Peto MacCallum Ltd.
CONSULTING ENGINEERS

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for
LAUZON PARKWAY UNDERPASS AT HIGHWAY 401
LAUZON PARKWAY EXTENSION
FROM EC ROW EXPRESSWAY TO HIGHWAY 3
TOWNSHIP OF TECUMSEH AND CITY OF WINDSOR
WINDSOR AREA, ONTARIO
G.W.P. 3017-09-00

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PML Ref.: 11TF002A
Index No.: 040FIR and 041FDR
GEOCRES No.: 40J2-130
December 4, 2013

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TABLE OF CONTENTS

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Lauzon Parkway Underpass at Highway 401
Lauzon Parkway Extension, From EC Row Expressway to Highway 3
Township of Tecumseh and City of Windsor
Windsor Area, Ontario
GWP 3017-09-00

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed Lauzon Parkway Underpass at Highway 401. The proposed underpass is part of the preliminary design and environmental assessment study to upgrade and realign the Lauzon Parkway from the EC Row Expressway to County Road 42 and its extension to Highway 3, a length of approximately 9 km. The study was carried out by Peto MacCallum Ltd. (PML) for MMM Group Limited (MMM) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed Lauzon Parkway Underpass over Highway 401 will be constructed as part of the Lauzon Parkway Extension to the east of Concession Road 9 and west of County Road 17 between Station 16+454 and 16+520, Lauzon Parkway chainage (refer to the Preliminary General Arrangement Drawing, Lauzon Parkway Highway 401 Underpass dated May 2013).

The purpose of this report was to summarize the subsurface stratigraphy encountered at the proposed structure site during the preliminary investigation.

2. SITE DESCRIPTION AND GEOLOGY

The proposed structure for the future Lauzon Parkway crossing at Highway 401 is located about 750 m east of the existing Concession Road 9 and Highway 401 underpass. The portion of the structure north of Highway 401 is located within the City of Windsor and the portion of the structure south of Highway 401 is located in the Township of Tecumseh, Essex County.

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BARRIE, HAMILTON, KITCHENER, TORONTO

Land use in the vicinity of the site includes the existing Highway 401 transportation corridor and farming activity to the north and south of the corridor. The local topography is generally flat to the north and south of Highway 401. The ground cover includes grasses in the Highway 401 right of way with bushes and stands of trees scattered throughout, particularly at north side of the structure where the Highway 401 right of way abuts the nearby farmland.

The project site is located in the Essex Clay Plain within the physiographic region known as the St. Clair Clay Plains. The soil cover at the project site is represented mainly by cohesive glacial till deposit over a sand and silt layer, mantling bedrock. The typical rock type in the project area is Middle Devonian limestone of the Paleozic era. The bedrock is at depths of more than 35 m at the site.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out during the period of June 19 to 21, 25 and 26, 2013. Two boreholes (LP-1 and LP-2) were drilled to 37.0 and 41.3 m at the locations shown on Drawing LP-1, appended.

The borehole locations were selected based on the General Arrangement (GA) drawings prepared by MMM dated May, 2013. The borehole locations and elevations were surveyed in the field by MMM. All elevations in this report are expressed in metres.

The boreholes were advanced using continuous flight hollow stem augers and ‘N’ casing through the soil cover with a track-mounted D-50 turbo drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field supervisor. One of the boreholes LP-1 was extended 3.4 m into bedrock using NQ diamond rock coring equipment supplemented by wash boring techniques.

Soil samples were recovered from the boreholes at regular 0.75, 1.5 or 3.0 m depth intervals using the standard penetration test method. Standard penetration tests and field vane tests were conducted to assess the strength characteristics of the substrata. Pocket penetrometer tests and field vane tests were carried out in the clayey soils to obtain representative test results on the in-situ shear strength. Pocket penetrometer testing was only considered within the very stiff upper soils where the capacity of the MTO field vane was exceeded. Soils were identified in accordance with the MTO soil classification manual procedures.

The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved. Groundwater level in the open boreholes following drilling could not be obtained as the boreholes were charged with water during drilling.

The boreholes were backfilled with a bentonite/grout mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment.

The recovered soil samples were returned to our laboratory in Toronto for detailed visual examination, laboratory testing and classification. The laboratory testing program included the following tests:

- Natural moisture content determinations (37)
- Atterberg Limits (7)
- Grain size distribution analyses (9)

The laboratory grain size distribution charts are presented in Figures LP-GS-1 to LP-GS-3 and Atterberg Limits results are presented in Figures LP-PC-1 and LP-PC-2. All of the test results are summarized on the Record of Borehole sheets.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, bedrock description, inferred stratigraphy, standard penetration in-situ vane, and penetrometer test results and groundwater observations. The results of laboratory particle size distributions, Atterberg limits and moisture content determinations are also shown on the Record of Borehole Sheets. The results of the Atterberg limits testing are listed in Table A.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawings LP-1. 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 topsoil overlying a cohesive clayey silt till underlain by a silty clay, over a cohesionless sand and silt till which extended to bedrock. The bedrock / probable bedrock surface was contacted at 37.9 and 37.0 m (elevation 148.1 and 149.2) in boreholes LP-2 and LP-2, respectively.

A summary of the findings is given below.

4.1 Topsoil

A 0.5 m thick topsoil layer was encountered surficially in boreholes LP-1 and LP-2 that extended to respective elevations of 185.5 and 185.7.

4.2 Clayey Silt Till (CL)

A clayey silt till deposit was encountered below the topsoil at 0.5 m (elevation 185.5 and 185.7) in boreholes LP-1 and LP-2 that extended to 20.5 and 31.7 m (elevation 165.5 and 154.5), respectively. In borehole LP-2 the clayey silt till was interbedded by a 6.0 m thick silty clay unit at

20.5 m (elevation 165.7) that extended to 26.5 m (elevation 159.7). The deposit was typically very stiff in the 3.4 and 4.0 m thick upper zones (locally hard in the upper zone of borehole LP-2), becoming stiff with depth. The deposit had SPT-'N' values ranging between 16 and 31 in the upper zones. In the remainder of the deposit had SPT-'N' values typically ranging between 7 and 25 (locally an SPT-'N' value of 57 was recorded due to a stone in the spoon tip) and shear strengths of 87 to 188 kPa were indicated by penetrometer and field vane testing. Although no cobbles are boulders were noted during drilling, the possibility of cobbles or boulders existing within the deposit should not be discounted.

The results of 7 grain size distribution analyses and 6 Atterberg Limit tests conducted on samples of the deposit are shown in respective figures LP-GS-1 and LP-PC-1. The liquid limit of clayey silt till samples ranged between 19 and 29 and the plastic limit varied from 12 to 15 with plasticity index values of 7 to 14. The moisture content determinations within the clayey silt till ranged between 9 to 22%.

4.3 Silty Clay (CI)

A 6.0 and 6.3 m thick silty clay deposit was encountered within / below the clayey silt till deposit at 20.5 m (elevation 165.5 and 165.7) in borehole LP-1 and LP-2. The silty clay extended to the sand and silt till at 26.8m (elevation 159.2) in borehole LP-1 and extended to the clayey silt till at 26.5 m (elevation 159.7) in borehole LP-2. The deposit was stiff to very stiff with SPT-'N' values of 6 to 9, and shear strengths of 72 to greater than 100 kPa indicated by field vane testing.

The results of a grain size distribution analysis and Atterberg Limit test conducted on a sample of the deposit are shown in respective figures LP-GS-2 and LP-PC-2. The Atterberg testing indicated a liquid limit 40, a plastic limit of 20 and a plasticity index value of 20. Moisture contents of 15 to 31% were recorded within the silty clay.

4.4 Sand and Silt Till

A 5.3 and 11.1 m thick sand and silt till deposit was encountered below the clayey silt till at 31.7 m and silty clay at 26.8 m (elevation 154.5 and 159.2) in boreholes LP-2 and LP-1, respectively. The sand and silt extended to bedrock at 37.9 (elevation 148.1) and probable bedrock at 37.0 m (elevation 149.2) in boreholes LP-1 and LP-2 respectively. The deposit was dense to very dense (SPT-'N' values of 34 to 72 blows for 18 cm) and had moisture contents of 10 and 24%. Although no cobbles are boulders were noted during drilling the possibility of cobbles or boulders existing within the deposit should not be discounted. The results of a grain size distribution analysis for a sample of the sand and silt are presented in Figure LP-GS-3.

4.5 Bedrock

Limestone bedrock was contacted at 37.9 m (elevation 148.1) at the north abutment (borehole LP-1) and inferred at 37.0 m (elevation 149.2) at the south abutment.

The bedrock was cored 3.4 m to a 41.3 m depth, elevation 144.7 in borehole LP-1. The measured core recovery and RQD from the recovered rock cores were both 100%, thus indicating excellent quality rock.

A detailed description of the rock core retrieved from borehole LP-1 is given in Table B, appended. A photograph of the rock core is also shown in Appendix A.

4.6 Groundwater

During augering, groundwater was observed at 13.7 and 14.8 m (elevation 172.3 and 171.4) in boreholes LP-1 and LP-2, respectively. Following the observation of groundwater during drilling, the boreholes were charged with drilling water for wash boring and rock coring and the water levels were not considered representative of the groundwater. The groundwater level is subject to seasonal fluctuation and rainfall patterns.

