GEOTECHNICAL INVESTIGATION
PROPOSED NAVIGATION TOWER
SQUIRREL ISLAND
WALPOLE ISLAND FIRST NATION

Submitted to:

Fisheries and Oceans Canada
Coast Guard
Central and Arctic Region
201 Front St. N, Suite 703
Sarnia, Ontario, N7T 8B1

Attention: Adam Wettges
Project Engineer

Submitted by:
AMEC Earth & Environmental
a division of AMEC Americas Limited
3096 Devon Drive, Unit 30
Windsor, Ontario
N8X 4L2

Tel +1 (519) 969-7530
Fax +1 (519) 969-0160

6 March 2009

SW0508053

Distribution:
- Coast Guard – 1 Electronic and 1 Hard Copy
- AMEC Earth & Environmental – 1 Electronic and 1 Hard Copy

TABLE OF CONTENTS

5.0 CLOSING REMARKS 10

1.0 INTRODUCTION

AMEC Earth & Environmental, a division of AMEC Americas Limited ("AMEC"), was retained by Fisheries and Oceans Canada (FOC) to complete a geotechnical investigation for a proposed replacement Aids to Navigation tower in Squirrel Island, in the Walpole Island First Nation. A key plan showing the general location of the site is provided on Figure 1. The new tower is to be situated near an existing roadway.

The purpose of the investigation was to determine the subsurface soil conditions at the site in order to prepare a geotechnical evaluation of the foundation soils along with general guidelines for the geotechnical aspects of the design. The scope of the fieldwork was to advance one sampled borehole to refusal followed by a 3 m core to prove the bedrock.

The purpose of the present information is to assist with the preliminary selection of the foundation options and associated construction methods. The final design and construction details should be established only after the assumptions in this report are confirmed by the field and laboratory testing as planned.

This report was prepared with the assumption that due consideration will be given to all applicable standards, codes, regulations of authorities having jurisdiction and with good engineering practice. Further, the recommendations and opinions in this report are applicable only to the subject project described above. The construction conditions discussed in this report are intended primarily to assist in the design decisions. Contractors should be aware that the data and their interpretations presented in this report may not be sufficient to assess all factors that may have an impact on the construction process.

There should be an ongoing liaison with AMEC during both the design and construction phases of this project to ensure that the recommendations in this report have been interpreted and implemented as intended. Also, if any further clarification and/or elaboration are needed concerning the geotechnical aspects of this project, AMEC should be contacted immediately.

The scope of this project is limited strictly to the geotechnical aspects of the proposed development.

2.0 INVESTIGATIVE PROGRAM

One borehole was completed on the roadway embankment approximately 12 m (40 ft) north of the proposed tower site, as shown on Figure 2. The fieldwork portion of the investigation was carried out on the 14th and 15th of December 2008. The location of the proposed tower in the marsh was not accessible by conventional drill rig therefore the borehole was advanced through the top of the roadway embankment, as near as possible (approximately 12.2 m north) to the coordinates provided by the client. The borehole was advanced using a self-propelled drill equipped with hollow stem augers and conventional soil sampling tools. Soil samples were taken at frequent intervals of depth following the Standard Penetration Test (ASTM D-1586) procedure. At select depths, samples were also taken using a 70 mm by 610 mm long, thin-walled sampler (Shelby Tube). This sampling method retrieves relatively undisturbed samples. At selected depths, field vane tests were taken to assess the in-situ shear strength of the soils.

After completion of the borehole, the augers were extracted, the hole was inspected for groundwater and caving, and the borehole was backfilled using bentonite-cement grout.

All samples were field logged, placed in airtight containers, and transported to our Sarnia laboratory for additional examination and testing.

The elevation of the ground surface at the borehole location was assigned an arbitrary value of El. 100.0 m.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site

Squirrel Island is part of the Walpole Island First Nation, which is located at the delta of the St. Clair River at the north end of Lake St. Clair. The island is low-lying and quite flat, with a system of dikes and drains constructed around the perimeter. The northern smaller portion of the island is drained and utilised as farmland and accommodates a few residential properties. The southern portion of the island is essentially marshland connected to the main river and lake system, and is situated above the general lowland farmland. Drainage from the farmland is pumped back into the marshland.

The main roadway leading to the proposed tower location is an earthen embankment that forms the main southern levy separating the marshland from the farmland. At the borehole location, the top of the embankment sloped downward approximately 0.3 m from south to north and the sides of the embankment were very steep. At the time of our fieldwork (14th and 15th December, 2008), the ice level on the south side of the embankment was approximately 0.6 m below the top of embankment and the water level in the ditch on the north side was approximately 1.55 m below the top of embankment.

Occasional flooding of the farmland occurs during prolonged wet seasons. The top of the road can become very muddy and inaccessible to conventional vehicular traffic.

3.2 Geological Background

The site is located within the St. Clair Delta, a subdivision of the physiographic region called the St. Clair Clay Plain (Chapman & Putnam, 1984) which is a flat-lying area composed of glaciolacustrine and alluvial clays and silts. The St. Clair Delta is a well-developed ‘bird’s foot’ delta consisting of fine sands and silts deposited over the proglaciar silty clay base by streams in higher level post-glacial lakes that occupied the basin. The delta has continued to undergo construction by the St. Clair River, which enters Lake St. Clair on the northwest flank of the delta (Quaternary Geology of the Wallaceburg-St. Clair Flats Area, Ontario Geological Survey, 1980).

The upper bedrock below the quaternary deposits is of the Upper Devonian Kettle Point Formation, which is a black, highly-fissile, non-calcareous shale with minor interbeds of grey-green silty shale and large spherical or sub-spherical limestone concretions known as “kettles.”

3.3 Soils

The findings in the borehole are detailed on the Borehole Log Sheets (Appendix A). The results of laboratory testing carried out on select samples are also shown on the Borehole Log Sheets. The following is only a summary of the conditions at the borehole.

