FOUNDATION INVESTIGATION AND DESIGN REPORT
for
GUELPH STREET OVERPASS EXTENSION AT HIGHWAY 85
HIGHWAY 7 & 85 IMPROVEMENTS
KITCHENER, ONTARIO
G.W.P. 3110-09-00

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PML Ref.: 10KF079
Index No.: 153FIR and 154FDR
GEOCRES No.: **40P8-230**
November 25, 2014

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FOUNDATION INVESTIGATION REPORT
for
Guelph Street Overpass Extension at Highway 85
Highway 7 & 85 Improvements
Kitchener, Ontario
GWP 3110-09-00

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1. INTRODUCTION

This report summarizes the results of a foundation investigation carried out at Highway 85 for the proposed Guelph Street overpass extension in Kitchener, Ontario. This extension is associated with the local east widening of Highway 85. The study is part of the proposed Highway 7 and 85 improvements. 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).

An existing Preliminary Foundation Investigation and Design Report was prepared by Thurber Engineering Ltd. (Thurber) dated June 2009 (Geocres No. 40P8-165) and included soils information beyond the existing Highway 85 embankment. This investigation was intended to supplement the previous investigation at the site and provide soils information for the existing embankment.

The purpose of this report is to summarize the subsurface stratigraphy encountered at the site of the proposed extension, during the investigation.

2. SITE DESCRIPTION AND GEOLOGY

The site lies on the east side of Highway 85 where the highway crosses Guelph Street, approximately 350 m north of Wellington Street in the City of Kitchener. Land use in the vicinity of the site includes the Highway 85 transportation corridor and numerous commercial properties to the east and west around the highway corridor. A natural gas main runs along the existing Guelph Street and through the proposed work area. The gasmain alignment is located about 4.5 m south of the existing north abutment.

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BARRIE, HAMILTON, KITCHENER, TORONTO

The local topography is generally flat at the site. The current difference in elevation between the Guelph Street and Highway 85 pavements is approximately 6.2 m, from elevation 318.7 on Highway 85 to elevation 312.5 on Guelph Street.

The project site is located within the physiographic region known as the Waterloo Hills. The surface of the Waterloo Hills is generally characterised by sandy hills, including sandy till ridges, kames and kame moraines, with outwash sands occupying the intervening hollows.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out on June 26, 2014 and comprised two boreholes drilled through the existing Highway 85 embankment to a depth of 9.8 and 10.4 m at the locations shown on Drawing G-1, appended.

The borehole locations were strategically located to provide soils data for the existing Highway 85 embankment and minimizing the impact on highway traffic. 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 solid stem augers with a truck-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field technician.

Soil samples were recovered from the boreholes at regular 0.75 and 1.5 m intervals following the standard penetration testing. Standard penetration tests were conducted to assess the strength characteristics of the substrata. 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.

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

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 (17)
- Grain size distribution analyses (5)
- Atterberg limit test (1)

The charts prepared using the results of the laboratory grain size distribution analyses and Atterberg Limit Test are presented in Figures G-GS-1 to G-GS-3 and G-PC-1, respectively. 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 101 and 102 for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test data, and groundwater observations. The results of laboratory grain size distribution analyses, Atterberg limit test and moisture content determinations are also shown on the Record of Borehole sheets.

Additionally, reference is also made to the Record of Borehole sheets and laboratory test results of the boreholes 08-003, 1 and 7 which were previously drilled at the site and presented in Thurber's June 2009 report. The Record of Borehole sheets and laboratory test results are presented in Appendix FIR-A.

The borehole locations and stratigraphic profile prepared from the borehole data are shown on Drawing G-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, these boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the current boreholes 101 and 102, drilled from the top of the existing Highway 85 embankment, comprised the existing pavement structure over compact to very dense sandy fill to 6.4 and 7.2 m over dense to very dense sandy silt / silty sand locally above very stiff silty clay which extended to the 9.8 and 10.4 m borehole termination depths.

The boreholes previously drilled from below the existing embankment level, boreholes 1, 7, and 08-003 presented in Thurber's June 2009 report and drilled east of Highway 85 generally revealed a compact to very dense silty sand / sandy silt which extended to 3.7 to 6.4 m, over stiff to hard clayey silt / silty clay (locally silty clay till) to 9.1 to 10.7 m, above a dense to very dense sandy silt to clayey silt (locally sandy silt till) which extended to the 13.4 to 15.4 m termination depth of the boreholes. Locally the existing Guelph Street pavement structure over compact to dense fill was encountered surficially in borehole 08-003 that extended to 2.0 m.

A summary of the findings is given below.

4.1 Pavement

The pavement encountered in boreholes 101 and 102 that were drilled from the Highway 85 surface included 180 and 250 mm of asphaltic concrete, underlain by 320 and 250 mm of sand and gravel in boreholes 102 and 101 respectively.

4.2 Fill

A 6.4 and 7.0 m thick unit of embankment fill including the pavement layers was encountered in borehole 101 and 102 that extended to elevation 312.4 and 311.2, respectively. The fill, a sand to silty sand fill was encountered beneath the pavement structure at 0.5 m in both boreholes. The fill was compact to very dense (SPT-'N' values of 15 to 53) and moist to wet (moisture contents of 5 to 15%). The results of grain size distribution analyses performed on 3 samples of the fill are presented on Figure G-GS-1.

A 2.0 m thick fill layer was also encountered surficially in borehole 08-003 that extended to elevation 310.5. The fill included 50 mm of asphaltic concrete (probably the paved shoulder of Guelph Street) underlain by silty sand. The fill was compact to dense (SPT-'N' values of 13 and 32) and moist (moisture contents of 8 to 10%).

4.3 Upper Silty Sand / Sandy Silt

A 3.2 and 2.6 m thick silty sand / sandy silt deposit was contacted beneath the fill in boreholes 102 and 101, at 7.2 and 6.4 m (elevations 311.2 and 312.4), respectively. The sandy silt / silty sand extended to the silty clay at 9.0 m (elevation 309.8) in borehole 101 and to the 10.4 m termination depth (elevation 308.0) in borehole 102. Sand and gravel layers, and cobbles and boulders were encountered within the deposit in boreholes 102. The material was dense to very dense (SPT-'N' values of 36 blows to 58 blows for 15 cm) and moist to wet (moisture contents of 9 to 18 %). The results of a grain size distribution analysis performed on a sample of the sand and gravel layer within the deposit in borehole 102 is presented on Figure G-GS-2.

A 3.7 to 4.6 m thick silty sand / sandy silt deposit was also contacted beneath a clayey silt layer at 2.3 m (elevation 310.2) in borehole 08-003 and surficially in boreholes 7 and 1. The deposit extended to depths ranging from 3.7 to 6.4 m (elevation 306.1 to 308.6). The deposit was compact to very dense (SPT-'N' of 14 to 56) and had moisture contents of 8 to 22%.

4.4 Clayey Silt / Silty Clay

Locally a 0.8 m thick layer of silty clay was contacted beneath the silty sand in borehole 101 at 9.0 m (elevation 309.8) that extended to the 9.8 m borehole termination depth (elevation 309.0). The silty clay was very stiff (SPT-'N' value of 24) and wetter than the plastic limit (moisture content of 22 %). The results of a grain size distribution analysis and Atterberg Limit Test performed on a sample of the silty clay is presented on Figures G-GS-3 and G-PC-1, respectively. Atterberg Limit Testing indicated that the silty clay had a liquid limit of 37, plastic limit of 17 and plasticity index of 20.

A 0.3 m thick clayey silt layer was contacted beneath the silty sand fill at 2.0 m (elevation 310.5) in borehole 08-003 that extended to 2.3 m (elevation 310.2). The clayey silt was stiff and had a moisture content of about 19%.

A clayey silt / silty clay deposit was also contacted beneath the sandy silt / silty sand at 3.7 to 6.4 m (elevations 306.1 to 308.6) in boreholes 1, 7, 08-003 that extended to 9.1 to 10.7 m (elevations 301.8 to 304.0). It is noted that the silty clay from borehole 08-003 was classified as a glacial till and the presence of cobbles and boulders within this deposit should be considered. The clayey silt / silty clay was very stiff to hard (SPT-‘N’ value of 26 to 62) and at the plastic limit to wetter than the plastic limit (moisture content of 18 to 22 %). Atterberg Limit Testing indicated that the clayey silt / silty clay had plastic limits between 15 and 20 and liquid limits of 25 to 43.

4.5 Lower Sandy Silt

A lower sandy silt deposit (locally sandy silt to clayey silt in borehole 1) was contacted beneath the clayey silt / silty clay in boreholes 1, 7 and 08-003 at 9.1 to 10.7 m (elevation 301.8 to 304.0). The deposit was at least 4.3 to 6.3 m thick and extended to the 13.5 to 15.4 m termination depth in all boreholes. It is noted that the material was classified as a glacial till in borehole 08-003 and the presence of cobbles and boulders within this deposit should be considered. The deposit was typically very dense, locally dense in the upper portion of the layer in borehole 1 (SPT-‘N’ values of 36 blows to 100 blows for 17.5 cm) and moist (moisture contents of 5 to 10%).

4.6 Groundwater

In the process of augering, water strikes were observed at of 4.0 and 8.5 m (elevations 314.8 and 309.9) in boreholes 101 and 102 respectively. Upon completion of augering, groundwater was measured in boreholes 101 and 102 at 4.9 and 7.9 m (elevation 313.9 and 310.5), respectively.

In the previous boreholes completed at the site, boreholes 1, 7 and 08-003 water levels were observed during drilling at 0.3 to 2.7 m (elevations 309.8 to 312.8).

The groundwater levels at the site are subject to seasonal fluctuation and precipitation patterns.

5. CLOSURE

Mr. A. Lo carried out the field investigation for this study under the supervision of Mr. A. DeSira, MEng, P. Eng., 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., revised to address MTO comments by D. Dundas, P.Eng. and reviewed by Mr. C. Nascimento, P.Eng., Project Manager and MTO Designated Principal Contact.

Yours very truly

Peto MacCallum Ltd.

LICENSED PROFESSIONAL ENGINEER
D. H. DUNDAS
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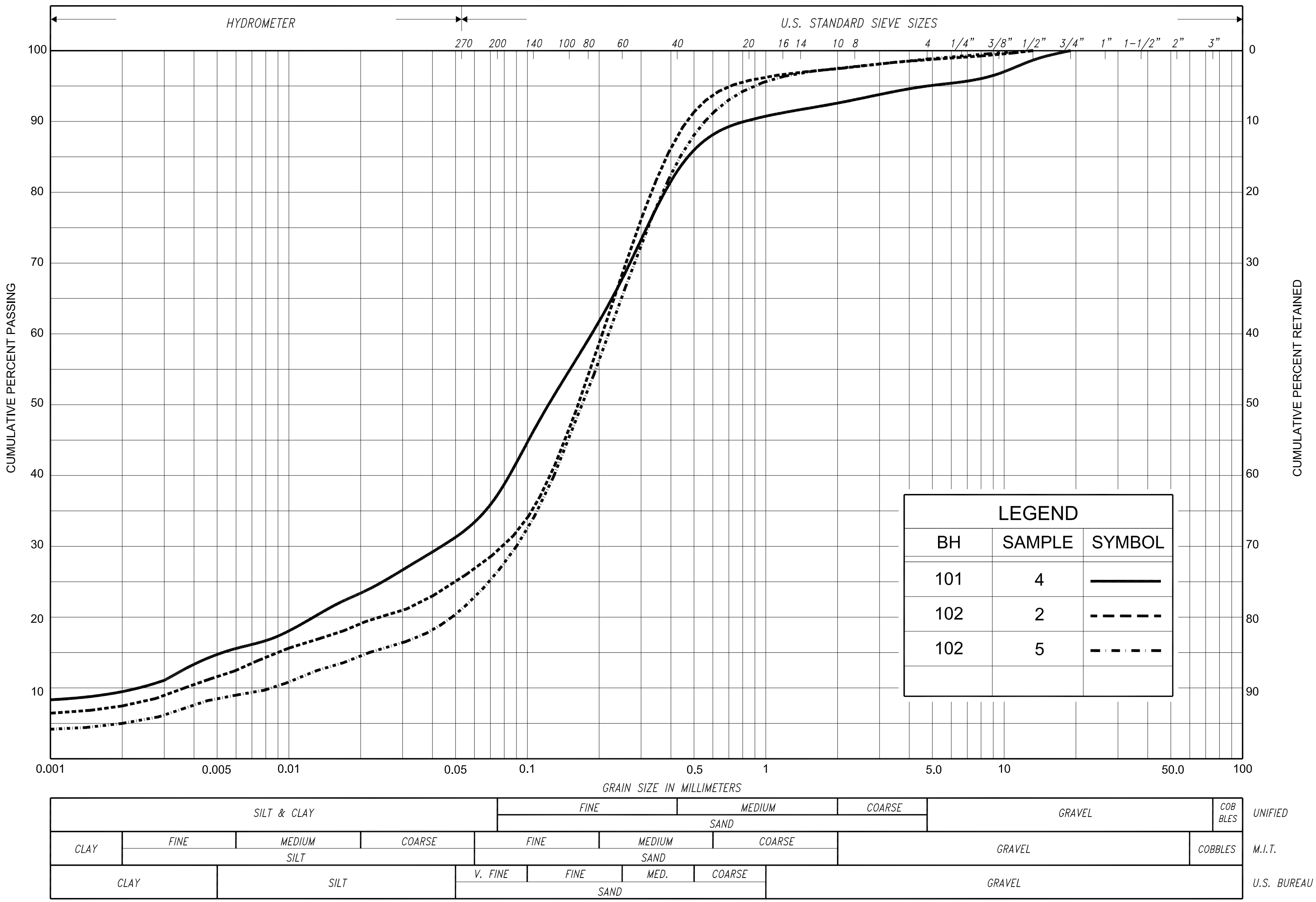
Nov 26. 2014
PROVINCE OF ONTARIO

Carlos Nascimento, P.Eng.
MTO Designated Principal Contact

DD/CN:dd-mi

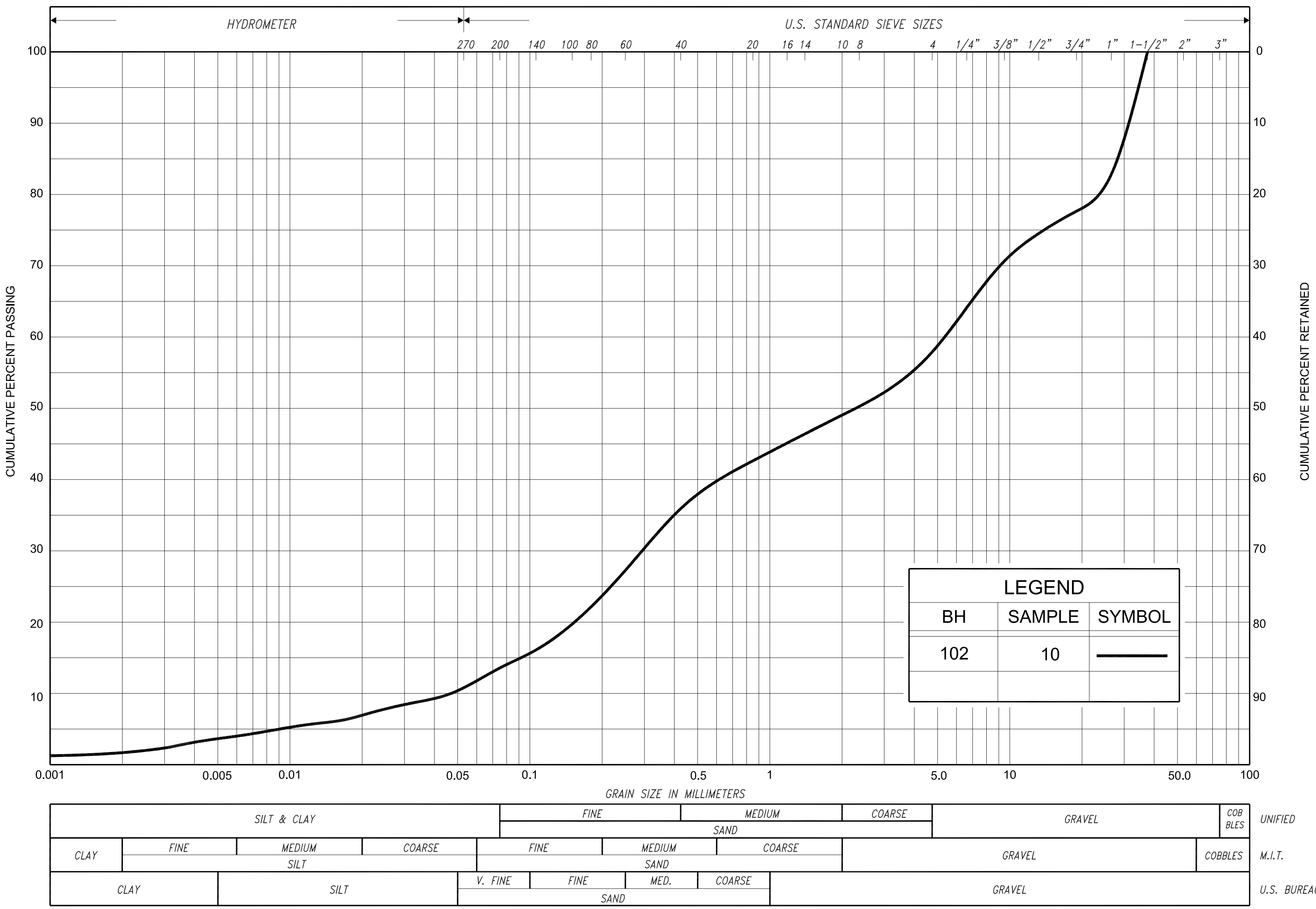


Ontario

GRAIN SIZE DISTRIBUTION

SAND, with silt, trace clay, trace gravel
(FILL)

