GEOTECHNICAL INVESTIGATION DRAFT REPORT
FOR
1-21 JOHN STREET, GRIMSBY ONTARIO

Prepared for

1000104674 ONTARIO INC

Prepared by

SIRATI & PARTNERS CONSULTANTS LIMITED

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The reader is referred to the "Limitation and Use of The Report" which follows the text and forms an integral part of the document.

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1. INTRODUCTION

Sirati & Partners Consultants Limited (SIRATI) was retained by 1000104674 Ontario Inc. (the Client) to undertake a geotechnical investigation in support of the proposed new mixed-use residential development at the property located at 1–21 John Street and 46–50 Ontario Street in Grimsby, Ontario (hereafter referred collectively to as the “Site”).

Based on architectural drawings provided by the Client, the proposed development consists of a 16-storey residential tower constructed above a multi-level structured parking garage, which includes an underground parking level (P1) and several above-grade parking levels (Levels 1–4). The development also incorporates ground-floor retail, office space on the lower levels, and indoor and outdoor amenity areas. A total of 313 residential units and 432 parking spaces are included in the proposed design.

This geotechnical investigation has been completed in accordance with the SIRATI work plan detailed in the engineering services proposal dated October 10, 2025. The purpose of the investigation is to assess the subsurface soil and groundwater conditions across the Site through the advancement of six (6) boreholes to depths of approximately 6 m or practical refusal, along with the installation of six (6) monitoring wells for groundwater evaluation. Laboratory testing, including moisture content, grain size analysis, and Atterberg limits, was carried out on select representative soil samples to support engineering interpretation.

Based on the factual data obtained from the field and laboratory programs, this report provides engineering recommendations for the design and construction of the proposed slab-on-grade structures, including guidance on foundation design, bearing capacity, settlement, and pavement design for associated site areas. The report also includes discussions on excavation and backfilling considerations, seismic site classification, and groundwater control requirements during construction, if applicable.

This report has been prepared for the Client and their designers. Third party use of this report without SIRATI consent is prohibited. The limitation conditions presented in **Appendix D** form an integral part of the report and they must be considered in conjunction with this report.

2. SITE DESCRIPTION AND LOCAL GEOLOGY

2.1 SITE DESCRIPTION

The subject Site is located at 1–21 John Street and 46–50 Ontario Street in Town of Grimsby in Ontario. The Site is located on the east side of Elm Street and along the north side of John Street, forming the northeast corner of the Elm Street–John Street intersection.

The property extends northward to the Canadian National Railway (CNR) corridor, which defines its northern boundary. Surrounding land uses consist of single detached residential dwellings to the south and east, and a mixture of commercial and public-use buildings to the west. The Site contains a combination of existing low-rise structures, paved areas, and disturbed or undeveloped ground characterized by bare soil, gravel, and intermittent vegetation, reflecting historical mixed uses and partial clearing.

Based on the architectural drawings provided by the Client (Drawing Set, June 2025), the proposed development consists of a mixed-use high-rise residential building located at 46, 48 and 50 Ontario Street and 1–21 John Street in Grimsby. The planned development includes a 16-storey residential tower constructed above a multi-level parking structure (Levels P1 through Level 4), along with ground-floor retail/commercial space, office space on the lower levels, and indoor and outdoor amenity areas distributed throughout the building. A total of 313 residential units is proposed, supported by approximately 432 parking spaces within the structured parking garage. Site plans indicate associated at-grade features such as a public green-space area, landscaped zones, loading areas, driveway access, and required setbacks from the adjacent Canadian National Railway corridor, including provision for a crash-wall barrier.

2.2 LOCAL GEOLOGY

The physiography of the area where the project is located is the Sand Plain according to Chapman and Putnam's Physiography of Southern Ontario 3rd Edition.

Within the project area, site geology as per *Ontario Geological Survey (OGS) Geology Map No. 2562 on the Quaternary Geology of Ontario, Southern Sheet (1:1,000,000 scale)* indicates the deposits consist of Halton Till (Unit 17), a Pleistocene till characterized as predominantly silt to silty clay matrix, high in carbonate content, and notably clast-poor, reflecting deposition by the Ontario–Erie lobe during late-glacial conditions.

According to the OGS Bedrock Geology of Ontario, the underlying bedrock in the Grimsby area consists of Paleozoic sedimentary rocks of the Clinton Group and the Cataract Group, comprising sandstone, shale, dolostone, and siltstone. These formations occur at substantial depth beneath the surficial deposits.

For design purposes, the frost penetration depth applicable to this region is typically taken as 1.0 m, in accordance with OPSD 3090.101.

3. FIELD AND LABORATORY WORK PROCEDURES

3.1 PERMITS, UTILITY LOCATES, TRAFFIC CONTROL

The borehole locations were predetermined and established in the field by SIRATI personnel. The borehole locations were selected to avoid conflicts with existing above ground and underground utilities, including water, sewer, water, gas, hydro, telephone and cable locations that were verified in the field using Ontario One-call and a private utility locater.

Prior to initiating the subsurface investigation activities SIRATI requested public utilities to be marked by utility operators in accordance with the Ontario One Call damage prevention laws. All applicable utility companies (gas, hydro, bell, network cables, pipeline, and municipal sewers, etc.) were contacted.

3.2 BOREHOLE AND MONITORING WELL INVESTIGATION

Prior to the start of fieldwork, proposed borehole/monitoring well locations were submitted to the Client for review and approval. Upon approval of the plan, SIRATI personnel staked out the borehole locations in the field. The final locations were adjusted, where required, to avoid existing underground utilities and to accommodate site access and surface conditions.

The as-drilled borehole and monitoring well locations, along with ground surface elevations, were surveyed and recorded by SIRATI. These locations, elevations, and corresponding termination depths and elevations are presented on the borehole records included in **Appendix A**.

A total of six (6) boreholes/monitoring wells, designated BH-MW-01 through BH-MW-06 were drilled between February 24 and February 25. The boreholes/monitoring wells were advanced to depths ranging from 6.53 m to 7.97 mbgs, as summarized in Table 4.1 (Refer to **Drawing 01** for the borehole location plan and **Appendix A** for the borehole and well logs).

Each borehole was completed as a monitoring well to facilitate groundwater level measurements and characterization of groundwater conditions at the Site. The wells were constructed using 50 mm diameter Schedule 40 PVC riser and screen sections, surrounded by a sand filter pack and sealed with bentonite and/or grout in accordance with Ontario Regulation 903 (as amended). Well construction details and screened intervals for each installation are provided on the borehole/monitoring well logs in **Appendix A**. Groundwater observations were made at the completion of drilling, and measured groundwater levels (where encountered) are documented on the borehole logs.

Drilling was carried out using solid-stem continuous flight augers by Elements Geo under the full-time direction and supervision of SIRATI personnel. Soil samples were collected at regular intervals using a 50 mm O.D. split-spoon sampler driven by a 63.5 kg automatic hammer falling 760 mm, consistent with the Standard Penetration Test (SPT) procedure (ASTM D1586). The number of hammer blows for the final 300 mm of penetration was recorded as the SPT "N-value." All samples were logged in the field and subsequently reviewed by SIRATI for classification and laboratory testing.

Table 3.1: Summary of SIRATI Boreholes

Borehole Number	Location	Coordinates		Borehole Depth (m)	Ground Surface Elevation (m)	Borehole Termination Elevation
		Northing (m)	Easting (m)			
BH-MW-01	5–21 John Street	4783517.6	617280.2	6.71	83.66	76.95
BH-MW-02		4783535.4	617260.3	6.53	83.45	76.92
BH-MW-03		4783488.2	617277.1	6.71	84.07	77.36
BH-MW-04		4783515.5	617150.2	7.97	86.77	78.80
BH-MW-05		4783486.1	617175.4	6.71	85.15	78.44
BH-MW-06		4783476.2	617227.0	6.71	84.53	77.82

3.3 LABORATORY TESTING PROGRAM

Prior to conducting geotechnical laboratory testing, the soil samples extracted from the boreholes were subjected to visual and tactile examination by an experienced SIRATI geotechnical engineer who confirmed the field descriptions and selected representative samples for detailed testing.

Geotechnical laboratory testing was conducted in accordance with the American Society for Testing and Materials (ASTM) and Canadian Council of Independent Laboratories (CCIL) applicable standards. Laboratory testing consisted of moisture content tests on all recovered soil samples, grain size distribution analyses (sieve and hydrometer testing) on eight (8) select soil samples and Atterberg Limits testing on five (5) selected soil samples to assess soil plasticity index properties.

The results of water content, grain size distribution and Atterberg Limits tests are reported on the boreholes records presented in **Appendix A**. The associated laboratory test results are provided in **Appendix B**.

4. SITE AND SUBSURFACE CONDITIONS

The complete borehole location plan is available in **Drawing 01**. Notes on soil descriptions are presented on **Enclosure 1**. The subsurface conditions of the boreholes forming the basis of this report are documented in the individual borehole logs provided in **Appendix A.**

The subsurface conditions in the boreholes are summarized in the following paragraphs.

4.1 SOIL CONDITIONS

The following presents the soil stratigraphy based on the observations of the boreholes drilled by SIRATI.

4.1.1 TOPSOIL

A surficial layer of topsoil was encountered in BH-MW-02, with thicknesses of 0.13 m. This material consists of dark brown, organic soil and is unsuitable for structural support. It should be completely removed from all construction and fill areas.

It should be noted that the thickness of the topsoil explored at the borehole location may not be representative of the entire site and should not be relied on to calculate the amount of topsoil to be stripped at the site.

4.1.2 Pavement

Borehole BH-MW-04 was advanced through the existing pavement and encountered a 100 mm thick layer of asphalt at the ground surface.

A 0.1 m thick granular fill layer consisting of gravelly sand material was encountered beneath BH-MW-04 and at ground surface in BH-MW-05 and BH-MW-06.

4.1.3 COHESIONLESS FILL

A layer of cohesionless fill material was encountered in all of the boreholes at the ground surface, except in BH-MW-02 and BH-MW-04 where it was encountered below the topsoil/ asphalt. The fill generally consisted of brown gravely sand, sand and silty sand containing variable amounts of sand and gravel.

The fill layer was encountered and extended to depths depth ranging between 1.52 mbgs and 5.33 mbgs. The fill was observed across the alignment within an overall elevation envelope, with the highest elevation of encounter at approximately 86.7 m (BH-MW-04) and the lowest at approximately 83.3 (BH-MW-02); terminations ranged from about 82.86 m (BH-MW-05) down to 81.4 (BH-MW-04).

Laboratory testing indicated that the natural moisture content of the cohesionless fill ranged from 6% to 30%, with an average moisture content of approximately 14.1%. The measured SPT ‘N’ values in the layer ranged from 8 blows per 300 mm of penetration to 50 blows per 102 mm of penetration, indicating loose to very dense state of compactness.

4.1.4 NON - COHESIVE TILL

A non - cohesive till deposit was encountered beneath below the fill materials in BH-MW-01 and BH-MW-02 only. The till deposit consists of sandy silt till with varying amount of gravel and clay. The till deposit was encountered and extended to a depth of 2.3 mbgs (Approximate Elevations 81.2 m).

