

REPORT

**FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED HIGH MAST LIGHT POLES
CARPOOL PARKING LOT AT QEW AND CASABLANCA BLVD.
REGIONAL MUNICIPALITY OF NIAGARA
AGREEMENT NO: 2007-E-0030
ASSIGNMENT NO: 4**

Submitted to:

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July, 2008

08-1181-0004 (4000b)

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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed high mast light poles in the area of the proposed carpool parking lot located at QEW and Casablanca Boulevard, in Grimsby, Ontario. This report addresses the foundation investigation carried out for the proposed high mast light poles, in Grimsby, Ontario.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Assignment No. # 4 for Agreement No. 2007-E-0030, issued on May 9, 2008.

2.0 SITE DESCRIPTION

The site is located along the south side of QEW, immediately southwest of the QEW interchange at Casablanca Blvd. The proposed carpool parking lot is bordered by the QEW W-S Ramp to the north, the South Service Road to the south, and Casablanca Blvd. to the east (Refer to Drawing 1 for site location).

The terrain of the proposed parking site is generally flat-lying and typically varies from about Elevation 87 m to Elevation 88 m. The present ground surface at the proposed high mast light pole on the west of the site (HML 1) and on the east of the site (HML 2) is about Elevation 87.5 m and Elevation 87.9 m, respectively. A wire fence separates the site from the adjacent road sections. At the time of the investigation, the ground surface was covered by sparse vegetation and a few small trees.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site on June 24, 2008, at which time one borehole (BH 1) was advanced between the two proposed high mast light poles, at the location shown on Drawing 2.

The field investigation was carried out using a truck-mounted CME-55 drill rig supplied and operated by GroundWorks Drilling LTD. The borehole was advanced to a depth of 6.1 m below the existing ground surface using 115 mm diameter continuous flight solid stem augers. Samples of the subsurface soils were obtained at depth intervals of 0.75 m to 1.5 m using a 50 mm outer diameter split-spoon sampler driven with an automatic hammer, as part of the Standard Penetration Test (SPT) procedure. The water level in the open borehole were observed throughout the drilling operation. The borehole was backfilled to the ground surface upon completion of the drilling operation using bentonite pellets, in accordance with Ontario Regulation 903 (as amended by O.Reg. 372).

The borehole was staked in the field by Golder at a location between the two high mast light poles. The corresponding ground surface elevation of the borehole was surveyed by Golder relative to Elevation 87.367 taken from the site plan¹ provided by MTO. The borehole location (MTM NAD83 northing and easting coordinates) and ground surface elevation (referenced to Geodetic datum) are detailed below and are presented on the Record of Borehole sheets.

<i>Borehole Number</i>	<i>Northing (m)</i>	<i>Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
BH-1	4784552.6	614014.88	87.06

The field work was supervised on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, arranged for the clearance of underground service locations, directed the drilling, sampling, and in situ testing operations, and logged the borehole. The soil samples were identified in the field, placed in labelled containers and transported to Golder's CCIL certified laboratory in Whitby for further examination and testing. Index and classification tests consisting of water content determinations, Atterberg limits and grain size distribution analyses were carried out on selected soil samples.

¹ MTO Site Plan, W.P. 2157-07-01, Sheet 3.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario*² and *Urban Geology of Canadian Cities*³.

The Iroquois Plain extends around the western shores of Lake Ontario; on the south side of the lake, the Plain is located between the present Lake Ontario shorebluffs and the foot of the Niagara Escarpment. The Plain is comprised of the flat to undulating lake bed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The overburden soils are underlain by red shale bedrock of the Queenston Formation⁴ (Geological Survey Map 1335A, Southern Ontario). This shale formation contains siltstone interlayers as well as "occasional patches of gypsum".⁵

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions as encountered in the borehole advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheet and on Figures 1 to 4 following the text of this report. The stratigraphic boundaries shown on the borehole record are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

² Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

³ J. Menzies and E.M. Taylor. "Urban Geology of St. Catharines-Niagara Falls, Region Niagara". In *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

⁴ Geological Survey Map 1335A, Southern Ontario.

⁵ J. Menzies and E.M. Taylor. "Urban Geology of St. Catharines-Niagara Falls, Region Niagara". In *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

In general, the surficial soils at the site consist of a thin layer of topsoil, underlain by a thin layer of clayey silt fill material. The fill material was in turn underlain by a clayey silt deposit underlain by red shale bedrock at shallow depth of about 1.9 m below the ground surface.

A more detailed description of the subsurface conditions encountered in the borehole is provided in the following sections.

4.2.1 Topsoil

Topsoil was encountered with a thickness of about 150 mm at the borehole location.

4.2.2 Fill Material

Underlying the topsoil, red to brown clayey silt fill material was encountered at Elevation 86.9 m, with a thickness of about 600 mm. In-situ water content measured on one fill sample was about 28 percent.

4.2.3 Clayey silt

Clayey silt, trace sand and gravel, was encountered in the borehole below the surficial clayey silt fill material. The surface of the deposit was encountered at a depth of 0.76 m below the ground surface corresponding to Elevation 86.3 m.