FIG No.	G-GS-1
HWY:	7 / 85
G.W.P. No.	3110-09-00

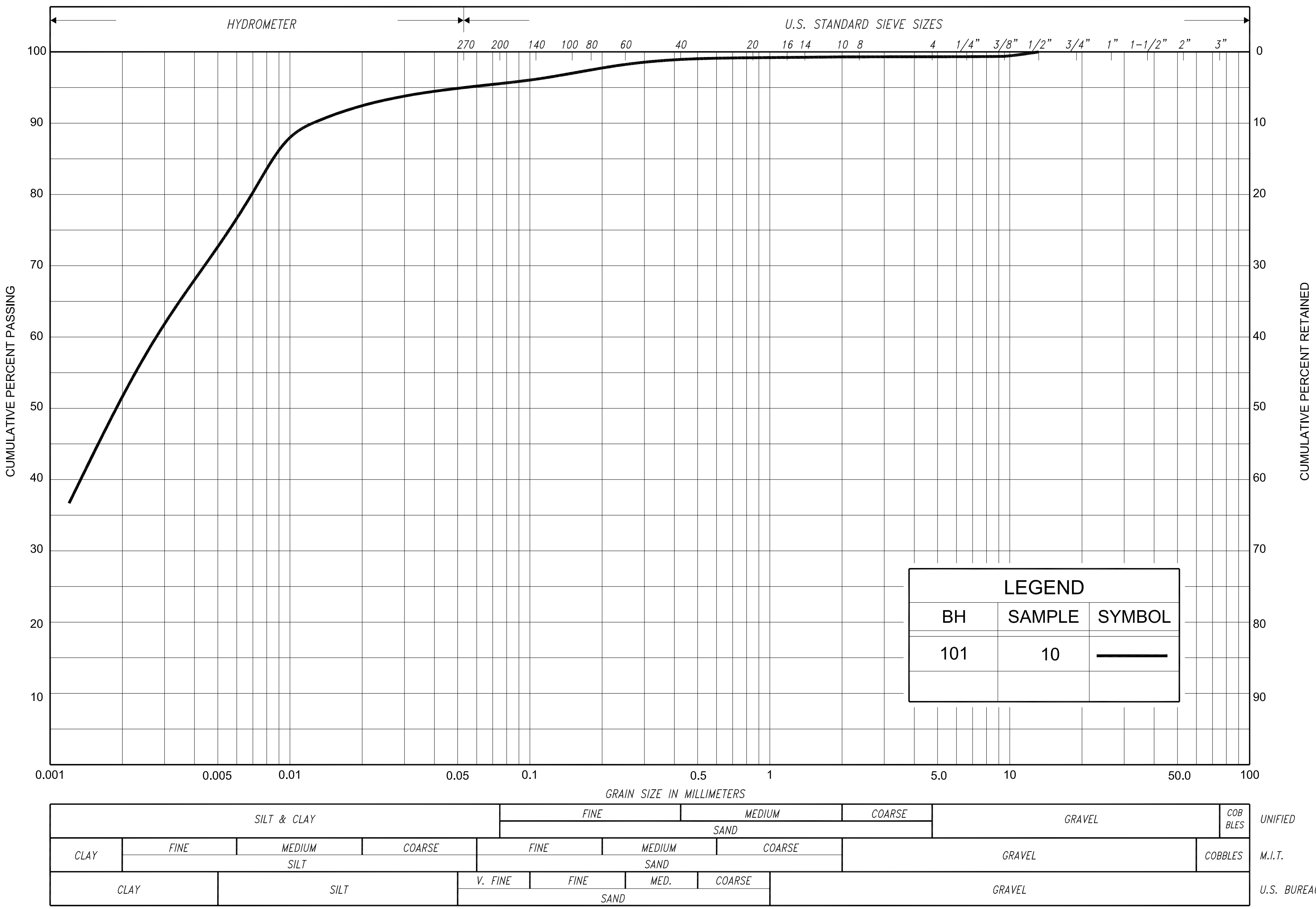


Ontario

GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL, some silt, trace clay

FIG No.	G-GS-2
HWY:	7 / 85
G.W.P. No.	3110-09-00

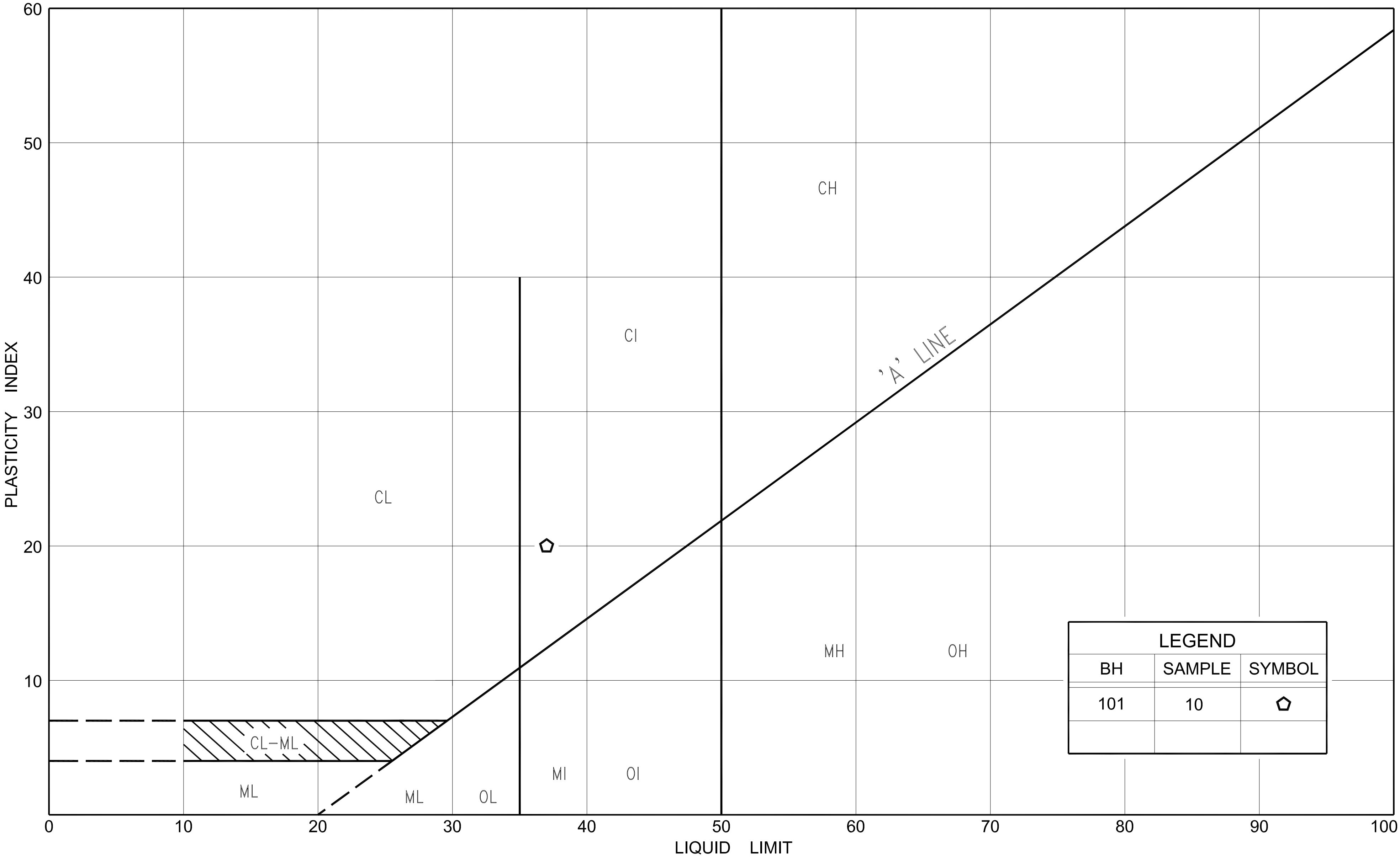


Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand, trace gravel (CI)

FIG No.	G-GS-3
HWY:	7 / 85
G.W.P. No.	3110-09-00



LEGEND		
BH	SAMPLE	SYMBOL
101	10	⬠

Ontario

PLASTICITY CHART

SILTY CLAY, trace sand, trace gravel (CI)

FIG No.	G-PC-1
HWY:	7 / 85
G.W.P. No.	3110-09-00

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
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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No. 101

METRIC

G.W.P. 3110-09-00 **LOCATION** Co-ords: 4 814 720.6 N; 226 065.0 E **ORIGINATED BY** A.L.

DIST London **HWY** 85 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** June 26, 2014 **CHECKED BY** B.R.G.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION
318.8	Ground Surface
0.0	250mm asphalt over 250mm sand and gravel
318.3 / 0.5	Dense Brown Moist (PAVEMENT FILL)
	Sand, trace to with silt trace clay, trace gravel. Compact Brown Moist
	Grey
	Brown to grey Wet
	(FILL)
312.4 / 6.4	Silty sand. Very dense Greyish brown Moist to wet
309.8 / 9.0	Silty clay trace sand, trace gravel. Very stiff Grey Moist
309.0 / 9.8	End of borehole

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	32	
2	SS	26	
3	SS	37	
4	SS	20	5 58 28 9
5	SS	33	
6	SS	21	
7	SS	15	
8	SS	36	
9	SS	51	
10	SS	24	1 3 44 52

Ground water conditions: water level observed during drilling at approx. elev. 315.0 *; water level measured after drilling at approx. elev. 314.0 *.

Elevation scale: 318, 317, 316, 315, 314, 313, 312, 311, 310, 309

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

* 2014 06 26

▽ Water level observed during drilling

▼ Water level measured after drilling

+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. 102

METRIC

G.W.P. 3110-09-00 **LOCATION** Co-ords: 4 814 776.1 N; 226 043.0 E **ORIGINATED BY** A.L.

DIST London **HWY** 85 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** June 26, 2014 **CHECKED BY** B.R.G.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
318.4	Ground Surface										
0.0	180mm asphalt over 320mm sand and gravel; Dense, Brown, Moist (PAVEMENT FILL)					318					
317.9 / 0.5	Sand, with silt, trace clay, trace gravel; Compact to very dense, Brown to grey, Moist	1	SS	20							
		2	SS	22		317	○	2	69	22	7
		3	SS	53			○				
		4	SS	43		316	○				
		5	SS	35		315	○	2	72	22	4
		6	SS	27		314	○				
		7	SS	49		313	○				
	(FILL)	8	SS	29		312	○				
311.2 / 7.2	Sandy silt, some gravel, cobbles and boulders; Very dense to dense, Dark grey to brown, Wet					311					
		9	SS	58/15cm	▼*	310					
					▽*						
	sand and gravel layers	10	SS	52		309	○	42	44	12	2
308.6 / 9.8	Silty sand; Dense, Brown, Wet	11	SS	36		308					
308.0 / 10.4	End of borehole										