The natural moisture content measured by a laboratory test ranged from 20% to 24% with an average of 22%. The measured SPT ‘N’ values in the layer ranged from 14 to 33 blows per 300 mm of penetration, indicating compact to dense state of compactness.

One (1) laboratory particle size distribution analysis was conducted on selected sample obtained from the non - cohesive till deposit. The results are provided in Table 5.1.

Table 4.1: Non - Cohesive Till Particle Size Distribution Analysis Results

Borehole ID	Sample ID	Sample Depth	Particle Size Distribution (ASTM D422 and ASTM D421)				Soil Classification
			Gravel %	Sand %	Silt %	Clay %	
BH-MW-01	SS03	1.5	8	39	48	5	Sandy silt till, trace gravel, trace clay

4.1.5 NON - COHESIVE NATIVE DEPOSITS

Non – cohesive native deposit was encountered in BH-MW-04 and BH-MW-05. This deposit consisted of sandy silt and sand containing variable amounts of gravel and clay, with occasional trace cobbles in selected location.

The thickness of this stratum extended to depths ranging from 2.3 m to 6.1 mbgs. The non – cohesive deposit was observed across the alignment within an overall elevation envelope, with the highest elevation of encounter at approximately 82.9 m (BH-MW-05) and the lowest at approximately 80.1 m (BH-MW-05); terminations ranged from about 82.1 m (BH-MW-05) down to 78.8 m (BH-MW-04).

Laboratory testing indicated natural moisture contents ranging from 7% to 13%, with an average of 10%. Standard Penetration Test (SPT) 'N' values within this deposit varied from 22 blows per 300 mm to greater than 50 blows per 56 mm, reflecting a compact to very dense state of compactness.

One (1) laboratory particle size distribution analysis was conducted on selected samples obtained from the Non – cohesive native deposit. The results are provided in Table 5.2.

Table 4.2: Non - Cohesive Native Particle Size Distribution Analysis Results

Borehole ID	Sample ID	Sample Depth	Particle Size Distribution (ASTM D422 and ASTM D421)				Soil Classification
			Gravel %	Sand %	Silt %	Clay %	
BH-MW-04	SS09	6.1	5	42	46	7	Sandy silt
BH-MW-05	SS04	3.05	2	64	29	5	Sandy silt

Atterberg limit test for a select sample of the subgrade materials is presented in **Appendix B**. A summary of the Atterberg limit test is also presented in the table below:

Table 4.3: Non - Cohesive Atterberg Limit Results

Borehole ID	Sample ID	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
BH-MW-04	SS09	Non - Plastic		

4.1.6 COHESIVE TILL DEPOSITS

Native cohesive till was encountered in all of the boreholes. The till deposit generally consisted of sandy lean clay till containing variable amounts of sand and gravel, with occasional trace cobbles in selected location.

The thickness of this stratum extended to depths ranging from 2.3 m to 6.2 mbgs. The cohesive till was observed across the alignment within an overall elevation envelope, with the highest elevation of encounter at approximately 82.2 m (BH-MW-06) and the lowest at approximately 78.9 m (BH-MW-05); terminations ranged from about 81.5 m (BH-MW-06) down to 76.9 m (BH-MW-02).

Laboratory testing indicated natural moisture contents ranging from 23% to 59%, with an average of 46%. Standard Penetration Test (SPT) 'N' values within this deposit varied from 22 blows per 300 mm to greater than 50 blows per 132 mm, reflecting a very stiff to hard consistency.

Six (6) laboratory particle size distribution analyses were conducted on selected samples obtained from the cohesive till deposit. The results are provided in Table 5.3.

Table 4.4: Cohesive Till Particle Size Distribution Analysis Results

Borehole ID	**Sample ID**	**Sample Depth**	**Particle Size Distribution (ASTM D422 and ASTM D421)**				**Soil Classification**
			Gravel %	**Sand %**	**Silt %**	**Clay %**	
BH-MW-01	SS04	2.29	3	38	43	16	Sandy lean clay till
BH-MW-02	SS05	3.05	3	23	51	23	Lean clay with sand till
BH-MW-03	SS07	6.10	2	36	41	21	Sandy lean clay till
BH-MW-04	SS08	5.33	2	31	47	20	Sandy lean clay till
BH-MW-06	SS06	4.57	2	34	49	15	Sandy lean clay till

Atterberg limit test for a select sample of the subgrade materials is presented in **Appendix B**. A summary of the Atterberg limit test is also presented in the table below:

Table 4.5: Cohesive Till Atterberg Limit Results

Borehole ID	**Sample ID**	**Liquid Limit (LL)**	**Plastic Limit (PL)**	**Plasticity Index (PI)**
BH-MW-01	SS04	31	17	14
BH-MW-03	SS07	29	15	14
BH-MW-04	SS08	30	19	11
BH-MW-06	SS06	24	15	9

4.2 GROUNDWATER CONDITIONS

Monitoring wells were installed in BH-MW-01 through BH-MW-06 immediately following borehole completion to facilitate ongoing groundwater level monitoring across the Site

Short-term (unstabilized) groundwater conditions were assessed during drilling and immediately upon completion of each borehole. Groundwater was encountered in BH-MW-01, BH-MW-02, BH-MW-03, BH-MW-05, and BH-MW-06 at depths ranging from 1.50 m to 2.90 mbgs, while no groundwater was encountered during the drilling of BH-MW-04. These depths correspond to groundwater elevations ranging from approximately 81.14 m to 82.43 m (mAMSL), based on the ground surface elevations reported on the borehole logs.

. Groundwater (GW) levels were subsequently measured on March 9, 2026, with readings indicating groundwater depths ranging from 1.84 m to 4.61 m mbgs, corresponding to elevations of approximately 81.06 m to 83.24 m.

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events.

Table 4.6: Summary of Groundwater Level Measurements

Borehole / Well ID	Borehole Depth (m)	Ground Elevation (mAMSL)	Well Depth (m)	Groundwater Depth (mBGS) / Elevation (m)			
				Upon Completion of Borehole	09-Mar-26	23-Mar-26	06-Apr-26
BH-MW-01	6.71	83.66	6.71	1.82/ 81.84	1.98/ 81.68	TBC	TBC
BH-MW-02	6.53	83.45	6.53	1.52/ 81.93	2.39/ 81.06	TBC	TBC
BH-MW-03	6.71	84.07	6.71	2.7/ 81.37	2.02/ 82.05	TBC	TBC
BH-MW-04	7.97	86.77	7.97	Dry	4.61/ 82.16	TBC	TBC
BH-MW-05	6.71	85.15	6.71	2.9/ 82.25	1.91/ 83.24	TBC	TBC
BH-MW-06	6.71	84.53	6.71	2.1/ 82.43	1.8/ 82.73	TBC	TBC

4.3 FROST SUSCEPTIBILITY OF SUBGRADE SOILS

The frost susceptibility of the subgrade soils is assessed using the Ministry of Transportation of Ontario’s guidelines, which are based on the presence of silt sized particles in the 75µm to 5µm range as outlined in Table 5.7. Based on the classification of the soils and the lab testing, it is assumed that the existing subgrade material is of Low to Moderate Susceptibility to Frost Heaving (LSFH to MSFH).

Table 4.7: MTO Frost Susceptibility Guidelines

Grain Size (75 – 5 µm)	Susceptibility to Frost Heaving
0 – 40%	Low (LSFH)
40 – 55%	Moderate (MSFH)
55 – 100%	High (HSFH)

Based on the above criteria the Site soils are assigned the frost susceptibility ratings provided in the following Table 5.8.

Table 4.8: Site Soils Frost Susceptibility

Soil Type	% Silt (75-5µm)	Location	Frost Susceptibility (75-5µm)
Silty Sand to Sandy lean clay till	19 – 42	BH-MW-01 - BH-MW-06	LSFH to MSFH
	LSFH–- Low Susceptibility to Frost Heaving MSFH – Moderate Susceptibility to Frost Heaving HSFH–- High Susceptibility to Frost Heaving		

As noted above in Table 5.8, most of the subgrade soils encountered on the project site have low to moderate susceptible (LSFH to MSFH) to frost. Based on the limited laboratory testing and the soil classification, the upper portion of the subgrade is assumed to be low to moderate susceptible to frost heaving (LSFH to MSFH).

5. ENGINEERING DISCUSSION AND RECOMMENDATIONS

Based on the architectural drawings provided by the Client, the proposed development consists of a 16-storey mixed-use residential building constructed above a multi-level parking structure, including one underground parking level (P1) and multiple above-grade parking levels (Levels 1–4). The proposed new development will replace the existing low-rise residential and commercial structures currently occupying the Site. Detailed structural information such as final finished floor elevations, column and wall loads, and foundation configurations are yet to be finalised and were not available at the time of preparing this report.

The following sections provide preliminary geotechnical comments and recommendations to support the proposed development concept. These recommendations are based on the subsurface conditions encountered during the investigation and are intended to assist in the design-development stage. A supplementary geotechnical investigation may be required once final structural loads, foundation layouts, and below-grade configurations are confirmed.

The recommendations provided herein are intended for use by the design team. Contractors bidding on or undertaking work at the Site should review the factual subsurface information, evaluate the adequacy of the data for their construction means and methods, and form their own interpretation of subsurface conditions as they relate to excavation, shoring, equipment selection, and sequencing. Any comments related to construction in this report are provided only to highlight factors that may influence design and should not be interpreted as instructions to the contractor.

Ongoing communication with SIRATI during detailed design and construction is recommended to ensure that the geotechnical recommendations are correctly interpreted and appropriately implemented as the project progresses.

The following sections provide comments and recommendations for the proposed development. These recommendations are preliminary and based on the geotechnical engineering findings at the time of this report.

5.1 SITE PREPARATION, GRADING, AND REUSE OF EXISTING SOILS

Based on the Plan of Survey (R.A. McLaren, O.L.S., July 17, 2024), existing ground elevations across the Site are generally flat to gently sloping from north toward the south/southeast, with typical grades ranging from 83.0 to 86.8 m referenced to Town of Grimsby BM 14012 (83.619 m). These grades are consistent with the as-drilled ground surface elevations recorded on the borehole/monitoring-well logs (ranging from 83.45 to 86.77 m), indicating minor local variations but no significant grade differentials across the development area.

Subsurface conditions encountered during the investigation indicate that the Site is underlain by asphalt and/or granular surface layers, and locally thin topsoil (e.g., BH-MW-02), overlying fill materials consisting of silty sand, sand with gravel, and mixed granular–silt fill to depths typically ranging between 0.1 m and 2.3 mbgs. Beneath the fill, native glacial till deposits were encountered, comprising silty sand till and sandy lean clay till, extending to the termination depths of the boreholes at approximately 6.5 m to 8.0 m.

The existing fill within the proposed building area is to be fully removed and the underlying native subgrade be inspected and proof rolled prior to any engineered fill placement. At the time of the fieldwork, organics were not encountered in the fill material, but if during the construction, organics are found within the fill or native soils then these soils should be removed from the footprint of the proposed building and pavement areas prior to site grading activities and should not be used as backfill. The suitability of the existing fill for reuse as engineered fill should be reviewed during excavation and any unsuitable fill (organic rich soil, construction debris and other deleterious materials) should be removed.

