DETAIL FOUNDATION INVESTIGATION AND DESIGN REPORT

for

HOWARD AVENUE OVERHEAD
SITE NO. 6-594
HOWARD AVENUE / CPR GRADE SEPARATION
GWP 3030-06-00
CITY OF WINDSOR, ONTARIO

PETO MacCALLUM LTD.
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PML Ref.: 07TF022A-1
Index No.: 148FIR and 149FDR
GEOCRES No.: 40J6-21
February 25, 2009

Peto MacCallum Ltd.
CONSULTING ENGINEERS

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for
Howard Avenue Overhead
Site No. 6-594
Howard Avenue / CPR Grade Separation
GWP 3030-06-00
City of Windsor, Ontario

1. INTRODUCTION

This report summarises the results of a detail foundation investigation carried out for construction of the Canadian Pacific Railway (CPR) grade separation bridges at Howard Avenue in the City of Windsor, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The grade separation will include bridges to carry the existing and future sets of railway tracks and a security / maintenance road running parallel to the railway over Howard Avenue.

The current plans call for Howard Avenue to pass under the CPR at approximate Station 10+298, Howard Avenue chainage. Information from borehole 1 drilled by Golder Associates Ltd. (Golder) in August 2006 during a preliminary investigation at the site and included in the RFP has been incorporated in this report.

The report provides detail subsurface information pertaining to the proposed overhead and security road bridge foundations and approaches. Other foundation facets of this project were reported separately to efficiently incorporate changes in the design. The following separate reports were prepared:

PML Ref. No.	Report Title
07TF022A-1	Canadian Pacific Railway Overhead
07TF022A-2	Retaining Walls
07TF022A-3	Road Cuts and Deep Sewers
07TF022A-4	Pumping Station
07TF022A-5	SWM Ponds
07TF022A-6	Watermain Tunnels

The Final Detail Foundation Investigation Report should be listed in SP 109F10.

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BARRIE, HAMILTON, KITCHENER, TORONTO

2. SITE DESCRIPTION AND GEOLOGY

The site is located on the CPR alignment at the crossing of Howard Avenue about 5 km north of the Highway 401 / Howard Avenue interchange in Windsor, Ontario. The alignment of the railway overhead is considered to be west-east.

Land use in the vicinity of the site includes residential and commercial properties and community facilities. The footprint of the proposed structures is currently the at-grade crossing of the transportation corridors of Howard Avenue and the CPR. A hydro corridor including high voltage power lines runs parallel and to the north of the CPR tracks.

The topography is relatively flat, slightly rising to the east. The unpaved ground beyond the CPR right-of-way, Howard Avenue and Memorial Drive is covered with grass and stands of mature trees.

The project site is situated in the Essex Clay Plain physiography region within the St. Clair Clay Plain. The native soils are mainly represented by cohesive glacial till deposits. The typical rock type in the project area is Middle Devonian limestone of the Paleozoic era. The bedrock is at depths of more than 35 m at the site.

3. INVESTIGATION PROCEDURES

Most of the field work for this study was carried out during the period of October 15 to 31, 2007 and comprised three boreholes advanced at the structure to depths of 41.4 to 45.4 m. It is noteworthy that drilling of borehole 107 put down in private lands and interrupted on October 17, 2007 at the landowner’s request was completed on October 8, 2008. The current boreholes were numbered in the 100-series (105, 107 and 108) to distinguish them from the boreholes drilled by Golder during the preliminary investigation. The relevant data from the preliminary investigation report including the record of borehole 1 and laboratory test results are enclosed in Appendix A. The borehole locations are indicated on Drawing ST-1, appended.

The locations of the boreholes were established in the field by Peto MacCallum Ltd. and cleared for the presence of underground services and utilities. A hydro corridor carrying high voltage power lines and running parallel to the CPR and the security road right-of-way limited access for drill rigs during the investigation. The ground surface elevations at the boreholes were provided by Callon Dietz Ltd., Ontario Land Surveyors. All elevations in this report are geodetic and expressed in metres.

The three current boreholes were advanced using continuous flight hollow stem augers and mud rotary drilling methods, powered by a truck-mounted CME-75 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. The boreholes were extended 2.8 to 6.6 m into bedrock using NQ diamond rock coring equipment. The borehole depths for the bridges are as follows:

Borehole No.	Depth, m		
	Auger	Rock Core(1)	Total
1	38.4	3.0	41.4
105	39.4	3.9	43.3
107	38.8	6.6	45.4
108	39.3	2.8	42.1

(1) NQ diamond rock coring equipment

Representative soil samples were recovered during drilling using a conventional split spoon sampler. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. Penetrometer and in situ vane shear tests were also performed to assess the shear strength of the cohesive soils. It is noted that the results of penetrometer tests may be lower than actual values due to sample disturbance.

Groundwater conditions at the borehole locations were assessed during drilling by visual examination of soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes. A piezometer was installed in borehole 107 after its completion in October 2008; five readings were taken during an 8-day period. The boreholes were backfilled with a bentonite/cement mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. In addition, 15 Atterberg limits tests and 16 grain size distribution analyses were conducted on selected samples, with the results presented in Figures PC-ST-1, GS-ST-1 and GS-ST-2. Further, unconfined compression tests were performed on three cohesive samples. The laboratory test results are shown on the Record of Borehole sheets.

4. SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classification, inferred stratigraphy, boundary elevations, standard penetration test data, penetrometer and in situ vane undrained shear strength values and groundwater observations. The results of laboratory unconfined compressive strength tests, Atterberg limits testing, grain size distribution analyses and moisture content determination are also shown on the Record of Borehole sheets. The borehole locations are indicated on Drawing ST-1.

The stratigraphic profile along the centreline of the structure alignment and characteristic cross-sections at both abutments are presented on Drawing ST-2. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial fill or topsoil underlain by an extensive deposit of clayey silt till mantling limestone bedrock. The soil referred to as silty clay till in the preliminary investigation (borehole 1) is described as clayey silt till in accordance with the MTO standard soil classification. The bedrock surface was contacted at depths of 38.4 to 39.4 m (elevation 148.7 to 150.2). The strata encountered are summarised below.

4.1 Fill

A pavement structure from an existing commercial parking area consisting of 130 mm thick asphaltic concrete and 630 mm thick granular material was reported in borehole 1. The granular fill overlay 760 mm of silty clay fill mixed with sand and gravel. The moisture content of the granular fill and silty clay fill was 5 and 17% respectively. The fill was penetrated at 1.5 m depth (elevation 187.0).

Surficial fill composed of sandy silt over slag and cinder was present in borehole 105 and of topsoil over sandy silt with organic inclusions in borehole 108. The fill was loose in relative density (SPT-‘N’ values of 7, 9) and had a moisture content of 16 and 29%. The fill was 400 and 700 mm in thickness and penetrated at elevation 187.7 and 187.4 respectively.

4.2 Topsoil

Topsoil was present surficially in borehole 107 drilled in a grassed area located northeast of the CPR and Howard Avenue intersection. Identified as low organic clayey sandy silt, the topsoil had a thickness of 200 mm and was penetrated at elevation 187.4.

4.3 Clayey Silt Till

Directly beneath the fill or topsoil at depths of 0.2 to 1.5 m (elevation 187.0 to 187.7) in all the boreholes was a major cohesive deposit of clayey silt till. Interlayered with silty sand till in borehole 107, this deposit had a total thickness of 36.9 to 39.0 m. The clayey silt till was penetrated at depths of 38.4 to 39.4 m (elevation 148.7 to 150.2), with boulders detected in borehole 105 just above bedrock at a depth of 38.8 m (elevation 149.3).

The consistency of the clayey silt till was typically very stiff in the upper 4 to 5 m thick zone and firm to stiff underneath. The results of in situ vane testing carried out in the lower zone of the deposit yielded undisturbed shear strength values in a typical range of 50 to 100 kPa (soil sensitivity of 2). Penetrometer tests on samples of the clayey silt till indicated a shear strength varying between 20 and 100 kPa. Unconfined compression testing on Shelby tube samples of the deposit gave undrained shear strength values of 31 to 51 kPa (strain at failure of 11 to 18%).

The results of Atterberg limits testing and grain size distribution analyses conducted during the current investigation on 15 cohesive samples are presented in Figures PC-ST-1 and GS-ST-1 respectively. The liquid limit of the clayey silt till ranged from 16 to 33 and plastic limit from 10 to 18, with a corresponding range in the plasticity index of 6 to 15. The moisture content of the deposit typically varied between 13 and 24%. The Atterberg limits and moisture content results including those from borehole 1 are given in Table 1.

4.4 Silty Sand Till

A layer of cohesionless silty sand till was encountered within the clayey silt till at a depth of 5.4 m (elevation 182.2) in borehole 107. This layer was 1.5 m thick and penetrated at 6.9 m depth (elevation 180.7). The till was compact in relative density (SPT-‘N’ value of 21).

The silty sand till had a moisture content of about 16%. The results of grain size distribution analysis performed on this material are presented in Figure GS-ST-2.

4.5 Bedrock

Bedrock was contacted below the clayey silt till at depths of 38.4 to 39.4 m (elevation 148.7 to 150.2) in all of the four boreholes. The bedrock surface is relatively flat, rising in the southeast direction from elevation 148.7 to 148.8 at boreholes 105, 107 and 108 to elevation 150.2 at borehole 1.

The bedrock comprises light grey Middle Devonian limestone. A detailed description of the bedrock is given in Table 2. Photographs of the rock cores retrieved during the current investigation are shown in Appendix B.

The measured core recovery varied between 63 and 100%. The RQD determined from rock cores in the current study ranged from 53 to 100%, thus indicating a fair to excellent quality rock. The borehole 1 log shows a poor to very poor quality rock in the upper 1.4 m thick zone below the bedrock surface, improving with depth to a good quality rock.

4.6 Groundwater

Perched water was detected in the process of augering at a depth of 1.1 m (elevation 187.5) in borehole 1 and 0.4 m depth (elevation 187.7) in borehole 105. Groundwater observations were not carried out in any of the boreholes upon completion of drilling in October 2007 because the boreholes were charged with drilling water. A piezometer was installed in borehole 107 after completion of drilling in October 2008. The water level readings taken in the piezometer during a period of eight days were as follows:

Date	Piezometric Water Level, m	
	Depth	Elevation
October 8, 2008	19.3	168.3
October 10, 2008	18.9	168.7
October 14, 2008	17.7	169.9
October 15, 2008	17.4	170.2
October 16, 2008	17.0	170.6

The slow rise in the observed water levels indicated that the native clayey silt till subsoil is relatively impervious. The readings were discontinued due to the long time required for completion of groundwater level stabilization. Based on the water content profile of the soil samples, it is anticipated that the groundwater at the site is at about 5.4 m depth, elevation 182.2.

Groundwater levels are subject to seasonal fluctuations and precipitation patterns.

5. CLOSURE

The field work was carried out under the supervision of Mr. M. Rapsey, Senior Technician, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. The drilling equipment was supplied by Aardvark Drilling Ltd. The laboratory work was carried out in the PML laboratory in Toronto.