5. CLOSURE

Mr. A. Lo carried out the field investigation for this study under the supervision of Mr. A. DeSira, MEng, and Mr. C. M. P. Nascimento, P. Eng., Project Manager. London Soil Drilling supplied the drill rig for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This Foundation Investigation Report was prepared by Mr. A. DeSira, MEng, P.Eng., and reviewed by Mr. C. M. P. Nascimento, P. Eng., Project Manager. 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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AD/CN/BRG:ad-nk

TABLE A

ATTERBERG LIMITS AND MOISTURE CONTENT TEST RESULTS

SOIL TYPE	BOREHOLE No.	SAMPLE No.	MOISTURE CONTENT (%)	LIQUID LIMIT (W_L)	PLASTIC LIMIT (W_P)	PLASTICITY INDEX (PI)
Clayey Silt Till (CL)	LP-1	5	16	29	15	14
		9	19	27	14	13
	LP-2	6	17	28	15	13
		11	19	27	14	13
		15	22	29	15	14
		19	14	19	12	7
Silty Clay (CI)	LP-1	16	32	40	20	20

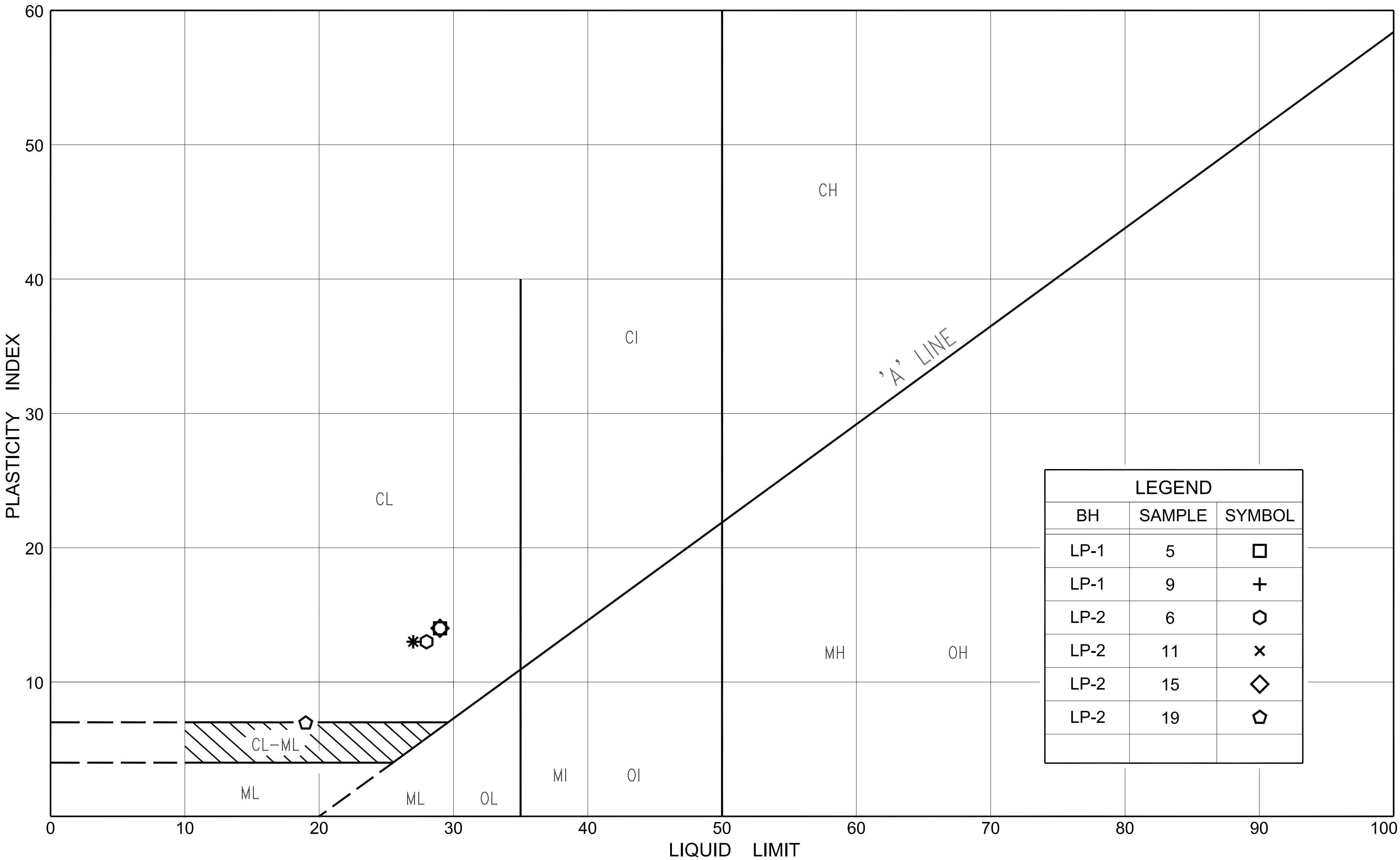
TABLE B

ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
LP-1	1	37.9(1) – 39.1	100	100	37.9 – 41.3	LIMESTONE: Light grey, fine crystalline to aphanitic, with few styolitic partings, small chert nodules, occasional fossils, high strength, unweathered, close to moderately spaced flat partings, rough planar, tight, excellent quality
	2	39.1 – 40.6	100	100		
	3	40.6 – 41.3	100	100		

Notes:
Drilled: June 19 to 21, 2013
Logged: July 18, 2013
RQD = Rock Quality Designation
37.9(1): Bedrock starts at 37.9 m

Originated: JO/SAT
Compiled: AL
Checked: AD/CN



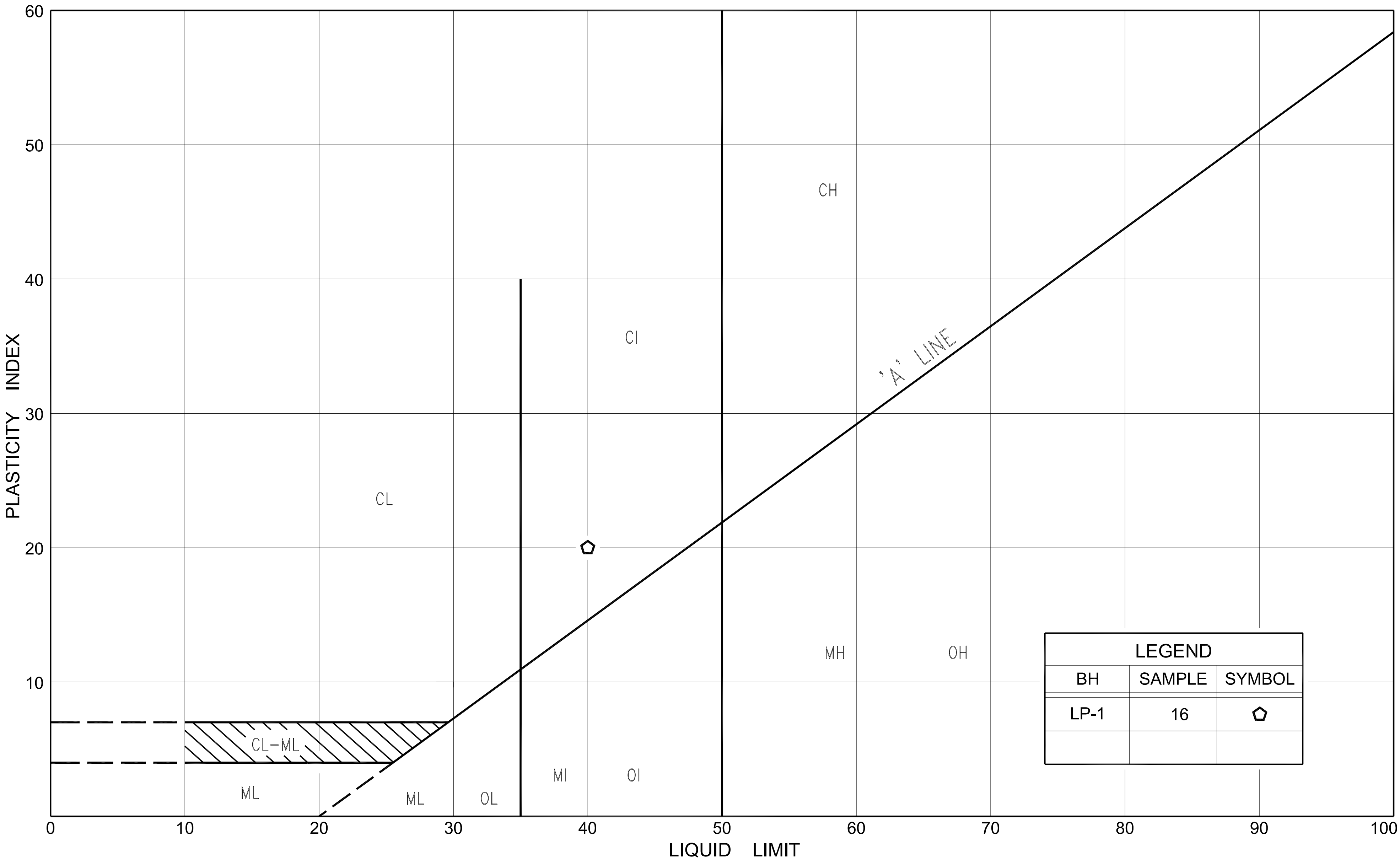
LEGEND		
BH	SAMPLE	SYMBOL
LP-1	5	□
LP-1	9	+
LP-2	6	⬡
LP-2	11	×
LP-2	15	◇
LP-2	19	⬠