This site is underlain by a deposit of typically stiff to hard silty clayey tills and dense to very dense sandy tills that is prone to long-term consolidation settlement when overstressed by external loads that are close to or exceed the difference between the existing subsurface stresses and the apparent pre-consolidation pressures.

External loads include loads imposed by building foundations (such as footings and floor loads), fill placement to achieve site grade raises and temporary or, especially, long-term lowering of groundwater levels. Managing ground settlement behavior in the context of planned grading for building floor slab or other areas of Site will be critical elements of appropriate site use designs. Any grade raise in proximity of the structures and underground utilities should not exceed 0.5 m, subject to inspection by qualified geotechnical personnel to confirm that the exposed soils are competent. It should be noted that a grade raise over large areas will increase stresses in the deeper soft to firm silty clay layers that will cause significant long-term settlements. A detailed settlement analysis is required to assess the impact of site grade raise higher than 0.5 m beneath planned structures or on the adjacent structures.

Prior to Site grading activity and earth filling, the exposed subgrade soils should be visually inspected, compacted, and proof rolled using large axially loaded equipment. Any soft, organic, or unacceptable material should be removed as directed by qualified geotechnical personnel and replaced with suitable fill materials compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD). Clean earth fill used to raise grades in the proposed building and pavement areas should be placed in thin layers (200 mm thick or less) and compacted by a heavy Sheepsfoot roller (clayey fill material) or smooth drum roller (granular fill) to 98 percent SPMDD.

Based on the characteristics encountered within the boreholes, the existing fill materials on site may not be suitable for reuse as structural backfill or fill under settlement sensitive areas, and will require further verification prior to construction. However, they can be used as general fill in landscaped areas. The native sandy lean clay till encountered at the Site are generally suitable for reuse as fill to raise site grades or as trench backfill during installation of buried services, provided they are free of organic material, and are within ±2% of the optimum moisture content. It should be anticipated that reworking of the soils may be necessary to facilitate compaction through slight wetting or drying as required, with the use of heavy sheepsfoot roller compactors. If free-draining and non-frost susceptible materials are required as backfill against foundation walls and retaining walls, the use of fine-grained soils such as the native sandy lean clay till may not be appropriate and imported granular fills will be required. Suitable frost tapers will have to incorporated into the backfill geometry if imported granular soils are used to backfill excavations.

If imported materials are required to raise site grades to design levels, then potential source sites should be evaluated for geotechnical and environmental quality prior to acceptance. It is recommended that any proposed fill considered for reuse on the Site comprise of clean earth material, free of topsoil and building rubble, and is at a moisture content ±2% of the laboratory optimum moisture content for compaction.

5.2 FOUNDATION SYSTEM

Based on information provided by the Client and the architectural drawings, the proposed development consists of a 16-storey residential tower with a multi level parking structure that includes one underground parking level, P1. The architectural section indicates that Level 1 is taken as the project reference and that the P1 formation lies about 4.5 metres below Level 1. Final foundation elevations, column loads, wall loads, and finished floor elevations were not available at the time of preparing this report. The recommendations below are therefore based on the existing grades from the survey and on subsurface information from boreholes BH MW 01 through BH MW 06.

The survey shows existing ground ranging from approximately 83.0 m to 86.8 m. The borehole logs show ground surface elevations between 83.45 m and 86.77 m. The excavation required to reach the P1 level will therefore vary across the site but will typically be on the order of 4 m to 5 mbgs. At the time of preparing this report details of the proposed finished floor elevations and footing / slab loading conditions were not available. To estimate the bearing capacities, it was assumed that the founding elevation will be at an approximate depth of 4.5 mbgs (80.5m AMSL).

Analysis of the geotechnical information indicates that fill materials were encountered to depths ranging from approximately 0.1 m to 2.3 mbgs, which corresponds to elevations of about 82.9 m to 81.4 m AMSL across the borehole locations. The fill materials consist of mixed silty sand, sand with gravel, granular silt, and construction debris in some locations, and are highly variable in both thickness and composition. These existing fills are not considered suitable to support the proposed building foundation system and must be removed from foundation and slab support zones to allow founding on competent native till.

SPT ‘N’ values within the native non-cohesive and native cohesive deposits ranged from 20 blows per 300 mm of penetration to values exceeding 50 blows at shorter penetration lengths, indicating compact to very dense conditions in the non-cohesive native deposits and very stiff to hard consistency in the cohesive native deposits.

Bedrock was not encountered in the drilled boreholes. The groundwater elevation was reported between approximately 1.34 to 7.69 mbgs in the monitoring wells as observed from October to December 2021.

For the subject Site, potentially the most settlement risk under building loads is expected to arise from long term consolidation of fine-grained native cohesive deposits where increases in effective stress occur. Settlement can be driven by structural loads, site grade raises, and groundwater drawdown during excavation and dewatering. Given the measured fines contents and moisture ranges in the native cohesive deposits and the observed groundwater depths of about 1.5 to 2.9 mbgs at several boreholes, long-term consolidation should be evaluated when final loads and finished grades are known. Where practical, grade raises beneath and adjacent to foundations should be limited to about 0.5 m, or alternatively supported by analysis to confirm acceptable total and differential settlements.

Based on the geotechnical investigation findings and considering the above-mentioned subsurface conditions, the following comments should be considered for the selection of foundation system for the proposed building. It should be noted that the structural loading is an important factor for selection of foundation system by the structural engineer in conjunction with the following geotechnical recommendations.

It should be noted that due to limited number of boreholes and the relatively small number of samples scheduled for lab testing, there are uncertainties about the variability of soil conditions across the site. Further investigation including more borings as well as more laboratory testing can potentially minimize the uncertainties and result in a less conservative design.

5.2.1 SHALLOW FOUNDATION SYSTEM

5.2.1.1 MAT FOUNDATION

For the proposed 16-storey building, the governing factors in foundation selection include the imposed structural loads, settlement criteria, particularly differential settlement, and the bearing capacity requirements for design, among other considerations. In view of the subsurface soil conditions encountered and the anticipated long-term consolidation settlement, the use of shallow strip footings or conventional spread footings is not recommended due to the potential for excessive differential settlement. Accordingly, a mat foundation system is recommended, as described below.

A raft foundation will act as the floor slab of the lowest parking level and is assumed to be founded at the P1 formation, which is about 4.5 m below Level 1 and corresponds to an assumed founding elevation of approximately 80.5 m AMSL, noting that existing ground varies across the Site.

Because the mat foundation extends over the entire footprint of the building, the total area of the foundation increases which will significantly lower the stress applied to the supporting soil. Because of the structural continuity of mat foundations, this type of shallow foundation reduces the risk of differential settlement. Moreover, where the bottom of the structure is located below the groundwater table, waterproofing is an important concern and because a mat foundation system is monolithic, it is much easier to waterproof. The weight of the mat also helps resist hydrostatic uplift forces from the groundwater. Groundwater was observed during drilling at depths on the order of 1.5 to 2.9 mbgs at several boreholes, so uplift and waterproofing design are required considerations at the P1 level.

The SLS values of 65 kPa and 90 kPa can be considered to limit the total settlement to 50 mm and 75 mm, respectively. It is noted that the final building plan dimensions are not known at this time; for calculations, a 15 m × 45 m raft foundation was assumed. Differential settlements in mat or raft foundations are largely controlled by the structural or flexural rigidity of the structure and mat; expected differential settlement can vary from about 50% of the maximum settlement for a flexible long base mat to negligible for a rigid mat.

A factored ULS soil resistance of 170 kPa with $\Phi = 0.5$ can be used for design purposes. This value is based on the assumption that the effective width of the raft is at least 15 m and that the mat bears on competent native non-cohesive and cohesive deposits at the P1 formation after removal of all fill and subgrade approval. In addition, the raft foundation should be designed to resist hydrostatic uplift pressure based on the recorded groundwater levels and the anticipated seasonal fluctuation.

It is noted that fine-grained native cohesive deposits are present in the boreholes and that a significant portion of total settlement can be long-term consolidation settlement where increases in effective stress occur. For the calculation of the above-mentioned soil bearing resistances, the effect of overburden removal due to excavation to the P1 foundation level has been included.

The value of modulus of subgrade reaction is not an intrinsic soil parameter; it depends on the interaction between the soil and the structure transferring the load to the soil, including foundation geometry, stiffness, and load distribution. The modulus of subgrade reaction is usually considered as a function of the area of soil loaded by the structure.

A modulus of subgrade reaction of 10 MPa/m for one foot plate size (kv,0.3) may be used for the design of the raft foundation supported on the undisturbed very stiff to hard native cohesive deposits. The subgrade modulus required for structural analysis can be estimated from the above-mentioned modulus of subgrade reaction for a one-foot square plate using a series of correction factors as recommended by numerous references including but not limited to CFEM (2006). An alternative method is to estimate the value of vertical modulus of subgrade reaction based on the calculated settlements of the mat foundation under uniform loading.

The proposed value should be considered in conjunction with the following comments:

- The values of modulus of subgrade reaction in the center of the mat are less than the proposed average value and along the perimeter are greater. It is recommended that the perimeter values of subgrade reaction be considered twice the central values in a way that the integral of all the values over the area of the mat is the same as the proposed value.

- It is recommended that the structural designer conduct a parametric study on the value of subgrade modulus to evaluate its effect on the mat design. ACI (1993) suggests varying the subgrade modulus from one-half the computed value to five or ten times the original value and basing the structural design on the worst case condition.
- Where individual concentrated loads are applied on the slab, complementary analysis should be carried out to estimate the range of influence of the load and to update the soil modulus of subgrade reaction.

These values and assumptions are preliminary and will be confirmed during detailed design once final loads, raft geometry, and P1 subgrade conditions are established.

5.2.2 DEEP FOUNDATIONS

5.2.2.1 HELICAL PILES

The proposed high-rise development may be supported on deep foundations such as helical piles, micropiles, or caissons where required. Helical piles consist of a steel shaft with one or more helical plates. The pile is rotated into the ground and advanced until it reaches competent native deposits, and the load capacity is confirmed through the installation torque achieved during advancement. Because helical piles displace soil rather than removing it, very little spoil is generated, reducing the need for off-site disposal. Helical anchors are generally designed as end-bearing elements, and any friction contribution from the overlying fill should be conservatively ignored. The helical plates may take advantage of the compact to very dense native non-cohesive deposits and very stiff to hard cohesive native deposits identified below the fill. The potential presence of cobbles or gravel lenses may create installation resistance in some locations.

The helical pile plates should be embedded at least 3 m into competent native non-cohesive or cohesive deposits where SPT N-values exceed 25 or where torque refusal is reached earlier. Based on the subsurface conditions and typical performance of an RS2875-type shaft, a helical pile extending to a depth of 6 mbgs, or earlier refusal, is expected to support an approximate Serviceability Limit States (SLS) load of about 115 kN and a factored Ultimate Limit State (ULS) geotechnical resistance of about 155 kN, subject to confirmation by field testing. These values are considered preliminary and shall be refined during detailed design.