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

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GD/CN/BRG/gd-mi

TABLE 1

ATTERBERG LIMITS AND MOISTURE CONTENT RESULTS

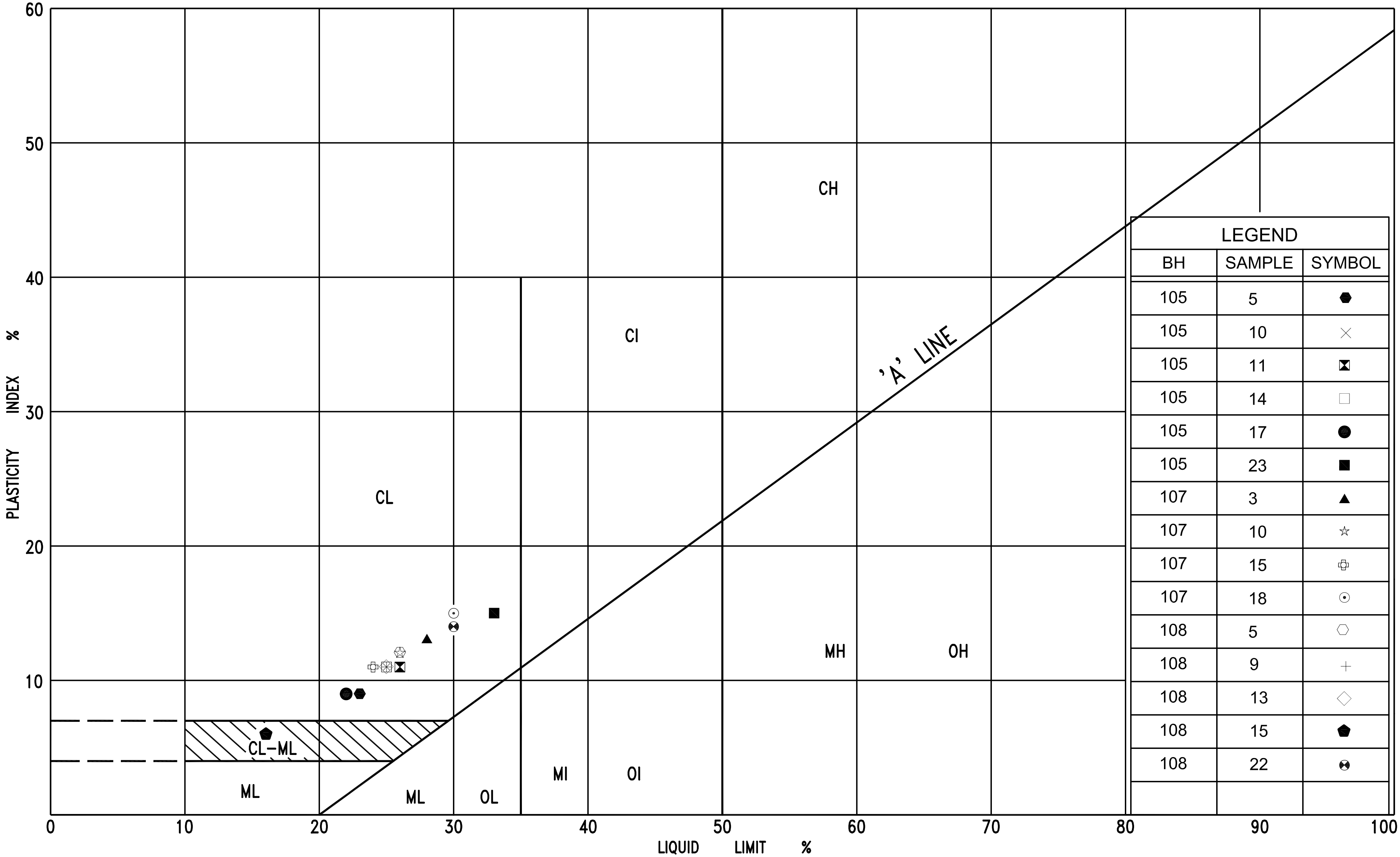
SOIL TYPE	BOREHOLE No.	SAMPLE No.	MOISTURE CONTENT (%)	LIQUID LIMIT (W_L)	PLASTIC LIMIT (W_P)	PLASTICITY INDEX (PI)
Clayey Silt Till	105	5	14	23	14	9
		10	19	25	14	11
		11	19	26	15	11
		14	18	25	14	11
		17	18	22	13	9
		23	24	33	18	15
	107	3	13	28	15	13
		10	20	26	14	12
		15	18	24	13	11
		18	20	30	15	15
	108	5	15	26	14	12
		9	19	25	14	11
		13	21	25	14	11
		15	16	16	10	6
		22	24	30	16	14
	1	10	19	25	14	11
		15	17	24	14	10

TABLE 2
ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
BH	**RC**	**DEPTH (m)**	**REC (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
105	24	39.4 – 40.7	100	90	39.4 – 43.3	LIMESTONE: Light grey, fine crystalline to aphanitic, with few styolitic partings, small chert nodules, occasional fossils, high strength, unweathered, close to moderate spaced flat partings, rough planar, tight, fair to good quality. (A 300 mm drop in core barrel and loss of water pressure was reported at 43 m depth during drilling. Some sand was observed near bottom of run. This is believed to be associated with an infilled vertical fissure rather than a continuous layer.)
	25	40.7 – 42.2	67	53		
	26	42.2 – 43.3	88	74		
107	22	38.8 – 39.3	100	100	38.8 – 39.4	LIMESTONE: Light grey, fine crystalline to aphanitic, with few styolitic partings, occasional fossils, high strength, unweathered, moderate spaced flat bedding layers, rough planar, tight, excellent quality.
	23	39.3 – 40.8	63	55		
	24	40.8 – 42.2	72	55	39.4 – 42.4	LIMESTONE WITH CLAY LAYERS: Limestone, as above, in 25 to 580 mm thick layers, interbedded with soft clay and/or sandy layers (typically 140 to 560 mm thick, total 990 mm), very close to moderate spaced flat bedding layers, rough planar, fair quality.
	25	42.2 – 43.9	98	98		
	26	43.9 – 45.4	100	100		
					42.4 – 45.4	LIMESTONE: Light grey to buff coloured, fine crystalline to aphanitic, occasional fossils, high strength, unweathered, wide spaced flat bedding layers, rough planar, tight, excellent quality.
108	23	39.3 – 40.5	100	100	39.3 – 42.1	LIMESTONE: Light grey becoming mottled brown, fine crystalline to aphanitic, with few styolitic partings, small chert nodules, high strength, unweathered, wide to moderate spaced flat partings, rough planar, tight, excellent quality.
	24	40.5 – 41.1	100	100		
	25	41.1 – 42.1	100	100		

RQD = Rock Quality Designation

Originated: JFW
Compiled: FP
Checked: GD / CN



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PLASTICITY CHART

CLAYEY SILT, some sand to sandy, trace gravel
(TILL)

FIG No. PC-ST-1
HWY: HOWARD AVENUE
G.W.P. No. 3030-06-00

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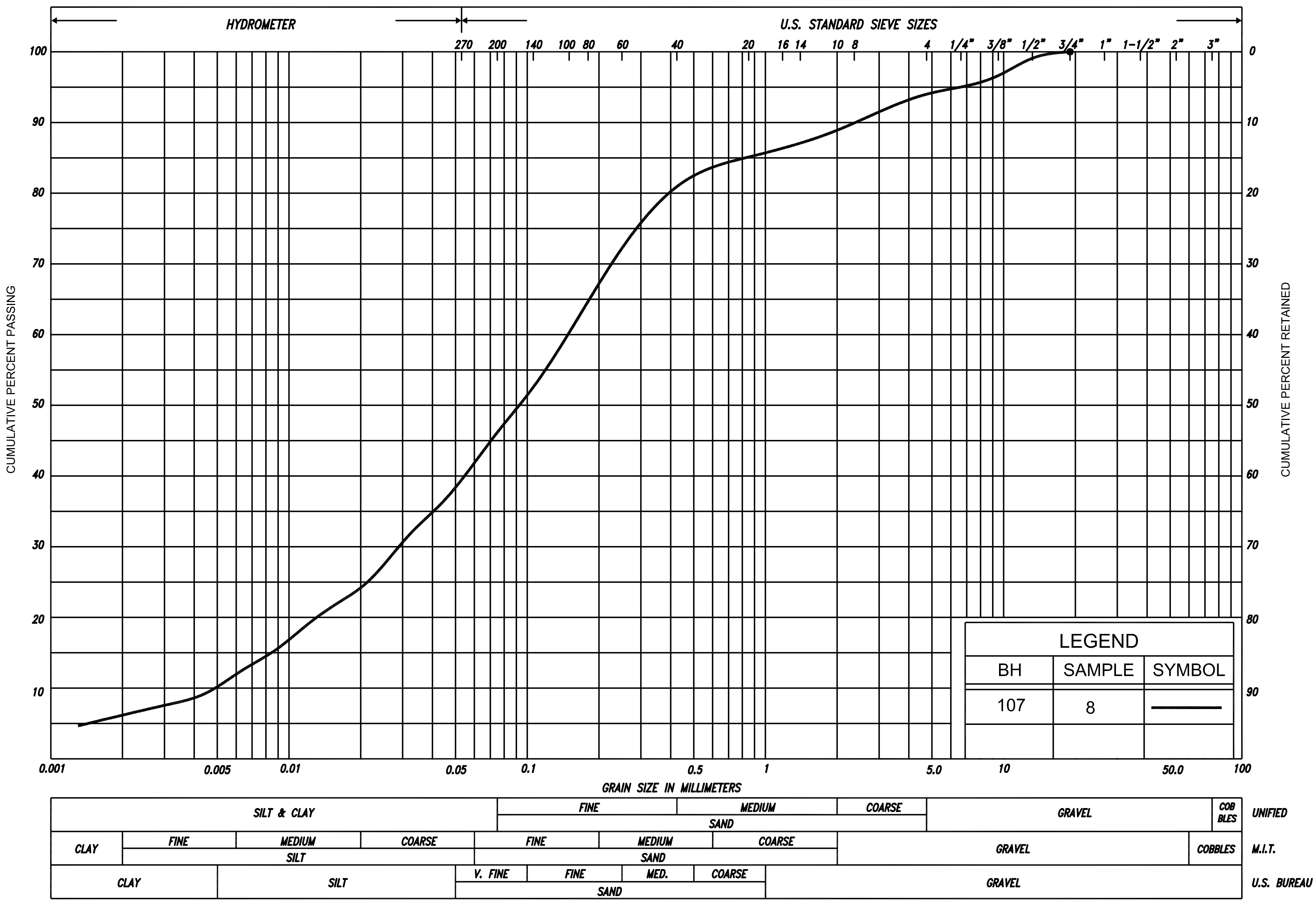
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand to sandy, trace gravel (TILL)

FIG No.	GS-ST-1
HWY:	HOWARD AVENUE
G.W.P. No.	3030-06-00



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GRAIN SIZE DISTRIBUTION

SILTY SAND , trace clay, trace gravel
(TILL)

FIG No.	GS-ST-2
HWY:	HOWARD AVENUE
G.W.P. No.	3030-06-00

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO

n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

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Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 105

METRIC

G.W.P. 3030-06-00 LOCATION Co-ords: 4 684 016 N; 334 030 E ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring COMPILED BY N.S.B

DATUM Geodetic DATE October 29 to 31, 2007 CHECKED BY C.N.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
188.1	Ground Surface
0.0	Sandy silt, trace clay Loose Brown Moist (FILL)
187.7 / 0.4	Slag and cinder Black Wet
	Clayey silt with sand, trace gravel Firm Brown Moist
	sandy Stiff to hard (TILL)
	Mottled grey
	Grey
	Firm to stiff
173.1	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
1	SS	7	Perched water at 0.4m	
2	SS	3		
3	SS	4		
4	SS	12		
5	SS	28	2 34 42 22	
6	SS	31		
7	SS	14		
8	SS	10		
9	SS	10		
	FV			
10	TW	PH	2 32 39 27	21.7
11	SS	5	3 31 38 28	
	FV			
12	SS	3		
	FV			
13	SS	4		
	FV			
14	SS	3	1 32 38 29	
	FV			

GROUND WATER CONDITIONS: ∇*

ELEVATION SCALE: 188, 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

Field vane values: 176 (sensitivity 2); sensitivity 2; 2; 2; 2

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

+7, ×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 105

METRIC

G.W.P. 3030-06-00	LOCATION Co-ords: 4 684 016 N; 334 030 E		ORIGINATED BY M.R.
DIST 32 HWY Howard Avenue	BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring		COMPILED BY N.S.B
DATUM Geodetic	DATE October 29 to 31, 2007		CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
173.1 / 15.0	Clayey silt, sandy trace gravel Stiff Grey Moist (TILL)	15	SS	3	173	
					172	
		16	SS	4	171	
					170	
		17	SS	3		1 33 40 26
					169	
					168	
					167	
		18	SS	6	166	
					165	
					164	
		19	SS	8	163	
					162	
	trace sand lenses of silt silty clay layer Firm				161	
	some sand, trace gravel Stiff Moist	20	SS	5	160	
					159	
158.1	Cont'd					

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; STRAT PLOT; GROUND WATER CONDITIONS.

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 105

METRIC

G.W.P. 3030-06-00 LOCATION Co-ords: 4 684 016 N; 334 030 E ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring COMPILED BY N.S.B

DATUM Geodetic DATE October 29 to 31, 2007 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
158.1 / 30.0	Clayey silt some sand, trace gravel Stiff to very stiff Grey Moist (TILL)				158		
		21	SS	10	157		
		22	SS	16	155–154		
		23	SS	9	152–151	wP–w–wL plotted	1 16 45 38
	boulders				150–149		
148.7 / 39.4	Limestone bedrock Unweathered High strength Fair to good quality	24	RC NQ	REC 100%	148		RQD 90%
		25	RC NQ	REC 67%	147		RQD 53%
	sand seams at depths of 41.9 and 43.0m	26	RC NQ	REC 88%	146–145		RQD 74%
144.8 / 43.3	End of borehole						

Cont'd

+7, ×5: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 105

METRIC

G.W.P. 3030-06-00 LOCATION Co-ords: 4 684 016 N; 334 030 E ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring COMPILED BY N.S.B

DATUM Geodetic DATE October 29 to 31, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				

* 2007 10 29

∇ Water level observed during drilling

** Borehole charged with drilling water

■ Penetrometer test

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

+7, ×5 : Numbers refer to Sensitivity

20 / 15 — ○ — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 107

METRIC

G.W.P. 3030-06-00 | LOCATION Co-ords: 4 684 048 N; 334 075 E | ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue | BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring | COMPILED BY M.N.

