



**FOUNDATION DESIGN REPORT
REHABILITATION OF THE MTO FERRY DOCK AT PELEE ISLAND
GWP 3029-05-00
DISTRICT 31, CHATHAM
for
TOTTEN SIMS HUBICKI ASSOCIATES**

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Figure 1 – Key Plan

Figure 2 – Typical Photograph

Figure 3 – Typical Section

Appendix A – Borehole Location Plan, Soil Strata Drawings and Record of Borehole Sheets

FOUNDATION DESIGN REPORT
for
Rehabilitation of the MTO Ferry Dock at Pelee Island
GWP 3029-05-00
District 31, Chatham

1. INTRODUCTION

Rehabilitation of the MTO mainland ferry dock at Pelee Island is planned. This report provides foundation engineering comments and recommendations for detailed design and construction of the work to be carried out to rehabilitate the dock.

The report was prepared for Totten Sims Hubicki Associates (TSH) on behalf of the Ministry of Transportation of Ontario (MTO).

The existing dock was constructed and commissioned in 1993 and consists of a sheet pile wall enclosure with a concrete pile cap and paved surface. The ferry dock is about 122 m long; the existing sheet pile wall is reported to be about 11 m deep.

Rehabilitation of the dock is required to address deficiencies related to:

- Over dredging and/or scour caused by the MV Jilmann ferry boat that uses the dock when transporting vehicles and passengers from the mainland to Pelee Island.
- The structural integrity of the sheet pile wall that was compromised by the loss of resistance in the passive zone of the sheet piles due to scour/over dredging.

Five options were considered for strengthening of the sheet pile wall during preliminary design (Preliminary Design Report prepared by Morrison Hershfield Limited dated April 2006; WP 3029-05-00):

Option	Description
1*	Toe pin with full height H piles
2	Install second level of tie backs
3	Toe pin with H piles cut off at Elevation 169.0
4	Place mass concrete/rockfill in the passive wedge below the dredge line
5	Install second sheet pile wall

* Recommended option

A concurrent study is in progress to evaluate options to address the ongoing settlement of the dock surface adjacent to the sheet pile wall. The results of this study will be issued under separate cover.

Pertinent design details documented in the Preliminary Design Report are noted in the following table:

	Elevation
200 year low water level	171.97
Design dredge level	169.00
Dredge level (November 2003)	167.25
Proposed dredge level	166.89
Base of sheet pile wall	164.50

2. SITE DESCRIPTION

The Pelee Island ferry dock is located near the intersection of West Dock Road and West Shore Road on the west side of Pelee Island, Ontario. Refer to Figure 1 for a location plan. The ferry dock complex is located on the west wharf and extends approximately 170 m out from shore. The dock has an asphalt paved surface and is enclosed on the south and west sides by steel sheet piling and a concrete cap. A typical photograph is provided on Figure 2.



3. DOCUMENT REVIEW

The subsurface stratigraphy along the alignment of the dock and related information reviewed during preparation of this report is itemized:

- Geocres 40G15-2 – Foundation Investigation and Design report for MTO Ferry Dock at Pelee Island, dated October 20, 2005; Work Order 01-33-001. This report contained information documented in reports dated November 1988 and January 1990 prepared for Public Works Canada.
- Preliminary Design Report – Rehabilitation of the Ferry Docks at Leamington, Kingsville and Pelee Island dated April 2006 (WP 3029-05-00).
- The RFP issued for rehabilitation of the MTO Ferry Docks at Leamington, Kingsville and Pelee Island, dated April 2006 (GWP 3029-05-00; PO 3005-E-0059).

4. SUBSURFACE CONDITIONS

A detailed description of the subsurface conditions at the ferry dock was provided in Geocres Nos. 40G15-2. The Borehole Location Plan, Record of Borehole Sheets and Soil Strata Drawings from these documents are provided in Appendix A.

The field investigation conducted during this study was in conformance with Section 6.8.2.1 of the Terms of Reference issued for the project. We consider the existing data to be sufficient to provide foundation engineering recommendations for strengthening of the dock.

Pertinent details are noted in the following table:

Geocres	Borehole No.	Native Soil Depth to/Elevation (m)	Bedrock Depth to/Elevation (m)	Length of Rock Core (m)	Total Depth/Elevation (m)
40G15-2	101 ¹	7.77 168.23	17.19 158.81	2.84	20.03 155.97
40G15-2	102 ¹	7.92 168.08	16.92 159.08	3.53	20.45 155.55
40G15-2	103 ¹	7.07 168.93	16.70 159.30	3.48	20.18 155.82
November 1988 Report	22 ²	5.18 168.73	15.18 158.73 ³	NC ⁴	15.24 158.67



Geocres	Borehole No.	Native Soil Depth to/Elevation (m)	Bedrock Depth to/Elevation (m)	Length of Rock Core (m)	Total Depth/Elevation (m)
January 1990 Report	301 ²	5.03 169.25	14.94 159.34	3.13	18.07 156.21
January 1990 Report	302 ²	5.49 168.79	15.88 158.40 ³	NC ⁴	16.46 157.82
January 1990 Report	303 ²	4.02 170.26	NE ⁴	NC ⁴	9.75 164.53

1. Boreholes drilled over the edge of the dock; depth to native soil is from the dock surface.
2. Borehole drilled from a barge.
3. Bedrock depth assumed.
4. NC – Not cored. NE – Not encountered.

5. ENGINEERING COMMENTS AND RECOMMENDATIONS

Pertinent information concerning the foundation design considerations for rehabilitation of the dock noted in the Preliminary Design Report and the Foundation Investigation and Design Report (Geocres 40G15-2) is summarized below:

	Elevation
<u>Sheet Piles</u>	
Top	176.00
Tip	164.50
<u>Dredge Level</u>	
Design	169.00
Actual (November 2003)	167.25
Proposed Dredge Level	166.89
Bedrock Elevation	158.8 to 159.3

A representative section of the dock wall along with a typical stratigraphic profile and recommended geotechnical parameters for design of measures to strengthen the wall, deduced from data provided in Geocres 40G15-2 are provided in Figure 3.

A brief description of the five options considered for strengthening of the dock wall during preliminary design are provided in Section 1. Installation of full height steel H piles in predrilled auger holes filled with concrete was the method recommended for rehabilitation of the dock.

The structural analysis conducted during detailed design of measures to rehabilitate the ferry dock must consider the lateral pressure imposed on the sheet pile wall by the soil and water (including short term variations due to ship traffic, natural variations in the lake water level, (storm events, seasonal and long term cycles), as well as the applied horizontal and vertical surcharge loads.

The lateral active and passive earth pressures (unfactored) imposed on the dock wall should be computed using the following equations:

Active Pressure

$$p_a = K_a (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_s$$

where K_a = active earth pressure coefficient (dimensionless)

γ = unit weight of retained soil
above the design water level (kN/m^3)

γ' = buoyant unit weight of soil
below design water level ($\gamma - \gamma_w$)

h_1 = depth below top of pier (m), above
design water level

h_2 = depth below design water level (m)

q = surcharge load (kN/m^2)

γ_w = unit weight of water
= 9.8 kN/m^3

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where ϕ = angle of internal friction of retained soil (37.5° for rock/sand fill)

δ = angle of friction between the soil and wall (27° for rock/sand fill)

Passive Pressure

$$p_p = \gamma_w h_w + K_p (\gamma' h_s + q) + C_s$$

where K_p = passive earth pressure coefficient

h_w = depth below design water level (m)

h_s = depth below design dredge level (m)

γ_w , γ' , q , and C_s were defined previously.