A full-scale load test must be completed on at least one sacrificial test pile to verify load–deflection behaviour and confirm the design load capacity prior to the installation of production piles. Full-time geotechnical monitoring is required during installation to document torque, depth, installation resistance, and final embedment conditions. A qualified design-build pile contractor should be retained to design, supply, and install the helical pile system.

5.2.2.2 CAISSONS

Caissons may be used to provide greater bearing capacity for the proposed development. Drilled caissons would be advanced through the unsuitable near surface materials and founded on verified competent native deposits at depth. The borehole logs show variable fill underlain by native non cohesive and cohesive deposits to the explored depths of about 6.5 to 8.0 metres. Final founding depth will be established by down hole inspection and proof at the time of drilling. Where uniform bearing or settlement performance requires it, the caisson may be advanced deeper until a consistent competent horizon is confirmed.

The caissons can be designed for a Serviceability Limit State (SLS) bearing pressure of 4 MPa and an Ultimate Limit State (ULS) factored resistance of 6 MPa on competent native deposits, subject to verification of bearing conditions at each shaft. A minimum length-to-diameter ratio of 3.0 is recommended for caissons unless a project-specific analysis supports a lower ratio.

If caisson foundations are used, a specialist piling contractor should provide temporary smooth-surface liners or casings to seal off any wet pockets within the fill or wet seams within fine-grained native soils and to allow safe cleaning and inspection of the base. Where cobbles or very dense granular lenses are present, coring or pre-drilling may be required to advance the casing and establish a sound base.

Foundations designed to the specified SLS capacity values are expected to settle less than about 25 mm total and about 19 mm differential, provided founding is on verified competent native deposits and construction follows the inspection requirements below.

Inspection requirement. Prior to concrete placement, the base of each caisson shall be inspected and approved by the Geotechnical Engineer to confirm that founding materials and cleaning

5.2.3 SLAB ON GRADE

Prior to the placement of the floor slab or any fill materials used to raise grades, the subgrade should be inspected by geotechnical staff for soft or loose areas. Areas found to be soft or containing unsuitable materials should be sub-excavated and replaced with compacted fill as described herein.

The floor slab should be placed on a 200 mm thick layer of well-graded granular base material consisting of 19 mm clear stone or crusher run limestone (or equivalent). For the structural design of the concrete slab-on-grade, a combined modulus of subgrade / granular base reaction coefficient (k) of 10 MPa/m can be used.

Due to the anticipated relatively shallow groundwater table at this site, a subfloor drainage system and waterproofing membrane will generally be required beneath the slab. The purpose of the subfloor drainage system is primarily to prevent a build-up of hydrostatic pressure below the floor slab so that the slab does not need to be designed to resist hydrostatic load. The drainage system must be designed to collect and dispose of groundwater at a rate sufficient to prevent build-up of hydrostatic pressure. The purpose of placing a waterproofing membrane below the slab is to minimize potential for seepage of groundwater or uncontrolled moisture migration through the slab and keep the basement dry.

As an alternative to a permanent subfloor drainage system, the basement can be supported on raft (mat) foundation (structural slab) and designed as a watertight tank. This will eliminate the need to install and maintain the subfloor drains but is otherwise likely to be more costly. In this case, the basement slabs will have to be designed to resist hydrostatic and uplift pressures.

Where a slab on grade is used adjacent to foundation walls, backfill should consist of suitable granular materials placed within about 2% of optimum moisture content, in lifts not exceeding 150 mm, and compacted to not less than 98% of Standard Proctor Maximum Dry Density. Field inspection by geotechnical personnel is recommended during placement and compaction.

5.3 LATERAL EARTH PRESSURE

Structures subject to unbalanced earth pressures such as, shoring systems, retaining walls and other similar structures must be designed to resist lateral earth and hydrostatic pressures. Based on the subsurface conditions encountered, the soil parameters provided in the table may be used to calculate lateral earth pressures:

Table 5.1 Lateral Earth Pressure Recommendations

Soil Type	Bulk Unit Weight	Effective Angle of Internal Friction (º)	Coefficient of Lateral Earth Pressure		
	γ (kN/m3)	ϕ	Active, K_a	At-rest, K_o	Passive, K_p
Fill – gravely sand, sand and silty sand	19	28	0.36	0.53	2.76
Native Cohesionless Deposits	20	30	0.33	0.5	3.00
Native cohesive deposits	21	29	0.35	0.52	2.9

Where walls are restrained against lateral movements, the coefficient of lateral earth pressure at rest Ko should be used. It is to be noted that large deformation will be required prior to the full mobilization of passive earth pressure. Therefore, a factor of safety is required to be applied to passive earth pressure if used for design purposes.

The provided lateral earth pressure values are based on assuming a horizontal backfill condition, vertical back-face of the retaining structure and smooth soil-wall interface only. If the design includes a sloping ground surface in front of the retaining structure, a backfill inclination, or an inclined back-face of the retaining wall, the earth-pressure coefficients will require modification. For the design of any temporary shoring systems, Chapter 26 of the Canadian Foundation Engineering Manual (CFEM 2006) provides guidelines for shoring designers regarding the distribution of the forces.

For the design of shoring system, the groundwater level should be assumed at the ground surface in the design of shoring systems and other design purposes.

5.4 SEISMIC SITE CLASSIFICATION

Based on the borehole data obtained during this geotechnical investigation and in accordance with the Ontario Building Code (OBC) provisions for seismic site response, the project site is classified as Site Class D (Stiff Soil) for preliminary design. This classification is based on the subsurface conditions observed in BH-MW-01 through BH-MW-06, including soil type and stiffness, absence of shallow bedrock within the explored depths of approximately 6.5 m to 8.0 m, and the recorded Standard Penetration Test N values that are generally in the range of about 20 to 40 with localized higher values at depth.

5.5 DEPTH OF FROST PENETRATION

The design depth of frost penetration in the area is 1.0 m as per the OPSD 3090.101. A permanent soil cover of 1.0 m or an appropriate thermal synthetic insulation system is required for frost protection of foundations (foundations in heated or unheated areas). During winter construction, all exposed subgrade surfaces intended to support foundations must be protected against freezing. Protection may be provided using loose straw, tarpaulins, insulated blankets, or other suitable methods to prevent frost penetration into the bearing soils.

5.6 SITE SERVICING

The native soils encountered in the boreholes are suitable for the support of proposed site services. The bedding and sand cover materials for the pipes should be adequately compacted to provide support and protection. Provided the base area of the sewer pipes and watermains are free of all loose and deleterious materials, the pipe bedding should comply with a Class B bedding configuration as per the requirements of OPSD 802.030 (rigid pipe) and/or OPSD 802.010 (flexible pipe).

Where the trench base has been disturbed by excavation, soft native soils, or groundwater seepage, the affected materials should be sub-excavated and replaced with properly compacted granular fill. If the native soils at pipe invert level are wet, clear stone may be used as bedding, wrapped with a suitable filter fabric to prevent migration of fines.

Where fill material is found at the pipe invert level, the fill material should be visually inspected during installation. Any wet, soft, highly organic or otherwise unsuitable fills should be sub-excavated and replaced with bedding materials or clean fills compacted to minimum of 95% SPMDD.

The presence of compressible native cohesive deposits together with site grade raises may result in long-term settlement that can affect the performance of buried services. The design of underground services should account for potential long-term settlements associated with final site grades, especially near transitions between fill and native soils. Where settlement sensitivity is high, construction sequencing or soil improvement beneath service corridors may be necessary.

5.7 EXCAVATION AND DEWATERING

The Occupational Health and Safety Act (OHSA) regulations require that if workmen must enter an excavation deeper than 1.2 m, the excavation must be suitably sloped and/or braced in accordance with the OHSA requirements. OHSA specifies maximum slope of the excavations for four broad soil types as summarized in the following table:

Table 5.2: OHSA requirements based on Soil types

Soil Type	Base of Slope	Maximum Slope Inclination
1	Within 1.2 metre of bottom	1 horizontal to 1 vertical
2	Within 1.2 metre of bottom of trench	1 horizontal to 1 vertical
3	From bottom of excavation	1 horizontal to 1 vertical
4	From bottom of excavation	3 horizontal to 1 vertical

OHSA Section 226 defines the four soil types as follows:

Table 5.3: Summary of OHSA Soil Types

Soil Type	Key Characteristics
Type 1	a) Hard and very dense, penetrated only with great difficulty by a small sharp object b) Low natural moisture content and high internal strength c) No signs of water seepage d) Typically excavated only with mechanical equipment
Type 2	a) very stiff, dense and can be penetrated with moderate difficulty by a small sharp object; b) has a low to medium natural moisture content and a medium degree of internal strength; and c) has a damp appearance after it is excavated.
Type 3	a) stiff to firm and compact to loose in consistency or is previously excavated soil; b) exhibits signs of surface cracking; c) exhibits signs of water seepage; d) if it is dry, may run easily into a well-defined conical pile; and e) has a low degree of internal strength.
Type 4	a) soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength; b) runs easily or flows, unless it is completely supported before excavating procedures; c) has almost no internal strength; d) wet or muddy; and e) exerts substantial fluid pressure on its supporting system. Ontario Regulation 213/91, s. 226 (5)

The OHSA defines soil types based on consistency, moisture condition, sensitivity, and stability. The native cohesive soils at this Site, dominated by sandy lean clay till, may generally be classified as Type 3 above the groundwater level and Type 4 below the groundwater level due to reduced strength and the potential for softening when saturated. Where the recommended side slopes cannot be achieved because of space constraints or to protect adjacent structures and property lines, the excavation sides must be supported by an engineered shoring system. Shoring must be designed in accordance with the CFEM and the OHSA Regulations for Construction Projects.

Groundwater levels observed in monitoring wells during drilling ranged from about 1.5 to 2.9 metres below existing grade, depending on location. Based on these observations, excavations for foundations or services may extend below the groundwater level, requiring temporary shoring and groundwater control.

Since the native cohesive soils at the site is relatively impervious and the seepage is expected to be small, it may be possible to control the groundwater using sump and pump methods. However, depending on the time of construction and during wet weather conditions, the fill materials can store perched groundwater that might be released into open excavations extending through these materials. The dewatering system must be designed and installed by a contractor specializing in dewatering during construction. The contractor should be responsible for selection, performance and detailed design of the dewatering system. The dewatering system should be designed to conform to the requirements of OPSS.PROV 517 (Dewatering for Excavations) and OPSS.PROV 518 (Control of Water from Dewatering Operations). It is recommended that prior to construction, trial test pits be dug along the proposed construction alignment in order to evaluate the expected groundwater inflow and to determine best means to achieve adequate dewatering.

Surface water should be directed away from open excavations and spoil piles should be kept a minimum of 1.0 m away from the top of any excavation to prevent excess loading on excavation sidewalls.

5.8 PAVEMENT DESIGN RECOMMENDATIONS

5.8.1 PAVEMENT STRUCTURE

The following table summarizes the minimum pavement structures recommended for the design of the potential driveways, parking areas, and access roads. The pavement designs include a Heavy Duty for access roads / driveways and a Light Duty for passenger car parking areas.