The nature and condition of the soil samples were based on visual and tactile examination, augmented with field tests (Standard penetration and field vane tests) and select laboratory tests (natural moisture contents).

The borehole was advanced through 1.1 m of grey-black silty sand mixed with roots and decaying vegetation (fibrous peat). These materials were saturated, and are inferred to be fill materials (probably from the immediate vicinity) based on the height of the embankment above the surrounding grades. Beneath this was 1.2 m of very soft, black organic clay with saturated seams of silt and sand and inclusions of peat with rootlets.

Underlying the embankment materials were deltaic soils consisting of silt and fine sand. These soils included traces of peat and rootlets, gradually diminishing with increasing depth. At approximately 5.3 m below the top of the embankment, the borehole encountered a major stratum of intermediate plasticity clay with silt laminations. The upper 2 m (+/-) formed a crust of stiff consistency, but the consistency quickly reduces to soft and very soft below. The moisture content of the clay varies from approximately 26% to 46%. Consolidation tests completed on these soils for other projects in the Walpole Island First Nation have indicated that the clay is slightly overconsolidated, most likely due to aging.

Beginning at a depth of 36.3 m, the borehole encountered compact to very dense till materials consisting of silt, sand and gravel in varying proportions. Beginning at a depth of approximately 41.1 m, natural gas was noted bubbling up through the drilling mud.

The borehole was terminated at a depth of 46.1 m below the top of embankment when a shale fragments were retrieved in the sampler. The blow counts at this depth were 50 blows per 50 mm of penetration.

3.4 Groundwater

Long-term groundwater table is determined by the river levels that fluctuate very near the ground surface. Short-term groundwater levels are influenced by the prevalent precipitation regime and by the local site drainage system. In adverse conditions flooding of the drained land occur almost periodically. The marshland area is usually covered by water at all times.

3.5 Seismic Design

The closest site listed in the Ontario Building Code 2006 (OBC 2006) is Wallaceburg, for which seismic design values are 5% damped base spectral acceleration $S_a(0.2) = 0.18$ and a design Peak ground Acceleration, PGA, of 0.110.

The Site Classification for the Seismic Response analysis should be considered Class E – Soft Soils.

3.6 Climatic Data

The depth of frost penetration below the ground surface at this site is assumed at an average of 1.15 m. For practical purposes, foundations should be protected from weathering by a minimum of 1.2 m of soil cover, or an equivalent layer of synthetic insulation.

Variations from the above noted depth of frost penetration might be considered, depending on various factors such as the type of backfilling materials or the temperature and moisture exposure of the area (i.e. prevailing winds or drifting snow). However, these variations do not generally become a concern unless special equipment and/or buried utilities have stringent requirements regarding the subsurface temperature and moisture regime (i.e. water lines or sensitive electrical utilities etc.). In such special situations further tests and analyses should be conducted on a case-by-case basis.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 Shallow Conventional Foundations

Conventional footings at this site are marginally feasible only for lightly-loaded conventional structures. In such cases the foundation grades should be established on native undisturbed inorganic soils, below the marshland peat. Although the present borehole was not advanced within the footprint of the proposed tower foundation, it may be assumed that the stiff silty clay stratum is continuous and will provide a relatively suitable bearing stratum at approximately 5.3 m below the top of embankment. For such soils, a preliminary footing design may be based on an average of 40 kPa net soil stress increase at the foundation level.

Based on the above and in conjunction with the indicated tower loads in the Request for Quotation, consideration may be given to the construction of a concrete mat foundation. The necessary size (diameter) of the mat is estimated to be on the order of 6.5 m (±) in order to provide stability to overturning and uplift, while keeping the net soil stress increase below 40 kPa. It was assumed that the loads provided were unfactored.

Another option to implement conventional foundations may consider anchor blocks and guy-wires. Such option would increase though the footprint of the facility, including the extent of the backfill pads necessary to create access and working surfaces within the marshland.

4.2 Pile Foundations

4.2.1 General Considerations

An alternative foundation is represented by steel piles driven to practical refusal on bedrock or within the dense to very dense till. From other projects in the region it is known that high capacity end bearing pile foundations are feasible at the site. The most usual pile system in this area consists of low-displacement end bearing steel driven piles (H piles) or steel pipe piles. It is anticipated that the piling will be carried out from the surface of a backfill pad built in the marsh.

The pile response under the applied loads depends on the pile type (structure and method of installation), foundation soils, foundation configuration (pile layout, spacing, structural connections, etc), and load types (static, dynamic, vertical-horizontal, etc).

If the piles are properly set to practical refusal, on bedrock, the available pile geotechnical capacity to compression loads is comparable to the pile structural capacity. However, the geotechnical resistance factors differ from the structural element factors. In particular, the resistance factor to axial compression would vary from 0.4 to 0.6 depending on the level of confidence in the mobilization of the geotechnical capacity. Piles driven only partially into the dense till without reaching bedrock (floating piles) may develop somewhat lower geotechnical capacity than piles set on the bedrock, however, a cost-capacity based analysis may show that "floating piles" option is preferable.

The pile foundation response for uplift and resistance to lateral loading relies on the soil-pile interaction mechanism. The determining design factors should be based on the strain-stress-strength characteristics of the weak silty clay deposit. Ideally, the lateral and uplift pile capacities should be determined on the basis of field tests. In the absence of such tests, semi-empirical methods may be used in conjunction with recognized design methods and sound engineering practices. Guidelines for geotechnical design are provided in the Canadian Foundation Engineering Manual (CFEM) using laboratory and in-situ test data. The resistance factor varies from 0.3 to 0.4, depending on the level of field confirmation by testing of the pile capacity.

Once the pile spacing is less than 8 pile diameters, the uplift capacity will decrease with decreasing distance between the adjacent piles and with the increase in number of piles under the footings. Complex soil-pile interaction analyses are necessary in these cases.