* 2014 06 23

▽ Water level observed during drilling

▼ Water level measured after drilling

Legend: 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

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

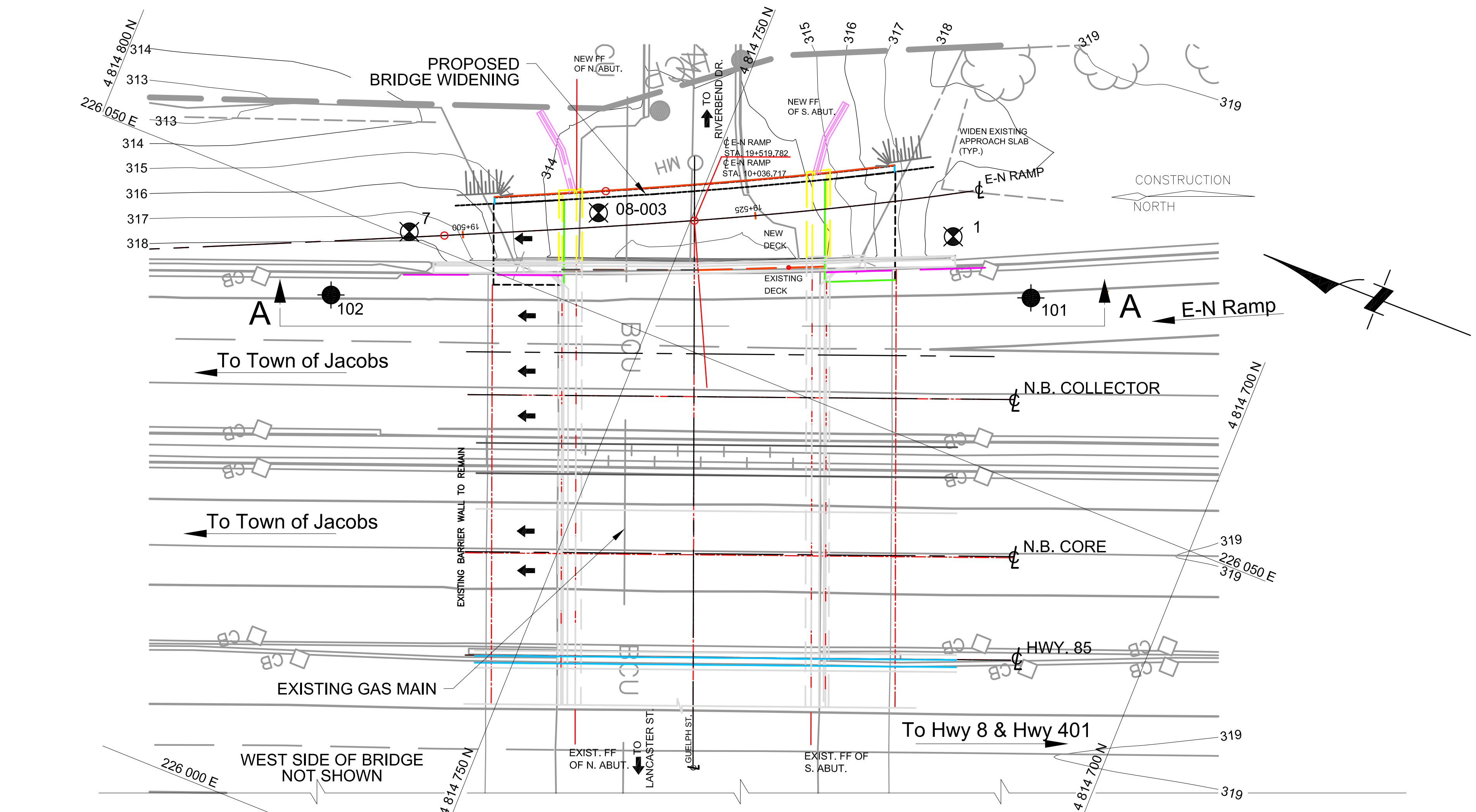
20, 15, 5, 10 (%) STRAIN AT FAILURE

CONT No
GWP No 3110-09-00

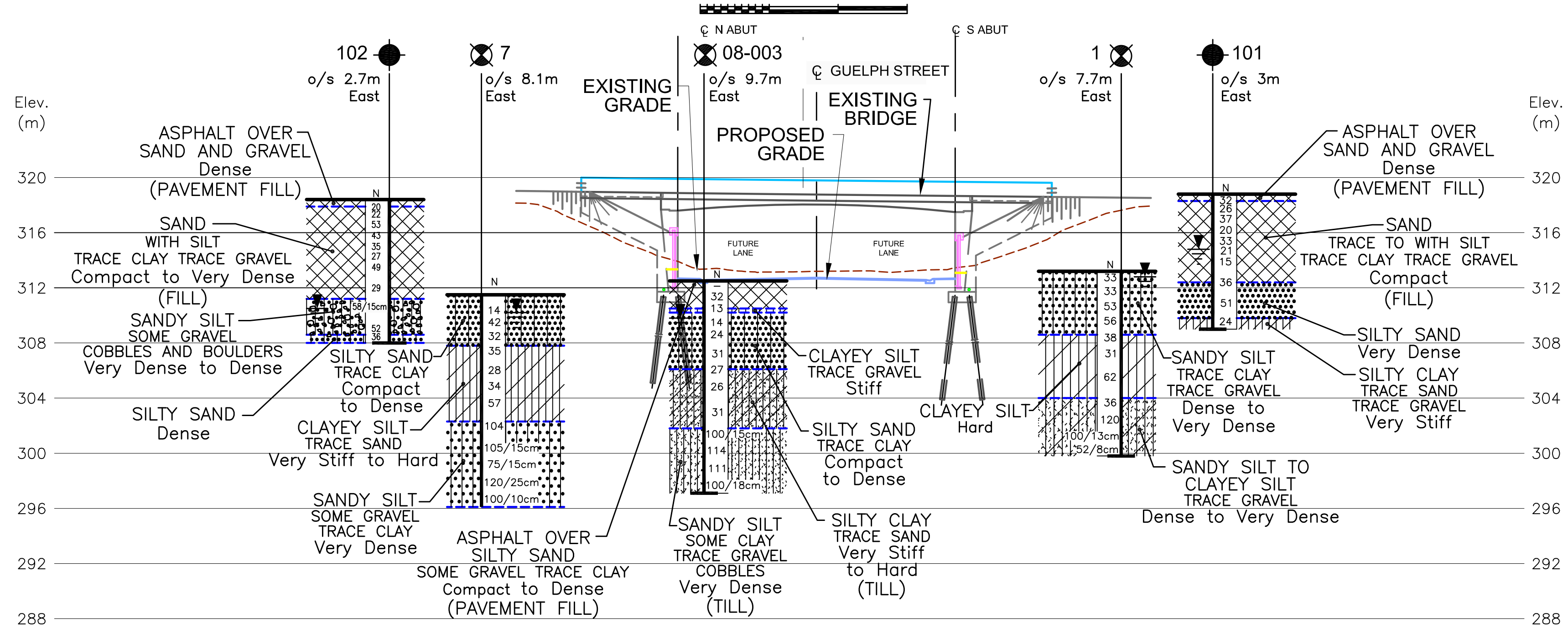
GUELPH STREET OVERPASS EXTENSION
HIGHWAY 85 WATERLOO TWP
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

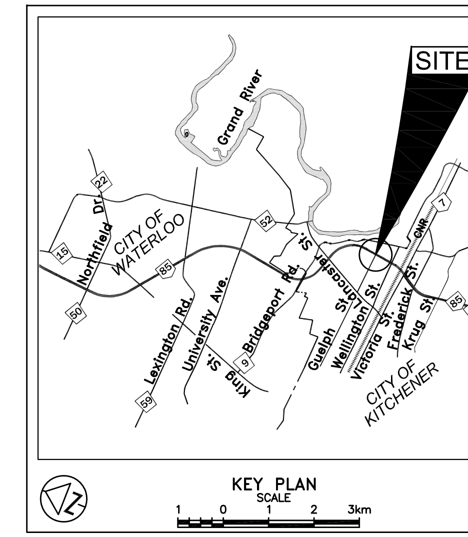


PLAN
SCALE



PROFILE A-A - ALONG N.B. COLLECTOR LANE OF HIGHWAY 85
AT GUELPH STREET OVERPASS

SCALE



KEY PLAN
SCALE

LEGEND

- Borehole
- Previous Geocres Boreholes by Others
- Borehole & 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 1966, June 2008 and June 2014
- Head
  ARTESIAN WATER
  Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
101	318.8	4 814 720.6	226 065.0
102	318.4	4 814 776.1	226 043.0
PREVIOUS GEOCRES BOREHOLES			
08-003	312.5	4 814 757.5	226 058.0
1	313.2	4 814 728.7	226 067.4
7	311.5	4 814 771.9	226 050.5

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 CONJUCTION WITH THE TEXT OF REPORT AND THE RECORD OF LOG OF BOREHOLES.
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.

LICENSED PROFESSIONAL ENGINEER D. H. DUNDAS PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER C M P Nascimento PROVINCE OF ONTARIO

Ref. MMM Drawing: H8114_NC01_Prelim.dwg;

REVISIONS DATE	BY	DESCRIPTION

Geocres No. 40P8-230

HWY No 85			DIST London
SUBM'D NA	CHECKED AD	DATE SEPT. 08, 2014	SITE
DRAWN NA	CHECKED AD	APPROVED CN	DWG G-1

APPENDIX FIR-A

Previous Boreholes and Laboratory Test Results from Geocres No. 40P8-165

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 08-003

METRIC

G.W.P. 408-88-00	LOCATION N 4 814 757.53 E 226 058.02		ORIGINATED BY ES
HWY 7	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY WM
DATUM Geodetic	DATE 2008.06.04 - 2008.06.05		CHECKED BY RPR

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 (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; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
312.5					
0.0 / 0.1	**ASPHALT:** (50mm)				
	Silty **SAND**, some gravel, trace clay Compact to Dense Dark Grey to Brown Moist (FILL) Strong gas odour	1	AS		
		1	SS	32	20 48 32 (SI+CL)
		2	SS	13	
310.5 / 2.0	Clayey **SILT**, trace gravel Stiff Greyish Brown				
310.2 / 2.3	Silty **SAND**, fine grained, trace clay Compact to Dense Brown Wet	3	SS	14	
		4	SS	24	
		5	SS	31	0 66 31 3
306.1 / 6.4	Silty **CLAY**, trace sand Very Stiff Dark Grey (TILL)	6	SS	27	
		7	SS	26	0 3 43 54
	Layer of sand: (225mm) Hard	8	SS	31	

Elevation scale: 312, 311, 310, 309, 308, 307, 306, 305, 304, 303

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

15-⊕-5 (20 / 10) (%) STRAIN AT FAILURE

ONTMT4S 6417R.GPJ 9/15/08

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 08-003

METRIC

G.W.P. 408-88-00 | LOCATION N 4 814 757.53 E 226 058.02 | ORIGINATED BY ES

HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY WM

DATUM Geodetic | DATE 2008.06.04 - 2008.06.05 | CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR SA SI CL
	Continued From Previous Page						
	Silty **CLAY**, trace sand Very Stiff Dark Grey (TILL)						
301.8 / 10.7	Sandy **SILT**, trace gravel, some clay, occasional cobbles Very Dense Grey Moist (TILL) Possible cobble at 11.1m	9	SS	100/ .150	302	o	
					301		
		10	SS	114	300	o	0 35 53 12
					299		
		12	SS	111	298	o	
297.1 / 15.4		13	SS	100/ .175		o	
	END OF BOREHOLE AT 15.4m. BOREHOLE OPEN AND WATER LEVEL AT 2.7m UPON COMPLETION. BOREHOLE BACKFILLED WITH GROUT TO 9.2m, BENTONITE TO 0.1m, THEN ASPHALT TO SURFACE.						

Column headings: SOIL PROFILE; 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

+³, ×³ Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

FORM OB-MT-126 (FORMERLY OB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-57 LOCATION N 204,340.7 53 , E 210,398,892 ORIGINATED BY D.T.W.

W.P. 638-64 BORING DATE June 9-13, 1966 COMPILED BY S.S.

DATUM Geodetic BOREHOLE TYPE Washboring and Cone Penetration CHECKED BY K.G.S.

SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FOOT	ELEV. SCALE	REMARKS
1027.4	Groundlevel					▽ 1026.2
0.0	Sandy silt. Traces of gravel and clay. Dense to very dense.	1	SS	33		Sa 9%
		2	SS	33	1020	Si 87%
		3	SS	53		Cl 4%
1012.4		4	SS	56		
15.0	Clayey silt with occasional layers of silty clay. Hard.	5	SS	38	1010	
		6	SS	31		Sa 0% Si 60% Cl 40%
997.4		7	SS	62	1000	
30.0	Sandy silt to clayey silt with occasional gravel. Dense to very dense.	8	SS	36		
		12	SS	120		
		9	SS	100/5"	990	Gr. 9% Sa 15% Si 36% Cl 40%
983.4		11	SS	52/3"		
44.0	End of borehole.				980	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

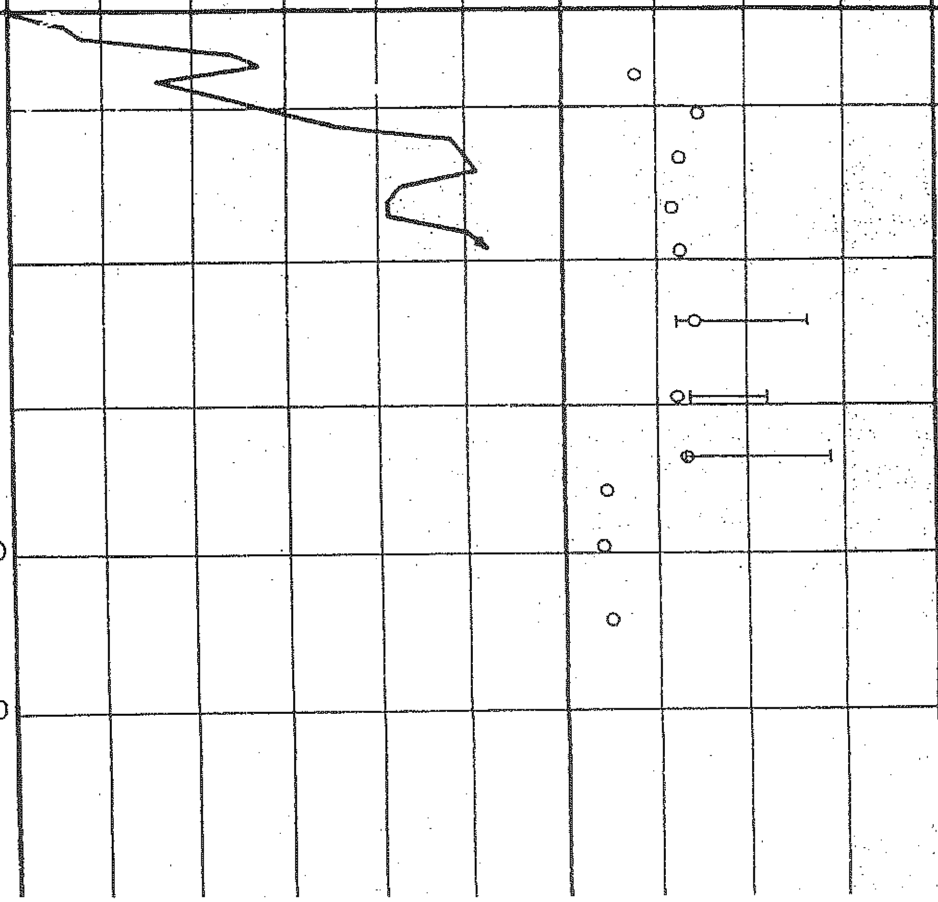
SHEAR STRENGTH P.S.F.

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

WATER CONTENT % 15 20 25

BULK DENSITY γ P.C.F.

STRAT. PLOT



FORM DB-MT-126 (FORMERLY DB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 66-F-57
W.P. 638-64
DATUM Geodetic

LOCATION N 204,479.162 E 210,355.594
BORING DATE June 9-10, 1966
BOREHOLE TYPE Washboring and Cone Penetration

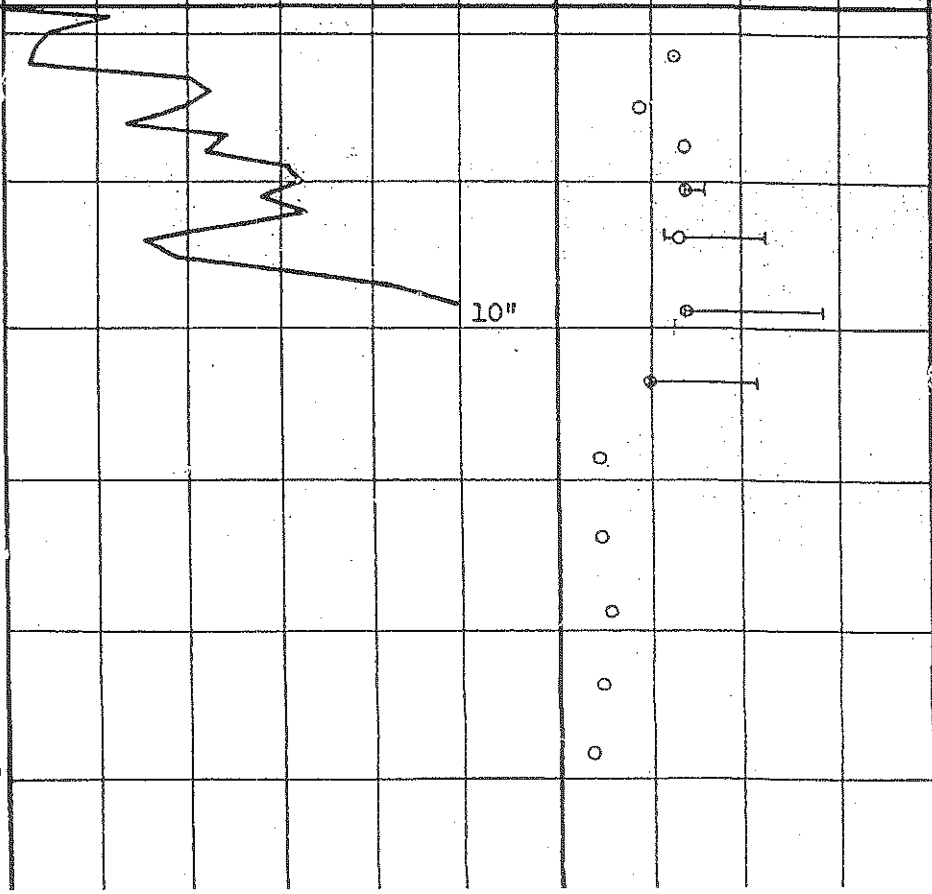
ORIGINATED BY D.T.W.
COMPILED BY S.S.
CHECKED BY K.G.S.

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
1022.0	Groundlevel					Sa34% Si61%
0.0	Silty fine sand with occasional clay.				1020	Cl 5%
		1	SS	14		▼1017.7
	Compact to dense.	2	SS	42		
		3	SS	32		
1010.0					1010	
12.0	Clayey silt with pockets of silt and traces of coarse sand.	4	SS	35		Sa 5%
		5	SS	28		Si 51%
						Cl 44%
		6	SS	34	1000	
	Stiff to hard.	7	SS	57		
992.0		8	SS	104	990	
30.0	Sandy silt with traces of gravel and clay.	9	SS	105/6"		Gr 19%
						Sa 33%
	Very dense.	10	SS	75/6"	980	Si 39%
						Cl 9%
		11	SS	120/10"		
961.6		12	SS	100/4"	970	
50.4	End of borehole.					

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT %: 15 30 45

BULK DENSITY γ P.C.F.