Ontario

PLASTICITY CHART

CLAYEY SILT, with sand to sandy, trace gravel (CL)
(TILL)

FIG No.	LP-PC-1
HWY:	401
G.W.P. No.	3017-09-00

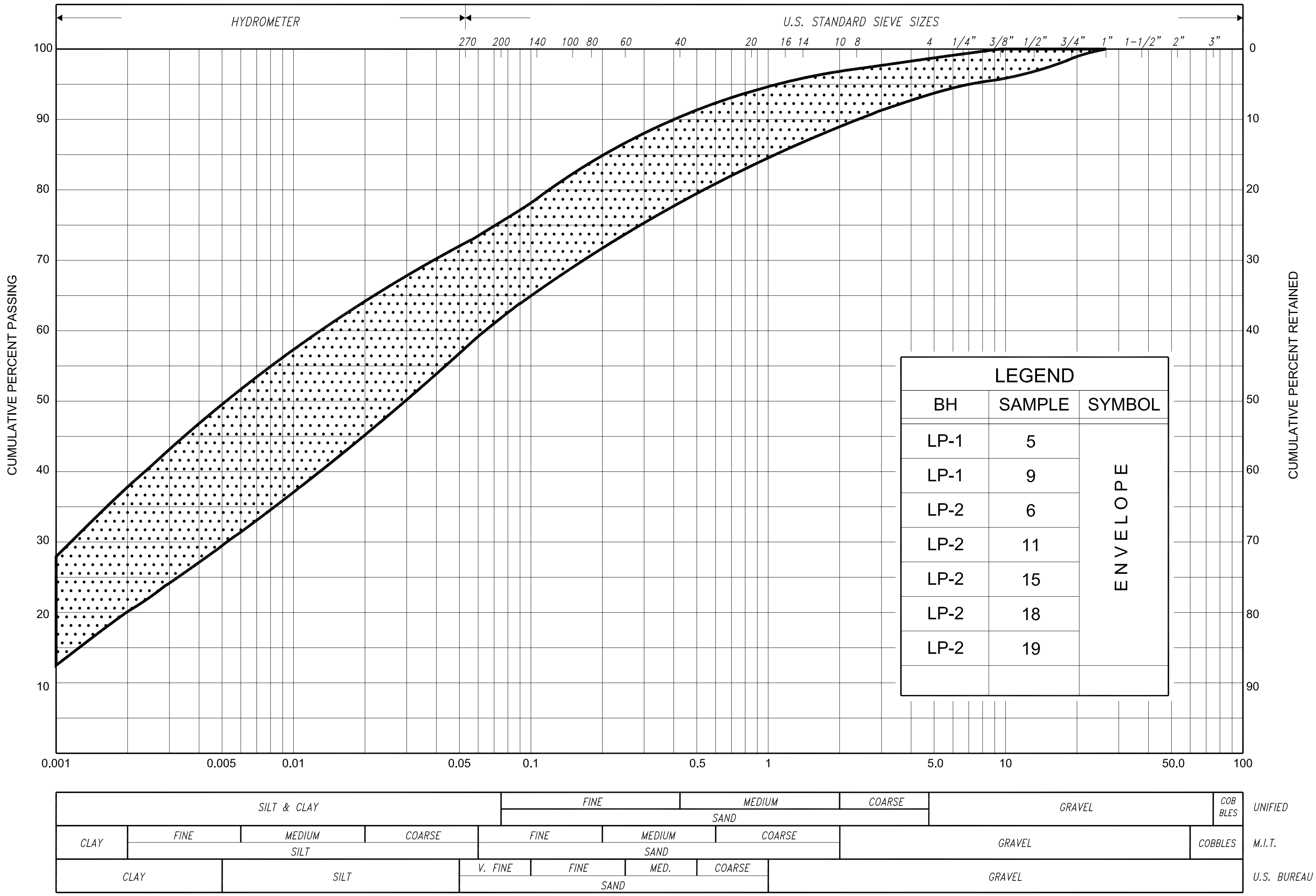


Ontario

PLASTICITY CHART

SILTY CLAY, trace sand (CI)

FIG No.	LP-PC-2
HWY:	401
G.W.P. No.	3017-09-00

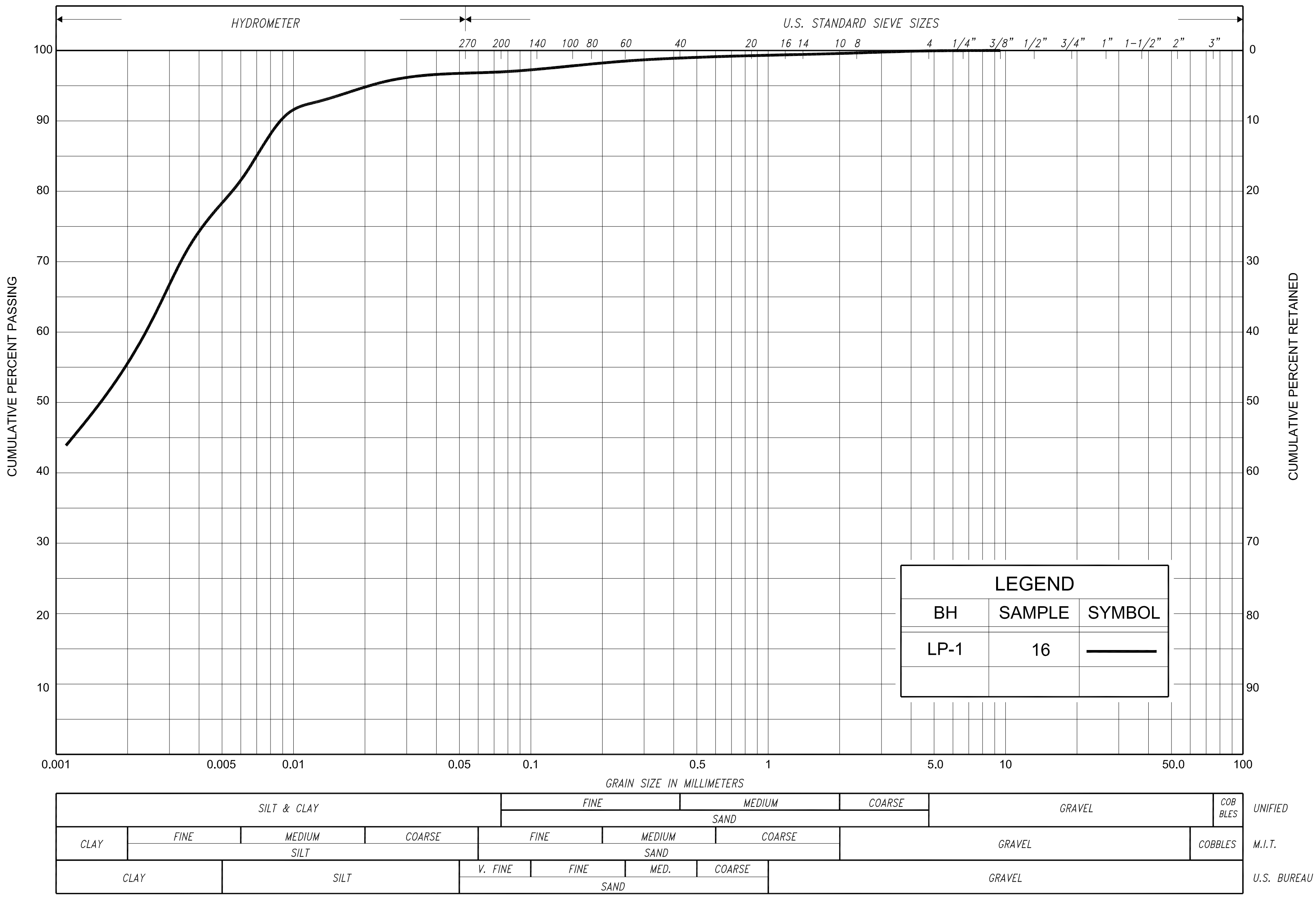


Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, with sand to sandy, trace gravel (CL)
(TILL)

