Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL AND ROCK PROBE INVESTIGATION
CONDOMINIUM DEVELOPMENT
700 PARIS STREET
SUDBURY, ONTARIO

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1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by Michael D. Allen Architect c/o 2226553 Ontario Inc. to carry out a geotechnical and rock probe investigation for a proposed condominium development. The subject property is located at 700 Paris Street in the City of Greater Sudbury, Ontario (see Figure 1).

This report is a revisio0on of our previous rock probe report (File No. 51-14-9026, December 3rd, 2014) entitled:

ROCK PROBE INVESTIGATION
PROPOSED CONDOMINIUM DEVELOPMENT
700 PARIS STREET
SUDBURY, ONTARIO

This revisions provides additional information with respect to the subgrade soils and the underlying bedrock Rock Quality Designation (RQD).

The exploratory geotechnical and rock probe investigation program was devised to collect subgrade soil samples and map the bedrock profile at the site by advancing two exploratory boreholes and eighteen rock probes. Based on the results of the exploratory borehole and rock probe investigation, geotechnical engineering recommendations are presented for the following items:

- Frost depth;
- Bearing capacity of the sub-strata;
- Appropriate types of foundations;
- Foundation factors for earthquake forces;
- Excavation procedures;
- Trench stability;
- Bedding and compaction requirements;
- Dewatering and drainage requirements;
- Geotechnical Construction Implications; suitability of on site soil to reuse as backfill;
- Unit density of soil and coefficients for lateral load design;
- Considerations for constructibility.

2.0 SITE AND BRIEF PROJECT DESCRIPTION

The property was the former site of the General Hospital. The south of the existing building was demolished consist to permit the construction of an underground parking garage and condominium building. The terrain at the site generally slopes in a easterly direction towards Ramsey Lake.

For discussion purposes, Paris Street is assumed to be running in a north-south direction at this location.

The subject property is bound by the following:

North - Facer Street, residential properties;
West - Paris Street, residential properties;
South - Municipal parking lot;
East - Bell Park, Ramsey Lake.

It is proposed to construct an eight floor condominium building that would be supported by a three storey underground parking garage. The condominium building will be serviced by the City of Greater Sudbury municipal services consisting of storm and sanitary sewers and municipal drinking water system.

3.0 FIELD INVESTIGATION

3.1 Rock Probes

The initial field investigation to advance rock probes was conducted on November 5th, 2014. The proposed initial rock probe program consisted of advancing twenty six (26) exploratory rock probes. Based on the depth of the current existing excavation and the rock probes locations along the east and south sections were not accessible. The final field investigation program consisted of advancing eighteen (18) exploratory rock probes to depths of up to 10.67 metres within the building footprint (See figure 2 for the location of the rock probes).

Prior to conducing the exploratory Rock Probes investigation, the underground services locates were provided by all members of Ontario One.

The rock probe location were marked in the field by Tulloch based on the building layout provided by Michael D. Allen Architect. The geodetic elevations of the borings were determined by Tulloch relative to the City of Greater Sudbury vertical controls and UTM Zone 17 NAD 83 CSRS datum.

The drilling work was carried out by Belanger Construction utilizing a hydrotrack drill rig. The operation was monitored by a Terraprobe technician who logged the probable bedrock depth.

3.2 Boreholes

The exploratory borehole investigation was carried out by Terraprobe between July 25 to 26th, 2016. The geotechnical investigation consisted of advancing the following exploratory boreholes (see figure 3 for the borehole locations):

1. Borehole 1 was advanced in close proximity of RP 72.
2. Borehole 2 was advanced in close proximity of RP 64.

Prior to conducing the exploratory borehole investigation, the underground services locates were provided by Ontario One.

The location of the boreholes were located in the field by Tulloch Geomatics. The elevations of the borings were determined relative to the City of Greater Sudbury vertical controls and UTM Zone 17 NAD 83 CSRS datum.

The drilling work was carried out by Landcore Drilling utilizing a truck mounted drill rig, equipped with conventional soil sampling equipment and rock coring equipment (NQ cores). The operation was monitored by a Terraprobe Engineer in Training (EIT) whom logged the borings and examined the samples as they were obtained. All samples obtained from these boreholes were sealed into plastic jars, and transported to the Terraprobe laboratory for detailed inspection and testing. All of the borehole samples were examined (tactile) in detail by the project engineer, and classified according to visual and index properties. The boreholes were backfilled once the soil samples were retrieved.

The Standard Penetration Test (SPT) was used to obtain samples of the strata penetrated in the exploratory boreholes, using the Split-Barrel Method technique as outlined in ASTM D1586. The soil samples were taken with a conventional 50 mm diameter split barrel sampler at 0.75 m intervals for the entire length of the boreholes. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole locations. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are therefore approximate.

The rock cores (NQ) were retrieved from each location and were placed in rock core boxes and transported to the Terraprobe laboratory for detailed inspection and classified according to visual and index properties.

Groundwater level observations are noted on the borehole logs in Appendix A.

4.0 SUBSURFACE CONDITIONS

4.1 Rock Probes

Details of the subsurface conditions encountered at the site are summarized below. The bedrock depth encountered in the rock probes are presented on the attached Rock Probe Log sheets in Appendix A.

It should be noted that the subsurface conditions are confirmed at the rock probe locations only. The stratigraphic boundaries indicated on the Rock Probe Log sheets are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil or rock type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced rock probes and will vary between and beyond the rock probe locations. The following discussion has been simplified in terms of the major soil and rock strata for the purposes of geotechnical design. It may not be possible to drill a sufficient number of rock probes and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling.

For this soil investigation, no soil samples were retrieved.

4.1.1 Probable Bedrock Subgrade Elevation

The following table presents the exploratory rock probe elevations and recorded depths:

Probable Bedrock Subgrade Elevation

Rock Probe Location	Surface Elevation (m)	Depth to Probable Bedrock (m)	Probable Bedrock Subgrade Elevation (m)
60	264.26	3.05	261.21
61	263.73	3.05	260.68
62	263.39	4.57	258.82
63	263.52	10.67	252.85
64	264.17	6.10	258.07

Rock Probe Location	Surface Elevation (m)	Depth to Probable Bedrock (m)	Probable Bedrock Subgrade Elevation (m)
65	265.13	3.96	261.17
66	265.17	2.44	262.73
67	266.09	2.44	263.65
68	264.94	1.22	263.72
70	264.96	1.83	263.13
71	264.11	1.52	262.59
72	264.01	2.44	261.57
73	264.14	3.96	260.18
74	264.43	3.05	261.38
75	263.89	9.75	254.14
76	264.00	1.22	262.78
77	265.04	0.00	265.04
78	264.13	1.22	262.91

The rock probes indicate that the underlying probable bedrock depth varies between 1.22 metres (RP 68,76 & 78) to 10.67 metres (RP 67) below the existing grades within the proposed building footprint. At RP 77, the bedrock was exposed.

It also indicates that the underlying probable bedrock subgrade generally slopes in a south east direction (towards RP63) dropping from a high of 262.91 m (RP78) to a low of 252.58 m (RP63) with some peaks (RP 77) and valleys (RP 75) that were noted.

The average depth of the probable bedrock is in the range of 3.47 metres (elevation 260.92 metres).

4.2 Boreholes

Details of the subsurface conditions encountered at the site are summarized below. The subsurface soil and groundwater conditions encountered in the boreholes are presented on the attached Log of Borehole sheets in Appendix B.

It should be noted that the subsurface conditions are confirmed at the borehole locations only. The stratigraphic boundaries indicated on the Log of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance typically represent a transition from one soil or rock type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes, and will vary between and beyond the borehole locations. The following discussion has been simplified in terms of the major soil and rock strata for the purposes of geotechnical design. 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 that could affect construction costs, techniques, equipment and scheduling.

All of the soil samples that were retrieved from this geotechnical investigation were tested in our soils laboratory to determine the water contents. In addition, grain size analysis and Atterberg Limits were conducted on selected soil samples. The results of this soil testing is presented in Appendix C.

4.2.1 Soil Stratigraphy

In general, fill materials were encountered in both boreholes. The fill materials extended up to 1.52 metres below the existing grades.

BH1 The upper stratum of fill material consisted of a brown to red compact dry SAND, GRAVEL and pieces of brick which extended up to 0.76 metres below the existing grades. The upper stratum of fill was underlain by a dense dark brown gravelly, silty SAND, trace clay Fill stratum that was moist and approximately 0.61 metres thick. Split spoon refusal was recorded at a depth of 1.37 metres. The gravelly, silty SAND stratum was underlain by bedrock consisting of dark grey Gabbro that had a good (RQD = 90%) to fair (RQD = 60%) quality and extended to the full depth of the borehole of 4.42 metres.

BH 2 The upper stratum of fill material consisted of a dark brown loose dry Sand, Gravel some silt which extended up to 0.76 metres below the existing grades. The upper stratum of fill was underlain by a loose brown sandy, silty GRAVEL, trace clay fill stratum that was moist and approximately 0.76 metres thick. The sandy, silty GRAVEL stratum was underlain by a loose light grey Clayey SILT, trace gravel, trace sand stratum that was wet and approximately 0.77 metres thick. The Clayey SILT stratum was underlain by a compact to dense brown SILT, trace clay, trace sand, trace gravel stratum that was wet and approximately 1.37 metres thick. Split spoon refusal was recorded at a depth of 3.66 metres. The Silt, trace clay, trace sand stratum was underlain by bedrock consisting of medium grey coloured Gabbro that had a fair (RQD = 62%) to good (RQD = 82%) quality and extended to the full depth of the borehole of 6.71 metres.

The following testing was conducted on representative soil samples:

1. Moisture contents.
2. Soil Gradations (hydrometers).

The following table presents the soil stratigraphy encountered at each borehole location:

Borehole Soil Stratigraphy

Borehole (Elev.)	Depth (m)	Subgrade Description	SPT Values 'N' or RQD %	Water Content %
BH1 (264.06)	0.00 - 0.76 0.76 - 1.37 1.37 - 2.90 2.90 - 4.42	1 - Fill - SAND, GRAVEL, brick, brown, moist, compact 2 - Fill - Gravelly, silty SAND, trace clay, dark brown, moist, dense 3 - Bedrock - Good quality dark grey Gabbro 4 - Bedrock - Fair quality dark grey Gabbro	49 90 % 60 %	16 18

Borehole (Elev.)	Depth (m)	Subgrade Description	SPT Values 'N' or RQD %	Water Content %
BH2 (264.08)	0.00 - 0.76	1 - Fill - SAND, GRAVEL, some silt, dark brown, dry, loose		14 17
	0.76 - 1.52	2 - Fill - Sandy, silty GRAVEL, trace clay, trace roots, brown, moist, loose.	8	22
	1.52 - 2.29	3 - Clayey SILT, trace gravel, trace sand, light grey, wet, loose	7	23
	2.29 - 3.66	4 - SILT, trace to some clay, trace sand, trace gravel, brown, wet, compact to dense	14 - 37	
	3.66 - 5.18	5 - Bedrock - Fair quality medium grey Gabbro	62 %	
	5.18 - 6.71	6 - Bedrock - Good quality medium grey Gabbro	82 %	

4.2.2 Bedrock Cores

The bedrock core retrieved from BH1 generally consist of an excellent to fair quality dark grey Gabbro (Sudbury Event, Mafic Intrusive Rocks, Nipissing Intrusive Rocks Group formation [1]).