The results of grain size distribution testing completed on one sample of the clayey silt are shown on Figure 1. Although boulders and cobbles were not encountered within the clayey silt deposit in the borehole advanced as part of this investigation, the deposit is glacially derived and may contain cobbles and boulders.

Atterberg limits testing was completed on one sample of the clayey silt deposit, and measured a plastic limit of 19 per cent, liquid limit of 30 per cent, and a corresponding plasticity index of 11 per cent. These results, which are plotted on the plasticity chart on Figure 2, classify the deposit as a clayey silt of low plasticity. In-situ water content measured on two samples ranged from 10 per cent to 22 per cent.

The SPT "N" values measured within the clayey silt deposit ranged from 18 blows per 0.3 m of penetration to 50 blows for 0.1 m of penetration, indicating that the deposit has a very stiff to hard consistency.

4.2.4 Shale Bedrock

Shale bedrock of the Queen Formation was encountered in the borehole (Elevation 85.1 m) at a depth of 1.9 m below ground surface and it was confirmed by split-spoon sampling.

The results of grain size distribution testing completed on one sample of the Shale are shown on Figure 3. Atterberg limits testing was completed on one sample of the shale, and measured plastic limit of 18 per cent, liquid limit of 27 per cent, and plasticity index of 9 per cent. These results are plotted on a plasticity chart on Figure 4 and indicate that the shale fines consist of clayey silt of low plasticity.

Although not noted in the split spoon sample collected, interlayers of strong to very strong limestone and siltstone are anticipated to be present within the Queenston Formation shale bedrock.

4.3 Groundwater Conditions

The water levels in the borehole (BH 1) advanced by Golder in June 2008 were noted during and after the drilling operations. The groundwater level measured in the open portion of BH 1 after the completion of drilling was at a depth of 4.6 m below the ground surface, as presented in The Record of Borehole Log and as detailed below at borehole location. The recorded water level in the open portion of the borehole is detailed below:

<i>Borehole Number</i>	<i>Ground Surface Elevation (m)</i>	<i>Depth to Water (m)</i>	<i>Water Level Elevation (m)</i>	<i>Date</i>
BH 1	87.1	4.6	82.5	June 24, 2008

It should be noted that these observations reflect the shallow groundwater conditions encountered in the borehole at the time of the field investigation and some seasonal fluctuations should be anticipated.

5.0 CLOSURE

This Foundation Design Report was prepared by Ms. Xiaofei Wu, P.Eng., and Mr. Darrin Sellick. Mr. Jorge Costa, P.Eng., the Designated MTO Foundation Contact for Golder, carried out a technical and quality control review of the report.



GOLDER ASSOCIATES LTD.



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XW/DRS/JMAC:xw

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July, 2008

08-1181-0004 (4000b)

PART B

**FOUNDATION DESIGN REPORT
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Golder Associates

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides parameters and recommendations for the geotechnical aspects of design of the proposed HML pole foundations to be located in the area of the proposed carpool parking lot on the southwest side of the QEW and Casablanca Boulevard interchange. The design parameters and recommendations have been developed based on interpretation of the factual data obtained during a subsurface investigation at the site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess and to design the proposed High Mass Light foundations. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project, or for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect the equipment selection, proposed construction methods, scheduling and the like.

6.2 Design of High Mast Light Pole Foundations

Based on the subsurface conditions encountered at the location of Borehole 1, the HML pole foundations may be designed in accordance with MTO's *Procedures for the Design of High Mast Pole Foundations*, dated June 1994.

The unfactored passive lateral earth pressure, P_p (kPa), distributed along the caisson may be calculated using the following equations, based on the stratigraphy and design parameters given in the tables below:

$$P_p = K_p \gamma d + 2 c_u / K_p \quad \text{above the groundwater table, and}$$

$$P_p = K_p \gamma d_w + K_p \gamma' (d - d_w) + 2 c_u / K_p \quad \text{below the groundwater table,}$$

where

K_p is the passive earth pressure coefficient;
 γ is the bulk unit weight (kN/m³);
 γ' is the effective unit weight below the groundwater level (kN/m³);
 d is the depth below the ground surface (m);
 d_w is the depth to the groundwater level (m); and
 c_u is the undrained shear strength (kPa).

<i>HML Location</i>	<i>Reference Borehole</i>	<i>Design Groundwater Level</i>	<i>Elevation Interval</i>	<i>Soil Stratum</i>	<i>Design Parameters</i>				
					c_u	ϕ'	K_p	γ	γ'
HML 1 HML 2	BH 1	Elev. 82.5 m	Above Elev. 86 m	Fill, Clayey Silt	100	30	3.0	20	10
			Below Elev. 86 m	Shale Bedrock	-	38	4.2	22	12

In the design of the foundations, the passive resistance within the upper 1.2 m below ground surface should be neglected to account for frost action. The unfactored lateral resistance should be calculated assuming an equivalent pile width equal to three times the caisson diameter. A resistance factor of 0.5 should be applied to this calculated lateral resistance in order to obtain the factored lateral geotechnical resistance.

