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PROJECT NO.: SM 301745-G January 11, 2022

ZELJKO HOLDINGS LIMITED
4728 DORCHESTER ROAD – UNIT 11B, 2ND FLOOR
Niagara Falls, Ontario
L2E 7H9

Attention: Jeremia Rudan

GEOTECHNICAL INVESTIGATION
PROPOSED CONDOMINIUM DEVELOPMENT
MAIN AND MURRAY STREET
NIAGARA FALLS, ONTARIO

Dear Mr. Rudan,

Further to your authorisation, SOIL-MAT ENGINEERS has prepared this geotechnical investigation report in connection with the above noted project. The fieldwork, reporting, and laboratory testing were conducted in general accordance with our proposal P301745, dated August 19, 2021. Our comments and recommendations, based on our findings at the borehole locations, are presented in the following paragraphs.

1. INTRODUCTION

We understand that the project will involve the construction of a condominium building approximately 25 storeys in height, with up to three underground parking levels and four podium parking levels, at the property located at Main Street and Murray Street (5614 and 5619 Murray Street and 6285 and 6289 Main Street) in Niagara Falls, Ontario. The purpose of this geotechnical investigation work was to assess the subsurface soil, bedrock, and groundwater conditions, and to provide our comments and recommendations with respect to the design and construction of the foundations for the proposed development, from a geotechnical point of view.

This report is based on the above summarised project description, and on the assumption that the design and construction will be performed in accordance with applicable codes and standards. Any significant deviations from the proposed project design may void the recommendations given in this report. If significant changes are made to the proposed design, this office must be consulted to review the new design with respect to the results of this investigation. It is noted that CEGP CONSULTANTS LTD.

has also conducted Phase One and Two Environmental Site Assessments (ESAs) for the subject site. As such, this report is not intended to provide any comment with respect to the environmental aspects of the site. The reports of which should be forwarded onto our office once they have been completed for our review and documentation.

2. PROCEDURE

The subsurface conditions were assessed in a total of eight [8] sampled boreholes advanced at the locations illustrated in the attached Drawing No. 1, Borehole Location Plan. It is noted that some of these boreholes were advanced as part of the Phase Two Environmental Site Assessment. The boreholes were advanced between September 27 and October 7, 2021 using continuous flight power auger equipment under the direction of a representative of SOIL-MAT ENGINEERS to termination or auger refusal on inferred bedrock at depths of between approximately 1.4 and 30.5 metres below the existing grade.

At two of the borehole locations [Borehole Nos. 3 and 7], approximately 3 to 5 metres of the bedrock was cored using NQ and HQ diamond barrel core equipment, to total depths of approximately 35 and 32 metres, respectively. Selected samples of the recovered core samples were assessed for Rock Quality Designation [RQD], and subjected to unconfined compressive strength testing at our laboratory. The results of this testing have been appended to the end of this report.

Upon completion of drilling, Borehole No. 3 was fitted with a well casing to allow for future shear wave velocity testing. Groundwater monitoring wells were installed at Borehole Nos. 1 and 5 to allow for future measurements of the static groundwater level. The monitoring wells consisted of 50-millimetre diameter PVC pipe, screened in the lower 4.6 to 9.1 metres. The wells were encased in well filter sand up to approximately 0.3 metres above the screened portion, then with bentonite ‘hole plug’ up to the surface and fitted with a protective steel ‘stick up’ casing. The remainder of the boreholes were backfilled in general accordance with Ontario Regulation 903, and the grade reinstated even with the surrounding ground surface.

Representative samples of the subsoils were recovered from the borings at selected depth intervals using split barrel sampling equipment driven in accordance with the requirements of the ASTM test specification D1586, Standard Penetration Resistance Testing. After undergoing a general field examination, the soil samples were preserved and transported to the SOIL-MAT laboratory for visual, tactile, and olfactory classifications. Routine moisture content tests were performed on all soil samples recovered from the borings, with hand penetrometer testing conducted on cohesive samples. In addition, three [3] selected samples were subjected to grain size analyses. The results of this testing have been presented in the attached Grain Size Analysis Nos. 1 to 3, inclusive, appended to the end of this report.

The boreholes were located in the field by representatives of SOIL-MAT ENGINEERS, based on accessibility over the site and clearance of underground services, as well as based on the requirements of the Phase Two ESA. The ground surface elevation at the borehole locations has been referenced to a site-specific temporary benchmark, described as the control monument 0126 in the concrete sidewalk at the northwesterly corner of Main Street and Murray Street, as illustrated on the attached Drawing No. 1. This benchmark was noted to have a geodetic elevation of 204.81 metres as per the Suda and Maleszyk Surveying Inc. topographic survey, File No. 18-153, that was provided to our office.

Details of the conditions encountered in the boreholes are presented in the Log of Borehole Nos. 1 to 8, following the text of this report. It is noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and therefore should not be construed as the exact planes of geological change.

3. SITE DESCRIPTION AND SUBSURFACE CONDITIONS

The subject site is located just west of the intersection of Main Street and Murray Street in Niagara Falls, Ontario and consists of the approximately 70 metre stretch of Murray Street, and the residential properties with Municipal addresses of 5614 and 5619 Murray Street and 6285 and 6289 Main Street. The 70 metre stretch of Murray Street heading west is an asphalt paved roadway noted to be in a generally fair condition, exhibiting some signs of age related distress including occasional longitudinal and transverse cracking. As noted above the four municipal addresses are currently occupied by single family dwellings, fronting either Murray Street or Main Street. These plots of land together form a wedge-shaped parcel west of the intersection of Main Street and Murray Street. The subject site is bordered to the north and east by Main Street, to the south by green space of the Hydro One facility, and to the west by a parking lot. The subject site is relatively flat and even, roughly level with the adjacent roadways.

The subsurface conditions encountered at the borehole locations are summarised as follows:

Topsoil

A surficial veneer of topsoil approximately 150 millimetres in thickness was encountered at Borehole Nos. 4, 5, and 6. It is noted that the depth of topsoil may vary across the site and from the depths encountered at the borehole locations. It is also noted that the term ‘topsoil’ has been used from a geotechnical point of view, and does not necessarily reflect the materials nutrient content or ability to support plant life.

Sand and Gravel Fill

A thin layer of sand and gravel fill was encountered at the surface of Borehole Nos. 3, 7, and 8. The sand and gravel fill was generally noted to be in a compact to dense condition, predominately associated with gravel surfaced shoulders of Murray Street and the driveway areas surrounding the existing dwelling at 6285 Main Street.

Pavement Structure

Borehole Nos. 1 and 2 were advanced through the existing pavement structure along the 70 metre stretch of Murray Street which was noted to consist of approximately 75 to 100 millimetres of asphaltic concrete, overlying approximately 200 millimetres of compact granular base.

Clayey Silt/Silty Clay Fill

Clayey silt/silty clay fill was encountered beneath the granular fill within Borehole Nos. 7 and 8. The fill material encountered was reddish brown to greyish brown in colour, contained trace sand and gravel, and was generally noted to be firm to stiff in consistency. The fill material encountered was proven to depths of up to approximately 3.0 metres below the existing ground surface where encountered. Fill deposits associated with existing buildings and underground infrastructure should be anticipated across the site.

Silt

Native silt was encountered beneath the topsoil, pavement structure, granular fill, or fill material within all boreholes. The native fine grained granular to slightly cohesive soil was reddish brown in colour, contained trace to some clay, trace sand and gravel, and was generally in a dense to very dense condition. The upper levels of the native material was generally noted to have a ‘reworked’ appearance, and may be fill associated construction of the existing roadway and buildings, and having been subjected to ongoing freeze-thaw cycles and traffic loads. The silt was proven to a depth of between approximately 9.5 to 14.7 metres at Borehole Nos. 1, 3, 5, and 7, and to termination at depths of approximately 1.4 to 3.4 metres below the existing ground surface in the remaining boreholes.

Sand

Native sand was encountered beneath the native silt at Borehole Nos. 1, 3, 5, and 7, at depths of approximately 9.4 to 14.7 metres below the existing ground surface. The native granular soils are reddish brown in colour, contained trace to some silt, trace clay and gravel, generally fine to medium in gradation with occasional coarse seams, and was generally in a dense to very dense condition. The sand soil was proven to practical auger or sample spoon refusal on limestone/dolostone bedrock at depths of approximately 29.5 to 30.5 metres below the existing ground surface within Borehole Nos. 3 and 7. Native sand was proven to termination within Borehole Nos. 1 and 5 at depths of between 12.8 to 14.3 metres below the existing ground surface. It is noted that a layer of coarse sand with fine gravel and cobbles was also encountered above the limestone bedrock.

Grain size analyses testing were conducted on selected samples of the native soils recovered from the boreholes. The results of this grain size and Atterberg limits testing can be found appended to the end of this report, and are summarized as follows:

TABLE A – GRAIN SIZE ANALYSES

Sample ID	Depth	% Clay	% Silt	% Sand	% Gravel	Hydraulic Conductivity, k [cm/s]	Estimated Infiltration Rate, [mm/hr]
BH3 SS6	7.6 m	18	78	4	0	10^{-7}	<10
BH3 SS13	21.3 m	3	23	68	6	10^{-4}	50 to 75
BH7 SS12	18.3 m	7	30	61	2	10^{-5}	40 to 50

The field and laboratory testing demonstrate the native soils in the upper levels to consist of a silt with some clay, with traces of sand and gravel, becoming a silty sand/sandy silt with traces of clay and gravel with depth. According to the Unified Soil Classification System (USCS), the soils are classified as M.L. – Inorganic silts, with slight plasticity in the upper levels, becoming S.M. – Silty sands, sand-silt mixtures with depth. The soils in the upper levels would generally behave as a low permeability to an effectively impermeable cohesive material, with occasional more permeable seams. The predominately sand soils present at depth would afford some infiltration, and potentially at rates higher than the conservative estimates noted above. However such higher values, if required, would need to be confirmed through more detailed in-situ testing.

A review of available published information [Quaternary Geology of Ontario, Southern Sheet Map 2556] indicate the subsurface soils to consist of coarse-textured glaciolacustrine deposits of sand and gravel, with minor silt and clay, consistent with our experience in the area, observations during drilling, and from the grain size analyses.

Limestone/Dolostone Bedrock

Limestone/dolostone bedrock was encountered at depths of approximately 29.5 to 30.5 metres below the existing grade at Borehole Nos. 3 and 7, respectively. The depths and elevations at which bedrock was encountered have been summarised as follows:

TABLE B – SUMMARY OF BEDROCK DEPTHS

Borehole No.	Ground Surface Elevation	Bedrock Depth	Bedrock Elevation
3	204.39 m	30.5 m	173.8 m
7	205.72 m	29.5 m	176.3 m

The bedrock was cored at Borehole Nos. 3 and 7. The bedrock cores were noted to yield recoveries of between approximately 82 to 100 per cent, with a Rock Quality Designation [RQD] of approximately 53 to 100 per cent, and generally 84 to 100 per cent, indicating a relatively fair to excellent quality bedrock, however, a horizontal fissure was encountered at a depth of approximately 32.3 metres in Borehole No. 7, roughly 2.8 metres into the bedrock. This is consistent with our experience on other nearby projects, having encountered a similar horizontal fissure within the bedrock. Unconfined compressive strength testing on selected portions of the recovered core samples yielded compressive strengths of approximately 13.3 to 76.6 MPa, with an average of 46.4 MPa. The results of this testing can be found appended to the end of this report, and have been summarised as follows:

TABLE C – SUMMARY OF BEDROCK CORING

Borehole No. 3

Depth of Core (m)	Elevation of Core (m)	Recovery	Rock Quality Designation (RQD)	Depth, Elevation of Tested Core Sample	Unconfined Compressive Strength
30.9 to 31.0	173.49 to 173.39	93%	84%	31.0 m, 173.4 m	69.8 MPa
31.7 to 31.8	172.69 to 172.59	93%	84%	31.8 m, 172.6 m	56.9 MPa
32.4 to 32.6	171.99 to 171.79	100%	94%	32.5 m, 171.8 m	76.6 MPa
33.3 to 33.4	171.09 to 170.99	100%	94%	33.4 m, 171.0 m	25.3 MPa

Borehole No. 7

Depth of Core (m)	Elevation of Core (m)	Recovery	Rock Quality Designation (RQD)	Depth, Elevation of Tested Core Sample	Unconfined Compressive Strength
30.6 to 30.7	175.12 to 175.02	82%	53%	30.7 m, 175.0 m	27.1 MPa
30.7 to 30.8	175.02 to 174.92	82%	53%	30.8 m, 174.9 m	69.1 MPa
31.9 to 32.0	173.82 to 173.72	100%	100%	32.0 m, 173.7 m	13.3 MPa
32.3 to 32.4	173.42 to 173.32	100%	100%	32.4 m, 173.3 m	32.9 MPa

Based on a review of the recovered core samples, as well as available published literature and past experience in the area, the bedrock consists of limestone and dolostone of the Guelph formation. The limestone/dolostone bedrock is grey, generally fractured and weathered in the upper levels, with occasional vugs and solution cavities. The bedrock is considered competent in terms of the foundation/excavation requirements for the proposed project, although occasional fissures and/or solution cavities have historically been encountered.

