Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
5613 & 5631-5633 VICTORIA AVENUE & 4874-4918 WALNUT STREET
NIAGARA FALLS, ONTARIO

Prepared For:	**Fugiel International Group Inc.** 6158 Allendale Avenue Niagara Falls, Ontario L2G 0A5
Attention:	Mr. Pawel Fugiel

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1.0 INTRODUCTION

Terraprobe Inc. was retained by Fugiel International Group Inc. to carry out a geotechnical investigation at the Property located at 5613 and 5613-5633 Victoria Avenue & 4874-4918 Walnut Street in Niagara Falls, Ontario, hereafter referred to as the 'the Property'. The location of the site is shown on the Site Location Plan, Figure 1. A proposal and cost estimate to carry out the work were outlined in our letter of May 21, 2021. Authorization to proceed with the investigation was provided by Mr. Fugiel on May 21, 2021.

The purpose of the work was to investigate and report on the subsurface soil, rock, and ground water conditions in a series of boreholes drilled at the site. Based on this information, advice is provided with respect to the geotechnical aspects of the proposed development. The anticipated construction conditions pertaining to excavation, shoring support, backfill and temporary ground water control are also discussed, but only with regard to how they might influence the proposed design.

Phase One & Two Environmental Site Assessments (ESA) and a hydrogeological assessment were also carried out concurrently with the geotechnical investigation and are being reported under separate cover.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Existing Site Conditions

The subject property is located at 5613 and 5613-5633 Victoria Avenue & 4874-4918 Walnut Street in Niagara Falls, Ontario. The property is generally rectangular in shape and occupies an area of approximately 1.4 hectares. The general arrangement of the site is shown on Figure 2. Ground surface in the area has a gentle slope going north to south. The overall slope is approximately 3% south.

The southeast portion of the Property is currently occupied by a convenience store with one basement level. It occupies approximately 225 m^2 of the 5,605 m^2 of the Property. The southern portion is occupied by a multi-commercial building with one main floor, an upper level and a basement. It occupies approximately 850 m^2 of the 5,605 m^2 of the Property. The remainder of the Property is in use as a parking lot.

2.2 Site Geology

Based on published geological information for the general area, near surface soil at and in the vicinity of the subject property generally consists of glaciolacustrine deposits comprised of sand, gravelly sand and gravel.[1] Beneath the overburden deposits is bedrock of the Lockport Formation consisting of sandstone,

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[1] Quaternary Geology, Niagara-Welland, Southern Ontario; Ministry of Natural Resources; Ontario Geological Survey; Map No. 2496; 1984.

shale, dolostone and siltstone.[2] The surface of the bedrock was encountered in all seven (7) boreholes at depths in the range of about 0.8 to 2.9 metres below the existing ground surface (m BGS), or between elevation 185.6 and 182.7 m.

2.3 Proposed Development

The proposed development features are shown on Figure 3, as derived from drawing A001 (dated August 18, 2021), prepared by Chamberlain Architect Services Limited. It is understood that the development presently under consideration would include 2 new buildings consisting of a 35-storey residential condominium building and a 34-storey hotel building. The 2 new buildings will rest on a 7-storey podium. We understand that 1 level of underground parking is presently being considered. The excavation will extend to the property limits; as such the overburden soils will have to be supported during the excavation.

3.0 PROCEDURE

The field work for this investigation was carried out on between June 22 and June 24, 2021, during which time three (3) boreholes (BH1, BH2D and BH3) were drilled to depths of about 20.2 to 20.4 m BGS. Terraprobe also advanced five (5) boreholes (BH2, BH4, BH5, BH6 and BH7) to depths of about 1.4 to 2.9 m BGS as part of a concurrent environmental investigation at the site. The locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the boreholes are shown on the Log of Borehole sheets presented in Appendix A.

The boreholes were drilled using track-mounted Geoprobe drill rig supplied and operated by a specialist drilling contractor. The boreholes were advanced using conventional interval augering and sampling techniques. Soil samples were recovered by split barrel sampling in accordance with ASTM D1586. In all of the boreholes, the overburden drilling and sampling were terminated upon practical refusal to conventionally auger which was assumed to coincide with the surface of the underlying bedrock.

The bedrock was explored in Boreholes 1, 2 and 3 by rotary core drilling with HQ sized rock coring equipment in core runs of 0.6 to 1.5 m lengths, and to depth of about 20.3 to 20.4 m BGS, or to elevations in the range of about 164.7 to 167.2 m. Our field staff logged and photographed the rock cores recovered from the boreholes. The Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD) values were recorded in accordance with the conventions used by the International Society for Rock Mechanics (ISRM).

Ground water observations were made in each borehole during and upon completion of drilling and sampling. In addition, a monitoring well was installed and sealed in Borehole 1, 2, 2D, 4, 6 and 7. The

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[2] Paleozoic Geology, Niagara, Southern Ontario; Ontario Division of Mines; Map No. 2344; 1976.

monitoring wells extended to depths of about 1.3 to 15.7 m BGS and were constructed of 50 mm diameter schedule 40 PVC screen and riser with a silica sand pack, and bentonite seal. The screened section of the wells ranged from 0.6 to 3.0 m long. The remainder of the monitoring well sections were sealed with bentonite to the existing ground surface. A conventional 50 mm diameter J-plug was used to seal the top of the wells and a stick-up steel well casing was installed at the ground surface and sealed with concrete. Details of the construction of the monitoring wells are presented on the attached corresponding borehole logs in Appendix A. The water levels were measured in the monitoring wells on July 7, 2021 by a member of our field staff, with subsequent readings taken on July 21, August 3 and August 19, 2021.

Boreholes that were not equipped with a monitoring well were decommissioned and sealed with bentonite pellets in accordance with Ontario Regulation 903, and sealed with nominally compacted commercial grade cold-mix asphalt patch at the pavement surface.

The field work was observed throughout by a member of our engineering staff who located the boreholes, arranged for the underground utility clearances at the borehole locations and cared for the samples obtained during the investigation. The borehole locations were located in the field in advance of drilling. The ground surface elevations at the borehole and monitoring well locations were inferred from a topographic survey of the site provided by J.D. Barnes Limited, dated March 8, 2021.

All of the samples recovered in the course of the investigation were brought to our Stoney Creek laboratory for further examination and laboratory testing. The results of laboratory testing of the recovered samples are shown on the Log of Borehole sheets in Appendix A. Uniaxial compressive strengths were determined on sixteen (16) sections of the cored bedrock, the results of which are shown on the corresponding Rock Core Logs in Appendix A.

4.0 SUBSURFACE CONDITIONS

The subsurface soil, bedrock, and ground water conditions encountered in the boreholes are presented on the attached Log of Borehole and Rock Core Log sheets in Appendix A. The stratigraphic boundaries indicated on the borehole and rock core logs are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in seven (7) generally evenly spaced boreholes and may vary between and beyond the borehole locations. The discussion has been simplified in terms of the major soil strata for the purposes of geotechnical design. For more specific subsurface details, refer to the enclosed Log of Borehole and Rock Core Log sheets in Appendix A.

4.1 Stratigraphy

The following stratigraphy is based on the results of the borehole findings, as well as the geotechnical laboratory testing conducted on selected representative soil samples. In general, the boreholes drilled at the site encountered asphalt pavements and fill, overlying a stratum of sandy silt to silty sand, and subsequently bedrock of the Lockport Formation.

4.1.1 Existing Pavements

All boreholes penetrated about 75 mm asphalt pavement at the existing ground surface underlain by about 150 to 250 mm of granular fill.

4.1.2 Fill

All boreholes encountered earth fill beneath existing pavements and granular fill to depths of about 0.8 to 2.9 m BGS or to about elevations 185.6 to 182.7 m. The fill generally consisted of silt, clayey silt, silty sand or sand with traces of clay, gravel and occasional brick and concrete pieces. Standard Penetration Testing carried out within the fill determined N values ranging from 2 to 39 blows per 0.3 m, generally indicating a loose to compact state of compaction. The higher N values were likely due to obstructions encountered while driving the split spoon. The in-situ water content of the samples of fill recovered from the boreholes ranged from about 4 to 19 percent.

4.1.3 Sandy Silt to Silty Sand

A stratum of sandy silt to silty sand was encountered in the boreholes BH1, BH2 and BH3 beneath the fill to depths of about 1.4 to 2.6 m BGS or to about elevations 184.9 to 183.1 m. It should be noted that Boreholes 2, 4, 5, 6 and 7 were terminated within this stratum. N values of ranging from 23 to greater than 100 blows per 0.3 m, generally indicating a compact to dense relative density. The natural water content of the samples of sandy silt to silty sand recovered from the boreholes ranged from about 5 to 16 percent.

4.1.4 Bedrock (Lockport Formation)

The augering and interval sampling method that was used for subsurface exploration on this project is conventionally accepted investigative practice. However, this method does not define the bedrock surface with precision, particularly in this instance where the surficial zone of the bedrock may be weathered. As best could be practically determined, the surface of the bedrock was encountered in all seven (7) boreholes at depths of about 1.2 to 2.9 m BGS or between about elevation 185.6 and 182.7 m. The N values determined at the bedrock surface were all greater than 100 blows per 0.3m. The following table shows those boreholes that were advanced to the bedrock or auger refusal interpreted as the top of the bedrock surface.

Top of Bedrock

Borehole #	**1**	**2D**	**3**	**4**	**5**	**6**	**7**
Depth (m)	1.4	2.0	2.6	1.6	1.2	2.0	2.9
Elevation (m)	183.6	183.5	184.9	182.7	184.7	185.6	184.4

The bedrock was continuously cored in boreholes BH1, BH2D and BH3 to depths of about 20.3 to 20.4 m BGS, or to elevations of 168.8 to 164.7 m. The bedrock consisted of grey limestone and dolostone of the Lockport Formation. The cored rock was observed to be fossiliferous, medium to thickly bedded, vuggy with calcite and/or gypsum, and with fine to medium crystalline. The TCR (total core recovery) ranged from 87 to 100 percent. The SCR (solid core recovery) ranged from 60 to 100 percent. The RQD (rock quality designation) value for the bedrock ranged from 52 to 100 percent, generally indicating a good to excellent quality. The discontinuities observed in the rock cores were typically horizontal and generally associated with the bedding planes. The joints were filled with clay and the joint spacing was generally close to moderately close. Uniaxial compressive strengths in the range of 61 to 147 MPa (average strength 120 kPa) were determined on sixteen (16) specimens of the rock cores.

4.2 Ground Water

Unstabilized ground water level observations were made in the open boreholes during and after drilling, as noted on the borehole logs. The water level measured within the installed wells is summarized below and are shown on the corresponding log of borehole sheet in Appendix A.

BH	Elevation of Well Screen (m)	Stratum Captured by Well Screen	Depth / Elevation of Water Level in Well (m)				
			During Drilling	July 07, 2021	July 21, 2021	August 3, 2021	August 19, 2021
1	184.3 – 183.4	Fill and Sandy silt	Dry	Dry	Dry	Dry	Dry
2	184.8 – 184.2	Fill and Sandy silt	Dry	Dry	Dry	Dry	Dry
2D	172.7 – 169.7	Bedrock	Dry	5.33 / 180.2	4.88 / 180.7	5.10 / 180.4	5.30 / 180.2
4	183.6 – 182.8	Fill	Dry	Dry	Dry	Dry	Dry
6	187.1 – 186.2	Fill	Dry	Dry	Dry	Dry	Dry
7	186.5 – 184.7	Fill	Dry	Dry	Dry	Dry	Dry

5.0 GEOTECHNICAL DESIGN

The following discussion is based on our interpretation of the factual data obtained during this investigation and is intended for the use of the design engineer only. Comments made regarding the construction aspects are provided only in as much as they may impact on design considerations. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

This report is based on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features, or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

The proposed development features are shown on Figure 3, as derived from drawing A001 (dated August 18, 2021), prepared by Chamberlain Architect Services Limited. It is understood that the development presently under consideration would include 2 new buildings consisting of a 35-storey residential condominium building and a 34-storey hotel building. The 2 new buildings will rest on a 7-storey podium. We understand that 1 level of underground parking is presently being considered. The excavation will extend to the property limits; as such the overburden soils will have to be supported during the excavation.

5.1 Site Preparation

The site is underlain by fill and very loose sand deposits that are not considered competent to support foundations, floor slabs on grade, or any settlement sensitive structures or underground plant. The nature of the site pre-grading will depend on the foundation design alternative selected as well as the scope of any work that may be required to address environmental issues.

Based on the current proposed design, preparation of the site will generally consist of the removal of existing asphalt pavements as well as any unsuitable fill, and cutting to achieve the design subgrade elevations. While specifically not encountered in the boreholes, it is possible that building foundations from former structures at the site could still exist within the overburden soils.

Some improvement of the condition of the near surface soil for the support of underground services and flexible pavements may be feasible. Such work would typically consist of the removal of any existing foundations, abandoned underground plant, as well as a portion of the fill, compacting the remaining fill to the extent possible from surface and restoring the site with engineered fill. The extent of fill that should be removed in these areas should be addressed by the geotechnical engineer at the time of construction.

5.2 Foundation Design Parameters

A building with 1 level of underground parking will penetrate the dolostone bedrock of the Lockport Formation. Spread footing foundations made on bedrock of the Lockport Formation can be designed for factored geotechnical resistance at ULS of 12 MPa. We recommend that there be a limit to the net allowable bearing pressure (SLS) to a maximum of 8 MPa, since we are aware of no specific load testing that has been carried out on this rock formation. The settlement of spread footings made on the bedrock will be negligible.

It is recommended that the minimum footing width for strip footings be 500 mm, and a minimum footing width of 900 mm be used for the design of spread footings, regardless of the structural requirements. Footings stepped from one level to another should be at a slope not exceeding 7 vertical to 10 horizontal.

Footings exposed to freezing temperatures must be provided with at least 1.2 metres of earth cover for frost protection or equivalent insulation. Where there are two or more levels of underground space, it is generally accepted practice to provide a minimum of 450 mm earth cover for perimeter footings and a minimum of 900 mm of earth cover for interior column foundations in “unheated” parking garages. Only in the immediate area of ventilation shafts is 1.2 metres of earth cover or equivalent insulation used.

5.3 Earthquake Design Parameters

Under Ontario Regulation 88/19, the ministry amended Ontario’s Building Code (O. Reg 332/12) to further harmonize Ontario’s Building Code with the 2015 National Codes. These changes are intended to help reduce red tape for businesses and remove barriers to interprovincial trade throughout the country. The amendments are based on code change proposals the ministry consulted in 2016 and 2017. The majority of the amendments came into effect on January 1, 2020, which includes structural sufficiency of buildings to withstand external forces and improve resilience.

