Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION AND ENGINEERING REVIEW OF SLOPE STABILITY 1268 LAKESHORE ROAD EAST ST. CATHARINES, ONTARIO

Prepared For: 2 Maplewood Drive
St. Catharines, Ontario
L2M 3PI

Attention: Mr. David Edwards

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1.0 INTRODUCTION

Terraprobe Inc. was retained by Mr. David Edwards to conduct a geotechnical investigation and engineering review of slope stability at 1268 Lakeshore Road East in St. Catharines, Ontario. A Site Location Plan is provided as Figure 1. A proposal and cost estimate to carry out the investigation were provided in our letter of July 8, 2010. Authorization to proceed with the work was provided by Mr. David Edwards on July 9, 2010. It is understood that the slope stability assessment will be a required as part of the permitting process for a new house, which is to replace a recently demolished dwelling.

The purpose of the work was to investigate and report on the subsurface soil and ground water conditions at four locations. Based on this information a slope stability assessment was completed to determine the long term stable top of slope. The geotechnical assessment was completed to satisfy the intent of the policy guidelines outlined in the document entitled Understanding Natural Hazards Guide (Ministry of Natural Resources, 2004) and the Niagara Peninsula Conservation Authority document entitled '*Polices, Procedures and Guidelines for the Administration of O.Reg. 155/06 and Land Use Planning Policy Document*', dated December 9, 2009. Advice is also provided with respect to the geotechnical aspects of the design of the foundations and floor slabs-on-grade for the new house.

2.0 SITE DESCRIPTION

The site was examined by a senior geotechnical engineer from our staff on July 21, 2010 in order to obtain general information regarding the existing slope features such as slope profile, slope drainage, water course features, vegetation cover, structures in the vicinity of the slope. Photographs illustrating various features of the study area are provided in Appendix A.

The property was irregular in shape and was situated on tableland adjacent to Lake Ontario to the north and the Fifteen Mile Pond corridor the west. The property was previously occupied by a residential dwelling, however the structure had been demolished and the site was vacant. At the time of our field reconnaissance, the tableland above the slope was generally flat and level with a gentle grade towards Lake Ontario.

2.1 Valley Slope

The valley to the east of the subject property consisted of the Fifteen Mile Pond corridor and the associated corridor of Sixteen Mile Creek. Both the Fifteen Mile and Sixteen Mile Creeks were characterized as steep-sided and flat-floored valleys. At the Lake Ontario shore, the mouth of the creeks were a few hundred metres of one another and the area was developed as Charles Daley Park. Based on the topographic survey by William A. Mascoe Surveying Ltd., the valley slopes immediately adjacent to the subject property had an approximate height of about 12 to 13 m. The overall inclination of the valley slopes were typically 1.8 horizontal to 1 vertical towards the south side and 1.6 horizontal to 1 vertical towards the north. The valley slopes were covered with a variety of grasses, bush and weed growth. There was no obvious evidence of failure scars in the slope face or erosion at the slope toe. Figure 3 present two representative cross-sections of the valley slope, Section A-A' and B-B'.

2.2 Shoreline Slope

The north property boundary consists of a shoreline slope which falls some 12 to 13 m in elevation to Lake Ontario. The shoreline consisted of concrete rubble which appeared to have been in-filled and capped with poured portland cement concrete. The concrete rubble beach was about 5 to 6 metres wide and about 1.4 to 1.8 m above the current water level of Lake Ontario. The water level at the time of the survey was 86.6m (local datum) or 74.82m (IGLD 1985). Figure 4 presents a representative cross-section of the shoreline slope, Section C-C'.

The shoreline on the west side of the slope, above the rubble beach, consisted of a slope with an overall inclination of about 1.2H:1V to 1.5H:1V (horizontal to vertical). The slope was relatively well vegetated with a variety of grasses, bush and weed growth. There was no evidence of deep seated instability of the slope such as large slumps, tension cracks and scarp zones. The overall slope appeared to be in a stable condition.

On the east side of the shoreline slope, there was a vertical concrete block wall above the concrete rubble beach. Construction details on the wall were not available. It is considered that the rubble beach does provide some level of protection to the wall against undermining. Based on the survey, the crest elevation of the wall was about 91.7 m (local datum) or about 5.1 m above the current water level of Lake Ontario. There was a relative gentle slope from the top of the concrete block wall to the physical top of slope of about 5 horizontal to 1 vertical.

3.0 PROCEDURE

The field work for this investigation was carried out on July 21, 2010, during which time four boreholes were drilled to depths of about 3.5 to 12.6 m below the existing ground surface. The locations of the boreholes are shown on the Site Plan, Figure 2. The results of the boreholes are shown on the Log of Borehole sheets following the text of this report.

The boreholes were drilled using a truck mounted power auger drill rig supplied and operated by a specialist drilling contractor. The boreholes were advanced using solid stem continuous flight augers. Standard Penetration testing and sampling were carried out at regular intervals of depth in each borehole using 50 millimetre outside diameter split spoon sampling equipment. After the drilling, sampling, and logging was completed, the boreholes were backfilled with auger cuttings, commercial grade pre-bagged sand and bentonite sealant.

The field work was monitored by a member of our engineering staff who located the boreholes, arranged for underground clearances at the borehole locations, logged the boreholes, and cared for the samples obtained. The ground surface elevations at the boreholes were inferred from spot elevations shown on the topographic plan prepared by William A. Mascoe Surveying Ltd. and were understood to have been referred to geodetic datum. Ground water observations were made in the boreholes as drilling proceeded. No provision was made for long-term ground water monitoring at the site.

All of the samples recovered in the course of the investigation were brought to our Stoney Creek laboratory for further examination and selective classification testing. A geotechnical laboratory testing program carried out as part of the investigation included water content determinations. The results of the water content determinations are shown on the Log of Borehole sheets.

4.0 SUBSURFACE CONDITIONS

The subsurface soil and ground water conditions encountered in the boreholes are presented on the attached Log of Borehole sheets. The stratigraphic boundaries indicated are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed at the borehole locations only, and may vary between and beyond the borehole locations.

4.1 Soil Conditions

The following discussion has been simplified in terms of the major soil strata for the purposes of geotechnical design. In general, the boreholes drilled at the site penetrated topsoil and fill overlying strata of sandy silt, silty sand, silty clay and clayey silt.

Topsoil

A 125 to 150 mm thick layer of topsoil was encountered at the ground surface in the boreholes.

Fill

Fill was encountered in boreholes 2, 3 and 4 to depths of about 0.8 to 1.2 metres below the existing ground surface. The fill consisted of sandy silt. The N values, as determined in the Standard Penetration testing carried out within the fill, ranged from 5 to 25 blows per 0.3m, inferring a loose to compact state of packing. The in-situ water content of the samples of fill recovered from the standard penetration testing ranged from about 7 to 13 percent.