As noted above, the bedrock in the area has historically been noted to contain horizontal fissures or fractures, as experienced in Borehole No. 7, indicated by significant wash water loss during coring activities approximately 2.8 metres into be bedrock, at elevation of approximately 173.4 metres. Furthermore, upon visual inspection of the core run within Borehole No. 3 (elevation of 172.3 to 170.9 metres), a thin vertical fissure was also noted. As such, based on the presence of both vertical and horizontal fissures encountered, in the event that the bedrock is to be utilised for foundations such as caissons, further assessment of the bedrock is strongly recommended to confirm the condition of the upper levels of the bedrock, and the extent of the such fissures.

Groundwater Observations

All of the shallow boreholes were recorded as open and 'dry', while the deeper boreholes were 'wet' at varying depths due to the nature of mud rotary drilling operations upon completion. It is noted that insufficient time would have passed for the static groundwater level to stabilise in the open boreholes. As noted above, Borehole Nos. 1 and 5 were fitted with groundwater monitoring wells to allow for future monitoring of the groundwater levels. The monitoring wells were equipped with data loggers for continuous groundwater level readings between October 7, 2021 to October 28, 2021. Manual monitoring well readings were taken from all of the installed monitoring well locations across the site on November 3, 2021, which indicated the wells to be 'dry' and have been summarized as follows:

TABLE D: SUMMARY OF GROUNDWATER LEVELS

Monitoring Well No.	Monitoring Well Depth [m]	Ground Surface Elevation [m]	November 3, 2021	
			Groundwater Depth [m]	Groundwater Elevation [m]
1	12.2	202.51	>12.2	<190.3
5	13.7	205.86	>13.7	<192.2

The groundwater readings summarised above as well as the groundwater data from the data loggers indicate groundwater greater than depths of 14 metres. Our experience in the area indicates groundwater levels as deep as 15 to 20 metres. Regardless, shallow aquifers and perched groundwater conditions may be encountered, especially during the 'wet' times of the year. Such shallower perched deposits would be relatively limited, and may result in initial greater flows of infiltration, however would not be considered to result in significant infiltration into open excavations.

4. FOUNDATION CONSIDERATIONS

SHALLOW FOUNDATIONS

It is understood that the project will involve the construction of a high-rise development with multiple underground levels extending to depths of up to approximately 9 to 12 metres below the existing ground surface. The native silt and sand soils present at the anticipated founding depth of 10 to 12 metres would be considered capable of supporting bearing capacities of up to 600 kPa [~12,000 psf] SLS and 900 kPa [~18,000 psf] ULS for the use of design of spread footings. Where spread footings using the above bearing capacities would result in a spread footing coverage of more than 50 per cent, consideration should be given to a raft slab to support the proposed building, utilising the above bearing capacities. If a flexible design approach is used, a conservative value of subgrade modulus of k = 70 MN/m^3 [~275 pci] may be considered. It is noted that more detailed investigations such as cone penetration testing may allow for refinement of these values, if required.

CAISSONS

In the event that spread footings or a raft slab are not feasible for the support of the proposed structure, deep foundations extending to the limestone/dolostone bedrock may be required to support the proposed high-rise structure. Caissons extending into the upper levels of the fractured and weathered limestone/dolostone bedrock encountered at depths of approximately 29.5 to 30.5 metres below the existing grade, may be utilised, however the presence of the horizontal fissure encountered should be taken into consideration. It is recommended that caissons be limited to a socket depth of no more than 0.5 metres into the bedrock, and be designed considering a conservative SLS and ULS bearing capacity of 1,500 kPa [~30, 000 psf]. Alternatively, caissons should be socketed to a minimum of 3 metres into the bedrock, beyond the horizontal fissure encountered. Where caissons are socketed below the depth of the horizontal fissure, they may be designed considering SLS and ULS bearing capacities of 3,000 kPa [~60,000 psf], based on the results of the unconfined compressive strength testing of the recovered bedrock core samples, pending a more detailed assessment of the bedrock. Higher bearing capacities may be available within the competent limestone/dolostone bedrock below the horizontal fissure, however confirmation of such bearing capacities would require additional coring of the bedrock. Where caissons are socketed into the competent bedrock, a minimum of 3.0 metre into the bedrock below the upper levels of highly weathered and fractured limestone/dolostone bedrock, skin friction may also be considered, using a value of 350 kPa [~7,000 psf]. As mentioned above, if deep foundations are required to extend to the limestone bedrock, it is strongly recommended that further coring investigations be undertaken in order to determine the extent of the fissures encountered.

Caisson excavations should be provided a steel liner to maintain the integrity of the open hole and limit or prevent the infiltration of water. The contractor should be prepared to provide such a steel liner over the entire length of the caisson. Some dewatering of open caisson excavations should be anticipated to be required, and given the relatively permeable nature of the sand soils above the rock and fractured condition of the rock surface it may not be possible to ‘seal’ off the liner to allow for dewatering of the open caissons. As such, the contractor should be prepared to place concrete by means of a ‘tremmie’ pipe method. The contractor should maintain a positive head of concrete in the liner while it is being removed to avoid the intrusion of loose materials [known as 'necking'] into the caisson. The base of the caissons should be thoroughly cleaned to remove all loose or disturbed material immediately prior to the placement of concrete. The installation of caissons should be monitored by a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD.

GENERAL FOUNDATION COMMENTS

It is noted that the SLS value represents the Serviceability Limit State, which is governed by the tolerable deflection [settlement] based on the proposed building type, using unfactored load combinations. The ULS value represents the Ultimate Limit State and is intended to reflect an upper limit of the available bearing capacity of the founding soils in terms of geotechnical design, using factored load combinations. There is no direct relationship between ULS and SLS; rather they are a function of the soil type and the tolerable deflections for serviceability, respectively. Evidently, the bearing capacity values would be lower for very settlement sensitive structure and larger for more flexible buildings. It is also noted that the SLS and ULS bearing capacities are equivalent for the limestone bedrock, as in order for serviceability limits to be realised, ultimate failure of the bedrock would have to occur.

All footings, caisson caps, grade beams, etc., exposed to the environment must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation to protect against frost damage. This frost protection would also be required if construction were undertaken during the winter months. All footings and foundations should be designed and constructed in accordance with the current Ontario Building Code.

With foundations designed as outlined above and as required by the Building Code, and with careful attention paid to construction detail, total and differential settlements should be small, and certainly well within normally tolerated limits of 25 and 20 millimetres, respectively, for the type of building and occupancy expected.

It is noted that the performance of deep foundation schemes is greatly dependent on the method, equipment, and workmanship utilized during construction. It is therefore essential that installation procedures for the deep foundations be monitored/evaluated by SOIL-MAT ENGINEERS.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this report and to allow changes to be made in the event that subsurface conditions differ from the conditions identified at the borehole locations.

It is recommended that our office be consulted during the detailed design stage of the foundations for various structures and given an opportunity to review the foundation design scheme to ensure it is consistent with the recommendations of this report.

5. LATERAL EARTH PRESSURE

The design of underground basement foundation walls, shoring, etc. should take into consideration the different parameters for the various soil conditions encountered, which have been summarised as follows:

TABLE E – SOIL PARAMETERS

Soil Strata	Depth	Unit weight γ	Friction angle ϕ	Cohesion c	ko	k_A	k_P
Silt	0 to 15 m	18.0 kN/m^3	34	0	0.44	0.28	3.5
Sand	15 to 31 m	19.0 kN/m^3	38	0	0.38	0.24	4.2

6. SEISMIC DESIGN CONSIDERATIONS

The structure shall be designed according to Section 4.1.8 of the Ontario Building Code, Ontario Regulation 332/12. As noted above, Borehole No. 3 was instrumented to allow for downhole shear wave velocity testing . The testing was conducted by Geophysics GPR and the results of which reported under a separate cover and appended to this report [GPR file T213541, dated January 6, 2022]. Based on the conditions encountered at the borehole locations, and the shear wave testing, the applicable seismic site class is C – Dense Soil to Soft Rock.

The seismic data from Supplementary Standard SB-1 of the Ontario Building Code for Niagara Falls are as follows:

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(5.0)$	$S_a(10.0)$	PGA	PGV
0.321	0.157	0.072	0.0320	0.0076	0.0030	0.207	0.121

7. EXCAVATIONS AND EXCAVATION SUPPORT CONSIDERATIONS

Excavations for the installation of foundations and underground services are expected to extend to depths of up to about 10 to 12 metres below the existing grade, in order to accommodate the up to 3 underground parking levels. Open excavations through the slightly cohesive silt soils above the groundwater level should be relatively straightforward, with the sides remaining stable for the short construction period at 45 to 60 degrees to the horizontal, while excavations through the fine-grained granular sand soils and fill materials would be expected to remain stable of up to angles of 45 degrees to the horizontal. During periods of heavy precipitation, or where perched wet seams are encountered, the excavation sides should be expected to 'slough-in' to as flat as 3 horizontal to 1 vertical, or flatter. All excavations must comply with the current Occupational Health and Safety Act and Regulations for Construction Projects.

Depending on the final design and limits of the new building, excavation support measures are expected to be required in order to maintain the stability of the excavations in proximity to adjacent roadways and structures. A speciality contractor or shoring consultant should be consulted with respect to the design of such a shoring system. For excavations above the static groundwater level typical soldier pile and timber lagging systems would likely be most effective. However, it is recommended that a filter fabric be incorporated behind the lagging boards to prevent the loss of finer silt and sand particles. The soil parameters outlined above should be considered in the design of shoring systems, such as soldier piles and timber lagging or caisson wall. Caissons may be designed for end bearing using the values provided above for the limestone bedrock or overburden soils. The shoring system should be monitored during construction, and the contractor should have a contingency plan in place to be implemented should deflections of the shoring system exceed the tolerable limits.

The shoring can be supported either by anchors extending into the overburden soil or by rakers extending into the excavation, although from a contractor's point of view, tie-back anchors would be preferred, provided they can be installed to avoid adjacent foundations and underground utilities. The shoring must be monitored for movements, and a plan must be available before the excavations begin, to rectify the shoring system should movements become apparent. It is noted that significant movements of the shoring system may take place if conventional rakers are used to support the shoring system since compression of the members and the supporting footings must occur before the rakers can begin to carry load. In this regard, anchors are preferred since they allow pre-stressing of the shoring system to the design load even before the excavations reach their final grade. Alternatively, the rakers can be designed to allow jacking in the design load, and thus minimising movements. The design bearing capacity of the footings supporting inclined rakers should be limited to one half of the bearing capacities presented above for spread footing foundations.

It is anticipated that the excavation for the structure will extend into the native sand. The base of the excavation in the compact to dense sand soils should remain firm and stable, however may be prone to localized disturbance from construction traffic, moisture conditions, etc. It is recommended that the excavation base be provided with a layer of granular material, such as Ontario Provincial Standard Specification [OPSS] Granular ‘B’, Type II [crushed bedrock], perhaps 200 to 300 millimetres thick and compacted to 100 per cent of its standard Proctor maximum dry density [SPMDD] to provide a stable ‘clean’ working surface.

Regardless, stabilisation of the excavation bases may be necessary, depending on the depth of excavation, the depth of the shoring system, dewatering methodologies employed and time of year of construction. In particular, excavation bases within the silt soils will be more prone to disturbance from construction traffic and weather, while excavation bases within the sand deposit would be less prone to disturbance. Consideration should be given to the provision of a layer of course crushed granular material approximately 300 millimetres thick should provide for a sufficiently stable working surface. This would provide for a clean and stable working surface for construction of forms and reinforcing steel placement. Alternatively, placement of a thin, ‘lean-mix] [~5 MPa] ‘mud slab’, over the base of the excavation would provide for a clean work area.

As noted above, the groundwater level is estimated to be below the depths of construction. Regardless, as noted above shallower perched groundwater deposits in the more permeable seams may be encountered. As such, some infiltration of groundwater from permeable seams into open excavations as well as from surface runoff should be anticipated. It should be possible to control groundwater infiltration via typical construction dewatering techniques such as pumping from sumps and ditches in the base of the excavation. Where such perched water deposits are encountered, a greater initial rate of infiltration should be expected, however would be expected to exfiltrate from the base of the excavation within the more permeable sandy soils, and/or be readily handled with a series of dewatering pumps.

8. FLOOR SLAB AND PERMANENT DRAINAGE

Where a raft slab foundation is not utilised, the building floor slabs may be constructed using conventional slab-on-grade techniques on a prepared subgrade. The exposed subgrade surface should be well compacted in the presence of a representative of SOIL-MAT ENGINEERS. Any soft 'spots' delineated during this work must be sub-excavated and replaced with quality backfill material compacted to 100 per cent of its standard Proctor maximum dry density. The subgrade level can then be raised to the design level with granular soils compacted to 100 per cent of its standard Proctor maximum dry density. Granular fill, such as an imported Ontario Provincial Standard Specification [OPSS] Granular 'B', Type II (crushed limestone bedrock) product, is preferred within the building footprint due to its relative insensitivity to weather conditions, ease in achieving the required degree of compaction, and its quick response to applied stresses.

