2.0	21.5	18.5	18.17	12.58298	9.73081	0.013047	0.03273
3-Aug-26	10:49 AM	5	19.0	2.0	21.5	17.0	16.69	12.83798	9.73081	0.013047	0.02091
3-Aug-26	10:59 AM	15	16.0	2.0	21.5	14.0	13.75	13.34798	9.73081	0.013047	0.01231
3-Aug-26	11:14 AM	30	15.0	2.0	21.5	13.0	12.77	13.51798	9.73081	0.013047	0.00876
3-Aug-26	11:44 AM	60	13.0	2.0	21.5	11.0	10.80	13.85798	9.73081	0.013047	0.00627
3-Aug-26	2:54 PM	250	11.0	2.0	21.5	9.0	8.84	14.19798	9.73081	0.013047	0.00311
4-Aug-26	10:44 AM	1440	8.0	2.0	21.5	6.0	5.89	14.70798	9.73081	0.013047	0.00132

Remarks: Specific Gravity assumed. As received moisture content, 4.9%.

Reviewed By: Daniel Boateng
Date: August 4, 2026

Stantec Consulting Ltd.
2781 Lancaster Rd, Suite 100 A&B, Ottawa ON K1B 1A7

August 10, 2026
File: 121625731

Client: GeoTerra File No. 2606145

Reference: ASTM D7012, Method C, Uniaxial Compressive Strength of Intact Rock Core Specimens
3 Selkirk Street, Ottawa, ON

The following table summarizes uniaxial compressive strength results for three intact rock cores.

Source	Depth	Compressive Strength (MPa)	Description of Break
BH26-1, RC1	11.30-11.54m	193.5	Columnar vertical cracking through both ends.
BH26-3, RC1	10.40-10.60m	82.8	Vertical cracking through both ends.
BH26-4, RC1	10.80-10.94m	100.6	Vertical cracking through both ends, small cone formed on one end.

Sincerely,

Stantec Consulting Ltd.

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

Environment Testing
146 Colonnade Rd, Unit 8, Ottawa, ON K2E 7Y1 (613) 727-5692

OFFICIAL CERTIFICATE OF ANALYSIS : 4794449

WORK REQUEST : 100455314

Report Date : 2026-08-11

GEOTerracs Inc
204-1339 Welington Street West
Ottawa, Ontario
K1Y 3B8
Attention : Alpesh Senghani

Reception Date : 2026-08-04
Project : 2606145/Riverain - Phase 2
Sampler : NA
PO Number : Not Applicable
Temperature : 19 °C

Analysis	Quantity	External Method
Chloride (Soil, Water Sol, ISE)	2	Modified from CSA A23.2-4B
Conductivity (Soil, Manual Meter)	2	Modified from MECP E3530
pH (Soil, 1:1, Manual Meter)	2	Modified from WESTERN REGION (S-2.20)
Resistivity (Soil, Calculation)	2	Modified from MECP E3530
Sulphate (Soil, Gravimetric)	2	Modified from 28-3, Methods of Soil Analysis

Sample status upon receipt :

9942984 9942985

Compliant

Notes :

- All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise stated.
- Eurofins Environment Testing Canada Inc. is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at https://directory.cala.ca/
- Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline or regulatory limits listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official guideline or regulation as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Legend :

RL : Reporting limit	N/A : Not applicable	* : Analysis conducted by external subcontracting
QC : Reference material (QC)	1 : Results in annex	^ : Analysis not accredited

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146 Colonnade Rd, Unit 8, Ottawa, ON K2E 7Y1 (613) 727-5692

OFFICIAL CERTIFICATE OF ANALYSIS - RESULTS

Client : GEOTerracs Inc
Project : 2606145/Riverain - Phase 2

Reception Date: 2026-08-04

		Eurofins Sample No :	9942984	9942985					
		Matrix :	Soil	Soil					
		Sampling Date :	2026-07-21	2026-07-22					
		Client Sample Identification :	BH26 - 1 - SS3	BH26 - 4 - SS4					
Anions	**RL**	**Unit**							
Sulphate^	0.01	%	0.06	0.12					

		Eurofins Sample No :	9942984	9942985					
		Matrix :	Soil	Soil					
		Sampling Date :	2026-07-21	2026-07-22					
		Client Sample Identification :	BH26 - 1 - SS3	BH26 - 4 - SS4					
Chloride	**RL**	**Unit**							
Chloride (Water Soluble)	0.002	%	<0.002	0.008					

		Eurofins Sample No :	9942984	9942985					
		Matrix :	Soil	Soil					
		Sampling Date :	2026-07-21	2026-07-22					
		Client Sample Identification :	BH26 - 1 - SS3	BH26 - 4 - SS4					
General Chemistry	**RL**	**Unit**							
Electrical Conductivity	0.05	mS/cm	0.49	0.89					
pH (1:1)	1		8.17	7.78					
Resistivity^	0.05	ohm-cm	2040	1120					

Approved by : ____________________
Emma-Dawn Ferguson, M.Sc.
Environmental Chemist