The factored passive resistance at ULS is $0.5 p$, the active pressure is unfactored (clause 6.9.1 of the Commentary to the Code).

The seismic coefficient for the conditions at this site is 1.5 (Type III soil profile as per clause 4.4.6 of the CHBDC, CAN/CSA-S6-00). The zonal acceleration ratio is 0.00.

The ferry dock is located in Seismic Performance Zone 1. The liquefaction potential of the silts and sands at the site was assessed using the procedure suggested by Seed and Idriss (1971) and, on this basis, it is considered that liquefaction of these soils is unlikely (clause 4.6.2 of the CHBDC).

The lateral earth pressures (active and passive) that act on a sheet pile wall are a function of the composition and engineering properties of the soil adjacent to the wall, the magnitude of friction developed between the soil and the sheet pile wall, the deflection of the wall and the flexibility of the sheeting.

Since the magnitude of deflection of the sheet pile wall is unknown and both the active and passive earth pressure coefficients are a function of the magnitude of wall movement, it is normal practice during design of sheet pile walls to ignore the friction that may develop between the soil and the steel sheeting. However, during rehabilitation of an existing sheet pile wall, an assessment of the 'mobilized' earth pressure coefficients (MEPC) is required to optimize the design for strengthening of the wall.

The MEPC should be evaluated by 'back analysis' of the existing sheet pile wall. We understand from review of the Preliminary Design Report that evidence of distress and/or significant movement of the existing wall was not detected.

A factor of safety of 1.0 is considered to be the value indicative of imminent failure. The role of the factor of safety in geotechnical analysis is to control movement; a factor of safety against failure of at least 1.5 is employed for studies of this type. Significant movement of the structure is normally evident in situations where the computed factor of safety is 1.1 to 1.2 and not discernible if it is in the range of 1.3 to 1.4.

Cognizant of the apparent satisfactory performance of the wall (measured deflections reported to be about 5 mm over a length of 1.2 m just above the dredge line), the factor of safety of the existing sheet pile wall against failure is considered to be at least 1.2. It is recommended therefore, that the MEPC be assessed by:

- i) Evaluate the computed factor of safety of the existing wall against failure using equations (1) and (2) and the geotechnical parameters noted in the following table. The friction developed between the sheeting and the soil should be ignored at this stage. It is imperative that the actual (unfactored) load and resistance is employed during this analysis. If the computed factor of safety is significantly less than 1.2, go to step ii).

Elevation		Total Unit Weight (γ kN/m ³)	Effective Cohesion (c' kPa)	Effective Friction Angle (ϕ')	Active Earth ² Pressure Coefficient (K_a)	Passive Earth ² Pressure Coefficient (K_p)
176.0	Top of Dock					
176.0 to 170.0	Rock fill and sand ¹	21.0	0	37.5	0.24	N/A
170.0 to 166.0	Silty clay	17.0	3 ³	28 ³	0.36	2.8
166.0 to 163.0	Clayey silt till	19.0	0	34	0.28	3.8
163.0 to 159.0	Sandy silt till	22.0	0	40	0.22	4.6
159.0	Top of bedrock					

1. Assumed – no data available from PDR based on information provided on the construction drawings.
 2. Wall friction ignored.
 3. These parameters are based on effective stress design. The silty clay is slightly over consolidated and will be permanently submerged. Hence, use of total stress parameters is considered to be suitable for the silty clay – $c_u = 40$ kPa $\phi' = 0$.
- N/A Not applicable.



- ii) Repeat the analysis in Step i) using the geotechnical parameters noted in the first column of the following Table for both the active and passive earth pressure coefficient. If the computed factor of safety is significantly less than 1.2, go to Step iii).

Elevation		Active Earth Pressure Coefficient ² (K _a)						Passive Earth Pressure Coefficient ² (K _p)					
176.0	Top of Dock												
176.0 to 170.0	Rock fill and sand ¹	0.23	0.23	0.22	0.22	0.21	0.21	N/A	N/A	N/A	N/A	N/A	N/A
170.0 to 166.0	Silty clay	0.35	0.34	0.33	0.32	0.32	0.31	3.2	3.6	4.0	4.5	4.7	5.1
166.0 to 163.0	Clayey silt till	0.27	0.26	0.25	0.24	0.24	0.23	4.4	5.2	6.0	6.75	7.5	8.2
163.0 to 159.0	Sandy silt till	0.22	0.21	0.20	0.19	0.19	0.18	6.0	7.7	9.7	11.5	12.4	13.8
159.0	Top of bedrock												

1. Assumed – no data available from PDR. To be confirmed.
 2. Wall friction included in increments to enable parametric analysis to assess realistic values for the actual mobilized wall friction – first column 15%, second column 30%, third column 45%, fourth column 60%, fifth column 75%, sixth column 90%.
- N/A Not applicable.

- iii) Repeat step ii) using the earth pressure coefficients in each successive column until the computed factor of safety against failure is at least 1.2.

The earth pressure coefficients employed to achieve a factor of safety of 1.2 are considered to be the MEPC.

The remedial work suggested in the Preliminary Design Report to minimize settlement of the dock surface in future calls for excavation of an approximate 3 m wide 2 m deep area behind the sheeting (PML Ref.: 06HF057, Draft Report dated November 23, 2006). Use of light weight fill as a means of reducing the active earth pressure imposed on the wall was considered. The available products are limited however due to the properties of the products normally employed – environmental characteristics of steel slag; buoyance of EPS.

Resistance to the active earth pressure imposed on the sheet pile wall will be provided by the passive earth pressure acting on the face of the embedded depth of the sheet pile wall below the

dredge level, the passive earth pressure developed on the concrete surrounding the H piles between the base of the sheeting and the bedrock surface (or toe of the H pile, whichever is less) and the 'horizontal bearing resistance' of the rock (if the H pile is socketted into bedrock) and/or soil if applicable (see subsequent paragraph). The passive resistance developed on the embedded portion of the sheeting should be computed using equation (2) and the MEPC. The passive resistance developed on the portion of the concrete surrounding the pile below the sheeting should be computed using equation (2) and the parameters provided in step I) subject to the limitations provided in the following paragraph.

The passive earth pressure will be developed over a width equivalent to three diameters and a height of six diameters of the concrete surrounding the H piles, provided the centre to centre spacing between the H piles is greater than five times the diameter of the concrete (transverse to the applied load). For this project, the spacing between piles will be less than five diameters and/or the space between the toe of the sheeting and the top of the bedrock may be less than six diameters; the computed passive resistance should be reduced to account for this situation in the following manner:

- i) Spacing less than 3 diameters:
 - Compute resistance based on a continuous caisson wall
- ii) Spacing less than 5 diameters and greater than 3 diameters, thickness of soil between the bedrock surface and the base of the sheeting is greater than 6 diameters:
 - Compute resistance for pile spaced at 5 diameters or greater (R_1)
 - Compute resistance for continuous caisson wall (R_2)
 - Reduce resistance in direct linear proportion between R_1 and R_2 for pile spacing between 3 diameters and 5 diameters. For example, if spacing is 4 diameters, resistance will be $(R_1 + R_2)/2$
- iii) Thickness of soil between the bedrock surface and the base of the sheeting is less than 6 diameters
 - Reduce computed resistance in direct linear proportion to the actual thickness of soil between the sheeting and bedrock surface.

The bedrock is fair to excellent quality (RQD typically greater than 50), thinly to medium bedded, medium strong to strong, dolomitic limestone of the Dundee Formation. The factored horizontal resistance of the bedrock at ULS is considered to be 3000 kPa. The resistance at SLS will be much higher and hence the factored resistance at ULS will govern.