As with all concrete floor slabs, there is a tendency for the floor slabs to crack. The slab thickness, concrete mix design, the amount of steel and/or fibre reinforcement and/or wire mesh placed into the concrete slab, if any, will therefore be a function of the owner's tolerance for cracks in, and movements of, the slabs-on-grade, etc. The 'saw-cuts' in the concrete floors, for crack control, should extend to a minimum depth of 1/3 of the thickness of the slab.

A moisture barrier will be required under the floor slabs such as the placement of at least 200 millimetres of compacted 20-millimetre clear crushed stone. At a minimum the moisture barrier material should contain no more than 10 per cent passing the No. 4 sieve. Where 'non-damp' floor slabs are required, as for instance under sheet vinyl floor coverings, etc., extra efforts will be required to damp proof the floor slab, as with the additional provision of a heavy 'poly' sheet, damp proofing sprays/membranes, drainage board products, etc. Where 'poly' sheets are used care should be taken to prevent puncturing and tearing and a sufficiently heavy gauge material be provided.

Curing of the slab-on-grade must be carefully specified to ensure that slab curl is minimised. This is especially critical during the hot summer months of the year when the surface of the slab tends to dry out quickly while high moisture conditions in the moisture barrier or water trapped on top of any 'poly' sheet at the saw cut joints and cracks, and at the edges of the slabs, maintains the underside of the slab in a moist condition.

It is important that the concrete mix design provide a limiting water/cement ratio and total cement content, which will mitigate moisture related problems with low permeance floor coverings, such as debonding of vinyl and ceramic tile. It is equally important that excess free water not be added to the concrete during its placement as this could increase the potential for shrinkage cracking and curling of the slab.

All basement foundation walls should be suitably damp proofed, including the provision of a 'dimple type' drainage board to promote rapid drainage to a perimeter drainage system. This may require the use of foundation wall systems intended for 'blind side' or 'single face' application against the excavation shoring. The perimeter drainage system should consist of 100-millimetre diameter perforated pipe, encased in a geofabric sock and covered with a minimum of 200 millimetres of a 20-millimetre clear crushed stone product, and the clear crushed stone in turn encased by a heavy filter geotextile product. The suppliers of the filter geotextile should be consulted as to the type best suited for this project. This office should examine the installation of the drains. Even a small break in the filtering materials could result in loss of fines into the drains with attendant performance difficulties, including settlements of the ground surface. The perimeter drains should outlet to a sump pit or retention tank a minimum of 150 millimetres below the underside of finished floor. The exterior grade around the structure should be sloped away from the structure to prevent the ponding of water against the foundation walls. The enclosed Drawing No. 2 shows schematics of the typical requirements for foundation construction with a basement level.

9. BACKFILL CONSIDERATIONS

The majority of excavated material will consist of the native silt, sand, and fill deposits encountered in the boreholes, as described above. These materials are generally considered suitable for use as engineered fill, service trench backfill, etc., provided the moisture content can be controlled to within 3 per cent of the material's standard Proctor optimum value, and the material is free of organics, construction debris, and otherwise deleterious materials. It is recommended that the fine-grained granular silt soils be separated from any predominately sand soils during excavation, to maintain the somewhat granular nature of the sandier soils, which would be more suitable for use in areas of restricted access. Depending on the weather conditions at the time of construction, some moisture condition of the excavated materials may be required to achieve acceptable compaction densities and minimise long-term settlement. Compaction of the cohesive clayey silt/silty clay soils will prove to be difficult in areas where access with compaction equipment is restricted.

It is noted that the silt soils encountered are not considered to be free draining, as demonstrated by the grain size analysis testing, and should not be used where this characteristic is necessary. The sand soils are generally considered to be well draining, however may be affected by variation in the silt content, and may present some difficulty in achieving effective compaction in areas of restricted access. Where this characteristic is required, the use of a free draining, well-graded granular material, such as an Ontario Provincial Standard Specification [OPSS] Granular 'B', Type II (crushed limestone bedrock) is recommended for backfill against foundation walls or to raise the interior grade to the design subgrade level. This material is more readily compacted in restricted access areas, and generally presents a more positive support condition for concrete floor slabs and exterior pavement. As noted above the sand soils encountered at depth would be suitable for this type of application, however this would be best assessed at the time of construction.

It is very important that the placement moisture content of the backfill soils be within 3 per cent of its standard Proctor optimum moisture content during placement and compaction to minimise long term subsidence [settlement] of the fill mass. Any imported fill required in service trenches or to raise the subgrade elevation should have its moisture content within 3 per cent of its optimum moisture content and meet the necessary environmental guidelines.

A representative of SOIL-MAT should be present on-site during the backfilling and compaction operations to confirm the provision of uniform compaction of the backfill material to project specification requirements. Close supervision is prudent in areas that are not readily accessible to compaction equipment, for instance near the end of compaction 'runs'. All structural fill should be compacted to 100 per cent of its standard Proctor maximum dry density [SPMDD]. Backfill within service trenches, areas to be paved, etc., should be compacted to a minimum of 98 per cent of SPMDD. The appropriate compaction equipment should be employed based on soil type, i.e. pad-toe for cohesive soils and smooth drum/vibratory plate for granular soils. A method should be developed to assess compaction efficiency employing the on-site compaction equipment and backfill materials during construction.

10. PAVEMENT DESIGN CONSIDERATIONS

All areas to be paved should be stripped of all organic or otherwise unsuitable materials. The exposed subgrade should be proofrolled with 3 to 4 passes of a loaded tandem truck in the presence of a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD., immediately prior to the placement of the sub-base material. Any areas of distress revealed by this or other means must be sub-excavated and replaced with suitable backfill material. Alternatively, the soft areas may be stabilised by placing coarse crushed stone and ‘punching’ it into the soft areas. Where the subgrade condition is poorer it may be necessary to implement more aggressive stabilisation methods, such as the use of coarse aggregate [50-millimetre clear stone, ‘rip rap’, etc.] ‘punched’ into the soft areas. The need for the treatment of softened subgrade will be reduced if construction is undertaken during the dry summer months and careful attention is paid to the compaction operations. The fill over shallow utilities cut into or across paved areas such as telephone, hydro, gas, etc. must also be compacted to 100 per cent of its standard Proctor maximum dry density.

Good drainage provisions will optimise the long-term performance of the pavement structure. The subgrade must be properly crowned and shaped to promote drainage to the subdrain system. Subdrains should be installed to intercept excess subsurface water and mitigate softening of the subgrade material. Surface water should not be allowed to pond adjacent to the outer limits of the paved areas.

The most severe loading conditions on the subgrade typically occur during the course of construction, therefore precautionary measures may have to be taken to ensure that the subgrade is not unduly disturbed by construction traffic. SOIL-MAT should be given the opportunity to review the final pavement structure design and subdrain scheme prior to construction to ensure that they are consistent with the recommendations of this report.

If construction is conducted under adverse weather conditions, additional subgrade preparation may be required. During wet weather conditions, such as during the Fall and Spring months, or during colder winter weather, it should be anticipated that additional subgrade preparation will be required, such as additional depth of Ontario Provincial Standard Specification [OPSS] Granular ‘B’, Type II (crushed limestone bedrock) sub-base material. It is also important that the sub-base and base granular layers of the pavement structure be placed as soon as possible after exposure, preparation, and approval of the exposed subgrade.

The suggested pavement structures outlined in Table F below are based on subgrade parameters estimated on the basis of visual and tactile examinations of the on-site soils and past experience. The outlined pavement structure may be expected to have an approximate ten to fifteen-year life, assuming that regular maintenance is performed. Should a more detailed pavement structure design be required, site specific traffic information would be needed, together with detailed laboratory testing of the subgrade soils.

TABLE F – RECOMMENDED PAVEMENT STRUCTURES

LAYER DESCRIPTION	**COMPACTION REQUIREMENTS**	**LIGHT DUTY SECTIONS**	**HEAVY DUTY [TRUCK ROUTE]**
Asphaltic Concrete Wearing course OPSS HL 3 or HL 3A	Min. 92 % Marshall MRD	40 millimetres	40 millimetres
Binder Course OPSS HL 8	Min. 92 % Marshall	50 millimetres	80 millimetres
Base Course OPSS Granular A	100% SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular B Type II	100% SPMDD	300 millimetres	450 millimetres

* Marshall MRD denotes Maximum Relative Density.
* SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

Depending on the anticipated traffic, a reduced light duty asphalt structure consisting of 65 millimetres of HL3 surface course may also perform sufficiently. This would be reasonable in areas subjected only to light vehicles such as cars for parking. Such a structure may have a reduced lifespan if subjected to heavier vehicles, and would also not allow for 'mill and pave' type operations for future rehabilitation.

Where asphalt pavement is to be constructed above the roof deck of the below grade parking level, the granular base layers recommended for the light duty pavement structure recommended above may be considered for both light duty and heavy duty areas. It is noted that in such cases the roof deck slab should be sufficiently sloped and/or provided with suitable subdrains, in order to promote rapid drainage of water from beneath the pavement. As well the roof slab should be provided with a suitable water proofing system.

To minimise segregation of the finished asphalt mat, the asphalt temperature must be maintained uniform throughout the mat during placement and compaction. All too often, significant temperature gradients exist in the delivered and placed asphalt with the cooler portions of the mat resisting compaction and presenting a honeycomb surface. As the spreader moves forward, a responsible member of the paving crew should monitor the pavement surface, to ensure a smooth uniform surface. The contractor can mitigate the surface segregation by 'back-casting' or scattering shovels of the full mix material over the segregated areas and raking out the course particles during compaction operations. Of course, the above assumes that the asphalt mix is sufficiently hot to allow the 'back-casting' to be performed.

11. SOIL EXPORT CONSIDERATIONS

It is understood that the development will incorporate up to three underground levels with excavations anticipated to extend to depths of up to approximately 10 metres, which would require the off-site disposal of surplus soil generated during construction. Based on the information above and the estimated building footprint area, approximately 25,000 cubic metres are anticipated to be generated during construction. Ontario Regulation 406/19 has recently come into effect, which governs the management of excess soils. As such, based on an approximate volume of 20,000 to 25,000 cubic metres of surplus soil generated, up to approximately 84 samples will need to be analyzed when conducting in-situ sampling, however this does not consider samples already recovered as part of the Phase Two ESA by CEGP.

Management of surplus soils requires the developer to conduct an assessment of the subject site, along with rigorous sampling and analysis, based on the volume of surplus soil generated, to support acceptance at an off-site location. Specifically, an Assessment of Past Uses, Sampling and Analysis Plan, Soil Characterisation Report, and Excess Soils Destination Report, will be required to support the acceptance of excess soils at an off-site property. Such testing and analysis can be conducted once development details have been finalised, based on volume of surplus soils to be generated, as well as the results of the Phase One and Two ESAs, currently being conducted by CEGP. If required, SOIL-MAT ENGINEERS may be retained to prepare such excess soil management reports and conduct the required analytical testing.