No traffic loading information was available at the time of preparing this report. As such, the recommended pavement structures have been developed based on assumed loading conditions consistent with the anticipated site use.

Table 5.4: Pavement Design Recommendation

Pavement Layer	Compaction Requirements	Light Duty Pavement Design	Heavy Duty Pavement Design
Surface Course Asphaltic Concrete HL3 (OPSS 1150)	91% to 96.5% Maximum Relative Density (OPSS 310)	40 mm	40 mm
Base Course Asphaltic Concrete HL8 (OPSS 1150)	92% to 97.5% Maximum Relative Density (OPSS 310)	50 mm	80 mm
Base Course: Granular ‘A’ or 19mm Crusher Run (OPSS1010	100% Standard Proctor Maximum Dry Density	150 mm	150 mm
Sub-base Course: Granular B or 50mm Crusher Run (OPSS1010)	98% Standard Proctor Maximum Dry Density	350 mm	450 mm

If pavement construction occurs in wet inclement weather, it may be necessary to provide additional subgrade support for construction traffic by increasing the thickness of the granular sub-base.

5.8.2 DRAINAGE

To maintain the integrity of the pavement at the Site, subdrains should be installed at all catch basins (3 m stubs in the upgradient direction) and along the perimeter of the parking lot. The invert of the subdrains should be at least 300 mm below the bottom of the subbase and should be sloped to drain to adjacent catch basins. The subdrains should be installed in a 300 mm by 300 mm trench lined by suitable geotextile and consist of a 150 mm diameter perforated pipe wrapped in a suitable geotextile and surrounded with a minimum thickness of 50 mm of free draining sand such as clear stone or concrete sand.

Grading adjacent to pavement areas should be designed so that water is not allowed to pond adjacent to the outside edges of the pavement. Also, the pavement subgrade should be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward the edge of pavement and toward catch basins.

6. GENERAL COMMENTS ON REPORT

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

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

The limitation conditions presented in **Appendix D** form an integral part of the report and they must be considered in conjunction with this report.

Drawings/Enclosures

Enclosure 1A: Notes on Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by Sirati & Partners Consultants Limited also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

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

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

Appendix A: GEOTECHNICAL BOREHOLE LOGS

SIRATI & PARTNERS
Geotechnical Hydrogeological & Environmental Solutions

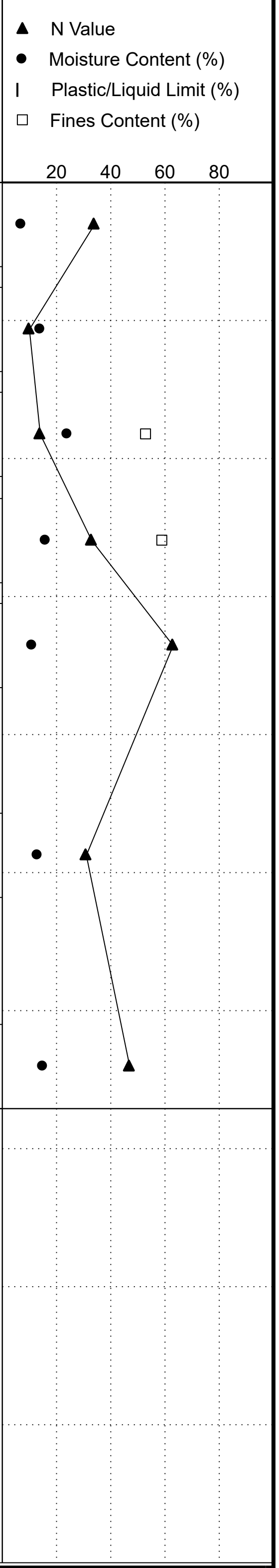
WELL NUMBER BH-MW-01

Sheet 1 of 1

CLIENT
PROJECT NUMBER SP25-01487-00
DATE STARTED 02-24-2026 **COMPLETED** 02-24-2026
DRILLING CONTRACTOR Elements Geo
DRILLING METHOD Solid stem auger
EQUIPMENT Diedrich D50 / Automatic SPT Hammer (63.5 Kg)
HOLE SIZE 150 mm
LOGGED BY JS **CHECKED BY** JS

PROJECT NAME Ground Investigation
PROJECT LOCATION 1-21 John Street, 46-48 & 50 Ontario Street, Grimsby
POSITION N: 4783517.6 m E: 617280.2 m (Bing / Google (WGS84))
GROUND ELEVATION 83.66 m UTM **FINAL DEPTH** 6.71 m
GROUNDWATER LEVELS:
▽ **AT TIME OF DRILLING**
▼ **AT END OF DRILLING** 1.80 m
▼ **AFTER DRILLING**

DEPTH (m)	ELEVATION (m)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
		SW-SM		Fill, Poorly graded sand with gravel and silt, brown, moist, dense -	SPT-SS1	100	25-10-24-26 (34)
0.76	82.90	SP-SM		Fill, sand silt, trace gravel, brown, very moist, compact	SPT-SS2	100	3-5-5-5 (10)
1.83 ▼	81.83	SM		Sandy silt till, trace gravel, trace clay, reddish brown, wet, compact	SPT-SS3	100	2-2-12-9 (14)
2.29	81.37	CL		Sandy lean clay till, trace gravel, brown, wet, hard	SPT-SS4	100	7-12-21-25 (33)
				Trace cobbles, grey	SPT-SS5	100	16-23-40-40 (63)
		CL			SPT-SS6	100	7-13-18-24 (31)
					SPT-SS7	100	11-20-27-35 (47)
6.71	76.95			Terminated at 6.71 m.			



NOTES

WELL NUMBER BH-MW-02

Sheet 1 of 1

CLIENT

PROJECT NUMBER SP25-01487-00

DATE STARTED 02-24-2026 **COMPLETED** 02-24-2026

DRILLING CONTRACTOR Elements Geo

DRILLING METHOD Solid stem auger

EQUIPMENT Diedrich D50 / Automatic SPT Hammer (63.5 Kg)

HOLE SIZE 150 mm

LOGGED BY JS **CHECKED BY** JS

PROJECT NAME Ground Investigation

PROJECT LOCATION 1-21 John Street, 46-48 & 50 Ontario Street, Grimsby

POSITION N: 4783535.4 m E: 617260.3 m (Bing / Google (WGS84))

GROUND ELEVATION 83.45 m UTM **FINAL DEPTH** 6.53 m

GROUNDWATER LEVELS:

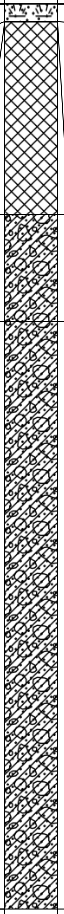
▽ **AT TIME OF DRILLING**

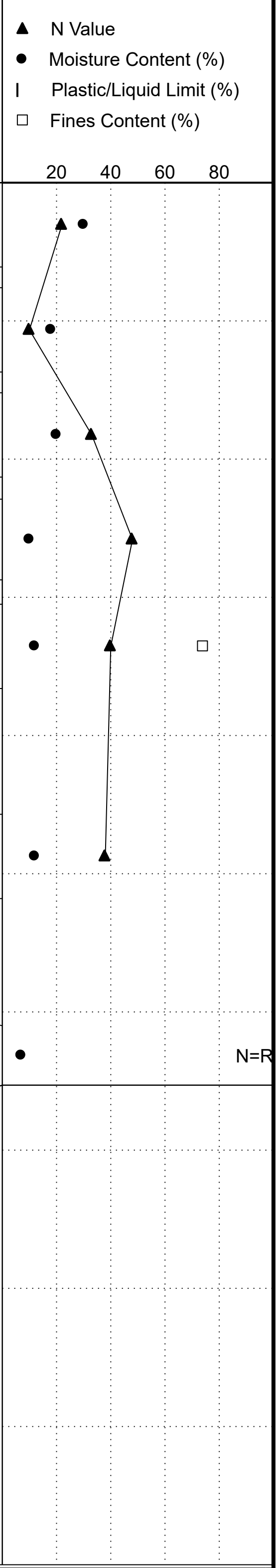
▼ **AT END OF DRILLING** 1.50 m

▼ **AFTER DRILLING**

DEPTH (m)	ELEVATION (m)	USCS	MATERIAL DESCRIPTION
0.13	83.32	OL	Topsoil, 130 mm thick
1.52	81.93	SM SM	Fill, sandy silt, trace to few clay, trace gravel, trace residual materials, trace rootlets, dark brown, moist, compact Reddish brown
2.29	81.16	SM	Sand silt till, trace to a few gravel, trace cobbles, trace clay, reddish brown, wet, dense
6.53	76.92	CL CL	Lean clay with sand till, trace cobbles, trace gravel, reddish brown, moist, hard Grey
			Terminated at 6.53 m.

SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
SPT-SS1	100	15-12-10-10 (22)
SPT-SS2	100	8-6-4-4 (10)
SPT-SS3	100	5-18-15-11 (33)
SPT-SS4	100	21-19-29-50 /134 mm (48)
SPT-SS5	100	9-16-24-26 (40)
SPT-SS6	100	7-16-22-34 (38)
SPT-SS7	100	13-36-50/132 mm (R)





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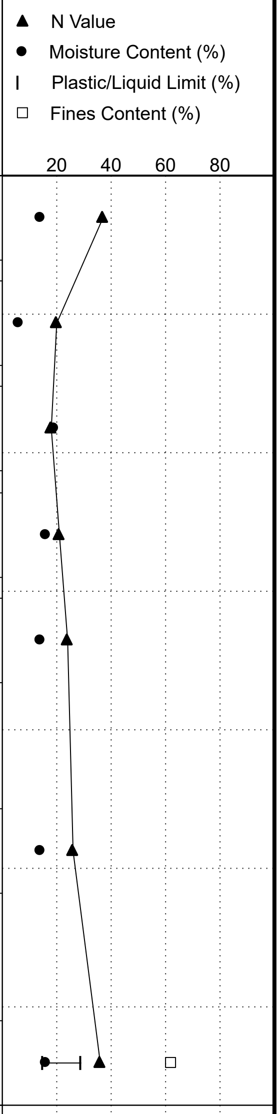
WELL NUMBER BH-MW-03

Sheet 1 of 1

CLIENT
PROJECT NUMBER SP25-01487-00
DATE STARTED 02-24-2026 **COMPLETED** 02-24-2026
DRILLING CONTRACTOR Elements Geo
DRILLING METHOD Solid stem auger
EQUIPMENT Diedrich D50 / Automatic SPT Hammer (63.5 Kg)
HOLE SIZE 150 mm
LOGGED BY JS **CHECKED BY** JS

PROJECT NAME Ground Investigation
PROJECT LOCATION 1-21 John Street, 46-48 & 50 Ontario Street, Grimsby
POSITION N: 4783488.2 m E: 617277.1 m (Bing / Google (WGS84))
GROUND ELEVATION 84.07 m UTM **FINAL DEPTH** 6.71 m
GROUNDWATER LEVELS:
▽ **AT TIME OF DRILLING**
▼ **AT END OF DRILLING** 2.70 m
▼ **AFTER DRILLING**

DEPTH (m)	ELEVATION (m)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
0.30	83.77	SP-SM		Fill, Poorly graded sand with gravel and silt, dark brown, moist, dense	SPT-SS1	100	31-20-17-17 (37)
		SM		Fill, sandy silt, trace gravel, brown, moist, dense			
1.52	82.55	SM		Reddish brown, trace cobbles, compact	SPT-SS2	100	6-11-9-7 (20)
2.29	81.78	SM		Fill, silty sand, trace to a few gravel, trace cobbles, trace clay, trace residual materials, reddish brown, very moist, compact	SPT-SS3	100	15-12-6-17 (18)
		CL		▼ Sandy lean clay till, trace cobbles, trace gravel, brown, wet, very stiff	SPT-SS4	100	3-9-12-16 (21)
		CL		Grey	SPT-SS5 SPT-SS6	100 100	4-10-14-19 (24) 6-11-15-22 (26)
6.71	77.36	CL		Hard	SPT-SS7	100	9-15-21-26 (36)
				Terminated at 6.71 m.			