The lateral pile capacity of an isolated vertical pile is expected to be rather low. The lateral soil support against the pile shaft within the upper organic deposits should be completely disregarded. The foundation capacity for lateral loads can be significantly increased by pile grouping and by using inclined (batter) piles.

At some sites in this region, the use of driven steel pipe piles has resulted in ground heave when the pipe piles are installed without prior pre-drilling of pilot holes, however, given the very weak soils at the site, pre-drilling might not be necessary in this case.

In the event that any distinct amounts of fill are placed around the tower base, negative skin friction (NSF) may develop on the piles. However, for properly designed and installed end bearing piles bearing on bedrock the NSF should not cause any significant impact.

The assumed pile capacities must be confirmed in the field by appropriate testing and inspection methods.

4.2.1 Examples of Pile Capacities

The following examples assume the steel mill rate Fy > 300 Mpa. The piles are assumed to be installed to refusal on bedrock using adequate hammer energy confirmed in-situ by appropriate testing (PDA, elastic rebound, etc). For the calculation purposes in these examples the pile tip was assumed at 42.5 m below grade (embedded in dense sand and gravel) and the invert of the pile cap at 2.5 m below grade. The piles were assumed widely spaced so that no capacity reduction due to stress overlapping was incorporated. The soil-pile gaps were assumed to be adequately grouted.

The conventional lateral capacity was determined for a free-headed vertical pile loaded by a horizontal load at the ground surface. In the case of a restrained pile head (such as the case of the piles structurally embedded within a rigid concrete pile cap), the conventional lateral capacity could increase by at least 50% compared to the free-headed pile.

Pile Type	**A_s (cm^2)**	**W (cm^3)**	**R_{uc} (kN)**	**M_y (kN-m)**	**H_{us} (kN)**	**H_{uc} (kN)**	**R_{ups} (kN)**	**R_{upc} (kN)**
HP 310x110	141	1537	2800-3500	460	270	155	1000-1200	300-400
		498		150	135	90		
HSS 219x8	53	209	1100-1350	63	75	50	560-670	185-225

A_s : ***Area of steel section***
W : ***Section modulus***
R_{uc} : ***Anticipated maximum Unfactored Geotechnical Capacity to compression***
M_y : ***Assumed Structural yield moment***
H_{us} : ***Anticipated conventional Unfactored Lateral Load capacity to static loads***
H_{uc} : ***Anticipated conventional Unfactored Lateral Load capacity to cyclic loads***
R_{ups}: ***Anticipated Unfactored Uplift Capacity to static loads***
R_{upc}: ***Anticipated Unfactored Uplift Capacity to cyclic loads***

To determine the Factored Resistances, typical resistance factors are provided in the CFEM.

From the above table it appears that one HP pile would comfortably offer necessary capacity to compressive loads, and perhaps to lateral loads under one leg of the tower. However, there will be necessary extra piles to provide the necessary anchoring capacities.

Our office has experience and will be happy to assist with detailed recommendations for the geotechnical pile design to static, dynamic and cyclic loads.

4.2.2 Driving Criteria

The selection of the proper driving equipment is essential in order to ensure that the pile is driven to the desired geotechnical capacity in an effective manner and without causing structural damage. No unusually 'hard' driving conditions are anticipated at this site. However, the glacial nature of the deeper layers carries the potential for scattered boulders or lenses of hard till at the base of the soft postglacial deposit. Therefore, the use of medium-gauge steel piles is recommended.

The optimal selection of the driving equipment should be finalized by field trial using PDA monitoring, or another recognized method. For preliminary purposes, the hammer should be rated at least 60 kJ/blow and 35 kJ/blow for the above examples of high capacity H-piles and lower capacity pipe piles, respectively.

4.3 Construction Considerations

It is anticipated that engineered backfill pads, and access embankments will be created within the marshland. Depending on the amount of water in the marsh and the depth of organics, there are different options available for the construction of the pads, such as:

- removal of the water and organics and replacement by engineered fill
- floating the embankment over the marsh (corduroy construction)
- displacement of the organics by surcharge

Temporary excavations within the existing fill and organic deposits should be classified as Type 4. Temporary excavations within dewatered engineered fills should be conducted in conformance with the work safety requirements applicable to Type 3 soils.

Below the groundwater table, seepage will occur. If flowing sand and silt conditions develop, then the soils should be classified as Type 4 soils. Consideration should be given to the potential for basal instability of excavations due to groundwater seepage through sand/silt and/or soft clay subgrade.

In the case of conventional foundations, positive dewatering using well point or other type of filtered sumps / cut-off drains must be used to prevent any upward flow from the bottom of the excavation. Alternatively, cut-off walls keyed into the deeper clayey soils must be considered.

Once the bearing surface is exposed and approved, a minimum of 150 mm of protective lean concrete (mud mat) should be placed. Any vehicular traffic over unprotected subgrade should be strictly prohibited.

Winter construction should include provisions to prevent freezing of the subgrade at all times.

The on-site sand and silt can be used as backfill material providing the cuttings are sorted from foreign materials, and providing the moisture content at the time of backfilling is within 2% of the optimum moisture content. It should be noted that these soils are currently wetter than the optimum range for compaction purposes. Once properly conditioned, the on-site sand and silt should be placed in lifts no thicker than 150 mm in loose state and compacted to at least 95% of the Standard Proctor maximum density (SPMDD). Subject to sufficient testing to prove otherwise, the anticipated minimum bulk unit weight of such backfills is 17 kN/m^3.

5.0 CLOSING REMARKS

All the above design recommendations are strictly intended to be used for preliminary feasibility studies. A close interaction between the Structural and Geotechnical Engineers is necessary to ensure conformance with the geotechnical assumptions and limitations considered.

For final design purposes adequate field investigations, laboratory testing and geotechnical analyses are necessary at the location of the tower.

We trust this report is complete within the terms of reference. However, if any questions arise regarding the content of this report, please do not hesitate to contact us.