Seismic hazard is defined in the 2012 Ontario Building Code (OBC 2012) by uniform hazard spectra (UHS) at spectral coordinates of 0.2 s, 0.5 s, 1.0 s and 2.0 s and a probability of exceedance of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g. shear wave velocity (vs), Standard Penetration Test (SPT) resistance, and undrained shear strength (su)) in the top 30 meters of the site stratigraphy below the foundation level, as set out in Table 4.1.8.4A of the Ontario Building Code (2012). There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g. sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain peak ground acceleration (PGA), peak ground velocity (PGV) site coefficients Fa and Fv, respectively, used to modify the UHS to account for the effects of site-specific soil conditions.

Based on the above noted information and assuming to building will be entirely supported on bedrock, it is recommended that the site designation for seismic analysis be ‘Site Class A’, as per Table 4.1.8.4.A of the Ontario Building Code (2012).

The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.B. to 4.1.8.4.I of the OBC 2012, as amended January 1, 2020, using linear interpolation for intermediate values of PGA.

5.4 Earth Pressure Design Parameters

The parameters used in the determination of earth pressures acting on retaining walls are defined below.

Parameter	Definition	Units
φ	internal angle of friction	degrees
γ	bulk unit weight of soil	kN / m^3
K_a	active earth pressure coefficient (Rankin)	dimensionless
K_o	at-rest earth pressure coefficient (Rankin)	dimensionless
K_p	passive earth pressure coefficient (Rankin)	dimensionless

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

Stratum/Parameter	ϕ	γ	Ka	Ko	Kp
Compact Granular Fill Granular 'B' (OPSS 1010)	32	21.0	0.31	0.47	3.25
Sandy Silt to Silty Sand or Similar Fill	30	19.0	0.33	0.50	3.00
Lockport Formation	28	25	n/a	n/a	n/a

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

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

where,
$\mathbf{P}$ = the horizontal pressure at depth, $\mathbf{h}$ (m)
$\mathbf{K}$ = the earth pressure coefficient,
$\mathbf{h_w}$ = the depth below the ground water level (m)
γ = the bulk unit weight of soil, (kN/m^3)
γ' = the submerged unit weight of the exterior soil, (γ - 9.8 kN/m^3)
$\mathbf{q}$ = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, acting in conjunction with the earth pressure, this equation can be simplified to:

$$\mathbf{P} = \mathbf{K}[\gamma\mathbf{h} + \mathbf{q}]$$

The factored geotechnical resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction **(R)** depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil **(tan ϕ)** expressed as: **R = N tan ϕ**. This is an unfactored resistance. The factored resistance at ULS is **R_f = 0.8 N tan ϕ**. The K value to be used for the design will depend on the rigidity of the wall.

5.4.1 **Rock Pressure**

If a structure is made such that it is cast directly into an excavation in the bedrock, the empirical approach for the design of foundation walls below bedrock level has been to use a uniform pressure distribution which is consistent with the maximum earth pressure calculated for the lowest level of soil in the profile. This approach is likely conservative in that the rock is effectively self-supporting, but is recognizes the practical requirement to have a foundation wall of a consistent width through the lower reach of the building.

Since the overburden soil cover on this site is minimal, it is expected that the structural requirements for the vertical capacity of the wall will govern wall design rather than lateral pressures. The bedrock is self-supporting in a vertical face and will exert no active pressure on the structure. It must be assumed that the hydrostatic forces on the basement walls would act to the base of the excavation in rock, unless the rock can be effectively drained by the perimeter drainage system where the rock face is exposed in the lower reaches of the excavation. To avoid this pressure the face of the rock should be continuously covered by a drainage core prefabrication that discharges under the floor at the lowest level of the building and any water which enters can then be conveyed to the building storm water discharge.

This approach does not recognize the potential for pressures on the basement walls due to the presence of locked-in horizontal stresses that are relieved when a rock cut is made. It is expected that there would be a sufficient time between the cutting of the rock face and construction of the building structure to allow the rock to de-stress. There is not much documented experience with stress relief and swell in the Lockport Formation. If there is not sufficient time lag between the excavation and the subsequent placing of footings, walls and the suspended slab levels, then special provisions need to be made with respect to time dependent rock stress relief in the design of basement walls of structures.

Where pits are made for sumps and elevators, or other such features which are incorporated within the major excavation, there must also be consideration of the potential for rock squeeze effects if the pits are to be cast directly against the rock face. For such structures, a compressible layer can be placed between the rock and the concrete or alternatively the local structures can be over-excavated and backfilled.

5.5 Slab on Grade Design Parameters

It is expected that the elevation of the finished floors of the underground parking structure will be within the bedrock. The modulus of subgrade reaction appropriate for slab on grade design in the rock is 80,000 kPa/m.

It is understood that the underground levels will be used primarily for parking, storage and for mechanical/electrical plant. On this basis it is anticipated that moisture sensitive floor coverings are not proposed for this level and it may not be necessary to incorporate a vapour barrier into the design of the floor slabs on grade. If moisture sensitive floor finishes are proposed, a capillary moisture barrier and drainage layer will be required beneath the slab. If a polyethylene barrier is selected for this application, care must be taken to ensure that the barrier is not damaged during concrete placement. The polyethylene barrier should be not less than 10 mill and must be supported by a layer of well-graded granular material to provide uniform support for the slab. It should be recognized that the use of a polyethylene barrier could have a negative impact on the quality of slab finish (i.e., curling).

Any buildings with below grade space must have both perimeter and subfloor drainage. The subfloor drainage system is made by placing the slab on a minimum 200 mm layer of 19 mm stone (OPSS 1004) compacted by vibration to a dense state. The nominal spacing of subdrains, in the order of 8 to 10 metres is expected to provide an adequate avenue for the removal of water beneath the slab. Basement drainage is required as discussed in the following Section 5.5.

All slabs on grade should be structurally separate from foundation walls and columns. Saw cut control joints should be incorporated into the slabs along column lines and at regular intervals. Interior load bearing walls should not be founded on the slab but on spread footings as outlined above.

5.6 Basement Drainage

To assist in maintaining dry basements and preventing seepage, it is recommended that exterior grades around the building be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 m. Provision of nominal subfloor drainage is required in conjunction with the perimeter drainage of the structure, to collect and remove the water that infiltrates at the building perimeter and under the floor. Perimeter and subfloor drainage are required throughout below grade areas.

It is recommended that the subfloor drainage system consists of minimum 100 mm diameter perforated pipes spaced at a maximum of 9 metres on centre. The pipes must be surrounded on all sides by a minimum of 100 mm of 19 mm clear stone (OPSS 1004), and the pipe inverts should be a minimum 300 mm below the base of the slab. The elevator pits can be drained separately with an independent lower pumping sump or can be designed as water proof structures which are below the drainage level.

Foundation walls must be damp-proofed in conformance to Section 5.8.2 of the Ontario Building Code (2012). Prefabricated drainage composites, such as Miradrain 2000 (Mirafi) or Terradrain 200 (Terrafix), should be incorporated between the shoring wall or rock face and the cast-in-place concrete foundation wall to make a drained cavity. Drainage from the cavity must be collected at the base of the wall in non-perforated pipes and conveyed directly to the sumps. The flow to the building storm water sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored excavation drainage details are provided in Appendix B.

The drainage system is a critical structural element, since it keeps water pressure from acting on the basement floor slab or on the foundation walls. As such, the sump that ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and these pumps must be on emergency power. The size of the pump should be adequate to accommodate the anticipated ground water and storm event flows. It is expected that the seepage can be controlled with typical widely available, commercial sump pumps.

5.7 Site Servicing

It is expected that site services will consist of storm and sanitary sewers and watermains, with relatively shallow inverts (less than 3 m). The invert elevation is expected to be within the bedrock. Excavations for underground services should be made as outlined in Section 6.1 of this report. The locations and depths of any building foundations which would potentially be affected by the proposed utilities should be identified prior to commencing the excavation.

5.7.1 Bedding

Bedding for the pipes should consist of well graded free draining granular material such as Granular A, which is compatible with the size and type of pipe. All bedding material should be uniformly compacted to at least 95 percent of standard Proctor maximum dry density.

It is possible that the excavation for some services would terminate in the fill. In this event, it will be necessary to sub-excavate all fill and replace it with engineered fill to ensure the service is properly supported and to minimize the potential of settlement. Engineered fill should consist of OPSS 1010 Granular A or Granular B Type II material placed and uniformly compacted to 98 percent of standard Proctor maximum dry density. Consideration could also be given to the use of lean concrete to restore the grade to the proposed invert elevation.

5.7.2 Backfill

Based on the results of the boreholes, it is assumed that the majority of excavated soil at the site from the construction of service trenches will consist of existing fill. Any fill containing topsoil and/or soil containing high amounts of organic material or debris should not be used or re-used for service trench backfill.

Service trench backfill should consist of clean earth, free of excessively wet or frozen soil and should be placed in lifts of 300 mm thickness or less and uniformly compacted to at least 95 percent of standard Proctor maximum dry density at placement water contents within 2 percent of the corresponding laboratory optimum water content for compaction. The upper 1m of the backfill forming any pavement subgrade should be uniformly compacted to 98 percent of standard Proctor maximum dry density.

It may be difficult to consistently achieve the degree of compaction specified above using existing fills on site or the native excavated soil as trench backfill, particularly in narrow trenches. For this reason, consideration could be given to using free draining granular material, such as Granular A or Granular B Type I (OPSS 1010) as trench backfill to allow for adequate, uniform compaction.

5.8 Pavement Design

5.8.1 Subgrade Preparation

Topsoil and deleterious fill should be stripped from all areas to be developed for new pavements. It is recommended that the subgrade be cut as cleanly as possible to minimize disturbance and be proof rolled with a static roller to identify any loose or disturbed areas. The preparation of the subgrade and the compaction of all fills should be monitored by the geotechnical engineer at the time of construction.

If fill is required to raise the grade, there may by some select on-site fill which could be used, provided it is free of topsoil and other deleterious material, and is at suitable placement water contents. The fill should be placed in large areas where it can be uniformly compacted in 300 mm thick lifts with each lift uniformly compacted to at least 95 percent of standard Proctor maximum dry density. The upper 1 m of backfill beneath areas to be developed as pavements should be compacted to 98 percent of standard Proctor maximum dry density.

Control of surface water is a significant factor in achieving good pavement life. Grading of adjacent pavement areas must be designed so that water is not allowed to pond adjacent to the outside edges of the pavement or curb. The existing earth fill and native soils have anywhere from a slight to severe frost susceptibility to frost heave, and pavement on these materials must be designed accordingly. The subgrade must be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward subgrade drains.

Continuous pavement subdrains should be provided along both sides of the driveway/access routes and drained into catch-basins to facilitate drainage of the subgrade and the granular materials. The subdrain invert should be maintained at least 0.3 metres below subgrade level. Subdrains should also be provided at all catch-basins within the parking areas..

5.8.2 Asphaltic Concrete Pavement Design

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions and for the design traffic loadings. On this basis, the following pavement component thicknesses are recommended for flexible pavements which will be subjected to “heavy duty” use (i.e., main site accesses and service accesses) and “light duty” use (ie car parking) constructed on a properly prepared clayey silt subgrade.

Minimum Asphaltic Concrete Pavement Structure

Pavement Layer	Compaction Requirements	Light Duty Minimum Component Thickness	Heavy Duty Minimum Component Thickness
Surface Course Asphaltic Concrete HL3 (OPSS 1150)	92-96.5% MRD	40 mm	40 mm
Base Course Asphaltic Concrete HL8 (OPSS 1150)	92-96.5% MRD	50 mm	60 mm
Base Course: Granular A (OPSS 1010) or 19mm Crusher Run Limestone	98% standard Proctor Maximum Dry Density (ASTM-D1557)	150 mm	150 mm
Subbase Course: Granular B Type II (OPSS 1010) or 50mm Crusher Run Limestone	98% standard Proctor Maximum Dry Density (ASTM-D1557)	300 mm	350 mm

Some adjustment to the thickness of the granular subbase material may be required depending on the condition of the subgrade at the time of the pavement construction. The need for such adjustments can be best assessed by the geotechnical engineer during construction.

Consideration should be given to delaying the placement of the final wearing surface for at least one year after construction of the binder course in order to minimize the effects of post construction settlement. Prior to placing the wearing surface, the binder course should be evaluated by the geotechnical engineer and remedial work carried out as required in preparation for final construction..

5.8.3 Drainage

Control of surface water is a significant factor in achieving good pavement life. Grading adjacent to pavement areas must be designed so that water is not allowed to pond adjacent to the outside edges of the pavement or curb. The subgrade must be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward subgrade drains or swales and/or ditches.

Continuous perimeter subdrains should be provided in paved areas and short perforated sub drains should be provided at all catch basins locations. The subdrain invert elevations should be maintained at least 0.3 metres below subgrade level.

It should be noted that in addition to a strict adherence to the above pavement design recommendations, a close control on the pavement construction process will be required in order to obtain the desired pavement life. It is therefore recommended that regular inspection and testing should be conducted during the construction to confirm material quality, thickness, drainage, and to ensure adequate compaction.

6.0 DESIGN CONSIDERATIONS FOR CONSTRUCTABILITY

6.1 Excavations

6.1.1 Overburden Soil

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III – Excavations, Sections 222 through 242. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes at this site, the existing fill is considered a Type 3 Soil, provided that effective ground water control is achieved where required and surface water is directed away from open excavations.

Where workers must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates maximum slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

6.1.2 Bedrock

The bedrock below the site is of medium to high strength and cannot be removed with conventional excavation equipment. This is not a rippable rock. Given the depth of the bedrock, which was found at depths of about 1.2 to 2.9 m BGS or between about elevation 185.6 and 182.7 m, serious consideration must be given to the potential for break the rock mass prior to excavation. An experienced contractor would have to be engaged to prepare a proposal for the works. Excavation of the dolostone/limestone rock by mechanical means (line drilling, hoe mounted hydraulic rams, jack hammering and/or rock splitting) may be feasible. Once the bulk of the rock has been removed then it should be expected that the detailed shapes for foundations and the edges of the excavation would be trimmed with hoe mounted hydraulic rams.