Where an undrained shear strength, c_u , is provided, the undrained capacity of the caisson should be checked to determine whether the drained or undrained case will govern. In this case, the lateral resistance for the length of the caisson within cohesive soil should be calculated assuming an internal angle of friction, $\phi' = 0$ degrees and an unfactored passive lateral pressure distribution equivalent to nine times the undrained shear strength acting over the actual width of the caisson. A resistance factor of 0.5 should be applied to this calculated lateral resistance in order to obtain the factored lateral geotechnical resistance.

6.3 Construction Considerations

It is recommended that a Non-Standard Special Provision (NSP) be included in the Contract Documents to warn the Contractor of the following items which are anticipated to affect the installation of the high mast light pole foundations at this site:

- **Presence of stronger interlayers of limestone/dolostone within shale bedrock:** Shale bedrock was encountered within the potential depth for caisson foundations in the vicinity of the proposed HML poles (based on results from Borehole BH 1). The shale bedrock in this area is known to contain stronger limestone and dolostone interlayers. Consideration of the presence of the stronger limestone and dolostone interlayers must be made in the selection of caisson installation equipment at these locations, if the foundation design for these two HML poles requires that the caissons be extended below the anticipated bedrock elevation.

A sample NSSP to address the above item is included in Appendix A.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Xiaofei Wu, P.Eng., and Mr. Darrin Sellick. Ms. Lisa Coyne, P.Eng. and Mr. Jorge Costa, P.Eng., the Designated MTO Foundation Contact for Golder, carried out an independent technical and quality control review.


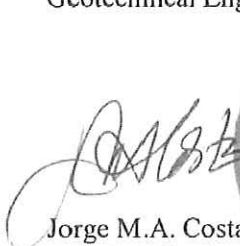
GOLDER ASSOCIATES LTD.



Xiaofei Wu, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Associate

A circular professional seal for a Licensed Professional Engineer in the Province of Ontario. The seal contains the name "J. M. A. COSTA" in the center.

Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Foundation Contact

XW/DRS/LCC/JMAC:xw

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CASABLANCA\REPORT\FOUNDATION REPORT - HML\04-1181-0004 (4000B) HML REPORT JULY 2008.DOC

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I	SAMPLE TYPE	III	SOIL DESCRIPTION		
AS	Auger sample	(a)	Cohesionless Soils		
BS	Block sample	Density Index (Relative Density)	N		
CS	Chunk sample		Blows/300 mm		
DO	Drive open		or Blows/ft.		
DS	Denison type sample		Very loose	0 to 4	
FS	Foil sample		Loose	4 to 10	
RC	Rock core		Compact	10 to 30	
SC	Soil core		Dense	30 to 50	
ST	Slotted tube		Very dense	over 50	
TO	Thin-walled, open	(b)	Cohesive Soils		
TP	Thin-walled, piston		Consistency	c _u , s _u	
WS	Wash sample		kPa	psf	
II	PENETRATION RESISTANCE		Very soft	0 to 12	0 to 250
Standard Penetration Resistance (SPT), N:			Soft	12 to 25	250 to 500
The number of blows by a 63.5 kg. (140 lb.)			Firm	25 to 50	500 to 1,000
hammer dropped 760 mm (30 in.) required			Stiff	50 to 100	1,000 to 2,000
to drive a 50 mm (2 in.) drive open			Very stiff	100 to 200	2,000 to 4,000
sampler for a distance of 300 mm (12 in.).			Hard	over 200	over 4,000
Dynamic Penetration Resistance; N _d :			IV. SOIL TESTS		
The number of blows by a 63.5 kg (140 lb.)			w	water content	
hammer dropped 760 mm (30 in.) to drive			w _p	plastic limit	
uncased a 50 mm (2 in.) diameter, 60° cone			w _l	liquid limit	
attached to "A" size drill rods for a distance			C	consolidation (oedometer) test	
of 300 mm (12 in.).			CHEM	chemical analysis (refer to text)	
PH:	Sampler advanced by hydraulic pressure	CID	consolidated isotropically drained triaxial test ¹		
PM:	Sampler advanced by manual pressure	CIU	consolidated isotropically undrained triaxial		
WH:	Sampler advanced by static weight of hammer		test with porewater pressure measurement ¹		
WR:	Sampler advanced by weight of sampler and	D _R	relative density (specific gravity, G _s)		
	rod	DS	direct shear test		
Piezo-Cone Penetration Test (CPT):			M	sieve analysis for particle size	
An electronic cone penetrometer with			MH	combined sieve and hydrometer (H) analysis	
a 60° conical tip and a projected end area			MPC	Modified Proctor compaction test	
of 10 cm ² pushed through ground			SPC	Standard Proctor compaction test	
at a penetration rate of 2 cm/s. Measure-			OC	organic content test	
ments of tip resistance (Q _t), porewater			SO ₄	concentration of water-soluble sulphates	
pressure (PWP) and friction along a			UC	unconfined compression test	
sleeve are recorded electronically			UU	unconsolidated undrained triaxial test	
at 25 mm penetration intervals.			V	field vane test (LV-laboratory vane test)	
			γ	unit weight	

Note:

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I GENERAL

π	= 3.1416
$\ln x$	natural logarithm of x
$\log_{10} x$ or $\log x$	logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
e	linear strain
e_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density \times acceleration due to gravity)	

(a) Index Properties (con't.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_i	sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength $= (\text{Compressive strength})/2$