FIG No.	LP-GS-1
HWY:	401
G.W.P. No.	3017-09-00

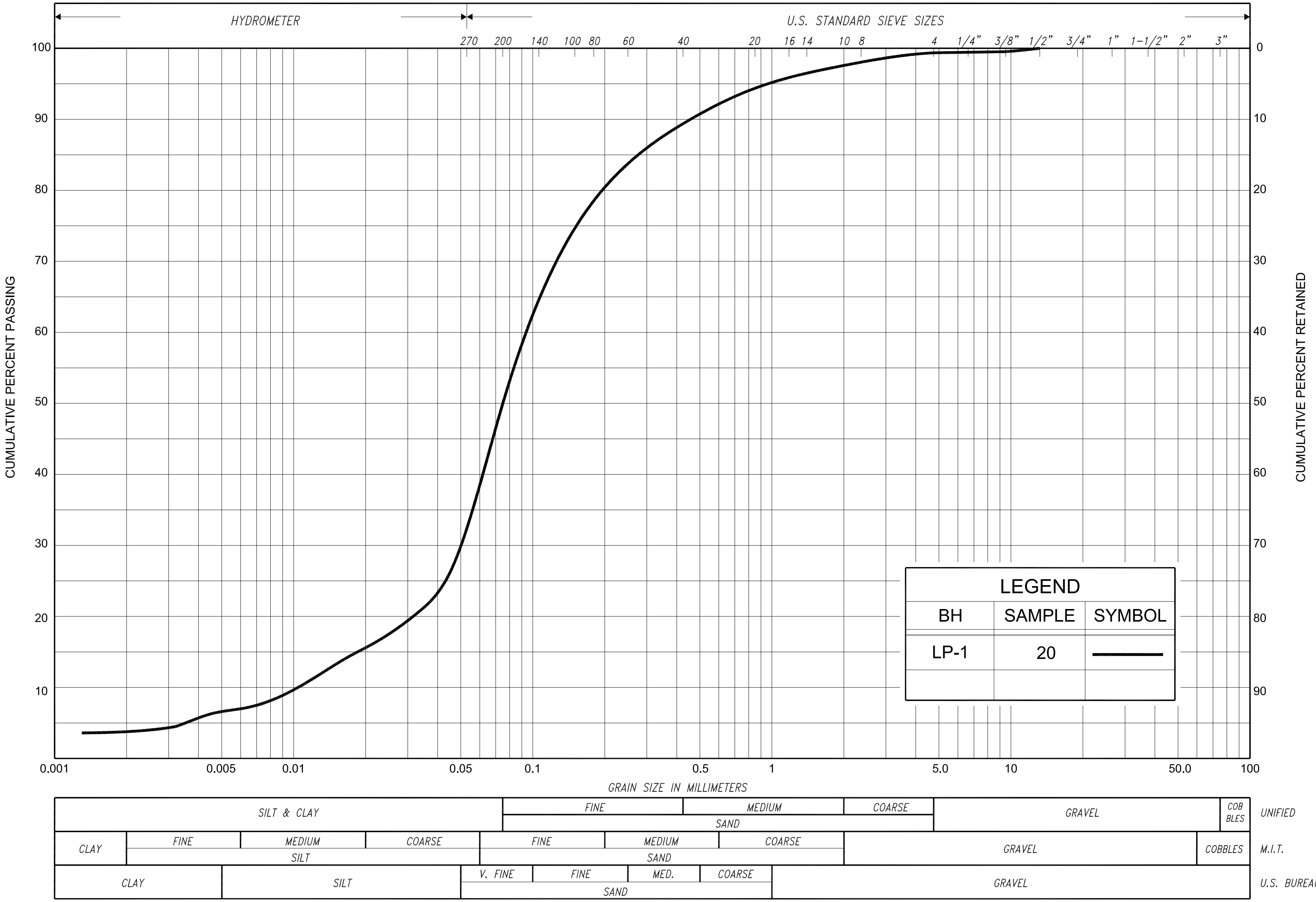


Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand (CI)

FIG No.	LP-GS-2
HWY:	401
G.W.P. No.	3017-09-00



Ontario

GRAIN SIZE DISTRIBUTION

SAND AND SILT, trace clay, trace gravel
(TILL)

FIG No.	LP-GS-3
HWY:	401
G.W.P. No.	3017-09-00

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.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 – 10	10 – 20	20 – 30	30 – 40	> 40
	TRACE	*SOME*	*WITH*	*ADJECTIVE (SILTY)*	*AND (AND SILT)*

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/0.3 m)	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:

RQD (%)	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	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_P	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_S	%	SHRINKAGE LIMIT	C_U	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_P	%	PLASTICITY INDEX $= w_L - w_P$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL				v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$	i	1	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	j	kN/m^3	SEEPAGE FORCE
e	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No LP-1

METRIC

G.W.P. 3017-09-00 LOCATION Coords: 4 678 466.9 N; 341 490.6 E ORIGINATED BY A.L.

DIST Chatham HWY 401 BOREHOLE TYPE C. F. H. S. A. + 'N' Casing + Wash boring COMPILED BY A.D.

DATUM Geodetic DATE June 19 to 21, 2013 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.0	Ground Surface						
0.0	Topsoil	1	SS	6			
185.5 / 0.5	Clayey silt with sand, trace gravel. Very stiff, Mottled brown/grey, Moist (TILL)	2	SS	17		185	
		3	SS	27		184	
		4	SS	24		183	
		5	SS	16			4 23 36 37
	Stiff, Grey					182	
		6	SS	11		181	
		7	SS	8		180	
			FV			179	
		8	SS	7		178	
			FV			177	
		9	SS	7			2 26 39 33
			FV			176	
		10	SS	7		175	
			FV			174	
		11	SS	11			
	silty sand pockets					173	
		12	SS	20	▽*	172	
171.0	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

5 × Numbers refer to Sensitivity

7 , :

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

RECORD OF BOREHOLE No LP-1

METRIC

G.W.P. 3017-09-00	**LOCATION** Coords: 4 678 466.9 N; 341 490.6 E		**ORIGINATED BY** A.L.
DIST Chatham	**HWY** 401	**BOREHOLE TYPE** C. F. H. S. A. + 'N' Casing + Wash boring	**COMPILED BY** A.D.
DATUM Geodetic	**DATE** June 19 to 21, 2013		**CHECKED BY**

SOIL PROFILE: ELEV / DEPTH, 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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	Shear strength plot	Remarks (GR SA SI CL)
171.0 / 15.0	Clayey silt with sand, trace gravel. Stiff, Grey, Moist (Cont'd.) (TILL) ____ ____ ____ ____ silty sand till pockets	13	SS	9		
			FV		> +	
		14	SS	14		
		15	SS	9		
			FV		> +	
165.5 / 20.5	Silty clay, trace sand. Stiff to very stiff, Grey, Moist to wet	16	SS	6		0 3 41 56
			FV		+ 3	
		17	SS	8		
			FV		> +	
159.2 / 26.8	Sand and silt trace clay, trace gravel. Dense, Grey, Wet (TILL)	18	SS	34		
156.0	Cont'd					

Elevation scale: 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157

7, 5 × : Numbers refer to Sensitivity

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No LP-1

METRIC

G.W.P. 3017-09-00 **LOCATION** Coords: 4 678 466.9 N; 341 490.6 E **ORIGINATED BY** A.L.

DIST Chatham **HWY** 401 **BOREHOLE TYPE** C. F. H. S. A. + 'N' Casing + Wash boring **COMPILED BY** A.D.

DATUM Geodetic **DATE** June 19 to 21, 2013 **CHECKED BY**

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
156.0 / 30.0	Sand and silt, trace clay, trace gravel. Dense, Grey, Wet (TILL) (Cont'd.)	19	SS	48	155	~22	
		20	SS	49	154, 153, 152	~21	1 49 46 4
	sand and gravel layers. Very dense	21	SS	50/13cm	151, 150, 149		
148.1 / 37.9	Limestone bedrock. Unweathered. High strength. Excellent quality	22	RC NQ	REC 100%	148, 147		RQD 100%
		23	RC NQ	REC 100%	146		RQD 100%
		24	RC NQ	REC 100%	145		RQD 100%
144.7 / 41.3	End of borehole						

Column headings: SOIL PROFILE (ELEV DEPTH, 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

* 2013 06 19 to 21

▽ Water level observed during drilling

■ 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 LP-2

METRIC

G.W.P. 3017-09-00 **LOCATION** Coords: 4 678 402.6 N; 341 464.8 E **ORIGINATED BY** A.L.

DIST Chatham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers + Wash boring **COMPILED BY** A.D.

DATUM Geodetic **DATE** June 25 and 26, 2013 **CHECKED BY**

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
186.2	Ground Surface
0.0	Topsoil
185.7 / 0.5	Clayey silt with sand, trace gravel Hard to very stiff Mottled brown/grey Moist (TILL)
	Stiff Grey Moist
	wet sand seams
171.2	silty sand pockets Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	22	
3	SS	31	
4	SS	27	
5	SS	18	
6	SS	11	2 22 39 37
7	SS	11	
8	SS	8	
	FV		
9	SS	7	
	FV		
10	SS	7	
	FV		
11	SS	7	2 26 39 33
	FV		
12	SS	8	
	FV		

ELEVATION SCALE: 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172

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

188
150

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

STRAT PLOT

+³, ×³: Numbers refer to Sensitivity

20
15 —○— 5 (%) STRAIN AT FAILURE
10

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No LP-2

METRIC

G.W.P. 3017-09-00	**LOCATION** Coords: 4 678 402.6 N; 341 464.8 E		**ORIGINATED BY** A.L.
DIST Chatham	**HWY** 401	**BOREHOLE TYPE** Continuous Flight Hollow Stem Augers + Wash boring	**COMPILED BY** A.D.
DATUM Geodetic	**DATE** June 25 and 26, 2013		**CHECKED BY**

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION (m)	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	GR	SA	SI	CL
171.2 / 15.0	Clayey silt with sand, trace gravel. Stiff, Grey, Moist (TILL) (Cont'd.)	13	SS	57**							
		14	SS	8							
			FV			+ 2					
		15	SS	9				4	21	40	25
			FV			> +					
165.7 / 20.5	Silty clay, trace sand. Stiff, Grey, Moist	16	SS	7							
			FV			+ 3					
		17	SS	9							
159.7 / 26.5	Clayey silt, sandy, trace gravel. Very stiff, Grey, Moist (TILL)	18	SS	25				6	31	41	22
156.2	Cont'd										

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (171, 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157); 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.