Sandy Silt, and Silty Sand

Physiographically the site constitutes a portion of a sand plain formed in the beach zones during a time when Lakes Ontario and Erie were at a higher post glacial elevation called the Lake Iroquois Period. The upper zone of this stratum is comprised of brown sandy silt. The deeper sediments of the soil stratigraphy beneath the site predominantly had a silty sand gradation. The N values determined within these strata ranged from 17 to 76 blows per 0.3 m. The natural water content of the samples ranged from about 6 to 30 percent, with an average natural water content of about 12 percent.

Silty Clay to Clayey Silt

Borehole 1 was terminated in stratum of silty clay to clayey silt. The N values determined within the silty clay or clayey silt strata ranged from 6 to 27 blows per 0.3 m. The natural water content of the samples ranged from about 17 to 31 percent.

4.2 Ground Water Conditions

The ground water conditions were observed in the boreholes during and following drilling. At the time of the investigation, ground water was encountered in borehole 1 at a depth of about 11 m below the existing grade level or at about elevation 88.9 m (local datum). All remaining boreholes were dry during and upon completion of drilling.

It should be noted that the conditions reported above may not necessarily represent stabilized ground water conditions or the conditions that will be encountered during construction. The ground water levels will vary due to seasonal effects and precipitation conditions.

5.0 STABILITY ANALYSIS

From a geotechnical engineering perspective, the setback for the proposed replacement structure should be established based on the potential for bank erosion to impact on the stability of the slope, the stability of the slopes, and top of bank land usage. The following presents an assessment of each of the setback components for estimating the total development setback specific to the site.

5.1 Erosion Allowance

An erosion allowance is recommended in areas where the water course position is within 15 m to the slope toe. A guideline table recommended for estimating the erosion allowance is presented as follows:

MINIMUM TOE EROSION ALLOWANCE - River within 15 m of Slope Toe *

Type of Material	Evidence of active erosion** or bankfull flow velocity > competent flow velocity***	No evidence of active erosion** or flow velocity << competent flow velocity***		
		Bankfull Width		
		< 5 m	5 - 30 m	> 30 m
1. Hard Rock (granite)	0 - 2 m	0 m	0 m	1 m
2. Soft Rock (shale, limestone) Cobbles, Boulders	2 - 5 m	0 m	1 m	2 m
3. Stiff/Hard Cohesive Soil (clays, clayey silt)	5 - 8 m	1 m	2 m	4 m
4. Soft/Firm Cohesive Soil Fine Granular (sand, silt) Fills	8 - 15 m	1 - 2 m	5 m	7 m

Notes:

* If a valley floor is > 15m width, still may require study or inclusion of a toe erosion allowance.

** Active Erosion is defined as: bank material is bare and exposed directly to stream flow under normal or flood flow conditions and, where undercutting, over steepening, slumping of a bank or high down stream sediment loading is occurring. An area may be exposed to river flow but may not display "active erosion" (i.e. is not bare or undercut) either as a result of well rooted vegetation or as a result of shifting of the channel or because flows are relatively low velocity. The toe erosion allowances presented in the right half of Table 2 are suggested for sites with this condition.

*** Competent Flow velocity; the flow velocity that the bed material in the stream can support without resulting in erosion or scour.

Consideration must also be given to potential future meandering of the watercourse channel.

Source:

"Geotechnical Principles for Stable Slopes" (Terraprobe, April 1998)
Prepared for: Ontario Ministry of Natural Resources, Lands and Natural Heritage Branch

The water course (pond) was noted to be typically 11 to 30 m wide and is located within 15 m of the slope toe for the majority of the study area. It is also noted the bank of the water course is comprised of very stiff silty clay to clayey silt. Therefore, in consideration of the prevailing site conditions, an erosion allowance of 2 m is recommended.

5.2 Stable Slope Allowance

A detailed engineering analyses of slope stability was carried out for a selected slope cross-section utilizing a commercially available slope modeling program GeoStudio 2007 (Version 7.03). The slope stability analyses is based on an effective stress limiting equilibrium analysis for long term slope stability using Morgenstern-Price, Spencer, Bishop and Jambu methods. The methods of analysis allow for the calculation of Factors of Safety for hypothetical or assumed failure surfaces through the slope. The analysis method is used to assess potential for movements of large masses of soil over a specific failure surface which is often curved or circular.

For a specific failure surface, the Factor of Safety is defined as the ratio of available strength resisting movement, divided by the gravitational forces tending to cause movement. A Factor of Safety of 1.0 represents a 'limiting equilibrium' condition where the slope is at the point of pending failure since the soil resistance is equal to the forces tending to cause movement. The analysis involves dividing the sliding mass into many thin slices and calculating the forces on each slice. The normal and shear forces acting on the slides and base of each slice are calculated. It is an iterative process that converges on a solution. It is usual to require a Factor of Safety greater than one (1) to ensure stability of the slope. The typical Factor of Safety used for engineering design of slopes for stability in building applications, range from about 1.3 to 1.5 for buildings situated close to the slope crest. The Niagara Peninsula Conservation Authority allow a minimum Factor of Safety of 1.5 for long term (effective stress) conditions.

The analysis was carried out by preparing a model of slope geometry and subsurface conditions and analyzing numerous different failure surfaces through the slope in search of the minimum or critical Factor of Safety for specific conditions. The pertinent data obtained from topographic mapping, slope profiles, and available subsurface information were input for the slope stability analysis. A series of calculations were carried out to examine the Factors of Safety for varying depths for potential failure surfaces.

The following average soil properties were assumed for the soil strata in the slope stability analysis. The soil strength parameters were used in the effective stress analysis for long-term slope stability.

Stratigraphic Unit	Soil Unit Weight (kN/cu.m)	Effective Shear Resistance, c' (kPa)	Effective Angle of Internal Friction ϕ' (degrees)
Sandy Silt, compact	19.0	0	33
Silty Sand, compact	19.5	0	32
Silty Clay to Clayey Silt, firm to very stiff	18.5	8	30

Cross-sections of the valley and shoreline slopes were derived from the topographic survey information provided by William A. Mascoe Surveying Ltd. at selected locations (Sections A-A', B-B' and C-C', Figures 2, 3 and 4). The slope at Section B-B' was considered to be the most vulnerable portion within the study area and was therefore selected for analyses. Graphical depictions of the slope stability analysis results are presented in Appendix B.

Based on the results of the analyses, the soil types and the anticipated ground water levels, an overall inclination of 1.8 horizontal to 1 vertical will be required to achieve a Factor of Safety of 1.5. Therefore, the stable slope allowance will be a total of 1.8 times the height of the slope measured horizontally from an inferred slope toe. The location of the stable top of slope determined on the basis of the stable slope profiles is shown on Figures 2, 3 and 4.

5.3 Development Setback

The third setback component is a spatial allowance for controlling top of bank land use that could potentially impact slope stability and to ensure that future development is not impacted by slope deformations. This setback also provides a means of access to the slope. The intent is that no filling, or structural development would be allowed to take place in this zone.

Policies for this component of the setback have been established by the Niagara Peninsula Conservation Authority in the document entitled '*Polices, Procedures and Guidelines for the Administration of O.Reg. 155/06 and Land Use Planning Policy Document*'. The policies that pertain specifically to this site are outlined under Policy No. 3.25 for development adjacent to valley lands and Policy No. 3.26 for development adjacent to shorelines.