PROJECT: 08-1181-0004 (4000b)

RECORD OF BOREHOLE BH-1

SHEET 1 OF 1

LOCATION: 4784552.6N 614014.88E

BORING DATE: June 24, 2008

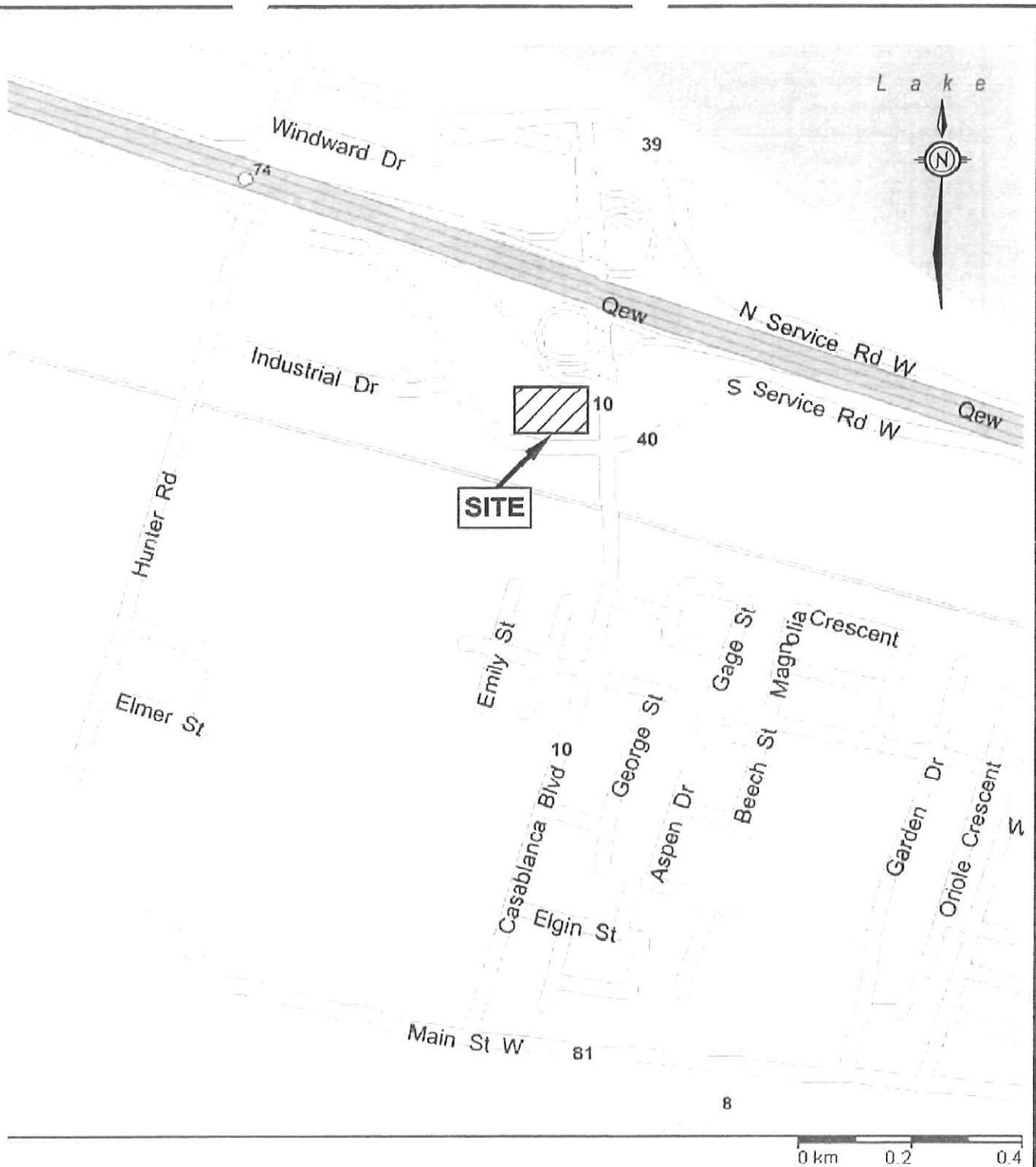
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SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER		TYPE	SHEAR STRENGTH				WATER CONTENT PERCENT					
								Cu, kPa				nat V. rem V. + 0.00 - 0.00					
								20 40 60 80				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³					
0		GROUND SURFACE		87.06													
		TOPSOIL		0.00													
		Red to brown clayey silt, trace sand, trace gravel, trace organics (FILL)		0.15	1	AS								27.3			
				86.30													