7 , 5 × : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No LP-2

METRIC

G.W.P. 3017-09-00 **LOCATION** Coords: 4 678 402.6 N; 341 464.8 E **ORIGINATED BY** A.L.

DIST Chatham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers + Wash boring **COMPILED BY** A.D.

DATUM Geodetic **DATE** June 25 and 26, 2013 **CHECKED BY**

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
156.2												
30.0	Clayey silt, sandy trace gravel Very stiff Grey Moist (TILL) (Cont'd.)					156						
		19	SS	17					2	34	44	20
						155						
154.5 31.7	Sand and silt trace clay, trace gravel Dense to very dense Grey Wet (TILL)					154						
						153						
		20	SS	38								
						152						
						151						
						150						
	sandy silt till layer	21	SS	72/18cm								
149.2 37.0	End of borehole Refusal on probable bedrock Sample 21: Sampler bouncing											

Chart 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; REMARKS & GRAIN SIZE DISTRIBUTION (%)

* 2013 06 25 & 26

▽ Water level observed during drilling

■ Penetrometer test

** Rock in spoon tip inflated 'N' value

7 , 5 × : Numbers refer to Sensitivity

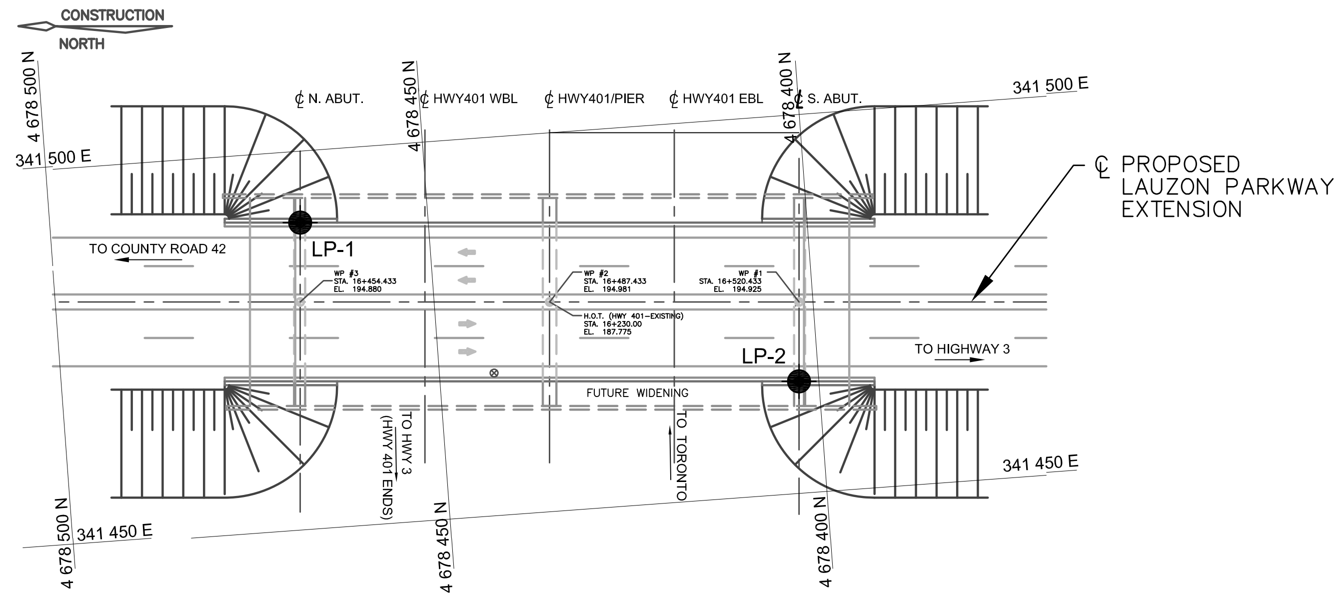
20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

CONT No
GWP No 3017-09-00

LAUZON PARKWAY UNDERPASS
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

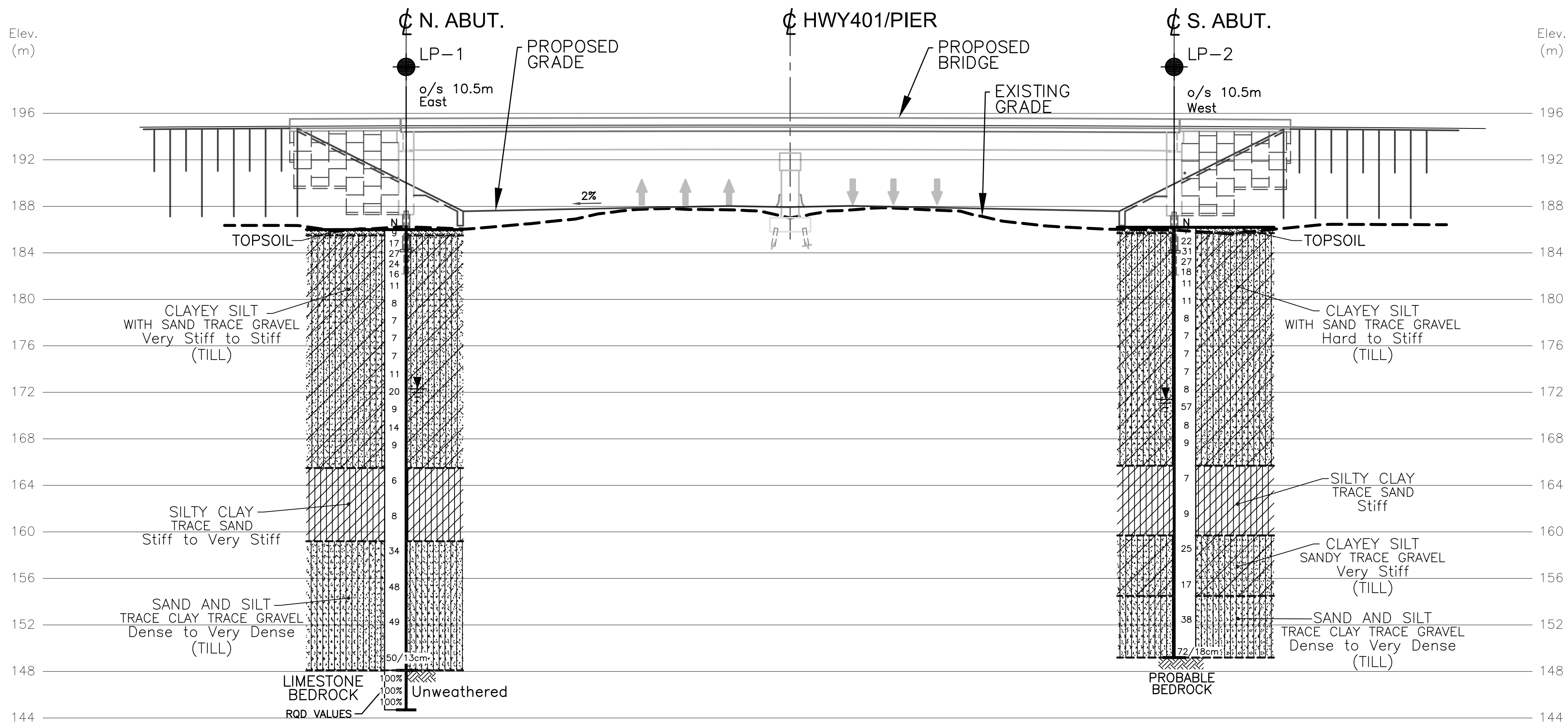
Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN

SCALE

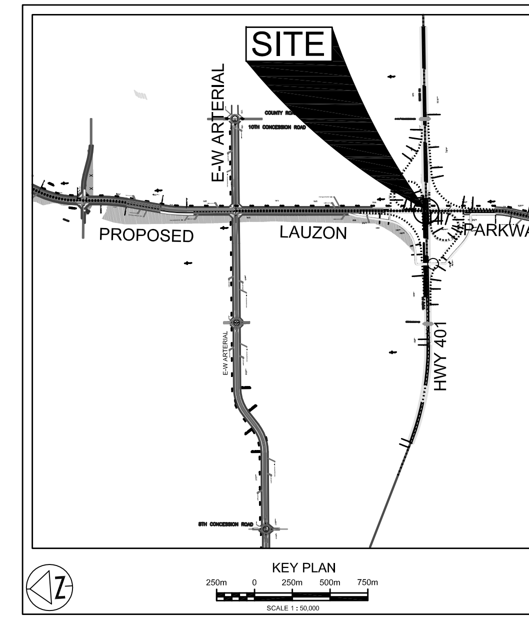
10 0 10 20m



℄ PROFILE ALONG LAUZON PARKWAY EXTENSION

SCALE

5 0 5 10m



KEY PLAN

250m 0 250m 500m 750m

LEGEND

- Borehole
- Cone
- Borehole and Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation June 2013
- * Water level not established
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