Environment Testing

146 Colonnade Rd, Unit 8, Ottawa, ON K2E 7Y1 (613) 727-5692

OFFICIAL CERTIFICATE OF ANALYSIS - QUALITY CONTROL

Client : GEOTerracs Inc
Project : 2606145/Riverain - Phase 2

Reception Date: 2026-08-04

Parameter	Unit	RL	Blank	QC		Matrix Spike		Duplicate	
				Recovery %	Range %	Recovery %	Range %	RPD %	Range %
Chloride (Soil, Water Sol, ISE)									
Method : Chloride, water soluble (Soil/Concrete, CSA, ISE). Internal method: AMCLCRE8.									
Chloride (Water Soluble)	%	0.002	<0.002	95	73-125			6	0-50
Associated Samples : 9942984, 9942985								Prep Date: 2026-08-06 Analysis Date: 2026-08-06	
Conductivity (Soil, Manual Meter)									
Method : Conductivity (soil, manual meter). Internal method: OTT-I-SOIL-WI53805.									
Electrical Conductivity	mS/cm	0.05	<0.05	116	76-124			1	0-40
Associated Samples : 9942984, 9942985								Prep Date: 2026-08-06 Analysis Date: 2026-08-06	
pH (Soil, 1:1, Manual Meter)									
Method : pH (Soil, 1:1 Water Extraction, Manual Meter). Internal method: OTT-I-SOIL-WI53805.									
pH (1:1)		1	9.41	102	98-102			0	0-40
Associated Samples : 9942984, 9942985								Prep Date: 2026-08-06 Analysis Date: 2026-08-06	
Sulphate (Soil, Gravimetric)									
Method : Sulphate (Soil, Gravimetric). Internal method: AMSO4SE2.									
Sulphate^	%	0.01	<0.01	104	90-110	97	70-130	18	0-40
Associated Samples : 9942984, 9942985								Prep Date: 2026-08-10 Analysis Date: 2026-08-11	

Where RPD % is reported as "-" the calculation is not available because one or both of the duplicates is within 5 times the RL.

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STANDARD CHAIN-OF-CUSTODY

146 Colonnade Road, Unit #8, Ottawa, ON, K2E 7Y1 - Phone: 613-727-5692, Fax: 613-727-5222

CLIENT INFORMATION

Company: Geoterra
Contact: Alpesh R.Senghani
Address: 300 275-Bank Street
Telephone: 6136194157 | Cell:
Email: #1: arsenghani@geoterracs.com
Email: #2: apahlavan@geoterracs.com
Project: 2606145/Riverain-Phase 2 | Quote #:

INVOICE INFORMATION (SAME AS CLIENT INFORMATION: YES ☐ NO ☐)

Company: Geoterra
Contact: Ahmed Lamrani | Email: #1: alamrani@geoterracs.com
Address: 300-275 Bank Street | Email: #2: accounting@geoterracs.com
Telephone: | PO #:

TURN-AROUND TIME (Business Days)

☐ 1 Day* (100%) ☐ 2 Day** (50%) ☐ 3-5 Days (25%) ☑ 5-7 Days (Standard)

Please contact Lab in advance to determine rush availability. *For results reported after rush due date, surcharges will apply: before 12:00 - 100%, after 12:00 - 50%. **For results reported after rush due date, surcharges will apply: before 12:00 - 50%, after 12:00 - 25%.

TCLP, SPLP, PFAS, and NP/NPE the rush surcharges are 100% (3 day) and 50% (4 day). For farm soils the rush surcharge is 100% (3-5 days). Regular TAT is 10 days.

REGULATION/GUIDELINE REQUIRED

☐ Sanitary Sewer, City:
☐ Storm Sewer, City:
☐ ODWSOG (Use DW COC if samples are for human consumption)
☐ PWQO
☐ O.Reg. 347 (TCLP)
☐ Other:

☐ O. Reg. 153/04
The sample results from this submission will form part of a formal Record of Site Condition (RSC) under O.Reg. 153/04. Analysis of full parameter list only Yes ☐ No ☐
Table # ____, Coarse / Fine, Surface / subsurface
Type: Com-Ind / Res-Park / Agri / GW / All Other / Sediment

☐ O. Reg 406 Excess Soils
Table # ________ Full depth/Strat/Ceiling/mSPLP Leachate
Type: Com-Ind /Res-Park /Agri/All Other
Category: Surface /Subsurface

The optimal temperature conditions during transport is 4 - 10°C. Sample(s) cannot be frozen, unless otherwise indicated or agreed upon with the Laboratory. This COC must not be used for drinking water samples. The COC must be complete upon submission of the samples, there will be a $25 surcharge if required information is missing (required fields are shaded in grey).

Occasionally, situations arise in which Eurofins Environment Testing Canada (Ottawa) is unable to process a sample after receipt. By signing this chain-of-custody form, the client agrees that Eurofins Environment Testing Canada (Ottawa) may subcontract samples to a laboratory that is similarly accredited. This subcontracted laboratory will perform the same analysis using the same or similar methodology. Agreements made in advance to subcontract to a specific laboratory will be honored.

Sample Details — Field Filtered →

Sample ID	Date/Time Collected	Sample Matrix	# of Containers	PHC F1 - F4	BTEX	VOCs	PAHs	PCBs	Metals + Inorganics	Metals only	mSPLP metals	mSPLP VOC	TCLP Metals	TCLP semi-volatiles	TCLP volatiles	Corrosivity				RN# (Lab Use Only)
BH26-1-SS3	2026/07/21	S	1													✔				9942984
BH26-4-SS4	2026/07/22	S	1													✔				985

(O.Reg.153/04 parameters: PHC F1 - F4, BTEX, VOCs, PAHs, PCBs, Metals + Inorganics, Metals only)

	PRINT NAME	LOCATION	SIGN	DATE/TIME	TEMP (°C)
Sampled By:					
Relinquished By:					
Received By:					
Received By:	S.S.ta	OTT	S/	Aug4/26 @ 10:25	19

COMMENTS: No redox or sulfide.

100455314

FO

Printed On : 2026-08-04 12:02:43

CUSTODY SEAL: ☐ YES ☐ NO | Ice packs submit ☐ Yes ☐ No