1		Very stiff to hard red to brown CLAYEY SILT, trace sand, trace gravel, trace rootlets		0.76	2	SO DO	18										
				86.30													
				86.30													
				86.30													
2		Red SHALE BEDROCK with intermittent limestone layers		1.91	3	SO DO	50/10										
				85.15													
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3				85.15													
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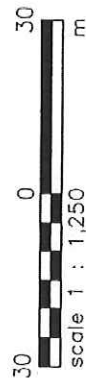
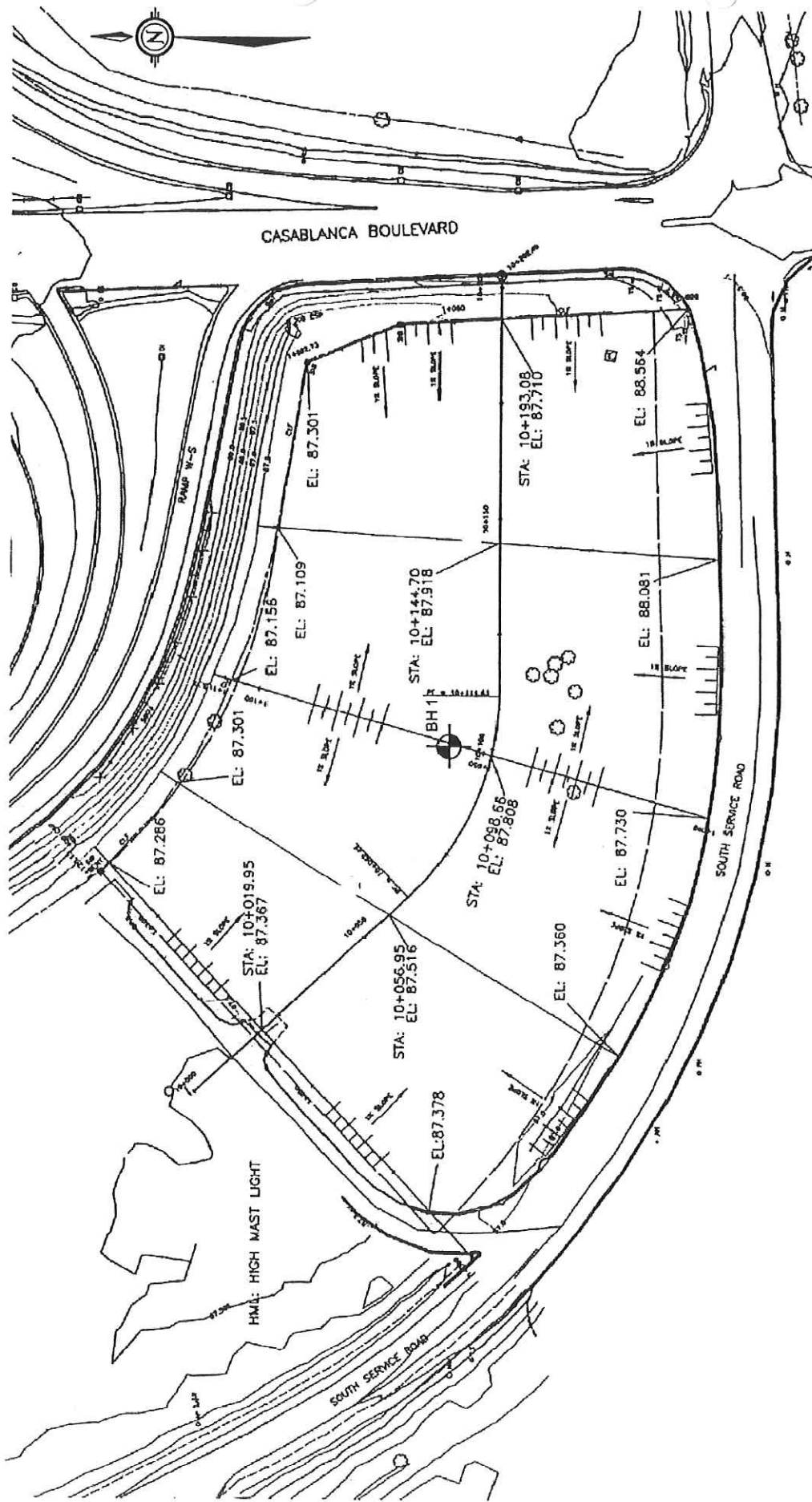
Drawing file: N:\CAD\2008\1181 engineering services\08-1181-0004\phase 4000b\08-1181-0004 (4000b)-002-fig 2008 07 Casablanca Car Pool Lot-tp FIG 1.dwg Jul 25, 2008 - 2:08pm