If the space between the base of the sheeting and the top of the bedrock exceeds six times the diameter of the pile, the computed lateral resistance could be increased by consideration of the horizontal bearing resistance of the soil between the bedrock surface and a distance equivalent to 6 diameters below the base of the sheeting. The factored horizontal bearing resistance of the strata that overlies the bedrock is considered to be (refer to Figure 3 for depths):

Clayey silt till 200 kPa

Sandy silt till 300 kPa

The following equation should be employed to evaluate the coefficient of horizontal subgrade reaction along the pile; modulus values to model the response of the soil in front of the sheeting (passive zone) by 'springs' are also provided:

Granular

$$k_s = n_h z/b$$

where k_s = coefficient of horizontal subgrade reaction kN/m³

n_h = coefficient related to soil density
 Refer to the following table for details

z = depth, m

b = pile width, m

Cohesive

$$k_s = \frac{67 c_u}{b}$$

where k_s = coefficient of horizontal subgrade reaction kN/m³

c_u = undrained shear strength of the clay
 Refer to the following table for details

b = pile width, m

Elevation		Soil Density ¹ Coefficient (n_h - kN/m ³)	Undrained Shear Strength (c_u - kPa)	Soil Modulus (k_s - mN/m ³)
176.0	Top of Dock			
176.0 to 170.0	Rock fill and sand	4,000 (6,000)	--	15
170.0 to 166.0	Silty clay	--	70	10
166.0 to 163.0	Clayey silt till	--	250	35
163.0 to 159.0	Sandy silt till	10,000	--	80
159.0	Top of bedrock			

1. Values shown are applicable to 'below water' situations which is the case for the conditions at this site except for the upper portion of the rockfill that is above the lake level; the soil density coefficient for the portion of the rockfill above the lake level is shown in parenthesis.

Use of a steel liner to advance the auger hole to the bedrock surface will be required. It is noteworthy that cobbles and boulders were identified in the hard till overlying the bedrock. Some difficulty drilling through these materials should be expected. Placement of concrete by the Tremie method will probably be necessary. The liner should be removed as the concrete is placed to ensure a void does not exist between the caisson and the surrounding soil. It is imperative that the top of the concrete within the line is maintained at least 1 m above the lake level as the liner is withdrawn to prevent 'collapse' of the sidewalls of the auger hole due to a differential hydraulic gradient.

Bedrock

$$k_s = n_h z/b$$

where n_h = coefficient related to rock quality, kN/m³ = 30,000 kN/m³

z = depth below bedrock surface, m

b = caisson diameter, m

5.1 Deep Seated Failure Consideration

Since the sheeting extends into the compact to dense sandy/clayey silt till, and the soldier piles will extend into the hard clay till or bedrock, it is considered that deep seated failure of the sheet pile wall is not a concern.

6. CLOSURE

The report was prepared by Mr. Dennis W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Sincerely

Peto MacCallum Ltd.



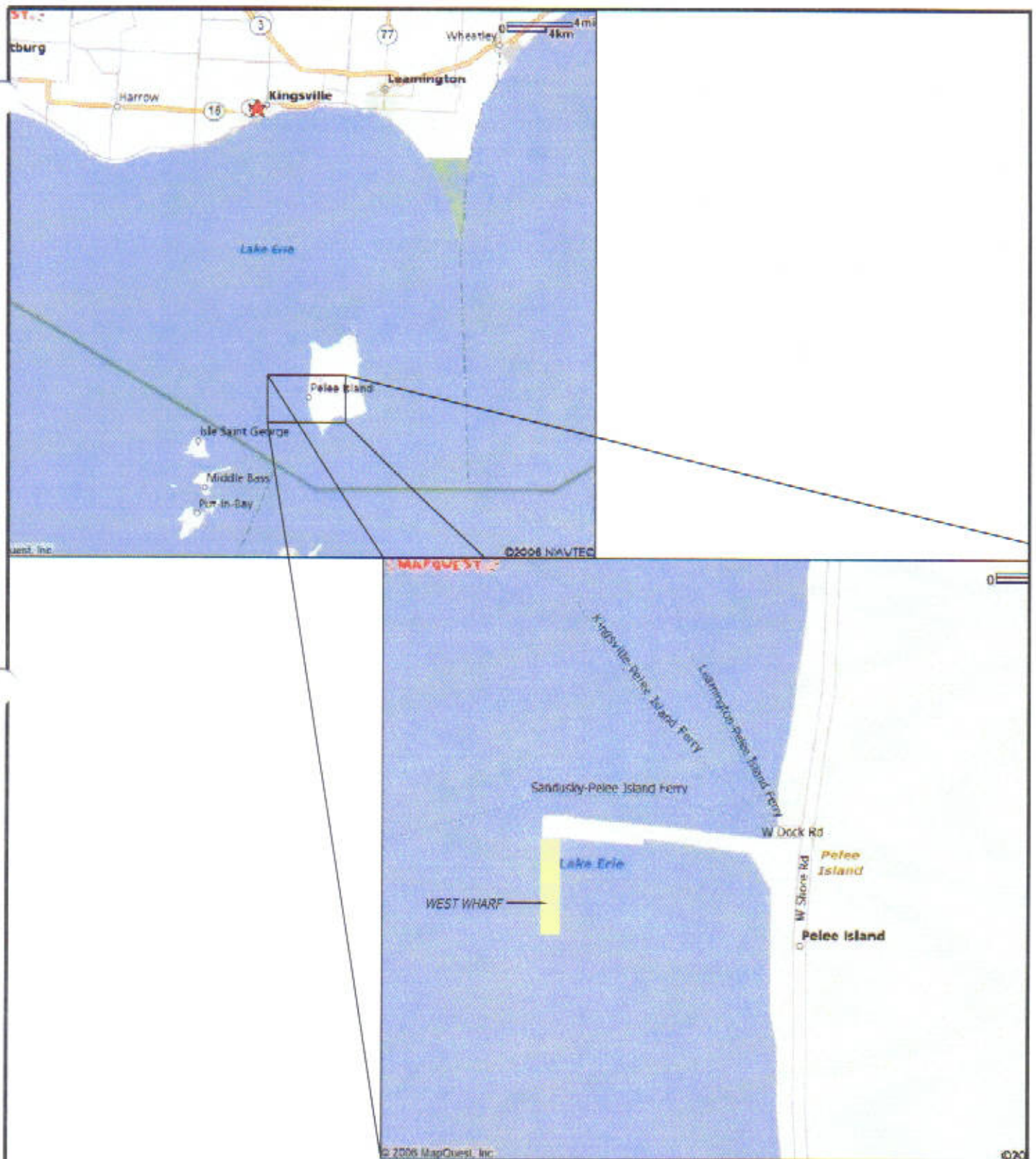
Dennis W. Kerr, MEng, P.Eng
Chief Foundation Engineer