Excavations made in bedrock can be vertical, provided the rock faces are scaled and maintained to preclude the possibility of spalls. Where this is not possible, protective mesh can be draped over the rock face when work is required in the area immediately beside the cut rock face. It has been assumed that the foundations will be cast neat in excavations made in the bedrock and that no backfill is required. If there is over excavation at the perimeter of the foundations, the excavated dolostone material is not a suitable material for backfill of the excavations. Over excavation below footings must be replaced with 25 MPa concrete. Over excavation beside footings can be restored with lean concrete (minimum 10 MPa unconfined compressive strength). The deepest footings and pits must be made first so that there is no chance of rock at higher elevations being disturbed by other excavation after foundations are placed.

6.1.3 Ground water

Based on the observed ground water conditions, we are recommending that lowest basement level be set above elevation 181.4 m, which is about 1 m above the reported ground water elevation. Deeper excavations will warrant the installation of additional ground water monitoring wells to better assess ground water conditions that would be expected to be encountered during construction and for long term discharge permits with the region/municipality (if required). Raising the basement level will also minimize the requirements for rock excavation and construction dewatering, and possibly eliminate the need for waterproofing.

6.1.4 Shoring Design Considerations

The depth of the overburden soil at this site varies from about 0.8 to 2.9 metres below existing grade (generally deepest at the location of Borehole Nos. 3 and 7 at the west corner of the site). As such, there is very little soil to be retained at the edges of the proposed excavations. The extent to which the municipality will allow sloping of the soil at the edge of the site will dictate whether any shoring must occur. If a straight edge excavation is required above the top of the bedrock, then a system of steel soldier piles and lagging pinned into the top of the rock and anchored would be the appropriate solution. Terraprobe can provide shoring design services for this project if requested. Rock anchorages are typically made in the Lockport Formation using a design bond stress of 1 MPa, without proof testing. Higher bond stresses are possible but proof testing of anchorages on a site-by-site basis is required. The use of soil anchors established in the overburden soil is not recommended.

6.2 Quality Control

6.2.1 Shoring

The City of Niagara Falls will require that the shoring installations be monitored during the period of construction to demonstrate that the shoring is performing adequately. Terraprobe has considerable experience in the provision of shoring instrumentation and monitoring services for a number of similar sites.

The provisions of the Ontario Building Code require that the construction of the earth retaining structures be monitored on a continuous basis. The shoring system constitutes an earth retaining structure as provided in Section 4.2.2.3 of the Ontario Building Code 2012. Terraprobe should be retained to provide this review as the shoring installations are made. It is an integral part of the geotechnical design function as it relates to shoring design considerations.

Assuming rock anchors will be used to support the shoring system on this site, a minimum of one anchor as each target anchorage level must be performance tested to verify the design adhesion used for the anchorages. This performance test anchor shall be consistent dimension in anchor and free stressing zones with the proposed production anchors and be provided with adequate tendon steel capacity to test the anchor to twice the design working load. The performance tests shall be monitored and evaluated by the geotechnical engineer. Production anchorages should not be installed until the performance test at each level has adequately demonstrated the design adhesion value. All production anchorages shall be monitored during stressing and evaluated by a geotechnical engineer.

6.2.2 Foundations

The proposed structures will be founded on conventional spread footings. All foundation installations must be reviewed in the field by Terraprobe, the geotechnical engineer, as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical engineering design function and is required by Section 4.2.2.2 of the Ontario Building Code 2012. If Terraprobe is not retained to carry out foundation engineering field review during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice contained in this report.

6.2.3 Slabs on Grade

The long-term performance of the slab on grade is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as practically possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and the compaction of all fill should be monitored by Terraprobe at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

6.2.4 General

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density (SPMDD). In situ determinations of density during fill and asphaltic pavement placement on site are required to demonstrate that the specified placement density is achieved. Terraprobe is a CNSC certified operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff.

Concrete will be specified in accordance with the requirements of CAN3 - CSA A23.1/2. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

Terraprobe staff can also provide quality control services for Building Envelope, Roofing and Structural Steel, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1.

7.0 LIMITATIONS AND USE OF REPORT

7.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

The drilling work was carried out by a drilling contractor and was observed and recorded by Terraprobe on a full-time basis. The boreholes were made by a continuous flight power auger machine using solid stem augers and NQ rock coring. The Terraprobe technician logged the boreholes and examined the samples as they were obtained. The samples obtained were sealed in clean, air-tight containers or rock core boxes and transferred to the Terraprobe laboratory, where they were reviewed for consistency of description by a geotechnical engineer. Ground water monitoring wells were installed in both boreholes to measure long-term ground water levels.

The samples of the strata penetrated were obtained using the Split-Barrel Method technique (ASTM D1586). The samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. A comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations.

It may not be possible to drill a sufficient number of boreholes, or sample and report them in a way that would provide all the subsurface information and geotechnical advice to completely identify all aspects of the site and works that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project must be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, and their approach to the construction works, cognizant of the risks implicit in the subsurface investigation activities.

7.2 Changes in Site and Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils, and frost protection.

The design parameters provided and the engineering advice offered in this report are based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained design consultants in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters, advice and comments relating to constructability issues and quality control may not be relevant or complete for the project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

7.3 Use of Report

This report is prepared for the express use of Fugiel International Group Inc. and their retained design consultants. It is not for use by others. This report is copyright of Terraprobe Inc., and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe.

It is recognized that the City of Niagara Falls, in their capacity as the planning and building authority under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

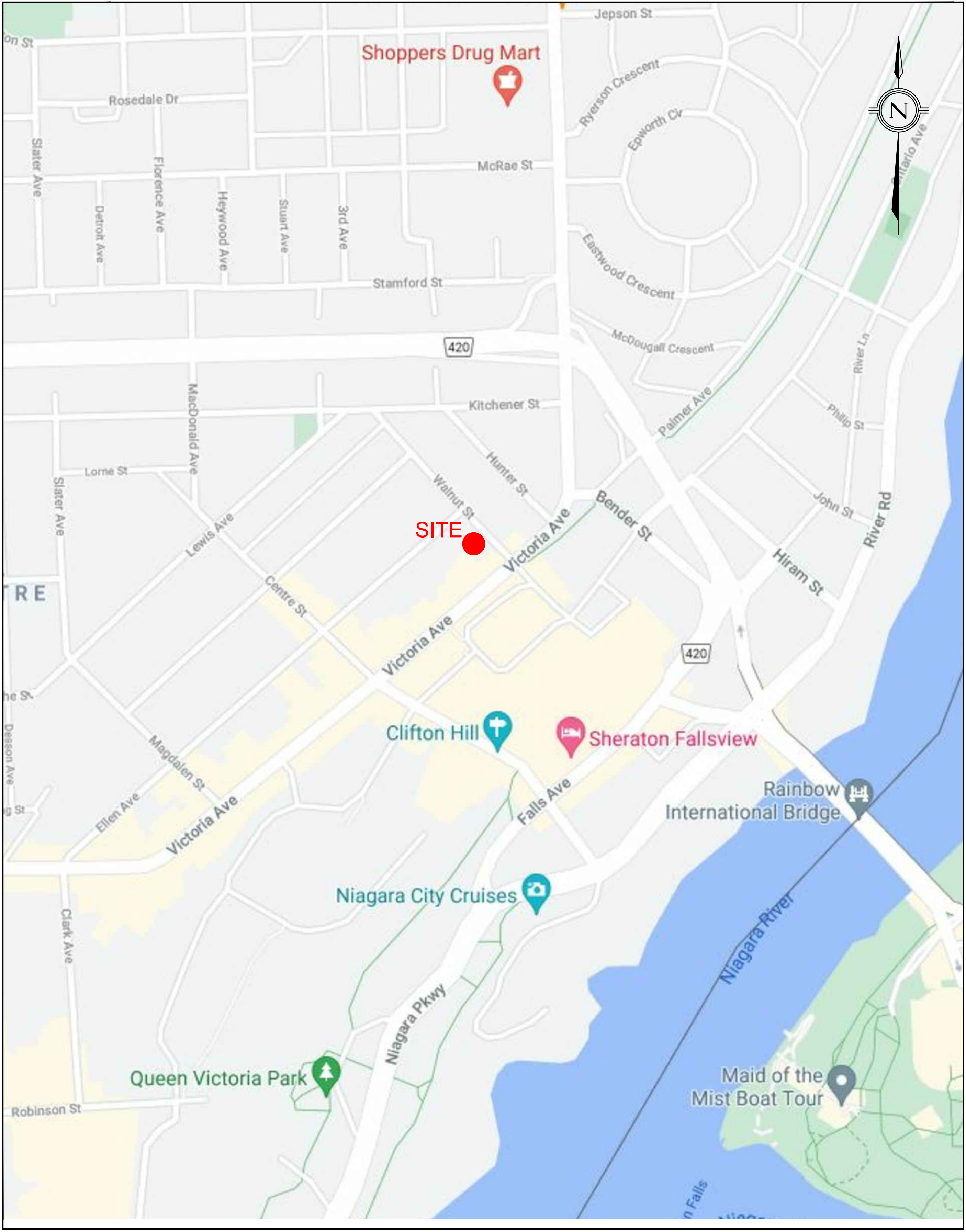
Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
P. T. CANNON
Aug 18/21
PROVINCE OF ONTARIO

Patrick Cannon, P. Eng.
Principal

FIGURES

Terraprobe Inc.



Terraprobe

903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R7
Tel: (905) 643-7560, Fax: (905) 643-7559

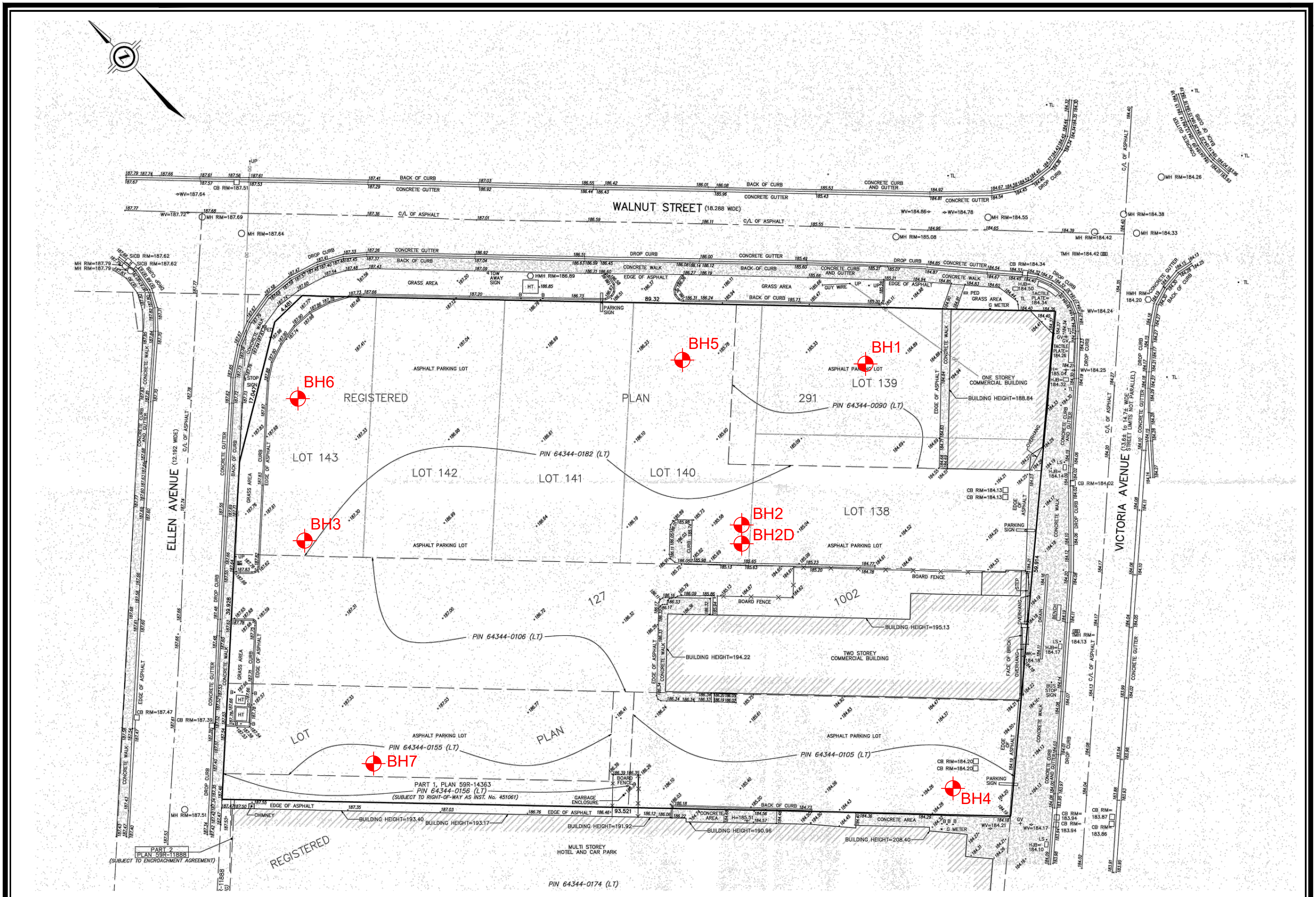
Title:

SITE LOCATION PLAN

5613 & 5631-5633 Victoria Avenue & 4874-4918 Walnut Street, Niagara Falls

File No. 7-21-0048-01

FIGURE : 1



LEGEND

BH1 ⌖ Borehole Location

Terraprobe

903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R7
Tel: (905) 643-7560, Fax: (905) 643-7559