Sincerely,
AMEC Earth & Environmental
a Division of AMEC Americas Limited

Matt Oldewening, P.Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
M. S. OLDEWENING
06 March 2009
PROVINCE OF ONTARIO

Dan Dimitriu, Ph.D., P.Eng.
Senior Engineer

matt.
oldewening

Digitally signed by matt.oldewening
DN: cn=matt.oldewening,
email=matt.oldewening@amec.com
Date: 2009.03.06 13:38:54 -05'00'

APPENDIX A

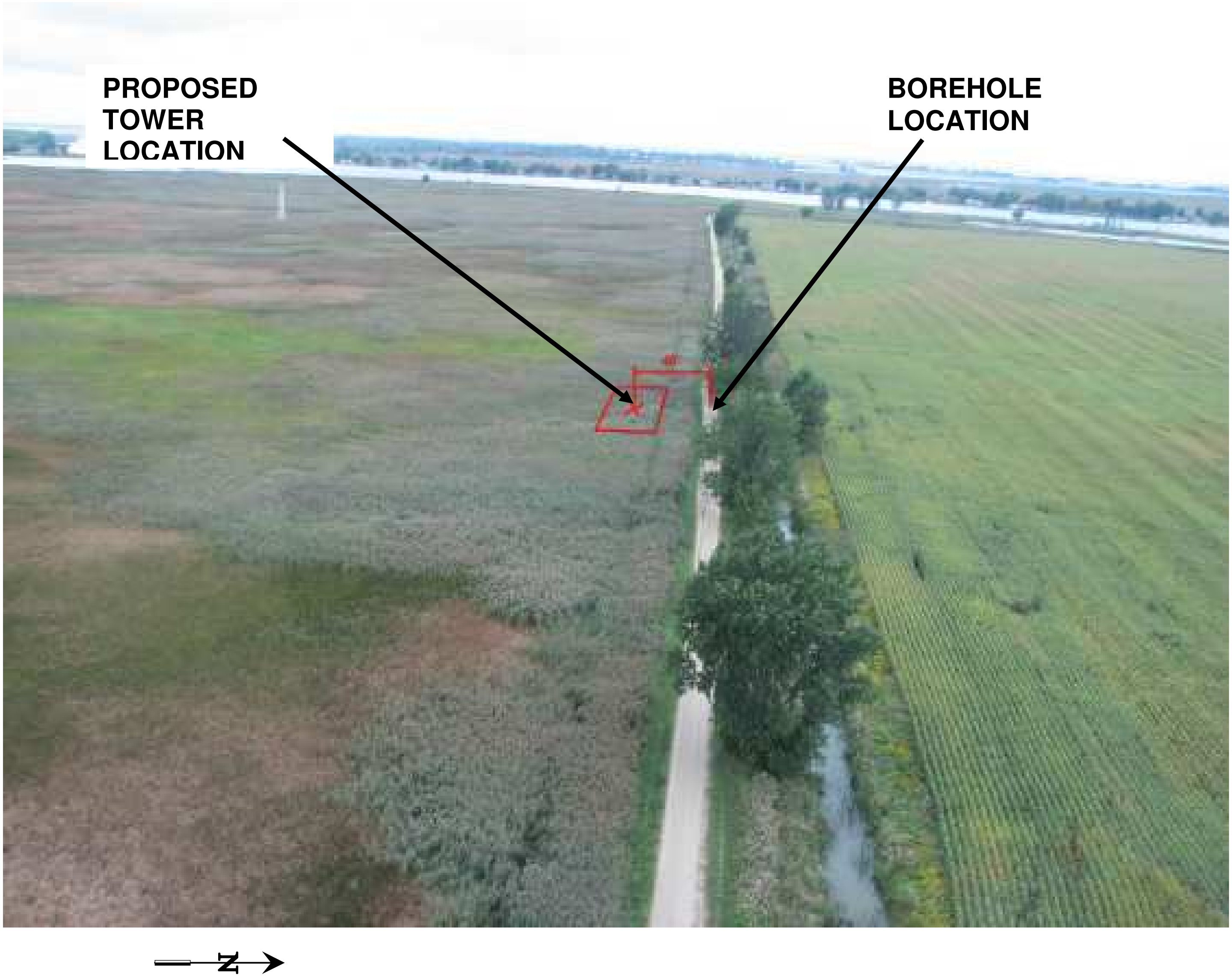
ENCLOSURES

Site Location

Closeup Site Location

figures provided by Coast Guard

amec	FIGURE 1	SITE LOCATION PLAN SQUIRREL ISLAND, WALPOLE ISLAND FIRST NATION	PROJECT:	GEOTECHNICAL INVESTIGATION SQUIRREL ISLAND REAR RANGE NAVIGATION TOWER		
3096 DEVON DRIVE, SUITE 30 **N8X 4L2** **WINDSOR, ON** **1-519-969-7530**	CLIENT:	COAST GUARD	DRAWN BY:	MO	DATE:	MARCH 2009
	JOB NUMBER:	SW0508053	REVIEWED BY:	MO	SCALE:	NTS



amec	FIGURE 2	BOREHOLE LOCATION PLAN SQUIRREL ISLAND, WALPOLE ISLAND FIRST NATION	PROJECT:	GEOTECHNICAL INVESTIGATION SQUIRREL ISLAND REAR RANGE NAVIGATION TOWER		
3096 DEVON DRIVE, SUITE 30 N8X 4L2 WINDSOR, ON 1-519-969-7530	CLIENT:	COAST GUARD	DRAWN BY:	MO	DATE:	MARCH 2009
	JOB NUMBER:	SW0508053	REVIEWED BY:	MO	SCALE:	NTS