Brian R. Gray, MEng, P.Eng.
MTO Designated Contact



DWK:lad



TOTTEN SIMS HUBICKI ASSOCIATES

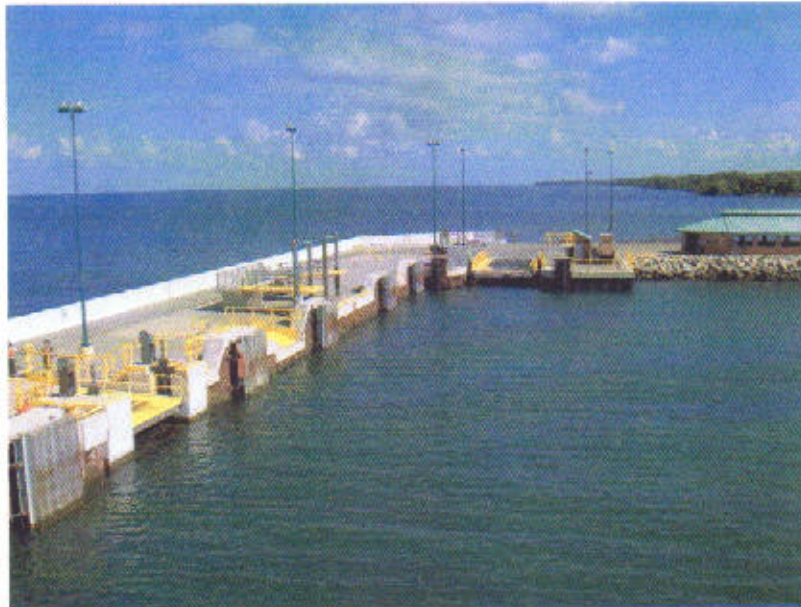
KEY PLAN

MTO FERRY DOCK, PEELE ISLAND, ONTARIO
GWP 3029-05-00

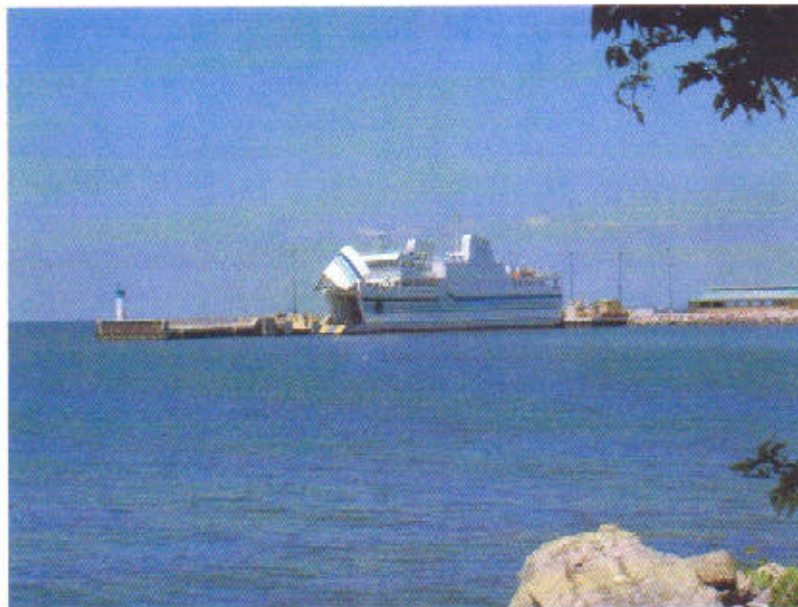


Peto MacCallum Ltd.
 CONSULTING ENGINEERS

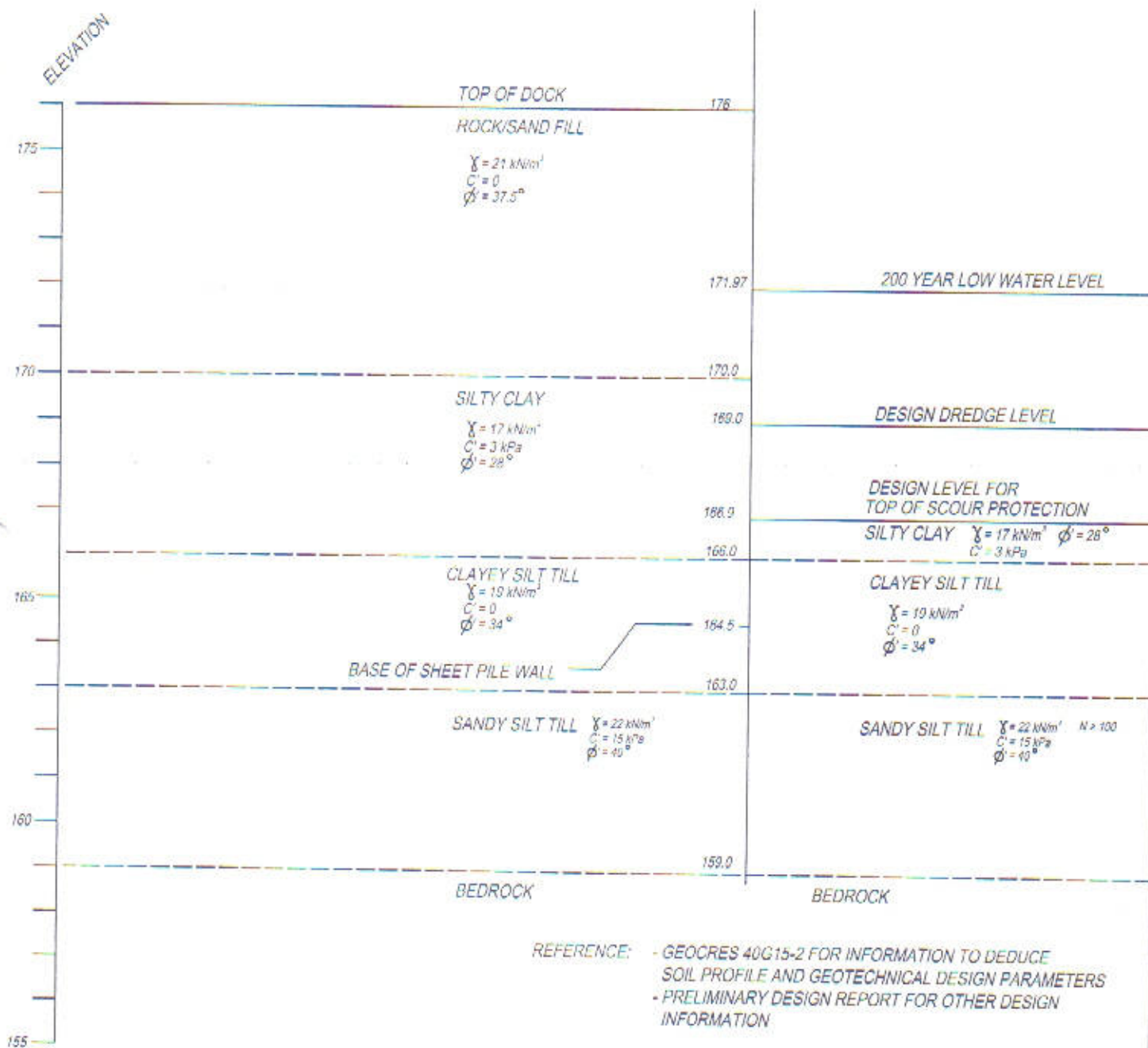
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CHECKED	DWK	OCT 2006	NTS	06HF051A	1
APPROVED	DWK				



Photograph 1 – View northwest of Pelee Island West Wharf.
(August 18, 2006)



Photograph 2 – View west of Pelee Island West Wharf.
(August 18, 2006)