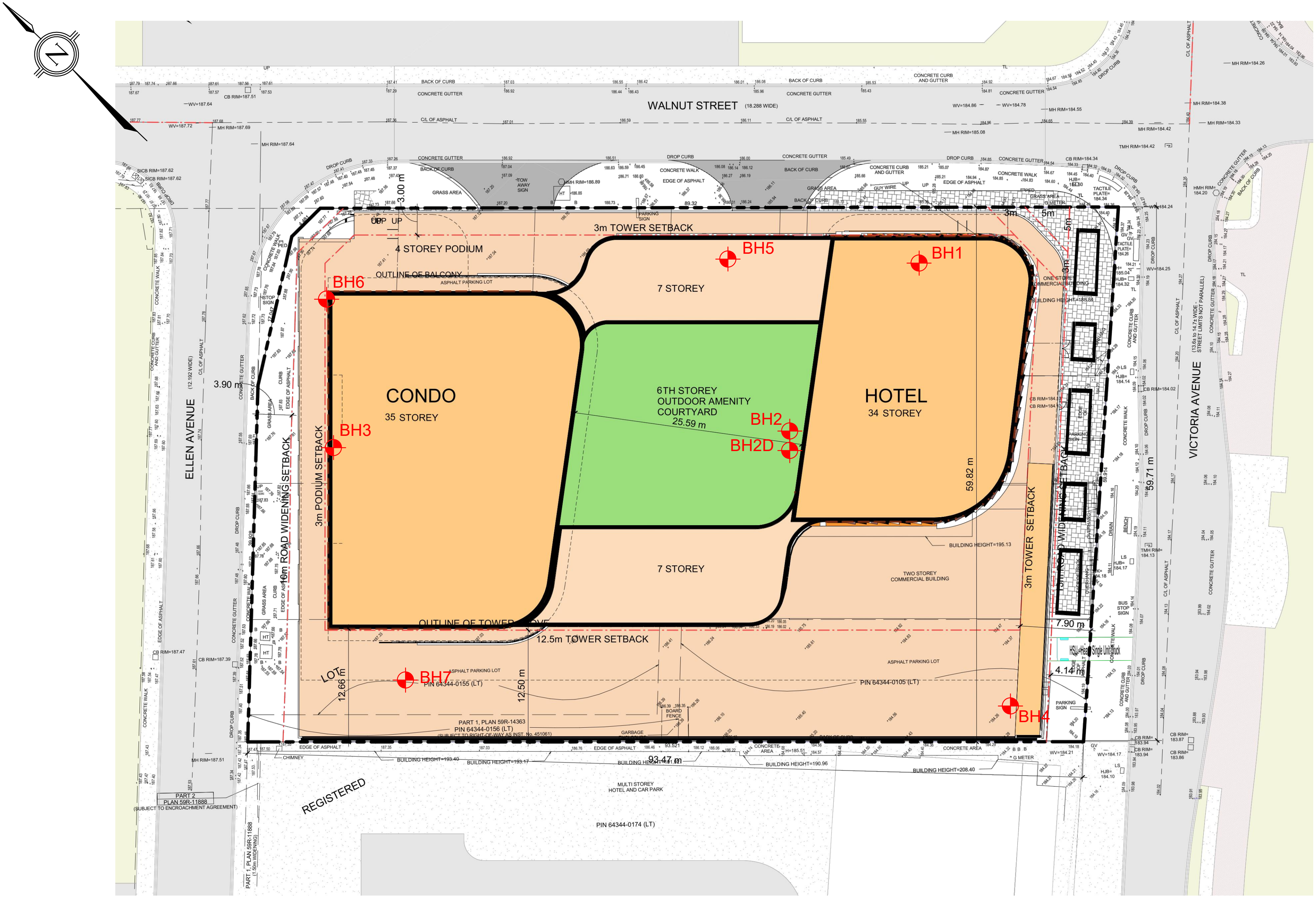
Title:

BOREHOLE LOCATION PLAN

5613 & 5631-5633 Victoria Avenue & 4874-4918 Walnut Street, Niagara Falls

File No. 7-21-0048-01

FIGURE : 2



LEGEND

BH1 Borehole Location

Terraprobe

903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R7
Tel: (905) 643-7560, Fax: (905) 643-7559

Title:

PROPOSED SITE DEVELOPMENT PLAN
5613 & 5631-5633 Victoria Avenue & 4874-4918 Walnut Street, Niagara Falls

File No. 7-21-0048-01

FIGURE : 3

LOGS OF BOREHOLES

APPENDIX A

Terraprobe Inc.

ABBREVIATIONS AND TERMINOLOGY

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

Consistency	'N' value	Undrained Shear Strength (kPa)
very soft	< 2	< 12
soft	2 – 4	12 – 25
firm	4 – 8	25 – 50
stiff	8 – 15	50 – 100
very stiff	15 – 30	100 – 200
hard	> 30	> 200

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silt*y*	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

▽	Unstabilized water level
▽	1st water level measurement
▽	2nd water level measurement
▼	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

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CORE LOG TERMINOLOGY

RECOVERY

TCR Total Core Recovery is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

SCR Solid Core Recovery is the total length of sound full-diameter core pieces obtained in a core run, expressed as a percentage of the length of that core run.

RQD Rock Quality Designation pertains to the sum of those pieces of sound core which are 10 cm or greater in length obtained in a core run, expressed as a percentage of the length of that core run.

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
QUALITY	**very poor**	**poor**	**fair**	**good**	**excellent**

JOINT CHARACTERISTICS

Orientation

horizontal / flat = 0 - 20°
dipping = 20 - 50°
vertical = 50 - 90°

Joint Aperture

Classification	Aperture
closed	< 0.5 mm
gapped	0.5 to 10 mm
open	> 10 mm

Joint Spacing (ISRM, 1981)

Classification	Spacing (m)
extremely close	< 0.02
very close	0.02 to 0.06
close	0.06 to 0.2
moderately close	0.2 to 0.6
wide	0.6 to 2
very wide	2 to 6
extremely wide	> 6

Joint Filling

Description	Approx φ
tight, hard, non-softening	25 - 35
oxidation, surface staining only	25 - 30
slightly altered, clay-free	25 - 30
sandy particles, clay-free	20 - 25
sandy and silty, minor clay	16 - 24
non-softening clays	6 - 12
swelling clay fillings	n/a

Degree of Weathering (after MTO, RR229 Evaluation of Shales for Construction Projects)

Zone	Degree	Description
Z1	unweathered	shale, regular jointing
Z2	partially weathered	angular blocks of unweathered shale, no matrix, with chemically weathered but intact shale
Z3		soil-like matrix with frequent angular shale fragments < 25mm diameter
Z4a		soil-like matrix with occasional shale fragments < 3mm diameter
Z4b	fully weathered	soil-like matrix only

Strength classification (after Marinos and Hoek, 2001)

Grade	Term	UCS (MPa)	Field Estimate (Description)
R6	extremely strong	> 250	can only be chipped by geological hammer
R5	very strong	100 - 250	requires many blows from geological hammer
R4	strong	50 - 100	requires more than one blow from geological hammer
R3	medium strong	25 - 50	can't be scraped, breaks under one blow from geological hammer
R2	weak	5 - 25	can be peeled / scraped with knife with difficulty
R1	very weak	1 - 5	easily scraped / peeled, crumbles under firm blow of geo. hammer
R0	extremely weak	< 1	indented by thumbnail

Terraprobe

LOG OF BOREHOLE 1

Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 1 of 1

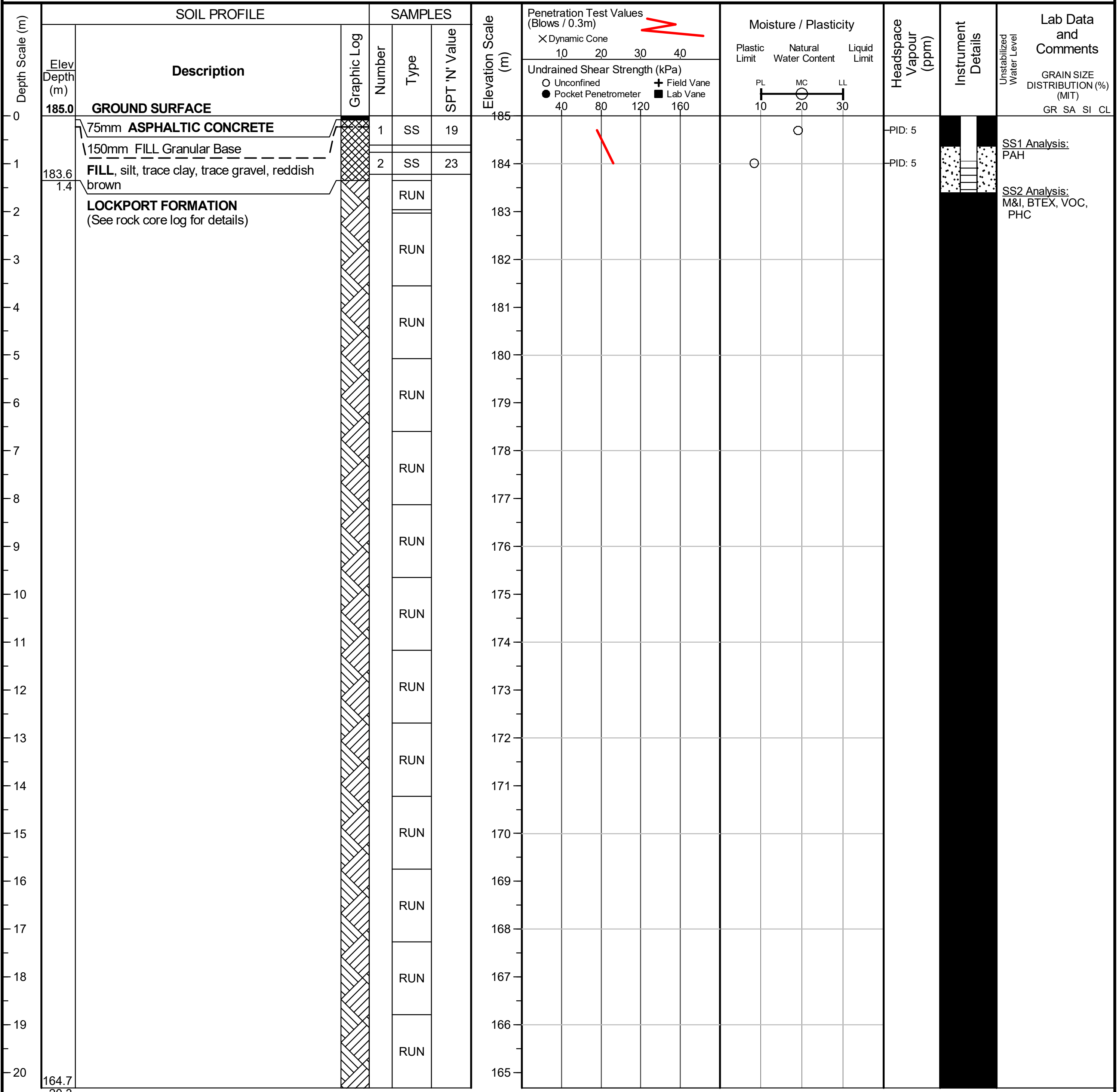
Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

Position : E: 656736, N: 4773009 (UTM 17T)
Rig type : Geoprobe
Elevation Datum : Geodetic
Drilling Method : Solid stem augers, HQ rock coring
Core Diameter : **HQ**, OD=96mm, ID=64mm

Elev Depth (m)	SOIL PROFILE Description
185.0	**GROUND SURFACE**
	75mm **ASPHALTIC CONCRETE**
	150mm FILL Granular Base
	FILL, silt, trace clay, trace gravel, reddish brown
183.6 1.4	**LOCKPORT FORMATION** (See rock core log for details)
164.7 20.3	

Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments
1	SS	19	PID: 5	SS1 Analysis: PAH
2	SS	23	PID: 5	SS2 Analysis: M&I, BTEX, VOC, PHC
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			



END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jul 7, 2021	dry	n/a
Jul 21, 2021	dry	n/a
Aug 3, 2021	dry	n/a

Terraprobe

ROCK CORE LOG 1

Project No. : 7-21-0048-01	Client : Fugiel International Group Inc.	Originated by : KG
Date started : June 21, 2021	Project : 5613 Victoria Avenue	Compiled by : TW
Sheet No. : 1 of 2	Location : Niagara Falls, Ontario	Checked by : TW

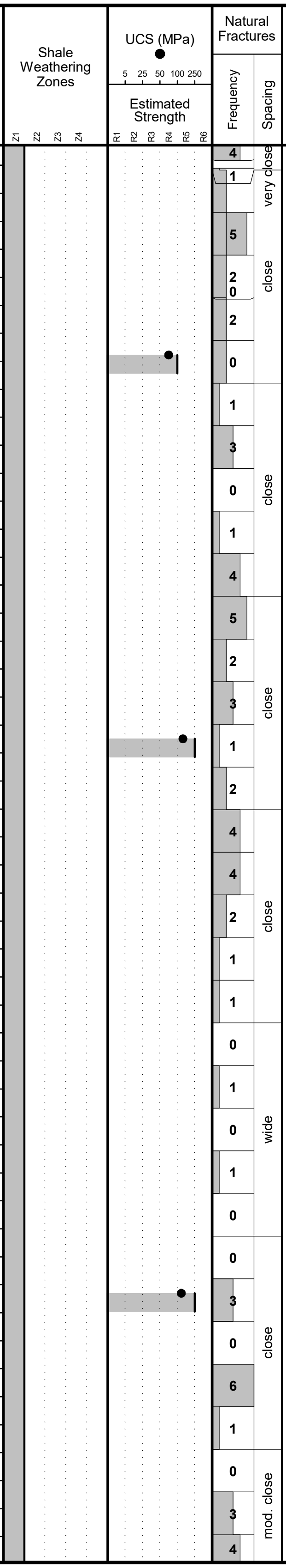
Position : E: 656736, N: 4773009 (UTM 17T)	Elevation Datum : Geodetic	Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Geoprobe	Drilling Method : Solid stem augers, HQ rock coring	

GENERAL DESCRIPTION

Rock coring started at 1.9m below grade

LOCKPORT FORMATION
dolomitic Limestone, grey, medium bedded to thickly bedded, crystalline, fossiliforous, vuggy; joints are horizontal

Run	Elev / Depth (m)	Recovery
	183.1	
R1	1.4 183.0 2.0	TCR = **100%** SCR = **60%** RQD = **60%**
R2	181.4	TCR = **100%** SCR = **89%** RQD = **68%**
R3	3.6 179.9	TCR = **100%** SCR = **93%** RQD = **72%**
R4	5.1 178.4	TCR = **100%** SCR = **87%** RQD = **68%**
R5	6.6 176.9	TCR = **100%** SCR = **88%** RQD = **52%**
R6	8.1 175.3	TCR = **100%** SCR = **98%** RQD = **91%**
R7	9.7 173.8	TCR = **100%** SCR = **98%** RQD = **78%**
R8	11.2	TCR = **100%** SCR = **98%** RQD = **97%**



Depth (m)	Elevation (m)	Laboratory Testing	Comments
2	183		
3	182	**3.4m to 3.5m:** UCS = 74.9 MPa	
4	181		
5	180		**5.0-5.1m:** *Weathered layer*
6	179	**6.1m to 6.2m:** UCS = 147 MPa	**6.5-6.6m:** *Vuggy Zone*
7	178		**6.8-6.9m:** *Vuggy Zone*
8	177		
9	176		**9.3-9.3m:** *Vuggy Zone*
10	175	**10.1m to 10.2m:** UCS = 133.7 MPa	
11	174		