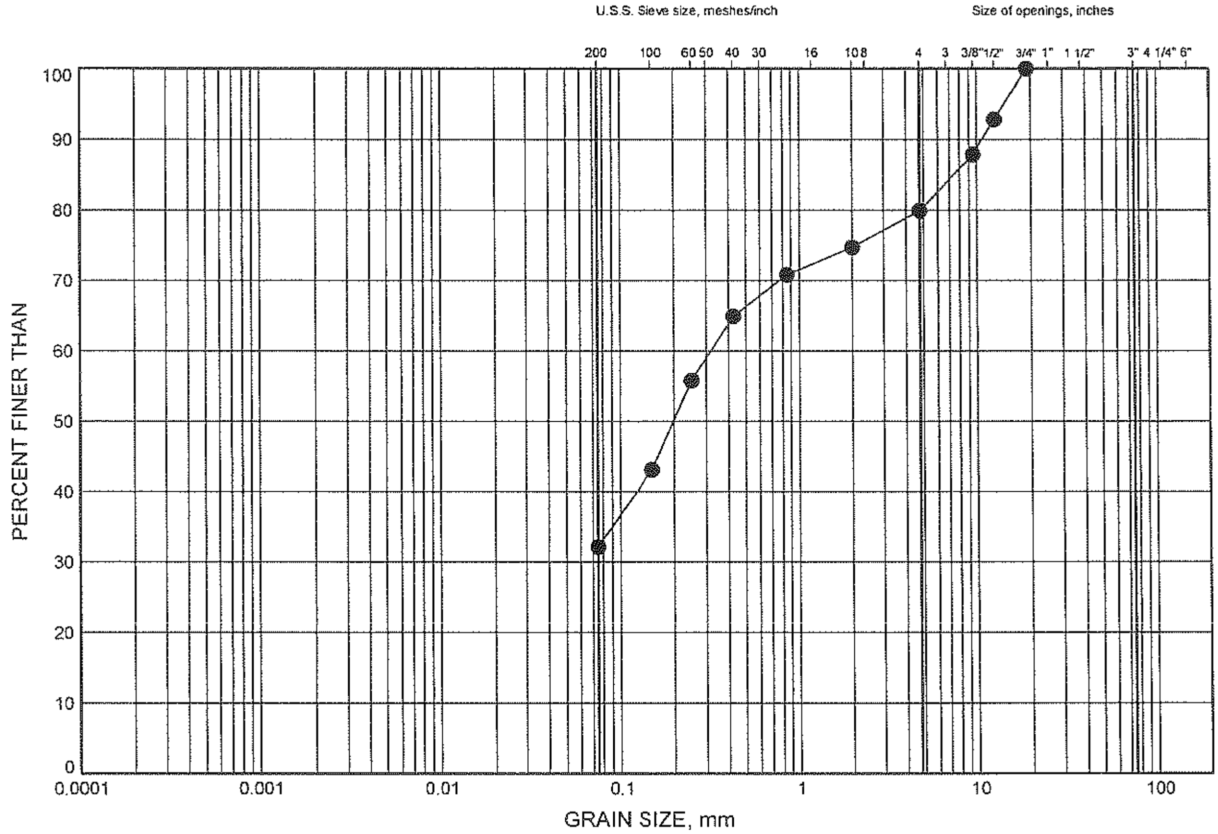


Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B1

Silty Sand Fill



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-003	1.07	311.43

W.P.# 408-88-00

Prepared By MFA

Checked By RPR

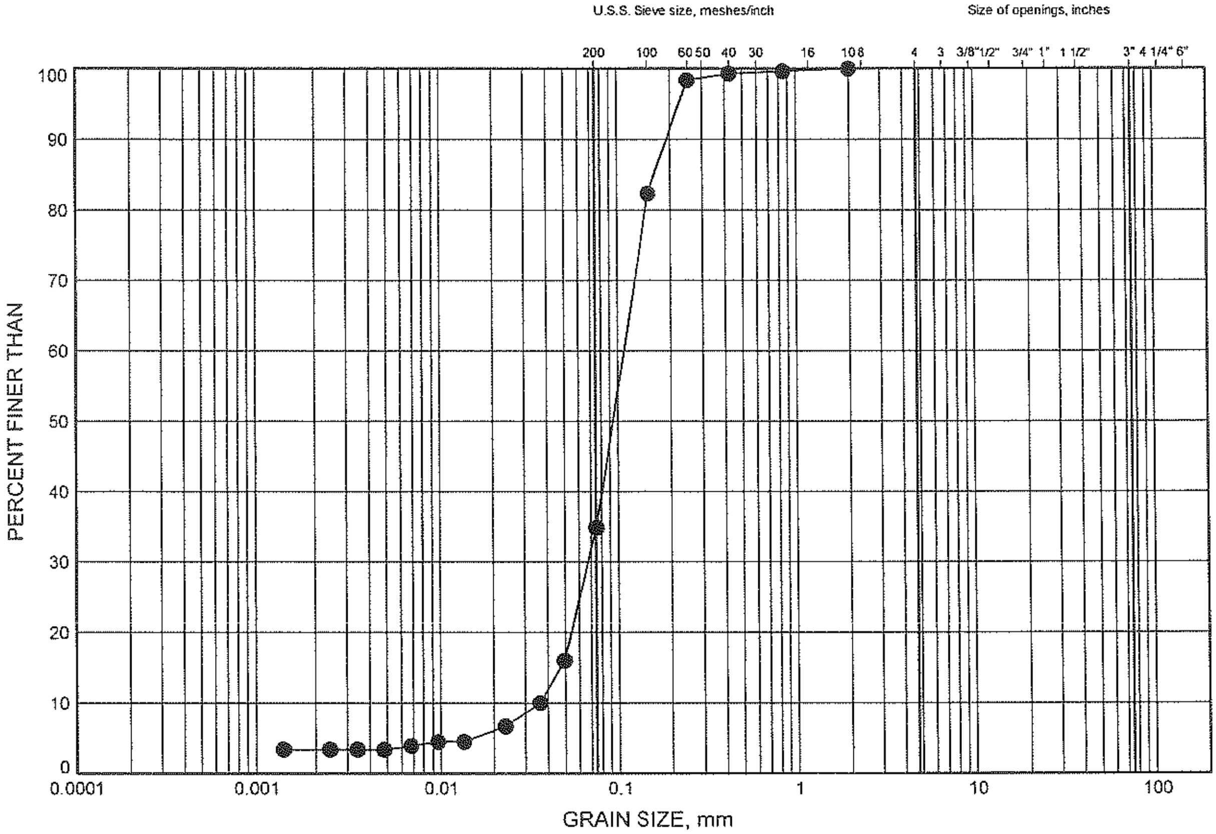
THURBER

GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 9/15/08

Highway 7 - New
GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-003	4.88	307.62

W.P.# 408-88-00
Prepared By MFA
Checked By RPR

THURBER

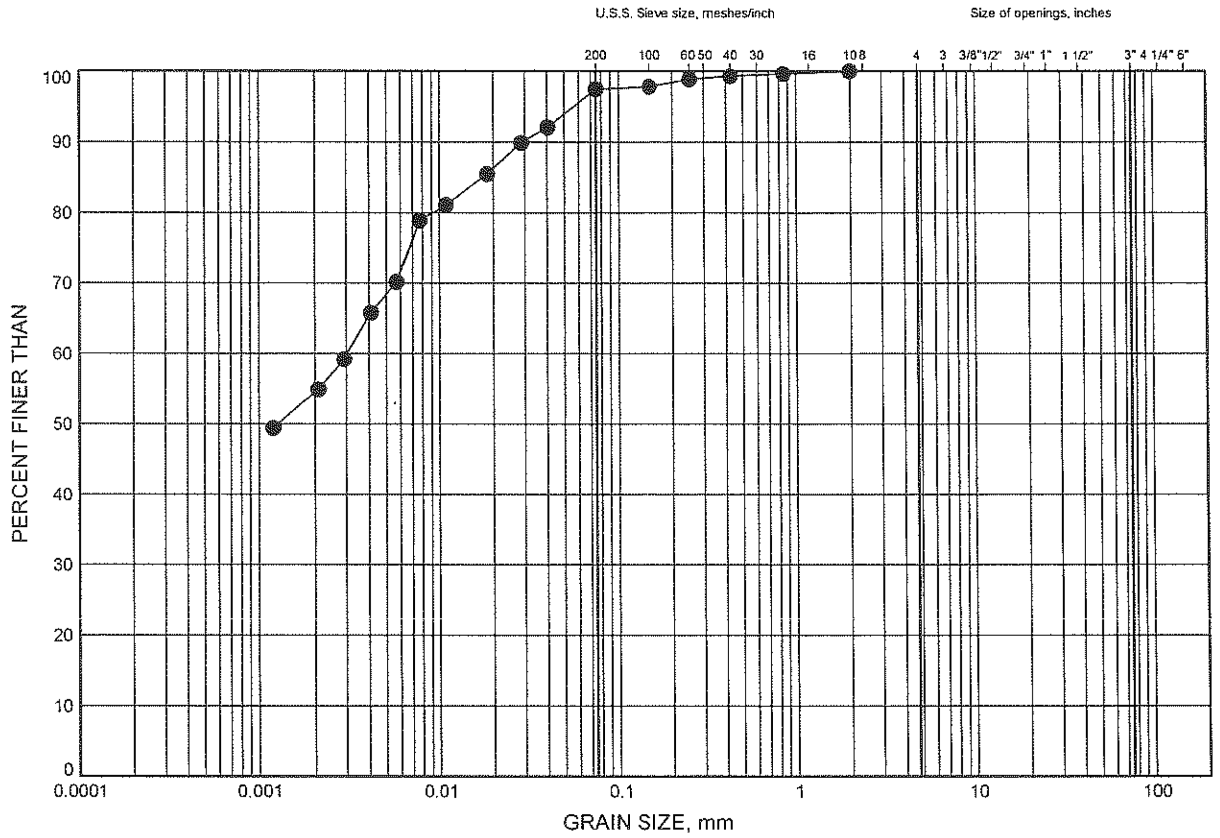
GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 9/15/08

Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty Clay Till



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-003	7.92	304.58

W.P.# 408-88-00
Prepared By MFA
Checked By RPR

THURBER

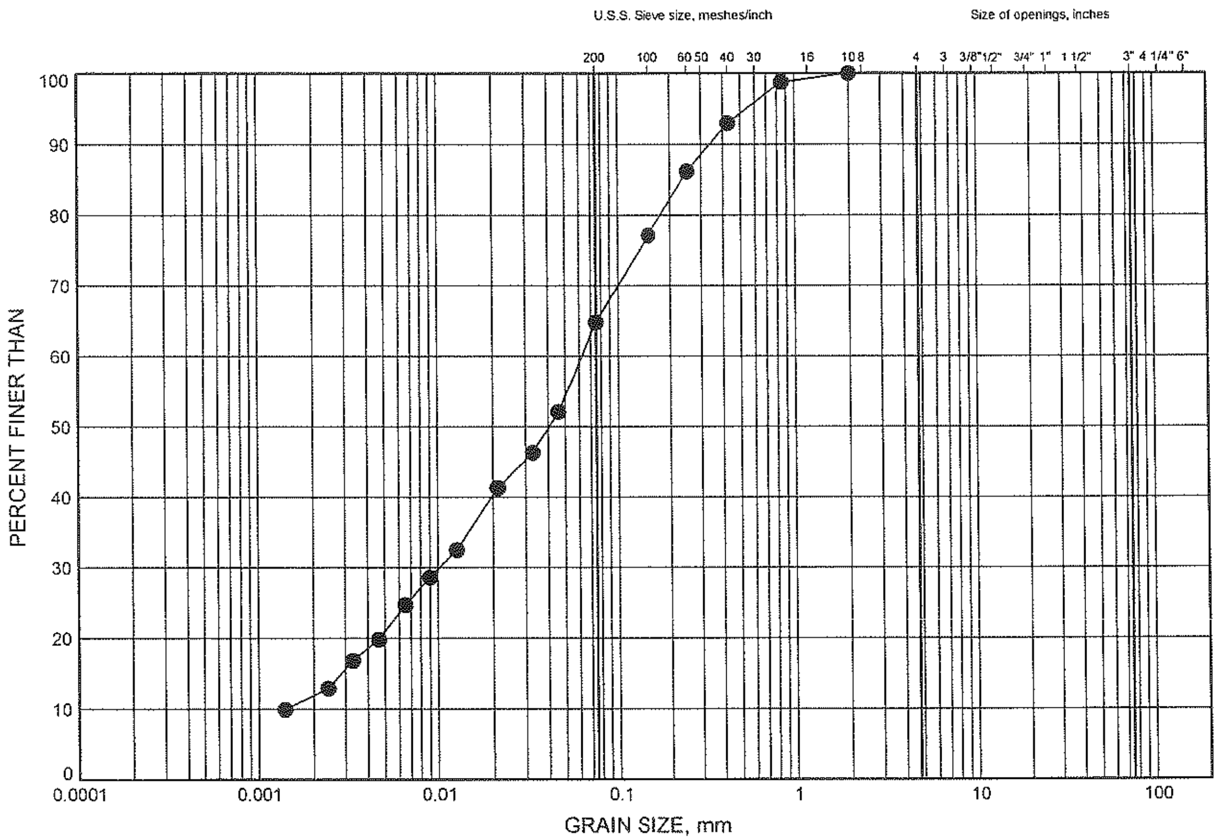
GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 9/15/08

Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B4

Sandy Silt Till



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-003	12.42	300.08

THURBER

W.P.# 408-88-00

Prepared By MFA

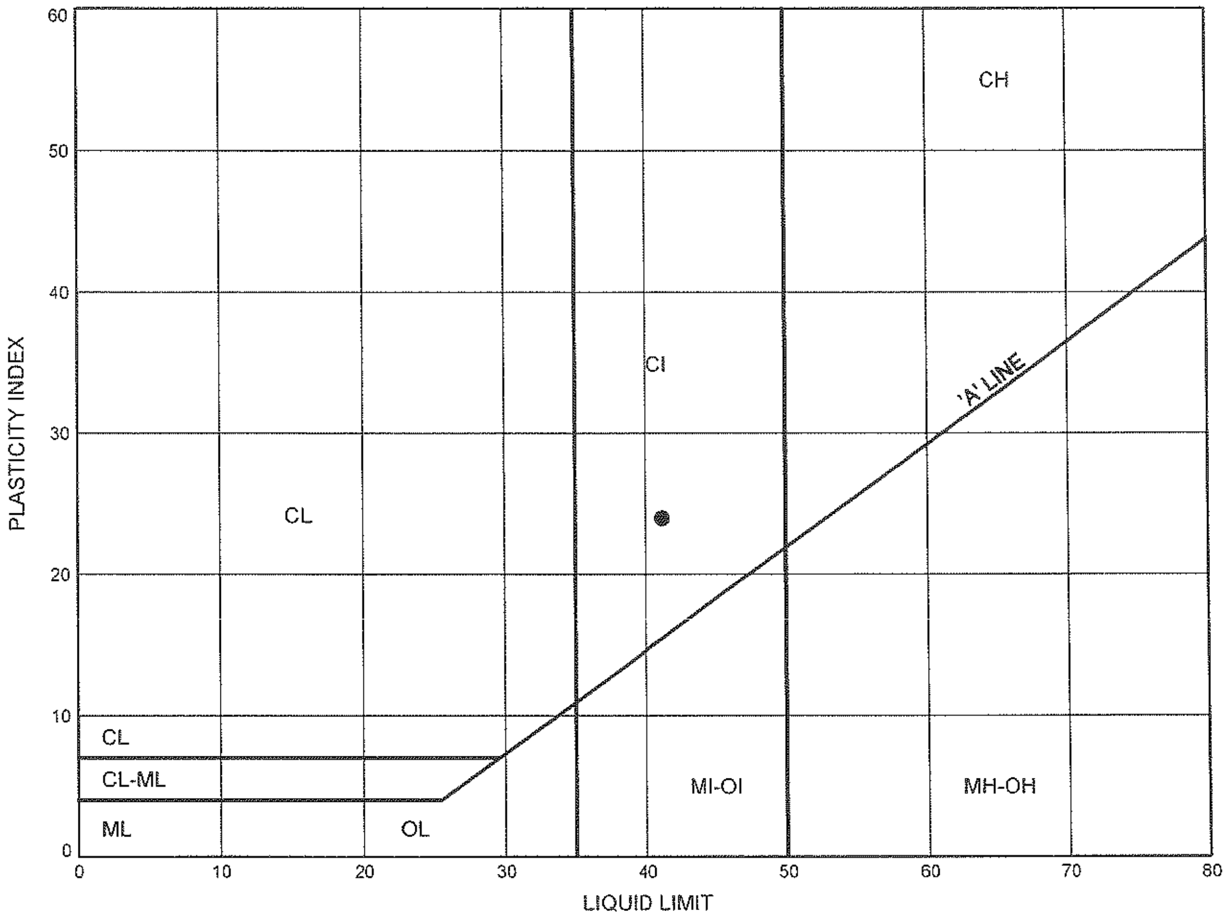
Checked By RPR

Highway 7 - New

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty Clay Till



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-003	7.92	304.58

THURBALT 6417R.GPJ 9/15/08

Date September 2008

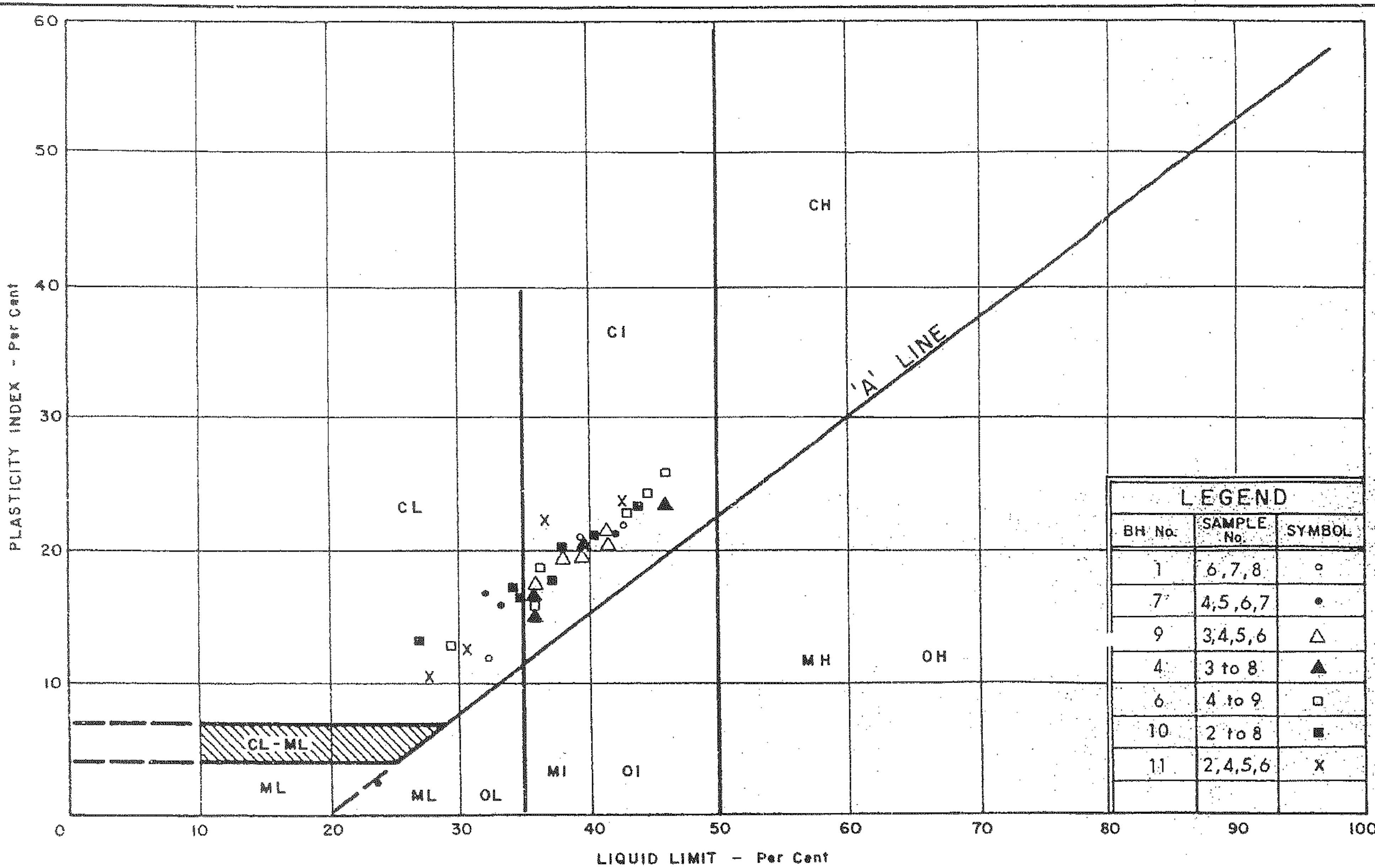
Project 408-88-00

THURBER

Prep'd MFA

Chkd. RPR

FORM OB-MT-252
(REV. JUNE 1965)
65-1624



LEGEND

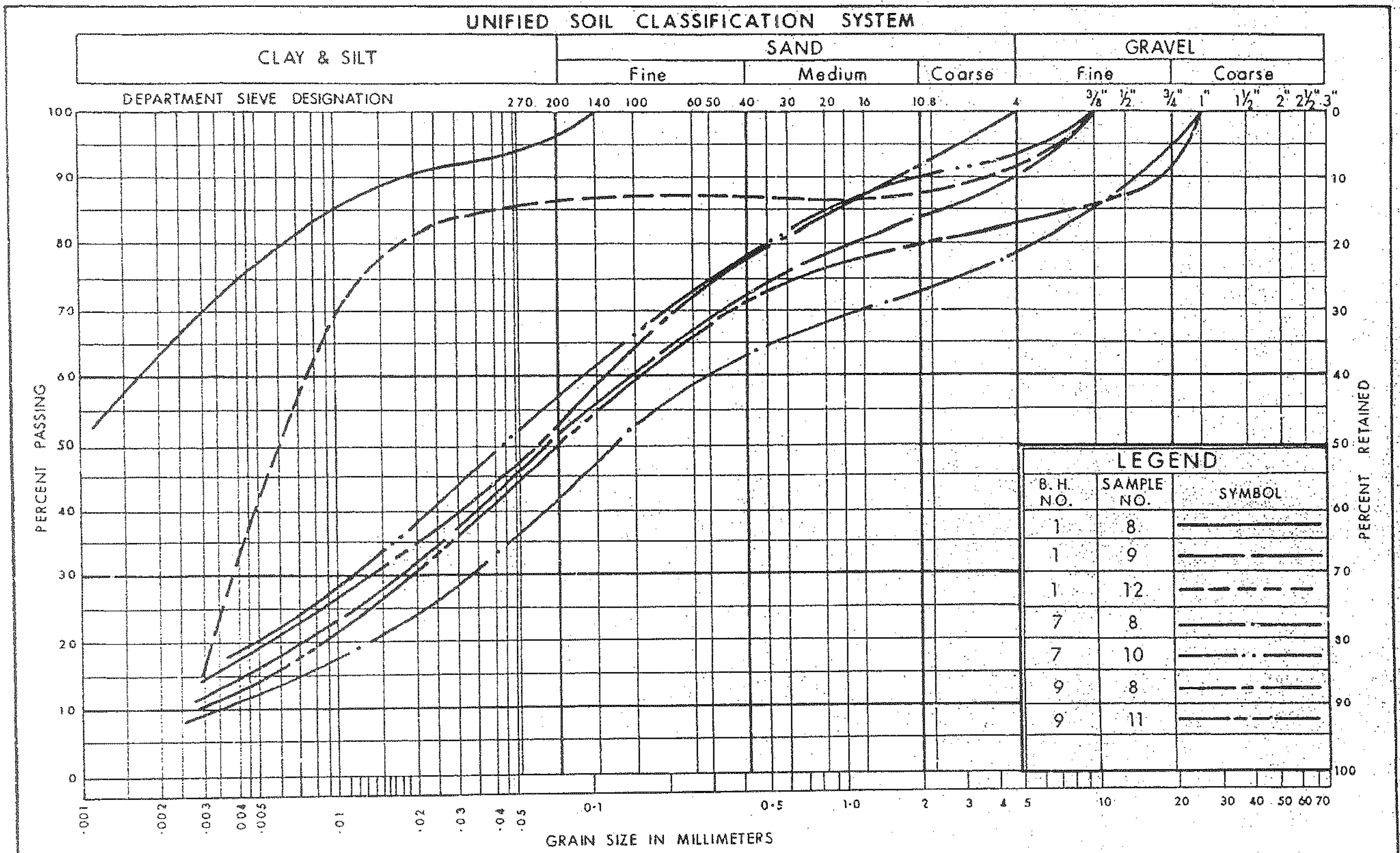
BH No.	SAMPLE No.	SYMBOL
1	6,7,8	○
7	4,5,6,7	•
9	3,4,5,6	△
4	3 to 8	▲
6	4 to 9	□
10	2 to 8	■
11	2,4,5,6	X

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

PLASTICITY CHART
SILTY CLAY
WITH TRACES OF SAND

W.P. No. 638-64
JOB No. 66-F-57

FORM OB MT-17
REV. APR. 1965



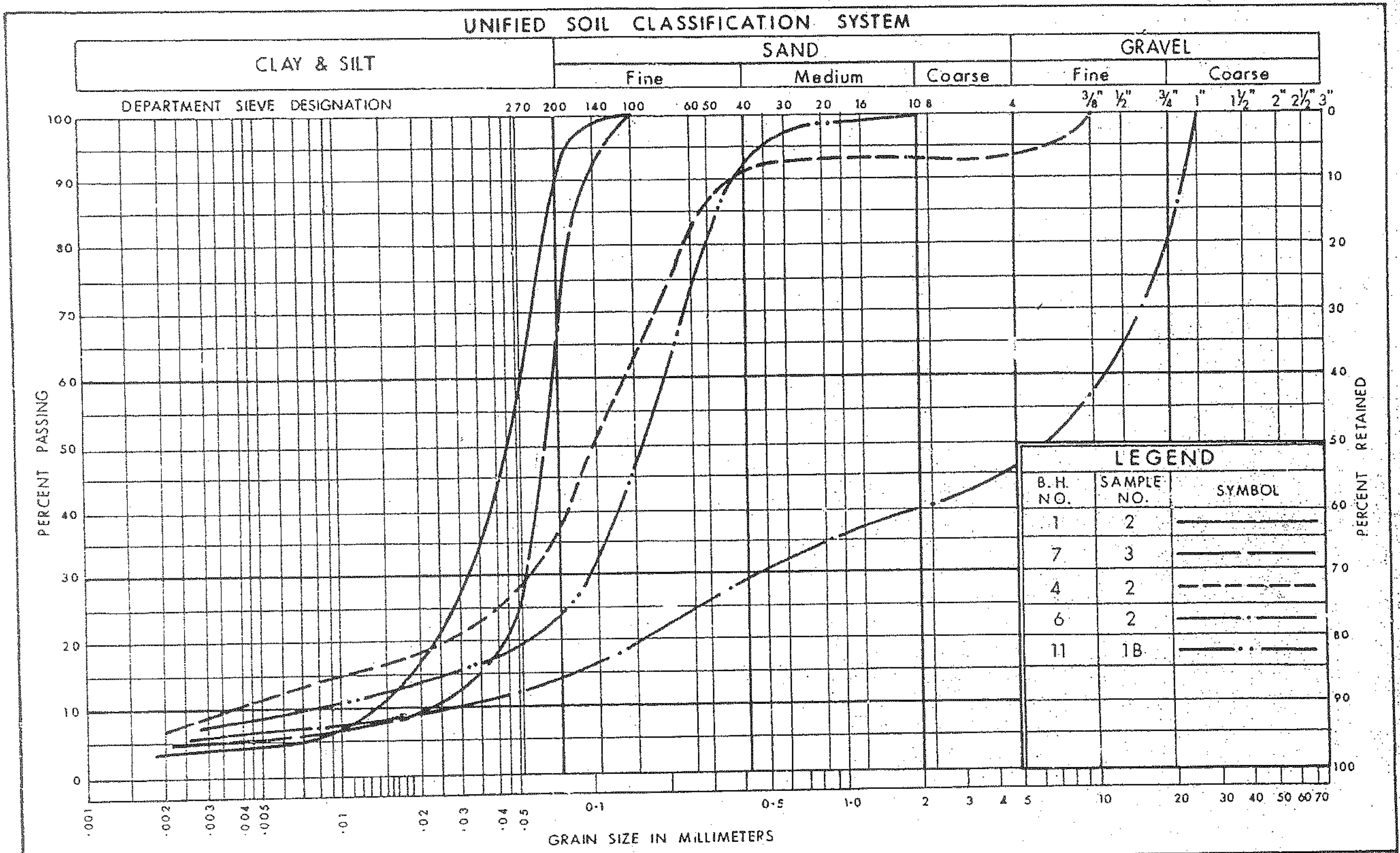
B.H. NO.	SAMPLE NO.	SYMBOL
1	8	
1	9	
1	12	
7	8	
7	10	
9	8	
9	11	

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

GRAIN SIZE DISTRIBUTION
SILTY SAND
WITH TRACES OF GRAVEL & SAND

W.P. No. 638-64
JOB No. 66-F-57

FORM OB MT-17
REV. APR. 1965



DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION ONTARIO	GRAIN SIZE DISTRIBUTION SANDY SILT TO SILTY SAND WITH TRACES OF GRAVEL & CLAY	W.P. No. 638-64
		JOB No. 66-F-57

GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILTY SAND
WITH TRACES OF GRAVEL & CLAY

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
for
GUELPH STREET OVERPASS EXTENSION AT HIGHWAY 85
HIGHWAY 7 & 85 IMPROVEMENTS
KITCHENER, ONTARIO
G.W.P. 3110-09-00

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PML Ref.: 10KF079
Index No.: 154FDR
GEOCRES No.: **40P8-230**
November 25, 2014

TABLE OF CONTENTS

FOUNDATION DESIGN REPORT
for
Guelph Street Overpass Extension at Highway 85
Highway 7 & 85 Improvements
Kitchener, Ontario
GWP 3110-09-00

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding the design and construction of the bridge foundations, RSS retaining walls and approach embankments for the proposed widening of the Guelph Street overpass on Highway 85 in the City of Kitchener, Ontario. 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).

Refer to Appendix FDR-D for the General Arrangement Drawing for the proposed work and for the RSS wall Drawing. The existing Guelph Street rigid frame overpass is to be widened to the east between Stations 19+510 and 19+530 along the proposed E-N Ramp from the New Highway 7 alignment. According to general arrangement drawings provided by MMM Group Limited (MMM) the 20 m wide single span bridge will be widened by approximately 6.5 and 8.0 m at the north and south abutments respectively. This widening will result in fills of about 5.3 and 5.6 m at the north and south abutments, respectively. The road grade of the proposed extension is expected to match the existing grade of Highway 85 near elevation 318.6 and 318.8 at the north and south abutments, respectively.

It is understood from MMM that the proposed bridge widening will comprise of a rigid frame structure to be consistent with the type of the existing bridge. Based on this, recommendations pertaining to integral and semi-integral abutments are not required and have not been included within this report.

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

In summary, the subsurface stratigraphy revealed in boreholes 101 and 102 drilled from the top of and through the existing Highway 85 embankment included the existing pavement structure over compact to very dense sandy fill to 6.4 and 7.2 m over dense to very dense sandy silt / silty sand locally above very stiff silty clay which extended to the 9.8 and 10.4 m borehole termination depths.

The boreholes 1, 7 and 08-003 were previously drilled from the existing Guelph Street level and were presented in Thurber's June 2009 report. In summary, these three boreholes revealed a compact to very dense silty sand / sandy silt which extended to 3.7 to 6.4 m, over stiff to hard clayey silt / silty clay (locally silty clay till) to 9.1 to 10.7 m, above a dense to very dense sandy silt to clayey silt (locally sandy silt till) which extended to the 13.5 to 15.4 m borehole termination depths.

Refer to Appendix FDR-C for a General Arrangement Drawing and a Layout of Footing Drawing for the existing bridge. The existing bridge is 44.5m long rigid frame with a span of 20m and a width of 65.8m founded on 324mm outside diameter piles driven to depths in the order of 10m.

Based on the Preliminary General Arrangement drawing provided by MMM, differential settlements between the widening and the existing bridge should be minimized. Consequently shallow spread footings on the native soils or placed engineered fill pads should not be used because of the possible settlement of the compact upper sandy zones of the subgrade. Rather, it is recommended that the abutments for the proposed bridge widening be founded on steel H-piles driven to practical refusal. Based on the soil stratigraphy encountered at the site it is considered feasible to support the proposed bridge widening on steel H-piles driven to practical refusal in the very dense sandy silt encountered at the north and south abutments as recommended in this report.

It is noted that the construction should avoid damaging the existing gas main running along Guelph Street through the widening area and is located approximately 4.5 m south of the face of the existing north abutment.

The seismic site coefficient for the bridge site is 1.0 [Soil Profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, Clause 4.4.6. 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 considered unlikely as well (clause 4.6.2 of CHBDC).

The foundation frost penetration depth in the Kitchener area is 1.4 m according to the MTO OPSD 3090.101.

2.2 Pile Foundation

The bridge extension should be founded on deep foundations to provide tolerable differential settlements between the existing bridge and the extension.

The following table indicates the advantages, disadvantages, risks/consequences and relative costs of considered alternatives for deep foundations.

Deep Foundation Type	Advantages	Disadvantages	Risks/Consequences	Relative Costs
driven steel H-piles	- conventional construction - superior performance in hard driving conditions	- vibrations	- pile driving vibration damage to existing bridge and gas main	- low
driven steel tube piles	- conventional construction	- vibrations - inferior performance in hard driving conditions	- pile driving vibration damage to existing bridge and gas main	- low
drilled shafts (caissons)	- high bearing resistance per unit	- high cost - more complex construction - higher potential of loss of ground	- loss of ground support causing settlement or distress to existing bridge and gas main	- high cost

The alternatives for deep foundations are driven steel H-piles (size 310HP110 or size 360HP152) or driven open ended steel tube piles subsequently filled with concrete (equivalent to size 324mm outside diameter). Drilled shafts (caissons) are not considered to be suitable for a bridge of this size and for consistency with the foundations for the existing bridge. Driven steel H-piles are preferred over driven steel tube piles because of their superior performance in hard driving conditions and to minimize the potential for vibrations during pile driving that could impact on the bridge or the gas main.