5.3.1 Valley Land Development Setback

For development adjacent to valley lands, it is understood that the NPCA present policy requires a 7.5 metre setback from the stable top of slope, except as provided in Policy No. 3.25.4., which states that replacement buildings may be permitted provided that:

i) *the replacement or addition does not encroach any closer to the stable top of slope than the existing development at it closest point;*
ii) *even if the existing development is closer than 7.5 metres to the stable top of slope, no new development is permitted within 7.5 metres to the stable top of slope in order to provide for an erosion allowance as per the Provincial Policy Statement;*
iii) *a geotechnical assessment by a qualified engineer (at the expense of the applicant), may be required to determine the location of the Stable Top of Slope and to determine if the proposed development will have a negative impact on the slope stability; and*
iv) *In cases were the building or structure can be reasonably relocated outside of the setback, the applicant will be encouraged to do so'.*

We have reviewed the available information for the site, including site topography and the site geology; completed a subsurface borehole investigation to identify the nature of the soil and ground water conditions beneath the site; and completed a detailed engineering analyses of slope stability to evaluate stable slope profile. Based on our field observations and engineering assessment, it is our opinion that the construction of a replacement building, located no closer than the stable top of slope, will not have any negative impact on the stability of the valley land slope.

5.3.2 Shoreline Development Setback

For development adjacent to the shoreline, Policy 3.26.4.2 states that replacement buildings may be permitted provided that:

a) Not permitted within the Stable Slope Allowance or the dynamic beach hazard.
b) May be permitted within the flooding hazard provided:
  i) Mean are provided to mitigate the wave uprush hazard (i.e. shutters installed on windows).
  ii) Means are provided to mitigate to 100 year lake level flood hazard (i.e. no openings are constructed within the structure below the regulatory 100 year flood elevation).
  iii) The NPCA is satisfied that no practical alternative exists to located the proposed structure outside of the flooding hazard.
c) May be permitted with the erosion allowance provided:
  i) It meets the requirements of the shore protection work standard to the maximum extent and level possible based on site-specific conditions;
  ii) it utilizes maximum lot depth and width;
  iii) as a minimum, uses a setback from the Stable Slope Allowance of 7.5 metres.
  iv) The NPCA is satisfied that no practical alternative exists to located the proposed structure outside of the erosion hazard.

Based on our interpretation of the proceeding policy, it is our opinion that the construction of a replacement building must be located no closer than 7.5 metres from the long term stable top of slope.

A coastal engineer should be consulted to determine the site specific erosion hazard limits for the site and examine the adequacy of the existing shoreline protection at the slope toe.

6.0 FOUNDATION DESIGN CONSIDERATIONS

The following discussion is based on our interpretation of the factual data obtained during this investigation and is intended for the use of the design engineer only. Comments made regarding the construction aspects are provided only in as much as they may impact on design considerations. Contractors bidding on or undertaking these works, should make their own assessment regarding the nature and adequacy of the factual information, as is affects their construction techniques, scheduling, equipment selection and the like.

It should be noted that only conceptual design information was available when this report was prepared. For this reason, some aspects of the following discussion are of a preliminary nature and should be reviewed at the final design stage.

6.1 Foundation Design Parameters

Based on the results of the boreholes it is considered feasible to support the building foundations on conventionally designed spread or strip footings bearing in the sandy silt. Existing topsoil, fill and any disturbed materials are unsuitable for the support of building foundations. All foundations designed to bear on the undisturbed sandy silt can be designed using a factored bearing resistance at Ultimate Limit States (ULS) of 225 kPa and a bearing resistance at Serviceability Limit States (SLS) of 150 kPa.

A minimum footing width of 450 mm is recommended for strip footings and a minimum footing width of 900 mm should be considered for spread footings. All footings in unheated areas must be provided with at least 1.2 metres of earth cover for frost protection or equivalent insulation. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided.

Some variability in the consistency and depth of the undisturbed strata is expected, particularly where former foundations and excavations exist. For this reason, it is important that all of the foundation excavations be inspected by Terraprobe to confirm that the fill and soft surficial strata have been fully penetrated and to identify any preparatory work required prior to placing the footing concrete. Where deeper excavations are required, the footings should be lowered in a series of steps with maximum vertical increments of 0.6 m and with a rise to run ratio of 1:2.

6.2 Floor Slabs of Grade

Concrete basement floor slabs can be constructed on grade. The slab must be placed on a capillary moisture barrier consisting of a minimum of 150 mm of 19 mm clear stone (OPSS 1004) compacted by a vibratory plate to a dense state. The floor slab design must incorporate subfloor drainage provisions to ensure that this clear stone does not pool ground water. This can be accomplished by installing subdrains beneath the basement floor slab or by constructing weep holes through the perimeter foundation walls to allowing drainage out to the perimeter foundation drains (weeping tile).

6.3 Foundation Walls

Walls subjected to unbalanced earth loads must be designed to resist earth pressures calculated based on the following equation: $P = K(\gamma h + q)$

where: P = the horizontal pressure at depth (h)
K = the earth pressure coefficient,
q = surcharge loading (kPa)
γ = the unit weight of the backfill (21 kN/m3)

The above equation assumes that free draining granular backfill such as Type I Granular "B" per OPSS Form 1010, is used and positive drainage is provided to ensure that there will be no hydrostatic pressure acting on the walls in conjunction with the earth pressure.

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

Soil	ϕ	γ (kN/m^3)	Ka	Ko	Kp
Compact Granular Fill Granular 'B' (OPSS 1010)	32	21	0.31	0.47	3.25
Sandy silt, silty sand or similar fill	30	19	0.33	0.5	3

In the case of a foundation wall with the top and bottom of the wall restrained, an active earth pressure coefficient (Ka) may be assumed for design purposes.

Heavy compaction equipment should not be used immediately behind the walls. The use of heavy compaction equipment close to the walls may cause deflection or damage to the wall.

6.4 Excavation

All excavations must be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. In this context, the near surface soil strata at the site, primarily consisting of sandy silt or silty sand, should generally be regarded as "Type 3 Soil", provided that effective ground water control is achieved where required and surface water is directed away from the excavation.

Where workmen must enter a trench or excavation carried deeper than 1.2 metres the trench or excavation must be suitably sloped and/or braced in accordance with the regulation requirements. Minimum support system requirements for steeper excavations are stipulated in Sections 235 thru 238 and 241 of the Act and Regulations and include provisions for timbering and shoring.

7.0 CLOSURE

This report was prepared by Terraprobe Inc. for the exclusive use of Mr. David Edwards, and his retained designers and consultants. The intent of the report was to provide geotechnical engineering information for the proposed development and on the stability of slopes at the subject site and not for other purposes. The report is site specific and not applicable to any other property.

The foundation construction should be monitored and evaluated by this office to verify that the conditions are consistent with the design assumptions. The on-site review of the condition of the bearing soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.3 of the Ontario Building Code 2006.