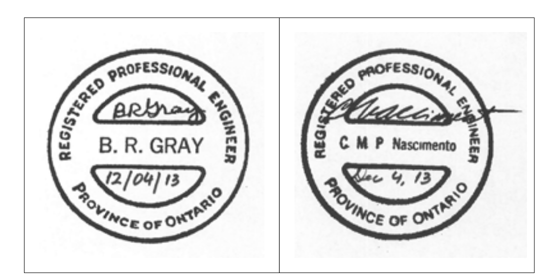
BH No	ELEVATION	NORTHINGS	EASTINGS
LP−1	186.0	4 678 466.9	341 490.6
LP−2	186.2	4 678 402.6	341 464.8

− NOTE −

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

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
3. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



REF MRC Drawing: S3211012-301-001GA.dwg

REVISIONS DATE	BY	DESCRIPTION

Geocres No. 40J2-130

HWY No 11			DIST Chatham
SUBM'D NA	CHECKED AD	DATE DEC. 04, 2013	SITE
DRAWN NA	CHECKED BRG	APPROVED CN	DWG LP-1

APPENDIX A

Rock Core Photographs

Photograph 1: Core retrieved from borehole LP-1. Rock cores 1 to 3 from 37.9 to 41.3 m. RQD values of 100%, indicating excellent rock quality.

PRELIMINARY FOUNDATION DESIGN REPORT
for
LAUZON PARKWAY UNDERPASS AT HIGHWAY 401
LAUZON PARKWAY EXTENSION
FROM EC ROW EXPRESSWAY TO HIGHWAY 3
TOWNSHIP OF TECUMSEH AND CITY OF WINDSOR
WINDSOR AREA, ONTARIO
G.W.P. 3017-09-00

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Index No.: 041FDR
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December 4, 2013

TABLE OF CONTENTS

PRELIMINARY FOUNDATION DESIGN REPORT
for
Lauzon Parkway Underpass at Highway 401
Lauzon Parkway Extension, From EC Row Expressway to Highway 3
Township of Tecumseh and City of Windsor
Windsor Area, Ontario
GWP 3017-09-00

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1. INTRODUCTION

This report provides preliminary foundation engineering analysis and recommendations for preliminary design and construction of the foundations and approach embankments for the proposed underpass on the Lauzon Parkway extension over Highway 401 between the City of Windsor and Township of Tecumseh, Ontario. This report was prepared for MMM Group Limited (MMM) on behalf of the Ministry of Transportation (MTO).

The preliminary plans call for the proposed underpass to be constructed between Station 16+454 and 16+520, Lauzon Parkway chainage. The proposed underpass is a two span structure with approximate length of 66 m between abutments and a width of 21 m (refer to the Preliminary General Arrangement Drawing, Lauzon Parkway Highway 401 Underpass dated May 2013).

The road grade of the Lauzon Parkway at the underpass location is planned at elevation 194.9 at the south and north abutments. The approach embankments of the structure are envisaged to be about 9 m above the existing grade.

In summary, the two boreholes revealed a surficial 0.5 m thick topsoil layer overlaying a clayey silt till deposit that extended to 20.5 and 31.7 m, elevation 165.5 and 154.5. The upper 3.4 and 4.0 m zones of the clayey silt till was a very stiff to hard crust above elevation 182.0 and 181.7 in boreholes LP-1 and LP-2, respectively and stiff to very stiff below. The clayey silt till was interbedded / underlain by a stiff to stiff 6.0 and 6.3 m thick silty clay layer at 20.5 m at the north and south abutments respectively. Underlying the silty clay at the north abutment and clayey silt till at the south abutment was a dense to very dense sand and silt till unit (5.3 and 11.1 m thick) which was underlain by high strength Limestone bedrock at 37.9 m (elevation 148.1) at the north abutment and probable bedrock at 37.0 m (elevation 149.2) at the south abutment.

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BARRIE, HAMILTON, KITCHENER, TORONTO

Based on the encountered subsurface conditions, the height of the embankments over the existing ground surface and the low geotechnical bearing resistance of the native soils, conventional spread footings are not considered feasible.

The underpass may be founded on deep foundations using steel H-piles driven to refusal on bedrock.

The use of drilled cast-in-place caissons should consider the potential delays caused by the groundwater control requirements, difficulties associated with the possibility of encountering cobbles and boulders and local natural gas deposits and the relatively long length (over 30 m) of caissons required. For preliminary design, this foundation alternative is not considered suitable for this site.

It should be expected that estimated total settlement due to consolidation of the clayey soils under the approximately 9 m high approach embankments will be about 120 to 140 mm. It is estimated this settlement will continue to occur for approximately 1 year following construction. Measures to reduce post-construction settlement, such as preloading, may need to be undertaken and should be investigated further during detailed design. For preliminary design, preloading the embankment prior to installation of piles should be considered to eliminate or reduce the negative skin friction loads on the abutment piles and to satisfy the MTO embankment surface settlement criteria for non-freeways.

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 current investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. In addition, further investigation should be carried out during the detailed design to confirm the subsoil, bedrock and groundwater conditions at the abutment and pier locations. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

The elevations referred in this report are expressed in meters. A list of the Ontario Provincial Standard documents referenced in this report is enclosed in Table 1.

2. FOUNDATIONS

2.1 General

It is considered that the upper soils are capable of providing a relatively low bearing resistance for bridge foundations. Consequently, it would be necessary to construct relatively large abutment and pier footings. These large footings combined with the approximately 9 m high approach embankments would create total and differential settlements greater than the acceptable 25 mm within the thick cohesive deposit underlying the till crust. Based on this, spread footings are not considered feasible at this site.

Founding the proposed underpass on steel H-piles driven to refusal on the bedrock is considered feasible for the abutments and central pier.

The assessment of the feasibility of using cast-in-place concrete drilled caissons bearing on the on the bedrock to support the underpass should consider the difficulties and potential delays caused by the groundwater control requirements, possibility of encountering cobbles and boulders in the glacial till deposits, possibility of encountering local natural gas pockets and relatively long length of caissons required.

Conventional, semi-integral and integral abutments are considered feasible at this site based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundations engineering perspective, use of integral abutments supported on piles driven to refusal on bedrock is the preferred type of abutment foundation for preliminary design.

The bridge site is located in Seismic Performance Zone 1. The liquefaction potential of the clayey soils was evaluated by considering the grain size distribution, liquid limit values and the ratio of water content to liquid limit. Based on research by Marcuson et al (1990), we believe that liquefaction of the fine grained soils is unlikely. The liquefaction potential of the granular soils was assessed using the procedure suggested by Seed and Idriss (1971) and is also considered unlikely (clause 4.6.2 of CHBDC).

The foundation frost depth in the Windsor area is 1.0 m according to the MTO OPSD 3090.101.

2.2 Pile Foundation

Steel H-piles could be used to support the foundation loads at the abutments and pier. The piles should be driven to refusal on bedrock found/inferred at depths from the existing ground surface of 37.9 m (elevation 148.1) at the north abutment and 37.0 m (elevation 149.2) at the south abutment.

The piles will be driven through native soils containing compressible clayey soils overlying dense to very dense sand and silt till. The existing grade at the south and north abutments will be raised about 9 m above the existing grade. Consequently, the development of downdrag load on the piles should be considered if the area is not preloaded and/or surcharged as recommended in Section 4.3 of this report.

Refer to Section 4.3 for a discussion and recommendations on the treatment of approach embankment settlements.

For preliminary purposes, the following factored geotechnical axial resistance at ULS for HP 310 x 110 steel piles is considered to be appropriate (refer to notes 5 and 6 in Section 3.3.3 of the Pile Driving Notes in the Structural Manual, June 2011):

PILE SECTION	FACTORED GEOTECHNICAL AXIAL RESISTANCE AT ULS (kN)
HP 310 x 110	2000

The geotechnical reaction at SLS allows for 25 mm compression of founding medium. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm deformation of the bedrock would be larger than the factored geotechnical resistance at ULS.

It is recommended that the piles be driven following preloading of the approach embankments to minimize post-construction settlements at the pile location and thus mitigate down-drag loads. Other methods such as surcharging, use of lightweight fill, wick drains or pile coatings could be employed to reduce down-drag loads. Monitoring of the pore water pressure and embankment settlement should be considered to ensure that settlement has occurred prior to pile installation.