Steel H-piles are feasible to be used to support the foundation loads at both abutments. The piles should be driven through the sand / sandy silt fill material, native sandy silt, silty sand, clayey silt, silty clay and into the very dense lower sandy silt deposit. It is expected that the piles will be driven about 2.0 m into the very dense soils before meeting refusal and therefore have pile tip elevations near 300.0 and 301.2 at the north and south abutments, respectively. Based on this and assuming the top of the pile cap will match the top of the existing footing, elevation 311.3, pile lengths of approximately 11.3 and 10.1 m should be expected at the north and south abutments, respectively.

The following factored geotechnical axial resistances at Factored ULS and SLS for the sections of steel piles listed below are considered to be appropriate for piles driven to practical refusal on the very dense sandy silt deposit.

PILE SECTION	FACTORED GEOTECHNICAL AXIAL RESISTANCE AT ULS (kN)	GEOTECHNICAL AXIAL REACTION AT SLS (kN)
HP 310 x 110	1600	1400
HP 360 x 152	1800	1600

Downdrag loads on the piles are anticipated to be negligible at the site because of the relatively small settlements expected at the site from the embankment widening.

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 should be installed and monitored in accordance with the requirements of OPSS 903. Pile driving shoes should be used to protect the piles in view of the cobbles and boulders present in the underlying glacial till soils.

A NSSP should be prepared to advise the contractor of the potential presence of boulders at this site. The recommended NSSP is attached in Appendix FDR-A.

Pile installation should consist of preaugering to elevation 309.0, which is below the invert of the gas main, prior to pile driving. Vibration monitoring is required for the gas main but not for the bridge as the vibration monitoring results from the gas main will be applicable to the existing bridge foundations. Vibration monitoring and settlement monitoring are required for both the gas main and for both abutments of the bridge. The pile driving note on the contract drawings should indicate that piles should be installed in preaugered holes to elevation 309.0, then driven to minimum depth equivalent to elevation 303.0m and then controlled by the Hiley Formula assuming an ultimate load of 2 x Factored Geotechnical Axial Resistance at ULS used in the design.

Peak Particle Velocities generated from pile driving activates should not exceed 50 mm/sec at the exposed gas main.

Pile caps should be provided with at least 1.4 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.

Although there was little direct evidence of their presence during drilling, glacial till deposits inherently contain cobbles and boulders. Hence, it is possible that a pile will achieve refusal at a higher elevation than anticipated due to encountering a cobble or boulder. If it is suspected that this is happening, the QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it should be referred to the design team for resolution.

If a pile fails to develop a specified resistance after being driven 2m beneath the anticipated pile tip elevation, driving of that pile should be immediately stopped and the Hiley formula calculation should be checked including all input values. If the Hiley calculations still indicate that the pile has not reached the specified resistance, the following procedure should be implemented:

a) Stop driving in that pile group for 48 hours (minimum).
b) After 48 hours, warm up the hammer on another pile and then restrike the subject pile and immediately take Hiley readings.
c) If the pile still does not reach the specified resistance, pile driving for that pile must stop and the QVE must immediately advise the CA who, in turn, should refer the issue to the design team.

2.2.1 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 / SILTY CLAY		**NATIVE SANDY SILT / SILTY SAND**	
Pile Section	HP 310	HP 360	HP 310	HP 360
Factored Lateral Resistance at ULS, kN	200	240	110	150
Lateral Resistance at SLS, kN	110	140	40	50

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

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

<u>Cohesionless (embankment fill):</u>

k_s = n_h z/b

n_h = coefficient related to soil density

= 12 MN/m^3 for granular fill

= 8 MN/ m^3 for compact to dense silty sand / sandy silt

z = depth, m

b = pile width, m

<u>Cohesive (clayey silt / silty clay):</u>

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

c_u = undrained shear strength of cohesive material

= 200 kPa for very stiff to hard clayey silt / silty clay

b = pile width, m

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters/widths. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

PILE SPACING IN DIRECTION OF LOADING d = PILE DIAMETER OR WIDTH	SUBGRADE REACTION REDUCTION FACTOR, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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 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 material above the top of the wall can be treated as a surcharge load (q in the preceding equation).

A weeping tile system (OPSS 405 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 Drawings for granular backfill at abutments (OPSD 3101.150).

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) is proposed to be constructed east of the proposed bridge widening section. A high performance, high appearance rated RSS wall should be employed.

Slope stability analysis for geometry with 5m high vertical wall supporting 2m high surcharge fill sloped at 2H:1V indicates an acceptable global safety factor of well over 1.5. Refer to the conceptual slope stability analysis in Appendix FDR-B for justification. Since the geometry analyzed is the most critical that would exist at the site, other slope geometries such as 2H:1V will yield higher safety margins.

The internal stability of the RSS will be the responsibility of the RSS supplier. 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.

An RSS wall supported on the compact to very dense sandy silt / silty sand at a 0.5 m (elevation 312.7 and 311.0 in boreholes 1 and 7, respectively) may be designed using a factored bearing resistance at ULS of 150 kPa and geotechnical reaction at SLS of 100 kPa.

Prior to placement of structural concrete and footing construction, 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 OPSS 902 should be followed.

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 magnitude.

The earth pressure coefficients provided above for granular materials and those given below for the compact to very dense sandy silt / silty sand are appropriate for the RSS wall.

Parameters	**Compact to Very Dense Silty Sand / Sandy Silt**
Angle of Internal Friction, degrees	30
Unit Weight, kN/m^3	20.0
Coefficient of Active Earth Pressure K_a	0.33
Coefficient of Earth Pressure At-Rest K_o	0.5
Coefficient of Passive Earth Pressure K_p	3

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.

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 the NSSP for RSS Walls in Appendix and for SP 599S23 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

4.1 General

It is anticipated that the approach embankments will be up to about 5.0 m high at the location of the widening. It is also assumed that the new embankments will be constructed using granular material that will be benched into the existing embankment in accordance with OPSD 208.010.

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 native inorganic soil.

Embankment fill should be placed and compacted in accordance with OPSS.PROV 206 and OPSS 501. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill.

It is considered that the approach embankments constructed on the existing compact to very dense soils in accordance with the foregoing recommendations will be stable and deliver a factor of safety over 1.3. The justification for this assessment is the slope stability analysis for the more severe geometry at the proposed RSS walls.

4.2 Embankment Settlements

Settlements resulting from the approximately 5 m high approach embankments should be expected as a result of consolidation of the new embankment fill and the underlying native compact to very dense sandy silt / silty sand and very stiff to hard clayey silt / silty clay.

The estimated magnitude of settlement of new granular material is in the order of 25 mm and the anticipated settlement of the underlying native material is expected to be about 15 to 20 mm. Therefore the total settlement at the proposed approach embankments is anticipated to be approximately 40 to 45 mm.

It is expected that most of this settlement will take place during or immediately following completion of the construction. Long term and differential settlements including transverse settlements are not expected to exceed to maximum allowable settlements referenced in the MTO's "Embankment Settlement Criteria for Design', dated July, 2010.

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 this purpose, the very stiff to hard clayey silt / silty clay are classified as Type 2 soils and the compact to very dense sand fill and silty sand / sandy silt are classified as a Type 3 soils 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.

In order to maintain traffic on Highway 85 it is expected that temporary roadway protection will be used during the extension of Highway 85 overpass. Temporary protection is feasible and should be constructed in accordance with OPSS 539. A minimum performance level of 2, according to OPSS 539 is recommended. The contractor is responsible for selection, preparation of a detailed design and performance for the roadway protection system.

The possibility of the existing granular fill migrating through the temporary roadway protection and difficulties associated with the presence of cobbles and boulders within the sandy silt, silty sand and possibility of cobbles and boulders within the glacial till soils encountered at the site should be considered by the contractor during the selection and installation of the temporary protection. A Non-Standard Special Provision (NSSP) should be added to the contract documents. The recommended NSSP is attached in Appendix FDR-A.

The following soil parameters may be assumed for shoring above the groundwater elevation:

Parameters	Compact to Very Dense Silty Sand / Sandy Silt
Angle of Internal Friction, degrees	30
Unit Weight, kN/m^3	20.0
Coefficient of Active Earth Pressure K_a	0.33
Coefficient of Earth Pressure At-Rest K_o	0.5
Coefficient of Passive Earth Pressure K_p	3

5.2 Groundwater Control

In the process of augering, water strikes were observed at of 4.0 and 8.5 m (elevations 314.8 and 309.9) in boreholes 101 and 102 respectively. Upon completion of augering, groundwater was measured in boreholes 101 and 102 at 4.9 and 7.9 m (elevation 313.9 and 310.5), respectively. In the previous boreholes completed at the site, boreholes 1, 7 and 08-003 water levels were observed during drilling at 0.3 to 2.7 m (elevations 309.8 to 312.8).

The Contractor should be responsible for lowering the prevailing groundwater elevation at the time of construction a minimum of 0.3 m below all excavations. It is anticipated that groundwater can be controlled through conventional sump pumping. Since the embankment is well above the prevalent water table, a Permit-To-Take-Water (PTTW) is not considered to be required.

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

6. CLOSURE

This Foundation Design Report was prepared by Mr. A. DeSira, MEng, P.Eng., revised to address MTO comments by D. Dundas, P.Eng. and reviewed by Mr. C. M. P. Nascimento, P. Eng., Project Manager and MTO Designated Principal Contact.

Yours very truly

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TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS.PROV 206	Construction Specification for Grading
OPSS 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 539	Construction Specification For Temporary Protection Systems
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
SP 599S23	Requirements for Materials, Quality Control and Quality Assurance Testing and Acceptance Criteria for Precast Concrete Facing Elements Including Panels
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail

APPENDIX FDR-A

Non-Standard Special Provisions (NSSP's)

DRAFT NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP – Potential for Cobbles and Boulders During Pile Driving

The Contractor shall be advised that cobbles and boulders were identified within the sandy silt / silty sand deposit at the site and that although no cobbles and boulders were encountered in the glacial till deposits the possibility of cobbles or boulders within the glacial till deposits should be considered. The contractor shall provide comprehensive pile driving supervision.

If there is evidence that a pile meets refusal on a cobble or boulder during pile driving, the contractor shall inform the Contract Administrator. The contractor shall be advised that piles meeting refusal on a cobble or boulder may need to be relocated, have their capacity reduced and / or require additional piles to be installed.

NSSP – Temporary Roadway Protection

The possibility of the existing granular fill migrating through the temporary roadway protection and difficulties associated with the presence of cobbles and boulders within the sandy silt, silty sand and possibility of cobbles and boulders within the glacial till soils encountered at the site should be considered by the contractor during the installation of the temporary protection systems.

NSSP – Monitoring of the Existing Bridge and Gas main

The possibility of damage to the existing bridge and gas main due to vibration caused by pile driving activities for the proposed structure shall be considered by the contractor during construction. Vibration of the exposed existing gas main in the vicinity of the pile driving operations and vibration and movement of the existing bridge in the vicinity of the pile driving operations should be monitored during pile driving.

The contractor shall halt pile driving and inform the Contract Administrator if vibrations with a peak particle velocity greater than 50 mm/sec are measured or if movements of settlement monuments on the bridge exceed 10mm.

NSSP – RSS Wall

RETAINED SOIL SYSTEM, TRUE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, FALSE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, HIGH PERFORMANCE - Item No.
BACKFILL FOR RETAINED SOIL SYSTEM, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
BACKFILL FOR RETAINED SOIL SYSTEM, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, LOW PERFORMANCE - Item No.
BACKFILL FOR RETAINED SOIL SYSTEM, LOW PERFORMANCE - Item No.

Non Standard Special Provision January, 2008

1.0 SCOPE

This special provision covers the requirements for the design and construction of Retained Soil Systems (RSS) walls and steep slopes.

Additional requirements for RSS precast concrete facing elements shall be as specified in the Contract documents.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 102 Weighing of Materials
OPSS 180 Management and Disposal of Excess Materials

- **Ontario Provincial Standard Specifications, Construction**

OPSS 501 Compacting

Canadian Standards Association Standards:

CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC)

- **Ministry of Transportation Publications:**

MTO Designated Sources of Materials (DSM)
Qualification Criteria for RSS

3.0 DEFINITIONS

For the purposes of this special provision the following definitions apply:

Alignment Elements: means components specified by the manufacturer that are constructed on the foundation for RSS to facilitate placing of the facing elements to the correct lines and grades, such as concrete levelling pads and soldier piles.
Approved Product Drawings: means the documentation for an RSS that has been submitted by the manufacturer and accepted by the Ministry for listing in the DSM, according to the Qualification Criteria for RSS.

Backfill for RSS: means the material specified by the manufacturer as part of the engineered materials comprising the backfill for the RSS.

Constructed Height: means the vertical distance between the foundation for RSS and the top of the currently placed and compacted backfill for RSS, measured at the point of the design height.

Corrective Work: means work carried out by the Contractor to repair deficiencies identified by the Owner during the RSS warranty period.

Design Checking Engineer: means the Engineer retained by the Contractor who checks the original design and working drawings.

Design Engineer: means the Engineer retained by the Contractor who produces the original design and working drawings.

Design Height: means the maximum difference in elevation between the foundation for RSS and the corresponding top of backfill for RSS, over the full length or perimeter of the RSS.

External Stability: means stability against deep-seated failure of the foundation for RSS, including adequate bearing capacity at specified settlements of the foundation.

Facing Elements: means components specified by the manufacturer that delineate the front face of the RSS and to which reinforcing elements may be attached, such as precast concrete panels, split-face concrete blocks, and geo-synthetic panels.

Foundation for RSS: means the base on which the RSS is constructed, such as excavation to a specified elevation and construction of a granular ‘A’ pad.

Internal Stability: means stability against failure of the engineered materials comprising the RSS, including adequate resistance against excessive elongation, breakage and pullout of the reinforcing elements.

Manufacturer: means the firm who supplies the design and proprietary components, and who specifies the backfill and other materials, for the RSS selected by the Contractor.

Manufacturer's Representative: means an individual with continuous full-time employment with the manufacturer for a period of at least three (3) years, and who is knowledgeable in the design and construction of the RSS selected by the Contractor.

Obstruction: means any part of the work and any existing condition within the Contract limits that affects the design, construction and performance of the RSS, such as structures, catch basins and manholes, drainage pipes and sewers, and utilities.

Performance Tolerance – Local: means the joint gap between any two constructed facing elements, measured at any point along the joint between the facing elements and perpendicular to the line of the joint.
Performance Tolerance – Global: means the vector distance between any point on the constructed RSS and the corresponding point on the theoretical RSS surface as defined in the Contract documents.

Placing Tolerances: means tolerances specified by the manufacturer on the placing of the RSS components and backfill for RSS to ensure compliance of the constructed RSS with the performance tolerances.

Reinforcing Elements: means components specified by the manufacturer that are placed within the backfill for RSS and connected to the facing elements to mechanically stabilize the backfill for RSS, such as metal tie strips, metal grids and geo-synthetic grids,

Retained Soil System (RSS): means a proprietary system listed in the DSM used to retain horizontal loads for applications such as true and false abutment structures, retaining walls and steep slopes; or, to retain vertical loads for applications such as embankments over soft ground.

RSS Superintendent: means the Contractor's authorized representative in responsible charge of the construction of the RSS.

Structure: means any bridge, culvert, tunnel, retaining wall, overhead sign, high mast light pole, wharf, dock, or any part thereof.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submissions

4.1.1 Working Drawings

The Contractor shall submit working drawings for all RSS. A separate submission shall be made for each RSS in the Contract. All submissions shall bear the seal and signature of the Design Engineer and the Design Checking Engineer.

The RSS Superintendent shall have a copy of the working drawings on site at all times during the construction of the RSS.

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

4.1.2 Working Drawing Requirements

Working drawings shall include at least the following:

- Statement from the manufacturer confirming the experience and expertise of the Design Engineer and Design Checking Engineer to provide design services for the manufacturer's RSS;
- All design, fabrication and construction drawings and specifications for the RSS;
- Location and value of the design height of the RSS;
- Defined lines and grades, type, and quantity in m^3 of the backfill for RSS;
- Details at obstructions, and connections to other structures, where shown in the Contract drawings;
- Statement of bearing resistance required by the RSS foundation according to the CHBDC;
- Statement of satisfactory internal and external stability;
- Placing tolerances for the RSS.

4.1.3 RSS Superintendent

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit in writing to the Contract Administrator the name(s) of the RSS Superintendent for each RSS in the Contract.

During construction of an RSS, the Contractor shall not change the RSS Superintendent for that RSS without written permission from the Contract Administrator. The Contractor shall submit in writing to the Contract Administrator the proposed change for RSS Superintendent at least one week prior to the actual change in RSS Superintendent.