The bedrock core retrieved from BH2 generally consist of a fair to good quality medium coloured grey Gabbro (Sudbury Event, Mafic Intrusive Rocks, Nipissing Intrusive Rocks Group formation[1]) that had been cleaned with compressed air to remove all loose debris and rock.

4.4 Groundwater

Based on the current site conditions, the current excavation filled up with surface water based on the depth of the excavation located up to 9.24 metes or more below Paris Street. We would estimate the groundwater table to be located approximately 1.45 metres (in BH 2) below the existing grade to near the bedrock surface interface (BH 1) with local perched areas depending on the permeability of the underlying native soils.

It should be noted that the ground water table is expected to fluctuate seasonally with higher levels expected during the spring and fall seasons.

[1] Ministry of Natural Resources, Ontario Geological Survey, Map 2491, Sudbury Geological Compilation, 1984.

5.0 GEOTECHNICAL DESIGN

The following discussions and recommendations are based on the factual data obtained from the investigation, and are presented for guidance of the design professionals only. The comments pertain to a specific project and location. This report is provided on the basis of these terms of reference and on the assumption that the preliminary 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 relevant to the interpretation 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.

Comments about construction are presented only to bring attention to aspects which might impact the design. Contractors bidding on or conducting work associated with this project should review the factual data presented in the preceding sections of the report, to assess their effect on proposed construction methods and scheduling.

5.1 Frost Protection

For the Sudbury area, the required frost protection is 1.80 metres of soil cover. As such, all exterior foundations and grade beams in unheated and heated areas constructed on undisturbed native soils or engineered fills must be provided with a minimum of 1.80 metres of earth cover for frost protection or alternative equivalent insulation in the City of Greater Sudbury. If required, Terraprobe can provide recommendations on the required equivalent insulation.

Footings and exterior columns placed on bedrock surfaces are not subjected to frost heave provided the footings are doweled into the bedrock.

5.2 Foundation Design - Underground Parking Garage Building

For this project, the proposed elevation for the underground parking garage first floor is in the range of 264.00 metres. The current excavation plateau elevation (based on the rock probe locations) was in the range of 263.39 metres (RP62) to 266.09 metres (RP67). This indicate that some excavation will be required to construct the underground parking garage foundation system.

For this project, we anticipate that some drilling and blasting will be required along the west and south sections of the building footprint. Allowances should be made for overbreak conditions. Due consideration should also be given to controlled blasting procedures in order to prevent potential damage to the surrounding environment. All blasts must be monitored and conducted as per the latest version of the Occupational Health and Safety Act and Regulations for Construction Projects (Part II- General Construction, Sections 196- 206).

In addition, we would recommend that a pre-blast survey (as per OPSS 120.07.03) of all neighbouring properties should be undertaken prior to conducting some drilling and blasting activities. The preconstruction survey will serve to protect the client from claims unrelated to the construction activities in the development of this property.

For this project, we recommend placing the underground garage and condominium building foundation system on:

A. On a series of micro piles advanced into the underlying bedrock subgrade in the deep bedrock areas. In the case of the micro pile, a steel casing is advanced and socketed into the underlying bedrock subgrade. The bedrock is then cored to a pre-determined depth based on the building loads and the entire column is filled with a grout mixture and reinforced with a Dywidag Threadbar® sized for the application.

   The number and size of the piles (and type) are determined based on the building loads and configuration. The design of the micro piles would be provided by the supplier in conjunction with the probable bedrock subgrade depth provided by Terraprobe in this report. Depending on the micro pile supplier, the grade beam and pile caps can also be designed from their engineer team.

B. Directly on the exposed bedrock in the areas of the exposed shallow bedrock subgrade.

5.3 Underlying Bedrock Characteristics

As noted in section 4.3, and based on local geological maps produced by the Ontario Geological Survey the local bedrock in the vicinity of the condominium development consist of a medium grey coloured to dark grey Gabbro.

The Gabbro bedrock can be assumed to have a unit weight, γ, of 26.50 kN/m^3 and a buoyant unit weight, γ’ (γ_{Gabbro} - γ_{water}), of 16.70 kN/m^3.

The Bulk Modulus of a Gabbro that can be utilized for design would be in the range of 50 GPa.

5.3.1 Coefficient of Friction on Bedrock

The coefficient of friction angle between the underside of a cast in place concrete footing and a relatively rough bedrock surface can be taken as tan φ of 43^0 (0.93) and for a smooth bedrock surface can be taken as tan φ of 30^0 (0.577).

5.3.2 Rock Anchors - Allowable Bond Stress

If rock anchors are required to provide additional uplift or lateral capacity, then the structural engineer will design the length and diameter of the rock anchors based on the bedrock characteristics. For rock anchors established in bedrock, three predominant modes of failures can occur:

1. Failure can occur between the grout and the dowel;
2. Or failure can occur between the grout and the rock.
3. The third mode would consist of a quasi-conical rock mass failure.

Field testing (pull out tests) have indicated that the bond developed between the grout and the dowel is typically twice that of the bond developed between the grout and the rock. Therefore, the design analysis should be based on the failure between the grout and the bedrock interface.

The allowable bond stress should be smaller than 1/30 times the unconfined compressive strength of the bedrock and the compressive strength of the grout material whichever is less and should not exceed 1.3 MPa. From previous knowledge of the bedrock in this area, a relatively conservative unconfined compressive strength of approximately 1.0 MPa may be used. The required bond length (L) for the anchor is a function of the core hole diameter (d) and can be calculated as follows:

$$\mathbf{L = P / (\pi)\, x(d)\, x(\tau_b)}$$

$L =$ length (m)
$P =$ working capacity of the anchor (kN)
$\tau_b =$ working bond stress (kPa)
$d =$ diameter of core hole (m)

Usually, the upper 300 mm of the bedrock, is not normally considered part of the bond length since this area is usually weathered/fractured. In this region, we usually assume that the ultimate bond strength will not develop based on the above calculation.

During construction, we recommend testing up to 10% of the rock anchors by conducting a pull out test to confirm the design strengths.

5.3.4 Bedrock Bearing Capacity

Some footings or grade beams may bear directly on the exposed shallow bedrock subgrade.

Foundations placed directly on bedrock should be established on a relatively level rock surface, i.e. generally sloping at an angle of less than approximately 10^0 from the horizontal. In some instances, foundation bases can be placed on bedrock sloping at angles up to 25^0 to 30^0 from the horizontal, provided dowels are incorporated to resist shear. Dowels should consist of a minimum 25M bar embedded a minimum of 1.0 metres into the underlying bedrock subgrade and grouted or epoxied. The spacing of the anchors can vary between 600 mm to 800 mm depending on the slope. Where rock slopes are at steeper angles, the rock surface is to be levelled to provide a stepped footing base.

As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide level benching over these areas by pouring lean concrete (minimum 10 MPa) prior to constructing the foundations. This decision is made on site, since each situation will depend on site specific bedrock conditions.

The Rock Quality Designation (RQD) of the cores that were retrieved ranged between 60% (fair) to 90% (good). Based on the lower bound RQD, the bearing capacity of the underlying bedrock would be in the range of 35 MPa (ULS).

Serviceability Limit States (SLS) does not apply for shallow foundations bearing directly on bedrock since the loads required for unacceptable settlements to occur would be much larger than the factored resistance at the Ultimate Limit States (ULS). Foundations installed in accordance with the above recommendations would be expected to experience very little settlements limited to the elastic deformation of the concrete.

5.4 Underground Parking Garage Foundation Grade Beams

It is anticipated that the grade beams will be supported by pile caps cast over the micro piles. In certain locations, it is anticipated that the bedrock will need to be drilled and blasted to accommodate the underground garage basement slab and foundation system. At these locations, the grade beams could bear upon exposed bedrock or on concrete columns bearing on the exposed bedrock. These transition zones would need to be designed once the final excavation elevation is completed.

Prior to pouring the concrete for the grade beams, the footing areas (original ground or engineered fill pad if applicable) should be cleaned of all deleterious materials such as topsoil, fill, softened, disturbed or caved materials, as well as any standing water.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided.

5.5 Underground Parking Garage Basement Slab

The current overburden soil that were assessed from the borehole investigation indicate some loose fill materials underlying some compact Silt soils. We are also aware that some of the fill materials consists of deleterious fill materials (bricks, concrete blocks) that were placed in the centre of the excavation to permit access to the site to enable the drilling of the rock probes.

We recommend that the underground garage basement slab should be designed as a structural slab (not bearing on the subgrade soils) by transferring the weight to the grade beams.

In areas were shallow bedrock is exposed, a section of the underground garage basement slab may be designed to bear upon an engineered fill placed over dense till soils or exposed sound bedrock.

5.5.1 Engineered Fill Placement

The engineered fill should consist of a Granular B Type II (OPSS MUNI 1010) placed in 150 mm lifts and compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD).

The engineered fill would be placed over the undisturbed dense till soils or bedrock subgrade. At the foundation level, sufficient engineered fill shall be constructed to ensure that it extends at least a distance equal to the full depth of the engineered fill laterally beyond the edge of any foundations, and that it extends outward within an area defined by a 1 to 1 line downward from the edge of any engineered fill.

Full time monitoring of the placement and compaction of the engineered fill is required for each lift of engineered fill. For a well graded blast rock fill and Granular B Type II, witnessing the chinking on a full time basis would be utilized to verify and approve the compactive effort.

5.6 Building Foundation Drainage

To assist in maintaining the building foundations dry from surface water seepage, it is recommended that exterior grades around the building be sloped away at a 2% gradient or more, for a distance of at least 2.0 metres. Roof drains should discharge a minimum of 1.5 metres away from the structure to a drainage swale or appropriate drainage outlet.

Since the underground garage building will consist of a basement, exterior perimeter foundation drains are required to drain the south west and north sides of the building. The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated pipe surrounded by a 19 mm diameter clearstone gravel (OPSS 1004) with a minimum cover of 150 mm (OBC section 9.14.3, Division B, pg B9-60). The perimeter weeping tile would drain into a sump pit located in the basement area of the underground garage. The perimeter foundation drains should discharge towards the rear section of the property to a swale or suitable drainage outlet. The perimeter drain installation and outlet considerations must conform to the Ontario Building Code and plumbing code requirements.