12. GENERAL COMMENTS

The comments provided in this document are intended only for the guidance of the design team. The material in it reflects SOIL-MAT ENGINEERS' best judgement in light of the information available at the time of preparation. The subsurface descriptions and borehole information are intended to describe conditions at the borehole locations only. It is the contractors' responsibility to determine how these conditions will affect the scheduling and methods of construction for the project. 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. SOIL-MAT ENGINEERS 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 trust that this geotechnical report is sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Scott Wylie, B. Eng., EIT

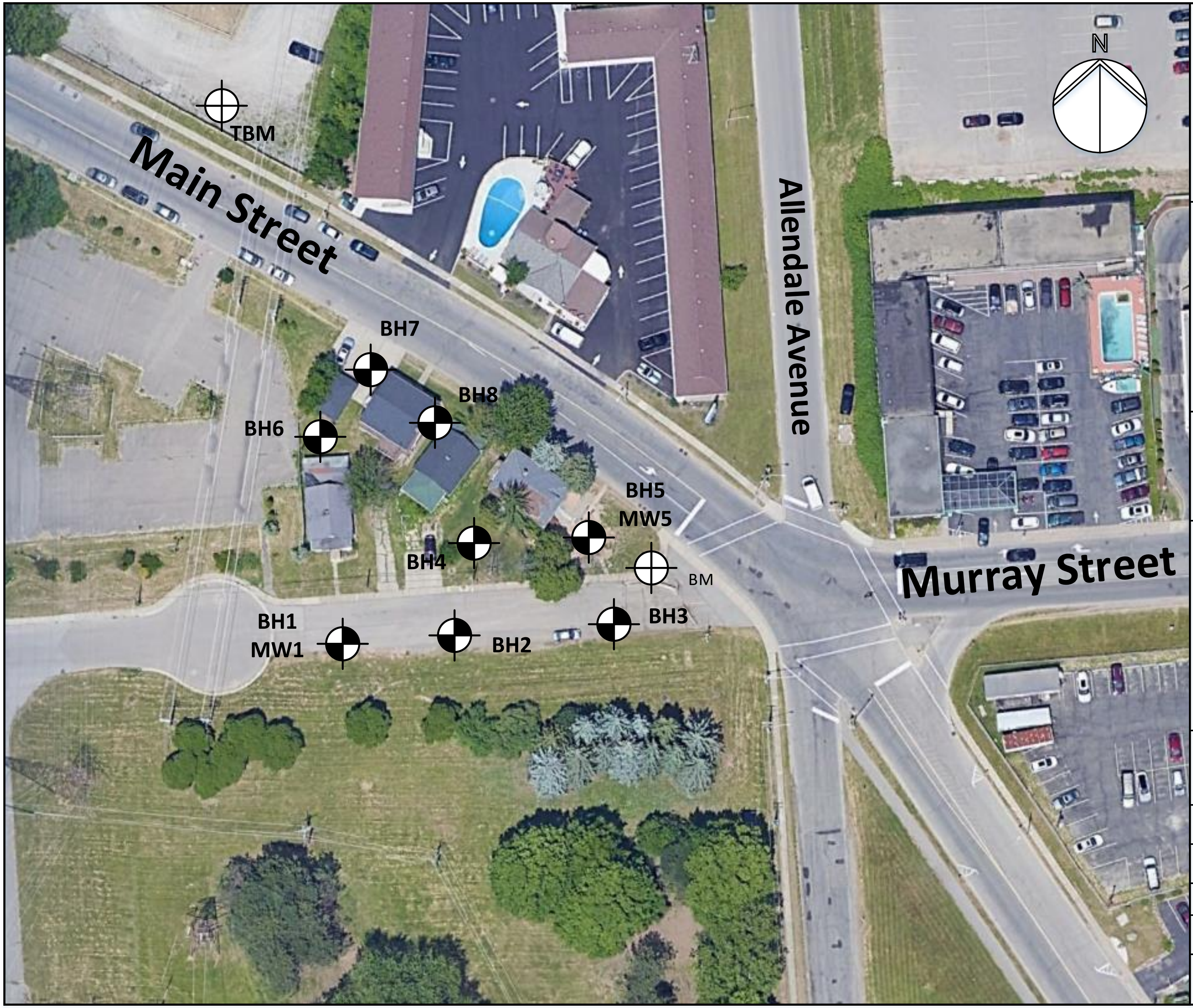
Kyle Richardson, P. Eng.
Project Engineer

LICENSED PROFESSIONAL ENGINEER
Jan 11/22
S. K. RICHARDSON
100179716
PROVINCE OF ONTARIO

Ian Shaw, P.Eng., QP_{ESA}
Senior Engineer

Enclosures: Drawing No. 1, Borehole Location Plan
Log of Borehole Nos. 1 through 8
Grain Size Analyses
Unconfined Compressive Strength Testing
Drawing No. 2 – Basement Perimeter Drainage

Distribution: Rudanco Hospitality Corporation [pdf]



LEGEND

BH# Borehole Location

BM Benchmark
Control Monument 0126
Elevation: 204.81m

NOTES

1. This drawing should be read in conjunction with Soil-Mat Engineers & Consultants Ltd. Geotechnical Report SM 301745-G.

2. Borehole locations are approximate.

SOIL-MAT
ENGINEERS & CONSULTANTS LTD.

Geotechnical Investigation
Proposed Condominium Development
Main Street and Murray Street
Niagara Falls, Ontario

Borehole Location Plan

Project No. SM 301745-G

Date: December 2021

Drawn: SW | Checked: KR

SM 301745-G Borehole Location Plan

Drawing No. 1

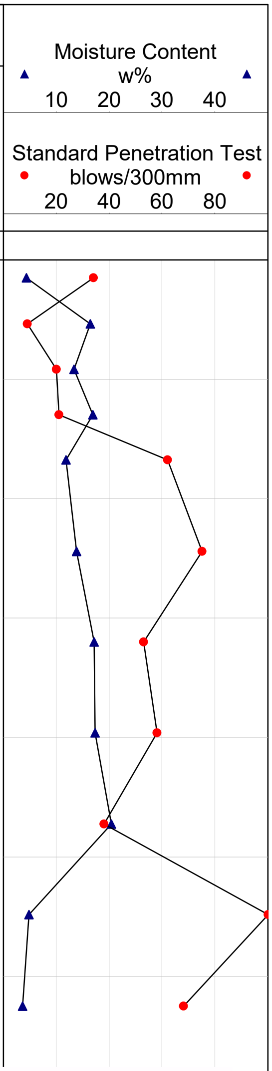
Log of Borehole No. 1

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771848
E: 655724

Elevation (m)	Description
202.51	Ground Surface
	Pavement Structure Approximately 100 millimetres of asphaltic concrete overlying 200 millimetres of compact granular base.
	Silt Reddish brown, trace to some clay, trace sand and gravel, reworked in the upper levels, dense to very dense.
193.10	**Sand** Brown, trace to some silt, trace gravel, coarse in gradation, dense to very dense.
189.70	End of Borehole

SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
SS	1	9,20,14,6	34	■		
SS	2	3,4,5,5	9	■		
SS	3	7,10,10,12	20	■		
SS	4	3,8,13,20	21	■		
SS	5	9,26,36,49	62	■		
SS	6	7,30,45,50	75	■		
SS	7	12,28,25,26	53	■		
SS	8	19,25,33, 50/5"	58	■		
SS	9	14,14,24,40	38	■		
SS	10	40,50/4"	100	■		
SS	11	25,33,35,40	68	■		



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 7, 2021 to termination at a depth of 12.8 metres.

2. Borehole was recorded as open and 'wet' at a depth of 9.1 metres upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

November 3, 2021 - Dry.

Drill Method: Solid Stem Augers
Drill Date: October 7, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

Soil-Mat Engineers & Consultants Ltd.
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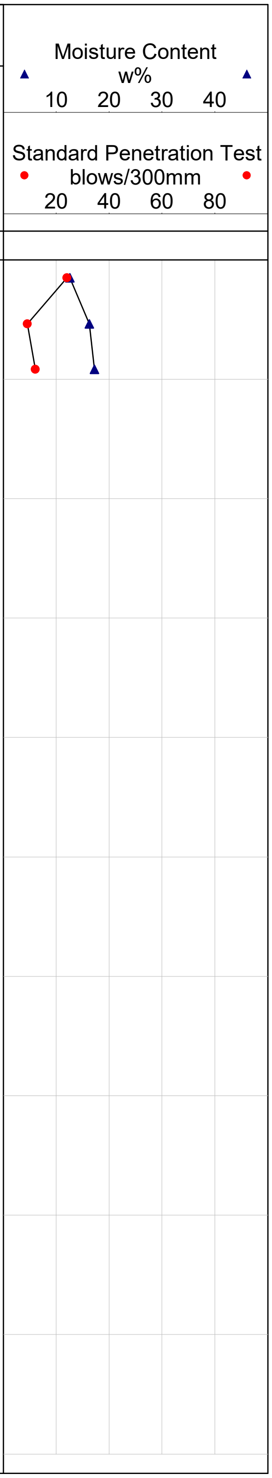
Datum: Geodetic
Field Logged by: EC/KJR
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 2

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771850
E: 655741

Depth (ft / m)	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0 / 0	203.18		Ground Surface								
			Pavement Structure Approximately 75 millimetres of asphaltic concrete overlying 200 millimetres of compact granular base.		SS	1	23,17,7,5	24	■		
					SS	2	4,5,4,4	9	■		
	201.00		**Silt** Reddish Brown, trace to some clay, trace sand and gravel, reworked in the upper levels, loose.		SS	3	2,4,5,8	9	■		
			End of Borehole								



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 7, 2021 to termination at a depth of 2.1 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 7, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

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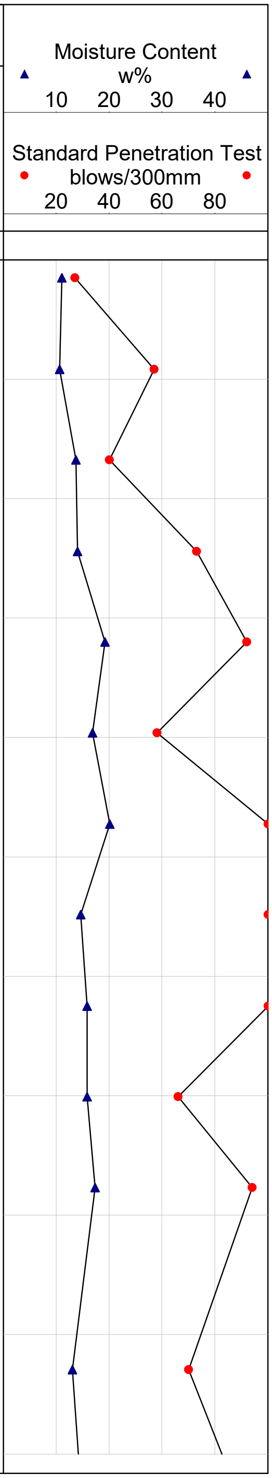
Sheet: 1 of 1

Log of Borehole No. 3

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771852
E: 655762

Depth	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
ft m	204.39		Ground Surface								
			Granular Fill Approximately 100 millimetres of compact granular fill.		SS	1	23,20,7,6	27	▨		
			Silt Reddish Brown, trace to some clay, trace sand and gravel, clayey inclusions, dense to very dense.		SS	2	8,23,34,50/5	57	▨		
					SS	3	12,18,22,32	40	▨		
					SS	4	17,36,37,32	73	▨		
					SS	5	23,43,49,41	92	▨		
					SS	6	12,20,38,39	58	▨		
					SS	7	50/6"	100	▨		
					SS	8	46,50/5"	100	▨		
	192.60		**Sand** Brown, trace to some silt, trace gravel, coarse in gradation, very dense.		SS	9	34,50/5"	100	▨		
					SS	10	19,30,36,36	66	▨		
					SS	11	26,46,48,37	94	▨		
					SS	12	21,33,37,41	70	▨		



Drill Method: Solid Stem Augers
Drill Date: September 28, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

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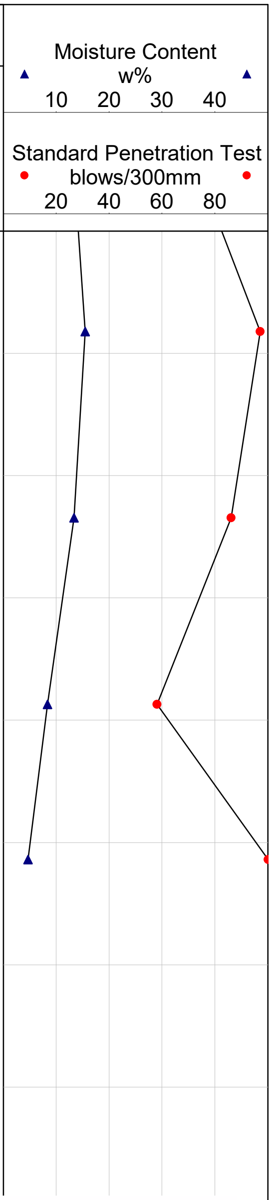
Sheet: 1 of 2

Log of Borehole No. 3

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771852
E: 655762

Elevation (m)	Description	Well Data	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
			SS	13	29,50,47,43	97	▨		
			SS	14	18,44,42,50/5	86	▨		
			SS	15	15,22,36,26	58	▨		
173.80			SS	16	50/1"	100			
172.70	**Limestone/Dolostone** Grey, weathered in upper levels, becoming more sound with depth. Core A: qu=69.8 MPa		NQ	1	RQD=84%		▨		
171.80	Core B: qu=56.9 MPa Core C: qu=76.6 MPa		NQ	2	RQD=94%		▨		
169.30	Core D: qu=25.3 MPa		NQ	3	RQD=95%		▨		
	End of Borehole								



NOTES:

1. Borehole was advanced using solid stem auger equipment on September 29, 2021 to termination at a depth of 30.5 metres. The bedrock was then cored using Nq core barrel equipment to a depth of approximately 35.0 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: September 28, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

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Datum: Geodetic
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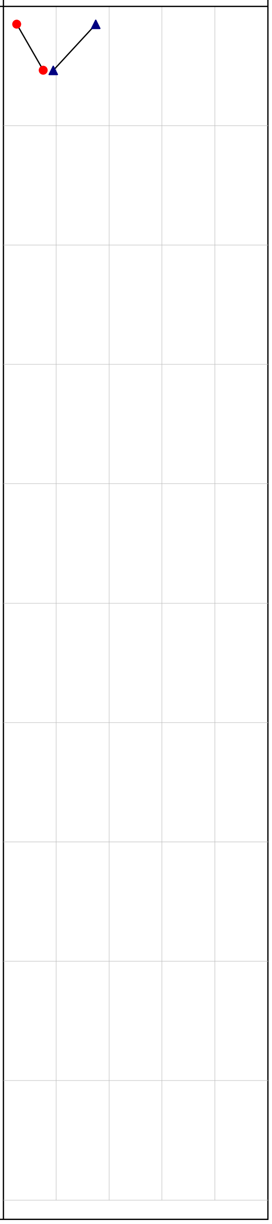
Checked by: KR
Sheet: 2 of 2