If the piles are installed prior to embankment construction and without preloading or other settlement mitigation methods, a preliminary downdrag load in the order of the following can be considered:

PILE SECTION	UNFACTORED DOWNDRAG LOAD (kN)
HP 310 x 110	1000

It is noted that the above downdrag load is considered for preliminary design purposes only and that further investigation into downdrag loading should be conducted during detailed design.

As indicated previously, although no cobbles and boulders were encountered in the borehole the possibility of cobbles or boulders within the till deposits should not be discounted. A NSSP should be prepared to advise the contractor of the potential presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in OPSS 903.

Any fill placed below the proposed grade for a working platform to drive piles should comprise OPSS Granular A material to allow installation of the piles without damage. Alternative granular materials such as Granular B Type II could be employed provided the maximum particle size does not exceed 75 mm.

The piles will be driven through dense to very dense soils and soils possibly containing cobbles and boulders and to refusal on bedrock and should be equipped with driving shoes. OPSS 903 calls for the use of OPSD 3000.100 (Driving Shoe Details for H-piles) or Titus H Bearing Pile Points Standard Model on piles driven to bedrock under these pile driving conditions.

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

Pile caps should be provided with at least 1.0 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.

2.2.1 Integral Abutment Considerations

For the integral abutment design, the H-piles should be driven to refusal on bedrock anticipated at the depths/elevations and axial resistance that are indicated in the previous section. The minimum 5.0 m long free pile length below the abutment stem which should be incorporated in the design will not be a concern at this site.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

2.2.2 Lateral Resistance

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The recommended lateral resistance is as follows:

	NATIVE CLAYEY SILT TILL
Pile Section	HP 310
Factored Lateral Resistance at ULS, kN	160
Lateral Resistance at SLS, kN	65

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

2.3 Comparison of Foundation Alternatives

A comparison of the relative advantages and disadvantages related to each of the foundation alternatives discussed in the preceding paragraphs is presented below.

ADVANTAGES	DISADVANTAGES
Footings on Cohesive Soils	
• Lower cost than deep foundations • Allows use of semi-integral abutments	• Relatively low geotechnical resistances will require wide footings and may render this alternative structurally impractical • Preloading required prior to footing construction • Approach embankment excavation would be required following preloading to construct footing, which may render this alternative impractical
Driven Piles	
• Allows use of integral and semi-integral abutments design and construction • Lower long-term maintenance costs of deck expansion joints with integral abutment design • Negligible settlements of foundations • Not affected by surficial soil variability	• More costly than shallow foundation alternatives • Heavy equipment for pile driving is required. • May require pre-augering through layers of boulders • Preloading or application of downdrag load required prior to construction
Caisson Foundations	
• Allows use of semi-integral abutments design and construction • Lower long-term maintenance costs of deck expansion joints with integral abutment design • Negligible settlements of foundations • Not affected by surficial soil variability	• Drilling must be advanced through very stiff to hard till possibly containing cobbles and boulders • Drilling may encounter natural gas deposits • May require temporary or permanent liner to prevent seepage inflow and softening of the caisson base • Dewatering may be required during construction (i.e. caisson caps), special techniques may be required if artesian conditions are encountered

From a foundation perspective driven piles are considered a feasible option to support the bridge.

3. ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure 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 (35º for Granular B Type II)
δ = angle of friction between the soil and wall (23.5º for Granular B Type II)

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

PARAMETERS	GRANULAR A OR GRANULAR B TYPE II
Angle of Internal Friction, degrees	35
Unit Weight, 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

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 earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).

A weeping tile system (MTO SP 405F03 and OPSD 3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the walls. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standards Drawing for granular backfill at abutments (OPSD 3101.150), as applicable.

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 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 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 and the 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 and appearance.

4. APPROACH EMBANKMENTS

4.1 General

It is anticipated that the approach embankments will be approximately 9 m high. It is also assumed that the new embankments will be constructed using native soils and/or granular materials. The subgrade revealed in the two boreholes typically comprises hard to firm clayey silt till.

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

Embankment fill should be placed and compacted in accordance with MTO's Special Provision 206S03, Earth Excavation, Grading and OPSS 501, Construction Specification for Compacting.

4.2 Slope Stability

Preliminary slope stability analyses were carried out for the south and north embankments for short-term (total stress analysis) and long-term (effective stress analysis) conditions assuming a granular fill material for embankment construction. Based on the soil data and laboratory tests conducted on selected samples, the table below summarizes the preliminary soil parameters applied to the analyses.

SOIL TYPE	UNIT WEIGHT (kN/m^3)	SHORT-TERM ANALYSIS		LONG-TERM ANALYSIS	
		COHESION (kPa)	FRICTION ANGLE (Degrees)	EFFECTIVE COHESION (kPa)	EFFECTIVE FRICTION ANGLE (Degrees)
Granular Fill	23	0	35	0	35
Clayey Silt Till (very stiff to hard)	21	150	0	8	29
Clayey Silt Till (stiff)	19	90	0	8	26

The stability of the approach embankment sections was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd. The software analyses numerous potential failure surfaces and establishes a minimum safety factor aided by user input.

For preliminary purposes and due to the similarity between the native soils and the embankment dimensions, the stability of both the north and south approach embankments were assessed using a single model. The results of the slope stability analyses are provided in Figures A-1 to A-2 attached in Appendix FDR-1 and listed below.

LOCATION	SHORT-TERM CONDITION FACTOR OF SAFETY	LONG-TERM CONDITION FACTOR OF SAFETY	FIGURE NO.
South/North Abutment (Side Slope 2H:1V)	2.68	-	A-1
	-	2.26	A-2

The preliminary factors of safety (FOS) values of 2.68 for the short-term and 2.26 for the long-term conditions at the south and north abutments are considered to be adequate for slope stability considerations. These values should be confirmed for detail design.

The embankments should be constructed in accordance with OPSD 200.020, 202.010 and OPSS 206. The side slopes of the approach embankments should be inclined no steeper than 2H: 1V for earth fill or granular fill. In addition, since the embankments exceed 8.0 m in height, construction of a mid-height bench (2 m minimum wide) will be required according to OPSD 202.010.

4.3 Embankment Settlements

Settlements of the platform and pavement surface resulting from the approximately 9 m high approach embankments should be expected as a result of consolidation of the new embankment fill and the underlying native cohesive clayey soils.

The estimated magnitude of settlement of new earth or granular fill material is in the order of 45 mm and the anticipated settlement of the clayey silt till is expected to be about 120 to 140 mm. Therefore the total settlement at the proposed approach embankments is anticipated to be approximately 165 to 185 mm for preliminary evaluation purposes.

It is expected that approximately 50% of this settlement will take place during or immediately following completion of the construction. The remaining settlement is anticipated to occur over a period of about 1 year. It is therefore suggested that measures such as preloading or surcharging be undertaken to reduce the post-construction settlement. These measures should be investigated further during the detailed design.

4.4 Embankment Erosion Considerations

It is considered that earth fill utilizing local native soils will be susceptible to surface erosion, in view of the silty nature of these soils. Earth fill slopes should be protected against surface erosion by sodding (OPSS 803) and suitable vegetation. Also refer to OPSS 804 for time constraints and type of seed and mulch required. Local areas of concentrated surface water flow should be protected with additional measures, such as rip-rap, rock protection or granular sheeting (OPSS 511).

5. CONSTRUCTION CONSIDERATIONS

5.1 Excavation

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations for worker protection and safety. For this purpose, the upper very stiff to hard clayey silt till is classified as Type 2 soil and the underlying stiff clayey silt till is classified as Type 3 soil according to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Any cobbles or boulders exposed on the excavation slope faces must be removed.

5.2 Groundwater Control

During augering, groundwater was observed at 13.7 and 14.8 m (elevation 172.3 and 171.4) in boreholes LP-1 and LP-2, respectively. Following the observation of groundwater during drilling, the boreholes were charged with drilling water for wash boring and rock coring and their water levels would not be representative. For preliminary purposes the groundwater level should be assumed to be immediately below the upper crust zone of the clayey silt till, at about 4.0 to 4.5 m depths, elevation 181.7 and 182.0 in the two boreholes drilled for this preliminary design. Perched water may also be present at higher levels. The groundwater level is subject to seasonal fluctuation and rainfall patterns.

Considering the relatively impervious nature of the native clayey silt till revealed in the boreholes at this site, groundwater should not be a concern at this site. Groundwater seepage or surface water that enters the excavations for construction of the abutments and pier should be readily handled by conventional sump pumping techniques.

Surface water run-off should be diverted away from the excavations to ensure that the foundations are constructed in the dry.