The exterior foundation backfill should extend a minimum lateral distance of 600 mm out from the foundation wall and grade beam and should consist of free-draining granular material, such as a Granular B Type I (OPSS 1010) or suitable alternative drainage cellular media. Since the garage parking structure will be constructed underground, the foundation walls will need to be water proofed (water stop detail).

5.7 Re-use of Excavated Material & General Backfill

Any topsoil/organic, fill and deleterious materials (building materials such as brick, concrete blocks, etc.) encountered at the site should not be reused as backfill in settlement sensitive areas, such as beneath the floor slabs, pavements and trench backfill areas. Theses material may be stockpiled and reused for landscaping purposes provide it is environmentally suitable to do so or removed from the site for disposal.

All backfill materials should consist of free draining material such as a Granular B Type I or Granular B Type II (OPSS MUNI 1010) which can be readily compacted. In settlement sensitive areas, such as beneath pavements and trenches, the backfill should be placed in lifts of 150 mm or less and compacted to a minimum of 100% of its SPMDD. It is recommended that inspection and testing be carried out during construction to confirm trench backfill quality, thickness and to ensure adequate compaction.

Should construction be conducted during the winter season, it is imperative to ensure that frozen material is not utilized as trench backfill.

5.8 Pipe Bedding

The buried services should be placed on conventional Class 'B' granular bedding as per the City of Greater Sudbury GSSD-1227.010 specifications for sewer pipes & water mains for good ground conditions. The granular bedding would be placed over an engineered fill or undisturbed native soils. In the case of a soil trench, where disturbance of the trench base has occurred, such as due to groundwater seepage, or construction traffic, the disturbed soils should be sub-excavated and replaced with suitably compacted granular fill.

Bedding details should conform to the latest version of the City of Greater Sudbury GSSD-1227.010 specifications.

5.9 Trench Backfill

Trench backfill above the springline of the pipe should conform to the latest version of the City of Greater Sudbury GSSD-1227.010 specifications. Backfilling of narrow trenches can be accomplished by reusing the excavated soils (provided they are not too wet) above the springline of the pipe to the underside of the roadway subbase materials provided the moisture content is maintained within 2% of optimum moisture content. If the native soils prove difficulty to compact with vibratory compaction equipment, it is recommended that a free draining material such as Granular B Type I (OPSS MUNI 1010) be used.

All fill should be placed in 150 mm lifts and compacted to a minimum of 95 percent Standard Proctor Maximum Dry Density (SPMDD). It needs to be noted that post-compaction settlement of fine grained fills on the order of 0.5 to 1.0 percent of the total height are common, even when adequately placed to specified compaction. It is best to schedule deep fill placement as far in advance of finish surfacing as possible for best grade integrity.

5.10 Earthquake Design Parameters

The current Ontario Building Code stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4A of the OBC (2006).

The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity measurements have been taken or alternatively estimated on the basis of rational analysis of undrained shear strength or penetration resistance.

$$\upsilon_{s\text{-}avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}} \qquad s_{u\text{-}avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{s_{ui}}} \qquad N_{avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Shear wave velocity | **Undrained shear strength** | **SPT N-values**

At this site, it is known the upper soil stratigraphy consists up to 3.0 metres or greater of soil with a loose to compact relative density with estimated average standard penetration resistance N values of less than 15. It is known that the deeper stratigraphy in this area is at least as competent as the existing stratum and that the competent bedrock consisting of igneous and metamorphic rocks could lie at depths of up to 10.67 metres (RP information) or greater below the existing grades.

In order to classify the bedrock as a Class A or B, the shear wave velocity of the actual bedrock formation must be measured on the site or on profiles of the same bedrock with equal or greater degree of weathering and fracturing. For this project, Terraprobe did not measure the shear wave velocity as part of the scope of work.

For a building designed to bear on micro piles driven into the underlying bedrock subgrade, the site designation for seismic analysis is Class C.

According to Tables 4.1.8.4.B and 4.1.8.4.C. of the same code, the applicable acceleration and velocity based site coefficients are tabulated below.

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
C	1.0	1.0	1.0	1.0	1.0

Site Class	**Values of F_v**				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
C	1.0	1.0	1.0	1.0	1.0

Values of F_a and F_v can be linearly interpolated for intermediate values of S_a between 0.2 and 1.0.

6.0 DESIGN CONSIDERATIONS FOR CONSTRUCTIBILITY

6.1 Site Work

It is recommended that the geotechnical aspects of the proposed works outlined within, be completed under appropriate geotechnical supervision to routinely check such items as subgrade preparation, fill compaction and material physical characteristics for compliance with the various recommendations and specifications presented within.

As noted, it is anticipated that some excavation for the services and underground parking garage foundations will require drilling and blasting in bedrock. Allowances should be made for overbreak conditions. Due consideration should also be given to controlled blasting procedures in order to prevent potential damage to the surrounding environment. All blasts must be monitored and conducted as per the latest Occupational Health and Safety Act and Regulations for Construction Projects (currently Nov. 1993, Part II- General Construction, Sections 196- 206).

In addition, we would recommend that a preconstruction survey of all neighbouring properties should be undertaken prior to conducting some drilling and blasting activities. The preconstruction survey will serve to protect the client from building damage claims unrelated to the construction activities in the development of this property.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the exposed soil in the foundation excavations and concrete must be provided.

6.2 Excavations

Where workmen must enter excavations carried deeper than 1.20 metres, the trench excavations should be suitably sloped and/or braced in accordance with the latest version of the Occupational Health and Safety Act and Regulations for Construction Projects (Part III - Excavations, Section 226). Alternatively, the excavation walls may be supported by bracing or close shoring or a trench box.

The Occupational Health and Safety Act recognizes four (4) broad classifications of soils, which are summarized as follows:

TYPE 1 SOIL

a. is hard, very dense, and only able to be penetrated with difficulty by a small sharp object;
b. has a low natural moisture content and a high degree of internal strength;
c. has no signs of water seepage; and
d. can be excavated only by mechanical equipment.

TYPE 2 SOIL

a. is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object;
b. has a low to medium natural moisture content and a medium degree of internal strength; and
c. has a damp appearance after it is excavated.

TYPE 3 SOIL

a. is stiff to firm and compact to loose in consistency or is previously excavated soil;
b. exhibits signs of surface cracking;
c. exhibits signs of water seepage;
d. if it is dry, may run easily into a well-defined conical pile; and
e. has a low degree of internal strength.

TYPE 4 SOIL

a. is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength;
b. runs easily or flows, unless completely supported before excavating procedures;
c. has almost no internal strength
d. is wet or muddy; and
e. exerts substantial fluid pressure on its supporting system.

Based on our previous test pit investigation report conducted at the site on October 1, 2013 (File No. 52-13-8196) and entitled:

Proposed Excavation Slope Stability Comments
St Joseph Hospital Building Demolition
700 Paris Street, Sudbury, Ontario

we would classify the compact fill materials (sand & gravel) and native soils (Silt and Sand) as a Type 3 soils above the groundwater table and Type 4 soils below under these guidelines.

Based on Type 3 soils; the excavations will need to be sloped at a minimum gradient of 1 horizontal to 1 vertical from the bottom of the excavation.

Based on Type 4 soils; the excavations will need to be sloped at a minimum gradient of 3 horizontal to 1 vertical from the bottom of the excavation.

Alternatively, the excavations may be shored by a support system complying with sections 235, 236, 237, 238, 239 and 241 under O. Reg. 231/91, s 234(1).

6.3 Anticipated Ground Water Management

From the observed water levels located in the middle section of the site, it is expected that some surface water could enter any temporary excavations for the grade beam and pile installations depending on the time of the year the construction takes place.

Generally, groundwater inflow can be controlled to a depth of up to approximately 600 mm below the water table by installing strategically placed sumps and pumping the collected water out of the excavations. Deeper excavations in this type of material will require more positive control, such as through well points and/or interlocking steel sheet piles. It is noted that excavations carried below the water table in cohesionless soil (silt, sand, sand and gravel) will experience loosening and sloughing of the base and sides, unless the ground water level is lowered first.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not undermine any adjacent structures. The contractor should submit their proposal to the prime consultant for review and approval prior to construction. A permit to take water may be required from the Ministry of the Environment. It is the responsibility of the contractor to make this application as required and any other applications from other Ministries or authorities as required (DFO, Conservation authorities, etc.).

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential adverse impacts on the environment.

It should be noted that the water table is expected to fluctuate seasonally with higher levels expected during the spring and fall seasons.

6.4 Temporary Shoring

For this project, it is anticipated that a temporary shoring design will be required to construct the underground parking garage structure along the west and south sides of the excavation limits. Once the building design is finalised, Terraprobe Design can provide this service.

6.5 Horizontal Earth Pressure

If required, walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

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

where:

P = the horizontal pressure at depth, h (m)
K = the earth pressure coefficient,
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)
q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, this equation can be simplified to:

$$P = K[\gamma h + q]$$

This equation assumes that free-draining granular backfill is used and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure.

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 ($\tan \varphi$) expressed as $R = N \tan \varphi$. This is an ultimate resistance value and does not contain a factor of safety.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The average values for use in the design of structure subjected to unbalanced earth pressures at this site are tabulated as follows:

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

Material Types and Strength Properties

Stratum/Parameter	φ	γ	K_a	K_o	K_p
Silt and Clay	26	18.5	0.39	0.56	2.56
Clayey/Sandy Silt or similar Fill	30	18.5	0.35	0.5	3
Silt and Sand/Sand	32	21.5	0.3	0.47	3.22
Granular B Type I (OPSS 1010)	34	21	0.28	0.44	3.54
Granular A (OPSS 1010)	38	22	0.24	0.38	4.2
Granular B Type II (OPSS 1010)	40	23	0.22	0.36	4.6

The values of the earth pressure coefficients noted above are for a horizontal grade behind the wall. The earth pressure coefficients for an inclined grade (retained soil) will vary based on its inclination.

Where permanent drainage for earth retaining walls is not install, hydrostatic pressure acting on the walls must be included in the above calculation; the unit weight of water, $\gamma_w = 9.81\ kN/m^3$. For sloping backfill, the Canadian Highway Bridge Design Code, section C 6.9 should be consulted for the design recommendations.

The surcharge effect from compaction equipment during construction must be taken into account. Where lighter compaction equipment and smaller lifts are used the surcharge effect will be minimized. This should be reviewed in detail by a structural engineer. Permanent earth retaining wall designs are to be carried out in accordance with the latest edition of the Canadian Foundation Engineering Manual and/or the Canadian Bridge Design Code.

6.6 Quality Control

The installation of the piles for the condominium building and any foundation excavations must be monitored by Terraprobe to ensure that the founding bearing capacities achieved are consistent with the design bearing capacity intended by the geotechnical engineer.