Log of Borehole No. 4

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771861
E: 655748

Depth (ft / m)	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	SAMPLE Number	SAMPLE Blow Counts	SAMPLE Blows/300mm	SAMPLE Recovery	SAMPLE PP (kgf/cm2)	SAMPLE U.Wt.(kN/m3)	Moisture Content ▲ w% 10 20 30 40 / Standard Penetration Test ● blows/300mm 20 40 60 80
0 / 0	204.24		Ground Surface									
0 / 0		▨	**Topsoil** Approximately 150 millimetres of topsoil.		SS	1	2,2,3,5	5	▨			
	202.90	▨	**Silt** Reddish Brown, trace to some clay, trace sand and gravel, reworked in the upper levels, compact.		SS	2	6,8,7,9	15	▨			
			End of Borehole									



NOTES:

1. Borehole was advanced using solid stem auger equipment on September 29, 2021 to termination at a depth of 1.4 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: September 29, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

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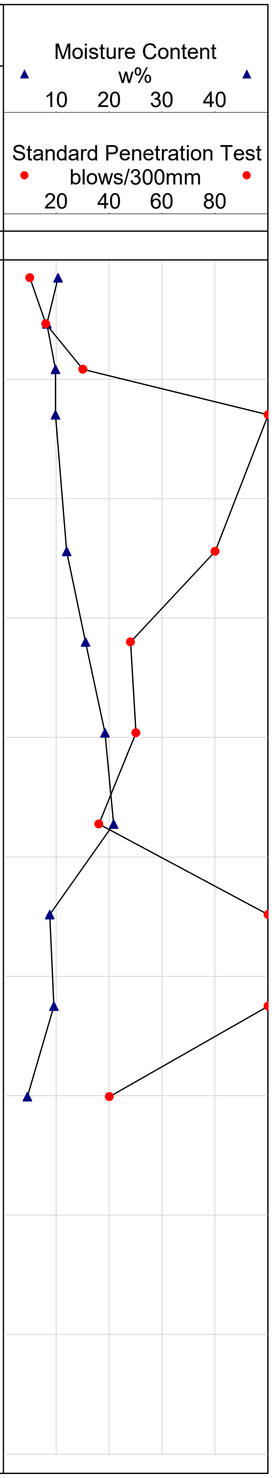
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 5

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771870
E: 655762

Depth (ft / m)	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0 / 0	205.86		Ground Surface								
			Topsoil Approximately 150 millimetres of topsoil.		SS	1	4,5,5,7,	10	▨		
					SS	2	5,7,9,9	16	▨		
			Silt Reddish brown, trace to some clay, trace sand and gravel, increasing clay content with depth, reworked in the upper levels, compact to very dense.		SS	3	6,11,19,24	30	▨		
					SS	4	16,42,50/4"	100	▨		
					SS	5	7,36,44,50/4	80	▨		
					SS	6	12,22,26,33	48	▨		
					SS	7	20,18,32,50/5	50	▨		
					SS	8	12,15,21,25	36	▨		
	195.60		**Sand** Brown, trace to some silt, trace clay and gravel, dense to very dense.		SS	9	18,40,50/5"	100	▨		
					SS	10	31,50/6"	100	▨		
					SS	11	12,20,20,22	40	▨		
	191.50		End of Borehole								



NOTES:

1. Borehole was advanced using solid stem auger equipment on September 28, 2021 to termination at a depth of 14.3 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

November 3, 2021 - Dry.

Drill Method: Solid Stem Augers
Drill Date: September 28, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

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Field Logged by: EC/KJR
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 6

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited
Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771878
E: 655723

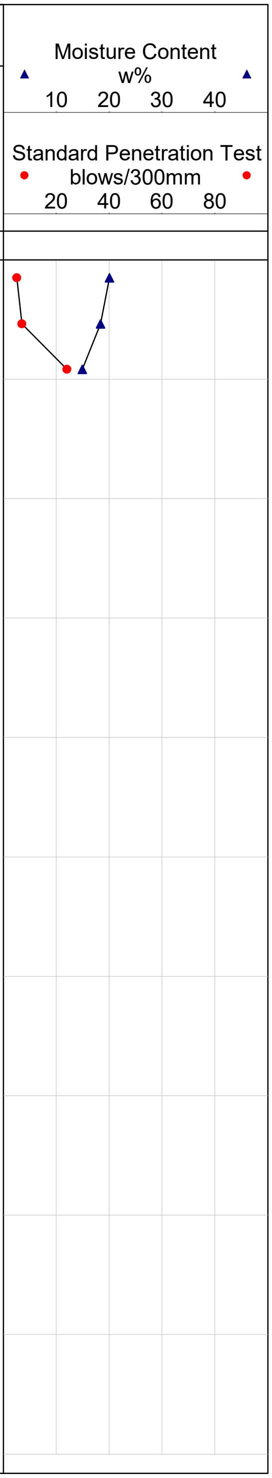
Depth (ft / m)	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	SAMPLE Number	SAMPLE Blow Counts	SAMPLE Blows/300mm	SAMPLE Recovery	SAMPLE PP (kgf/cm2)	SAMPLE U.Wt.(kN/m3)
0	204.13		Ground Surface								
			Topsoil Approximately 150 millimetres of topsoil.		SS	1	1,2,3,2	5	■		
			Silt Reddish brown, trace to some clay, trace sand and gravel, trace organics and reworked in the upper levels, loose to compact.		SS	2	2,3,4,6	7	■		
					SS	3	5,10,14,20	24	■		
	201.30		End of Borehole								

NOTES:

1. Borehole was advanced using solid stem auger equipment on September 28, 2021 to termination at a depth of 2.9 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.



Drill Method: Solid Stem Augers
Drill Date: September 28, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

Soil-Mat Engineers & Consultants Ltd.
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E: info@soil-mat.ca

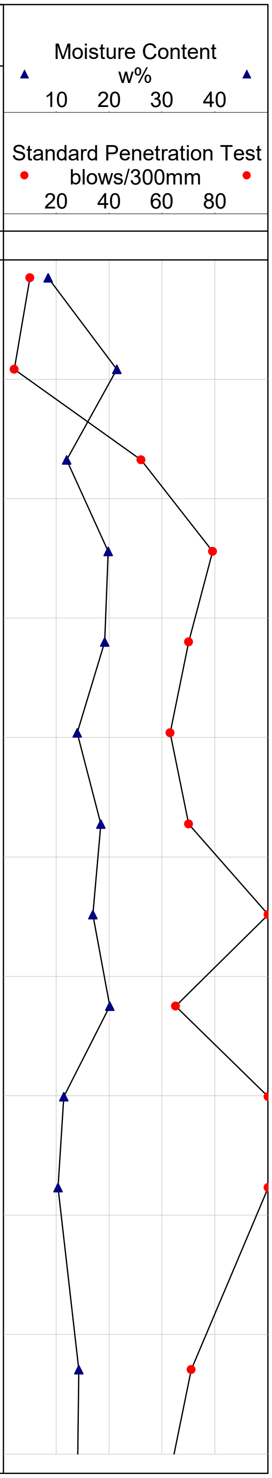
Datum: Geodetic
Field Logged by: EC/KJR
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 7

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771886
E: 655747

Elevation (m)	Description	Type	Number	Blow Counts	Blows/300mm	PP (kgf/cm2)
205.72	Ground Surface					
	Sand and Gravel Fill Approximately 150 millimetres of compact granular fill.	SS	1	7,7,3,2	10	
204.14	**Clayey Silt/Silty Clay Fill** Reddish brown to greyish brown, trace sand and gravel, firm.	SS	2	2,2,2,2	4	1.0
	Silt Reddish brown, trace to some silt, trace sand and gravel, clayey inclusions, very dense.	SS	3	12,22,30,34	52	
		SS	4	23,34,45,50	79	
		SS	5	28,32,38,50	70	
		SS	6	21,31,32,46	63	
		SS	7	23,31,39,42	70	
		SS	8	20,50/5"	100	
		SS	9	22,31,34,30	65	
		SS	10	42,50/3"	100	
191.00	**Sand** Brown, trace to some silt, trace clay and gravel, fine in gradation in the upper levels, becoming coarse with depth, dense to very dense.	SS	11	50/6"	100	
		SS	12	22,31,40,44	71	



Drill Method: Solid Stem Augers
Drill Date: September 30, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

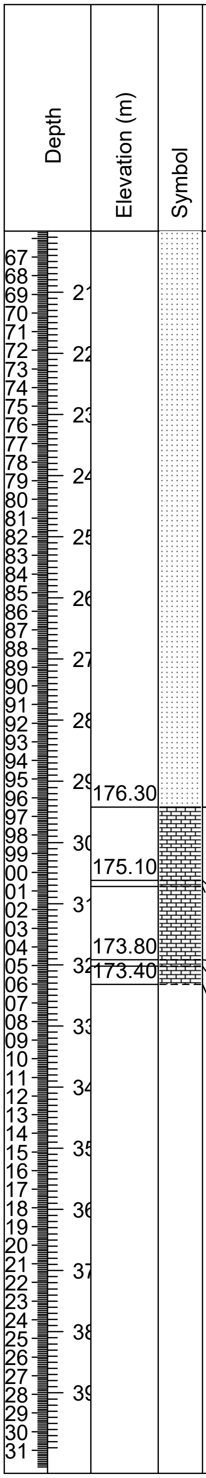
Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

Datum: Geodetic
Field Logged by: EC/KJR
Checked by: KR
Sheet: 1 of 2

Log of Borehole No. 7

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771886
E: 655747



Limestone/Dolostone

Grey, highly weathered in upper levels, becoming more sound with depth.

Core A: qu=27 MPa

Core B: qu=69.1 MPa

Core C: qu=13.2 MPa

Core D: qu=32.9 MPa

End of Borehole

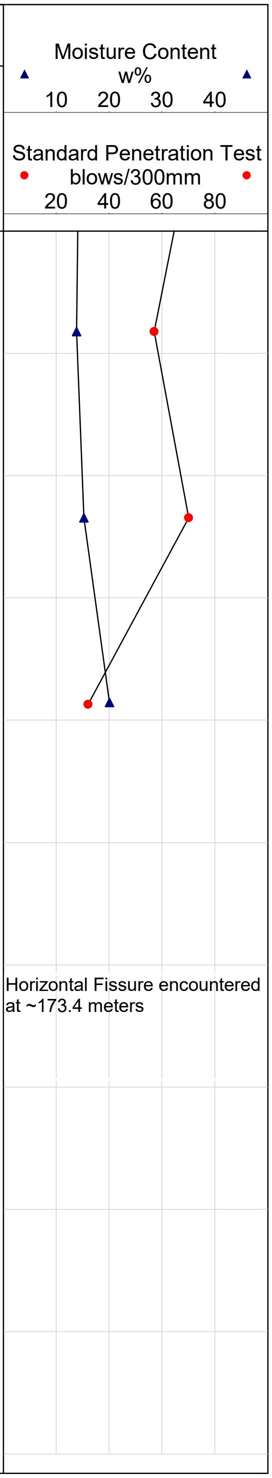
NOTES:

1. Borehole was advanced using solid stem auger equipment on September 30, 2021 to termination at a depth of 29.5 metres. The bedrock was then cored using Hq core barrel equipment to a depth of approximately 32.5 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
	SS	13	21,28,29,39	57	■		
	SS	14	22,31,39,35	70	■		
	SS	15	15,16,16,17	32	■		
	HQ	1	RQD=53%		■		
	HQ	2	RQD=95%		■		
	HQ	3	RQD=100%		■		



Horizontal Fissure encountered at ~173.4 meters

Drill Method: Solid Stem Augers
Drill Date: September 30, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

Datum: Geodetic
Field Logged by: EC/KJR
Checked by: KR
Sheet: 2 of 2

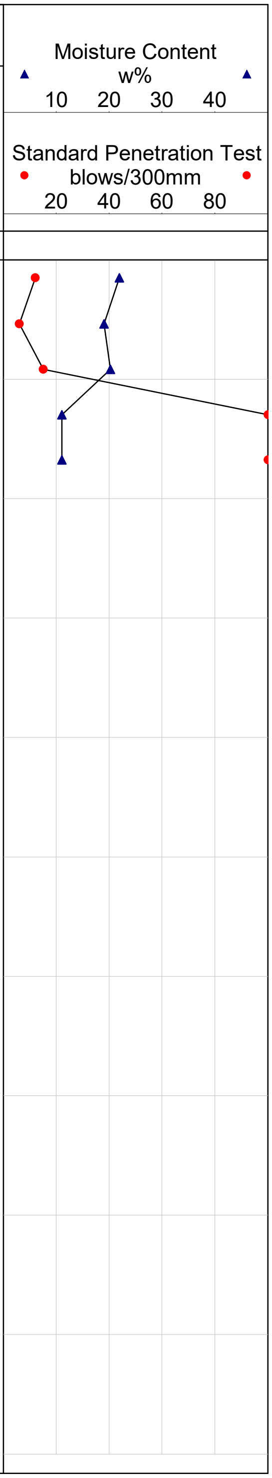
Log of Borehole No. 8

Project No: SM 301745-G
Project: Proposed Condominium Development
Location: Main and Murray Street, Niagara Falls
Client: Zeljko Holdings Limited