(continued next page)

ROCK CORE LOG 1

Project No. : 7-21-0048-01
Client : Fugiel International Group Inc.
Originated by : KG
Date started : June 21, 2021
Project : 5613 Victoria Avenue
Compiled by : TW
Sheet No. : 2 of 2
Location : Niagara Falls, Ontario
Checked by : TW

Position : E: 656736, N: 4773009 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Geoprobe
Drilling Method : Solid stem augers, HQ rock coring

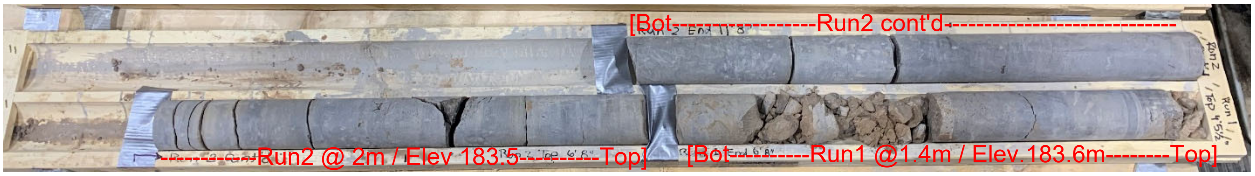
GENERAL DESCRIPTION

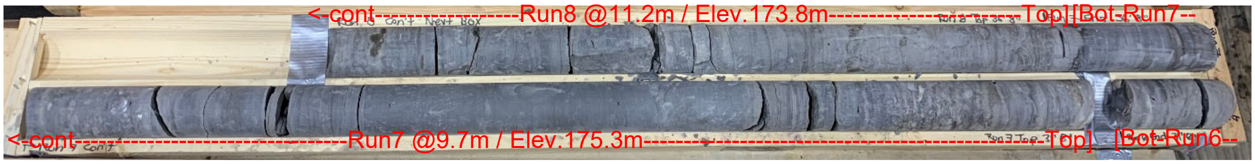
LOCKPORT FORMATION
dolomitic Limestone, grey, medium bedded to thickly bedded, crystalline, fossiliforous, vuggy; joints are horizontal

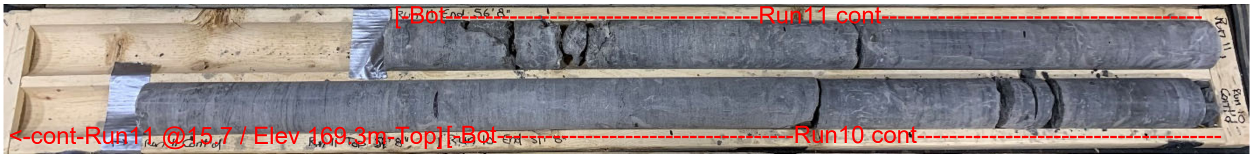
Run	Elev Depth (m)	Recovery	Natural Fractures Frequency	Spacing	Laboratory Testing	Comments
R8	172.3 12.7	TCR = **100%** SCR = **98%** RQD = **97%**	4 0 0	mod. close		
R9	170.8 14.2	TCR = **100%** SCR = **92%** RQD = **69%**	4 4 0 2 0	close		**13.6-13.7m:** *Calcite deposits*
R10	169.3 15.7	TCR = **100%** SCR = **92%** RQD = **88%**	1 1 3 1 0	mod. close	**14.3m to 14.5m:** UCS = 146.7 MPa	**14.2-14.3m:** *Calcite deposits*
R11	167.7 17.3	TCR = **100%** SCR = **88%** RQD = **83%**	0 1 1 1 1	mod. close		**16.9-17.1m:** *Vuggy Zone*
R12	166.2 18.8	TCR = **100%** SCR = **97%** RQD = **93%**	0 0 0 3 0	mod. close		**18.0-18.0m:** *Calcite deposits*
R13	164.7 **20.3m**	TCR = **100%** SCR = **100%** RQD = **72%**	1 4 2 3 2	close	**18.8m to 18.9m:** UCS = 114.8 MPa	

END OF COREHOLE

Borehole 1 Core Photos



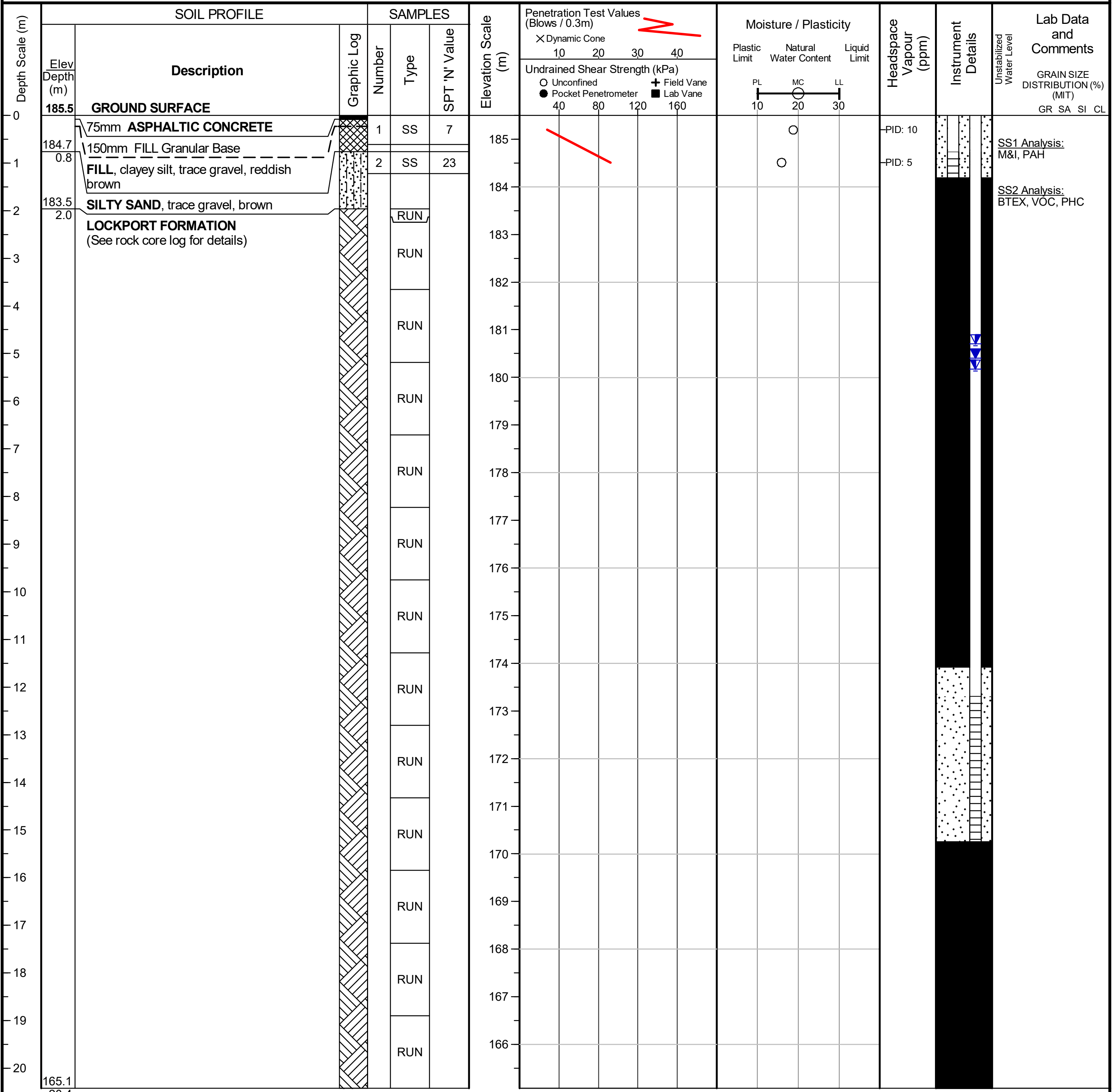




LOG OF BOREHOLE 2

Project No.	: 7-21-0048-01	Client	: Fugiel International Group Inc.	Originated by	: KG
Date started	: June 21, 2021	Project	: 5613 Victoria Avenue	Compiled by	: TW
Sheet No.	: 1 of 1	Location	: Niagara Falls, Ontario	Checked by	: TW

Position : E: 656715, N: 4773007 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Geoprobe
Drilling Method : Solid stem augers, HQ rock coring



SOIL PROFILE		SAMPLES		
Elev Depth (m)	Description	Number	Type	SPT 'N' Value
185.5	**GROUND SURFACE**			
	75mm **ASPHALTIC CONCRETE**	1	SS	7
184.7 0.8	150mm FILL Granular Base **FILL**, clayey silt, trace gravel, reddish brown	2	SS	23
183.5 2.0	**SILTY SAND**, trace gravel, brown **LOCKPORT FORMATION** (See rock core log for details)		RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
			RUN	
165.1 20.4			RUN	

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)

GR SA SI CL

SS1 Analysis:
M&I, PAH

SS2 Analysis:
BTEX, VOC, PHC

END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

W1 WATER LEVELS

Date	Water Depth (m)	Elevation (m)
Jul 7, 2021	dry	n/a
Jul 21, 2021	dry	n/a
Aug 3, 2021	dry	n/a

W2 WATER LEVELS

Date	Water Depth (m)	Elevation (m)
Jul 7, 2021	5.3	180.2
Jul 21, 2021	4.8	180.7
Aug 3, 2021	5.1	180.4

ROCK CORE LOG 2

Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 1 of 2

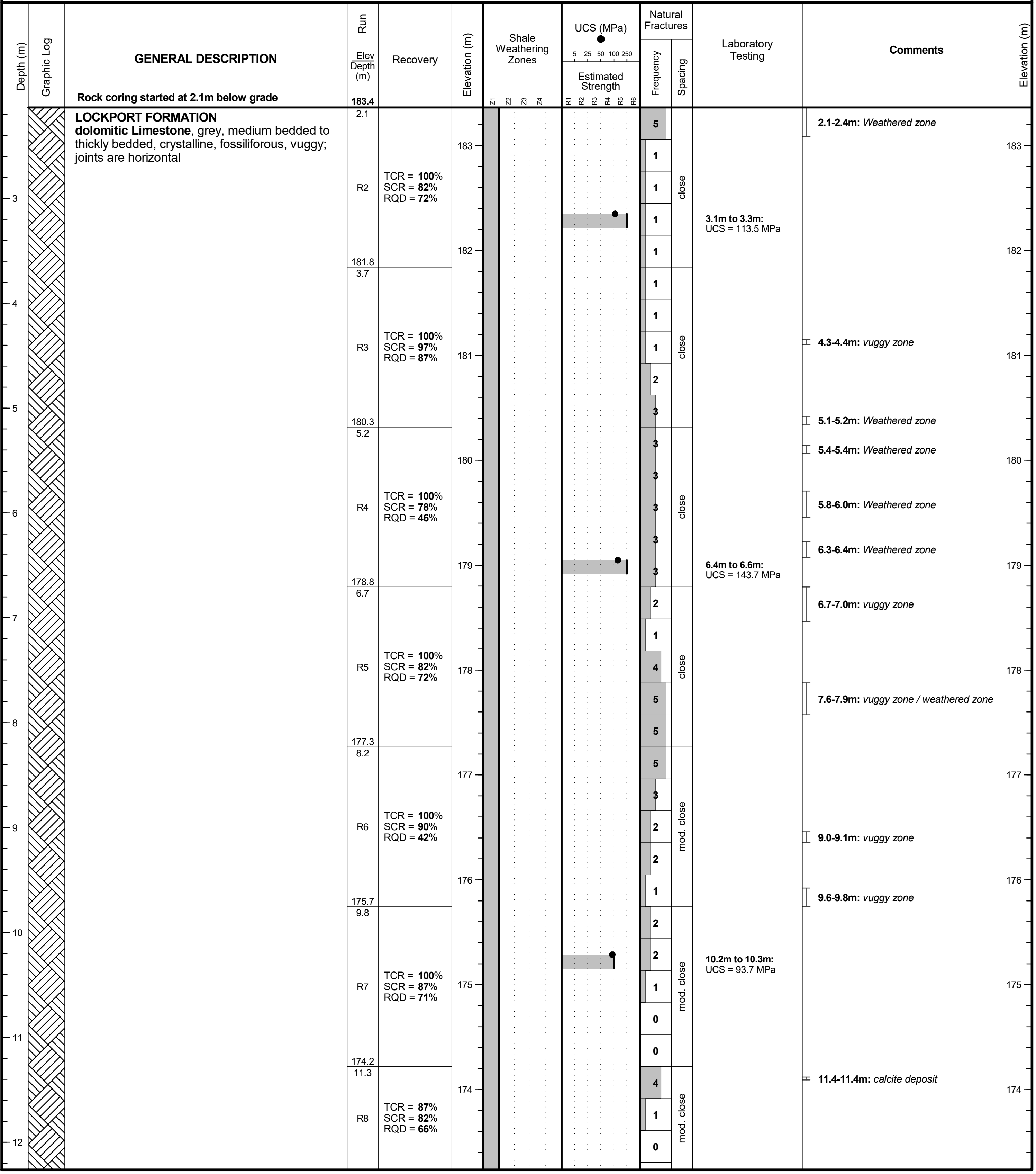
Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

Position : E: 656715, N: 4773007 (UTM 17T)
Rig type : Geoprobe

Elevation Datum : Geodetic
Drilling Method : Solid stem augers, HQ rock coring

Core Diameter : **HQ**, OD=96mm, ID=64mm



GENERAL DESCRIPTION

Rock coring started at 2.1m below grade

LOCKPORT FORMATION
dolomitic Limestone, grey, medium bedded to thickly bedded, crystalline, fossiliforous, vuggy; joints are horizontal

Run	Elev Depth (m)	Recovery
	183.4	
R2	2.1 – 181.8	TCR = **100%** SCR = **82%** RQD = **72%**
R3	3.7 – 180.3	TCR = **100%** SCR = **97%** RQD = **87%**
R4	5.2 – 178.8	TCR = **100%** SCR = **78%** RQD = **46%**
R5	6.7 – 177.3	TCR = **100%** SCR = **82%** RQD = **72%**
R6	8.2 – 175.7	TCR = **100%** SCR = **90%** RQD = **42%**
R7	9.8 – 174.2	TCR = **100%** SCR = **87%** RQD = **71%**
R8	11.3	TCR = **87%** SCR = **82%** RQD = **66%**