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Geotechnical Hydrogeological & Environmental Solutions

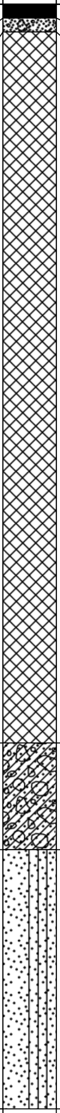
WELL NUMBER BH-MW-04

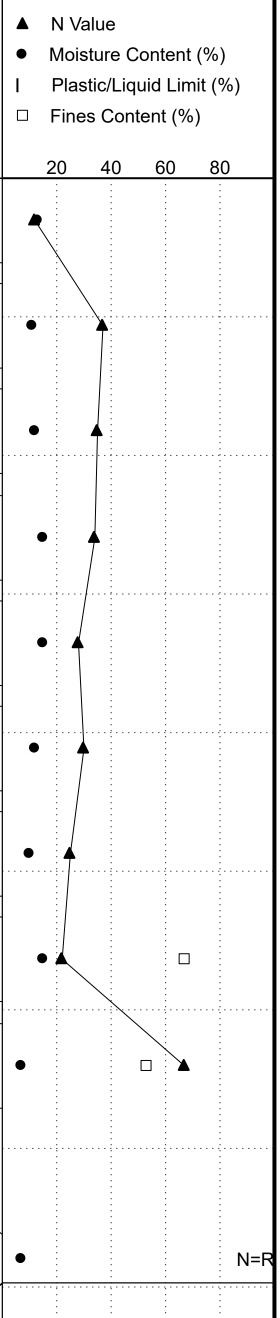
Sheet 1 of 1

CLIENT
PROJECT NUMBER SP25-01487-00
DATE STARTED 02-25-2026 **COMPLETED** 02-25-2026
DRILLING CONTRACTOR Elements Geo
DRILLING METHOD Solid stem auger
EQUIPMENT Diedrich D50 / Automatic SPT Hammer (63.5 Kg)
HOLE SIZE 150 mm
LOGGED BY HE **CHECKED BY** JS

PROJECT NAME Ground Investigation
PROJECT LOCATION 1-21 John Street, 46-48 & 50 Ontario Street, Grimsby
POSITION N: 4783515.5 m E: 617150.2 m (Bing / Google (WGS84))
GROUND ELEVATION 86.77 m UTM **FINAL DEPTH** 7.97 m
GROUNDWATER LEVELS:
▽ **AT TIME OF DRILLING**
▼ **AT END OF DRILLING** Not Encountered
▼ **AFTER DRILLING**

DEPTH (m)	ELEVATION (m)	USCS	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
0.10	86.67	-	Asphalt, 100 mm thick	SPT-SS1	100	4-6-6-9 (12)
0.20	86.57	-	Granular fill, sand and gravel, 100 mm thick			
		SM	Fill, silty sand, trace to a few clay, oxidized, brown, moist, compact			
			Trace cobbles, trace gravel, trace brick pieces, brown, moist, dense	SPT-SS2	100	9-17-20-20 (37)
				SPT-SS3	100	9-15-20-22 (35)
		SM		SPT-SS4	100	8-15-19-21 (34)
		SM	Compact	SPT-SS5	100	6-13-15-19 (28)
		SM	Dark brown	SPT-SS6	100	7-14-16-17 (30)
				SPT-SS7	100	5-14-11-15 (25)
5.33	81.44	CL	Sandy lean clay till, trace gravel, brown, moist, very stiff	SPT-SS8	100	5-10-12-13 (22)
6.10	80.67	SM	Sandy silt, trace cobbles, trace gravel, trace clay, reddish brown, moist, very dense	SPT-SS9	100	6-26-41-54 (67)
				SPT-SS10	100	29-42-50/56 mm (R)
7.97	78.80		Terminated at 7.97 m.			





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SIRATI & PARTNERS
Geotechnical Hydrogeological & Environmental Solutions

WELL NUMBER BH-MW-05

Sheet 1 of 1

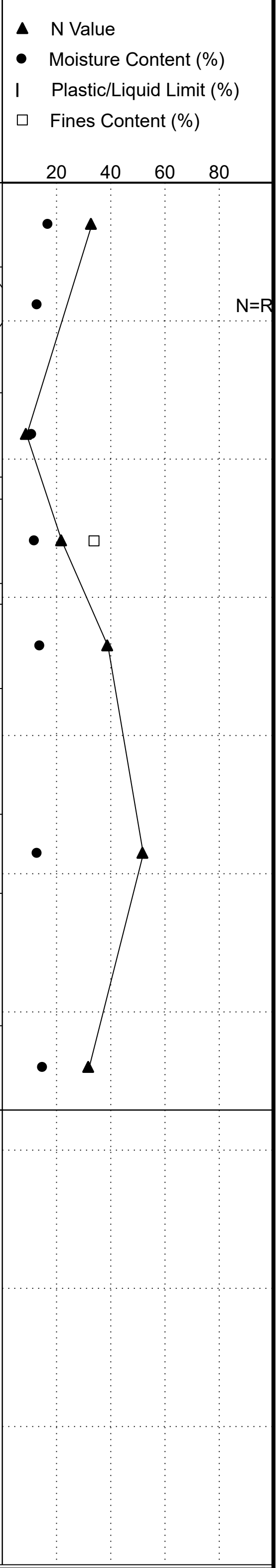
CLIENT
PROJECT NUMBER SP25-01487-00
DATE STARTED 02-25-2026 **COMPLETED** 02-25-2026
DRILLING CONTRACTOR Elements Geo
DRILLING METHOD Solid stem auger
EQUIPMENT Diedrich D50 / Automatic SPT Hammer (63.5 Kg)
HOLE SIZE 150 mm
LOGGED BY HE **CHECKED BY** JS

PROJECT NAME Ground Investigation
PROJECT LOCATION 1-21 John Street, 46-48 & 50 Ontario Street, Grimsby
POSITION N: 4783486.1 m E: 617175.4 m (Bing / Google (WGS84))
GROUND ELEVATION 85.15 m UTM **FINAL DEPTH** 6.71 m
GROUNDWATER LEVELS:
▽ **AT TIME OF DRILLING**
▼ **AT END OF DRILLING** 2.90 m
▼ **AFTER DRILLING**

DEPTH (m)	ELEVATION (m)	USCS	MATERIAL DESCRIPTION
0.15	85.00	-	Granular fill, sand and gravel, 150 mm thick
		SM	Fill, sand, trace to a few silt, trace clay, trace wood fragments, trace rootlets, trace brick pieces, dark brown, moist, dense
		SP	Fill, silty sand, trace to a few clay, trace cobbles, trace gravel, trace wood fragments, trace rootlets, reddish brown, moist, very dense
2.29	82.86	SM	Trace brick pieces, oxidized, loose
3.05	82.10	SM	Silty sand, trace to a few clay, trace gravel, reddish brown, moist, compact
5.03	80.12	CL	Lean clay with sand till, trace gravel, reddish brown, moist, hard
6.25	78.90	SP	Fine sand, trace to few silt, trace to few clay, grey, moist to very moist, very dense to compact at 6.1 m
6.71	78.44	CL	Lean clay with sand till, trace gravel, brown, moist, hard

Terminated at 6.71 m.

SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
SPT-SS1	100	19-25-8-3 (33)
SPT-SS2	100	4-50/104 mm (R)
SPT-SS3	100	4-3-6-8 (9)
SPT-SS4	100	2-8-14-20 (22)
SPT-SS5	100	8-17-22-25 (39)
SPT-SS6	100	5-13-39-50/ 122 mm (52)
SPT-SS7	100	10-15-17-23 (32)



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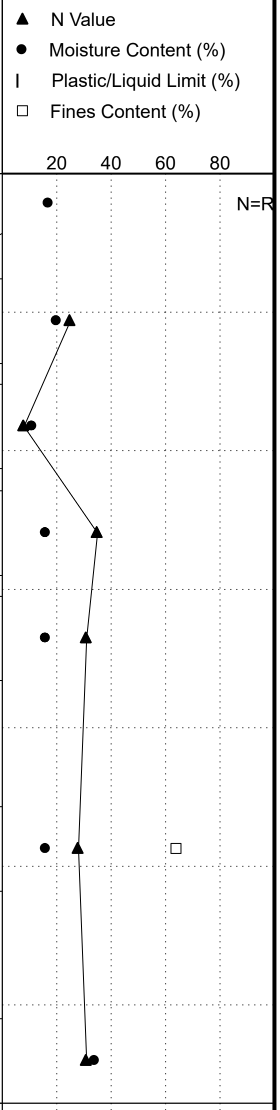
WELL NUMBER BH-MW-06

Sheet 1 of 1

CLIENT
PROJECT NUMBER SP25-01487-00
DATE STARTED 02-25-2026 **COMPLETED** 02-25-2026
DRILLING CONTRACTOR Elements Geo
DRILLING METHOD Solid stem auger
EQUIPMENT Diedrich D50 / Automatic SPT Hammer (63.5 Kg)
HOLE SIZE 150 mm
LOGGED BY HE **CHECKED BY** JS

PROJECT NAME Ground Investigation
PROJECT LOCATION 1-21 John Street, 46-48 & 50 Ontario Street, Grimsby
POSITION N: 4783476.2 m E: 617227.0 m (Bing / Google (WGS84))
GROUND ELEVATION 84.53 m UTM **FINAL DEPTH** 6.71 m
GROUNDWATER LEVELS:
▽ **AT TIME OF DRILLING**
▼ **AT END OF DRILLING** 2.10 m
▼ **AFTER DRILLING**

DEPTH (m)	ELEVATION (m)	USCS	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
0.15	84.38	-	Granular fill, sand and gravel, 150 mm thick	SPT-SS1	100	11-25-50/13 2 mm (R)
			Fill, poorly graded sand with gravel, trace cobbles, trace silt, trace brick pieces, dark brown, moist, dense			
		SM	Fill, silty sand, trace to a few gravel, trace to a few clay, trace cobbles, trace rootlets, oxidized, brown, moist, compact	SPT-SS2	100	5-10-15-11 (25)
		-				
		SM	Reddish brown, moist to very moist	SPT-SS3	100	4-5-3-2 (8)
2.29	82.24	CL-ML	Silty clay with sand, trace cobbles, trace gravel, brown, moist (wet spoon), hard	SPT-SS4	100	6-15-20-24 (35)
3.05	81.48	CL	Sandy lean clay till, trace gravel, reddish brown, moist, hard	SPT-SS5	100	10-16-15-16 (31)
		CL	Very stiff	SPT-SS6	100	6-11-17-22 (28)
		CL	Hard	SPT-SS7	100	6-14-17-24 (31)
6.71	77.82		Terminated at 6.71 m.			