Project Manager: Kyle Richardson, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4771877
E: 655738

Elevation (m)	Description
205.27	Ground Surface
204.60	**Sand and Gravel Fill** Approximately 150 millimetres of topsoil.
	Clayey Silt/Silty Clay Reddish brown, trace to some sand and gravel, stiff.
201.90	**Silt** Reddish brown, trace to some clay, trace sand and gravel, clayey inclusions, loose to very dense.
	End of Borehole

Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
SS	1	2,6,6,4	12	▨		
SS	2	2,3,3,4	6	▨		
SS	3	2,3,12,22	15	▨		
SS	4	10,50/6"	100	▨		
SS	5	21,36,50/5"	100	▨		



NOTES:

1. Borehole was advanced using solid stem auger equipment on September 27, 2021 to termination at a depth of 3.4 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: September 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Davis Drilling

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130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

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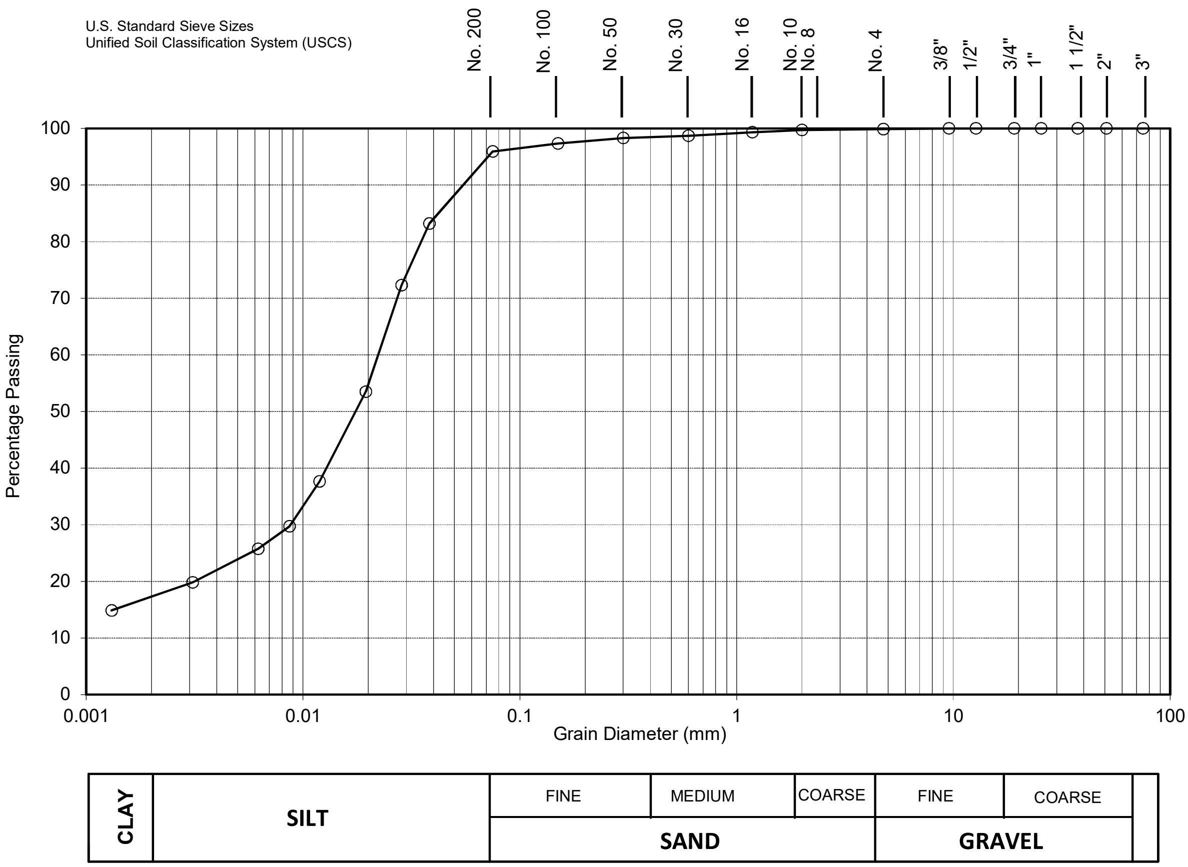
Field Logged by: EC/KJR
Checked by: KR
Sheet: 1 of 1

Mechanical & Hydrometer Analyses



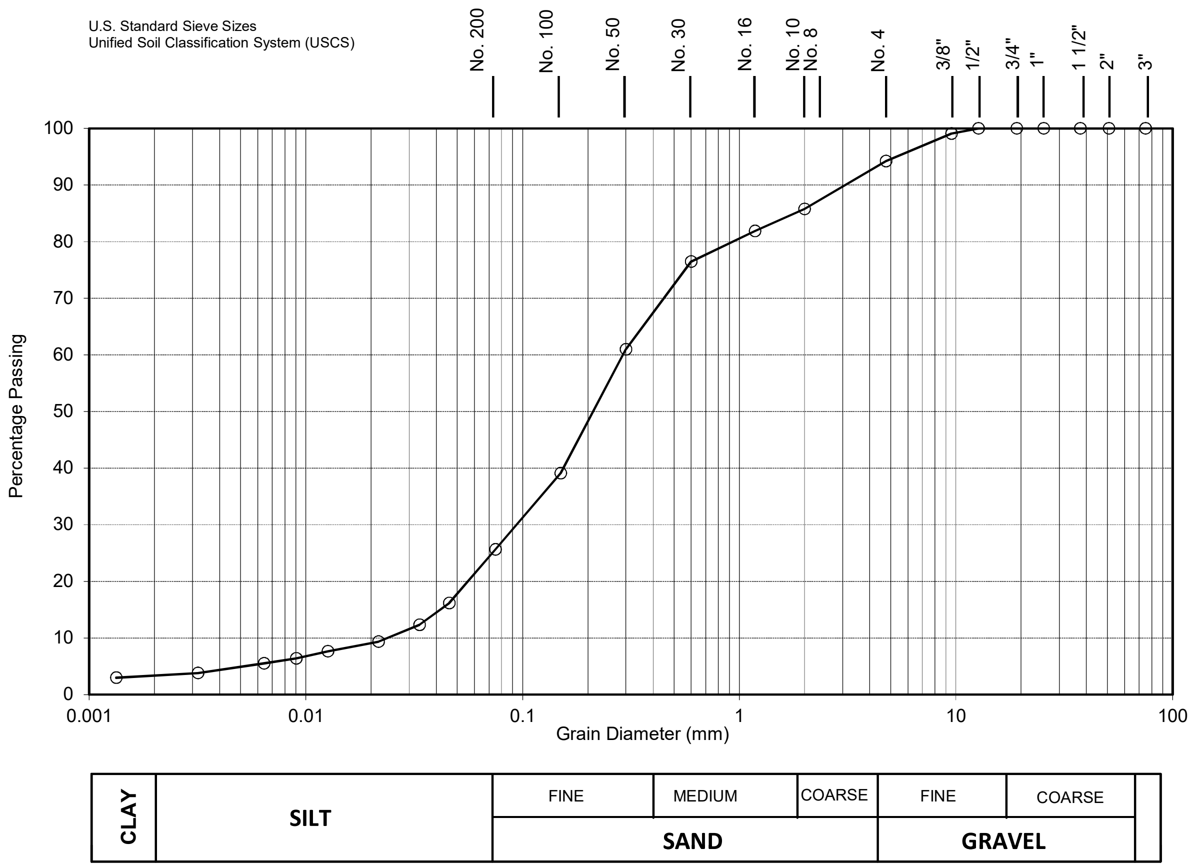
Lab No.:	**21- 406**	Notes: Depth 25'			
Borehole No.:	**3**				
Sample No:	**6**				
CLAY [%]:	**18**	Soil Description: **Reddish Brown Silt w/ some Clay and a trace of Sand** **M.L. - Inorganic silts and very fine sands, clayey silts with slight plasticity to**			
SILT [%]:	**78**				
SAND [%]:	**4**				
GRAVEL [%]:	**0**	Estimated Infiltration Rate [mm/hr] :	**< 10**	Estimated Permeability, k [cm/s]	**10^{-7}**
D_{10} (Effective Diam. in mm):	**0.0007**	Coefficient of Uniformity C_U:	**31.4**	Coefficient of Curvature C_C:	**4.7**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Proposed Condo - Main & Murray, Niagara Falls ON

October 2021	**Grain Size Analysis No. 1**	**Project No.: SM 301745-T**

Mechanical & Hydrometer Analyses



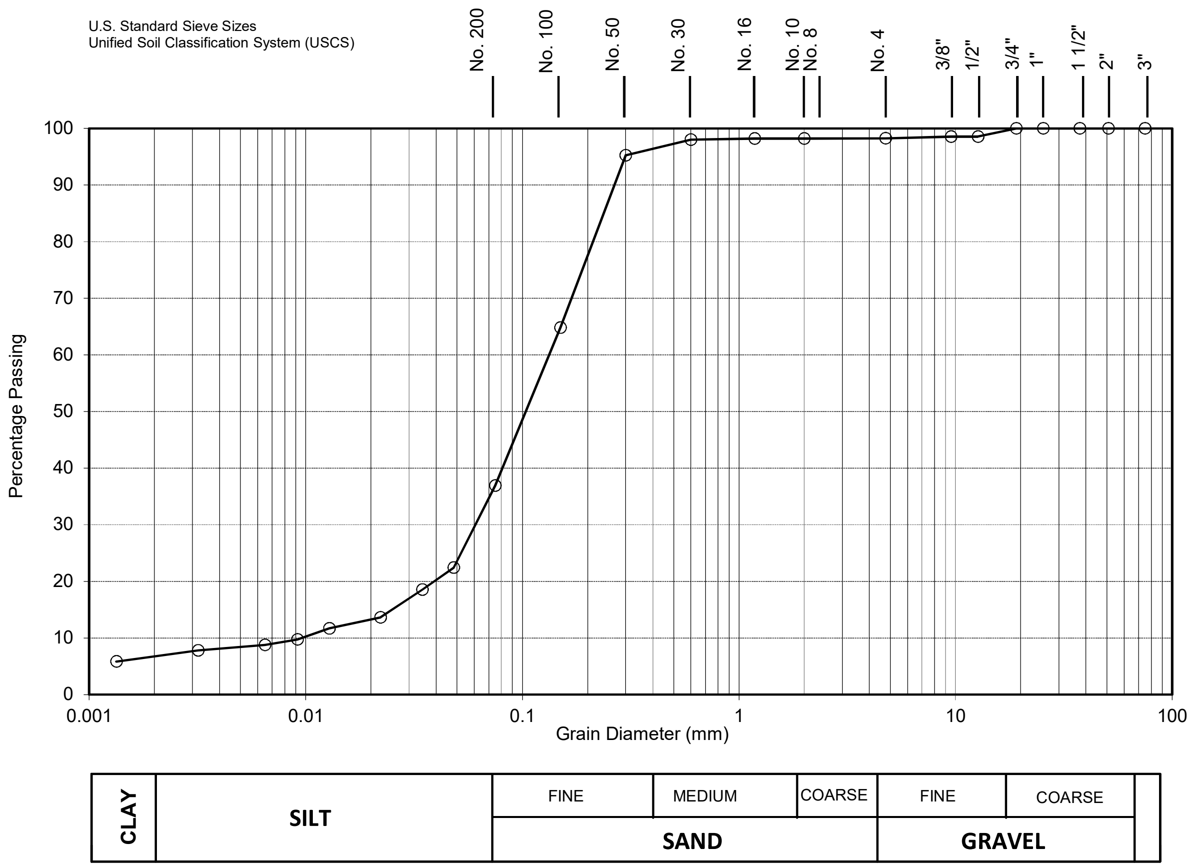
Lab No.:	**21- 407**	Notes: Depth 70'			
Borehole No.:	**3**				
Sample No:	**13**				
CLAY [%]:	**3**	Soil Description:	**Reddish Brown Silty Sand w/ traces of Gravel and Clay** **S.M. - Silty sands, sand-silt mixtures**		
SILT [%]:	**23**				
SAND [%]:	**68**				
GRAVEL [%]:	**6**	Estimated Infiltration Rate [mm/hr] :	**50 to 75**	Estimated Permeability, k [cm/s]	**10^{-4}**
D_{10} (Effective Diam. in mm):	**0.023**	Coefficient of Uniformity C_U:	**12.6**	Coefficient of Curvature C_C:	**1.3**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Proposed Condo - Main & Murray, Niagara Falls ON