4.1.4 Manufacturer's Representative

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit in writing to the Contract Administrator the name(s) of the manufacturer's representative for each RSS in the Contract.

For each occasion the Contractor arranges for the manufacturer's representative to be on site, the Contractor shall submit 48 hours advance notice in writing to the Contract Administrator giving the dates and locations the manufacturer's representative will be on site.

4.1.5 Certificates of Conformance

For each RSS in the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the QVE upon completion of the RSS.

4.1.6 Milestone Inspection

For each RSS in the Contract, the Contractor shall submit to the Contract Administratora Milestone Inspection Report following an Interim Inspection by the QVE at each of the following milestones, and prior to commencement of subsequent operations on that RSS:

a) Layout and marking of all lines and grades needed to construct the RSS; and construction of the alignment elements, where applicable;
b) Delivery and storage on site of facing elements and reinforcing elements, where applicable;
c) Installation of the facing elements; placement and compaction of the backfill for RSS; and installation of the reinforcing elements, where applicable;

For RSS where the design height is greater than 5.0 m, the Contractor shall submit a series of Written Permissions to Proceed for milestone c) corresponding to the constructed height of the RSS at 5.0 m, 10.0 m, and 15.0 m, as applicable, up to and including the design height.

The Milestone Inspection submissions in no way supersede the inspection and testing intervals required for the construction of the RSS, as specified in the working drawings.

4.1.7 RSS Warranty

The Contractor shall submit a warranty to the Owner to address all deficiencies identified by the Owner related to the performance of the RSS for a period of 36 months from the date of certification of completion of the Contract.

4.1.8 Repair Procedures for Corrective Work

At least two weeks prior to commencement of any corrective work at an RSS during the warranty period, the Contractor shall submit to the Manager of Contracts, for information purposes only, three copies of his repair procedures for that RSS.

The repair procedures shall include a description of the cause and fully detail the corrective work required to correct the deficiencies identified by the Owner.

The repair procedures shall bear the seal and signature of an Engineer (who may be different than the Design Engineer and Design Checking Engineer), and be signed by the manufacturer’s representative.

4.2 Design

4.2.1 General

The Contractor shall be responsible for the design of the RSS and for ensuring the RSS as designed is compatible with the work.

The geometric requirements of the RSS, such as lines and grades of the facing elements and typical cross-sections, shall be as specified in the Contract drawings.

The foundation for RSS shall be as specified in the Contract documents.

4.2.2 RSS Selection

The Contractor shall select an RSS from the DSM that meets the Application, Performance and Appearance requirements for that RSS, as specified in the Contract drawings.

The Contractor shall select an RSS from the DSM designated as either ‘A’ (Accepted) or ‘DE’ (Demonstration). RSS designated as ‘DE’ status require inspection, instrumentation and monitoring of the constructed RSS, and reporting of the findings to the Ministry by the manufacturer, according to the Qualification Criteria for RSS.

Where there is more than one RSS in the Contract, the Contractor shall select the RSS from the same DSM listing, including type and colour of facing elements, according to the following groupings:

a) All RSS covered under the same tender item number(s) for payment;

b) All RSS with the same Performance and Appearance requirements that abut the same structure, existing and/or part of the work.

4.2.3 Performance Tolerances

Performance tolerances for the RSS shall be according to Table 1.

• **TABLE 1 – PERFORMANCE TOLERANCES FOR RSS**		
Performance Requirement	Performance Tolerance (mm)	
	Local	Global
Abutments	Joint Gap[1] ± 5	≤ 20
High	Joint Gap[1] ± 10	≤ 30
Medium	N/A	≤ 50
Low	N/A	≤ 100

Note 1.: Joint Gap shall be as specified in the working drawings.

4.2.4 Obstructions

The Contractor shall be responsible for developing design details of the RSS at obstructions, for all obstructions shown in the Contract drawings.

Where an obstruction is shown in the Contract drawings but not located to sufficient accuracy for the design of the RSS, the Contractor shall locate the obstruction in the field to sufficient accuracy as required to design the RSS.

4.2.5 Foundation Report

A Foundation Investigation Report that describes the subsurface conditions at the RSS is available, as specified in the Contract documents.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

5.0 MATERIALS

5.1 General

All materials for the selected RSS shall be according to the Approved Product Drawings for that RSS.

6.0 EQUIPMENT

6.1 Restriction on Skid-Steer Vehicles

Skid-steer vehicles will not be permitted on any area where the depth of backfill for RSS over installed reinforcing elements is less than 0.5 m.

7.0 CONSTRUCTION

7.1 General

The RSS shall be constructed according to the working drawings and this Special Provision.

Construction of the RSS shall not commence until the Contractor has submitted all applicable Certificates of Conformance for the foundation for RSS.

7.2 RSS Superintendent

The Contractor shall schedule his operations such that the construction of an RSS is at all times under the responsible charge of an RSS Superintendent who has been advised on site by the manufacturer's representative as to the required procedures for the construction of that RSS, for the specified operations and time periods.

7.3 Manufacturer's Representative

The manufacturer's representative shall be on site to advise the RSS Superintendent as to the procedures and placing tolerances required for the construction of the RSS.

For each RSS in the Contract, the Contractor shall arrange for the manufacturer's representative to be on site at commencement of each of the following operations, for a time period of three (3) working days per operation or until the operation is complete, whichever is less:

a) Layout of the RSS; and construction of the alignment elements, where applicable;
b) Installation of the facing elements;
c) Placement and compaction of the backfill for RSS; and installation of the reinforcing elements, where applicable.

Whenever there is a change in the RSS Superintendent during construction of an RSS, the Contractor shall arrange for the manufacturer's representative to return to the site for the same operations and time periods as at commencement.

7.4 Backfill for RSS

Backfill for RSS shall be placed within the lines and grades shown on the working drawings. All backfill for RSS shall be compacted according to OPSS 501.

Unless otherwise shown in the Contract drawings, the Contractor shall not place backfill for RSS against an adjacent concrete structure that is part of the work until the concrete in that structure has obtained a compressive strength at least 70% of the concrete strength specified in the Contract.

7.5 Management of Excess Materials

Management of excess materials shall be according to OPSS 180.

7.6 Corrective Work

At least one week prior to commencement of any corrective work at an RSS during the warranty period, the Contractor shall submit written notice of commencement to the Manager of Contracts.

The Contractor shall repair all deficiencies according to the repair procedures for corrective work. All corrective work shall be done within the RSS warranty period, unless prevented by seasonal shutdown, in which case the corrective work shall be done during the first eight weeks of the following construction season.

The Contractor shall provide access to the corrective work for inspection by the Owner when requested.

8.0 QUALITY ASSURANCE

8.1 Acceptance Criteria at End of the RSS Warranty Period

The Owner will accept the RSS at the end of the RSS warranty period if none of the deficiencies listed in Table 2 are found during the warranty inspections. Where deficiencies are found, the RSS will not be accepted until the Contractor has carried out corrective work to repair the deficiencies.

TABLE 2 – RSS DEFICIENCIES	
Number	Description of Deficiency
1.	Performance tolerance exceeds tolerances given in Table 1.
2.	Damaged facing elements and damaged alignment elements, where applicable.
3.	Dead and dying vegetative elements that are an integral part of the RSS.

8.2 Warranty Inspections

Throughout the warranty period the Owner will carry out warranty inspections of the RSS for deficiencies as per Table 2. The Owner will notify the Contractor as to the date and time of the inspection(s) and the Contractor may, at his discretion, be present during the inspection(s).

Within two weeks following a warranty inspection the Owner will notify the Contractor in writing of all deficiencies that require corrective work.

9.0 MEASUREMENT FOR PAYMENT

9.1 Actual Measurement

9.1.1 Backfill for Retained Soil System, High Performance
Backfill for Retained Soil System, Medium Performance
Backfill for Retained Soil System, Low Performance

Measurement will be of the mass in tonnes of the material placed within the theoretical lines and grades shown in the stamped working drawings. The method of determining the mass shall be according to OPSS 102.

10.0 BASIS OF PAYMENT

10.1 Retained Soil System, True Abutment - Item
Retained Soil System, False Abutment - Item
Retained Soil System, Wall/Slope, High Performance – Item
Retained Soil System, Wall/Slope, Medium Performance – Item
Retained Soil System, Wall/Slope, Low Performance – Item

Payment at the contract price for the above tender items shall be full compensation for all labour, equipment and material to do the work, including all costs associated with the manufacturer's representative on site.

Payment for construction of the foundation for RSS will be made under the appropriate tender items in the Contract.

No payment will be made for corrective work, including investigation of deficiencies, design of repairs, site access, traffic staging and removal of existing work, except where the corrective work is required as a result other than an act or fault of the Contractor.

10.2 Backfill for Retained Soil System, High Performance – Item
Backfill for Retained Soil System, Medium Performance – Item
Backfill for Retained Soil System, Low Performance – Item

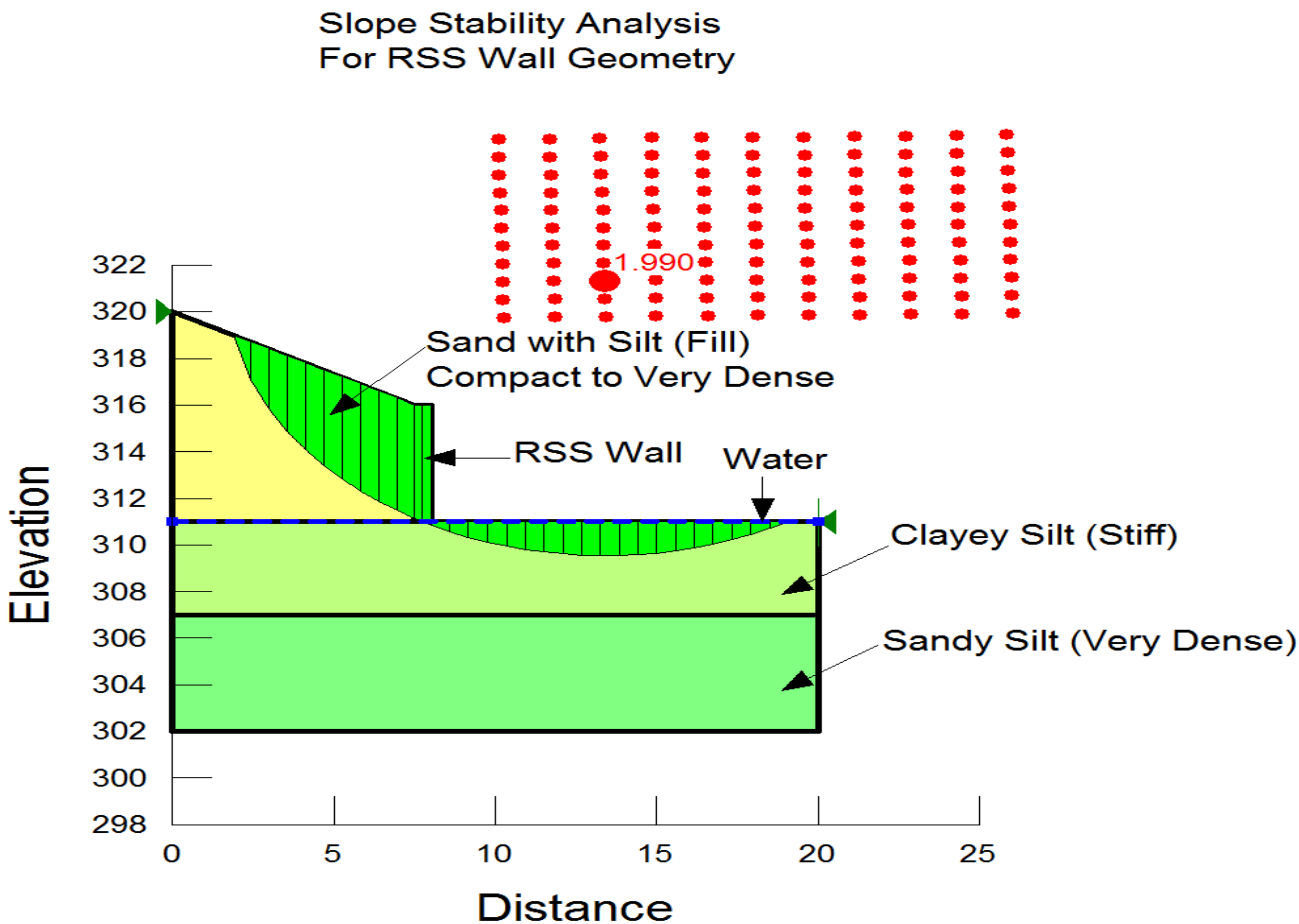
Payment at the contract price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

When the Contract does not contain a separate tender item for backfill for RSS, the contract price for the RSS contract items in which the backfill for RSS is incorporated shall include full compensation for all labour, equipment and material required to place and compact the backfill for RSS.

WARRANT: Always with these tender items.

APPENDIX FDR-B

Global Stability of RSS Walls


Slope Stability Analysis
For RSS Wall Geometry
1.990
Sand with Silt (Fill)
Compact to Very Dense
RSS Wall
Water
Clayey Silt (Stiff)
Sandy Silt (Very Dense)
Elevation
322
320
318
316
314
312
310
308
306
304
302
300
298
0
5
10
15
20
25
Distance

APPENDIX FDR-C

General Arrangement and Footing Layout of Existing Bridge

Contract # 67-101

General Arrangement
Drawing D-6044-1
March/67

Layout of Footings

Drawing D-6044-3

March/67

	PILE SCHEDULE			
	LOAD	NO.	LENGTH	TYPE.
N. ABUT.	50T	9	32'·0"	12¾"Φ Steel Tube.
	do.	23	33'·0"	do
	do.	64	34'·0"	do.
	do.	10.	35'·0"	do.
S. ABUT.	do.	9	32'·0"	do.
	do.	23	33'·0"	do.
	do.	66	34'·0"	do.
	do.	10.	35'·0"	do.
RET. WALLS.	do.	14	35'·0"	do.
	do.	28	36'·0"	do.