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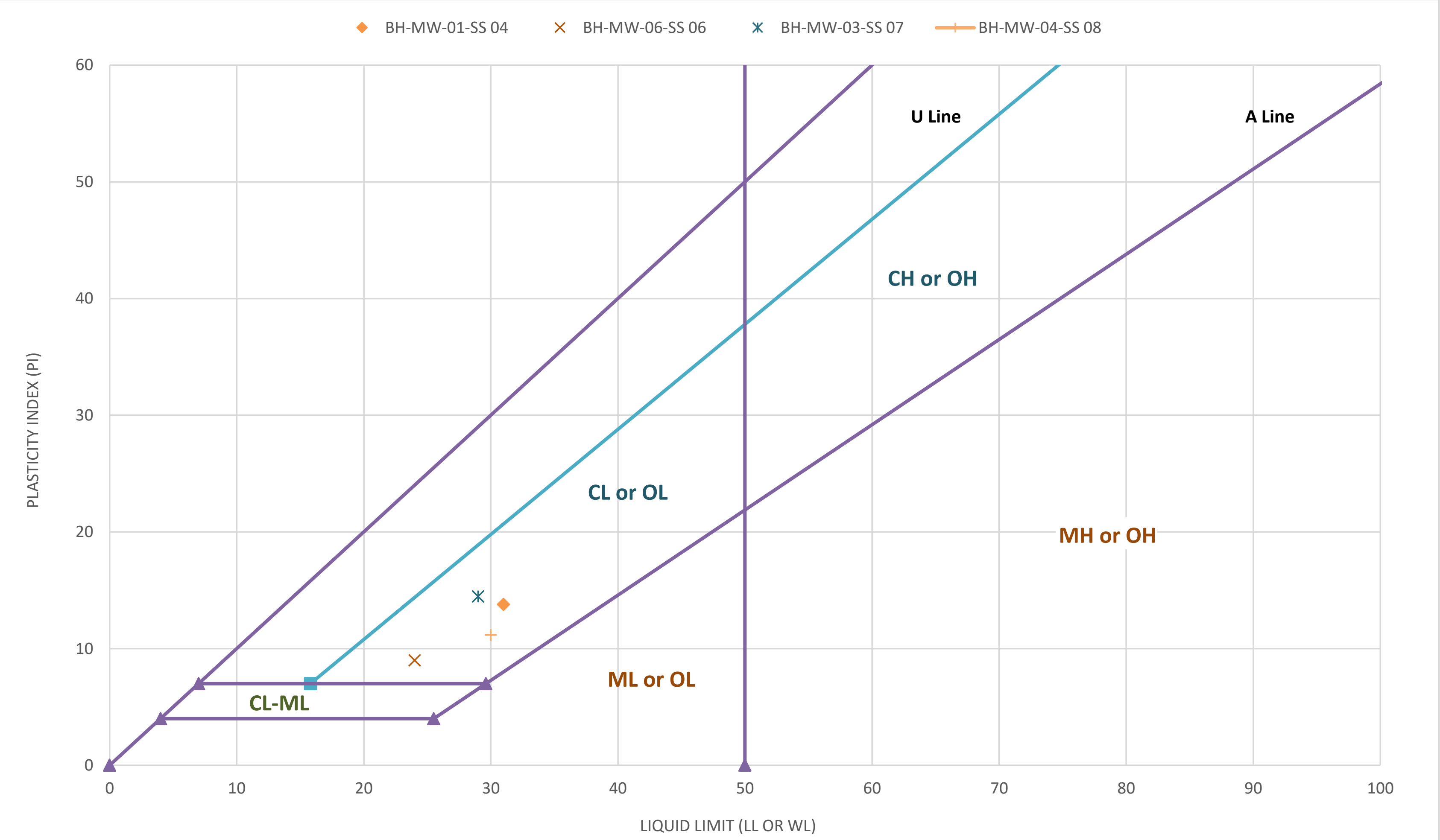
Appendix B: GEOTECHNICAL LABORATORY TESTING