RECORD OF BOREHOLE No. 1

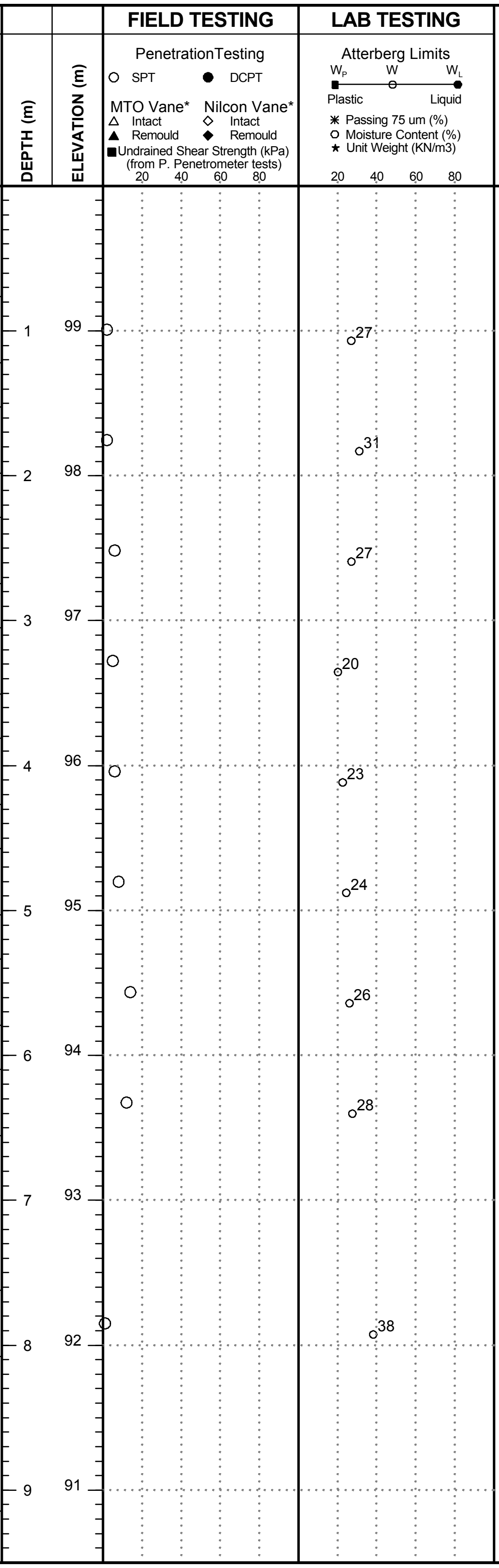
Project Number: **SW0508053**
Project Client: **Coast Guard**
Project Name: **Squirrel Island Rear Range Navigation Tower**
Project Location: **Walpole Island First Nation**

Drilling Location: **Squirrel Island**
Drilling Method: **200 mm Hollow Stem Augers**
Drilling Machine: **CME 75**
Date Started: **14 Dec 08** Date Completed: **15 Dec 08**

Logged by: **LEC**
Compiled by: **TLP**
Reviewed by: **MSO**
Revision No.: **0, 6/3/09**

Local Ground Surface Elevation: 100.0 m

LITHOLOGY PROFILE		SOIL SAMPLING					
DESCRIPTION	Elevation / Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value	Moisture Content (%)	COMMENTS
Grey-black SILTY SAND with roots and decaying vegetation, saturated (FILL)	98.9	SS	1	100	2	27	
Very soft, black ORGANIC CLAY with saturated seams of silt and sand, peat inclusions and rootlets (Possible FILL and/or disturbed)	1.1	SS	2	100	2	31	
	97.7						
Loose SILT and SAND, saturated with peat and rootlets	2.3	SS	3	100	6	27	
	97.0						
Loose, grey fine SAND, saturated trace rootlets	3.0	SS	4	54	5	20	
		SS	5	54	6	23	
silt lenses/laminations		SS	6	100	8	24	
	94.7						
SILTY CLAY grey	5.3						
stiff		SS	7	67	14	26	
silt laminations up to 15mm		SS	8	100	12	28	
------------- very soft							
		SS	9	100	1	38	
		TW	10	75			



INSTRUMENTATION INSTALLATION

AMEC Earth & Environmental
A division of AMEC Americas Limited
3096 Devon Drive, Suite 30
Windsor, Ontario N8X 4L2
Tel 519-969-7530
Fax 519-969-0160
www.amec.com

∇ No freestanding groundwater measured in open borehole on completion of drilling.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Explanation of Borehole Log'.