DATUM Geodetic | DATE October 15 to 17, 2007 and October 6 to 8, 2008 | CHECKED BY C.N.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
187.6	Ground Surface
0.0	Topsoil
187.4 / 0.2	Clayey silt with sand, trace gravel. Stiff to hard, Brown, Moist (TILL). Mottled grey; Grey
182.2 / 5.4	Silty sand trace clay, trace gravel. Compact, Grey, Wet (TILL)
180.7 / 6.9	Clayey silt with sand, trace gravel. Firm to stiff, Grey, Moist (TILL)
172.6	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	Shear strength (kPa) / Sensitivity	Water content	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	12		○		
2	SS	13				
3	SS	24		○ with plastic and liquid limits		3 30 40 27
4	SS	25		○		
5	SS	37		○		
6	SS	30		○		
7	SS	12		○		
8	SS	21		○ Non-plastic		6 48 40 6
9	SS	3		○		
	FV		+ 2			
10	TW	PH	○	○ with plastic and liquid limits	21.2	5 29 40 26
	FV		+ 2			
11	SS	6				
	FV		+ 2			
12	SS	4		○		
	FV		+ 2			
13	SS	3		○		
	FV		+ 2			

ELEVATION SCALE: 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 107

METRIC

G.W.P. 3030-06-00 | LOCATION Co-ords: 4 684 048 N; 334 075 E | ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue | BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring | COMPILED BY M.N.

DATUM Geodetic | DATE October 15 to 17, 2007 and October 6 to 8, 2008 | CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
172.6 15.0	Clayey silt with sand, trace gravel Firm to stiff Grey Moist (TILL)										
		14	SS	4	172						
			FV			Field vane +2					
		15	SS	4	171		Wp–w–WL	5	30	41	24
		16	SS	5	169		w				
		17	SS	7	166		w				
		18	SS	7	163		Wp–w–WL	2	23	40	35
		19	SS	7	160		w				
157.6	Cont'd				158						

Elevation scale: 172, 171, 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158

+7, ×5 : Numbers refer to Sensitivity

20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 107 METRIC

G.W.P. 3030-06-00 LOCATION Co-ords: 4 684 048 N; 334 075 E ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring COMPILED BY M.N.

DATUM Geodetic DATE October 15 to 17, 2007 and October 6 to 8, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
157.6 / 30.0	Clayey silt with sand, trace gravel; Stiff to very stiff; Grey; Moist (TILL)					
		20	SS	11	o (~19)	
		21	SS	17	o (~19)	
148.8 / 38.8	Limestone bedrock; Unweathered; High strength; Fair to excellent quality	22	RC NQ	REC 100%		RQD 100%
		23	RC NQ	REC 63%		RQD 55%
		24	RC NQ	REC 72%		RQD 55%
		25	RC NQ	REC 98%		RQD 98%
142.6	Cont'd	26	RC NQ	REC 100%		RQD 100%

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³; GR SA SI CL

$+^7$, $\times^5$: Numbers refer to Sensitivity; 20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 107

METRIC

G.W.P. 3030-06-00 LOCATION Co-ords: 4 684 048 N; 334 075 E ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring COMPILED BY M.N.

DATUM Geodetic DATE October 15 to 17, 2007 and October 6 to 8, 2008 CHECKED BY C.N.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
142.6 45.0											
142.2 45.4	End of borehole										

* 2008 10 16

▼ Water level measured after drilling

■ Penetrometer test

Piezometer Legends :

Bentonite seal

Filter sand

Screen

Bentonite bed

Native bed

Water Level Readings :

Date	Depth (m)	Elev.
10/08/2008	19.3	168.3
10/10/2008	18.9	168.7
10/14/2008	17.7	169.9
10/15/2008	17.4	170.2
10/16/2008	17.0	170.6

C.F.H.S.A: denotes Continuous Flight Hollow Stem Augers

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 108

METRIC

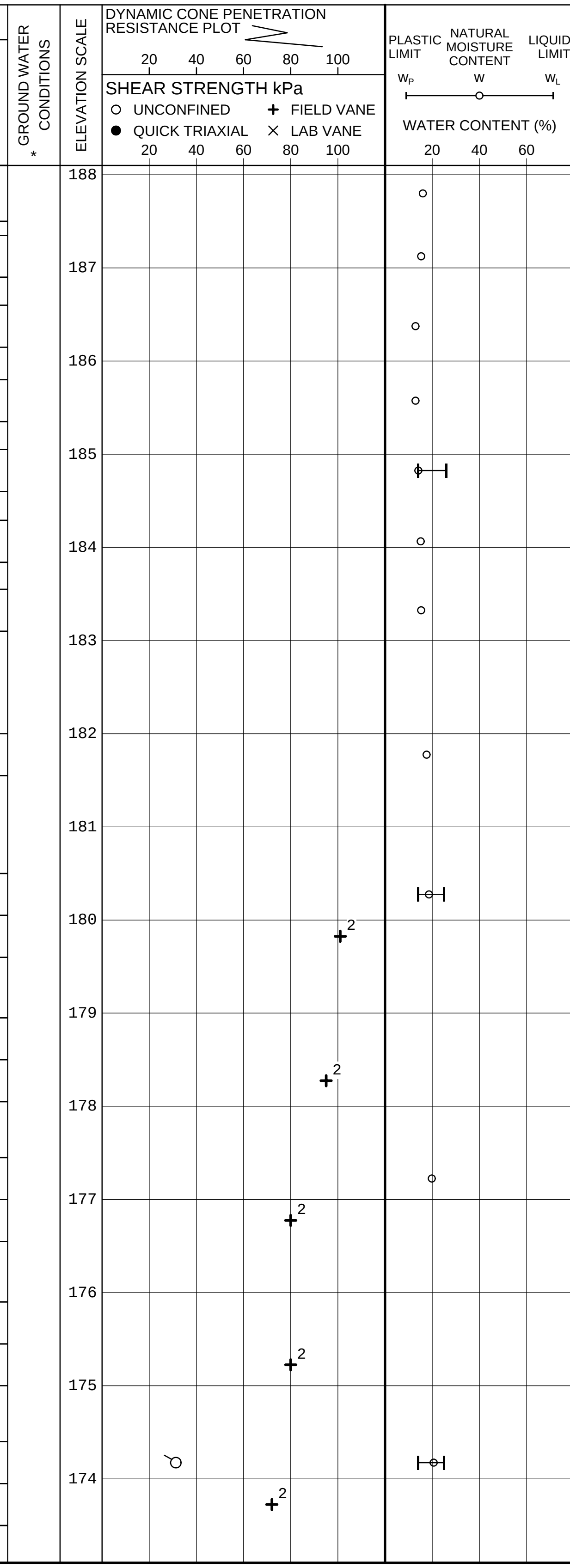
G.W.P. 3030-06-00	LOCATION Co-ords: 4 684 049 N; 334 023 E	ORIGINATED BY M.R.
DIST 32 HWY Howard Avenue	BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring	COMPILED BY N.S.B
DATUM Geodetic	DATE October 16 to 19, 2007	CHECKED BY C.N.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
188.1	Ground Surface
0.0	Topsoil
187.9 / 0.2	Sandy silt, trace clay organics, ashes, coal; Loose; Mottled black/brown; Moist (FILL)
187.4 / 0.7	Clayey silt, sandy trace gravel; Stiff to hard; Brown; Moist (TILL)
	Very stiff to stiff; Grey
173.1	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
1	SS	9		
2	SS	13		
3	SS	15		
4	SS	14		
5	SS	28	6 33 37 24	
6	SS	38		
7	SS	21		
8	SS	11		
9	SS	8	2 30 42 26	
	FV			
10	SS	8		
	FV			
11	SS	5		
	FV			
12	TW	PH		
	FV			
13	TW	PH	10 29 37 24	23.1
	FV			



+7, ×5 : Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 108

METRIC

G.W.P. 3030-06-00	LOCATION Co-ords: 4 684 049 N; 334 023 E		ORIGINATED BY M.R.
DIST 32 HWY Howard Avenue	BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring		COMPILED BY N.S.B
DATUM Geodetic	DATE October 16 to 19, 2007		CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH / WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
173.1 / 15.0	Clayey silt, sandy trace gravel. Stiff to very stiff, Grey, Moist (TILL)				173		
		14	SS	3			
			FV			Field vane 165, sensitivity 5	
					172		
		15	SS	4	171		10 46 32 12
					170		
		16	SS	3			
					169		
					168		
					167		
		17	SS	12			
					166		
					165		
					164		
		18	SS	6			
					163		
					162		
	trace sand silty clay layer. Firm						
					161		
	some sand, trace gravel. Stiff to very stif	19	SS	5			
					160		
					159		
158.1	Cont'd						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

+7, ×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 108

METRIC

G.W.P. 3030-06-00 | LOCATION Co-ords: 4 684 049 N; 334 023 E | ORIGINATED BY M.R.

DIST 32 HWY Howard Avenue | BOREHOLE TYPE C.F.H.S.A. + Mud Rotary + NQ Coring | COMPILED BY N.S.B

DATUM Geodetic | DATE October 16 to 19, 2007 | CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
158.1 / 30.0	Clayey silt, some sand, trace gravel. Stiff to very stiff, Grey, Moist (TILL)				*	158		
		20	SS	11		157		
		21	SS	14		155–154	w ≈ 19	
		22	SS	13		152–151	wP–w–wL ≈ 16–23	1 12 55 32
148.8 / 39.3	Limestone bedrock. Unweathered. High strength. Excellent quality	23	RC NQ	REC 100%		148		RQD 100%
		24	RC NQ	REC 100%		147		RQD 100%
		25	RC NQ	REC 100%				RQD 100%
146.0 / 42.1	End of borehole					146		

* Borehole charged with drilling water

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

+7, ×5 : Numbers refer to Sensitivity. 15–○–5 (20 above, 10 below): (%) STRAIN AT FAILURE

METRIC

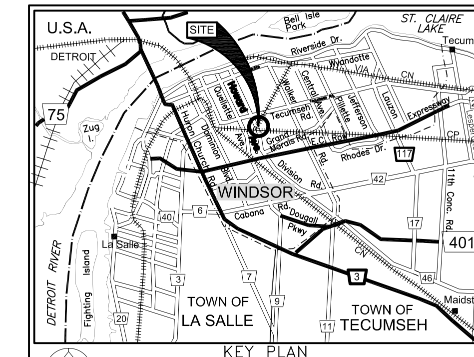
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
GWP No 3030-06-00