October 2021	**Grain Size Analysis No. 2**	**Project No.: SM 301745-T**

Mechanical & Hydrometer Analyses



Lab No.:	21- 408	Notes: Depth 60'			
Borehole No.:	7				
Sample No:	12				
CLAY [%]:	7	Soil Description:	Reddish Brown Silty Sand w/ traces of Clay and Gravel S.M. - Silty sands, sand-silt mixtures		
SILT [%]:	30				
SAND [%]:	61				
GRAVEL [%]:	2	Estimated Infiltration Rate [mm/hr] :	40 to 50	Estimated Permeability, k [cm/s]	10^{-5}
D_{10} (Effective Diam. in mm):	0.0010	Coefficient of Uniformity C_U:	140.0	Coefficient of Curvature C_C:	26.6

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Proposed Condo - Main & Murray, Niagara Falls ON

October 2021 | **Grain Size Analysis No. 3** | **Project No.: SM 301745-T**

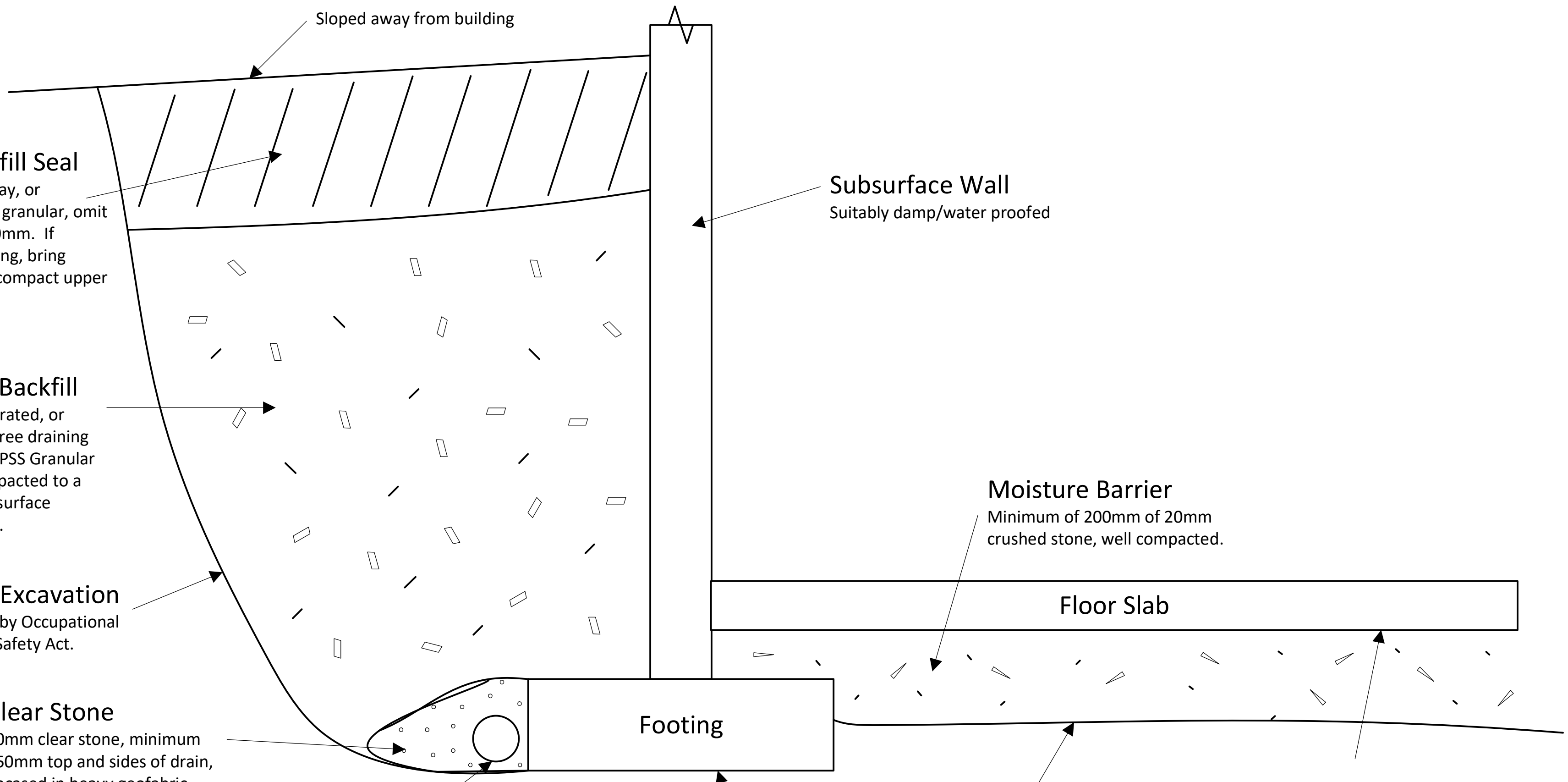
Core	Date Tested	Average Diameter	Average Height	Dry Weight	Break Load		Aspect Ratio Factor	Cross-Sectional Area	Calculated Volume	Calculated Unit Weight	Calculated Density	Unconfined Compressive Strength
		D	H	(mass)	Q_{ult}			A	V	γ_{conc}	ρ_{conc}	
	BH1 Box1	[mm]	[mm]	[g]	[lb]	[N]	C_F	[m^2]	[m^3]	[kN/m^3]	[kg/m^3]	[MPa]
A BH8-106'	Oct-07-21	63.0	129	1033.0	23030	102437.4	1.00	0.00312	0.00040	25.2	2568.9	32.9
B BH8-100'8"	Oct-07-21	47.0	133	625.8	26950	119873.6	1.00	0.00173	0.00023	26.6	2712.1	69.1
C BH8-104'8"	Oct-07-21	63.0	146	1095.8	9320	41455.4	1.00	0.00312	0.00046	23.6	2407.7	13.3
D BH3-104'	Oct-07-21	47.0	119	532.6	22200	98745.6	1.00	0.00173	0.00021	25.3	2579.7	56.9
E BH3 - 101'3"	Oct-07-21	47.0	98	436.6	27240	121163.5	1.00	0.00173	0.00017	25.2	2567.9	69.8
F BH3 - 108'1"	Oct-07-21	47.0	130	551.5	9860	43857.3	1.00	0.00173	0.00023	24.0	2445.2	25.3
G BH3 - 106'5"	Oct-07-21	47.0	137	617.0	29870	132861.8	1.00	0.00173	0.00024	25.5	2595.8	76.6
H BH8 - 100'5"	Oct-07-21	63.0	129	1072.3	19020	84601.0	1.00	0.00312	0.00040	26.1	2666.6	27.1

Date cast:	
Date cored:	
Date received:	October, 2021
Project/Contract No.:	SM 301745-G
Structure:	

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

COMPRESSIVE STRENGTH TEST - BEDROCK CORES

Project No.: SM 301745-G **Lab Number: 21-1300** October, 2021



Ground Surface
Sloped away from building

Impermeable Backfill Seal
Well compacted clay/silty clay, or equivalent. If original soil is granular, omit seal and compact upper 600mm. If pavement adjacent to building, bring Granular 'B' to surface and compact upper 1 metre to 100% SPMDD.

Subsurface Wall
Suitably damp/water proofed

Select Compacted Backfill
Free of organic, frozen, saturated, or otherwise unsuitable soil. Free draining granular material, such as OPSS Granular 'B' (Type II) preferred. Compacted to a minimum of 95% SPMDD if surface settlement can be tolerated.

Moisture Barrier
Minimum of 200mm of 20mm crushed stone, well compacted.

Limit of Excavation
As required by Occupational Health and Safety Act.

Clear Stone
20mm clear stone, minimum 150mm top and sides of drain, encased in heavy geofabric.

Perimeter Drain
150mm diameter weeping tile or pipe equivalent, leading to positive sump or outlet. Invert at least 150mm below underside of floor slab.

Pour flush with original undisturbed soil.

Subgrade
Competent native soil or well compacted fill.

Vapour Barrier
Where 'non-damp' floors are required, provide heavy poly sheeting.

NOT TO SCALE

Soil-Mat

Soil-Mat Engineers & Consultants Ltd.

Project No.:	SM 301745-G
Date:	December 2021

Typical Design Requirements
Drainage and Backfill for Basement Walls

Drawing No. 2

GEOPHYSICS GPR INTERNATIONAL INC.
6741 Columbus Road
Unit 14
Mississauga, Ontario
Canada L5T 2G9
Tel.: (905) 696-0656
Fax: (905) 696-0570
gprtor@gprtor.com
www.geophysicsgpr.com

January 6th, 2022 GPR file: T213541

Kyle Richardson, P.Eng.
Project Manager
Soil-Mat Engineers & Consultanys Ltd.
130 Lancing Drive
Hamilton, Ontario
L8W 3A1

RE: Downhole seismic survey in borehole BH03 at Main and Murray Street, Niagara Falls, Ontario

Dear Mr. Richardson:

Geophysics GPR International Inc. has been requested by Soil-Mat Engineers & Consultants Ltd. to carry out a downhole seismic survey at the above site in Niagara Falls, Ontario. Figure 1 shows the approximate location of the test borehole.

The survey was performed on December 24th, 2021.

The purpose of the testing was to measure the in-situ shear-wave (S-wave) velocities to determine seismic site class. The ASTM D7400-14 Down-hole Seismic Testing test method was applied.

The following paragraphs describe the survey design, the principles of the test method, the methodology for interpreting the data, and provide a culmination of the results in table and chart format.

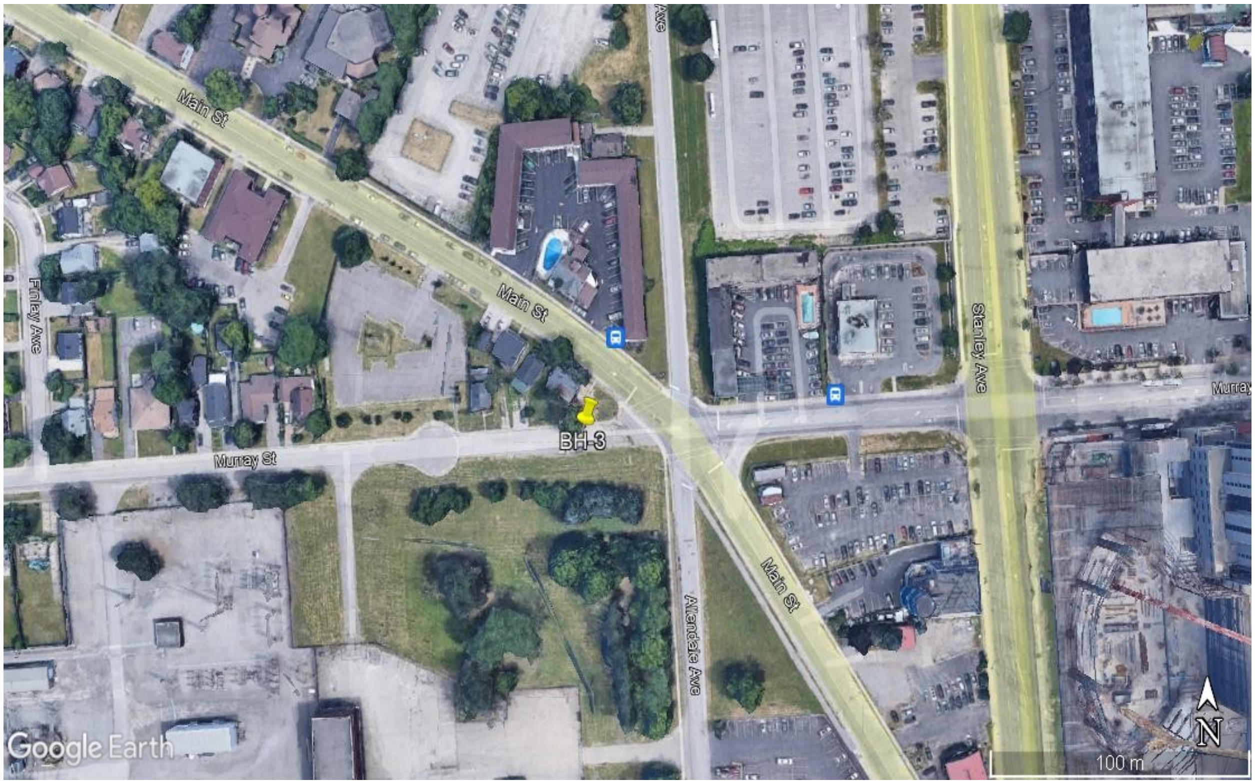


Figure 1: Approximate location of the borehole test, Niagara Falls, ON.

METHODOLOGY

Seismic Down-hole

The purpose of the investigations was to provide the seismic shear-wave (S-wave) velocity depth profile to generate an interpreted layer model.