The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2, Division B, of the 2006 Ontario Building Code. If Terraprobe is not retained to carry out foundation evaluations 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 design recommendations contained in this report.

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density as determined by ASTM D698. Terraprobe operates a CCIL (Canadian Council of Independent Laboratories) certified aggregates laboratory. In situ determinations of density during fill placement on site are recommended to demonstrate that the specified densities are achieved. Terraprobe is a CNSC licensed 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. For a Granular B Type II (OPSS 1010) witnessing the proof rolling on a full time basis would be utilised to verify and approve the compactive effort.

It has been assumed that concrete for the this structure will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

7.0 STATEMENT OF LIMITATIONS AND RISK

7.1 Procedures

This reports presents geotechnical design recommendations for the constructibility of the proposed condominium development. It does not consider any environmental issues that may or not be present on the site. It is the responsibility of the client to assess any environmental potential issues on this property and was not part of the scope of work for this investigation.

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 geotechnical engineering discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

Any bedrock elevation and ground water observations discrepancies in relation to the findings in the field are not the responsibility of Terraprobe. The client must assume the risk of such description discrepancies findings and be prepared to adjust to potential extra costs to remedy the findings under the direction of Terraprobe. The data presented in the rock probe logs are based on non continuous sampling. There is consequently some interpolation of the probable bedrock depth and indications of changes in stratigraphy as described are therefore approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing program 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 rock probes are similar to those found at the rock probe locations. The conditions that Terraprobe has interpreted to existing between rock probes may differ from those that actually exist.

It may not be possible to advance a sufficient number of rock probes and boreholes and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should 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, cognizant of the risks implicit in the subsurface investigation activities.

7.2 Changes In Site And Scope

It must also 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. Ground water conditions are particularly susceptible to change as a result of season variation and alterations in drainage conditions.

The engineering discussion and design parameters recommendations that have been provided are based on the factual data obtained from the site investigation (consisting of rock probes and exploratory boreholes) conducted by Terraprobe and are intended for use by the owner and their retained designers in the design phase of the project.

Since the project is still in the design stage, all aspects of the project relative to the subsurface conditions cannot be anticipated. If there are changes to the project scope and development features the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant to the revised project or complete.

Terraprobe must be retained to review the implications of changes with respect to the contents of this report and must be retained to review the design drawings and specifications prior to construction.

8.0 CLOSURE

This report was prepared for the express use of our client Michael D. Allen Architect c/o 2226553 Ontario Inc. and their retained design consultants. This report is copyright of Terraprobe and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe.

Michael D. Allen Architect c/o 2226553 Ontario Inc. and their retained design consultants are authorized users.

We trust that the foregoing is sufficient for your present requirements. If you have any questions or if we can be of further assistance, please do not hesitate to contact us.

Yours truly,

Terraprobe Inc.

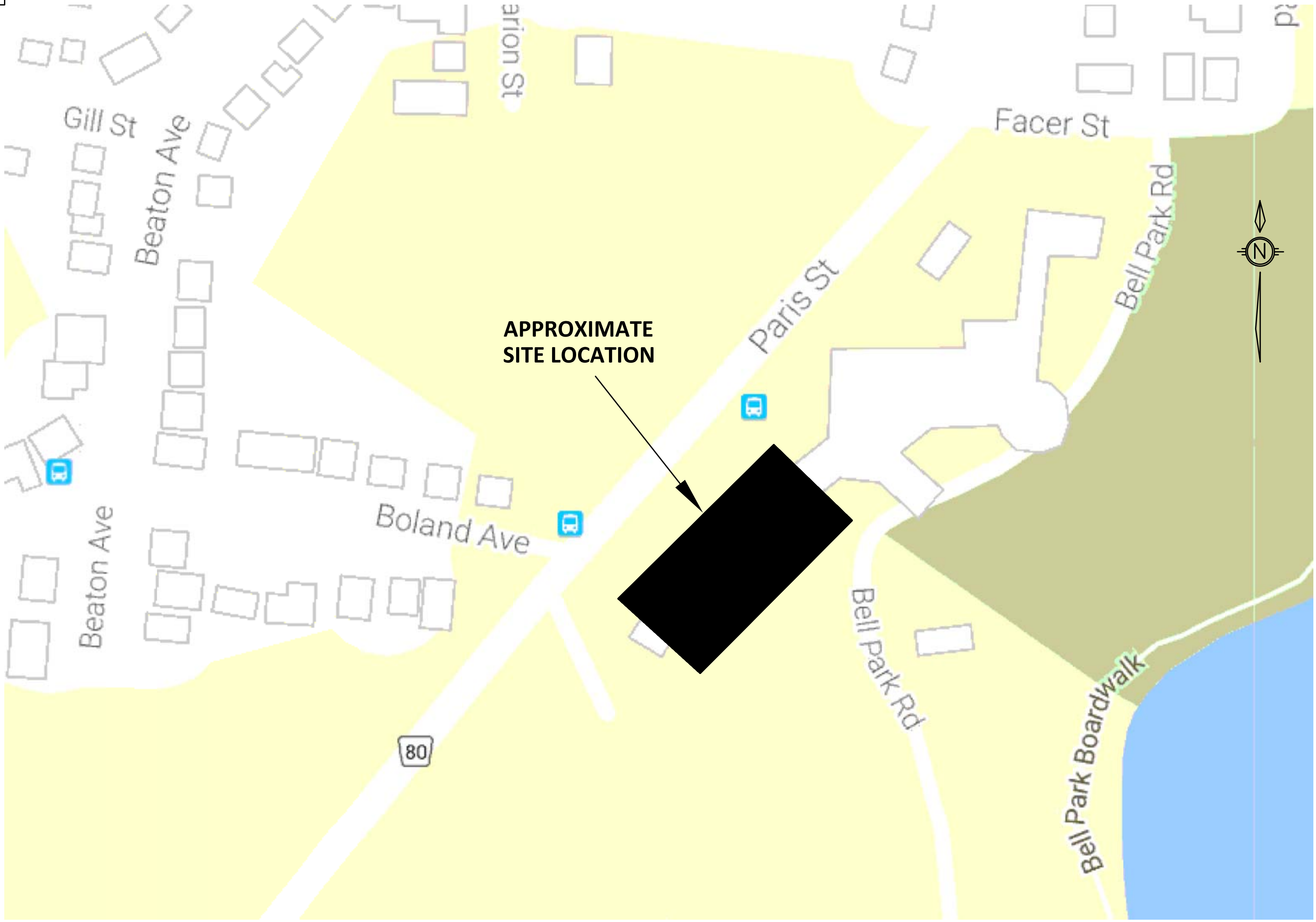
Denis Paquette, P.Eng.
Principal, Sudbury Branch Manager

Denis Paquette

Digitally signed by Denis Paquette
DN: cn=Denis Paquette, o=Terraprobe Inc., ou, email=dpaquette@terraprobe.ca, c=CA
Date: 2016.08.10 22:49:38 -04'00'

FIGURES

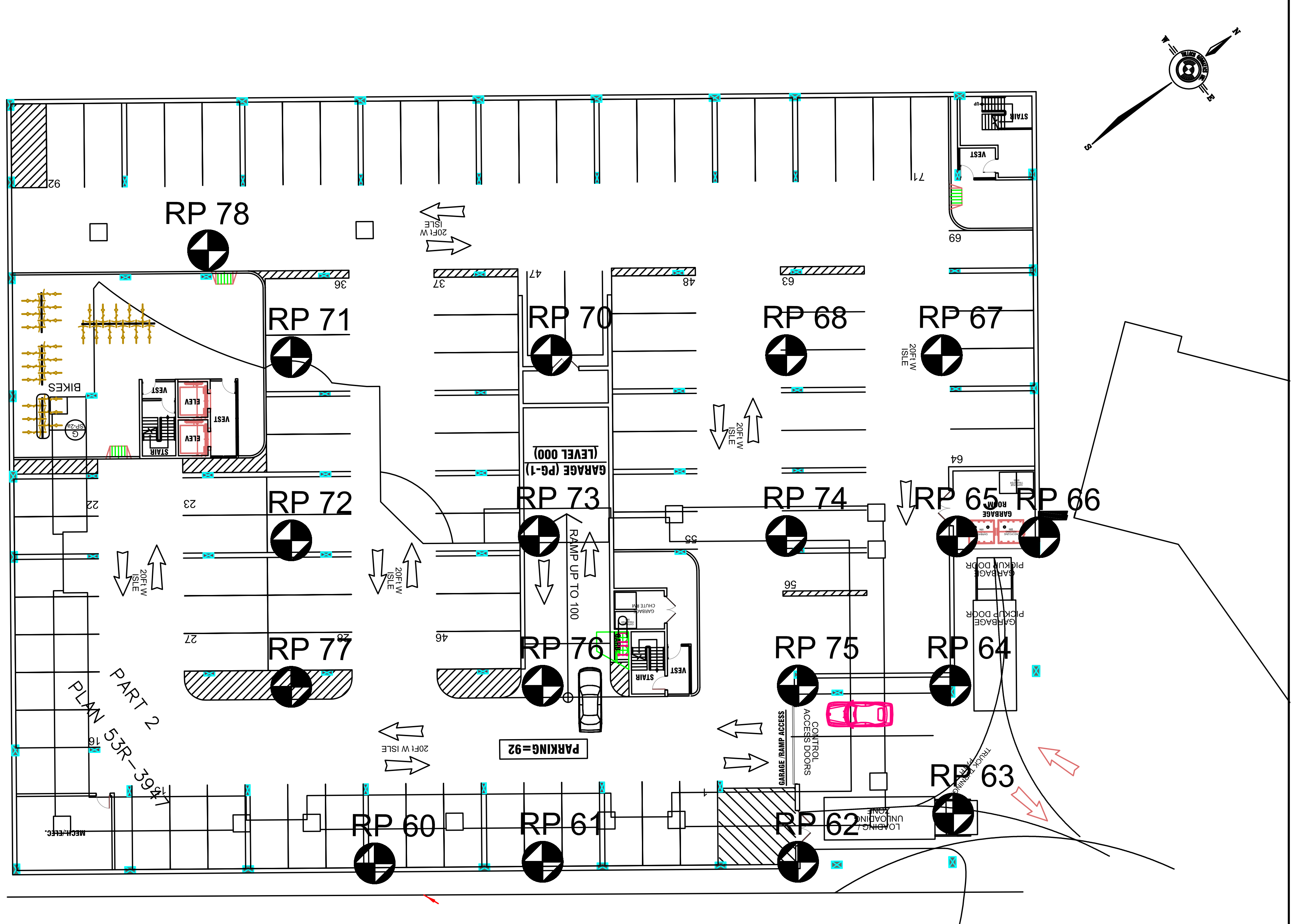
TERRAPROBE INC.



Map provided by Google Maps

NTS

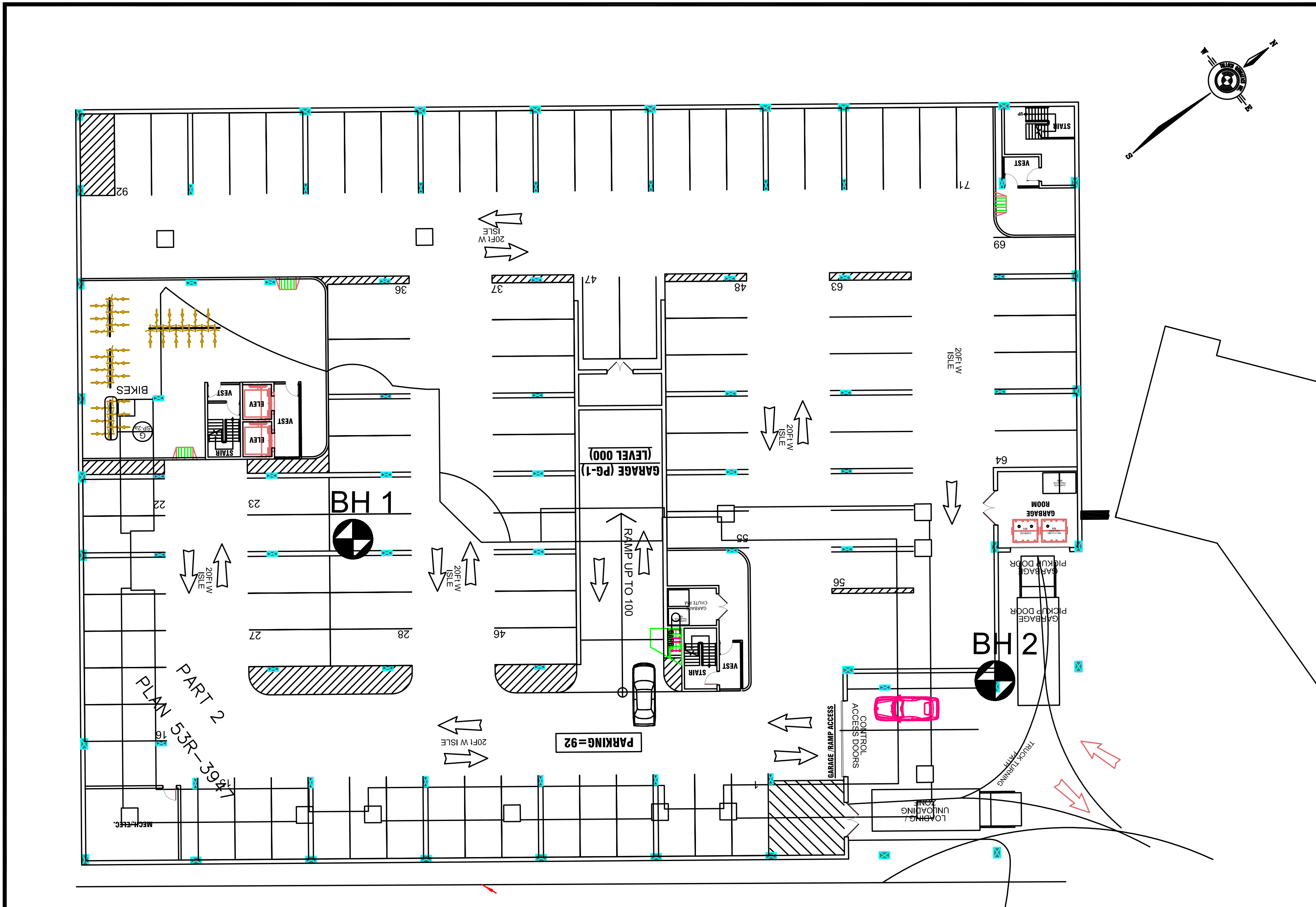
Terraprobe	Title/Name Location: Site Location Plan – Proposed Condominium Development 700 Paris Street, Sudbury, Ontario	Date: July 25, 2016	Project No.: 5–16–0115–01	Figure: 1



Drawing provided by Tuloch

Drawing NTS

Terraprobe	Title/Name Location: Rock Probe Location Plan – Proposed Codominium Development 700 Paris Street, Sudbury, Ontario	Date: November 5, 2014	Project No.: 51–14–9026	Figure: 2



Drawing provided by Tuloch

Drawing NTS

Terraprobe	Title/Name Location: BH Location Plan – Proposed Codominium Development 700 Paris Street, Sudbury, Ontario	Date: June 23, 2016	Project No.: 5–16–0115–01	Figure: 3

APPENDIX A

Rock Probe Logs

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GEOTECHNICAL INVESTIGATION
Rock Probe Logs
Proposed Condominium Development
700 Paris Street
Sudbury, Ontario

ROCK PROBE 60

Location: See Figure 2
Elevation: 264.26 m

DEPTH	DESCRIPTION
0.00 to 3.05 m	Interpreted as granular fill underlain by native soils
3.05 m	Probable bedrock

ROCK PROBE 61

Location: See Figure 2
Elevation: 263.73 m

DEPTH	DESCRIPTION
0.00 to 3.05 m	Interpreted as granular fill underlain by native soils
3.05 m	Probable bedrock

ROCK PROBE 62

Location: See Figure 2
Elevation: 263.39 m

DEPTH	DESCRIPTION
0.00 to 4.57 m	Interpreted as granular fill underlain by native soils
4.57 m	Probable bedrock

ROCK PROBE 63

Location: See Figure 2
Elevation: 263.52 m

DEPTH	DESCRIPTION
0.00 to 10.67 m	Interpreted as granular fill underlain by native soils
10.67 m	Probable bedrock

ROCK PROBE 64

Location: See Figure 2
Elevation: 264.17 m

DEPTH	**DESCRIPTION**
0.00 to 6.10 m	Interpreted as granular fill underlain by native soils
6.10 m	Probable bedrock

ROCK PROBE 65

Location: See Figure 2
Elevation: 265.13 m

DEPTH	**DESCRIPTION**
0.00 to 3.96 m	Interpreted as granular fill underlain by native soils
3.96 m	Probable bedrock

ROCK PROBE 66

Location: See Figure 2
Elevation: 265.17 m

DEPTH	**DESCRIPTION**
0.00 to 2.44 m	Interpreted as granular fill underlain by native soils
2.44 m	Probable bedrock

ROCK PROBE 67

Location: See Figure 2
Elevation: 266.09 m

DEPTH	**DESCRIPTION**
0.00 to 2.44 m	Interpreted as granular fill underlain by native soils
2.44 m	Probable bedrock

ROCK PROBE 68

Location: See Figure 2
Elevation: 264.94 m

DEPTH	**DESCRIPTION**
0.00 to 1.22 m	Interpreted as granular fill underlain by native soils
1.22 m	Probable bedrock

ROCK PROBE 70

Location: See Figure 2
Elevation: 264.96 m

DEPTH	**DESCRIPTION**
0.00 to 1.83 m	Interpreted as granular fill underlain by native soils
1.83 m	Probable bedrock

ROCK PROBE 71

Location: See Figure 2
Elevation: 264.11 m

DEPTH	**DESCRIPTION**
0.00 to 1.52 m	Interpreted as granular fill/native soils
1.52 m	Probable bedrock

ROCK PROBE 72

Location: See Figure 2
Elevation: 264.01 m

DEPTH	**DESCRIPTION**
0.00 to 2.44 m	Interpreted as granular fill underlain by native soils
2.44 m	Probable bedrock

ROCK PROBE 73

Location: See Figure 2
Elevation: 264.14 m

DEPTH	**DESCRIPTION**
0.00 to 3.96 m	Interpreted as granular fill underlain by native soils
3.96 m	Probable bedrock

ROCK PROBE 74

Location: See Figure 2
Elevation: 264.43 m

DEPTH	**DESCRIPTION**
0.00 to 3.05 m	Interpreted as granular fill underlain by native soils
3.05 m	Probable bedrock

ROCK PROBE 75

Location: See Figure 2
Elevation: 263.89 m

DEPTH	DESCRIPTION
0.00 to 9.75 m	Interpreted as granular fill underlain by native soils
9.75 m	Probable bedrock

ROCK PROBE 76

Location: See Figure 2
Elevation: 264.00 m

DEPTH	DESCRIPTION
0.00 to 1.22 m	Interpreted as granular fill underlain by native soils
1.22 m	Probable bedrock

ROCK PROBE 77

Location: See Figure 2
Elevation: 265.04 m

DEPTH	DESCRIPTION
0.00	Exposed Bedrock

ROCK PROBE 78

Location: See Figure 2
Elevation: 264.13 m

DEPTH	DESCRIPTION
0.00 to 1.22 m	Interpreted as granular fill underlain by native soils
1.22 m	Probable bedrock

APPENDIX B

Borehole Logs

TERRAPROBE INC.

Terraprobe Inc.

ABBREVIATIONS, TERMINOLOGY, GENERAL INFORMATION

BOREHOLE AND TEST PIT LOGS

SAMPLING METHOD

SS	split spoon
ST	Shelby tube
AS	auger sample
WS	wash sample
RC	rock core
WH	weight of hammer
PH	pressure, hydraulic

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.).

SOIL DESCRIPTION - COHESIONLESS SOILS

Relative Density	'N' value
very loose	< 4
loose	4 - 10
compact	10 - 30
dense	30 - 50
very dense	> 50

SOIL DESCRIPTION - COHESIVE SOILS

Consistency	Undrained Shear Strength, kPa	'N' value
very soft	< 12	< 2
soft	12 - 25	2 - 4
firm	25 - 50	4 - 8
stiff	50 - 100	8 - 16
very stiff	100 - 200	16 - 32
hard	> 200	> 32

SOIL COMPOSITION

	% by weight
'trace' (e.g. trace silt)	< 10
'some' (e.g. some gravel)	10 - 20
adjective (e.g. sandy)	20 - 35
'and' (e.g. sand and gravel)	35 - 50

TESTS, SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
φ'	angle of internal friction
c'	cohesion shear strength
C_c	compression index

GENERAL INFORMATION, LIMITATIONS

The conclusions and recommendations provided in this report are based on the factual information obtained from the boreholes and/or test pits. Subsurface conditions between the test holes may vary.

The engineering interpretation and report recommendations are given only for the specific project detailed within, and only for the original client. Any third party decision, reliance, or use of this report is the sole and exclusive responsibility of such third party. The number and siting of boreholes and/or test pits may not be sufficient to determine all factors required for different purposes.

It is recommended Terraprobe be retained to review the project final design and to provide construction inspection and testing.

Terraprobe

ROCK CORE 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

Joint Spacing (adapted from *Bieniawski 1989, ISRM 1981*)

Classification	Spacing
very close	< 60 mm
close	60 – 200 mm
moderately close	0.2 to 0.6 m
wide	0.6 to 2 m
very wide	> 2 m

Natural Fracture Frequency (per 0.3 m)
Refers to the number of natural fractures (joints, faults, etc.) which are present per 0.3m. Ignores mechanical or drill-induced breaks, and closed discontinuities (e.g. bedding planes).

Orientation

Orientation	Angle from horiz.
horizontal/flat	0 - 20°
dipping	20 - 50°
vertical	50 - 90°

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	2□ - 25
sandy□and silty, minor clay	1□ - 24
non-softening clays	6 - 12
swelling clay fillings	n/a

Joint Aperture

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

Planarity

- Planar
- Undulating
- Stepped
- Irregular
- Discontinuous

Roughness

- Very rough
- Rough
- Smooth
- Slickensided
- Polished

Coating	Description
clean	no filling
veneer	< 1 mm filling
coating / infill	> 1 mm filling

GENERAL

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

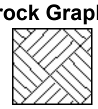
Strength classification (*after Marinos and Hoek, 2001*)

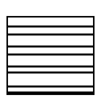
Grade	Term	UCS (MPa)	Field Estimate (Description)
R6	extremely strong	> 250	can only be chipped by geological hammer
R5	very strong	100 - 250	requires many blows from geological hammer
R4	strong	50 - 100	requires more than one blow from geological hammer
R3	medium strong	25 - 50	can't be scraped, breaks 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

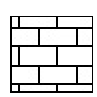
Bedding Thickness (*Quarterly Journal of Engineering Geology, Vol 3, 1970*)

Very thickly bedded	> 2 m	Medium bedded	200 – 600mm	Very thinly bedded	20 – 60mm	Thinly Laminated < 6mm
Thickly bedded	0.6 – 2m	Thinly bedded	60 – 200mm	Laminated	6 – 20mm	

Bedrock Graphic Legend


Inferred bedrock


Shale


Limestone

Terraprobe

LOG OF BOREHOLE 1

PROJECT: Condo Development

DATE: June 25 & 26, 2016

CLIENT: Panoramic

EQUIPMENT: CME 850 Track Mounted

LOCATION: 700 Paris Street, Sudbury, Ontario

ELEVATION DATUM: Geodetic FILE: 5-16-0115-01

DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUE	CORE RECOVERY %	R.Q.D %	WATER CONTENT (%)	STANDPIPE INSTALLATION OR REMARKS
264.06 0.00	Compact Brown/ red Moist FILL: Sand, gravel, pieces of brick	1	AS					
263.30 0.76	Dense Dark Brown Moist Fill – Gravelly, Silty SAND, trace clay	2	SS	49				Estimated Groundwater Table 1.20 metres
262.69 1.37	SS refusal at 1.37m on infered bedrock Bedrock coring commence at the depth of 1.37 m below grade RUN 1 – Good quality Dark grey Gabbro	1	NQ		100%	90%		
261.16 2.90	RUN 2 – Fair quality Dark grey Gabbro	2	NQ		80%	60%		
259.64 4.42	End of Borehole							

PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH kPa: FIELD VANE – + Q – ●; POCKET PEN – * U – ○; 20 40 60 80

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

DEPTH SCALE IN METRES: 0 1 2 3 4 5 6 7 8 9

NOTES: Ground water level not recorded in consideration of use of water for bedrock coring procedures

Terraprobe

LOG OF BOREHOLE 2

PROJECT: Condo Development

CLIENT: Panoramic

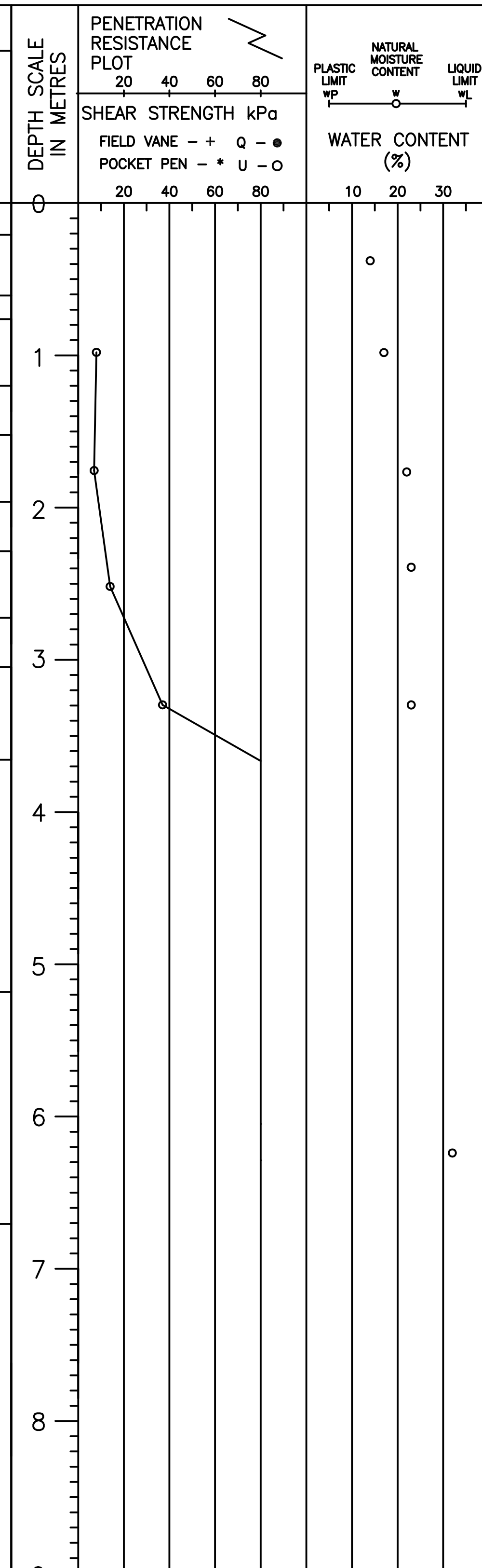
LOCATION: 700 Paris Street, Sudbury, Ontario

DATE: June 25, 2016

EQUIPMENT: CME 850 Track Mounted

ELEVATION DATUM: Geodetic FILE: 5-16-0115-01

DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUE	CORE RECOVERY %	R.Q.D %
264.08 0.00	Loose Dark Brown Dry FILL: Sand, gravel, some silt	1	AS			
263.32 0.76	Loose Brown Moist Fiil – Sandy, Silty GRAVEL, trace clay, trace roots	2	SS	8		
262.56 1.52	Loose Light grey Wet Clayey SILT, trace gravel, trace sand	3	SS	7		
261.79 2.29	Compact Brown Wet SILT, some clay, trace sand	4	SS	14		
	Dense trace clay, trace gravel	5	SS	37		
260.42 3.66	SS refusal at 3.66m on infered bedrock Bedrock coring commence at 3.66 m RUN 1– Fair quality Medium grey Gabbro	1	NQ		89%	62%
258.90 5.18	RUN 2– Good quality Medium grey Gabbro	2	NQ		91%	82%
257.37 6.71	End of Borehole					



STANDPIPE INSTALLATION OR REMARKS

Estimated Groundwater Table 1.45 metres

NOTES:

APPENDIX C

Soil Laboratory Results

TERRAPROBE INC.

Terraprobe

WATER CONTENT TEST FORM

PROJECT: **Condominium Development**
LOCATION: **700 Paris Street, Sudbury, Ontario**
CLIENT: **Michael D. Allen Architect c/o 2226553 Ontario Inc.**

FILE NO.: **5-16-0155-01**
LAB NO.: **6270**
SAMPLE DATE: **July 25, 2016**
SAMPLE BY: **D.T.**
TEST DATE: **August 2, 2016**
TESTED BY: **T.E.**

BOREHOLE NUMBER		**1**	**1**
SAMPLE NUMBER		**1**	**2**
DEPTH OF SAMPLE (m)		**0.2 - 0.5**	**0.76 - 1.22**
WT. OF WET SOIL + TARE (g)	A	101.41	669.90
WT. OF DRY SOIL + TARE (g)	B	91.90	630.80
WEIGHT OF TARE (g)	C	30.65	410.90
WATER CONTENT (%)	A-B/B-C*100	**16%**	**18%**

BOREHOLE NUMBER		**2**	**2**	**2**	**2**	**2**
SAMPLE NUMBER		**1**	**2**	**3**	**4**	**5**
DEPTH OF SAMPLE (m)		**0.2 - 0.6**	**0.76 - 1.22**	**1.52 - 1.98**	**2.29 - 2.75**	**3.05 - 3.51**
WT. OF WET SOIL + TARE (g)	A	83.32	650.80	664.70	668.00	658.80
WT. OF DRY SOIL + TARE (g)	B	76.83	614.80	618.60	619.40	612.40
WEIGHT OF TARE (g)	C	30.55	407.40	411.00	410.70	407.80
WATER CONTENT (%)	A-B/B-C*100	**14%**	**17%**	**22%**	**23%**	**23%**

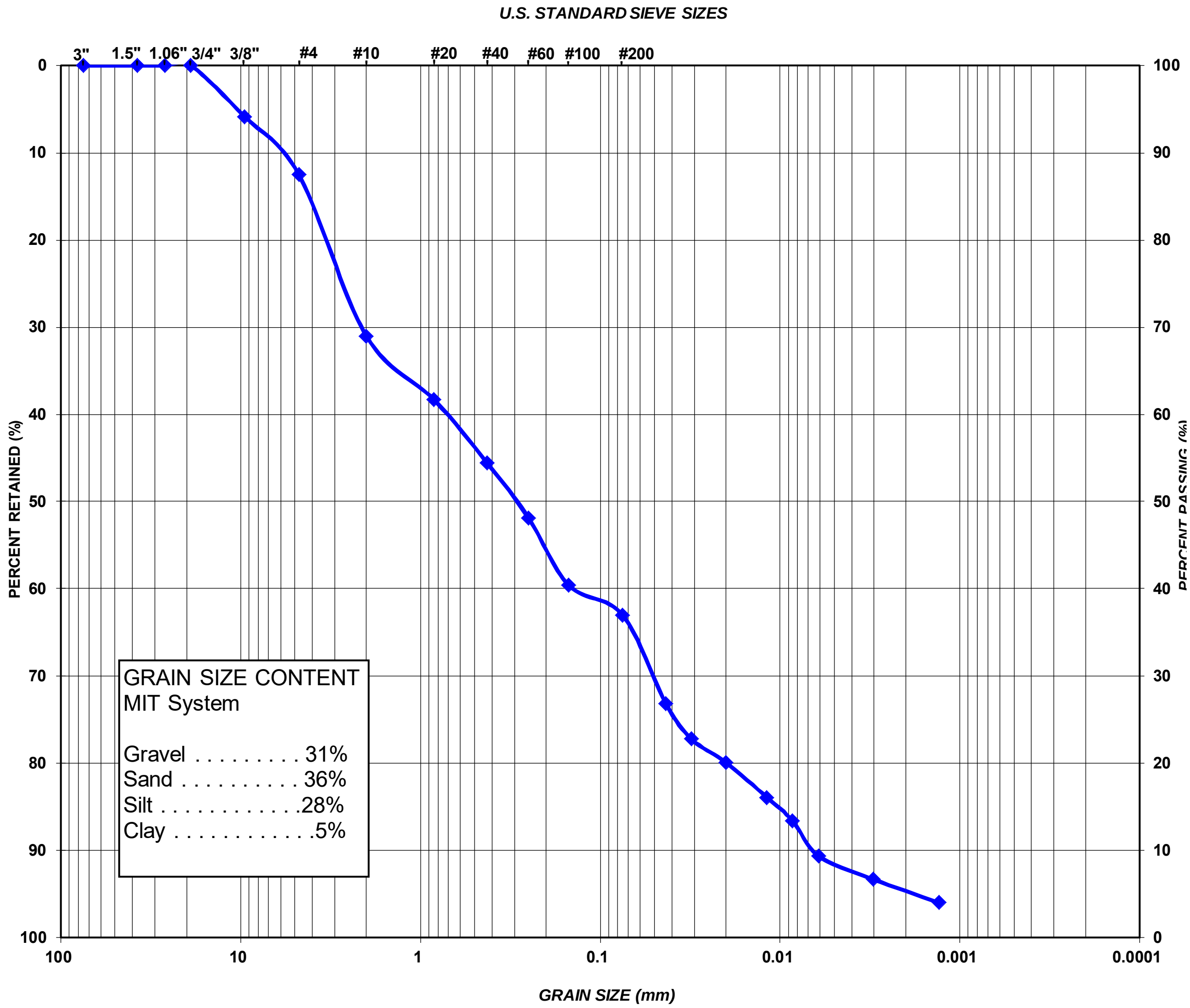
COMMENT:

Terraprobe

HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Condominium Development**	FILE NO.:	**5-16-0115-01**
LOCATION:	**700 Paris Street, Sudbury, Ontario**	SAMPLE DATE:	**July 25, 2016**
CLIENT:	**Michael D. Allen Architect c/o 2226553 Ontario Inc.**	SAMPLED BY:	**D.T.**
BOREHOLE NUMBER:	**1**	TEST DATE:	**August 3, 2016**
SAMPLE NUMBER:	**2**	TESTED BY:	**T.E.**
SAMPLE DEPTH (m):	**0.76 - 1.22**	LAB NO.:	**6270**
SAMPLE DESCRIPTION:	**Gravelly, Silty SAND, trace clay**		

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	CBBLS	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		GRAVEL			SAND				

UNIFIED SYSTEM	CBBLS	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
		GRAVEL		SAND			

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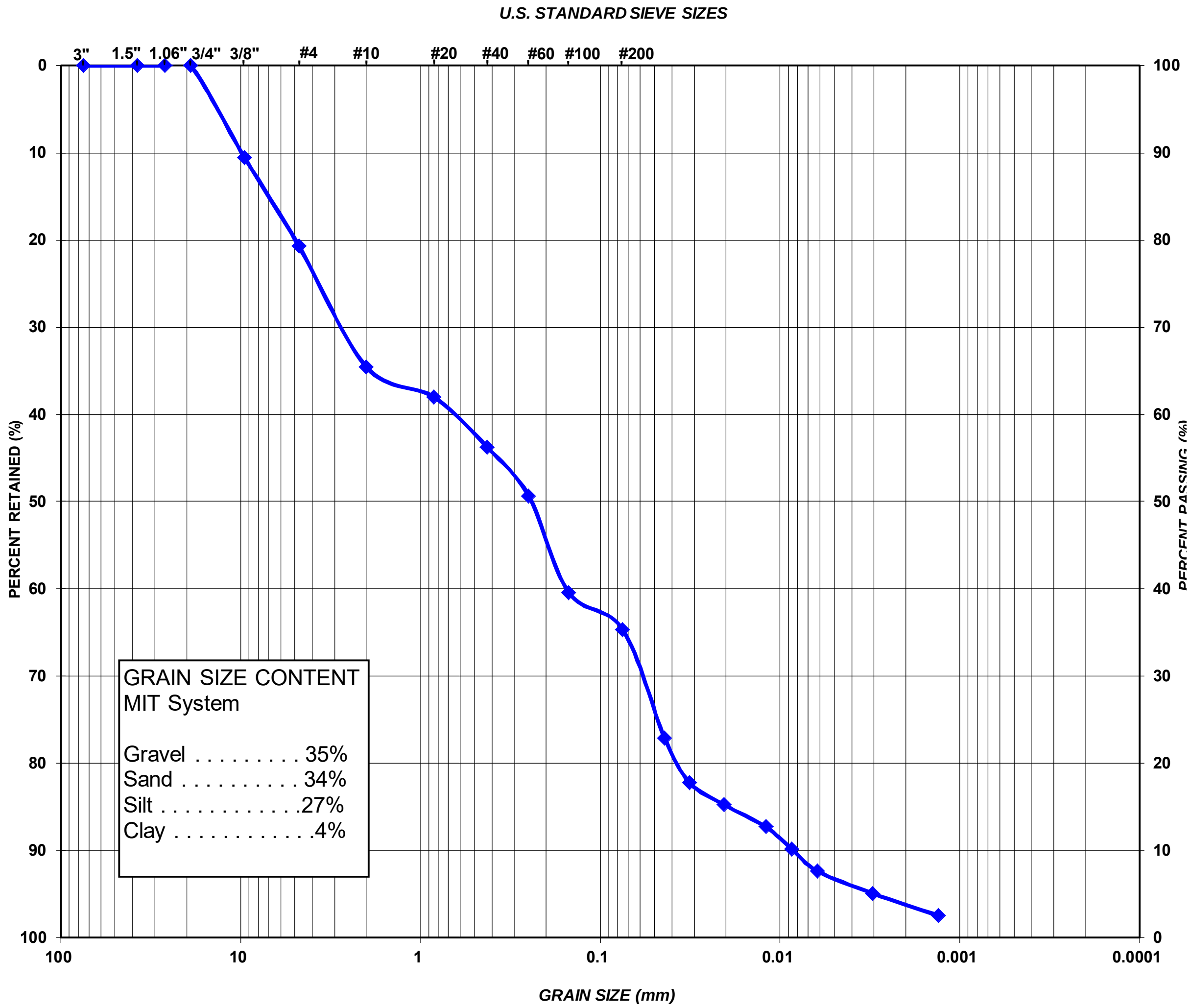
HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Condominium Development**	FILE NO.:	**5-16-0115-01**
LOCATION:	**700 Paris Street, Sudbury, Ontario**	SAMPLE DATE:	**July 25, 2016**
CLIENT:	**Michael D. Allen Architect c/o 2226553 Ontario Inc.**	SAMPLED BY:	**D.T.**
BOREHOLE NUMBER:	**2**	TEST DATE:	**August 3, 2016**
SAMPLE NUMBER:	**2**	TESTED BY:	**T.E.**
SAMPLE DEPTH (m):	**0.76 - 1.22**	LAB NO.:	**6270**
SAMPLE DESCRIPTION:	**Sandy, Silty GRAVEL, trace clay**		

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	CBBLS	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		GRAVEL			SAND				

UNIFIED SYSTEM	CBBLS	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
		GRAVEL		SAND			

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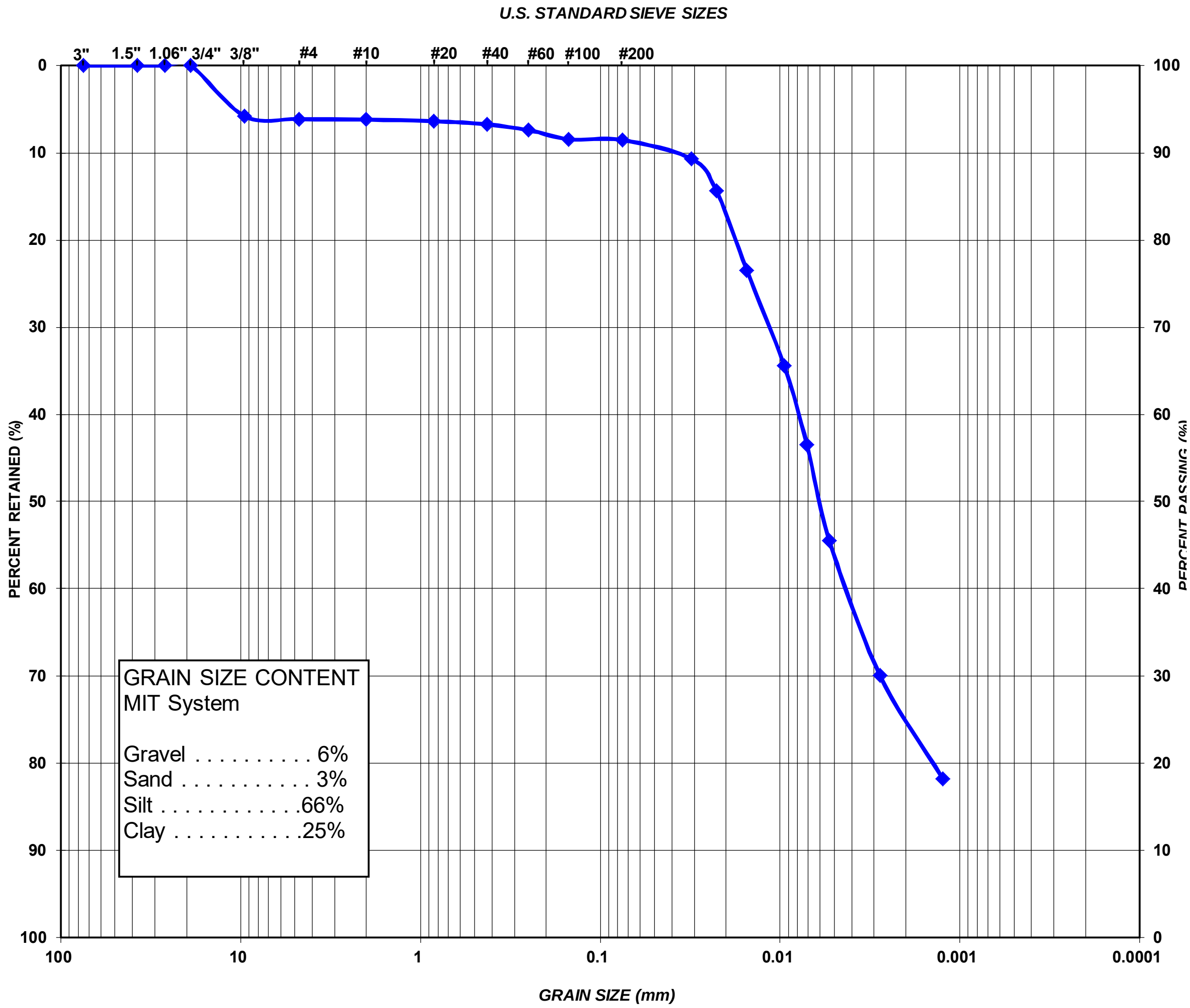
HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Condominium Development**	FILE NO.:	**5-16-0115-01**
LOCATION:	**700 Paris Street, Sudbury, Ontario**	SAMPLE DATE:	**July 25, 2016**
CLIENT:	**Michael D. Allen Architect c/o 2226553 Ontario Inc.**	SAMPLED BY:	**D.T.**
BOREHOLE NUMBER:	**2**	TEST DATE:	**August 3, 2016**
SAMPLE NUMBER:	**3**	TESTED BY:	**T.E.**
SAMPLE DEPTH (m):	**1.52 - 1.98**	LAB NO.:	**6270**
SAMPLE DESCRIPTION:	**Clayey SILT, trace gravel, trace sand**		