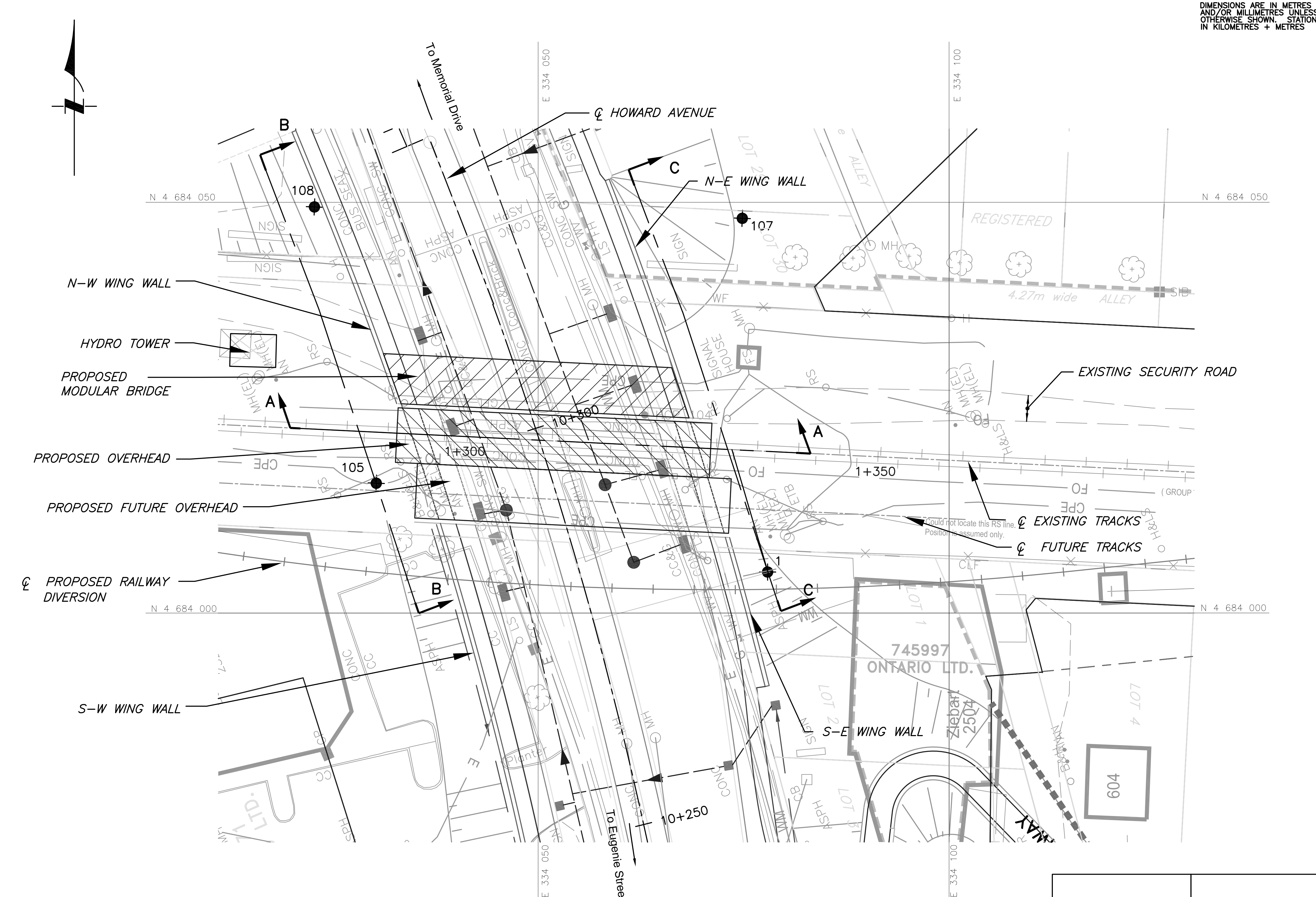
HOWARD AVENUE OVERHEAD
HOWARD AVENUE/CPR GRADE SEPARATION
BOREHOLE LOCATIONS

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



KEY PLAN
SCALE
2 0 2 4 6km



PLAN
SCALE
5 0 5 10m

LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N — Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE — Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation Oct. 2007 and Oct. 2008; Borehole 1: Aug. 2006
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	COORDINATES NORTHINGS	COORDINATES EASTINGS
1	188.6	4 684 005	334 078
105	188.1	4 684 016	334 030
107	187.6	4 684 048	334 075
108	188.1	4 684 049	334 023

– NOTE –
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REGISTERED PROFESSIONAL ENGINEER
C M P Nascimento
Feb 24, 09
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
Feb 24, 2009
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

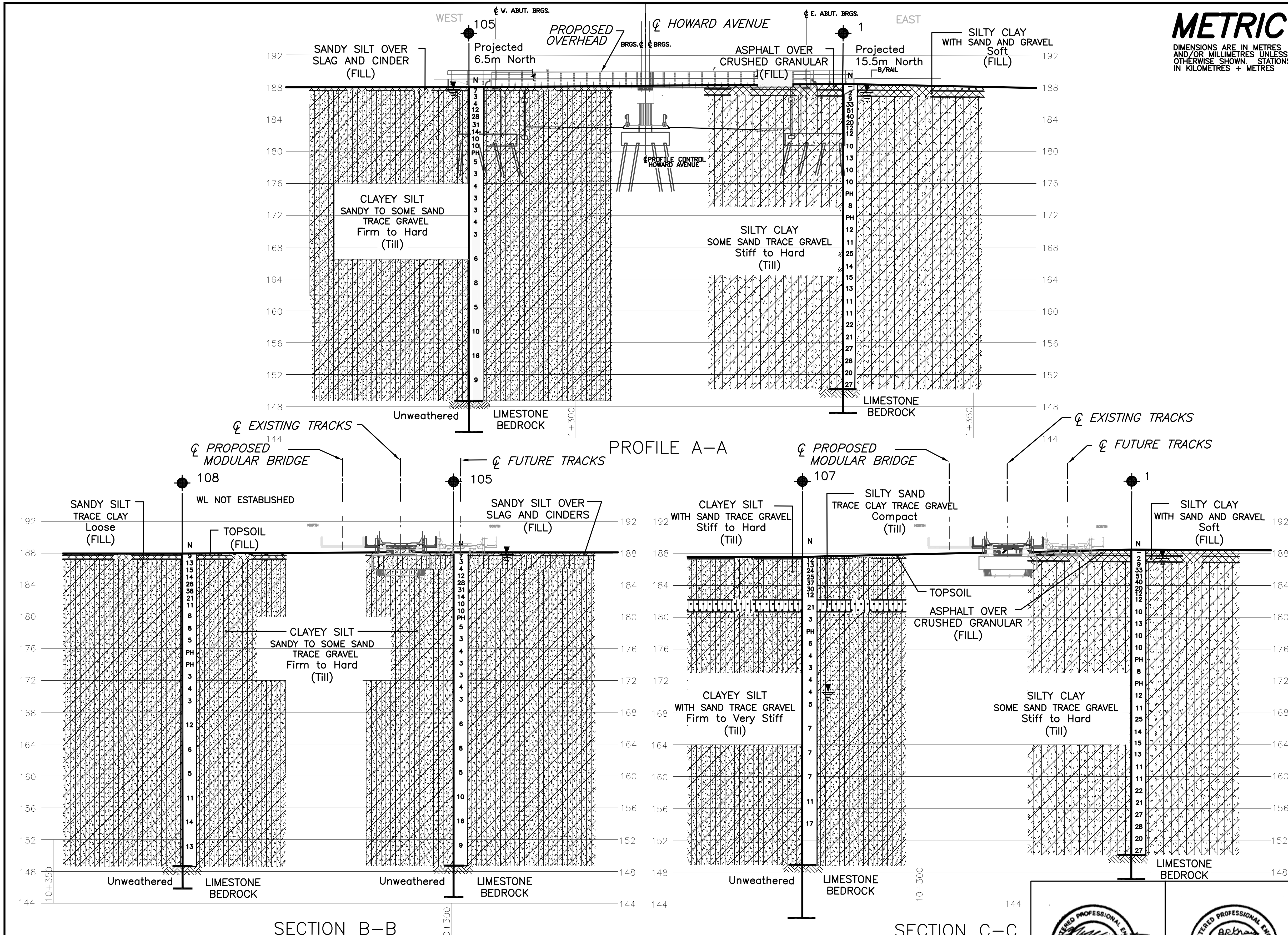
Geocres No. 40J6-21

HWY No HOWARD AVENUE			DIST LONDON
SUBM'D GD	CHECKED GD	DATE FEB. 24, 2009	SITE 6-594
DRAWN NA	CHECKED CN	APPROVED BRG	DWG ST-1

REF No MRC DRAWINGS: H6933XA01.dwg; H6933XB01.dwg; H6933XN01.dwg; H6933xu01.dwg; H6933XY2.dwg and H6933Xd2-prop-req.dwg; dated May 13, 2008

NOTES:

1. REFER TO DRAWING ST-2 FOR PROFILE A-A AND SECTIONS B-B, C-C.
2. BOREHOLE 1 WAS DRILLED BY GOLDER ASSOCIATES IN AUGUST 2006.
3. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No

GWP No 3030-06-00

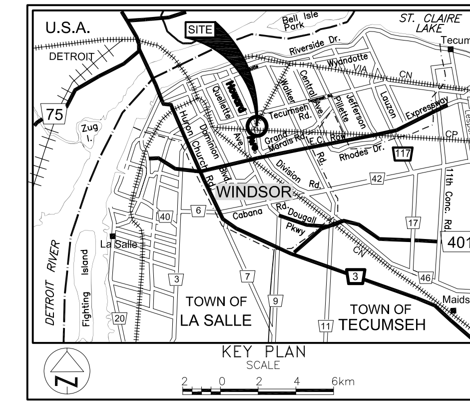
HOWARD AVENUE OVERHEAD

CPR/HOWARD AVENUE GRADE SEPARATION

SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



LEGEND

●	Borehole
⊕	Dynamic Cone Penetration Test (Cone)
⊕	Borehole & Cone
N	Blows/0.3m (Std. Pen Test, 475J/blow)
CONE	Blows/0.3m (60° Cone, 475J/blow)
▽	W L at time of investigation Oct. 2007 and Oct. 2008; Borehole 1 : Aug. 2006
▽	Head
▽	ARTESIAN WATER Encountered
ǀ	PIEZOMETER

BH No	ELEVATION	COORDINATES NORTHINGS	EASTINGS
SEE DRAWING ST–1 FOR DETAILS			

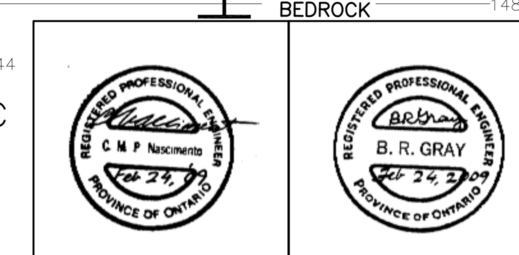
– NOTE –

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

NOTES:

1. REFER TO DRAWING ST–1 FOR BOREHOLE LOCATIONS PLAN.
2. BOREHOLE 1 WAS DRILLED BY GOLDER ASSOCIATES IN AUGUST 2006.
3. LONGITUDINAL PROFILE ALONG SECURITY ROAD BRIDGE WAS NOT REPRODUCED IN THIS DRAWING.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION. PROPOSED PIER VIEWS PROJECTED IN SECTIONS B–B AND C–C FOR ILLUSTRATION PURPOSES ONLY.

SCALE
5 0 5 10m



REF No MRC DRAWINGS: H6933XA01.dwg; H6933XB01.dwg; H6933XN01.dwg; H6933xu01.dwg; H6933XY2.dwg and H6933Xd2–prop–req.dwg, dated May 13, 2008; S6933–300–001GA.dwg dated Nov. 28, 2008

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No. 40J6–21

HWY No	HOWARD AVENUE			DIST	LONDON
SUBM'D GD	CHECKED GD	DATE FEB. 24, 2009		SITE	6–594
DRAWN NA	CHECKED CN	APPROVED BRG		DWG	ST–2

APPENDIX A

DATA FROM PRELIMINARY INVESTIGATION BY GOLDER ASSOCIATES LTD.