6. CLOSURE

This Foundation Design Report was prepared by Mr. A. DeSira, MEng, P.Eng. and reviewed by Mr. C. M. P. Nascimento, P. Eng., Project Manager. 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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AD/CN/BRG:ad-nk

TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
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
OPSD 200.020	Earth/Shale Grading-Divided Rural
OPSD 202.010	Slope Flattening Using surplus Excavated Material on Earth or Rock Embankment
OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail

TABLE 2

GRADATION SPECIFICATION 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

APPENDIX FDR-1

Slope Stability Diagrams

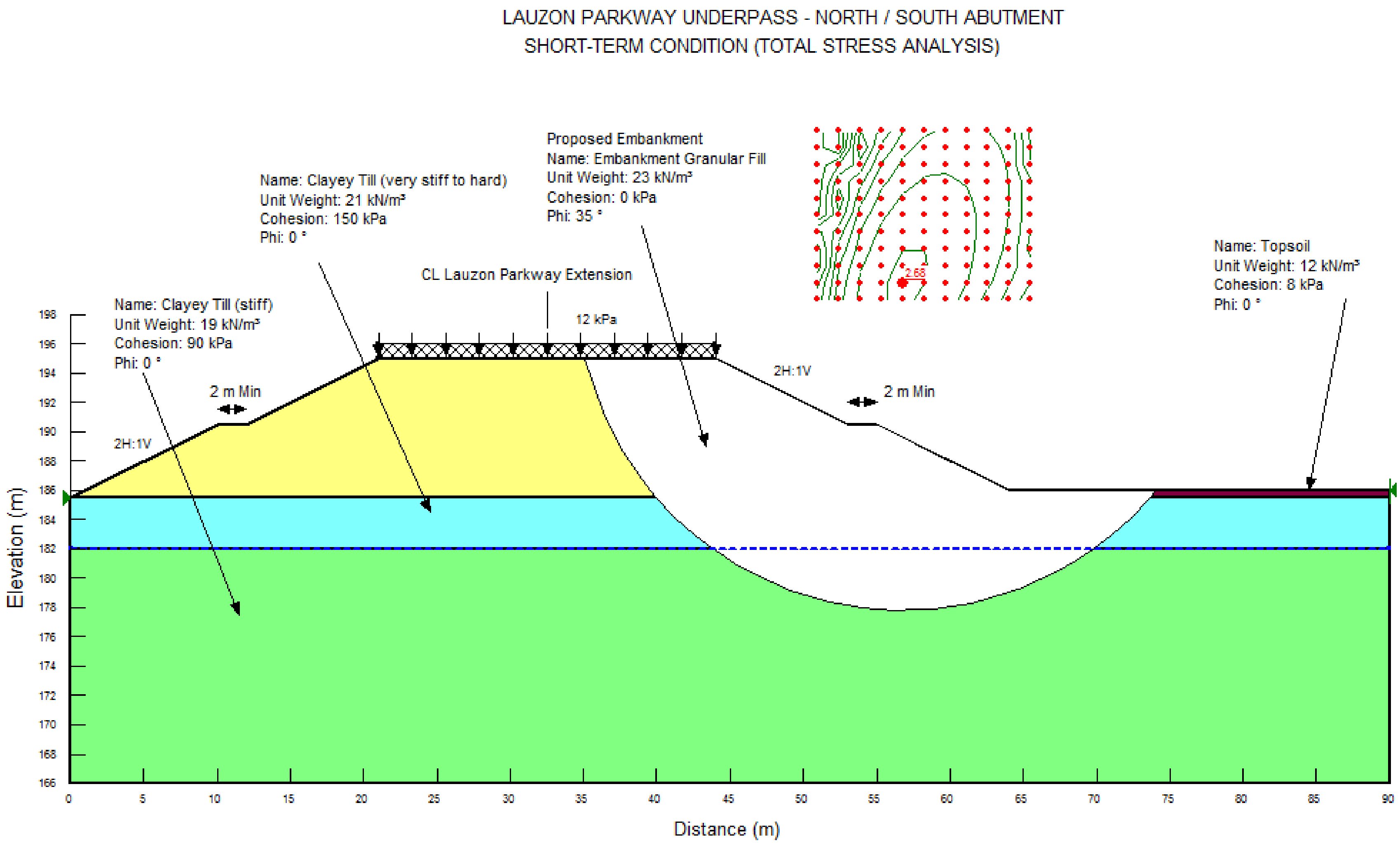
LAUZON PARKWAY UNDERPASS - NORTH / SOUTH ABUTMENT
SHORT-TERM CONDITION (TOTAL STRESS ANALYSIS)
Proposed Embankment
Name: Embankment Granular Fill
Unit Weight: 23 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Name: Clayey Till (very stiff to hard)
Unit Weight: 21 kN/m³
Cohesion: 150 kPa
Phi: 0 °
CL Lauzon Parkway Extension
12 kPa
Name: Clayey Till (stiff)
Unit Weight: 19 kN/m³
Cohesion: 90 kPa
Phi: 0 °
Name: Topsoil
Unit Weight: 12 kN/m³
Cohesion: 8 kPa
Phi: 0 °
2.68
2H:1V
2 m Min
2H:1V
2 m Min
Elevation (m)
Distance (m)

FIGURE A-1

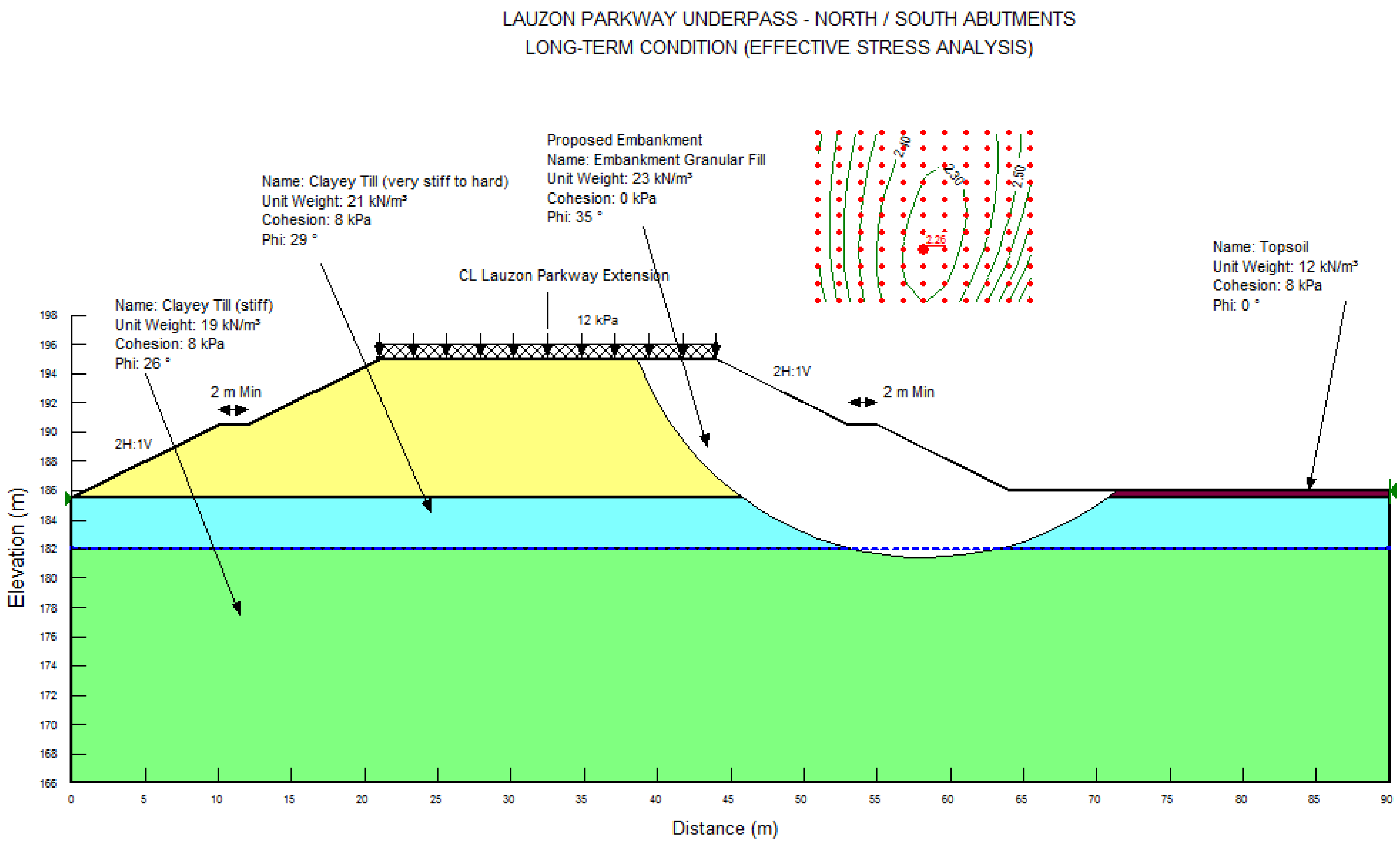
LAUZON PARKWAY UNDERPASS - NORTH / SOUTH ABUTMENTS
LONG-TERM CONDITION (EFFECTIVE STRESS ANALYSIS)
Proposed Embankment
Name: Embankment Granular Fill
Unit Weight: 23 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Name: Clayey Till (very stiff to hard)
Unit Weight: 21 kN/m³
Cohesion: 8 kPa
Phi: 29 °
CL Lauzon Parkway Extension
12 kPa
Name: Clayey Till (stiff)
Unit Weight: 19 kN/m³
Cohesion: 8 kPa
Phi: 26 °
Name: Topsoil
Unit Weight: 12 kN/m³
Cohesion: 8 kPa
Phi: 0 °
2 m Min
2H:1V
2H:1V
2 m Min
2.25
Elevation (m)
Distance (m)

FIGURE A-2