TOTTEN SIMS HUBICKI ASSOCIATES

TYPICAL SECTION

MTO FERRY DOCK, PELEE ISLAND, ONTARIO



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 CONSULTING ENGINEERS

DRAWN	MS	DATE	SCALE	JOB NO.	FIGURE NO.
CHECKED	DWK	OCT 2006	AS SHOWN	06HF051A	3
APPROVED	DWK				



Appendix A

Borehole Location Plan, Soil Strata Drawings

and Record of Borehole Sheets



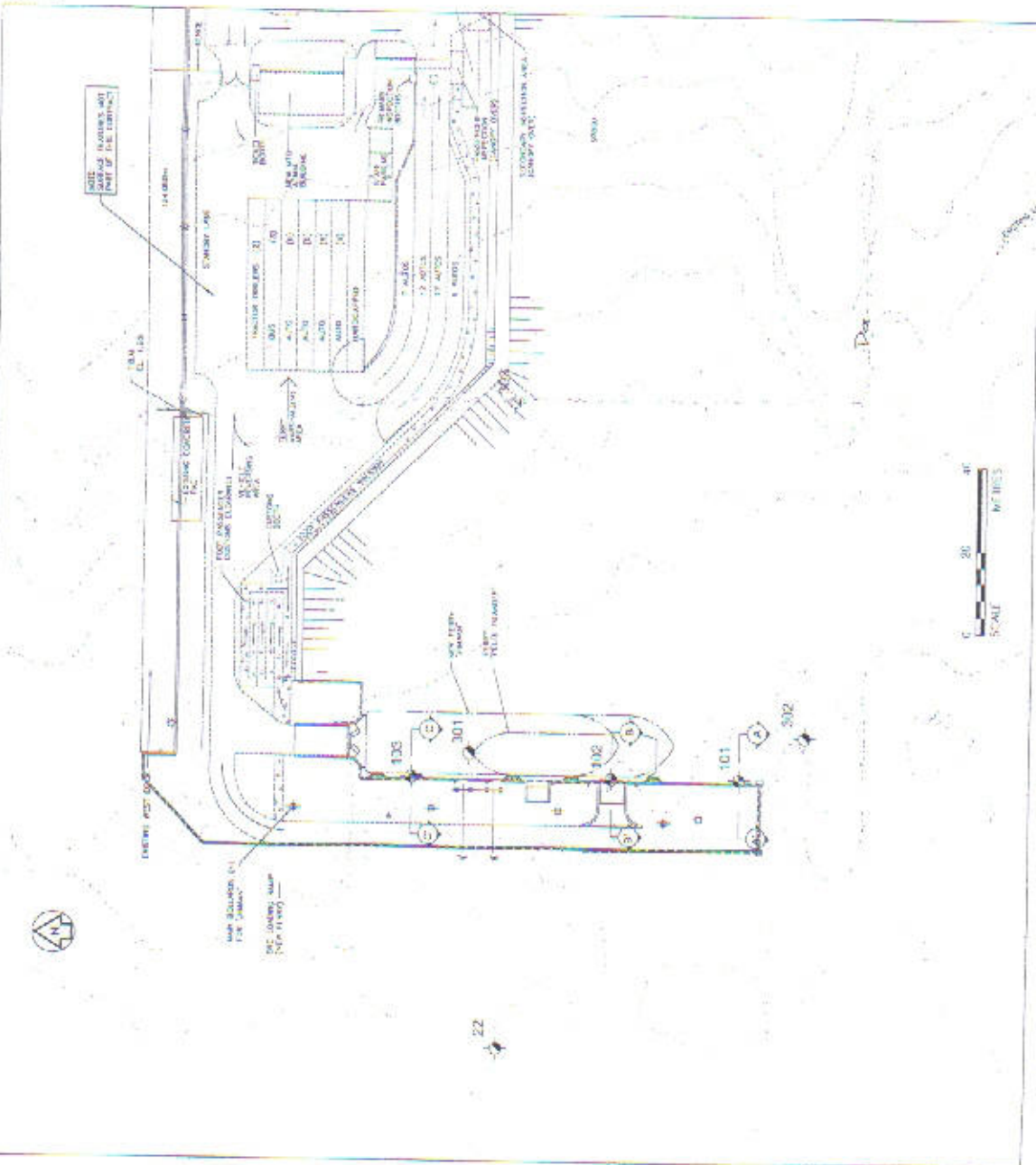
LEGEND
Boathouse Location - Indicated by
Symbol - Indicated by Symbol

NO.	ELEVATION (MGS. 1985)	20' OFFSHORE (MGS. 1985)	20' OFFSHORE (MGS. 1985)
101	171.00	N/A	N/A
102	174.00	N/A	N/A
103	178.00	N/A	N/A
104	174.00	N/A	N/A
105	174.00	N/A	N/A
106	174.00	N/A	N/A
107	174.00	N/A	N/A
108	174.00	N/A	N/A
109	174.00	N/A	N/A
110	174.00	N/A	N/A

NOTES:
1. The boathouse location and elevation data were obtained from a survey conducted in 2011.
2. The boathouse location and elevation data were obtained from a survey conducted in 2011.
3. The boathouse location and elevation data were obtained from a survey conducted in 2011.

NO.	DATE	BY	REVISION
1	12/15/2011	Golden Associates Ltd.	Initial Design

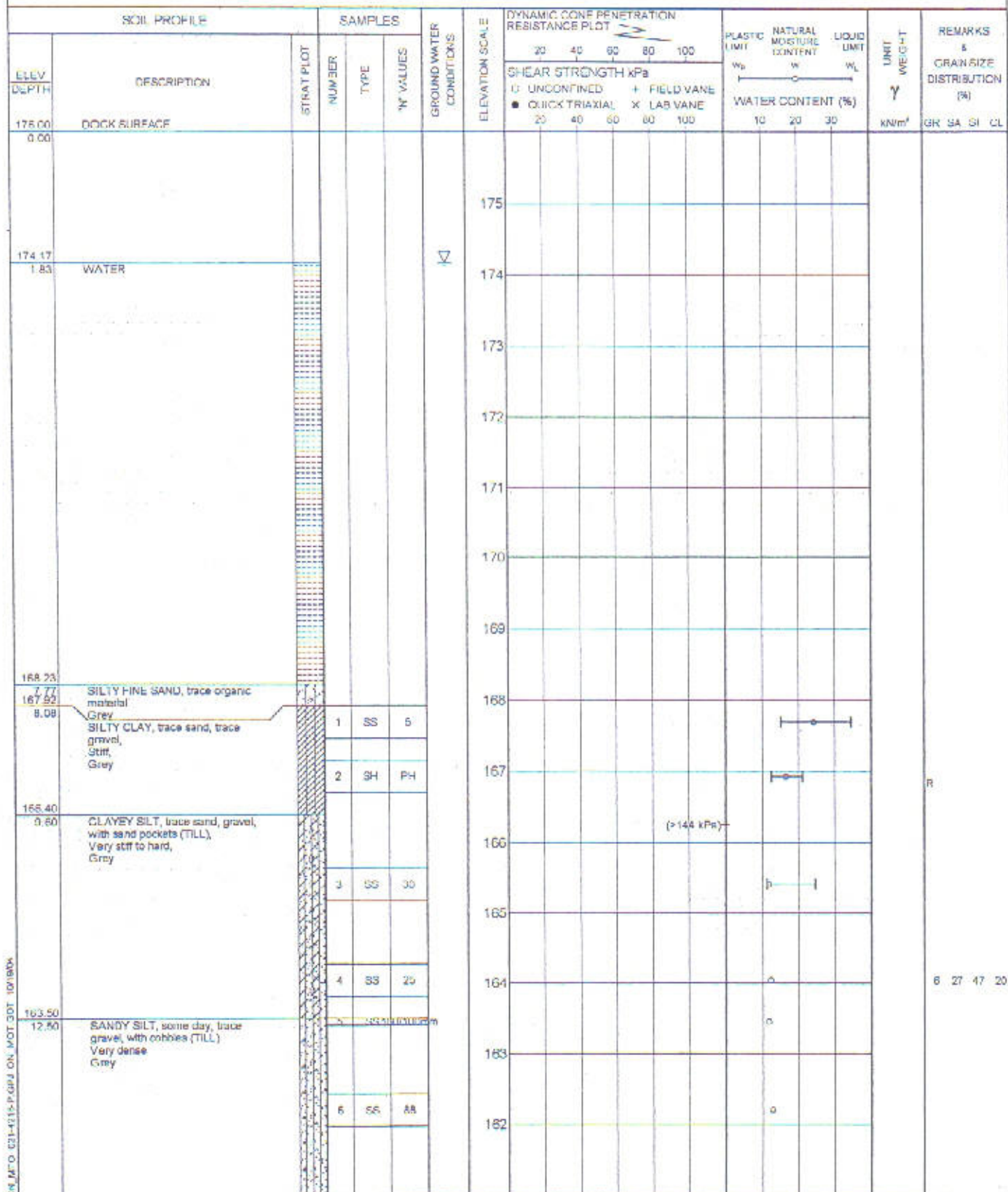
NO.	DATE	BY	REVISION
1	12/15/2011	Golden Associates Ltd.	Initial Design