Atterberg's Limits Test Report

ASTM D4318-10



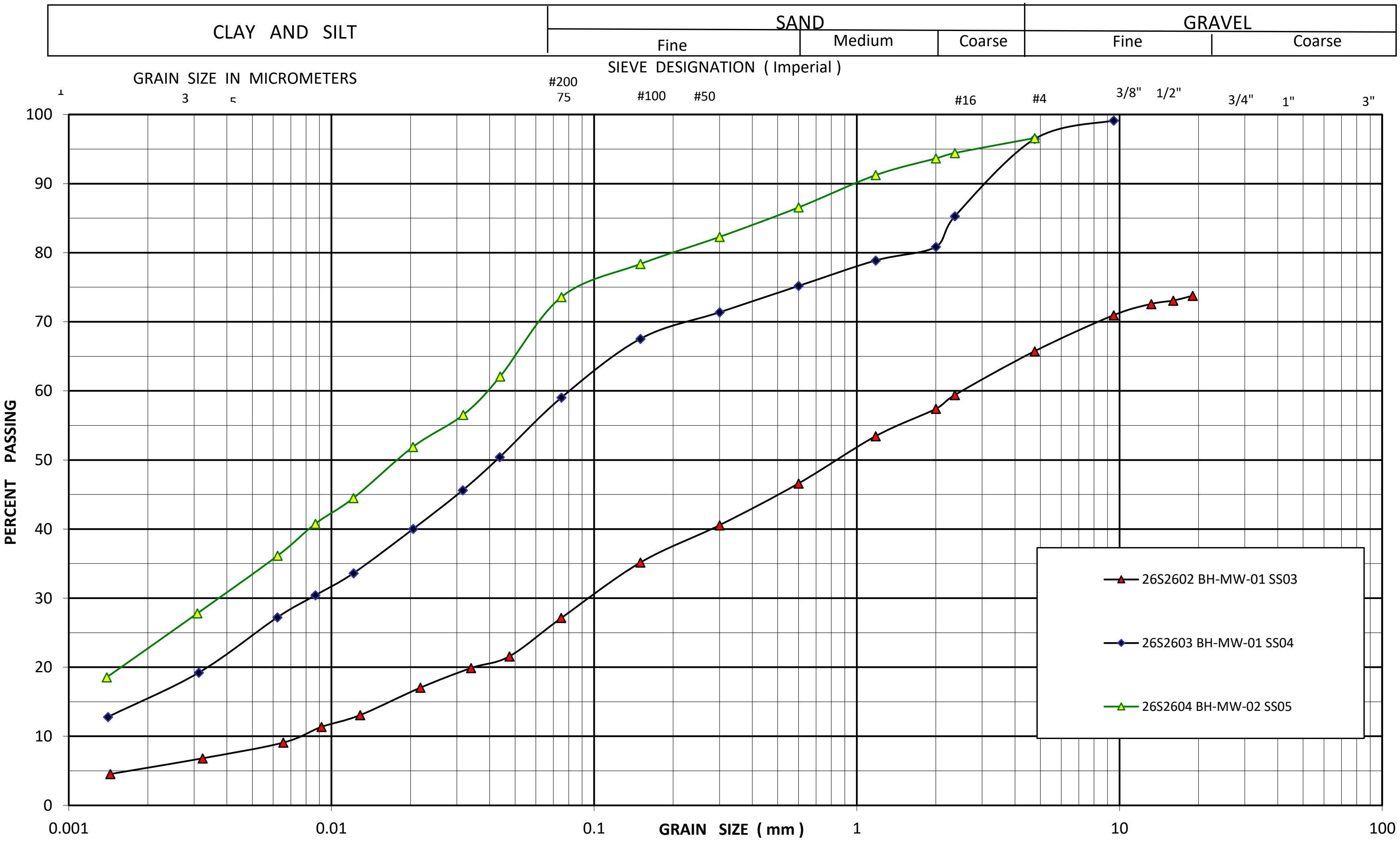
SIRATI & PARTNERS

Geotechnical Hydrogeological & Environmental Solutions

Date	:	09-03-2026
Project No.	:	SP25-01487-00
Figure No.	:	1

GRAIN SIZE DISTRIBUTION

UNIFIED SOIL CLASSIFICATION SYSTEM

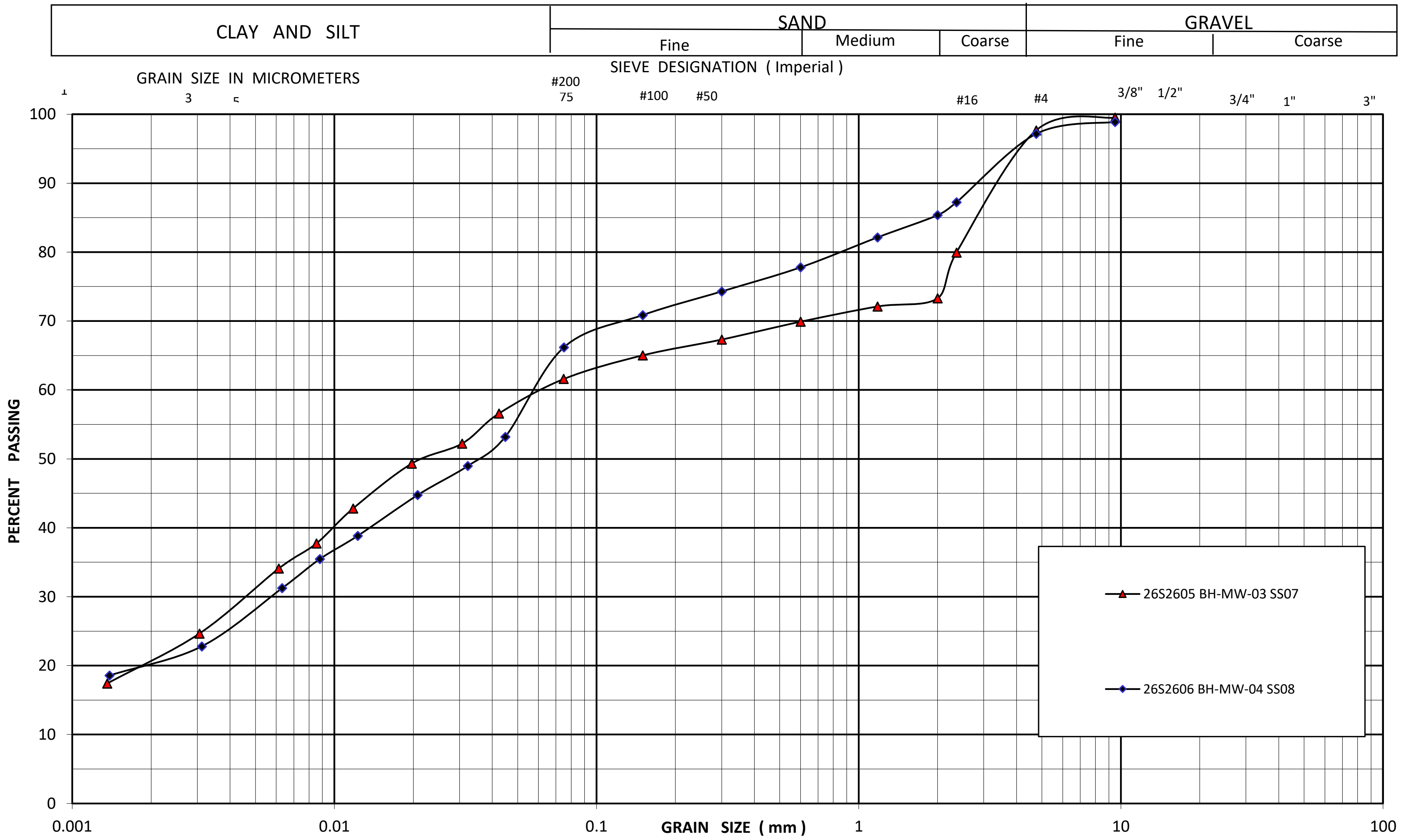


Project No.	:	SP25-1487-00
Date	:	02 March 2026
Figure No.	:	1

SIRATI & PARTNERS

GRAIN SIZE DISTRIBUTION

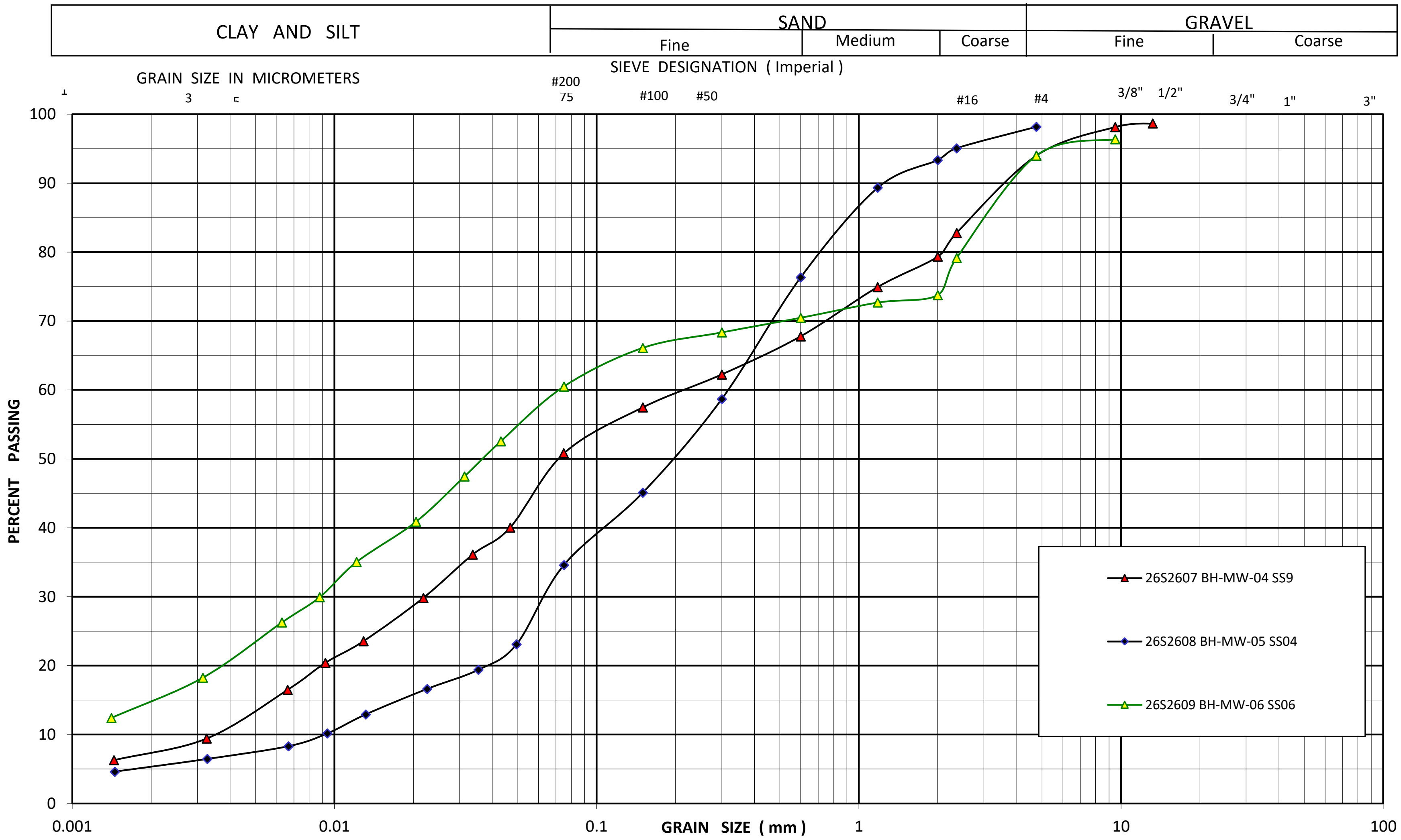
UNIFIED SOIL CLASSIFICATION SYSTEM



Project No.	:	SP25-1487-00
Date	:	02 March 2026
Figure No.	:	2

GRAIN SIZE DISTRIBUTION

UNIFIED SOIL CLASSIFICATION SYSTEM



Project No.	:	SP25-1487-00
Date	:	02 March 2026
Figure No.	:	3

Appendix C: Guidelines on Engineered Fill

GENERAL REQUIREMENTS FOR ENGINEERED FILL

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

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

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

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

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

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

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

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

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

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

6. Full-time geotechnical inspection by SIRATI during placement of engineered fill is required. Workcannot commence or continue without the presence of the SIRATI representative.

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

8. A bearing capacity of 150 kPa at SLS (225 kPa at ULS) can be used provided that all conditions outlined above are adhered to. A minimum footing width of 500 mm (20 inches) is suggested and footings must be provided with nominal steel reinforcement.

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

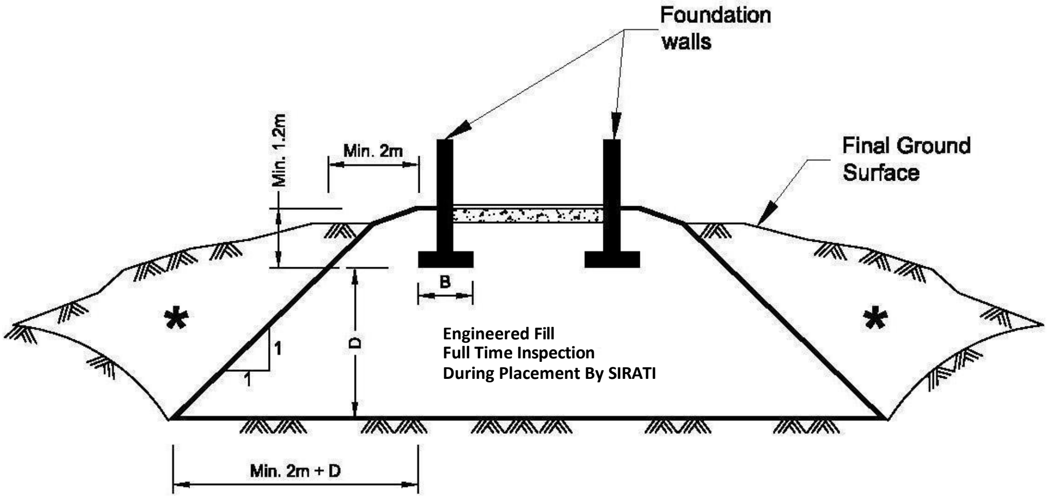
10. After completion of the engineered fill pad a second contractor may be selected to install footings. The prepared footing bases must be evaluated by engineering staff from SIRATI prior to footing concrete placements. All excavations must be backfilled under full time supervision by SIRATI to the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

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

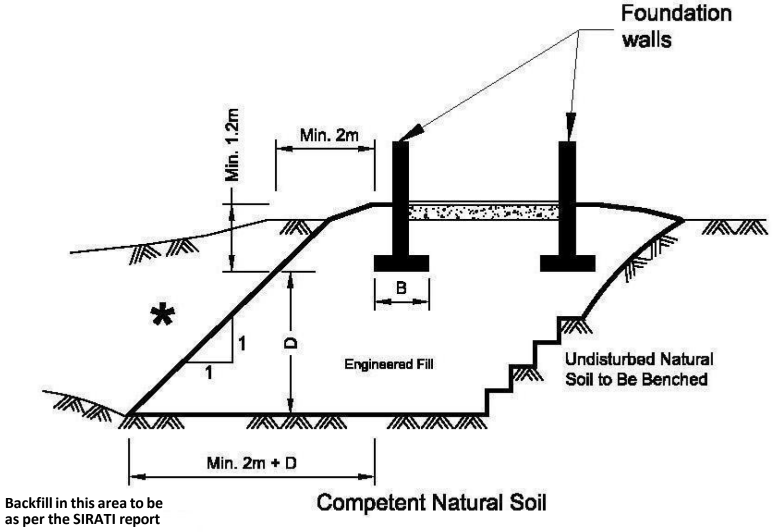
12. If there is a delay in construction, the engineered fill pad must be inspected and accepted by the geotechnical engineer. The location of the structure must be reconfirmed that it remains within the pad.

13. The geometry of the engineered fill as illustrated in these General Requirements is general in nature. Each project will have its own unique requirements. For example, if perimeter sidewalks are to be constructed around the building, then the projection of the engineered fill beyond the foundation wall may need to be greater.

14. These guidelines are to be read in conjunction with Sirati & Partners Consultants Limited (SIRA TI) report attached.



Competent Natural Soil To Be Confirmed By SIRATI



Appendix D: Limitation and Use of the Report

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

The conclusions and recommendations given in this report are based on information determined at the borehole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the borehole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc. Professional judgement was exercised in gathering and analyzing data and formulation of recommendations using current industry guidelines and standards. Similar to all professional persons rendering advice, SIRATI cannot act as absolute insurer of the conclusion we have reached. No additional warranty or representation, expressed or implied, is included or intended in this report other than stated herein the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SIRATI accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

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

SIRATI engagement hereunder is subject to and condition upon, that SIRATI not being required by the Client, or any other third party to provide evidence or testimony in any legal proceedings pertaining to this finding of this report or providing litigations support services which may arise to be required in respect of the work produced herein by SIRATI. It is prohibited to publish, release or disclose to any third party the report produced by SIRATI pursuant to this engagement and such report is produced solely for the Client own internal purposes and which shall remain the confidential proprietary property of SIRATI for use by the Client, within the context of the work agreement.