Natural Fractures – Frequency: 5, 1, 1, 1, 1, 1, 1, 1, 2, 3, 3, 3, 3, 3, 3, 2, 1, 4, 5, 5, 5, 3, 2, 2, 1, 2, 2, 1, 0, 0, 4, 1, 0

Natural Fractures – Spacing: close, close, close, close, mod. close, mod. close, mod. close

Laboratory Testing

3.1m to 3.3m: UCS = 113.5 MPa

6.4m to 6.6m: UCS = 143.7 MPa

10.2m to 10.3m: UCS = 93.7 MPa

Comments

2.1-2.4m: *Weathered zone*

4.3-4.4m: *vuggy zone*

5.1-5.2m: *Weathered zone*

5.4-5.4m: *Weathered zone*

5.8-6.0m: *Weathered zone*

6.3-6.4m: *Weathered zone*

6.7-7.0m: *vuggy zone*

7.6-7.9m: *vuggy zone / weathered zone*

9.0-9.1m: *vuggy zone*

9.6-9.8m: *vuggy zone*

11.4-11.4m: *calcite deposit*

file: 7-21-0048-01 5613 victoria ave nfalls.gpj

(continued next page)

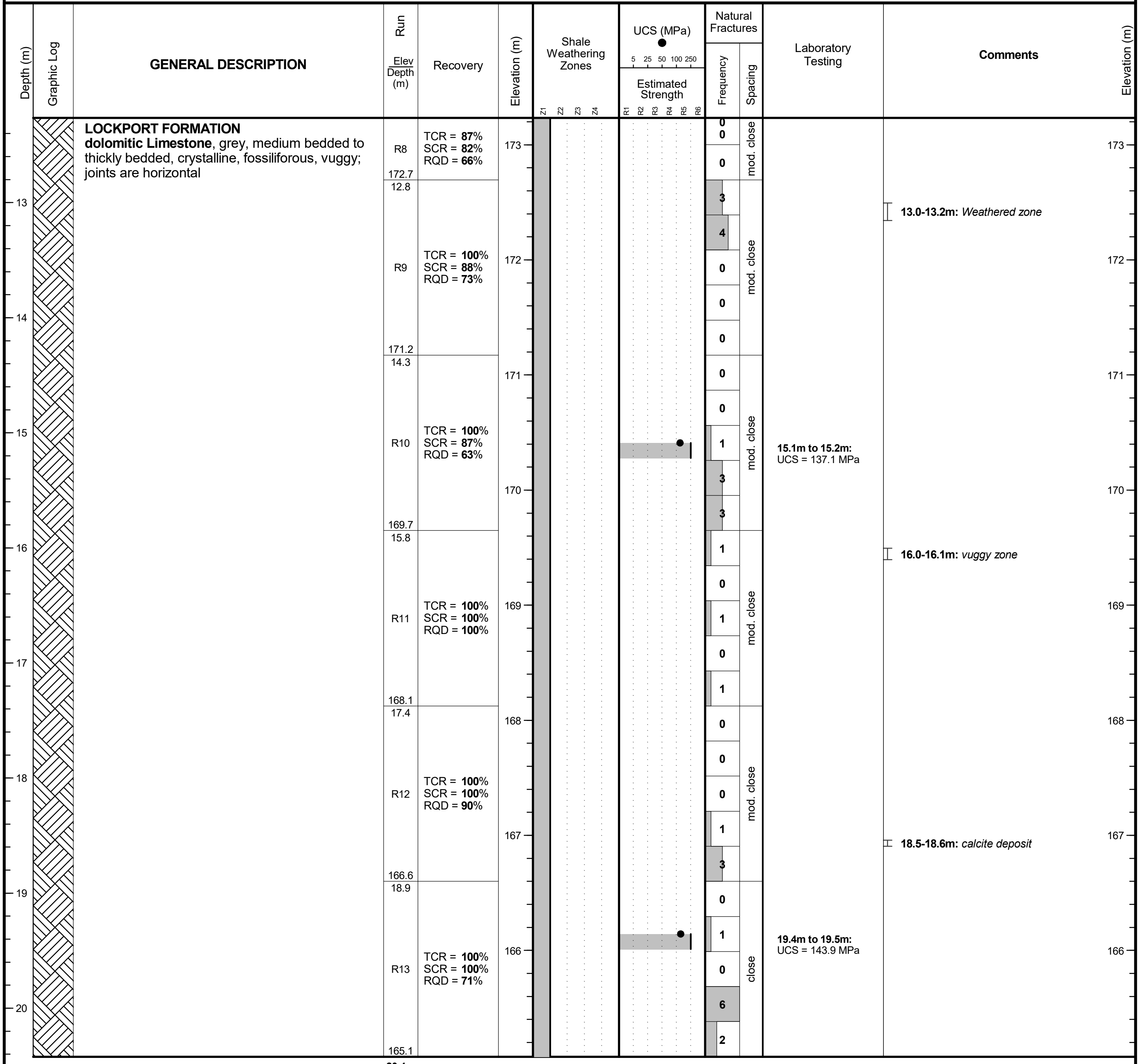
ROCK CORE LOG 2

Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 2 of 2

Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

Position : E: 656715, N: 4773007 (UTM 17T)
Rig type : Geoprobe
Elevation Datum : Geodetic
Drilling Method : Solid stem augers, HQ rock coring
Core Diameter : **HQ**, OD=96mm, ID=64mm



GENERAL DESCRIPTION

LOCKPORT FORMATION
dolomitic Limestone, grey, medium bedded to thickly bedded, crystalline, fossiliforous, vuggy; joints are horizontal

Run	Elev / Depth (m)	Recovery
R8	172.7	TCR = **87%** SCR = **82%** RQD = **66%**
R9	12.8 171.2	TCR = **100%** SCR = **88%** RQD = **73%**
R10	14.3 169.7	TCR = **100%** SCR = **87%** RQD = **63%**
R11	15.8 168.1	TCR = **100%** SCR = **100%** RQD = **100%**
R12	17.4 166.6	TCR = **100%** SCR = **100%** RQD = **90%**
R13	18.9 165.1	TCR = **100%** SCR = **100%** RQD = **71%**

20.4m

END OF COREHOLE

Laboratory Testing

15.1m to 15.2m:
UCS = 137.1 MPa

19.4m to 19.5m:
UCS = 143.9 MPa

Comments

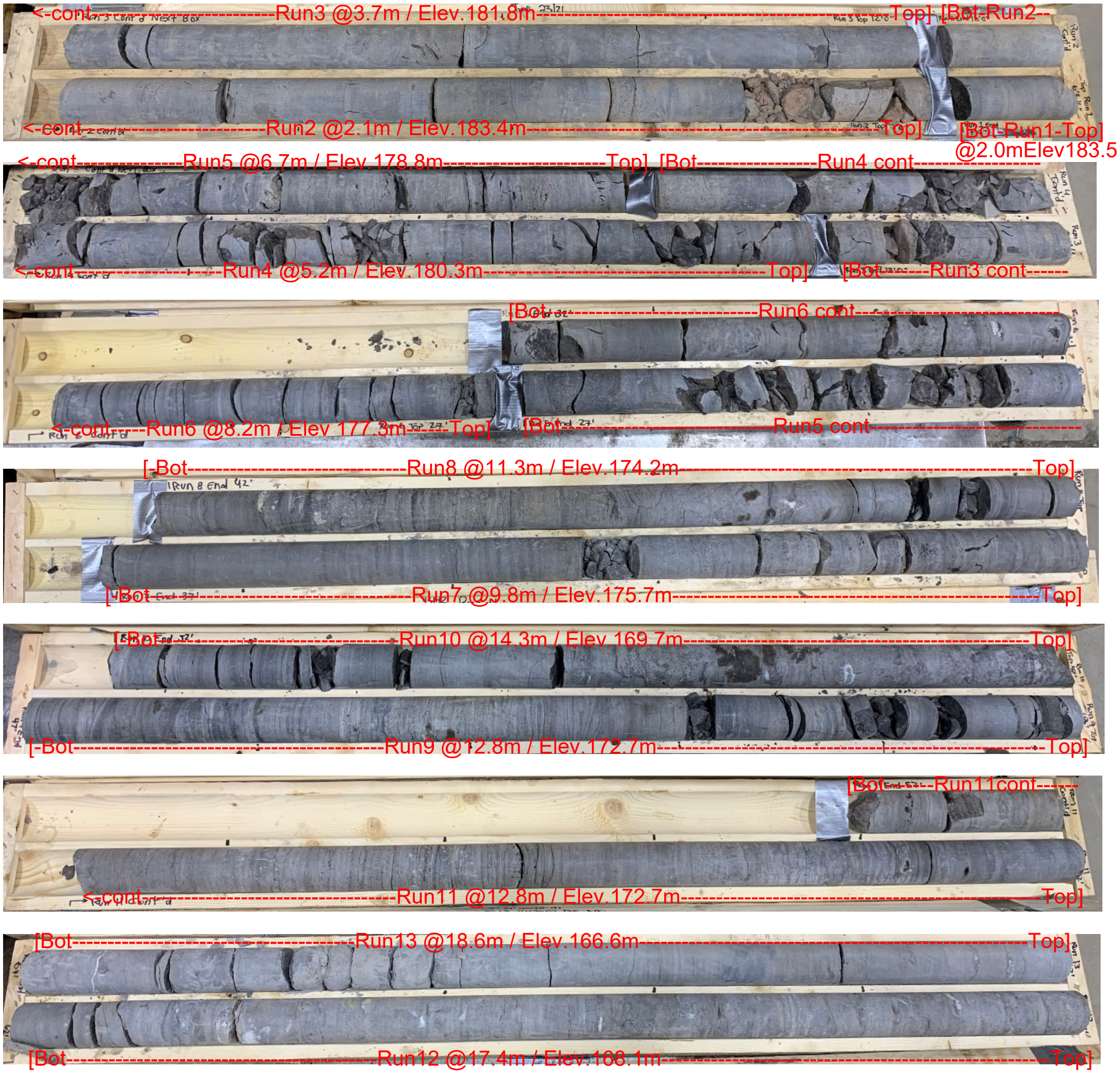
13.0-13.2m: *Weathered zone*

16.0-16.1m: *vuggy zone*

18.5-18.6m: *calcite deposit*

file: 7-21-0048-01 5613 victoria ave nfalls.gpj

Borehole 2 Core Photos



LOG OF BOREHOLE 3

Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 1 of 1

Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

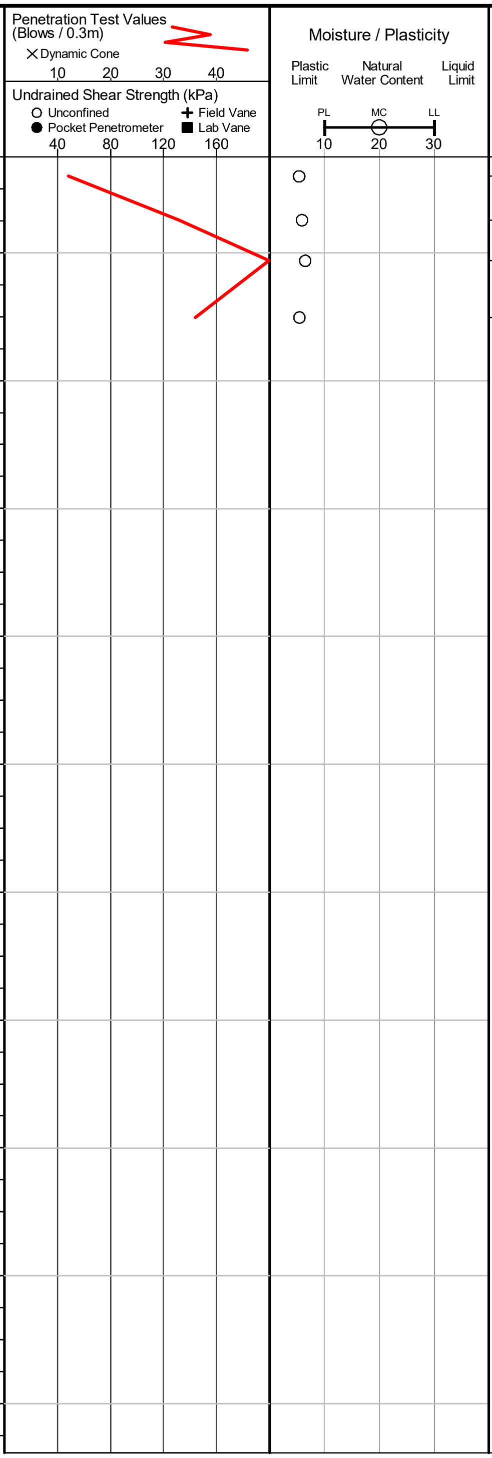
Position : E: 656677, N: 4773045 (UTM 17T)
Rig type : Geoprobe
Elevation Datum : Geodetic
Drilling Method : Solid stem augers, HQ rock coring
Core Diameter : **HQ**, OD=96mm, ID=64mm

SOIL PROFILE

Elev Depth (m)	Description
187.5	**GROUND SURFACE**
	75mm **ASPHALTIC CONCRETE**
	230mm FILL Granular Base
	FILL, clayey silt with sand and gravel, with concrete pieces, brownish red and grey
185.1 2.4	**SILTY SAND**, brown
184.9 2.6	**LOCKPORT FORMATION** (See rock core log for details)
167.2 20.3	

SAMPLES

Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments
1	SS	12	PID: 0	
2	SS	33	PID: 25	
3	SS	50 / 50mm	PID: 20	
4	SS	36	PID: 25	SS4 Analysis: M&I
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			
	RUN			



Depth Scale (m): 0 – 20
Elevation Scale (m): 187 – 168
Instrument Details
Unstabilized Water Level
GRAIN SIZE DISTRIBUTION (%) (MIT)
GR SA SI CL