Base map by Microsoft Streets and Trips, 2008

ALL LOCATIONS ARE APPROXIMATE

PROJECT		Casablanca Car Pool Parking Lot Grimsby, Ontario	
TITLE		KEY PLAN	
 Golder Associates Whitby, Ontario	PROJECT No 08-1181-0004 (4000b)		FILE No 002
	DESIGN		SCALE AS SHOWN REV
	CADD	JT July 2008	
	CHECK	DEH Jy 2008	
	REVIEW		DRAWING 1



PROJECT

Casablanca Car Pool Parking Lot
Grimsby, Ontario

TITLE

LEGEND



BOREHOLE LOCATION IN PLAN

BOEHOLE LOCATION PLAN

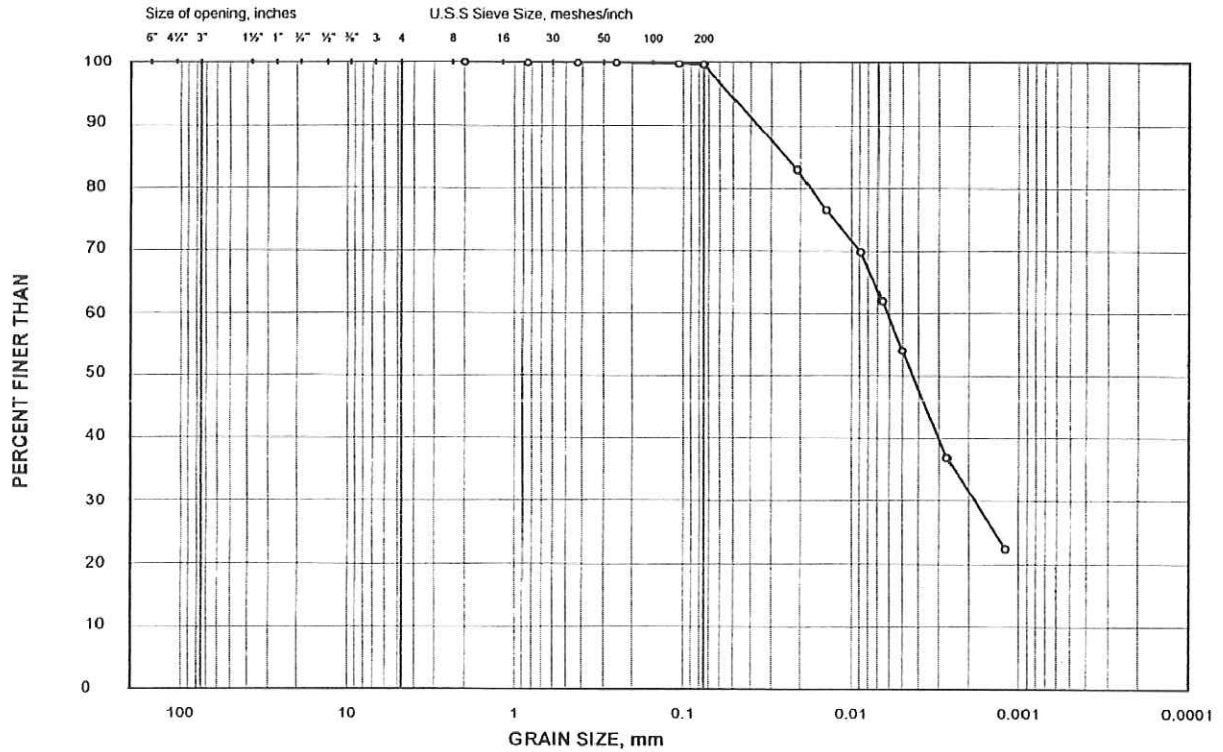
PROJECT No. 08-1181-0004(4000b)		FILE No.	001
DESIGN	SCALE	AS SHOWN	REV.
CADD	July 2008	JT	
CHECK	PRS		
REVIEW			



DRAWING 2

GRAIN SIZE DISTRIBUTION CLAYEY SILT

FIGURE 1



COBBLE SIZE	coarse	fine	coarse	medium	fine	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

LEGEND

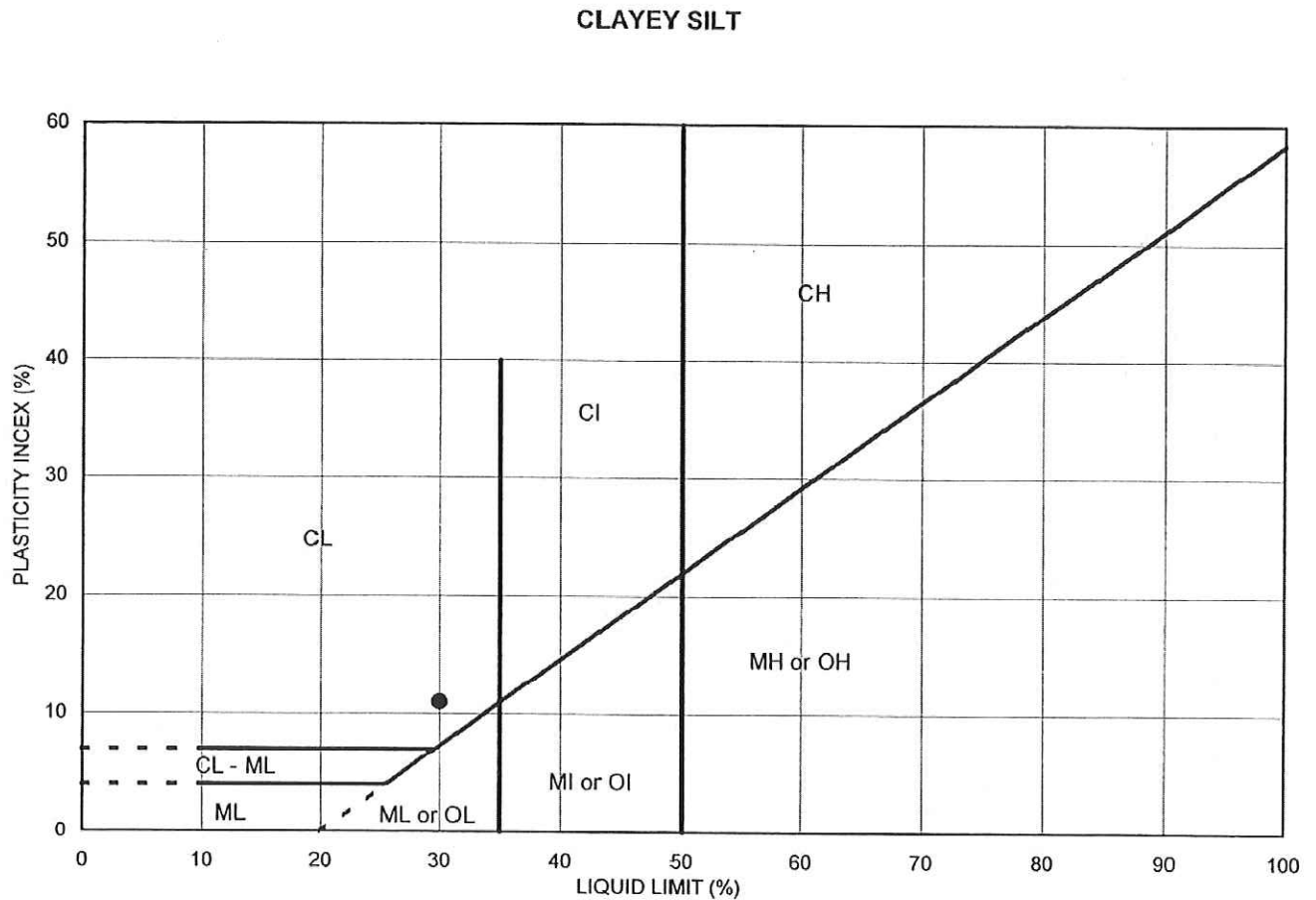
SYMBOL	BOREHOLE	SAMPLE #	ELEVATION (m)
o	1	3	Elev. 85.3