We trust this report meets with your requirements. Should you have any questions regarding the information presented, please do not hesitate to contact our office.

Yours truly,

Terraprobe Inc.



Patrick Cannon, P.Eng.
Associate

pc

LOGS OF BOREHOLES

Terraprobe Inc.



Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering & Testing

ABBREVIATIONS, TERMINOLOGY & GENERAL INFORMATION

Sampling Method
SS - split spoon
ST - shelby tube
AS - auger sample
RC - rock core

Penetration Resistance
Standard Penetration Resistance ('N' values) is defined as the number of blows by a hammer of 63.5kg mass (140lbs) falling freely for a distance of 0.76m (30 inches) required to advance a standard 50mm (2inch) diameter split spoon sampler for a distance of 0.3m (12 inches).

Dynamic Cone Penetration Resistance is defined as the number of blows by a hammer of 63.5kg mass (140 lbs) falling freely for a distance of 0.76m (30 inches) required to advance a conical steel point to 50mm diameter and with 60 degrees sides of 'A' size drill rods for a distance of 0.3m (12 inches).

Soil Description

Cohesionless Soils

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

Cohesive Soils

Consistency	**Undrained Shear Strength (kPa)**
very soft	<12
soft	12 - 25
firm	25 - 50
stiff	50 - 100
very stiff	100 - 200
hard	>200

Soil Composition	**% By Weight**
'trace' (eg. trace silt)	<10
'some' (eg. some gravel)	10 - 20
'adjective' (eg. sandy)	20 - 35
'and' (eg. sand and gravel)	35 - 50

General Information
The recommendations provided in this report are based on the factual information obtained from the boreholes and on the general information provided for the proposed project.

Site investigations by means of boreholes and/or test pit identifies subsurface conditions at the location and time of sampling only. Ground conditions at locations away from the boreholes and test pit may vary.

Recommendations are made by interpretation of this factual data for specific conditions such as size, configuration and location of the proposed project. Changes in project conditions should be reviewed by the Geotechnical consultant as they may affect the recommendations provided.

In order to identify possible changes in ground conditions between the sample locations and their effect on the project, it is recommended that site inspections be carried out during construction by qualified Geotechnical personnel.

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LOG OF BOREHOLE 1

PROJECT No: 7-10-5060
CLIENT: Mr. David Edwards
LOCATION: 1268 Lakeshore Road, Burlington, Ontario

BORING DATE: July 21, 2010
ELEVATION DATUM: Local
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 75 - Truck - 150mm Solid Stem Augers

DEPTH SCALE IN METRES	DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	NUMBER	TYPE	"N" VALUE
0	**GROUND SURFACE**		99.9			
	150mm Topsoil		0.0			
	compact, brown **SANDY SILT** occasional seams or layers of silty sand		0.2	1	SS	21
1				2	SS	21
2				3	SS	24
				4	SS	24
3				5	SS	25
			96.1			
4	compact to dense, grey **SILTY SAND**		3.8	6	SS	24
				7	SS	76
5						
6				8	SS	17
7			92.3			
	firm, grey **SILTY CLAY TO CLAYEY SILT**		7.6	9	SS	6
8						
	becoming very stiff at 9.1m depth		90.9			
9	**Borehole Continued on next page...**		9.0			

PENETRATION RESISTANCE PLOT
SHEAR STRENGTH kPa
20 40 60 80

WATER CONTENT (%)
w_P w w_L
10 20 30

INSTALLATION INFORMATION

UTM Co-ordinates:
17 636770 E
4782568 N

A. CUMMINGS
-10-5060-1.DWG

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LOG OF BOREHOLE 1

PROJECT No: 7-10-5060
CLIENT: Mr. David Edwards
LOCATION: 1268 Lakeshore Road, Burlington, Ontario

BORING DATE: July 21, 2010
ELEVATION DATUM: Local
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 75 - Truck - 150mm Solid Stem Augers

DEPTH SCALE IN METRES	DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	NUMBER	TYPE	"N" VALUE	PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (20 40 60 80)	WATER CONTENT (%) (wP, w, wL; 10 20 30)	INSTALLATION INFORMATION
9	**Borehole Continued...**		90.9						
			9.0						
				10	SS	27		o	
	very stiff, grey **SILTY CLAY TO CLAYEY SILT**								Jul.21/10
10									
				11	SS	14		o	
11									
12									
				12	SS	15		o	
			87.3						
	END OF BOREHOLE		12.6						
13									
14									
15									
16									
17									
18									

NOTES:
Water level in open borehole at elevation 88.9m after drilling.

A. CUMMINGS

7-10-5060-1.DWG

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LOG OF BOREHOLE 2

PROJECT No: 7-10-5060
CLIENT: Mr. David Edwards
LOCATION: 1268 Lakeshore Road, Burlington, Ontario

BORING DATE: July 21, 2010
ELEVATION DATUM: Local
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 75 - Truck - 150mm Solid Stem Augers
DEPTH SCALE IN METRES

DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	NUMBER	TYPE	"N" VALUE
GROUND SURFACE		99.8			
150mm Topsoil		0.0			
compact, brown **FILL** sandy silt, some topsoil		0.0 / 99.0	1	SS	25
compact, brown **SANDY SILT** trace to some clay		0.8 / 98.1	2	SS	13
compact, brown **SANDY SILT** occasional seams or layers of silty sand		1.7 / 96.3	3	SS	48
			4	SS	24
			5	SS	32
END OF BOREHOLE		3.5			

PENETRATION RESISTANCE PLOT
SHEAR STRENGTH kPa
20 40 60 80

WATER CONTENT (%)
w_P w w_L
10 20 30

INSTALLATION INFORMATION

UTM Co-ordinates:
17 636790 E
4782574 N

NOTES:
Borehole dry upon completion of drilling.

SHEET 1 OF 1

A. CUMMINGS
7-10-5060-2.DWG

Terraprobe

LOG OF BOREHOLE 3

PROJECT No: 7-10-5060
CLIENT: Mr. David Edwards
LOCATION: 1268 Lakeshore Road, Burlington, Ontario

BORING DATE: July 21, 2010
ELEVATION DATUM: Local
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD	DEPTH SCALE IN METRES	DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUE	PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (20 40 60 80)	WATER CONTENT (%) (wP, w, wL; 10 20 30)	INSTALLATION INFORMATION
CME 75 - Truck - 150mm Solid Stem Augers	0	**GROUND SURFACE**		99.9						
		125mm Topsoil		0.0						
		loose to compact, brown **FILL** sandy silt, sand and gravel		0.0	1	SS	13		o	
	1				2	SS	5		o	
				98.7						
		compact to dense, brown **SANDY SILT** occasional seams or layers of silty sand and clayey silt		1.2.	3	SS	24		o	
	2				4	SS	34		o	
	3				5	SS	37		o	
				96.4						
		END OF BOREHOLE		3.5						
	4									
	5									
	6									
	7									
	8									
	9									

UTM Co-ordinates:
17 636773 E
4782541 N

NOTES:
Borehole dry upon completion of drilling.