NOTES:
1. The boathouse location and elevation data were obtained from a survey conducted in 2011.
2. The boathouse location and elevation data were obtained from a survey conducted in 2011.
3. The boathouse location and elevation data were obtained from a survey conducted in 2011.



PROJECT 021-4216-1-3 RECORD OF BOREHOLE No 101 1 OF 2 METRIC
G.W.P. WP LOCATION REFER TO BOREHOLE LOCATIONS - DRAWING 1 ORIGINATED BY D.W.
DIST HWY N/A BOREHOLE TYPE ROTARY DRILLING (HW CASING), TRI-CONE COMPILED BY WDF
DATUM I.G.L.D. DATE September 13, 2004 - September 14, 2004 CHECKED BY AMH



CM, MFO 021-4216-P.GRD ON POT 30T 10/19/04

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 101

2 OF 2

METRIC

PROJECT 021-4216-1-3

C.W.P. WP

LOCATION REFER TO BOREHOLE LOCATIONS - DRAWING 1

ORIGINATED BY D.W.

DIST HWY N/A

BOREHOLE TYPE ROTARY DRILLING (NW CASING), TRI-CONE

COMPILED BY WDF

DATUM I.G.L.D.

DATE September 13, 2004 - September 14, 2004

CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100		10 20 30				
168.81			7	SS	100									
			8	SS	144									
17.19														
	Fresh, microcrystalline to fine grained, thin to medium bedded, light grey to brown, faintly to moderately porous, medium strong to strong, argillaceous DOLOMITIC LIMESTONE, occasionally fossiliferous and interclastic with pitted to vuggy zones and calcite nodules (DUNDEE FORMATION)		9	SS 110/200mm										
			10	CORE	-									
			11	CORE	-									
155.97														
20.03	END OF BOREHOLE													

ON MTD 021-4216 P.3-2 CH. MOT. OUT 12/19/04

METRIC

PROJECT 021-4215-1-3

GW F WP

LOCATION

REFER TO BOREHOLE LOCATIONS - DRAWING 1

ORIGINATED BY DJM

DIST HWY NA

BOREHOLE TYPE ROTARY DRILLING (NW CASING), TRI- CONE

COMPILED BY WWT

DATUM 1 GLE

DATE _____

September 14, 2004 - September 14, 2004

CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT. PLCT	NUMBER	TYPE	"N" VALUES			20	40					
178.00 0.00	DOCK SURFACE													
174.17 1.83	WATER													
188.08 7.92	SILTY CLAY, trace sand, trace gravel, Silt Grey		1	SS	5									
166.86 9.45	CLAYEY SILT, trace sand, gravel, with cobbles (TILL), Very stiff to hard, Grey		2	SH	PH									
			3	SH	PH									
			4	SS	38									
			5	SS	29									
			6	SS	34									
163.05 12.05	SANDY Silt, trace to some clay, trace gravel, with cobbles (TILL) Very dense Grey		7	SS 105/200mm										
			8	SS	110									

ON_MTO 021-4210-F GP: ON_MCT 3DT 1019004

Continued Next Page:

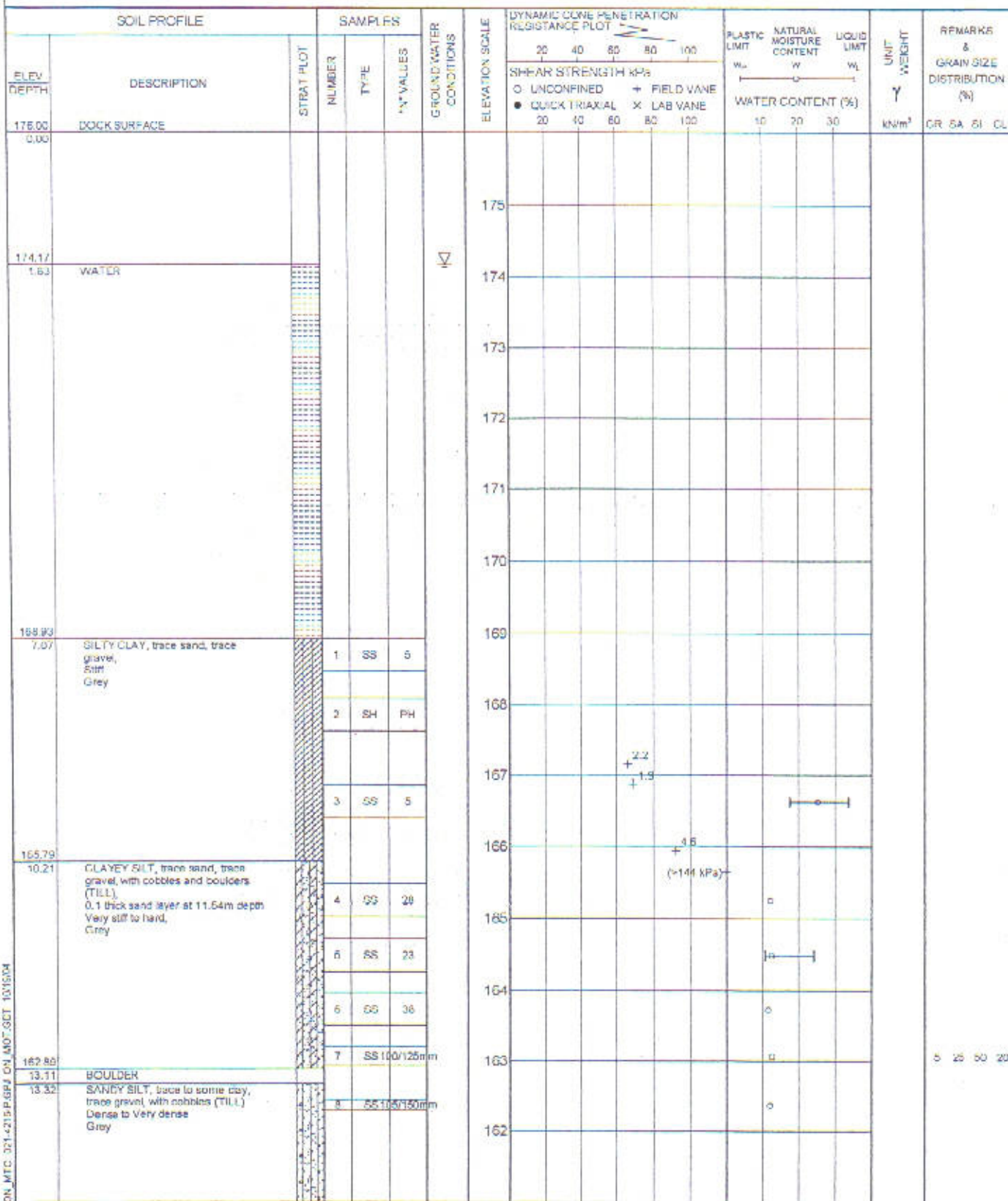
○ 3% STRAIN AT FAILURE

PROJECT 0214216-1.3 RECORD OF BOREHOLE No 102 2 OF 2 METRIC
G.W.P. WP LOCATION REFER TO BOREHOLE LOCATIONS - DRAWING 1 ORIGINATED BY D.M.
DIST HWY N/A BOREHOLE TYPE ROTARY DRILLING (NW CASING), TRI-CONE COMPILED BY WOF
DATUM I.G.L.D. DATE September 14, 2004 - September 14, 2004 CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
EL. EV. DEPTH	DESCRIPTION	STRAT. PLOT	N. MEER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100							
								0 UNCONFINED + FIELD VANE • QUICK TRIAXIAL X LAB VANE							
								20 40 60 80 100							

+ 3 x 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 021-4215-1-3 RECORD OF BOREHOLE No 103 1 OF 2 METRIC
G.W.P. WP LOCATION REFER TO BOREHOLE LOCATIONS - DRAWING 1 ORIGINATED BY DJM
DIST HWY N/A BOREHOLE TYPE ROTARY DRILLING (NW CASING), TRI-CONE COMPILED BY WCF
DATUM I.G.L.D. DATE September 15, 2004 - September 16, 2004 CHECKED BY AMH



Continued Next Page

+ 3, X 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 021-4216-1-3 RECORD OF BOREHOLE No 103 2 OF 2 METRIC
G.W.P. WP LOCATION REFER TO BOREHOLE LOCATIONS - DRAWING 1 ORIGINATED BY DJM
DIST HWY N/A BOREHOLE TYPE ROTARY DRILLING (NW CASING), TRI-CONE COMPILED BY WDF
DATUM I.G.L.D. DATE September 15, 2004 - September 16, 2004 CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N-VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								□ UNCONFINED	+ FIELD VANE	×						LAB VANE	20	40
			9	SS	48													
159.30 16.70	Fresh, microcrystalline to fine grained, thin to medium bedded, light grey to brown, faintly to moderately porous, medium strong to strong, argillaceous DOLOMITIC LIMESTONE, occasionally fossiliferous and intercalated with pitted to vuggy zones and calcite nodules (DUNDEE FORMATION)		10	SS	100/100/100													
			11	CORE	-		53	53	53									
				12	CORE	-		100	100	100								
				13	CORE	-		50	7/8	7/8								
155.82 20.18	END OF BOREHOLE																	

ON MTO 021-4216-1-3.PDF ON MTO 021-4216-1-3

RECORD OF BOREHOLE 22

SHEET 1

LOCATION - SEE FIGURE 1

BORING DATE - SEPT. 18, 1988

DATUM - LGLD

SAMPLER - HAMMER, 35.5kN DROP, 760mm

PENETRATION TEST HAMMER, 35.5kN DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, CM/SEC		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3M	SHEAR STRENGTH CU, KPa 25 50 75 100	nat.V. - + O - rem.V. - O U - O	WATER CONTENT, PERCENT wp 10 20 30 40		
0		WATER LEVEL (LAKE)	173.91								
			0.00								
2		-WATER-									
4											
6		Loose gray SAND occ. gravel and shell.	168.73								
			168.42	1	50 DO	4					
8			8.49								
10		Stiff gray SILTY CLAY trace sand occ. gravel.		2	50 DO	5					
12			165.69								
			8.22	3	50 DO	12					
14		Stiff to hard gray CLAYEY SILT trace sand occ. gravel, cobble (TILL).		4	50 DO	18					
16			160.19								
			13.72	5	50 DO	19					
18				6	50 DO	64					
20		Very stiff gray SILTY CLAY occ. gravel occ. silt partings	158.73								
			16.24	7	50 DO	18					
22		END OF BOREHOLE (PROBABLY LIMESTONE BEDROCK)									

(Golder Report No. 881-3342)

"Note: This Drawing has been Reduced"

LAKE LEVEL
AT ELEV. 173.91
DURING DRILLING

ROTARY DRILLING
1 1/2" TRIP CONE / 1 1/2" CASING 6.86

MH

0
15-20 PERCENT AXIAL STRAIN AT FAILURE
10

(Golder Report No. 881-3342)
"Note: This Drawing has been Reduced"

LAKE LEVEL
AT ELEV. 173.91
DURING DRILLING

MH

0
15-20 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE

LOGGED R.J.M.