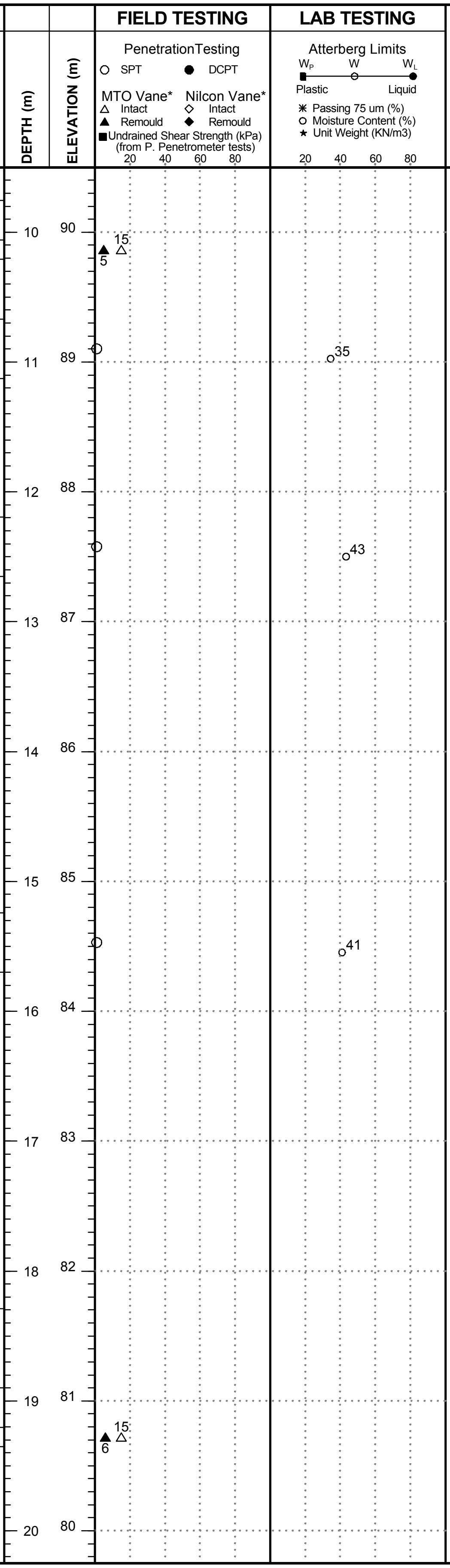
Scale: 1 : 50

Continued on Next Page

RECORD OF BOREHOLE No. 1

Project Number: **SW0508053** Drilling Location: **Squirrel Island** Logged by: **LEC**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value
▨	SILTY CLAY				
		VT			
		SS	11	100	1
		SS	12	100	1
		SS	13	100	1
		TW	14	100	
		VT			



INSTRUMENTATION INSTALLATION

COMMENTS

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Explanation of Borehole Log'.

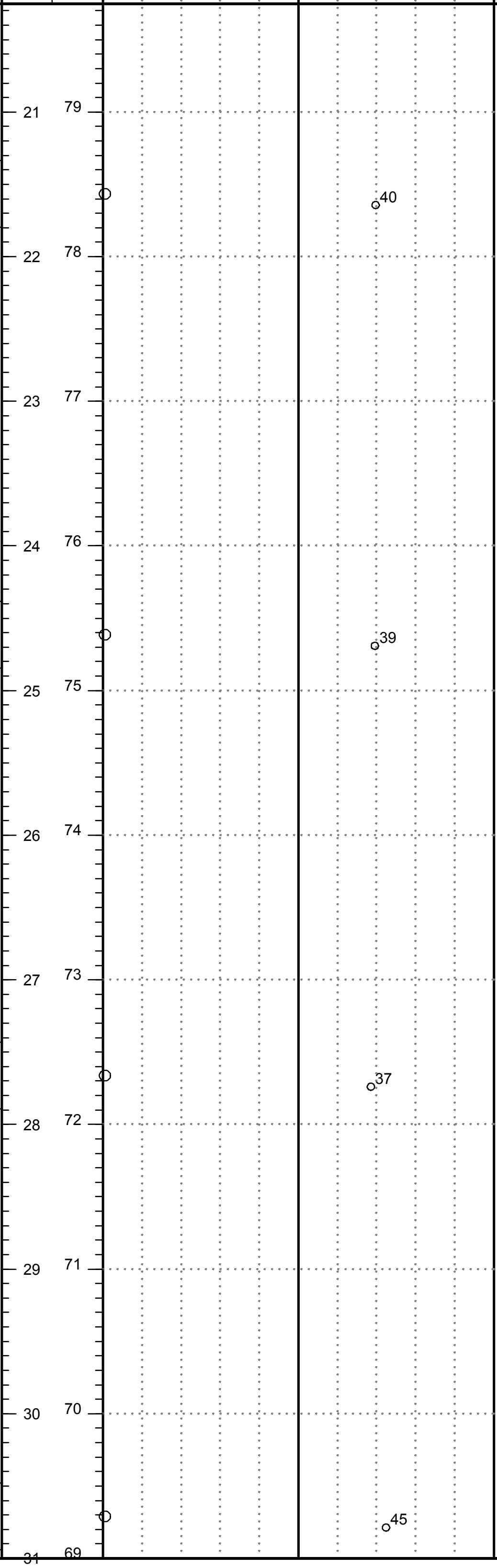
Scale: 1 : 50

Continued on Next Page

RECORD OF BOREHOLE No. 1

Project Number: **SW0508053** Drilling Location: **Squirrel Island** Logged by: **LEC**

LITHOLOGY PROFILE		SOIL SAMPLING						FIELD TESTING	LAB TESTING		
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value	DEPTH (m)	ELEVATION (m)	PenetrationTesting ○ SPT ● DCPT MTO Vane* Nilcon Vane* △ Intact ◇ Intact ▲ Remould ◆ Remould ■ Undrained Shear Strength (kPa) (from P. Penetrometer tests) 20 40 60 80	Atterberg Limits W_P W W_L Plastic Liquid ✱ Passing 75 um (%) ○ Moisture Content (%) ★ Unit Weight (KN/m3) 20 40 60 80	INSTRUMENTATION INSTALLATION	COMMENTS
▧	SILTY CLAY										
		SS	15	100	1				40		
		SS	16	100	1				39		
		SS	17	100	1				37		
		SS	18	100	1				45		



Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Explanation of Borehole Log'.