SHEET 1 OF 1

A. CUMMINGS

7-10-5060-3.DWG

Terraprobe

LOG OF BOREHOLE 4

PROJECT No: 7-10-5060
CLIENT: Mr. David Edwards
LOCATION: 1268 Lakeshore Road, Burlington, Ontario

BORING DATE: July 21, 2010
ELEVATION DATUM: Local
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 75 - Truck - 150mm Solid Stem Augers

DEPTH SCALE IN METRES	DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	"N" VALUE	PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (20 40 60 80)	WATER CONTENT (%) (wP, w, wL; 10 20 30)	INSTALLATION INFORMATION
0	**GROUND SURFACE**		100.0						
	150mm Topsoil		0.0						
	compact, brown **FILL** sandy silt and topsoil		0.0	1	SS	14			
			99.2						
1	compact, brown **SANDY SILT** occasional seams or layers of silty sand and clayey silt		0.8	2	SS	17			
				3	SS	19			
2				4	SS	28			
3				5	SS	36			
			96.5						
	END OF BOREHOLE		3.5						
4									
5									
6									
7									
8									
9	**UTM Co-ordinates:** 17 636793 E 4782558 N								

NOTES:
Borehole dry upon completion of drilling.

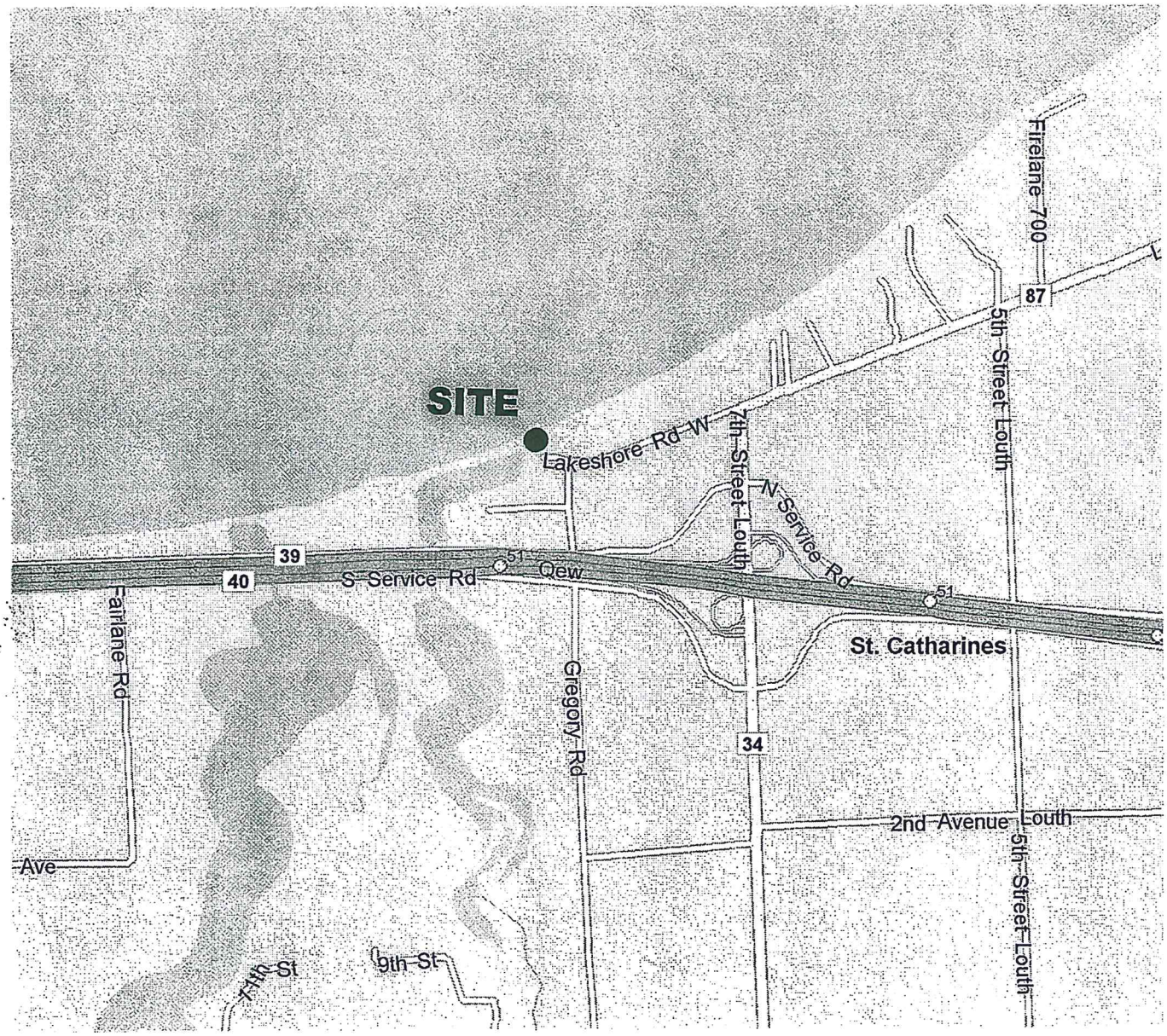
SHEET 1 OF 1

7-10-5060-4.DWG A. CUMMINGS

FIGURES

Terraprobe Inc.