RECORD OF BOREHOLE 301

SHEET 1

LOCATION: S.M. Pore 4

BORING DATE: June 22, 1989

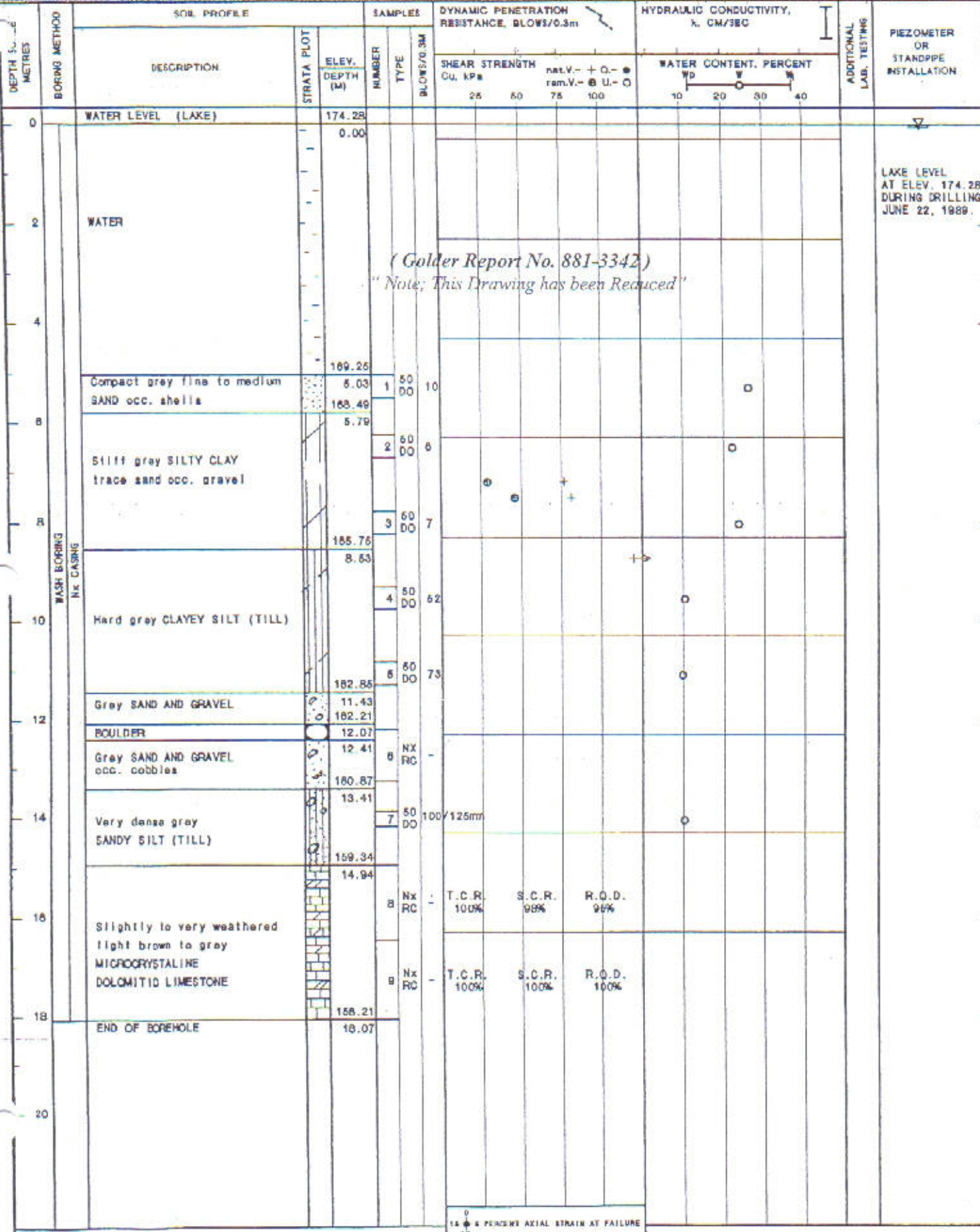
DATUM: 161.0

SAMPLER: HAMMER, 335 kg DROP: 760mm

PENETRATION TEST: HAMMER, 63 kg DROP: 760mm



PROJECT 881-3342-1



LAKE LEVEL AT ELEV. 174.28 DURING DRILLING JUNE 22, 1989.

0 10 20 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

LOGGED D.J.M.

RECORD OF BOREHOLE 302

SHEET 1



LOCATION: Saw. Pit No. 4

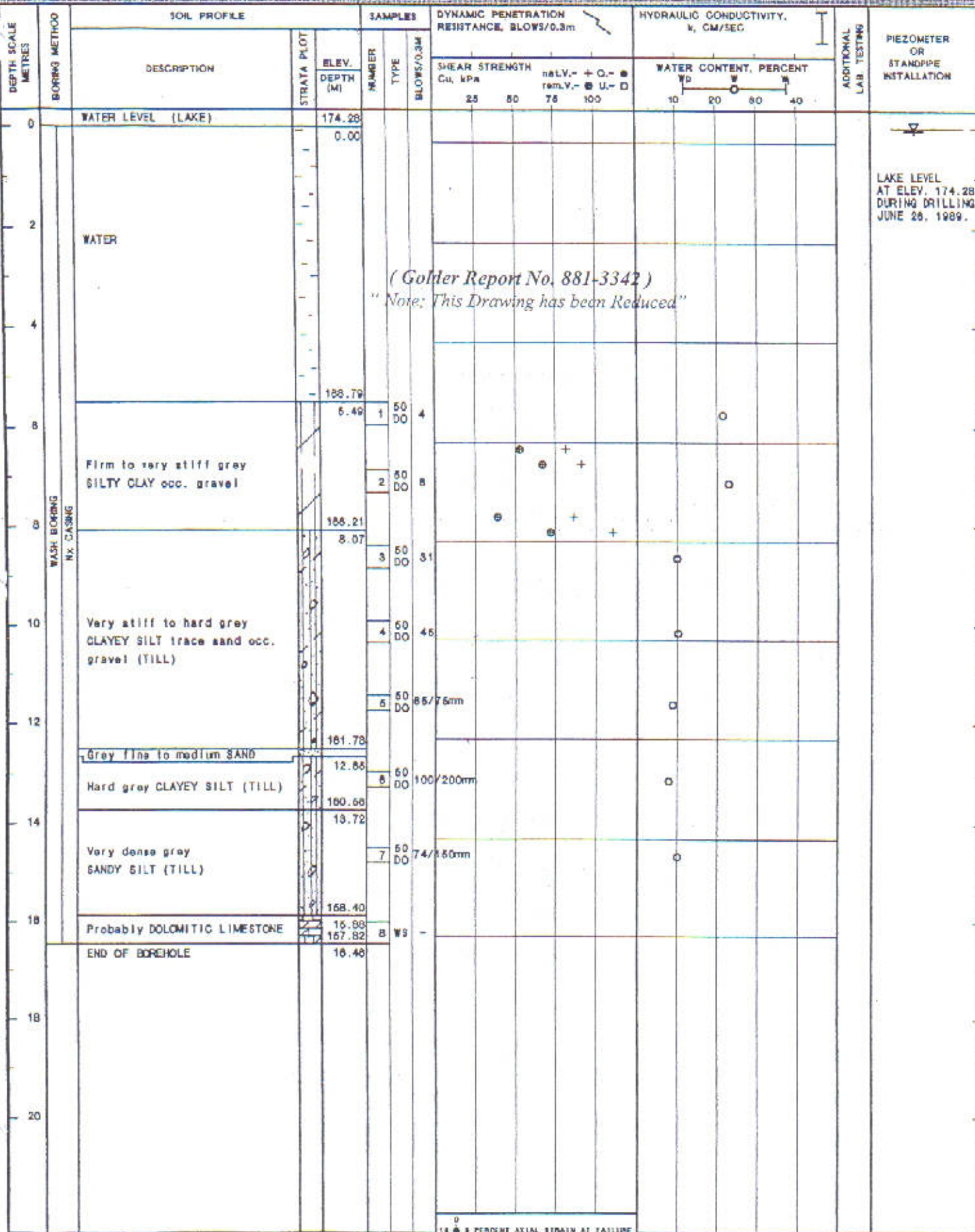
BORING DATE: June 26, 1989

DATUM: ELLD

SAMPLER: HAMMER, 85kg, DROP, 760mm

PENETRATION TEST: HAMMER, 85kg, DROP, 760mm

PROJECT: 881-3342-1



DEPTH SCALE

Golder Associates

LOGGED: D.J.M.

CHECKED: [Signature]

RECORD OF BOREHOLE 303

SHEET 3

LOCATION See Figure 4

BORING DATE June 26, 1989

DATUM 1985

SAMPLER HAMMER, 85.3kg, DROP, 750mm

PENETRATION TEST HAMMER, 85.3kg, DROP, 750mm



PROJECT 881-3342-1

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, CM/SEC		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3M	SHEAR STRENGTH Cal, kPa 25 50 75 100	WATER CONTENT, PERCENT			
								nat.V.- + O.- ● rem.V.- ● U.- ○	10 20 30 40		
0		WATER LEVEL (LAKE)	174.28								
			0.00								
2		WATER									
4			170.28								
		Very loose fine to medium SAND	4.02	1	50 DO	3					
			4.57	2	50 DO	6					
6		Firm to very stiff grey SILTY CLAY occ. gravel									
			166.51								
8			7.77	4	39 DO	45					
		Hard grey CLAYEY SILT (TILL)									
			164.53								
			9.78	5	50 DO	41					
10		END OF BOREHOLE									
12											
14											
16											
18											
20											

(*Golder Report No. 881-3342*)

" Note; This Drawing has been Reduced "

LAKE LEVEL
AT ELEV. 174.28
DURING DRILLING
JUNE 26, 1989.

0
16 → 8 PERCENT AXIAL STRAIN AT FAILURE

(Golder Report No. 881-3342)
"Note: This Drawing has been Reduced"

LAKE LEVEL
AT ELEV. 174.28
DURING DRILLING
JUNE 26, 1989.

0
16-18 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE

LOGGED D.J.M.