Date July ,2008
Project 08-1181-0004 (4000b)

Drawn XW
Checked DRS

PLASTICITY CHART (MTO)

FIGURE 2



LEGEND

SYMBOL	BOREHOLE #	SAMPLE #	ELEVATION (m)
•	1	3	Elev. 85.3

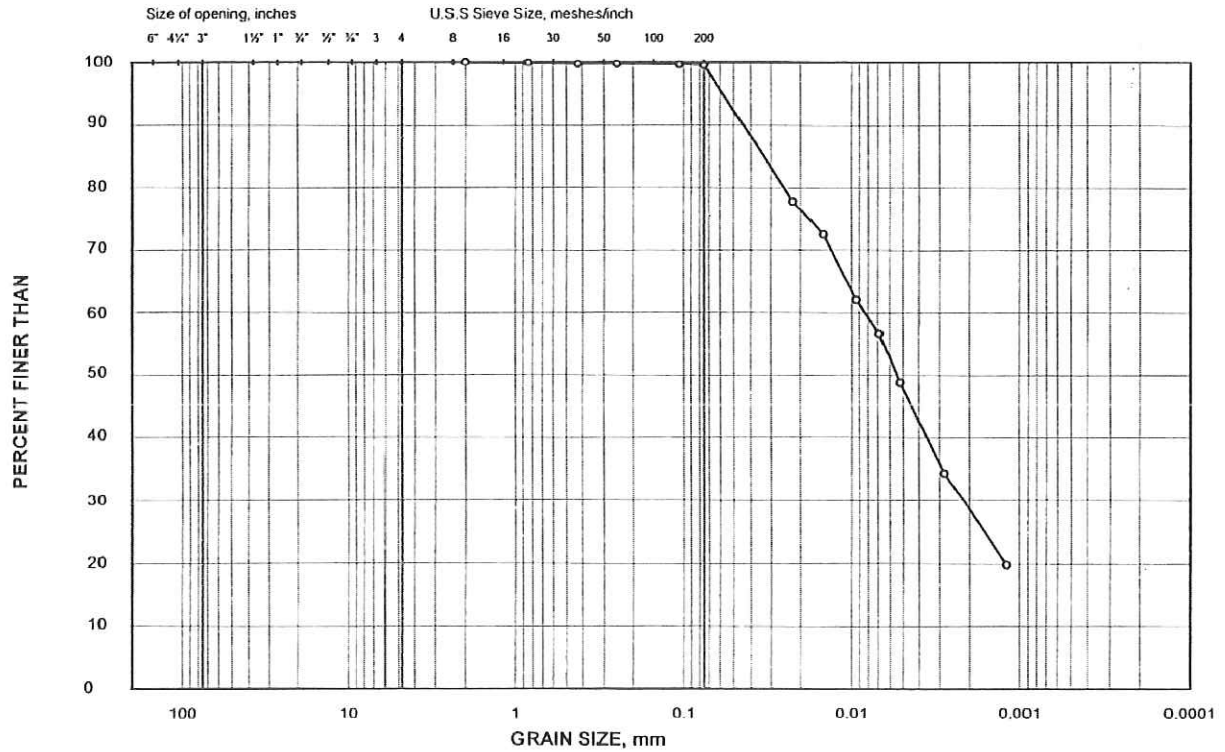
Date July, 2008
Project 08-1181-0004 (4000b)