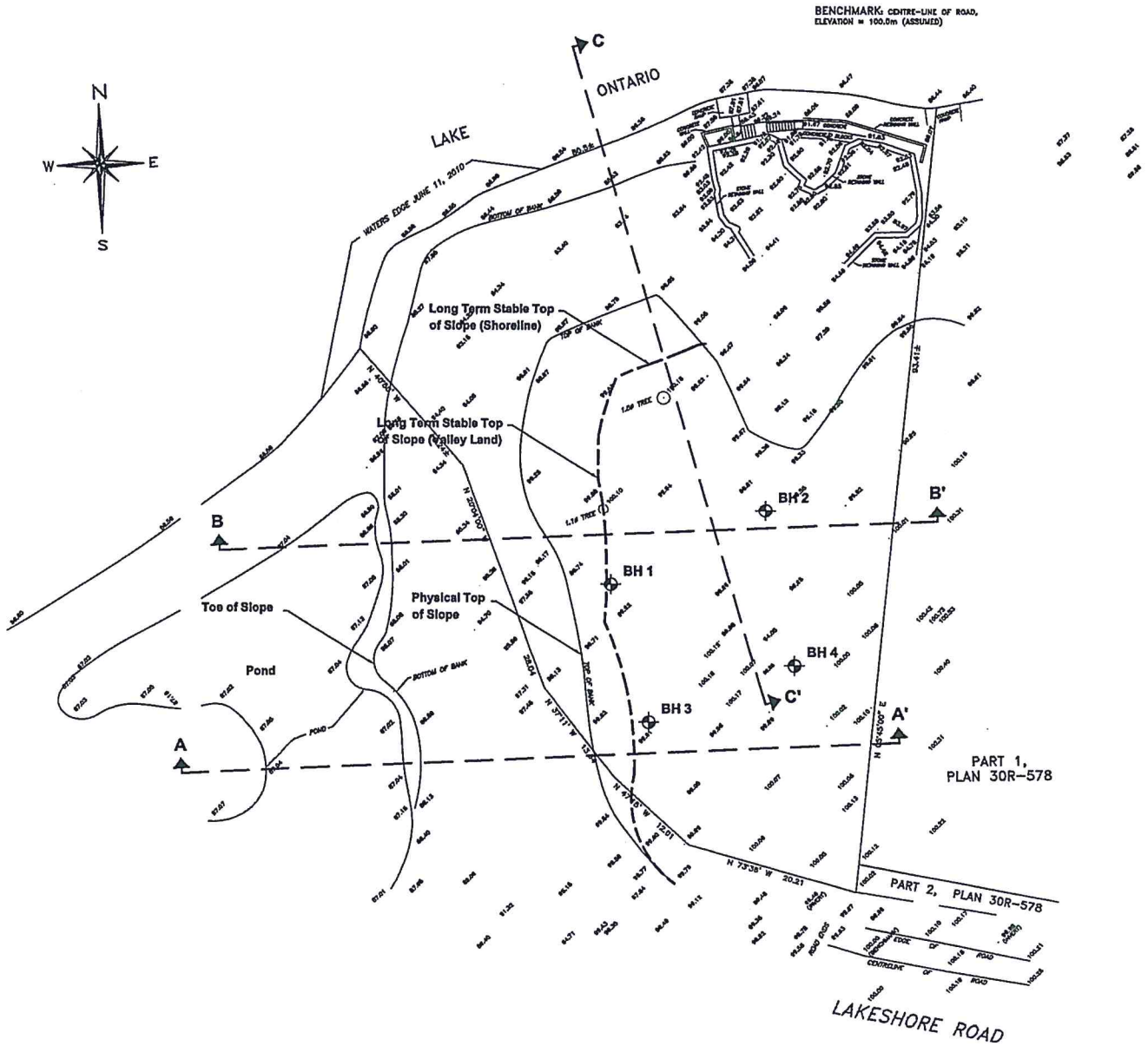




SITE LOCATION PLAN
1268 LAKESHORE ROAD
BURLINGTON, ONTARIO

Terraprobe
903 Barton Street, Unit 22
Stoney Creek, Ontario, L8E 5P5
(905) 643-7560 / Fax (905) 643-7559

Drawn By:	AC	Scale:	N.T.S.	Project No.:	7-10-5060
Checked By:	PC	Date:	AUGUST 2010	Figure No.:	1



SOURCE

WILLIAM A. MASCOE SURVEYING INC.
ONTARIO LAND SURVEYOR

SHOWING EXISTING BUILDINGS & ELEVATIONS
PART OF LOT 8,
BROKEN FRONT CONCESSION
GEOGRAPHIC TOWNSHIP OF LOUTH
IN THE
CITY OF ST. CATHARINES
REGIONAL MUNICIPALITY OF NIAGARA

METRIC

DISTANCES SHOWN ON THIS PLAN ARE IN METRES
AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048

NOTE

THE SURVEY WAS COMPLETED JULY 4, 2010

ELEVATON NOTE

ELEVATIONS ARE INFERRED TO LOCAL BENCHMARK
CENTRE-LINE OF ROAD
ELEVATION = 100.0m (ASSUMED)