PROJECT: 06-1140-156

LOCATION: SEE LOCATION PLAN

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 1

BORING DATE: AUGUST 15/ 16, 2006

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
0		PAVEMENT SURFACE	188.55
	POWER AUGER / HOLLOW STEM	ASPHALT	0.00
		Grey, crushed granular base (FILL)	0.13
1		Soft, grey, silty clay, mixed with sand and gravel (FILL)	187.79 / 0.76
2		Stiff, mottled brown and grey, SILTY CLAY, some sand, trace gravel, fissured with silt pockets (TILL)	187.03 / 1.52
3–4		Hard, brown, SILTY CLAY, some sand, trace gravel, occ. silt pockets and fissures (TILL)	186.42 / 2.13
5–9	MUD ROTARY / HW CASING; MUD ROTARY / HQ CASING	Very stiff to stiff, grey, SILTY CLAY, some sand, trace gravel (TILL)	184.28 / 4.27

— CONTINUED NEXT PAGE —

SAMPLE NUMBER	TYPE	BLOWS/0.3m
1	AS	
2	SS	2
3	SS	9
4	SS	33
5	SS	51
6	SS	40
7	SS	20
8	SS	12
9	SS	12
10	SS	10
11	SS	13

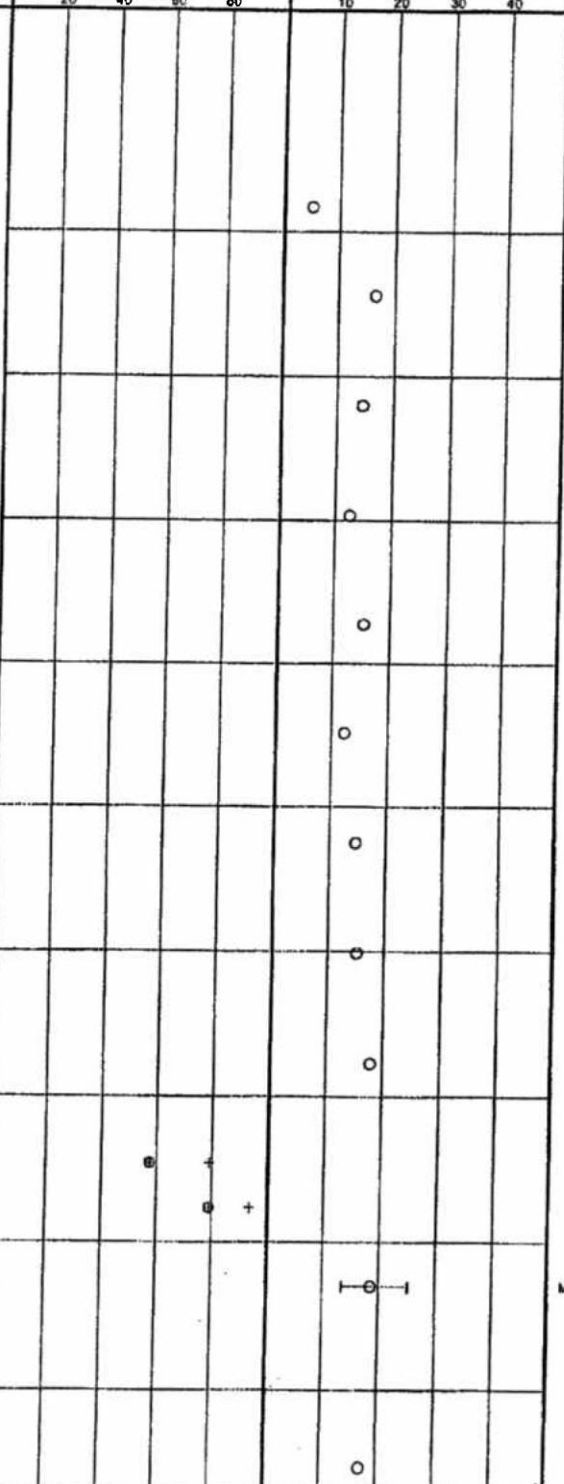
ELEVATION: 188, 187, 186, 185, 184, 183, 182, 181, 180

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 10 20 30 40



ADDITIONAL LAB TESTING: MH (sample 10)

INSTALLATION AND GROUNDWATER OBSERVATIONS:

August 15, 2006

Minor water seepage into borehole encountered at about elevation 187.48m during drilling on August 15, 2006

LDN_BHS 06-1140-156.GPJ GLDR_CAN.GDT 10/10/06 DATA INPUT: Tony Mastroianni

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: C.C.

CHECKED: [signature]

PROJECT: 06-1140-156

RECORD OF BOREHOLE 1

SHEET 2 OF 5

LOCATION: SEE LOCATION PLAN

BORING DATE: AUGUST 15/ 16, 2006

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		— CONTINUED FROM PREVIOUS PAGE —										
10	MUD ROTARY / HQ CASING	Very stiff to stiff, grey, SILTY CLAY, some sand, trace gravel (TILL)						179				
					12	SS	10	178				
11								177				
12					13	SS	10	176				
13					14	75 TO	PH	175				
14								174				
15					15	SS	8	173			MH	
16					16	75 TO	PH	172				
17								171				
18					17	SS	12	170				
19												
		— CONTINUED NEXT PAGE —										

DEPTH SCALE

1 : 50

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LDN_BHS 06-1140-156.GPJ GLDR_CAN.GDT 10/10/06 DATA INPUT: Tony Mastroianni

PROJECT: 06-1140-156

LOCATION: SEE LOCATION PLAN

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 1

BORING DATE: AUGUST 15/ 16, 2006

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)
		— CONTINUED FROM PREVIOUS PAGE —	
20	MUD ROTARY, HQ CASING	Very stiff to stiff, grey, SILTY CLAY, some sand, trace gravel (TILL)	167.67 / 20.88
21–26	MUD ROTARY, HQ CASING	Stiff to very stiff, grey, SILTY CLAY, numerous silt pockets, some sand, trace gravel (TILL)	162.11 / 26.44
27–29	MUD ROTARY, HQ CASING	Stiff to very stiff, grey, SILTY CLAY, occ. silt seams or partings (TILL)	
		— CONTINUED NEXT PAGE —	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
18	SS	11
19	SS	25
20	SS	14
21	SS	15
22	SS	13
23	SS	11
24	SS	11

ELEVATION: 169, 168, 167, 166, 165, 164, 163, 162, 161, 160

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20, 40, 60, 80

>96+

>96+

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 10, 20, 30, 40

ADDITIONAL LAB. TESTING

INSTALLATION AND GROUNDWATER OBSERVATIONS

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: C.C

CHECKED:

PROJECT: 06-1140-156

LOCATION: SEE LOCATION PLAN

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 1

BORING DATE: AUGUST 15/ 16, 2006

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE / SHEAR STRENGTH	WATER CONTENT PERCENT
		— CONTINUED FROM PREVIOUS PAGE —						
29.5	MUD ROTARY, HQ CASING	Very stiff, grey, SILTY CLAY, some sand, trace gravel, occ. sand seams (TILL)	159.05 29.50				>96+	
30.3				25	SS	22		
32.0				26	SS	21		O
33.5				27	SS	27		O
35.0				28	SS	28		O
36.5				29	SS	20		O
37.11		Very stiff, grey, SILTY CLAY, some sand, occ. gravel, numerous silt pockets or seams (TILL)	151.44 37.11					
38.0				30	SS	27		O
38.40		Limestone BEDROCK	150.15 38.40	31	NQ RC		TCR (%) 82; R.Q.D. (%) 35	
		— CONTINUED NEXT PAGE —						

DEPTH SCALE 1 : 50

Golder Associates

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CHECKED: [signature]

LDN_BHS 06-1140-156.GPJ GLDR_CAN.GDT 10/10/06 DATA INPUT: Tony Mestroianni

PROJECT: 06-1140-156

LOCATION: SEE LOCATION PLAN

SAMPLER HAMMER, 63.5kg, DROP, 760mm

RECORD OF BOREHOLE 1

BORING DATE: AUGUST 15/ 16, 2006

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ELEVATION	T.C.R. (%)	R.Q.D. (%)	HYDRAULIC CONDUCTIVITY / WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		— CONTINUED FROM PREVIOUS PAGE —											
	MUD ROTARY HQ CASING	Limestone BEDROCK			31	NQ RC			62	35			
40					32	NQ RC		149	100	0			
41					33	NQ RC		148	100	86			
		END OF BOREHOLE		147.20 41.36									
42													
43													
44													
45													
46													
47													
48													
49													

Chart headers: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80; HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40

DEPTH SCALE

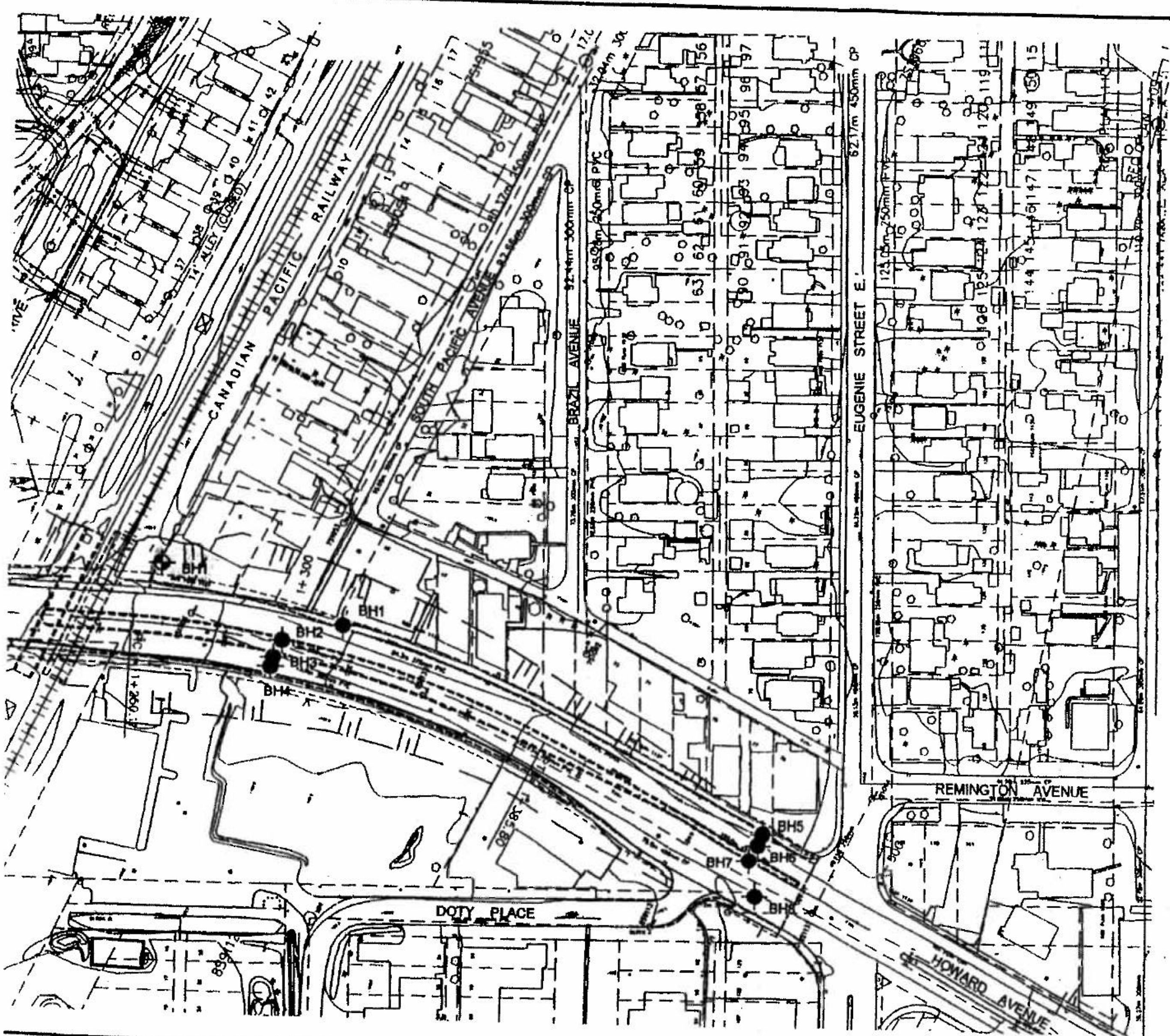
1 : 50

Golder Associates

LOGGED: C.C.

CHECKED: [signature]

LDN_BHS 06-1140-156.GPJ GLDR_CAN.GDT 10/10/06 DATA INPUT: Tony Mastroianni



LEGEND

BOREHOLE LOCATION (Current Investigation)

BOREHOLE LOCATION (Previous Investigation) Report Number 901-4047

NOTES

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

ALL LOCATIONS APPROXIMATE.