Basic Theory

The seismic down-hole method relies on the accurate measurement of the transit time for a generated wave to travel from a shot-point on the surface to a receiver (geophone) at sequential depths within a borehole. The velocities at which the waves propagate are then determined from the arrival times of the impulse signals. Arrival time records are recorded separately for waves with preferential shear (S) wave components and compressional (P) wave components. The seismic "P" wave velocity depends mainly on volumetric elastic ratio of the constituent soil particles and pore water. The seismic "S" wave velocity depends more on the structural elasticity of the material, which is influenced by the size, form and tightness of the particles (for the case of unconsolidated sediments). Unlike the P-wave, a polarized S-wave is easily generated.

Survey Design

A tri-axial geophone, containing two orthogonal horizontal geophones, for detecting the shear (S) wave arrivals, and a vertical geophone for detecting the compressional (P) wave arrivals, was used as the receiver. The geophone was held firm to the borehole casing by a motorized wall-lock.

Data were recorded with an ABEM Pro 2 seismograph. The sampling interval was set to 40 µs with 8192 samples for a total record length of 327 ms with a pre-trigger delay of 10 ms. The seismic source was located 0.7 m from the borehole.

A 10-lb sledgehammer and a propelled energy generator (PEG) were used as energy sources with a minimum of three stacks per shot.

Three seismic records were recorded at each one metre interval with the tri-axial geophone beginning at the bottom of the borehole and ending at one meter depth. There are records for the downward hammer blow, which generates a very strong compressional wave from surface to the receiver. The second hammer strike is applied to the side of a block of wood that is orientated perpendicular to the borehole and offset approximately 0.7 meters from the hole. The block stays rigid on the ground by putting a weight (person) on top. The last strike is applied to the opposite side of the block. The second and third strokes generate polarized shear waves of opposite polarity to aid in identification. Figure 2 is an example of a single seismic shot record from a preferential P-wave generated shot.

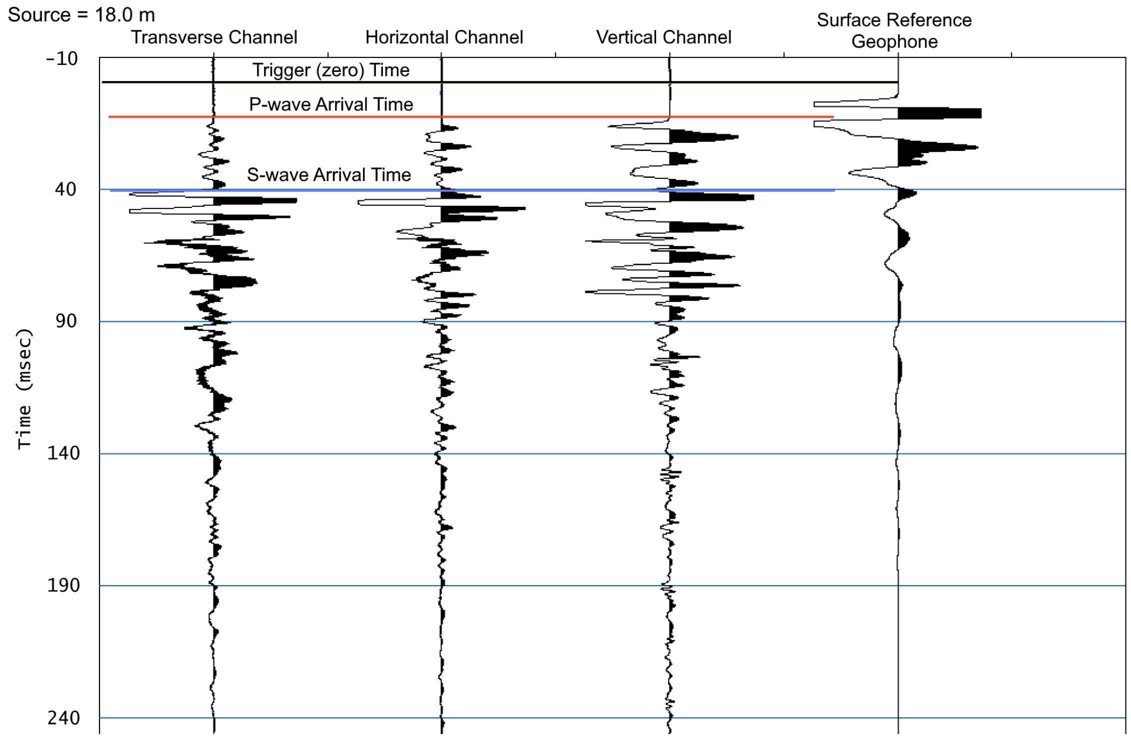


Figure 2: Typical example shot record, preferential P-wave generated

Interpretation Method and Accuracy of Results

Interpretation of the down-hole seismic data involves identifying the first arrival times of the P-waves and S-waves from the shot records at each depth interval. The seismic traces from each depth interval can be combined into one image with the S-wave traces from opposite polarizations overlapped to aid in the interpretation.

The preferred method for analyzing down-hole data is to produce time-distance plots and calculate the velocities from the slope of the best-fit lines. The selection of the best-fit lines can be visually interpreted by the analyst or can be computer aided. This investigation used the geologic contacts identified in available borehole data as the primary method for assigning layer contacts along with visual interpretation of changes in slope of the time-distance plots. The *ReflexW* software developed by Dr. Karl Jozef Sandmeier was used for interactive interpretation of the velocity layer contacts and inversion calculations for determining the layer velocities accounting for refractions and pick confidence. ReflexW was used to pick the first arrivals of both the P- and S-waves. A detailed description of the software used for the interpretation can be found in the ReflexW Manual which can be found on Sandmeier website.

RESULTS & CONCLUSIONS

The lowest geophone depth was at 29.0 m. The signal-to-noise ratio for the downhole seismic data was good.

The inversion model for the S-wave and P-wave data are presented in Figures 3 and 4.

Table 1 presents the modeled data summary with depths in meters.

Table 1: Seismic velocity summary table for borehole BH03

Depth (m)	Interval (m)	Model Velocity (m/s)	Thickness (m)	Cumulative Thickness (m)	Delay for Modelled Vs (s)	Cumulative Delay Time (s)	Vs to given Depth (m/s)
0.0	3.0	364.0	3.0	3.0	0.00824	0.00824	364.0
3.0	7.0	455.0	4.0	7.0	0.00879	0.01703	411.0
7.0	12.0	639.0	5.0	12.0	0.00782	0.02486	482.7
12.0	24.0	493.0	12.0	24.0	0.02434	0.04920	487.8
24.0	29.0	553.0	5.0	29.0	0.00904	0.05824	497.9
29.0	30.0	553.0	1.0	30.0	0.00181	0.06005	499.6

The V_S30 value for the seismic test is presented in Table 2. The V_S30 values are based on the harmonic mean of the shear wave velocities over the 30 m. The V_S30 value is calculated (as outlined in Commentary 'J' sentence 4.1.8.4(2)-101 of the National Building Code) by dividing the total depth of interest (e.g. 30 m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response.

Table 2: Calculated Vs30 values (m/s) from the down-hole data

Borehole	**V_S30**	**Seismic Site Class (based on 30 m interval below ground level)**
BH03	500 m/s +/- 5%	**C**

Based on the V_S30 values (as determined through the down-hole seismic with consideration for the estimated error) and table 4.1.8.4.A of the National Building Code of Canada, 2015 Edition, the seismic site classification is within class "C" ($360 < V_S30 < 760$ m/s).

It must be noted that the site classification provided in this report is based on the V_S30 value as derived from the down-hole seismic method and may be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, the presence of sensitive and/or liquefiable soils, more than 3 m of soft clays, high moisture content, etc.The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2015 Edition for more information on the requirements for site classification.

Processing of the seismic data was performed by Duro Zeljkovic, GIT. This report has been written by Lhoucin Taghya, P. Geo.

Lhoucin Taghya

Sincerely,

Lhoucin Taghya, P.Geo.

Geophysicis

PROFESSIONAL GEOSCIENTIST

LHOUCIN TAGHYA (LT)

PRACTISING MEMBER

2872

ONTARIO

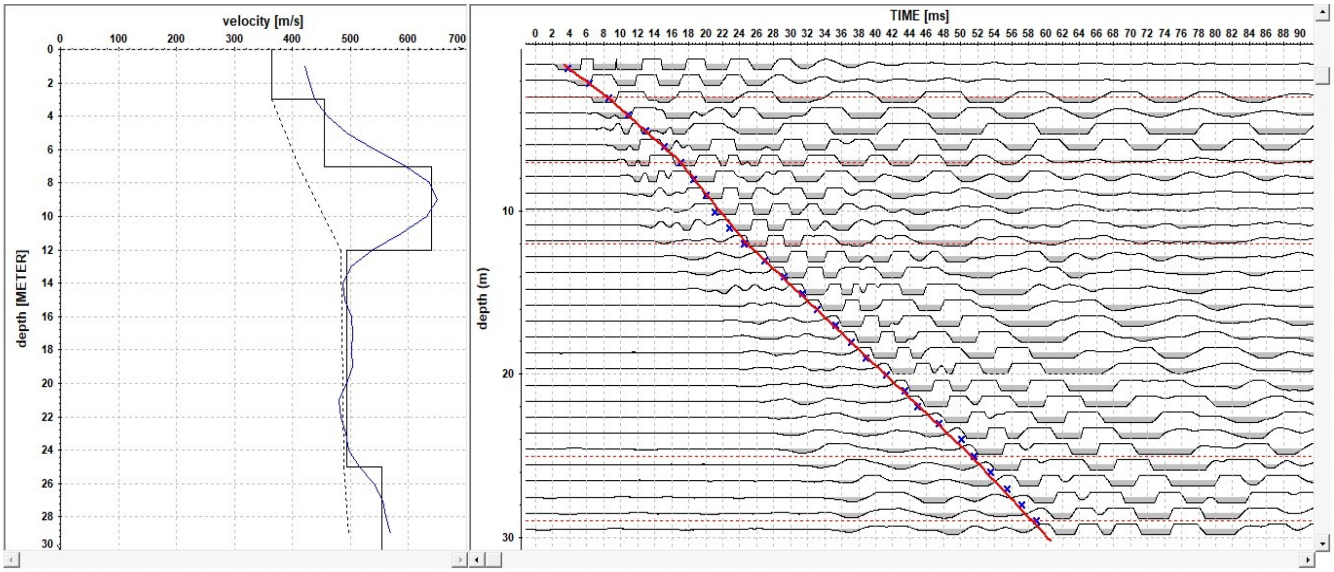


Figure 3: Borehole BH03 S-wave Velocity Inversion Model

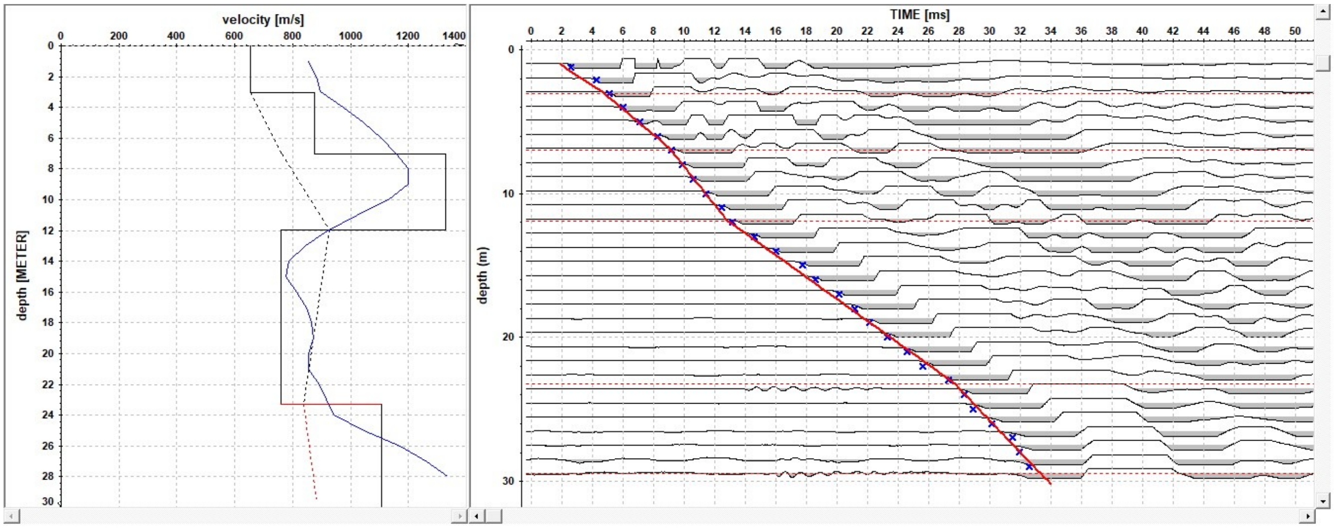


Figure 4: Borehole BH03 P-wave Velocity Inversion Model