LEGEND

A A' — Slope Section

BH 1 — Borehole Location

Stable Top of Slope

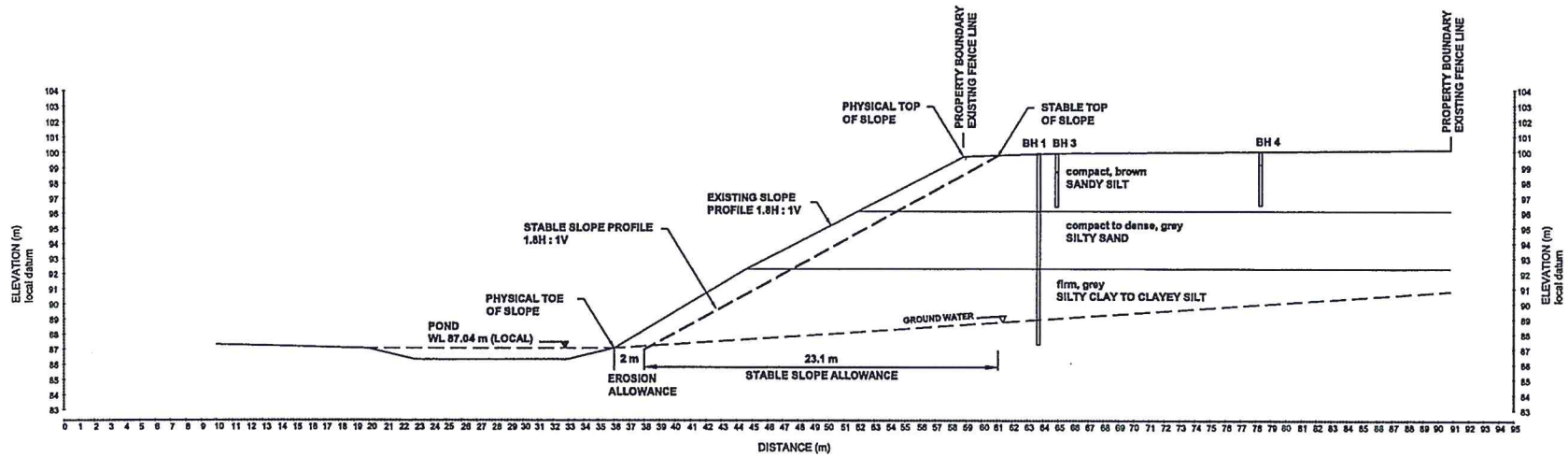
SCALE

0 2 4 6 8 10 20 30
metres

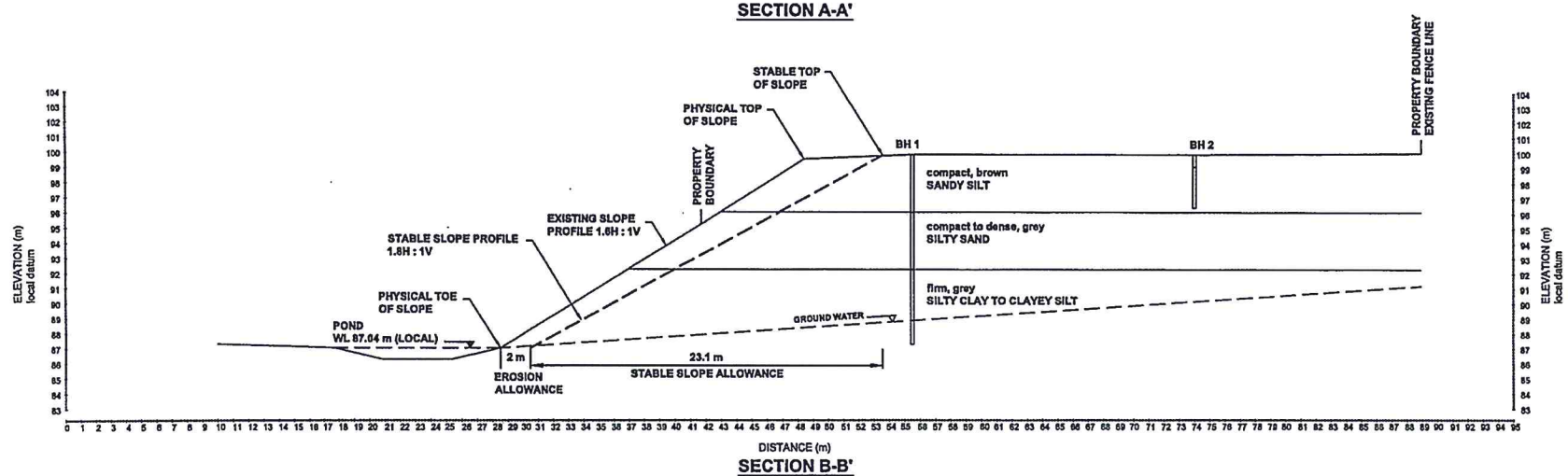
SITE PLAN
1268 LAKESHORE ROAD
BURLINGTON, ONTARIO

Terraprobe
903 Barton Street, Unit 22
Stoney Creek, Ontario, L8E 5P5
(905) 643-7560 / Fax (905) 643-7559

Drawn By: AC	Scale: 1:500	Project No.: 7-10-5060
Checked By: PC	Date: AUGUST 2010	Figure No.: 2



SECTION A-A'

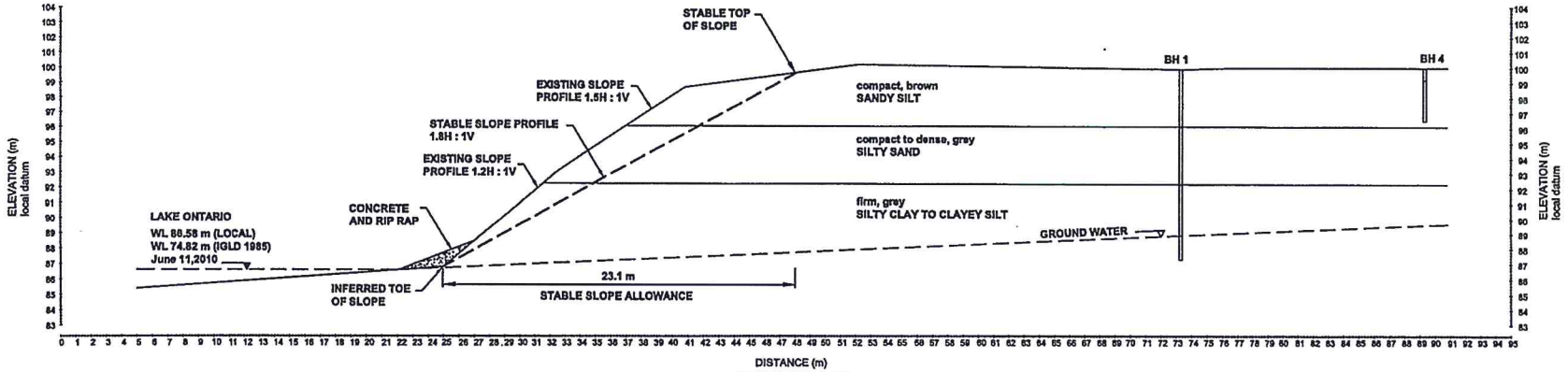


SECTION B-B'

SECTION A-A' & B-B'
1268 LAKESHORE ROAD
BURLINGTON, ONTARIO

Terraprobe
903 Barton Street, Unit 22
Stoney Creek, Ontario, L8E 5P5
(905) 643-7560 / Fax (905) 643-7559

Drawn By: AC	Scale: 1:300	Project No.: 7-10-5060	
Checked By: PC	Date: AUGUST 2010	Figure No.: 3	



SECTION C-C'

SECTION C-C'
1268 LAKESHORE ROAD
BURLINGTON, ONTARIO

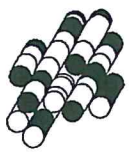
Terraprobe
903 Barton Street, Unit 22
Stoney Creek, Ontario, L8E 5P5
(905) 643-7560 / Fax (905) 643-7559

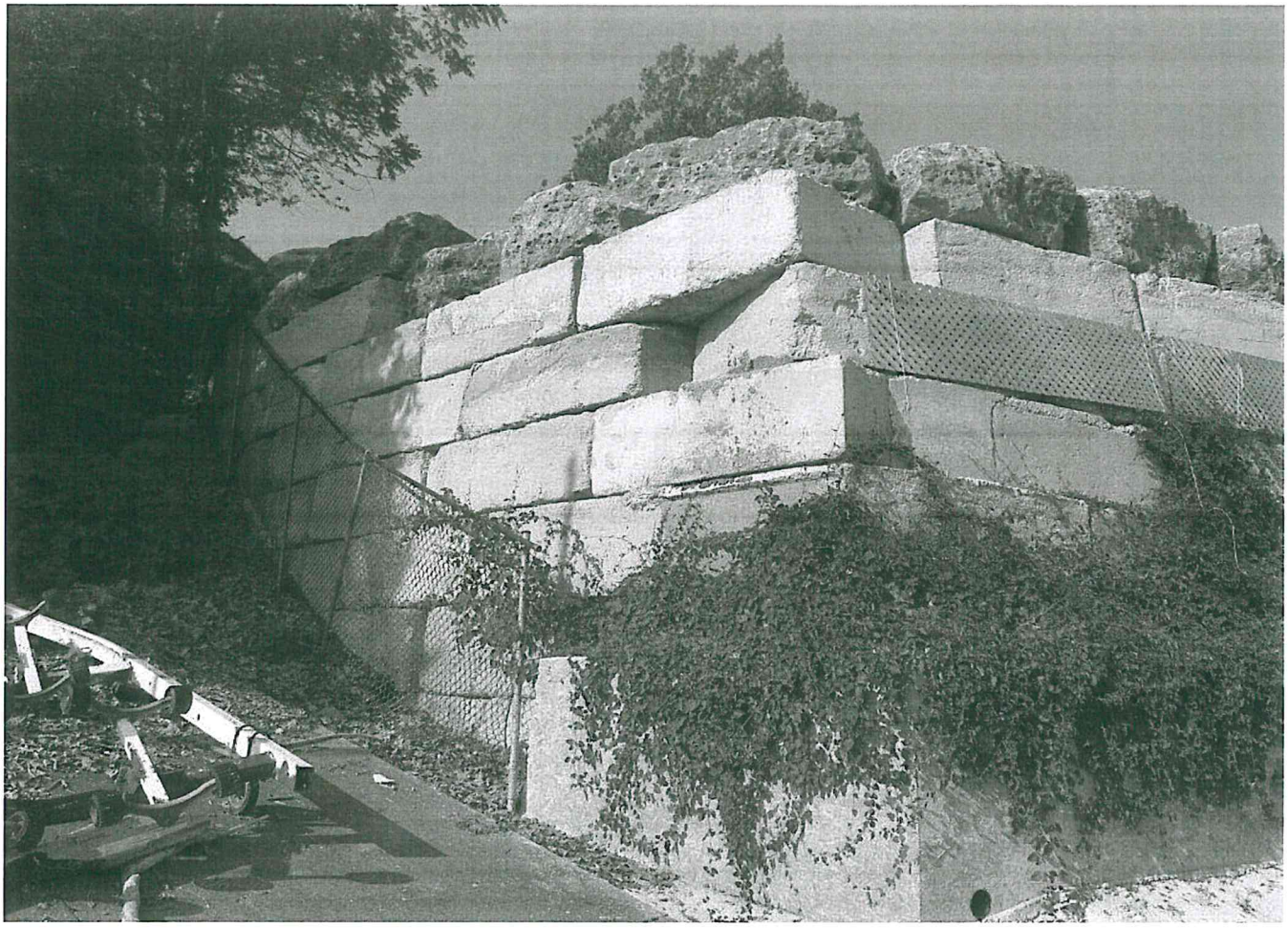
Drawn By:	AC	Scale:	1:300	Project No.:	7-10-5060
Checked By:	PC	Date:	AUGUST 2010	Figure No.:	4

PHOTOGRAPHS

APPENDIX A

Terraprobe Inc.



















SLOPE STABILITY ANALYSIS RESULTS

APPENDIX B

Terraprobe Inc.



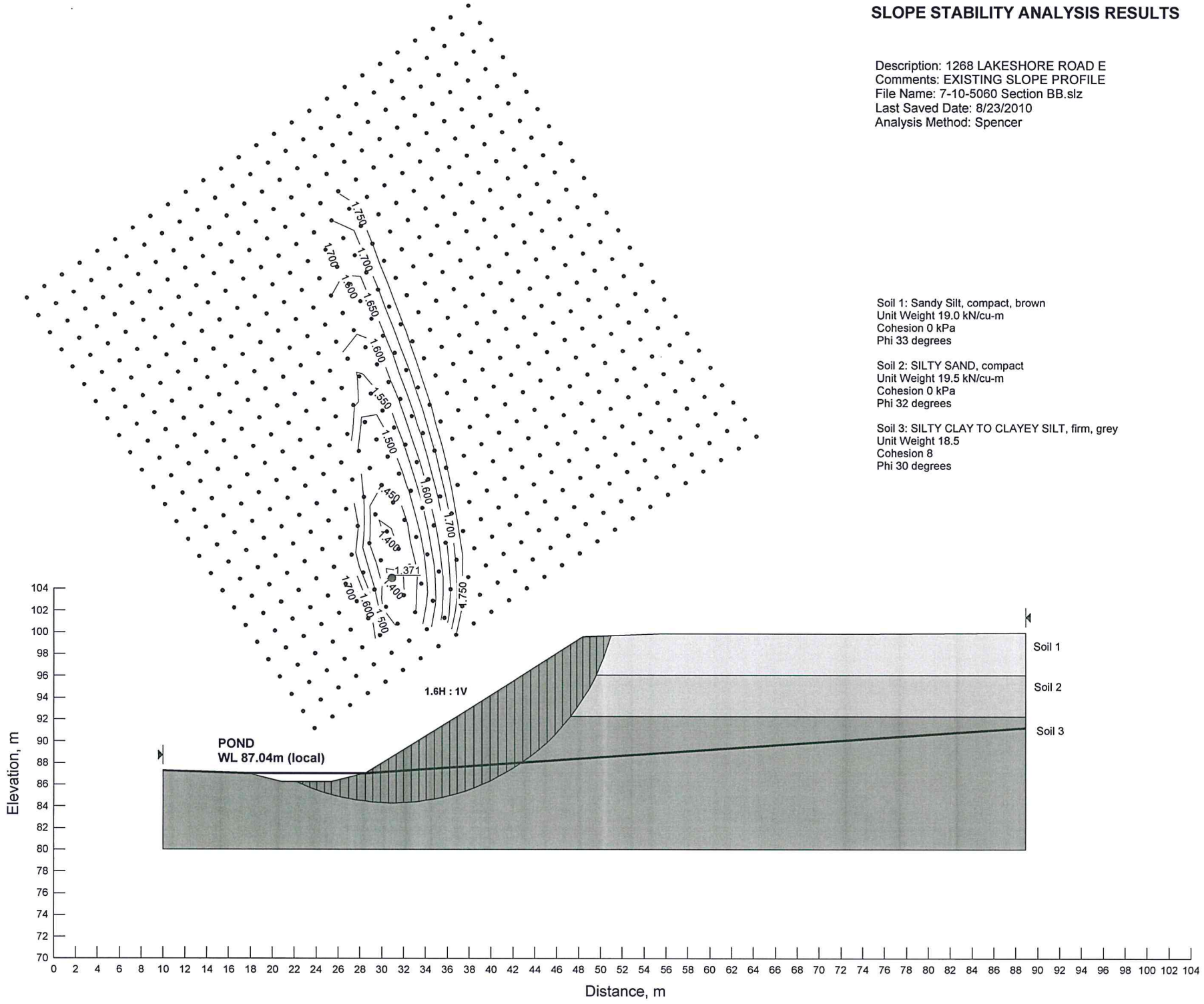
SLOPE STABILITY ANALYSIS RESULTS

Description: 1268 LAKESHORE ROAD E
Comments: EXISTING SLOPE PROFILE
File Name: 7-10-5060 Section BB.slz
Last Saved Date: 8/23/2010
Analysis Method: Spencer

Soil 1: Sandy Silt, compact, brown
Unit Weight 19.0 kN/cu-m
Cohesion 0 kPa
Phi 33 degrees

Soil 2: SILTY SAND, compact
Unit Weight 19.5 kN/cu-m
Cohesion 0 kPa
Phi 32 degrees

Soil 3: SILTY CLAY TO CLAYEY SILT, firm, grey
Unit Weight 18.5
Cohesion 8
Phi 30 degrees



SLOPE STABILITY ANALYSIS RESULTS

Description: 1268 LAKESHORE ROAD E
Comments: STABLE SLOPE PROFILE
File Name: 7-10-5060 Stable Slope.slz
Last Saved Date: 8/23/2010
Analysis Method: Spencer

Soil 1: Sandy Silt, compact, brown
Unit Weight 19.0 kN/cu-m
Cohesion 0 kPa
Phi 33 degrees

Soil 2: SILTY SAND, compact
Unit Weight 19.5 kN/cu-m
Cohesion 0 kPa
Phi 32 degrees

Soil 3: SILTY CLAY TO CLAYEY SILT, firm, grey
Unit Weight 18.5 kN/cu-m
Cohesion 8 kPa
Phi 30 degrees