REFERENCES

CAD PLAN SUPPLIED BY: DILLON CONSULTING LIMITED
RECEIVED: September 18, 2006

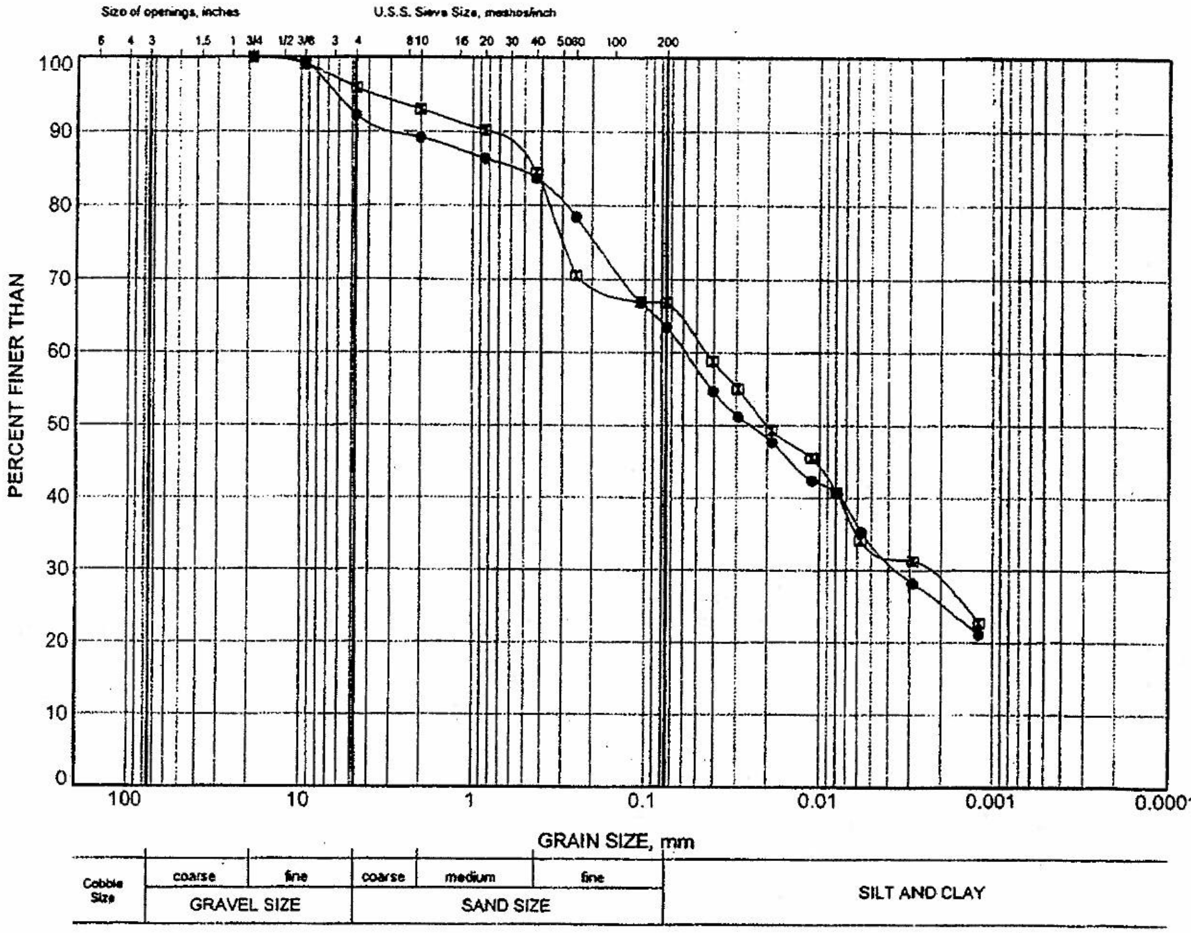
25 0 25
SCALE: 1:1250 METRES

PROJECT
DILLON CONSULTING LIMITED
HOWARD AVENUE GRADE SEPARATION AT
CANADIAN PACIFIC RAILWAY, WINDSOR, ONTARIO

TITLE
LOCATION PLAN

Golder Associates
Windsor, Ontario

PROJECT No. 06-1140-156			FILE No.
DESIGN			SCALE AS SHOWN / REV. 0
CADD	T.M.	OCT/04/06	FIGURE 2
CHECK			
REVIEW			



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	10	180.7
⊠	1	15	173.5

PROJECT

DILLON CONSULTING LIMITED
HOWARD AVENUE GRADE SEPARATION AT
CANADIAN PACIFIC RAILWAY, WINDSOR, ONTARIO

TITLE

GRAIN SIZE DISTRIBUTION

UPPER GREY SILTY CLAY (TILL)

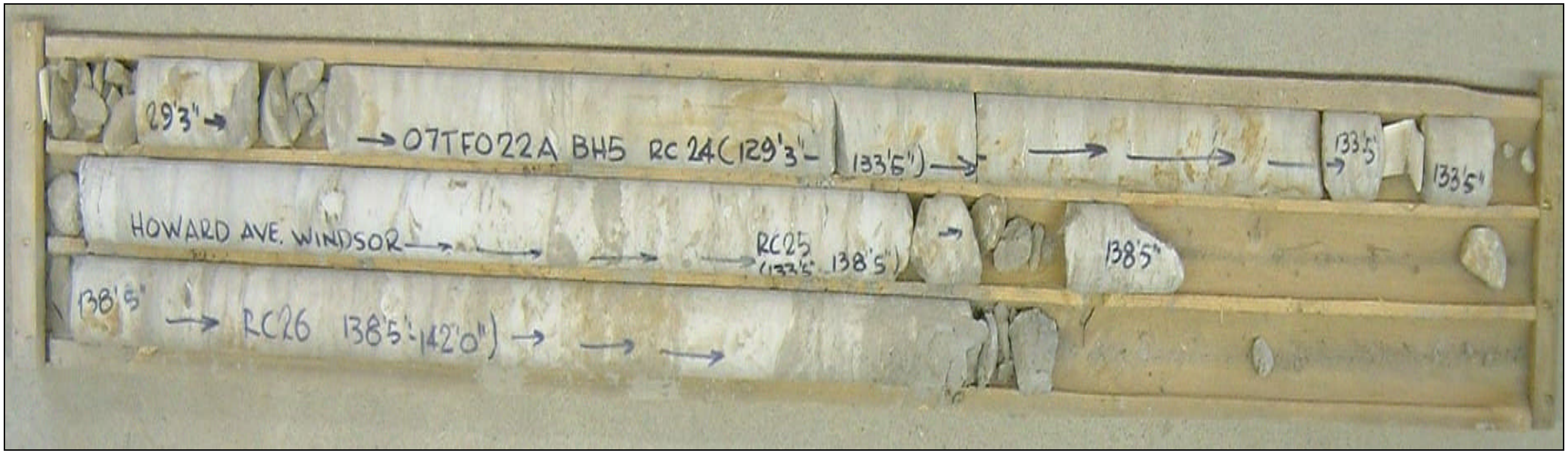
Golder Associates
WINDSOR, ONTARIO

PROJECT No.	06-1140-156		FILE No.	06-1140-156.GPJ
			SCALE	N/A
DRAWN	TM	10/10/06	REV	
CHECK		10/10/06	FIGURE	3

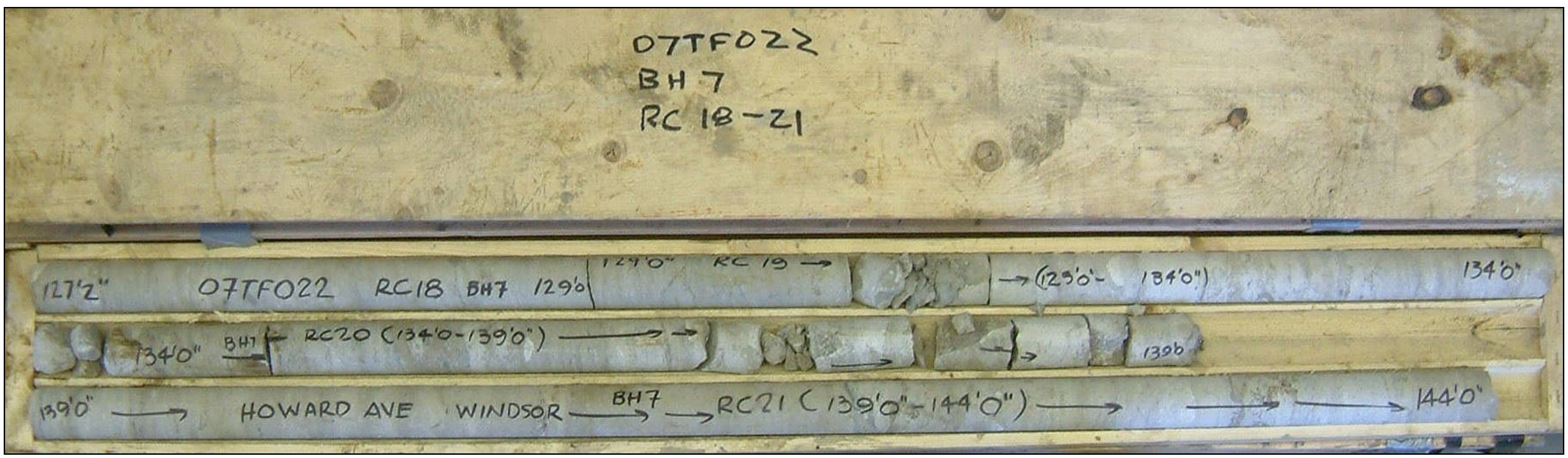
LON_GSD_NEW_GLDR_LON.GDT

APPENDIX B

ROCK CORE PHOTOGRAPHS

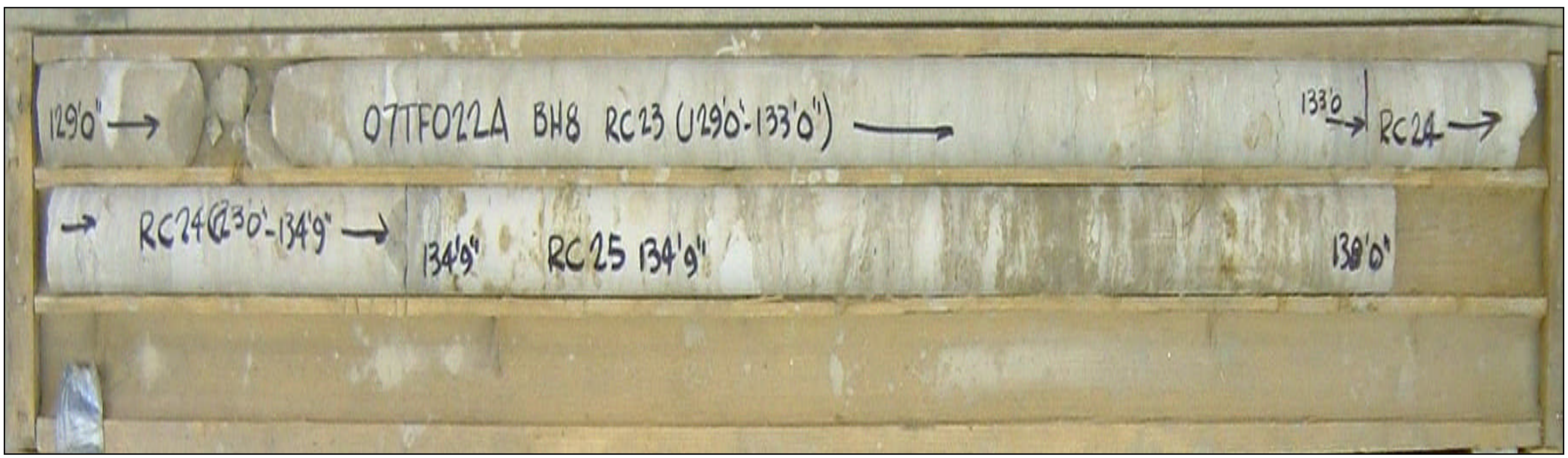


Photograph 1: Borehole 105, rock cores 24, 25 and 26



Photograph 2: Borehole 107, rock cores 18 to 21

Photograph 3: Borehole 107, rock core 22



Photograph 4: Borehole 108, rock cores 23, 24 and 25

DETAIL FOUNDATION DESIGN REPORT

for

HOWARD AVENUE OVERHEAD
SITE NO. 6-594
HOWARD AVENUE / CPR GRADE SEPARATION
GWP 3030-06-00
CITY OF WINDSOR, ONTARIO

PETO MacCALLUM LTD.
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Distribution:
5 cc: McCormick Rankin Corporation (MRC) for distribution to MTO, Project Manager – WBIIG (London) + 1 digital copy (pdf)
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PML Ref.: 07TF022A-1
Index No.: 149FDR
GEOCRES No.: 40J6-21
February 25, 2009

TABLE OF CONTENTS

DETAIL FOUNDATION DESIGN REPORT
for
Howard Avenue Overhead
Site No. 6-594
Howard Avenue / CPRGrade Separation
GWP 3030-06-00
City of Windsor, Ontario

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of foundations, abutments and approach fill embankments for the Canadian Pacific Railway (CPR) grade separation bridges at Howard Avenue in the City of Windsor, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The current plans call for the Howard Avenue alignment to be lowered and the overhead structure to carry railway traffic over Howard Avenue at approximate Station 10+298, Howard Avenue chainage. The road-rail grade separation also involves a bridge for security road traffic, a railway detour and reconstruction of the Howard Avenue / Memorial Drive intersection. It is planned to construct 25 to 80 m long wing walls extending parallel to Howard Avenue. Storm sewers between the abutments will be 1200 mm in diameter.

This report provides the recommendations for the bridge foundations and CPR diversion. Recommendations for other components of the project are provided separately.

The proposed overhead is envisaged to be a two span structure with a total length of 39.0 m (ref. ‘Howard Avenue / CPR Grade Separation: General Arrangement’ drawings prepared by MRC in November 2008). A future overhead is also planned to the south of the proposed overhead. A modular bridge is to be erected on the north side and parallel to the proposed overhead to carry light maintenance vehicles for security purposes. This bridge will be a two span structure with a length of about 36.5 m (Drawing 2 of the same GA drawings).