Golder Associates

Drawn XW
Checked DRG

GRAIN SIZE DISTRIBUTION SHALE

FIGURE 3



COBBLE SIZE	coarse	fine	coarse	medium	fine	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

LEGEND

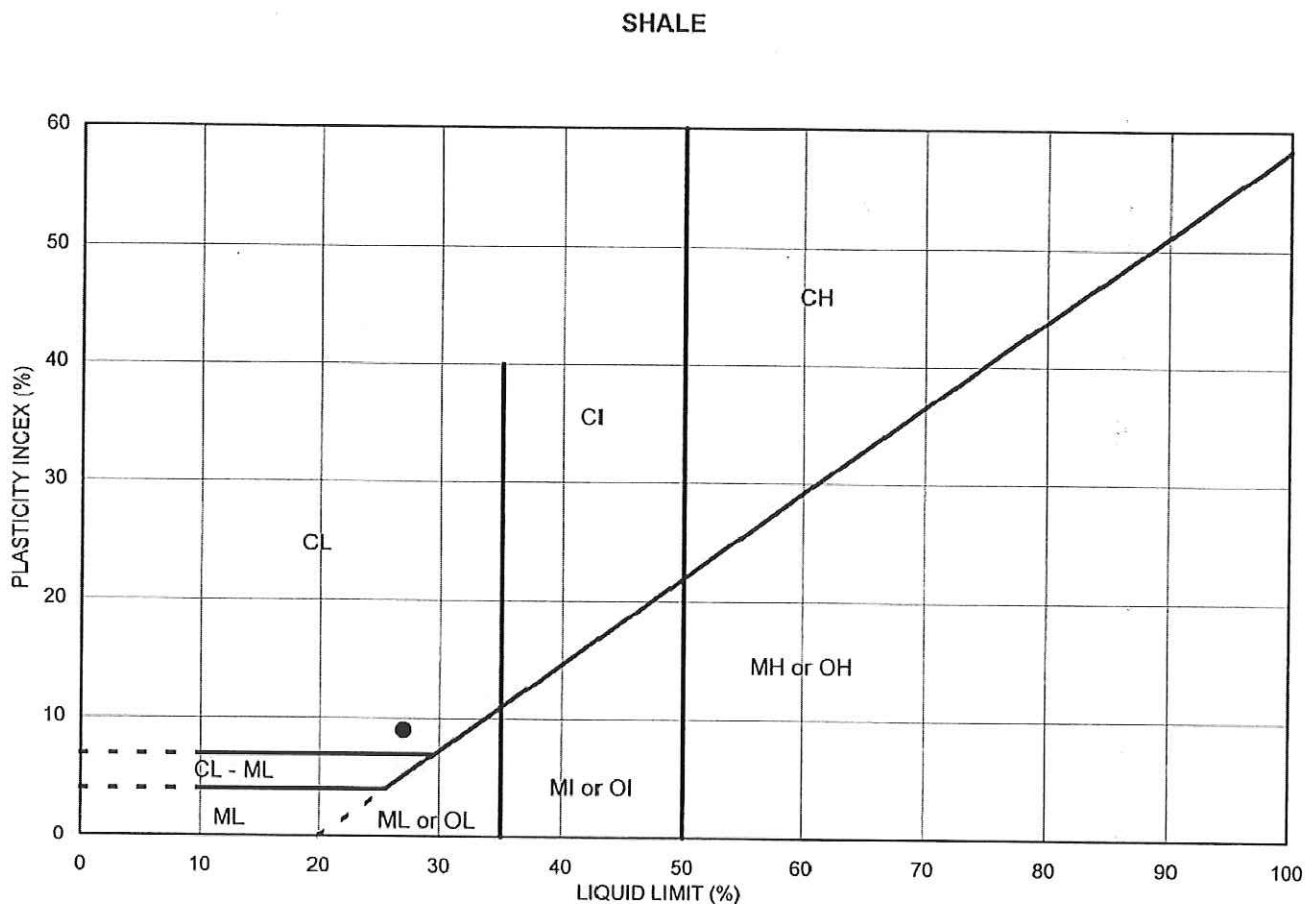
SYMBOL	BOREHOLE	SAMPLE #	ELEVATION (m)
°	1	6	Elev. 82.4

Date July ,2008
Project 08-1181-0004 (4000b)

Drawn XW
Checked DBS

PLASTICITY CHART (MTO)

FIGURE 4



LEGEND

SYMBOL	BOREHOLE #	SAMPLE #	ELEVATION (m)
•	1	6	Elev. 82.4

Date July, 2008
Project 08-1181-0004 (4000b)

Golder Associates

Drawn XW
Checked DPS

APPENDIX A
NON-STANDARD SPECIAL PROVISIONS

**PRESENCE OF STRONGER INTERLAYERS OF LIMESTONE/DOLOSTONE WITHIN
SHALE BEDROCK DURING HML FOUNDATION INSTALLATION - Item No.**

Special Provision

Shale bedrock was encountered within the caisson foundation depth for the HML poles. The shale bedrock in this area is known to contain stronger interlayers of limestone and dolostone. Consideration of the presence of the stronger interlayers must be made in the selection of caisson installation equipment at these locations.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION