Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
PROPOSED FINANCE AND NPB HEARING BUILDING
BEAVER CREEK INSTITUTION
TOWN OF GRAVENHURST, ONTARIO

Prepared For: Public Works and Government Services Canada
Ontario Region
Professional & Technical Program
4900 Yonge Street
Toronto, Ontario
M2N 6A6

Attention: Mr. Rudy Pitton, Project Manager

File No. 3-10-6157
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DISTRIBUTION

2 cc: Public Works and Government Services Canada
Ontario Region
1 cc: Terraprobe Inc

Terraprobe Inc.

Greater Toronto
11 Indell Lane
Brampton, Ontario L6T 3Y3
(905) 796-2650 Fax 796-2250
brampton@terraprobe.ca

Hamilton - Niagara
903 Barton Street, Unit 22
Stoney Creek, Ontario L8E 5P5
(905) 643-7560 Fax 643-7559
stoneycreek@terraprobe.ca

Central Ontario
220 Bayview Drive, Unit 25
Barrie, Ontario L4N 4Y8
(705) 739-8355 Fax 739-8369
barrie@terraprobe.ca

Northern Ontario
1012 Kelly Lake Rd.
Sudbury, Ontario P3E 5P4
(705) 670-0460 Fax 670-0558
sudbury@terraprobe.ca

www.terraprobe.ca

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1. INTRODUCTION

Terraprobe Inc. was retained by Public Works and Government Services Canada to carry out a geotechnical investigation for the proposed Finance and NPB Hearing Building, to be constructed at the Beaver Creek Institution. The property is located at 2000 Beaver Creek Drive, Town of Gravenhurst, Ontario. Authorization to carry out this assessment was provided by Public Works and Government Services Canada on December 17, 2010.

The purpose of this assessment is to determine the soil and groundwater conditions in the area of the proposed development in order to address geotechnical aspects of design and construction. This geotechnical report will provide comments and recommendations with regard to foundations, backfill, pavement design and groundwater.

2. SITE AND PROJECT DESCRIPTION

The proposed Finance and NPB Hearing building will be located to the east and south east of existing building structure 'BC16'. The proposed structure will be a single storey, slab-on-grade 400.0m^2 office building. Also, a 32 foot (9.75m) high self-supporting tower will be constructed to the east of the proposed building structure. The tower is proposed to have a 27" (0.69m) diameter and will have a weight of approximately 377 lbs (171kg).

A plan drawing of the site as completed by Stantec Architecture Ltd. (dated January 18, 2010, Project 1400 10092) was supplied by Stantec for use within this report. Also, a topographic drawing of the site as completed by Public Works and Government Services Canada (Project No. R.003197.001) was supplied by Stantec for use by Terraprobe.

The location of the Beaver Creek Institution is shown on Figure 1 and the location of the proposed structure and borehole locations is presented on Figure 2.

3. FIELD WORK

Prior to carrying out the drilling program, the site was visited on January 11, 2011. All of the proposed boreholes were staked and underground services cleared with appropriate local utilities and with a private utility locator contractor. Suggested borehole locations were provided to Terraprobe by Stantec, and the borehole locations were selected in the field by Terraprobe with regard to existing site services.

A truck mounted power auger was mobilized to the site on January 24, 2011. A total of three (3) boreholes were advanced to a maximum depth of 5.0m below existing ground surface. The boreholes locations are provided on Figure 2.

Standard Penetration Testing was carried out at regular 0.75 to 1.5m intervals in each borehole. All soil samples obtained during the investigation process were sealed in plastic containers and returned to our laboratory for further evaluation and testing. Moisture contents were determined for all soil samples obtained.

A slotted standpipe type piezometer was installed in Borehole BH1 upon completion. A return visit to the site was made on January 31, 2011 to measure static water levels.

The ground surface elevation at each borehole location was extrapolated from the provided topographical drawing as completed by Public Works and Government Services Canada, and provided to Terraprobe by Stantec.

4. SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered are summarized on the attached borehole logs. It should be noted that these conditions are confirmed at the borehole locations only and could vary between and beyond these borehole locations.

Also, the changes in stratigraphy identified on the borehole logs have been inferred from non continuous sampling. In this regard, these lines should be interpreted as inferred transitions from one type to another and not taken as exact planes of geologic change.

Surficial asphalt topping was identified across the proposed building site. The asphalt layer was found to range in thickness from between 100mm to 150mm. Sand and granular type driveway structure fill was identified below the asphalt topping at the borehole locations. The granular type fill ranged in thickness from between 300mm (BH1 and BH2) to 900mm (BH2).

Insitu fill was identified below the granular fill soil at all borehole locations. The insitu fill is identified as sand, trace to some gravel, trace silt. The fill was found in a loose state ('N' values at 8 blows per 300m). The insitu fill extended to approximately 0.6m below existing ground surface at BH1 (276.7m), to 0.9m below existing ground surface at BH2 (276.8m) and 2.7m below existing ground surface at BH3 (274.8m).

The native soil encountered at the study site is classified as fine to coarse sand, trace to some gravel layers, trace silt. The native sand soil was found to be in a compact state. The native sand soil was in a moist to wet condition at 4.3m below existing ground surface at BH1, 4.7m below existing ground surface at BH2 and at 4.6m below existing ground surface at BH3. It should be noted that the groundwater table (currently at approximately 272.8m) could fluctuate seasonally and be higher during wetter seasons.

Insitu services such as a water service main, storm sewer, and electrical service line are located within the proposed building structure. Insitu fill associated with these buried services are anticipated.

5. DISCUSSION AND RECOMMENDATIONS

The following discussion and recommendations are provided for use by the design engineers only. Contractors bidding on this project or developing construction schedules should provide their own interpretation of the data and/or provide their own investigations if they feel warranted.

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

5.1 Foundation Installation

A soil bearing pressure of 200 kPa (SLS) is recommended for the design of the conventional spread footings founded on the native sand soils (below the insitu fill materials), at approximately ±276.7m at BH1, ±276.8m at BH2 and ±274.8m at BH3).

The bearing capacities provided are not Ultimate Limit State (ULS) values. Assuming typical strip/spread footing widths of 0.8 to 2.5m the corresponding ultimate limit states (factored) values are approximately 1.5 x the Serviceability Limit State (SLS). If excessively large and/or highly loaded spread footings are being proposed, ULS values need to be verified with Terraprobe.

Prior to placing foundation concrete, the founding subgrade should be cleaned of all deleterious materials such as unsuitable fill, existing buried services, softened, disturbed or caved materials, existing subsurface drainage structures, asphalt and any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided.

It is recommended that all excavated foundation bases must be evaluated by a qualified geotechnical engineer to ensure that the founding soils exposed at the excavation base are consistent with the design bearing pressure intended by the geotechnical engineer.

All foundations exposed to freezing temperatures must be provided with a minimum of 1.4 metres of earth cover for frost protection or alternative equivalent insulation.

All footings should be stepped along a line of 7 vertical to 10 horizontal imaginary line or flatter where variable founding levels take place. Step footings should be anticipated for this proposed building as the building footprint contains existing buried site service and insitu fills to a depth of approximately 2.7m below existing ground surface.

No major groundwater problems are anticipated for excavation to the proposed founding levels above the groundwater table. Minor seepage and surface runoff should be handled adequately with properly filtered sumps placed at the base of the excavation.

Backfilling of the footing wall excavations (and under-floor) is recommended to be placed in 200mm thick lifts, compacted to minimum of 95% Standard Proctor Maximum Dry Density (SPMDD) to proposed sub-grade elevations. Based on the attached sieve analyses, the granular type fill materials (free of rubble) and the native sand soil (free of topsoil and silt lumps) is generally suitable for re-use as backfill against foundations.

It is not recommended to place foundations and floor slabs on existing services and service trenches. Backfill within existing service trenches is recommended to be re-engineered in the building footprint. The re-engineered fill is considered to be engineered fill.

The granular type fill materials (free of rubble) identified on site and the native sand soil (free of topsoil) is suitable for re-use as engineered fill in the proposed building footprint. Engineered fill must be placed in maximum loose lift thicknesses of 150mm, compacted uniformly to a minimum of 100% of SPMDD at the optimum water content or ±2% of the optimum water content. Prior to the placement of engineered fill, all unsuitable subgrade soils must be removed, the subgrade must be proof-rolled and approved to receive fill. Foundations placed on engineered fill soil can be designed using a soil bearing pressure of 150kPa (SLS).

The proposed foundation type for the proposed tower is unknown at this time (pier foundation or pad foundation).

The foundation for the proposed tower may be founded on undisturbed native soils at a depth of approximately 1.4m below existing ground surface at the BH2 location, assuming a maximum soil bearing pressure of 200kPa (SLS).

If a gravity type foundation is being considered to resist overturning and/or lateral loads from loading on the tower, the passive resistance from the earth fill can be calculated assuming a unit weight of approximately 19kN/m^3 for the soil and an internal friction of 33° minimum. Consider a passive earth pressure coefficient (Kp) of approximately 3.0 for the marginally compacted native soil and/or compacted sand backfill.

5.2 Excavations and Backfill

Excavations will need to be carried out for the construction of the footings and servicing. The excavations will encounter fill and native sandy soils.

For the most part, these soils above the water level or where dewatered should be classified as a Type 3 Soil according to the Occupational Health and Safety Act. In this regard, temporary excavation side slopes above the groundwater level should be sloped at 1:1 (horizontal to vertical) inclination or flatter from the base of the excavation to ground surface.

Temporary excavations should not extend below an imaginary line drawn down at 7 vertical to 10 horizontal from existing footings or services without first underpinning or providing temporary shoring and/or bracing.

The moisture content of the native sand soils and sandy insitu fills encountered during this investigation are on the low side of optimum (above the groundwater level). They will generally be suitable to be placed and compacted as general fill, backfill around foundations and in service trenches. The insitu fill soils (free of rubble) and native sand soil above the groundwater level (free of silt lumps) is suitable to be used to raise the grade for driveways, parking areas and slab-on-grade areas.

General earth fills that are imported or used to raise grades on the site or internal backfill should be placed in a maximum 200mm loose lift and be compacted uniformly to a minimum of 95% SPMDD.

Testing and inspection by Terraprobe during this operation should be provided in order to document the specified compaction that is achieved and provide recommendations and suggestions with respect to how to optimize the proposed earth works.

Prior to placement of any type of earth fill to raise grade for slab-on-grade floors it is recommended that the exposed subgrade be inspected by Terraprobe and proof-rolled. Any soft or weak spots, insitu fills, existing insitu service pipes and asphalt and concrete curbs should be further excavated and replaced with approved earth fill materials.

5.3 Slab-on-Grade Floors

The native sand encountered at the site is generally suitable for the support of a slab-on-grade floor design and subgrade fill to raise grades. It is assumed that the asphalt driveway topping will be removed from site. It is essential that the subgrade area is heavily compacted and proof rolled prior to placing under floor subgrade fill and proposed floor slabs.

Conventional lightly loaded concrete floor slabs should be placed on at least 150mm of granular base (OPSS Granular 'A' or 19mm crusher run limestone) compacted to a minimum of 98% SPMDD. Any subgrade area containing excessive amounts of deleterious/organic material, unsuitable insitu fill or rubble (concrete, asphalt etc.) must be subexcavated. A proof roll of the subgrade should be conducted. Any soft or wet subgrade areas which deflect excessively during the proof roll should be locally subexcavated and backfilled with approved earth fill compacted to a minimum of 95% SPMDD. The final subgrade must be compacted to a minimum of 95% SPMDD prior to the placement of granular base.

5.4 Seismic Loading for Design

The Ontario Building Code (2006) 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 Ontario Building Code (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 the stratigraphy consists of at least 5.0 metres of compact sand soil with a penetration resistance averaging about 20 blows per 300 mm of penetration. It is known that the deeper stratigraphy in this area is at least as competent.

Although not encountered during this study, the site is underlain by bedrock of the Canadian Shield (migmatite and pegmatite).

For seismic design purposes the weighted average penetration resistance can be taken as about 20 blows per 300 mm for the upper 30 metres and the site designation for seismic analysis is Class D (OBC 4.1.8.4 Table 4.1.8.4.A).

The site has been classified as Class D according to Table 4.1.8.4.A of the Ontario Building Code (2006). 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				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
D	1.3	1.2	1.1	1.1	1

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
D	1.4	1.3	1.2	1.1	1.1

5.5 Pavement Design

The pavement subgrade in the proposed development is expected to consist of sandy soils compacted to a minimum of 98% SPMDD.

The sandy insitu fill and sandy native soils encountered on the site may be utilized for subgrade preparation provided they do not contain excessive amounts of organics and deleterious materials, and their insitu moisture content is within 3% of the optimum moisture content. The pavement subgrade should be proof-rolled with a heavy rubber tire vehicle (such as a grader), and any loose, soft, wet or unstable areas should be sub-excavated, and backfilled with similar clean earth fill placed in 150mm lifts and compacted to a minimum of 98% SPMDD.

We understand that the proposed site development will include restructuring the paved access and parking areas in the area of the new building structure. The following parking and access pavement structure is recommended for the study site.

HL3	surface asphalt	40mm
HL8	binder asphalt	50mm
Granular 'A'	base	150mm
Granular 'B'	subbase	250mm

If the final subgrade is clean sand (ie: less than 8% fines) an alternative pavement design may be appropriate, comprising of 40mm HL3, 50mm HL8 and 200mm Granular 'A' base material. This can be assessed when final grading is near completion with subsequent soil sampling and grain size analysis completed by Terraprobe. The grain size analysis as completed as part of this investigation (appended) suggests that this alternative pavement design will likely be a reasonable option if desired. The grain size analysis completed on sample 3 from BH1 has met the OPSS Granular 'B' (Type 1) specifications.

The granular materials should be placed in lifts 150 mm thick or less and be compacted to a minimum of 100% SPMDD for granular materials. Asphalt materials should be rolled and compacted to 97% Marshall Bulk Density based on nuclear density testing.

The need for adequate subgrade drainage cannot be over-emphasized. The subgrade must be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward subgrade drains. Grading adjacent to the pavement areas should be designed to ensure that water is not allowed to pond adjacent to the outside edges of the pavement. Pavement subdrains leading to catchbasins are recommended to facilitate drainage of the subgrade and the granular materials.

The above pavement design thicknesses are considered adequate for the entrance and parking lot. However, if the pavement construction occurs in wet, winter or inclement weather, it may be necessary to provide additional subgrade support for heavy construction traffic by increasing the thickness of the granular sub-base, base or both. Further, traffic areas for construction equipment may experience unstable subgrade conditions. These areas may be stabilized utilizing additional thickness of the granular materials.

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

We trust that this report is adequate for your present requirements. If you should have any questions, or if we can be of further assistance, please do not hesitate to contact the undersigned.

Sincerely,

Terraprobe Inc.

PROFESSIONAL GEOSCIENTIST
SEAN G. O'MARA
PRACTISING MEMBER
1108
ONTARIO

Sean O'Mara, P.Geo.

Kirk R. Johnson, P. Geo, P. Eng.
Associate

REGISTERED PROFESSIONAL ENGINEER
May 19/11
K. R. JOHNSON
PROVINCE OF ONTARIO

SOM/ct

Barrie Office

BOREHOLE LOGS

Terraprobe Inc.

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ABBREVIATIONS, TERMINOLOGY, GENERAL INFORMATION

BOREHOLE 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.

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LOG OF BOREHOLE ..1..

PROJECT NAME: Beaver Creek, NPB Hearing Bldg
PROJECT No.: 3-10-6157
CLIENT: Public Works and Government Services Canada
BORING DATE: January 24, 2011
LOCATION: Gravenhurst, Ontario
ELEVATION DATUM: Geodetic

BORING METHOD: CME55 Crawler-mounted Drill Rig / Solid Stem Augers

Depth scale (m)	Soil profile description	Elev. / Depth (m)	Sample Number	Type	"N" Value	Penetration resistance (x)	Water content (o)	Installation information
0	GROUND SURFACE	277.3						
	150mm – Asphalt	0.0						
	300mm– SAND&GRAVEL, trace silt, FILL Brown Compact Moist	276.7 / 0.6	1	AS			o	
1	SAND, trace silt, FILL Brown Dense to Compact Moist to Wet		2	SS	41	x	o	Bentonite Seal
2	SAND, trace gravel, trace silt, gravelly layers at 1.5 to 2.0m, laminated zones		3	SS	29	x	o	
			4	SS	22	x	o	
3			5	SS	25	x	o	
4								▽ 4.3m; ▼ 4.5m
5		272.3	6	SS	24	x	o	
	End of Borehole	5.0						

Penetration resistance plot: 20 40 60 80
Shear strength kPa: nat.V – +, rem.V – ⊕, Q – ●, U – o; 20 40 60 80
Water content (%): WP, W, WL; 10 20 30

1. Borehole remained open upon completion of drilling.

2. Water level noted at 4.3m during drilling.

3. Water level on January 31, 2011 measured at 4.5m (272.8m).

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LOG OF BOREHOLE ..2..

PROJECT NAME: Beaver Creek, NPB Hearing Bldg
PROJECT No.: 3-10-6157
CLIENT: Public Works and Government Services Canada
BORING DATE: January 24, 2011
LOCATION: Grevenhurst, Ontario
ELEVATION DATUM: Geodetic

BORING METHOD: CME55 Crawler-mounted Drill Rig / Solid Stem Augers

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	"N" VALUE	WATER CONTENT (%)	INSTALLATION INFORMATION
GROUND SURFACE	277.7					
100mm – Asphalt 300mm– SAND&GRAVEL, trace silt, FILL Brown Compact Moist	0.0	1	AS			
GRAVELLY SAND, trace silt, FILL Brown Compact Moist	276.8 0.9	2	SS	24		
SAND, trace to some gravel, trace silt		3	SS	28		
		4	SS	23		
		5	SS	19		
	272.7	6	SS	20		
End of Borehole	5.0					1. Borehole remained open upon completion of drilling. 2. Water level noted at 4.7m during drilling.

Penetration resistance plot scale: 20 40 60 80
Shear strength kPa: nat.V – + Q – ● rem.V – ⊕ U – O; scale 20 40 60 80
Water content (%): WP W WL; scale 10 20 30

SHEET 1 OF 1

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LOG OF BOREHOLE ..3..

PROJECT NAME: Beaver Creek, NPB Hearing Bldg | PROJECT No.: 3-10-6157
CLIENT: Public Works and Government Services Canada | BORING DATE: January 24, 2011
LOCATION: Gravenhurst, Ontario | ELEVATION DATUM: Geodetic

Boring method: CME55 Crawler-mounted Drill Rig / Solid Stem Augers

Depth scale (m)	Description	Strata plot	Elev. / Depth (m)	Sample number	Type	"N" value	Penetration resistance / shear strength	Water content (%)	Installation information
0	GROUND SURFACE		277.5						
	100mm – Asphalt 300mm– SAND&GRAVEL, trace silt, FILL		0.0	1	AS			O	
	Brown Loose Moist SAND, trace gravel, trace silt, FILL			2	SS	50/150mm		O	
				3	SS	8	x	O	
			274.8	4	SS	8	x	O	
2.7	Brown Compact Moist to Wet SAND, trace gravel, trace silt		2.7	5	SS	21	x	O	
			272.5	6	SS	21	x	O	
5.0	End of Borehole		5.0						1. Borehole remained open upon completion of drilling. 2. Water level noted at 4.6m during drilling.

Penetration resistance plot: 20 40 60 80
Shear strength kPa: nat.V – + ; rem.V – ⊕ ; Q – ● ; U – O ; 20 40 60 80
Water content (%): P — ○ — L ; 10 20 30

SHEET 1 OF 1

FIGURES

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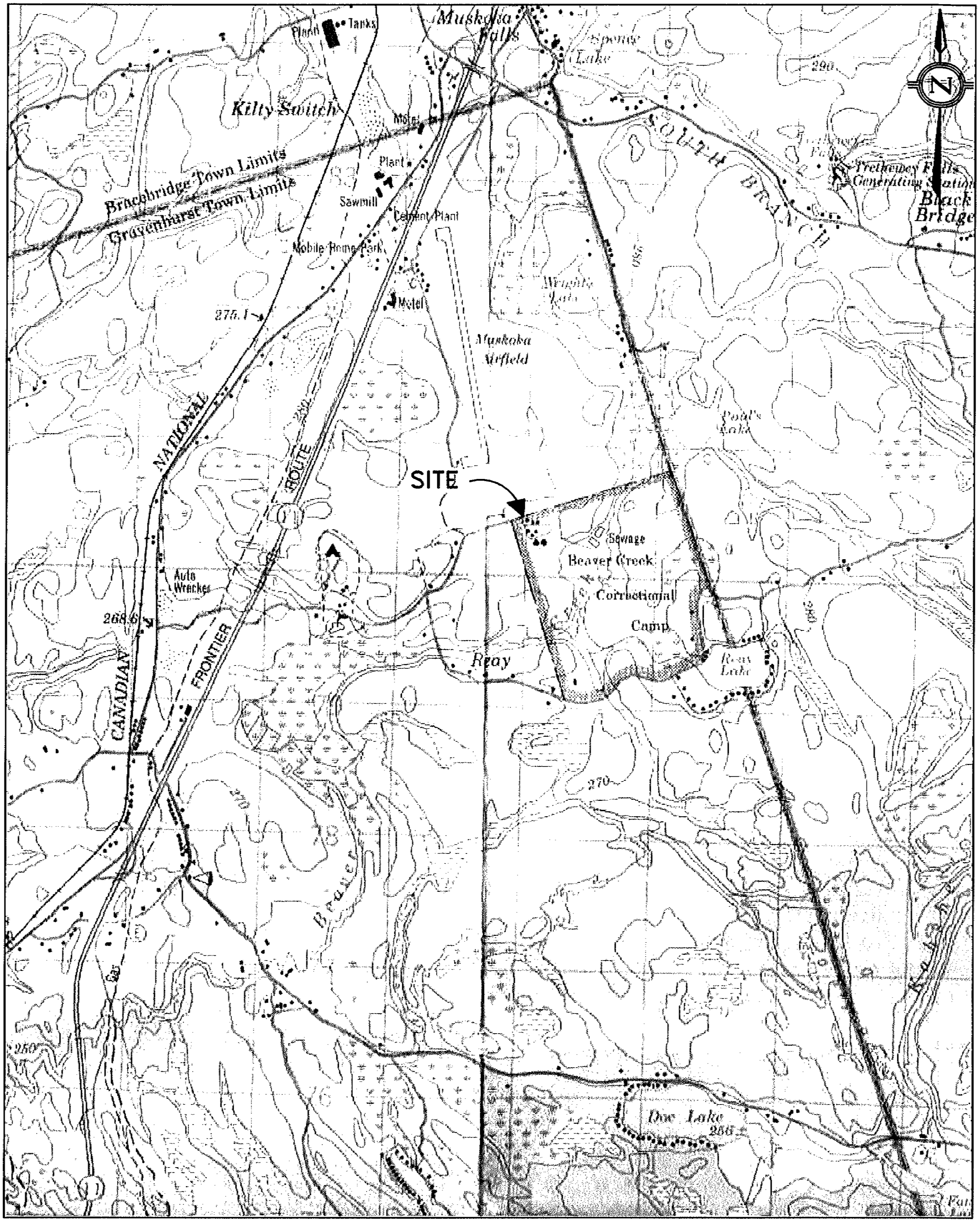

FEBRUARY 2011

SITE LOCATION PLAN

3-10-6157

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FIGURE 1

FEBRUARY 2011 BOREHOLE LOCATION PLAN 3-10-6157

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FIGURE 2

SIEVE GRADATION ANALYSIS

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SIEVE GRADATION ANALYSIS
TEST RESULTS

PROJECT : **Beaver Creek: NPB Hearing Building**
LOCATION: **Gravenhurst, ON**
CLIENT : **Public Works & government Services Ontario Region Professional and Technical Program**
SAMPLE MATERIAL: **Native sand, trace gravel, trace silt**

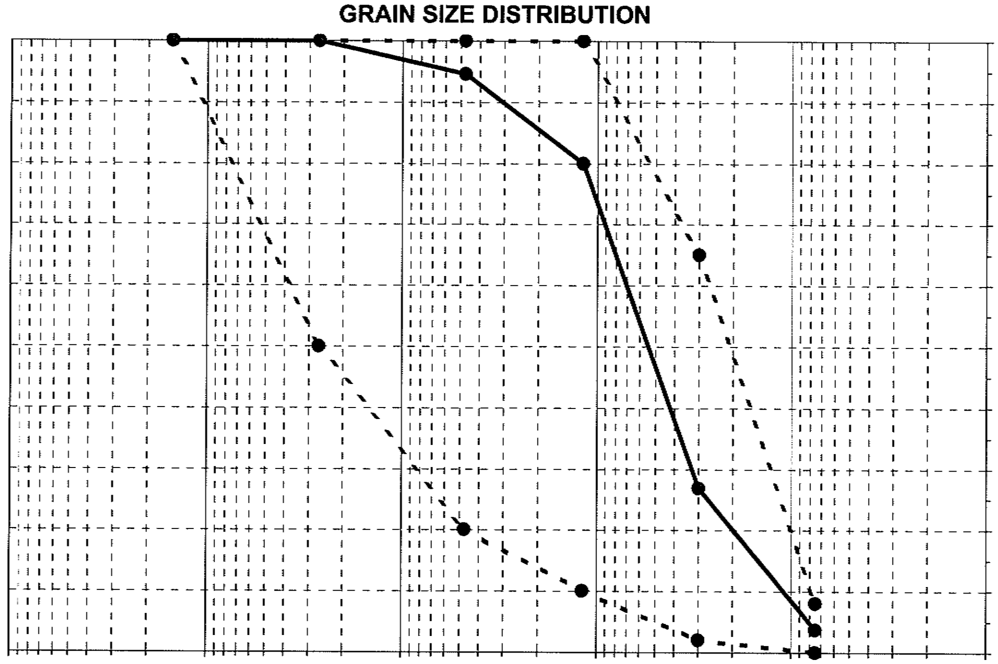
SAMPLE SOURCE: **Borehole 1, sample #3**

FILE NO. **3-10-6157**
LAB NO: **139**

SAMPLE DATE: **Jan-24-11**

SAMPLED BY: **B.H.**

GRAIN SIZE DISTRIBUTION



GRAIN SIZE, mm

SIEVE SIZE mm	PERCENT PASSING SPECIFIED MIN.	MAX.	SAMPLE	NOTES: GRANULAR 'B' (Type 1) OPSS FORM 1010
150.0	100	100	100	
26.5	50	100	100.0	Sample tested conforms to
4.75	20	100	94.6	OPSS 1010 for gradation
1.18	10	100	79.9	
0.300	2	65	26.9	
0.075	0	8	3.7	Note: Boldface denotes not meeting specifications

TTBarrie MAR 2007

Reviewed By:____________