The top of rail at the crossing is specified to be at elevation 189.1, some 0.4 m above the existing level. The new approach embankments to the structure for the future railroad are envisaged to be less than 2 m high (interpreted from the railway grade and ground surface elevations). The road grade on Howard Avenue at the overhead location will be near elevation 182.4, about 6.7 m below the top of rail level.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial fill or topsoil underlain by an extensive deposit of clayey silt till mantling limestone bedrock. The consistency of the clayey silt till was typically very stiff in the upper 4 to 5 m thick zone and firm to stiff underneath. The bedrock surface was contacted at depths of 38.4 to 39.4 m (elevation 148.7 to 150.2). Boulders were encountered just above bedrock at 38.8 m depth (elevation 149.3) in borehole 105 put down at the west abutment.

Based on the results of the field investigation, conventional design and construction of foundations to support loadings from the overhead structure and modular bridge is considered feasible at this site.

The composition of the soil at or below approximate elevation 181 comprised firm to stiff clayey silt till. It may, subject to the foundation loads and the composition of the subgrade soil at the actual foundation locations, be feasible to support the overhead structure on spread footings. However, use of end-bearing piles driven to bedrock is considered to be the preferred foundation system from a foundation engineering perspective since the resistance of the soil is relatively low for spread footings and excessive post-construction differential settlements may occur.

The modular bridge abutments may be founded on conventional spread footings. In view of the relatively light loading, the abutments of the modular bridge may also be placed on perched footings bearing on the fill employed for the support of the RSS wall provided that the fill is constructed as a structural fill pad. The pier footings for the modular bridge should be placed beyond the zone of influence of the proposed sewers to preclude adverse effects on existing structures as per clause 6.4.2 of the Canadian Highway Bridge Design Code (CHBDC) 2006 Edition.

Wing walls constructed as RSS walls may experience differential settlements in relation to the abutments. The RSS wall areas should be preloaded to reduce settlements under the fill and facilitate sufficient subgrade strength gain to allow construction of the fill with an adequate factor of safety against slope failure.

As indicated above, the presence of storm sewers between the abutments may affect spread footings if utilised. In this case the relative locations of the piers and storm sewers should be carefully assessed.

To facilitate structural work at the site of the proposed overhead, a temporary railway diversion south of the existing CPR alignment is planned. Special measures may need to be implemented to minimise adverse impacts on the railway tracks during construction. It is anticipated that a protection system by means of shoring will be required. The lateral resistance of the shoring may be provided by way of a braced excavation support system utilising ground anchors.

It is noted that the pile driving will occur in a partially residential area and may require special measures to be implemented to limit noise and air pollution from pile driving operations. The contractor should be advised of these limitations with a Special Provision item to select the pile driving equipment accordingly.

The "red flag" issues outlined in the preceding paragraphs and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

The foundation frost penetration depth at this site is 1.2 m. The seismic site coefficient for the subsurface conditions is assessed to be 1.0 – Type I soil profile as per clause 4.4.6 of the CHBDC.

The site is located in Seismic Performance Zone 1. The liquefaction potential of the clayey soils was evaluated by consideration of the grain size distribution (percentage of particles < 0.005 mm in size), liquid limit values and the ratio of the water content to the liquid limit. Based on the research by Marcuson et al (1990), we believe that liquefaction of the fine grained soils (more than 35% of the soil particles passing the No. 200 sieve) is unlikely. The liquefaction potential of the granular soils assessed using the procedure suggested by Seed and Idriss (1971) is considered unlikely as well (clause 4.6.2 of CHBDC).

The standard specifications referenced in this report are listed in Table 1.

2. FOUNDATIONS

2.1 Piles

A driven pile system consisting of steel H-piles is considered to be an appropriate means of supporting the centre pier and both abutments of the overhead structure. Cognisant of the depth to a competent bearing stratum, construction of integral abutments supported on end-bearing piles founded on bedrock may be considered at this site if the railway horizontal loads are not excessive.

The H-piles driven to refusal on bedrock anticipated at elevation 148.7 to 150.2 should be designed using the following axial resistance at ultimate limit states (ULS) for three pile sections:

	Factored Axial Resistance at ULS, kN
HP 310 x 110	2000
HP 310 x 132	2400
HP 310 x 174	3200

Boulders were identified in the clayey silt till just above the bedrock surface in one borehole. It is considered that damage to the piles when driving into the clayey silt till is unlikely, although boulders were encountered locally in borehole 105. As a consequence, application of a reduction factor to account for potential damage during driving is not necessary. The above pile sections are only recommended due to the pile length and the fact that heavier sections are less likely to be damaged by boulders during installation.

The resistance at serviceability limit states (SLS) normally allows for 25 mm compression of the pile and founding medium. Considering the bedrock to be a non-yielding material and the pile length of about 32 m, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm axial deformation of the pile is larger than the factored resistance at ULS.

Any fill placed below grade for a working platform to drive the piles should comprise Granular A or Granular B Type II or Type III with a maximum 75 mm nominal size to enable driving of the piles and minimise the potential for damage during pile installation.

The soil adjacent to the upper portion of the piles is expected to comprise firm to stiff clayey silt till. To accommodate movement of the integral abutment, <u>if utilised</u>, it is recommended that two concentric CSPs that extend at least 3 m below the bottom of the abutment be placed around the pile to create an annular space. The inner CSP of 600 mm diameter should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements shown in Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

The piles likely to be installed from open cut excavation due to low headroom under the existing hydro cables will be about 32 m long and driven through an extensive deposit of clayey silt till with little evidence of cobbles or boulders. Therefore, it is considered, based on our extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed.

It is noted that the pile driving in residential areas may require limitations on noise and emissions of air pollution from diesel pile driving hammers. The contractor should be advised of these limitations with a Special Provision item to select the driving equipment accordingly.

The H-piles should be equipped with driving shoes as per OPSD 3000.100 or the Titus ‘H’ Bearing Pile Points, Standard model, in accordance with SP 903S01 because the piles should be driven to refusal on bedrock.

The piles should be installed and monitored in accordance with the requirements of SP 903S01. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices, and should be performed on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Resistance to lateral loads may be provided in part by mobilisation of passive resistance along the pile. The following lateral resistance for the three pile sections driven through the stiff clayey silt till is recommended:

Factored Lateral Resistance at ULS	=	160 kN
Lateral Resistance at SLS	=	80 kN

If greater resistance is required, batter piles should be installed.

The coefficient of horizontal subgrade reaction, k_s, for the granular backfill and the underlying cohesive deposit present at the site should be computed using the following equations to evaluate the point of contraflexture:

Cohesionless (embankment fill):

k_s = $n_h\, z/b$
n_h = coefficient related to soil density
= 12 MN/m^3 for granular fill
z = depth, m
b = pile width, m

Cohesive (clayey silt till):

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

c_u = undrained shear strength of cohesive material
= 60 kPa for stiff clayey silt till
b = pile width, m

2.2 Spread Footings

Supporting the abutment and pier foundation loads of the proposed modular bridge structure on conventional spread footings placed in the clayey silt till may be considered at the site. Footings founded in the typically very stiff clayey silt till revealed at or below approximate elevation 182 should be designed using the following resistance values (maximum 2.5 m wide footings):

Factored Geotechnical Resistance at ULS, kPa	= 250
Geotechnical Resistance at SLS, kPa	= 150

In view of the relatively light loading, the abutments of the proposed modular bridge may be placed on perched footings bearing on the fill employed for the support of the RSS walls provided that this fill is constructed as a structural fill pad.

The engineered fill should comprise Granular A material placed in maximum 200 mm thick lifts, compacted to 100% standard Proctor maximum dry density, and extended laterally to a line originating at least 1 m from the top edge of footing and inclined downward at 45° to the horizontal. The material should be placed and compacted in accordance with OPSS 501 (Method A). This scheme is illustrated in the attached Figure 1.

The RSS wall areas should be preloaded to reduce settlements under the fill and facilitate sufficient subgrade strength gain to allow construction of the fill with an adequate factor of safety against slope failure. The interior facing of the RSS wall panels should be lined with a layer of extruded polystyrene at least 75 mm thick to provide adequate frost protection.

The bearing resistance for a minimum 2.5 m wide footing constructed on a minimum 3.5 m thick pad of structural fill is as follows:

Factored Geotechnical Resistance at ULS	=	900 kPa
Geotechnical Resistance at SLS	=	350 kPa

The resistance at SLS normally allows for 25 mm of compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.7 is recommended for footings on granular fill.

All footings subject to frost action should be provided with at least 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Prior to placement of structural concrete, all foundation excavations should be examined by qualified geotechnical personnel to verify the competency of the founding surface. The procedures for excavation and backfilling of structures specified in SP 902S01 should be followed.

3. ABUTMENT WALLS

The abutment walls and wing walls should be designed to resist the unbalanced horizontal earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p (kPa), may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:

$$p = K(\gamma h + q) + Cp + Cs$$

where K = coefficient of lateral earth pressure (dimensionless)
γ = unit weight of free-draining granular material, kN/m^3
h = depth below final grade, m
q = surcharge load, kPa, if present
Cp = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where Ø = angle of internal friction of retained soil (35^o for Granular A or Granular B Type II or Type III)
δ = angle of friction between the soil and wall (23.5^o for Granular A or Granular B Type II or Type III)

Free-draining granular material should be used as backfill behind the wall. The following parameters are recommended for design:

PARAMETERS	GRANULAR A or GRANULAR B TYPE II or TYPE III
Internal Friction Angle, Ø (degrees)	35
Unit weight, γ (kN/m^3)	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient of Passive Earth Pressure, K_p	3.69

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments if utilised. The coefficient of earth pressure at rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. Refer to Figure C6.16 of the CHBDC for this computation.

A subdrain system (SP 405F03) should be installed to minimise the build-up of hydrostatic pressure behind the wall. The subdrain tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to the structure should be performed in conformance to Ontario Provincial Standards specifications for granular backfill at abutments (OPSD 3101.150). As noted earlier, Granular A should be employed within the limits of driven piles.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to OPSS 501 and SP 105S10 for additional information in this regard.

3.1 RSS Walls

A retained soil system (RSS) could also be constructed at this site. A high performance, high appearance rated RSS wall should be employed.

The founding material is expected to comprise granular engineered fill or firm to very stiff clayey silt till. The geotechnical parameters employed to design the RSS will be dependent upon the type of backfill required for internal stability of the proprietary system as well as the soil contiguous to the RSS system that will govern global stability, overturning and/or sliding of the base.

The earth pressure coefficients provided above for granular materials and those given for the clayey silt till in Section 6 "Railway Diversion Considerations" are appropriate for the RSS wall. The bearing resistance and foundation levels recommended for footings founded on structural fill or on the clayey silt till is considered to be suitable for the RSS.

The horizontal force at the base of the RSS will be resisted in part by the friction force developed through the granular backfill or along the interface between the granular backfill and the founding soil, subject to site specific design details. Unfactored friction factors of 0.7 and 0.5 are considered to be appropriate for the granular backfill and at the granular/soil interface, respectively. The global stability should be assessed using the geotechnical parameters noted above. A design groundwater level of 500 mm above the weeping tile outlet should be used.

The RSS supplier should be responsible for specifying the type of backfill material employed, taking into consideration the engineering properties of the proprietary product, the design life of the structure, the pullout resistance required, drainage requirements and settlements of the approach embankments.

The requirements for design and construction of the RSS wall specified in SP 599S22 and SP 599S23 as well as MTO RSS Design Guidelines dated September 2008 should be followed. The supplier of the RSS should also be responsible for the detail design of the structure (backfill, reinforcement, internal and external stability) and for providing drawings to show pertinent information such as location, length, height, elevations, performance level, appearance, etc.

4. APPROACH EMBANKMENTS

It is anticipated that the existing approach embankments for the railroad and security road will be re-instated to the present levels. The new embankment for the future railway tracks will be constructed with earth borrow or granular material (at the abutments) and ballast. The design calls for the embankment to be about 1m high at both abutments. The subgrade revealed in the boreholes drilled at the site consists of typically stiff clayey silt till.

Any topsoil and other deleterious material at the abutment locations and along the alignment of the approach fill should be stripped prior to placement of the embankment fill on the inorganic native soils.

In general, the embankments should be constructed following conventional MTO procedures (OPSD 200.010, 201.010, 202.010 and SP 206S03) or the applicable CPR procedures for railway ballast and rail track construction. These shallow embankment slopes inclined no steeper than 2 horizontal to 1 vertical should be stable.

Maximum settlements of the approach fill as a result of the consolidation of the underlying cohesive deposit induced by the low embankment loads are estimated to be less than 25 mm. No bearing capacity problems are anticipated.