END OF BOREHOLE
Auger refusal on inferred bedrock

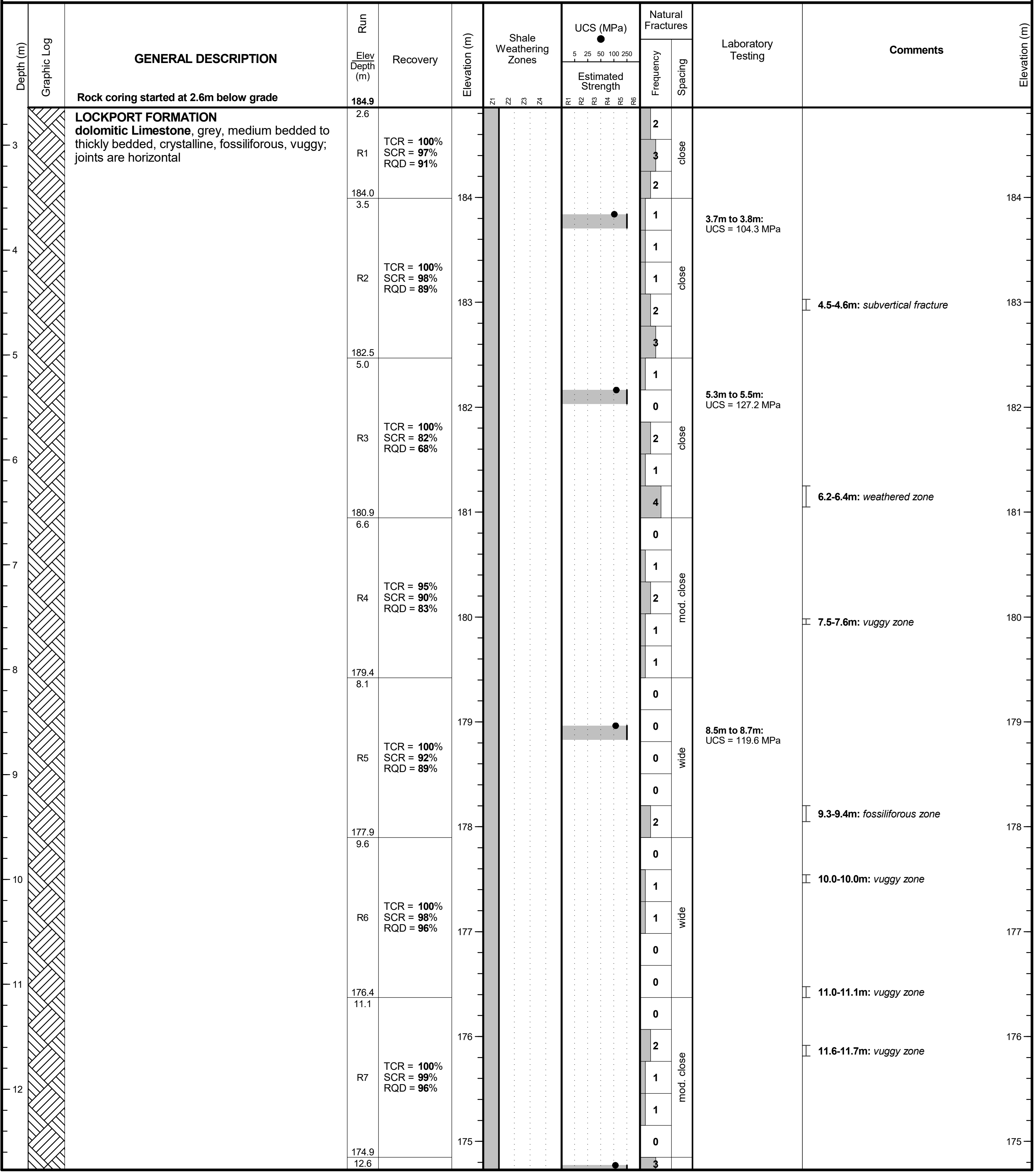
Borehole was dry and open upon completion of drilling.

Terraprobe

ROCK CORE LOG 3

Project No. : 7-21-0048-01
Client : Fugiel International Group Inc.
Originated by : KG
Date started : June 21, 2021
Project : 5613 Victoria Avenue
Compiled by : TW
Sheet No. : 1 of 2
Location : Niagara Falls, Ontario
Checked by : TW

Position : E: 656677, N: 4773045 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Geoprobe
Drilling Method : Solid stem augers, HQ rock coring



GENERAL DESCRIPTION

Rock coring started at 2.6m below grade

LOCKPORT FORMATION
dolomitic Limestone, grey, medium bedded to thickly bedded, crystalline, fossiliforous, vuggy; joints are horizontal

Run	Elev Depth (m)	Recovery	Frequency	Spacing
	184.9			
R1	2.6 – 184.0	TCR = **100%** SCR = **97%** RQD = **91%**	2 3 2	close
R2	3.5 – 182.5	TCR = **100%** SCR = **98%** RQD = **89%**	1 1 1 2 3	close
R3	5.0 – 180.9	TCR = **100%** SCR = **82%** RQD = **68%**	1 0 2 1 4	close
R4	6.6 – 179.4	TCR = **95%** SCR = **90%** RQD = **83%**	0 1 2 1 1	mod. close
R5	8.1 – 177.9	TCR = **100%** SCR = **92%** RQD = **89%**	0 0 0 0 2	wide
R6	9.6 – 176.4	TCR = **100%** SCR = **98%** RQD = **96%**	0 1 1 0 0	wide
R7	11.1 – 174.9	TCR = **100%** SCR = **99%** RQD = **96%**	0 2 1 1 0	mod. close
	12.6		3	

Laboratory Testing

3.7m to 3.8m:
UCS = 104.3 MPa

5.3m to 5.5m:
UCS = 127.2 MPa

8.5m to 8.7m:
UCS = 119.6 MPa

Comments

4.5-4.6m: *subvertical fracture*

6.2-6.4m: *weathered zone*

7.5-7.6m: *vuggy zone*

9.3-9.4m: *fossiliforous zone*

10.0-10.0m: *vuggy zone*

11.0-11.1m: *vuggy zone*

11.6-11.7m: *vuggy zone*

(continued next page)

file: 7-21-0048-01 5613 victoria ave nfalls.gpj

ROCK CORE LOG 3

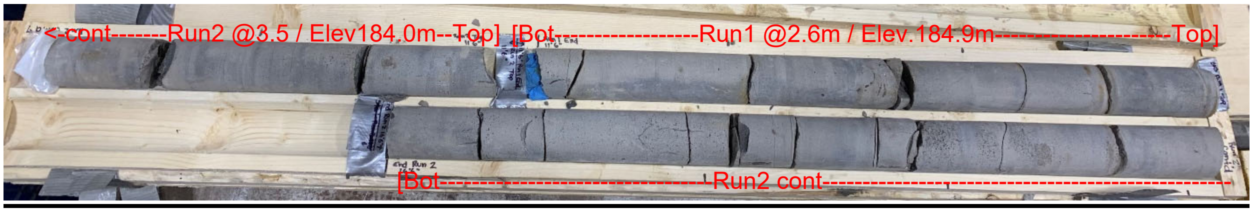
Project No. : 7-21-0048-01
Client : Fugiel International Group Inc.
Originated by : KG
Date started : June 21, 2021
Project : 5613 Victoria Avenue
Compiled by : TW
Sheet No. : 2 of 2
Location : Niagara Falls, Ontario
Checked by : TW

Position : E: 656677, N: 4773045 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Geoprobe
Drilling Method : Solid stem augers, HQ rock coring

Depth (m)	GENERAL DESCRIPTION	Run	Elev Depth (m)	Recovery	Natural Fractures Frequency	Spacing	Laboratory Testing	Comments
12.7–14.2	**LOCKPORT FORMATION** **dolomitic Limestone**, grey, medium bedded to thickly bedded, crystalline, fossiliforous, vuggy; joints are horizontal	R8	173.3 14.2	TCR = **100%** SCR = **100%** RQD = **72%**	3 1 0 1 5	close	**12.7m to 12.9m:** UCS = 116.6 MPa	
14.2–15.7		R9	171.8 15.7	TCR = **100%** SCR = **97%** RQD = **67%**	2 0 2 2 2	close		
15.7–17.2		R10	170.3 17.2	TCR = **100%** SCR = **100%** RQD = **89%**	0 0 0 0 3	mod. close		**16.3-16.6m:** *fossiliforous zone*
17.2–18.7		R11	168.8 18.7	TCR = **100%** SCR = **100%** RQD = **93%**	1 2 1 1 0	mod. close	**17.4m to 17.5m:** UCS = 144 MPa	
18.7–20.3		R12	167.2 **20.3m**	TCR = **92%** SCR = **89%** RQD = **78%**	2 1 1 0 0	mod. close	**19.5m to 19.6m:** UCS = 61.1 MPa	**19.8-19.9m:** *Calcite deposits*

END OF COREHOLE

Borehole 3 Core Photos



LOG OF BOREHOLE 4

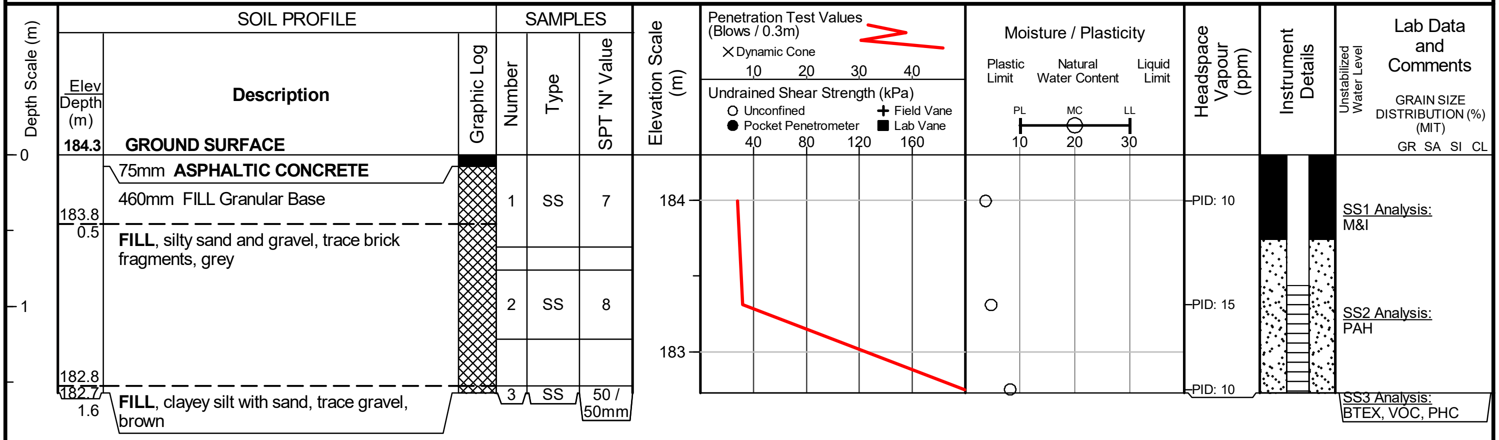
Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 1 of 1

Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

Position : E: 656708, N: 4772970 (UTM 17T)
Rig type : Geoprobe

Elevation Datum : Geodetic
Drilling Method : Solid stem augers



SOIL PROFILE		SAMPLES			Headspace Vapour (ppm)	Lab Data and Comments
Elev Depth (m)	Description	Number	Type	SPT 'N' Value		
184.3	**GROUND SURFACE**					
	75mm **ASPHALTIC CONCRETE** 460mm FILL Granular Base	1	SS	7	PID: 10	SS1 Analysis: M&I
183.8 0.5	**FILL**, silty sand and gravel, trace brick fragments, grey	2	SS	8	PID: 15	SS2 Analysis: PAH
182.8 182.7 1.6	**FILL**, clayey silt with sand, trace gravel, brown	3	SS	50 / 50mm	PID: 10	SS3 Analysis: BTEX, VOC, PHC

END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jul 7, 2021	dry	n/a
Jul 21, 2021	dry	n/a
Aug 3, 2021	dry	n/a

Terraprobe

LOG OF BOREHOLE 5

Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 1 of 1

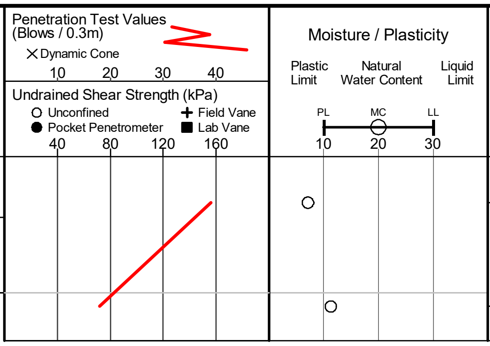
Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

Position : E: 656720, N: 4773026 (UTM 17T)
Rig type : Geoprobe

Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value	Elevation Scale (m)	Headspace Vapour (ppm)	Instrument Details	Unstabilized Water Level	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
0	185.9	**GROUND SURFACE**									
		75mm **ASPHALTIC CONCRETE**									
	185.6 0.3	200mm FILL Granular Base									
		FILL, sandy silt, trace clay, trace gravel, trace brick fragments, brown	▨	1	SS	39		PID: 10			SS1 Analysis: PAH
1	185.1 0.8	**FILL**, clayey silt, trace gravel, reddish brown	▨	2	SS	18	185	PID: 10			SS2 Analysis: M&I, VOC, PHC
	184.7 1.2										



END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

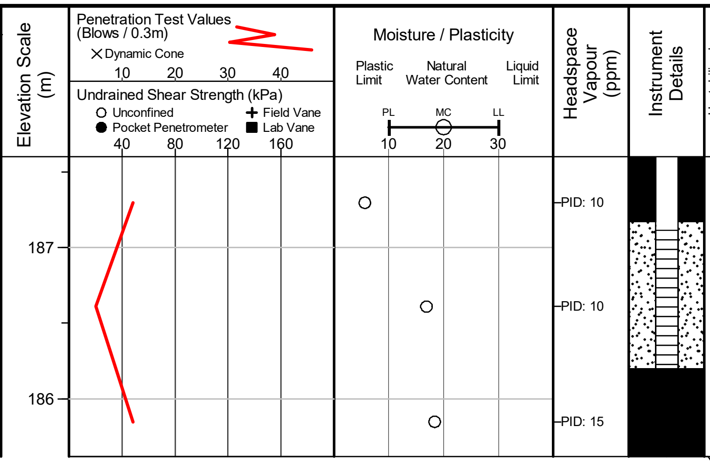
Terraprobe

LOG OF BOREHOLE 6

Project No. : 7-21-0048-01
Client : Fugiel International Group Inc.
Originated by : KG
Date started : June 21, 2021
Project : 5613 Victoria Avenue
Compiled by : TW
Sheet No. : 1 of 1
Location : Niagara Falls, Ontario
Checked by : TW