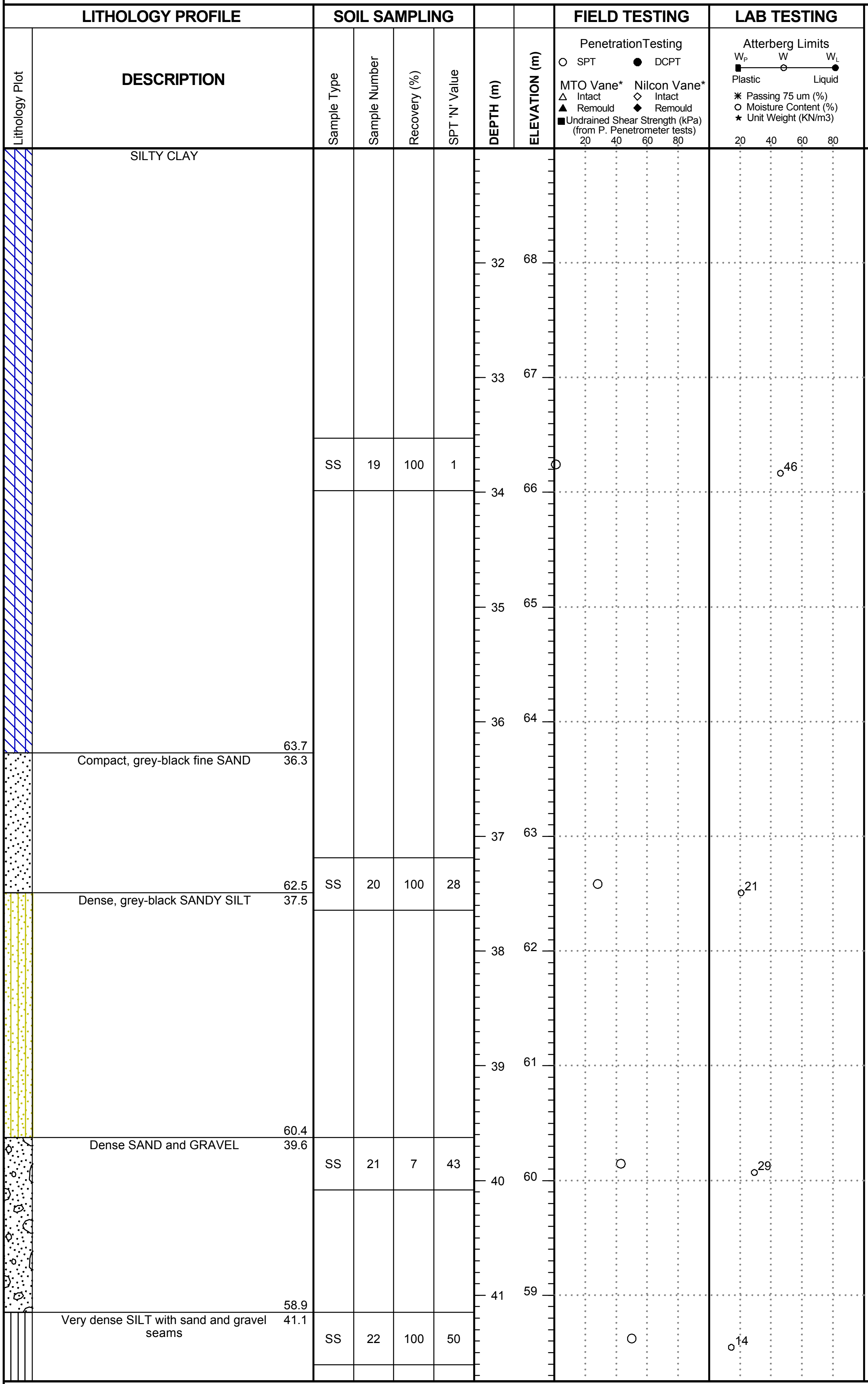
Scale: 1 : 50

Continued on Next Page

RECORD OF BOREHOLE No. 1

Project Number: **SW0508053** Drilling Location: **Squirrel Island** Logged by: **LEC**

DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value	DEPTH (m)	ELEVATION (m)	LAB TESTING	COMMENTS
SILTY CLAY								
	SS	19	100	1			46	
Compact, grey-black fine SAND					36.3	63.7		switch to 'H' casing and tricone at 35m
	SS	20	100	28			21	
Dense, grey-black SANDY SILT					37.5	62.5		
								end for the day on 14 Dec 08 start 15 Dec 08
Dense SAND and GRAVEL					39.6	60.4		
	SS	21	7	43			29	no recovery on SPT, split spoon driven again
Very dense SILT with sand and gravel seams					41.1	58.9		natural gas bubbling through drilling mud
	SS	22	100	50			14	50 blows per 50mm of penetration



Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Explanation of Borehole Log'.

Scale: 1 : 50

Continued on Next Page

RECORD OF BOREHOLE No. 1

Project Number: **SW0508053** Drilling Location: **Squirrel Island** Logged by: **LEC**

LITHOLOGY PROFILE		SOIL SAMPLING						FIELD TESTING	LAB TESTING		
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value	DEPTH (m)	ELEVATION (m)	PenetrationTesting ○ SPT ● DCPT MTO Vane* Nilcon Vane* △ Intact ◇ Intact ▲ Remould ◆ Remould ■ Undrained Shear Strength (kPa) (from P. Penetrometer tests) 20 40 60 80	Atterberg Limits W_P W W_L Plastic Liquid ✱ Passing 75 um (%) ○ Moisture Content (%) ★ Unit Weight (KN/m3) 20 40 60 80	INSTRUMENTATION INSTALLATION	COMMENTS
	Very dense SILT with sand and gravel seams										
						42	58				
		SS	23	100	82	43	57	○	○ 11		
						44	56				
						45	55				
	54.0	SS	24	100	50	46	54	○	○ 11		50 blows per 50mm of penetration
	SHALE 46.0 53.9										
	END OF BOREHOLE - REFUSAL 46.1										
						47	53				
						48	52				
						49	51				
						50	50				
						51	49				
						52	48				

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Explanation of Borehole Log'.

Scale: 1 : 50

APPENDIX B

GENERAL REPORT NOTES

EXPLANATION OF BOREHOLE LOG

This form describes some of the information provided on the borehole logs, which is based primarily on examination of the recovered samples, and the results of the field and laboratory tests. Additional description of the soil/rock encountered is given in the accompanying geotechnical report.

GENERAL INFORMATION

Project details, borehole number, location coordinates and type of drilling equipment used are given at the top of the borehole log.

SOIL LITHOLOGY

Elevation and Depth

This column gives the elevation and depth of inferred geologic layers. The elevation is referred to the datum shown in the Description column.

Lithology Plot

This column presents a graphic depiction of the soil and rock stratigraphy encountered within the borehole.

Description

This column gives a description of the soil stratums, based on visual and tactile examination of the samples augmented with field and laboratory test results. Each stratum is described according to the *Modified Unified Soil Classification System*.