The backfill to the abutments should comprise Granular A or Granular B Type II material placed in conformance to the requirements of SP 206S03 and OPSS 501 to minimise post-construction settlements.

5. EXCAVATION AND GROUNDWATER CONTROL

5.1 Excavation

Construction of deep foundations is considered to necessitate open cut excavation allowing for piles to be driven from the excavated level due to low headroom under the existing hydro cables.

Excavation for installation of piles and construction of pile caps or spread footings is expected to extend through the fill and into the clayey silt till to a depth of up to 8 m from existing grades.

The fill and firm to stiff clayey silt till are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes inclined at 45° to the horizontal should generally be employed. Flatter side slopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction.

5.2 Permanent Cut Slopes

Permanent cut slopes will be required for this project. The stable slope angle in the long term has been assessed using effective stress parameters. The stability analyses carried out for 8 m high slopes indicate that an adequate factor of safety of 1.5 is achieved in case of permanent slopes cut at an inclination of 3 horizontal to 1 vertical (3H:1V). Taking into account the slope height, a horizontal bench located mid-way from the crest of slope may be considered. It is noted that ensuring the acceptable stability of the benched slope will necessitate lowering of the groundwater by means of a drain installed beneath the back of the benched area and extending below the toe of slope.

Shallow cuts within the upper zone of the very stiff clayey silt till are expected to be stable at steeper slopes. Thus, a 4 m high slope could be cut at an inclination of 2H:1V with a safety factor of 1.5.

5.3 Groundwater Control

Perched water was detected in the process of augering at 1.1 m depth (elevation 187.5) in borehole 1 and at a depth of 0.4 m (elevation 187.7) in borehole 105. Groundwater was not observed in any of the boreholes upon completion of drilling. The water level in a piezometer installed in borehole 107 in October 2008 rose by 2.3 m to elevation 170.6 during an 8-day period. It is noteworthy that groundwater levels are subject to seasonal fluctuations and precipitation patterns.

Considering the relatively impervious nature of the native clayey silt till revealed in the boreholes at this site, groundwater seepage or surface water that enters the excavations for construction of the abutments and centre pier should be readily handled by conventional sump pumping techniques. It is noted, however, that the excavation extending into the silty sand till encountered at elevation 182.2 in borehole 107 may require more positive groundwater control measures such as a well-point system, sheet piling or equivalent.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

6. RAILWAY DIVERSION CONSIDERATIONS

To facilitate structural work at the site of the proposed overhead, a temporary railway diversion south of the existing CPR alignment is planned. Special measures may need to be implemented to minimise adverse impacts on the railway tracks during construction. It is anticipated that a track protection system including shoring will be required to support the walls of the excavation and adjacent CPR tracks.

The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence. The recommended design earth pressure distribution for singly and multi-braced walls, for the conditions that exist at the site, are presented in Figures 2 and 3 respectively. Recommendations concerning design and construction of the braced excavation support systems are provided in the figures.

The geotechnical parameters to be used in designing the wall are given in the following table. It is noted that the values for the very stiff clayey silt till are relevant to design of deadman anchors if employed within the upper 4 to 5 m thick zone of the native soils.

PARAMETERS	**CLAYEY SILT TILL**	
	Firm to Stiff	**Very Stiff**
Angle of Internal Friction, degrees	28	32
Unit Weight, kN/m^3	19.0	20.0
Coefficient of Active Earth Pressure K_a	0.36	0.31
Coefficient of Earth Pressure At-Rest K_o	0.53	0.47
Coefficient of Passive Earth Pressure K_p	2.77	3.25

Additional lateral resistance could be provided by installing tiebacks anchored in the stiff clayey silt till. The unfactored pull-out resistance (R) of anchors grouted in cohesive material can be estimated using the following equation:

$$R = \alpha \, A_s \, L_s \, c_{\underline{u}}$$

where α = reduction factor related to undrained shear strength
= 0.6 for clayey silt till

A_s = circumference of cross-section of fixed length of anchor, m^2/m

L_s = fixed length of anchor, m

C_u = average undrained shear strength of cohesive soil over fixed length of anchor, kPa
= 60 kPa

A resistance factor of 0.4 should be applied to the computed anchor capacity to determine the ULS resistance. Reference is made to SP 999S26 for design, installation and testing of ground anchors.

The ground surface adjacent to the excavation is expected to experience some inward movement and vertical settlement. The magnitude of movements adjacent to a braced cut can be limited by selection of an appropriate lateral earth pressure coefficient provided good quality workmanship and construction practice is employed. The anticipated magnitude of movements is as follows.

	Movement (% of Excavation Depth)
Lateral Movement	
Braced Excavation	0.2
Anchored Wall	0.1
Vertical Movement	0.05

Construction procedures should be specifically suited to limit any consequent settlement of the subgrade behind the excavation face.

The proposed railway diversion should be implemented in accordance with AREMA Manual for Railway Engineering.

7. CLOSURE

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

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TABLE 1

STANDARD SPECIFICATIONS REFERENCED IN REPORT

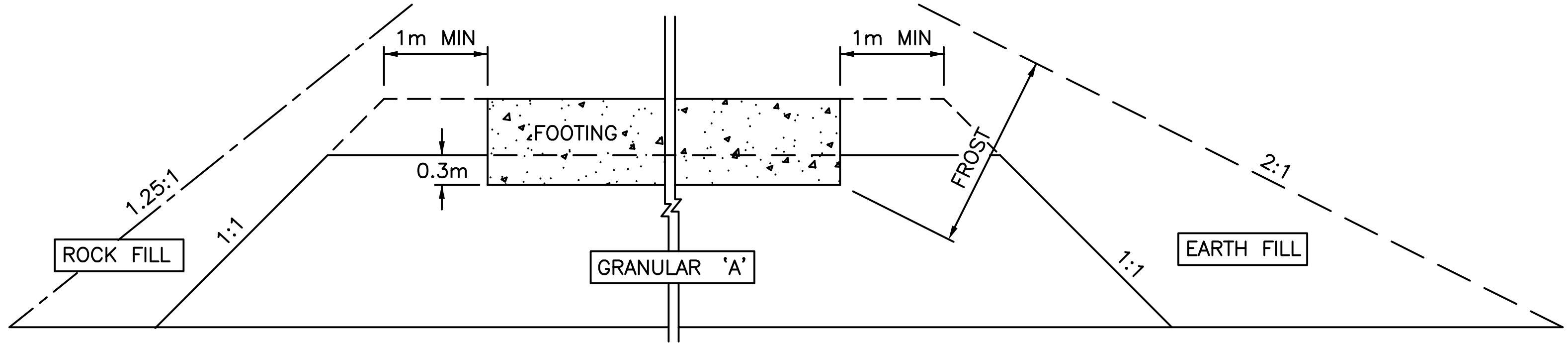
DOCUMENT	TITLE
OPSS 501	Construction Specification for Compacting
SP 105S10	Construction Specification for Compaction
SP 109F10	Structural Reference Plans and Reports
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)
SP 599S23	Requirements for Materials, Quality Control and Quality Assurance Testing and Acceptance Criteria for Precast Concrete Facing Elements Including Panels
SP 902S01	Excavation and Backfilling of Structures
SP 903S01	Construction Specification for Piling
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock
OPSD 200.010	Earth/Shale Grading – Undivided Rural
OPSD 201.010	Rock Grading-Undivided Rural
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD-3101.150	Minimum Granular Backfill Requirements - Walls Abutment

TABLE 2

GRADATION SPECIFICATIONS FOR SAND FILL IN PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

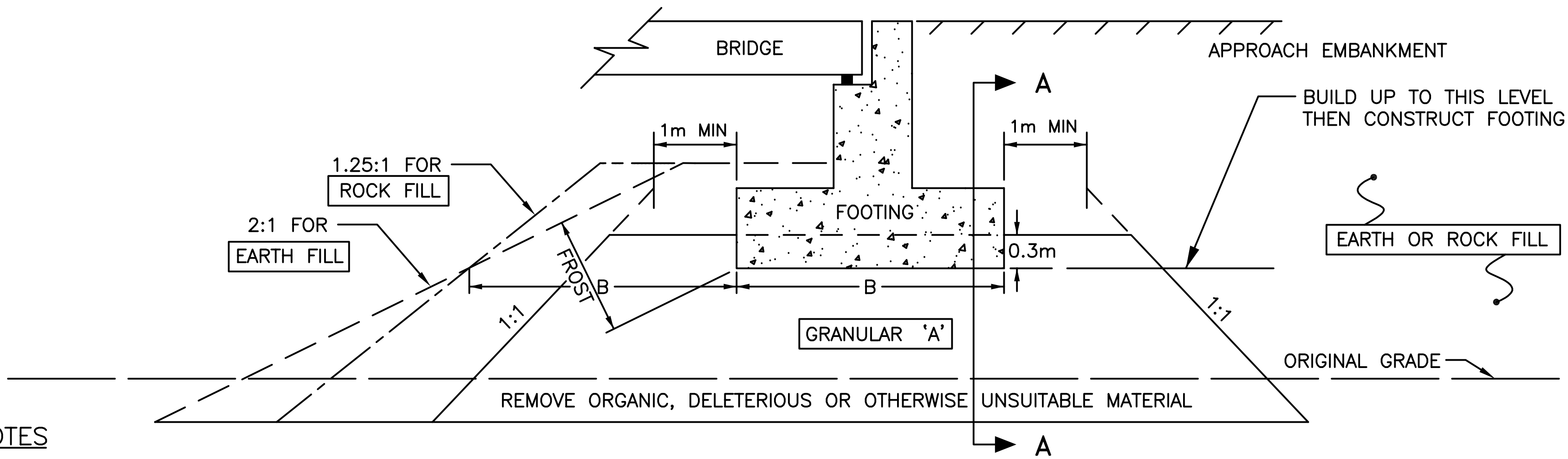
MTO Sieve Designation	Percentage Passing by Mass
2 mm (#10)	100
600 μm (#30)	80 – 100
425 μm (#40)	40 – 80
250 μm (#60)	5 – 25
150 μm (#100)	0 – 6

Note: From MTO Report S0-96-01, Revision 1 – July, 1996.



CROSS SECTION A–A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

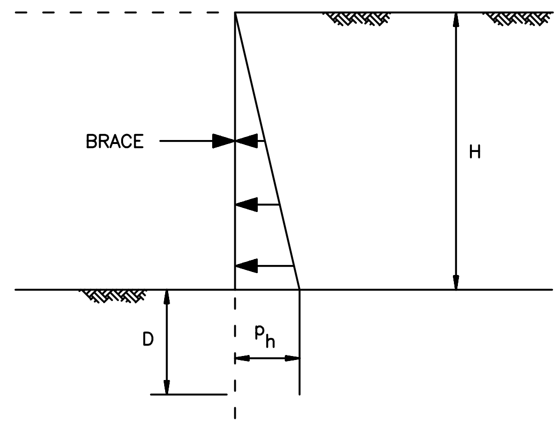
1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and apppropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM



p_h = design lateral earth pressure
= $K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

γ = 19.0 kN/m^3

K = 0.36

LATERAL EARTH PRESSURE DISTRIBUTION

SINGLY BRACED CUTS IN STIFF CLAYS

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN: N.A.	DATE	SCALE	JOB NO.	FIGURE NO.
CHECKED: G.D.	FEB. 2009	N.T.S.	07TF022A	2
APPROVED: C.N.				

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
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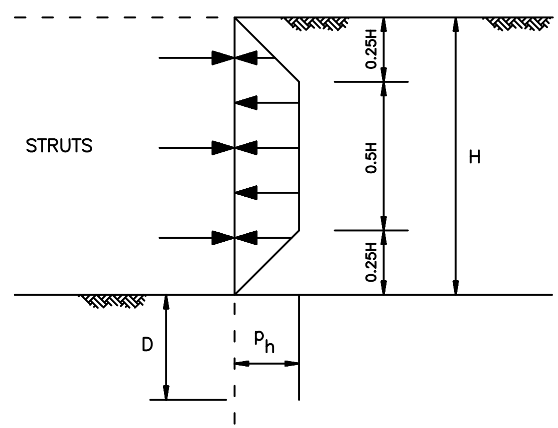

p_h = design lateral earth pressure
= $0.4\,\gamma H$

where

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

$\gamma = 19.0\ \ \mathrm{kN/m^3}$

LATERAL EARTH PRESSURE DISTRIBUTION

MULTI-BRACED CUTS IN STIFF CLAYS

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DRAWN: N.A.	DATE	SCALE	JOB NO.	FIGURE NO.
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APPROVED:				