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	CBBLS	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		GRAVEL			SAND				

UNIFIED SYSTEM	CBBLS	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
		GRAVEL		SAND			

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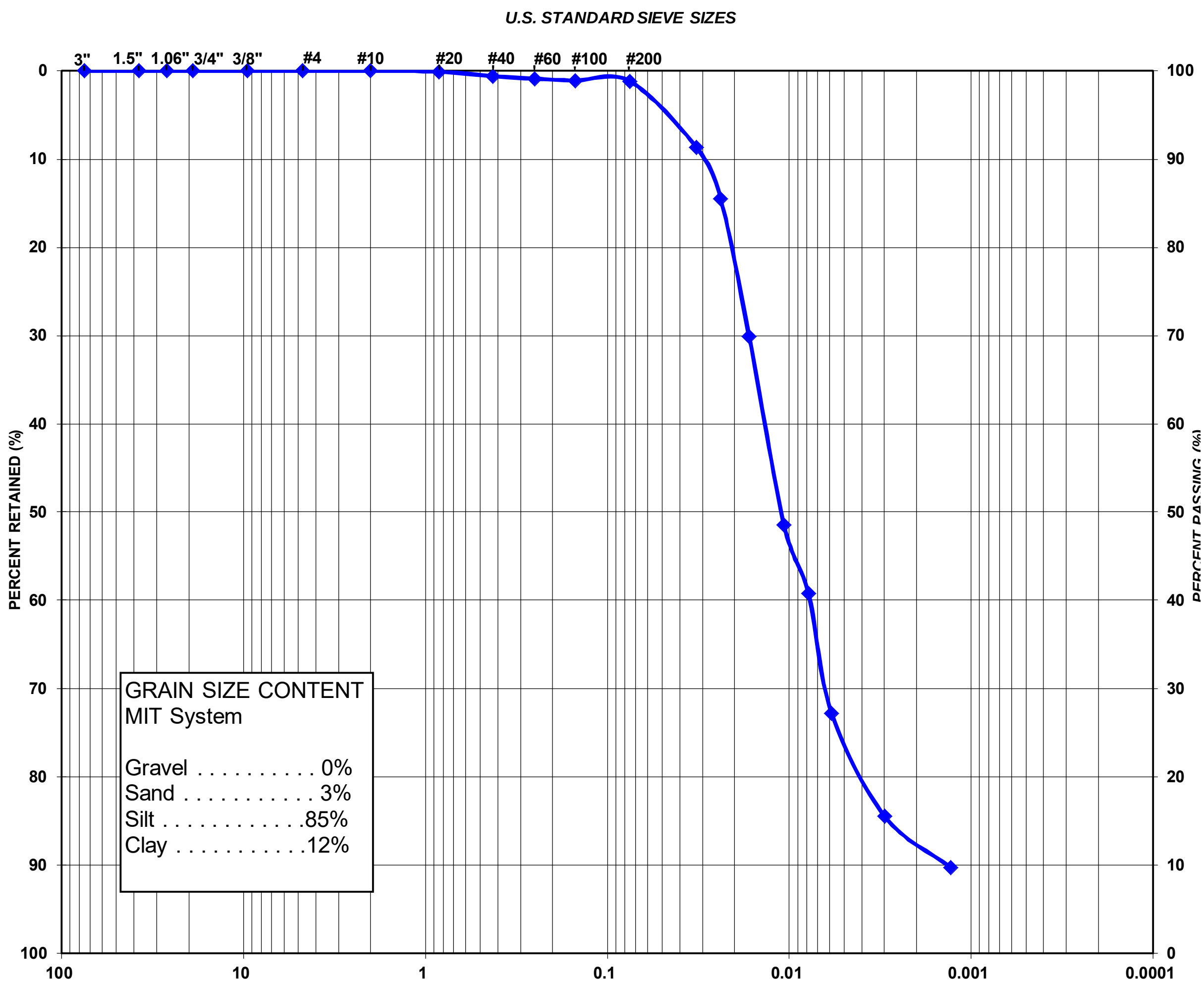
HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Condominium Development**	FILE NO.:	**5-16-0115-01**
LOCATION:	**700 Paris Street, Sudbury, Ontario**	SAMPLE DATE:	**July 25, 2016**
CLIENT:	**Michael D. Allen Architect c/o 2226553 Ontario Inc.**	SAMPLED BY:	**D.T.**
BOREHOLE NUMBER:	**2**	TEST DATE:	**August 3, 2016**
SAMPLE NUMBER:	**4**	TESTED BY:	**T.E.**
SAMPLE DEPTH (m):	**2.29 - 2.75**	LAB NO.:	**6270**
SAMPLE DESCRIPTION:	**SILT, some clay, trace sand**		

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	CBBLS	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		GRAVEL			SAND				

UNIFIED SYSTEM	CBBLS	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
		GRAVEL		SAND			

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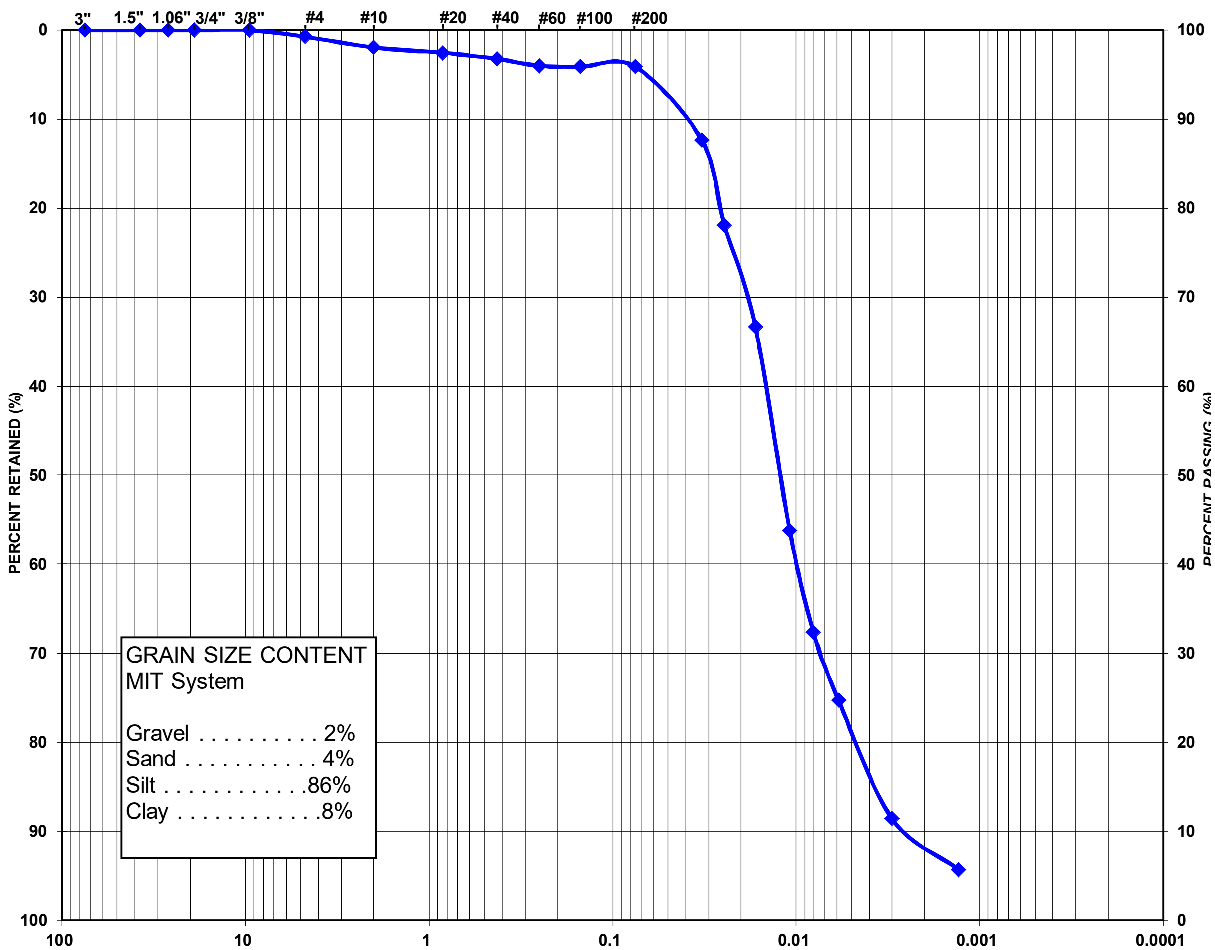
HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Condominium Development**	FILE NO.:	**5-16-0115-01**
LOCATION:	**700 Paris Street, Sudbury, Ontario**	SAMPLE DATE:	**July 25, 2016**
CLIENT:	**Michael D. Allen Architect c/o 2226553 Ontario Inc.**	SAMPLED BY:	**D.T.**
BOREHOLE NUMBER:	**2**	TEST DATE:	**August 3, 2016**
SAMPLE NUMBER:	**5**	TESTED BY:	**T.E.**
SAMPLE DEPTH (m):	**3.05 - 3.51**	LAB NO.:	**6270**
SAMPLE DESCRIPTION:	**SILT, trace clay, trace sand, trace gravel**		

GRAIN SIZE DISTRIBUTION

U.S. STANDARD SIEVE SIZES



GRAIN SIZE (mm)

MIT SYSTEM	CBBLS	COARSE GRAVEL	MEDIUM GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

UNIFIED SYSTEM	CBBLS	COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT AND CLAY