The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined as follows *(Ref. Canadian Foundation Engineering Manual)*:

Compactness of Cohesionless Soils	SPT N-Value
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of	Undrained Shear Strength	
Cohesive Soils	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	Over 200	Over 4000

Soil Sampling

Sample types are abbreviated as follows:

SS	Split Spoon	TW	Thin Wall Open (Pushed)	RC	Rock Core	GS	Grab Sample
AS	Auger Sample	TP	Thin Wall Piston (Pushed)	WS	Washed Sample	AR	Air Return Sample

Additional information provided in this section includes sample numbering, sample recovery and numerical testing results.

Field and Laboratory Testing

Results of field testing (e.g., SPT, pocket penetrometer, and vane testing) and laboratory testing (e.g., natural moisture content, and limits) executed on the recovered samples are plotted in this section.

Instrumentation Installation

Instrumentation installations (monitoring wells, piezometers, inclinometers, etc.) are plotted in this section. Water levels, if measured during fieldwork, are also plotted. These water levels may or may not be representative of the static groundwater level depending on the nature of soil stratum where the piezometer tips are located, the time elapsed from installation to reading and other applicable factors.

Comments

This column is used to describe non-standard situations or notes of interest.

AMEC Earth & Environmental
3096 Devon Drive, Suite 30
Windsor, ON N8X 4L2
Ph: (519) 969-7530
Fax: (519) 969-0160
www.amec.com

Rev. 6 Oct. '08

MODIFIED * UNIFIED CLASSIFICATION SYSTEM FOR SOILS

*The soil of each stratum is described using the Unified Soil Classification System (Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S Army. Vol. 1 March 1953.) modified slightly so that an inorganic clay of "medium plasticity" is recognized.

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4$; $C_C = \frac{(D_{30})^2}{D_{10} X D_{60}} = 1$ to 3
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (WITH SOME OR MORE FINES)	GM	SILTY GRAVELS, GRAVEL-SAND- SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6$; $C_C = \frac{(D_{30})^2}{D_{10} X D_{60}} = 1$ to 3
			SP	POORLY GRADED GRAVELS, GRAVEL- SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		$W_L < 50\%$	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		$30\% < W_L < 50\%$	CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		$W_L < 50\%$	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		$W_L < 50\%$	OH	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGH ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

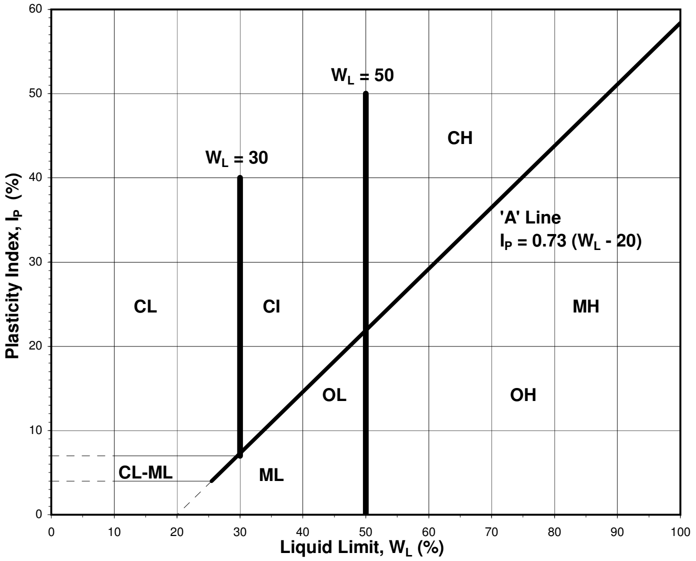
SOIL COMPONENTS

FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS		
		PASSING	RETAINED	PERCENT	DESCRIPTOR
GRAVEL	COARSE	76 mm	19 mm	35-50 20-35	AND Y/EY
	FINE	19 mm	4.75 mm	10-20 1-10	SOME TRACE
SAND	COARSE	4.75 mm	2.00 mm		
	MEDIUM	2.00 mm	425 µm		
	FINE	425 µm	75 µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 µm			

OVERSIZED MATERIAL

ROUNDED OR SUBROUNDED: COBBLES 76 mm TO 200 mm BOULDERS > 200 mm	NOT ROUNDED: ROCK FRAGMENTS > 76 mm ROCKS > 0.76 CUBIC METRE IN VOLUME

Plasticity Chart for Soil Passing 425 Micron Sieve



AMEC Earth & Environmental
3096 Devon Drive, Suite 30
Windsor, ON N8X 4L2
Ph: (519) 969-7530
Fax: (519) 969-0160
www.amec.com

Note 1: Soils are classified and described according to their engineering properties and behaviour.

Note 2: The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual (4th Edition, Canadian Geotechnical Society, 2006.)

Rev. 6 Oct. '08

APPENDIX C

LIMITATIONS OF REPORT

Limitations of Report

The conclusion and recommendations presented in this report are based on the information determined at the test hole locations. The information contained within this report in no way reflects the environmental aspects of the site and/or soils, unless specifically reported upon. Subsurface and groundwater conditions between and beyond these locations may differ from those encountered at the specific locations tested. Conditions may become apparent during construction, which were not detected and could not be anticipated at the time of the site investigation. It is, therefore, recommended practice that the Soil Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those conditions encountered in the test holes.

The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development etc.

The Design recommendations provided in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not have been available at the time this report was prepared, we recommend that we be retained during the final stage of design to verify that the design is consistent with our recommendations, and that the assumptions made in our analysis are valid.

The comments given in this report on potential construction problems and possible methods of construction are intended only for the guidance of the designer. The scope of work and the number of test holes may not be sufficient to determine all of the factors that may affect construction methods and costs (e.g. the thickness of surficial topsoil and fill layers can vary markedly and unpredictably). The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretations of the presented factual information and draw their own conclusions as to how the subsurface conditions may affect their work.