Position : E: 656689, N: 4773056 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Geoprobe
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	SAMPLES Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments
0	187.6	**GROUND SURFACE**					
	187.4 0.2	75mm **ASPHALTIC CONCRETE**					
		230mm FILL Granular Base	1	SS	12	PID: 10	
		FILL, sand and gravel, grey					
	186.8 0.8	**FILL**, silt, trace clay, gravel, trace glass fragments, reddish brown	2	SS	5	PID: 10	SS2 Analysis: PAH
1							
	186.1 1.5	**FILL**, clayey silt, trace gravel, trace sand	3	SS	12	PID: 15	SS3 Analysis: M&I, BTEX, VOC, PHC
	185.6 2.0	...wet at 1.8m depth					



END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jul 7, 2021	dry	n/a
Jul 21, 2021	dry	n/a
Aug 3, 2021	dry	n/a

file: 7-21-0048-01 5613 victoria ave nfalls.gpj

Terraprobe

LOG OF BOREHOLE 7

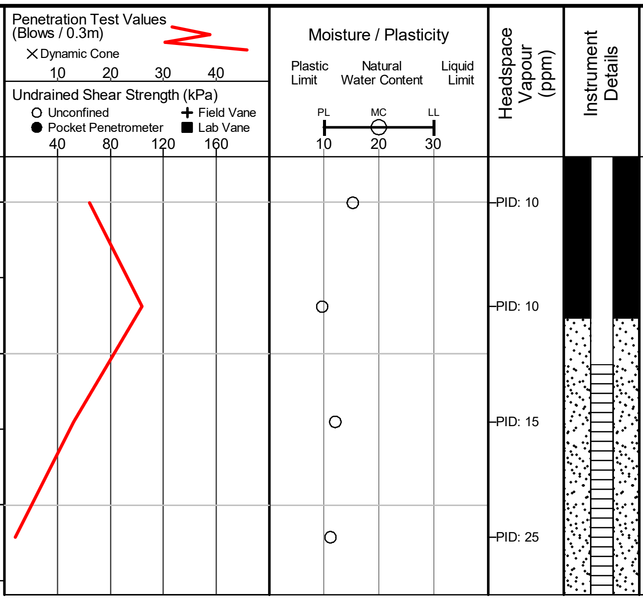
Project No. : 7-21-0048-01
Date started : June 21, 2021
Sheet No. : 1 of 1

Client : Fugiel International Group Inc.
Project : 5613 Victoria Avenue
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : TW
Checked by : TW

Position : E: 656660, N: 4773021 (UTM 17T)
Rig type : Geoprobe
Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	Description	Number	Type	SPT 'N' Value	Elevation Scale (m)	Headspace Vapour (ppm)	Lab Data and Comments
	187.3	**GROUND SURFACE**						
0		75mm **ASPHALTIC CONCRETE**						
	187.0 0.3	230mm FILL Granular Base	1	SS	16	187	PID: 10	
		FILL, silty clay, reddish brown						
1	186.5 0.8	**FILL**, silt, trace clay, trace gravel, faint petroleum hydrocarbon odour, reddish brown	2	SS	26		PID: 10	SS2 Analysis: M&I, BTEX, PAH, VOC, PHC
						186		
			3	SS	13		PID: 15	SS3 Analysis: M&I
2						185		
			4	SS	2		PID: 25	SS4 Analysis: BTEX, VOC, PHC
	184.4 2.9							



END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

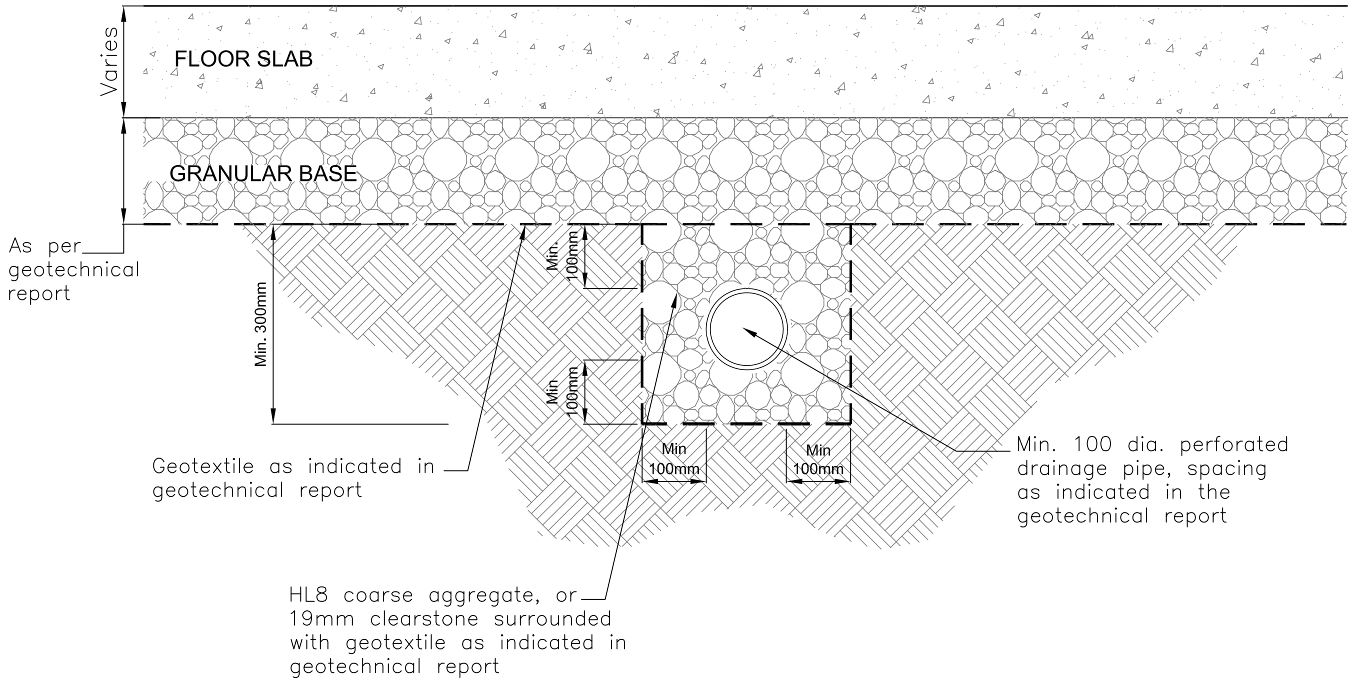
Date	Water Depth (m)	Elevation (m)
Jul 7, 2021	dry	n/a
Jul 21, 2021	dry	n/a

file: 7-21-0048-01 5613 victoria ave nfalls.gpj

BASEMENT DRAINAGE DETAILS

APPENDIX B

Terraprobe Inc.



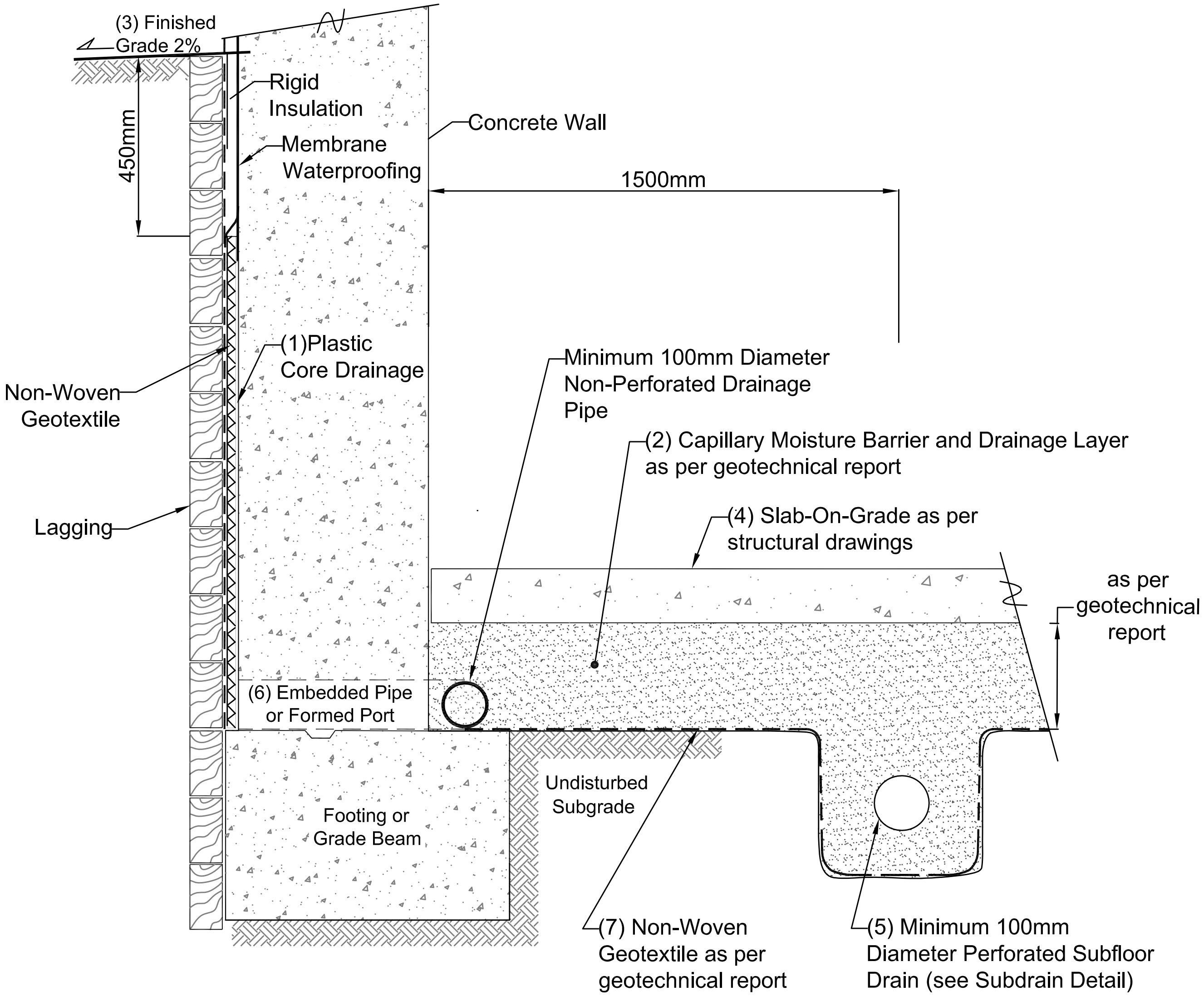
Schematic Only
Not to Scale

Terraprobe

11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

BASEMENT SUBDRAIN DETAIL



NOTES

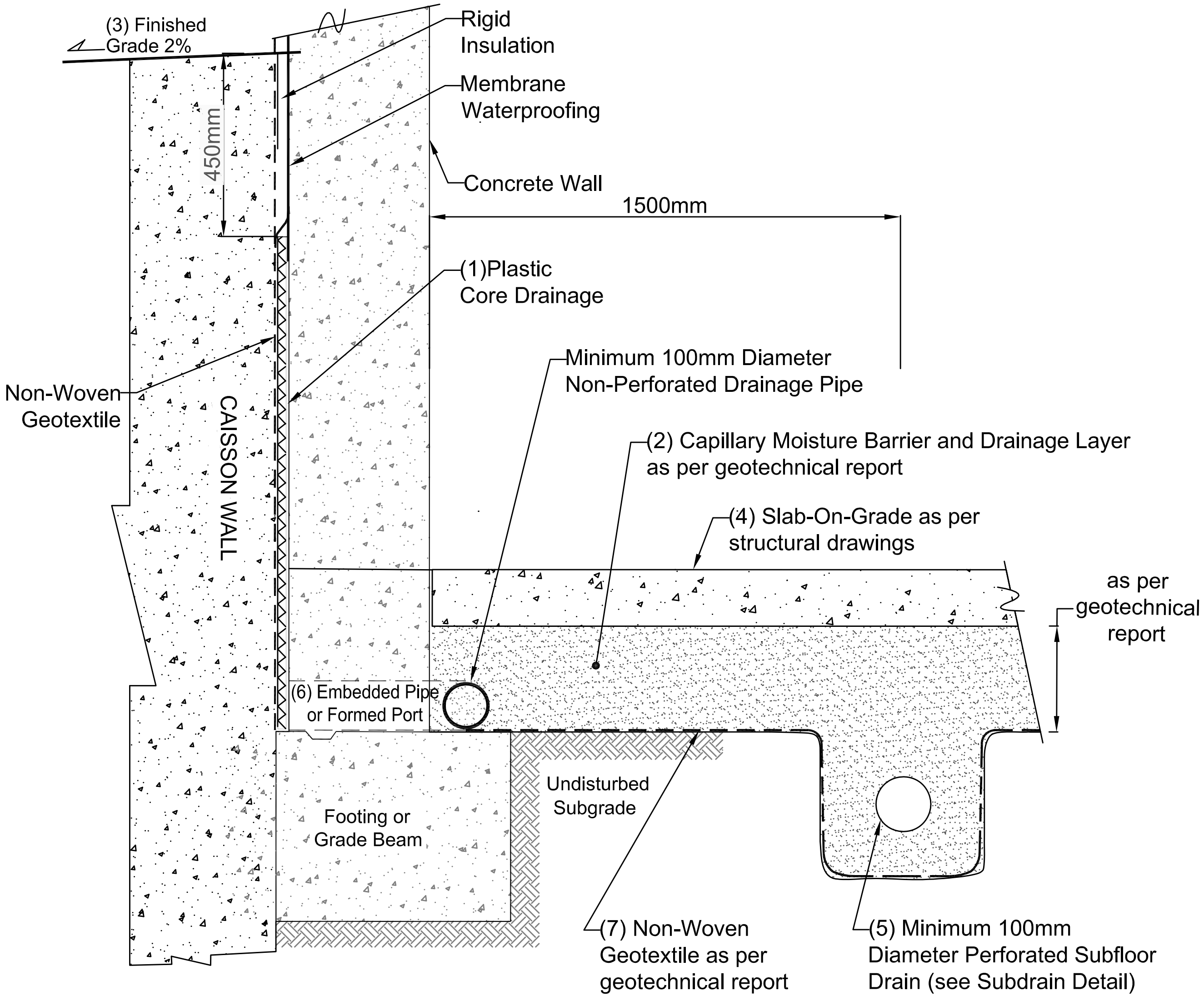
1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.
2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS 1010) compacted to 98% SPMDD where vehicular traffic is required.
3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.
4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.
5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report.
6) Embedded pipes/formed ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in non-perforated pipe.
7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

N.T.S.

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM



NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS 1010) compacted to 98% SPMDD where vehicular traffic is required.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report.

6) Embedded pipes/formed ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in non-perforated pipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

N.T.S.

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
CAISSON WALL SHORING SYSTEM