Appendix FDR-C

APPENDIX FDR-D

General Arrangement and RSS Walls for Proposed Bridge Extension

CONT. No.
WP No. 3110-09-00

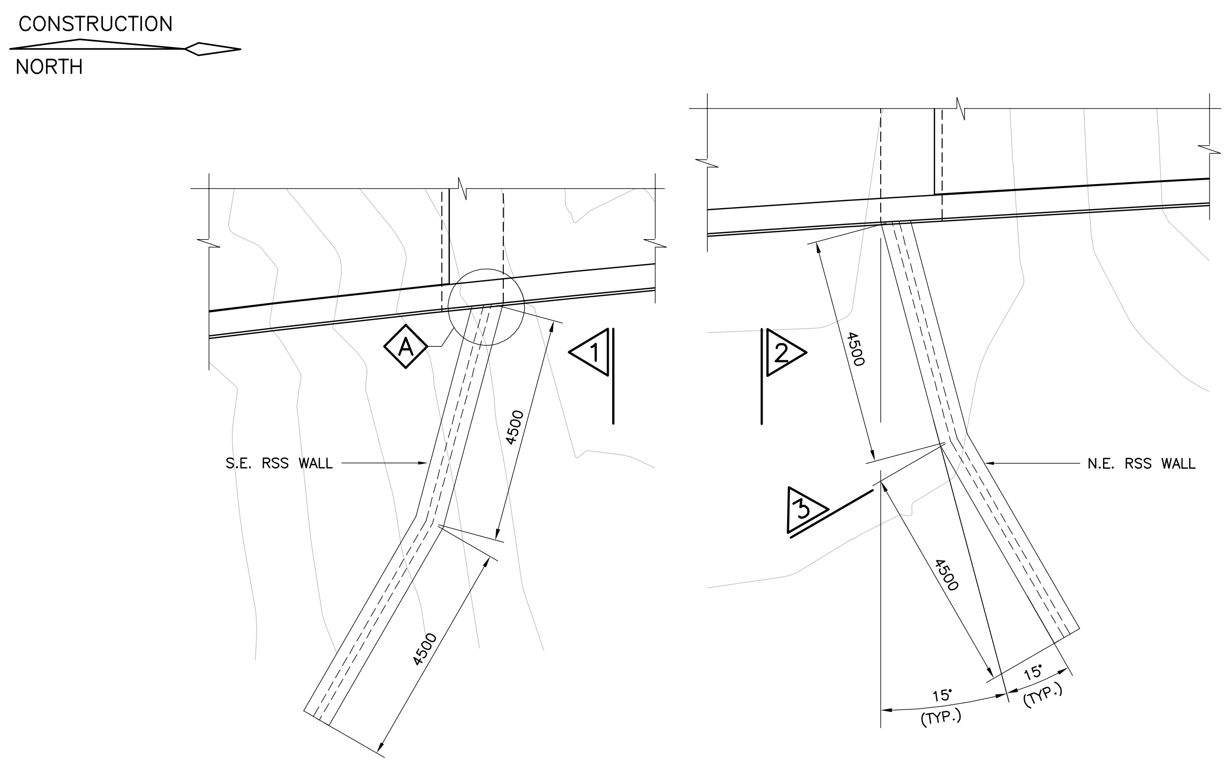
HIGHWAY 85
GUELPH ST. OVERPASS NBL
REHABILITATION

RSS WALLS

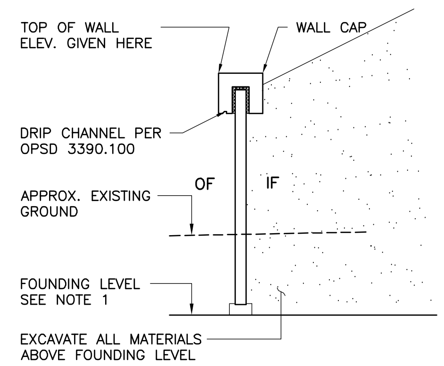
SHEET 126

MMM GROUP

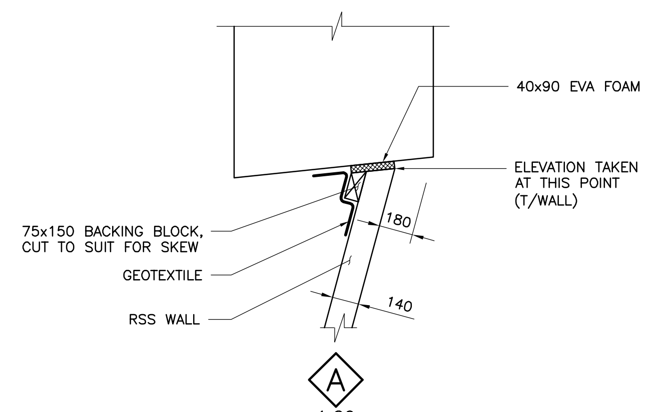
METRIC



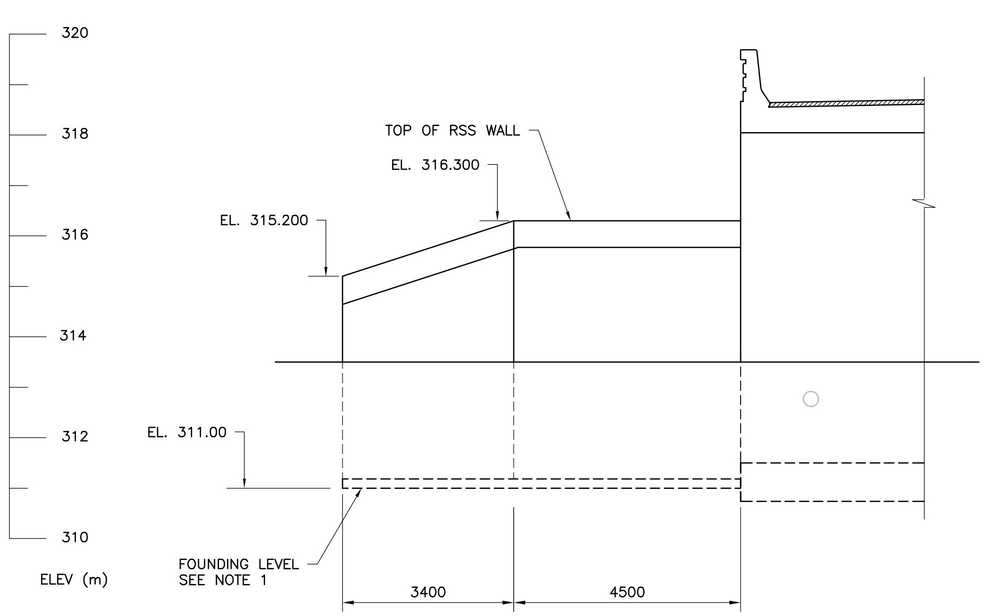
PLAN OF RSS WALLS
1:75



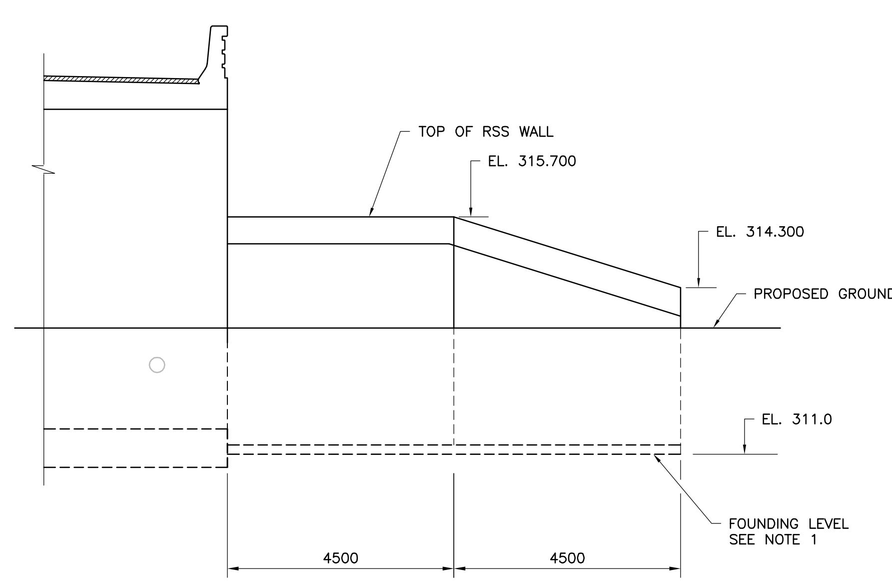
3
1:50



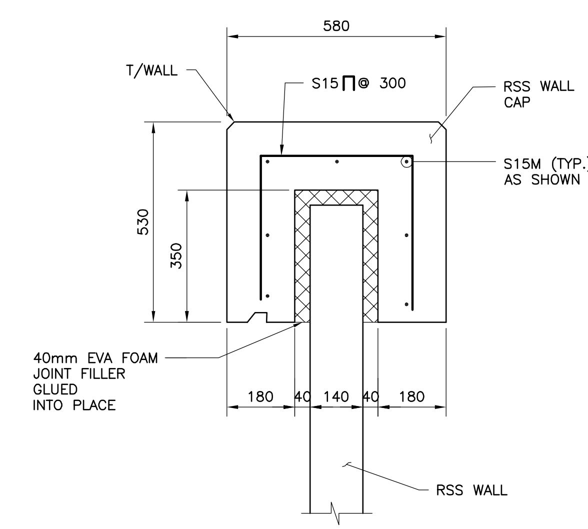
A
1:20



1 S.E RSS WALL ELEVATION
1:75



2 N.E RSS WALL ELEVATION
1:75



CAP DETAILS
1:10

NOTE:

1. BOTTOM OF RSS WALL MAY BE HIGHER THAN "FOUNDING LEVEL", PROVIDED THAT GRANULAR 'A' COMPACTED TO 100% SPMDD (WITH MOISTURE CEMENT WITHIN 2% OF OPTIMUM) IS PLACED FROM "FOUNDING LEVEL" TO UNDERSIDE OF RSS WALL.

RETAINED SOIL SYSTEM:

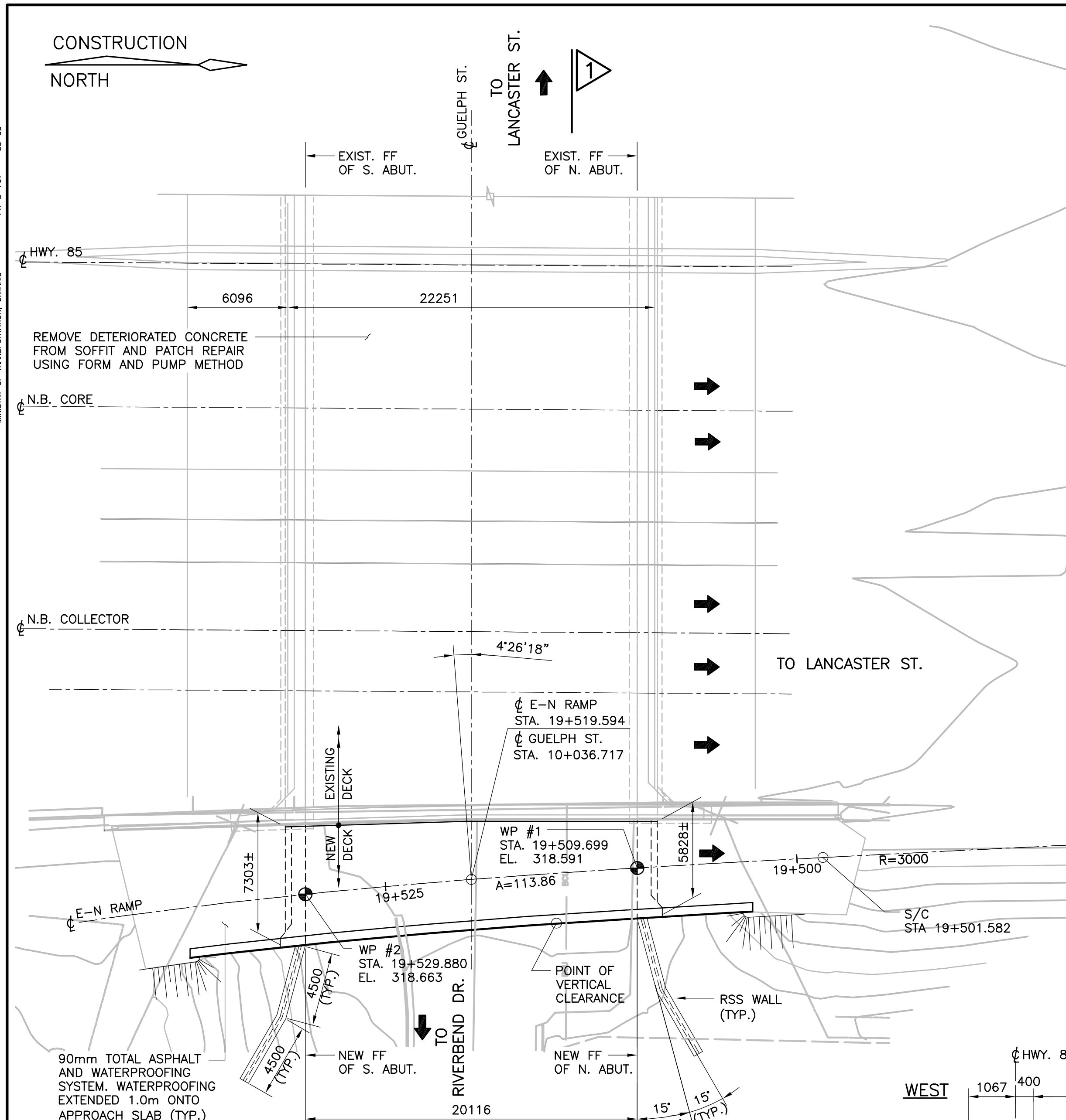
RSS WALL SHALL HAVE THE FOLLOWING ATTRIBUTES:

APPLICATION: FALSE ABUTMENT/RETAINING WALL
PERFORMANCE: HIGH
GEOMETRY: VERTICAL
APPEARANCE: HIGH

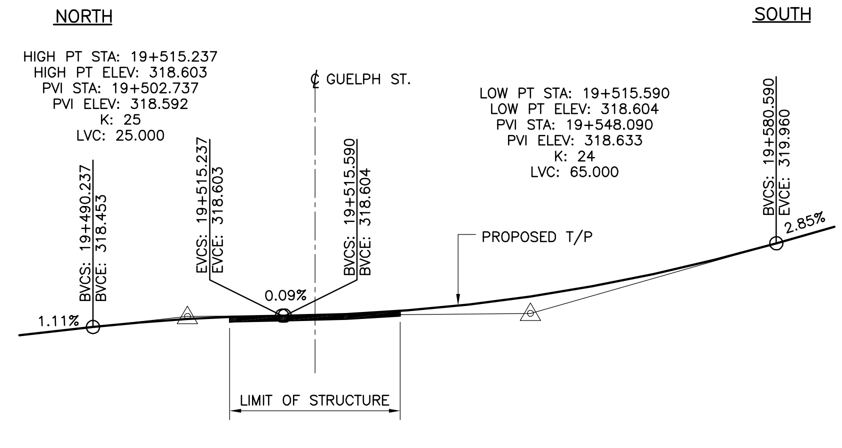
EFFECTS OF LIVE LOAD SURCHARGE SHALL BE INCLUDED IN DESIGN PER CHBDC.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

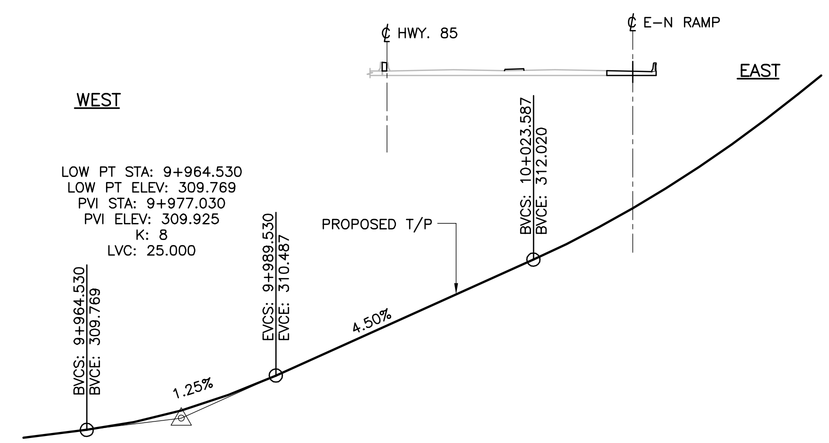
REVISIONS		DESCRIPTION	
DESIGN WK	CHK AY	CODE CHBDC 2010	LOAD CL-625-ONT	DATE NOV/14
DRAWN GL	CHK DF	SITE 33-238/1 STRUCT	SCHEME	DWG R3-10



PLAN
1:200



PROFILE OF E–N RAMP
N.T.S



PROFILE OF GUELPH ST.
N.T.S

LIST OF ABBREVIATIONS:

T/D – DENOTES TOP OF DECK
ABUT. – DENOTES ABUTMENT
TYP. – DENOTES TYPICAL
SBGR – DENOTES STEEL BEAM GUIDE RAIL

CONSTRUCTION NOTES:

- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
- SAWCUTS, WHERE INDICATED, SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS.
- CONCRETE SEALER SHALL BE FEDERAL STANDARD, 26493 FOR BARRIER WALL AND FASCIA AND 26373 FOR SUB–STRUCTURES.
- BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK SLAB IS IN PLACE AND HAS REACHED 25 MPa STRENGTH.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GRATER THAN 500mm.
- CONSTRUCT ABUTMENTS AND WINGWALLS TO THE BEARING SEAT ELEVATIONS. THE CONTRACTOR SHALL SUPPLY TEMPORARY LATERAL BRACING FOR THE ABUTMENTS. FORMWORK AND LATERAL RACING SHALL NOT BE REMOVED UNTIL THE CONCRETE IN DECK HAS REACHED 25 MPa STRENGTH.

LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATA
3. REMOALS
4. FOUNDATIONS LAYOUT AND REINFORCING
5. FRAME DETAILS I
6. FRAME DETAILS II
7. WINGWALLS
8. MISCELLANEOUS DETAILS
9. BARRIER WALL WITHOUT RAILING – PL3
10. RSS WALLS
11. PILE DRIVING CONTROL
12. STANDARD DETAIL
13. EMBEDDED WORK ELECTRICAL

DISTRICT CONT. No. WP No. 3110–09–00		
HIGHWAY 85 GUELPH ST. OVERPASS NBL REHABILITATION		SHEET
GENERAL ARRANGEMENT		117
MMM GROUP		METRIC

GENERAL NOTES:

CLASS OF CONCRETE:

UNLESS OTHERWISE NOTED 35 MPa

CLEAR COVER:

FOOTINGS	100 ± 25
DECK	
TOP	70 ± 20
BOTTOM	50 ± 10
REMAINDER	70 ± 20
(UNLESS OTHERWISE NOTED)	

REINFORCING STEEL:

- REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED
- STAINLESS STEEL SHALL BE TYPE 316 LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa. BAR MARKS WITH PREFIX 'S' DENOTES STAINLESS STEEL BARS.
- UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR BLACK REINFORCING STEEL BARS SHALL BE CLASS B.
- BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12–1 UNLESS INDICATED OTHERWISE.

RETAINED SOIL SYSTEM:

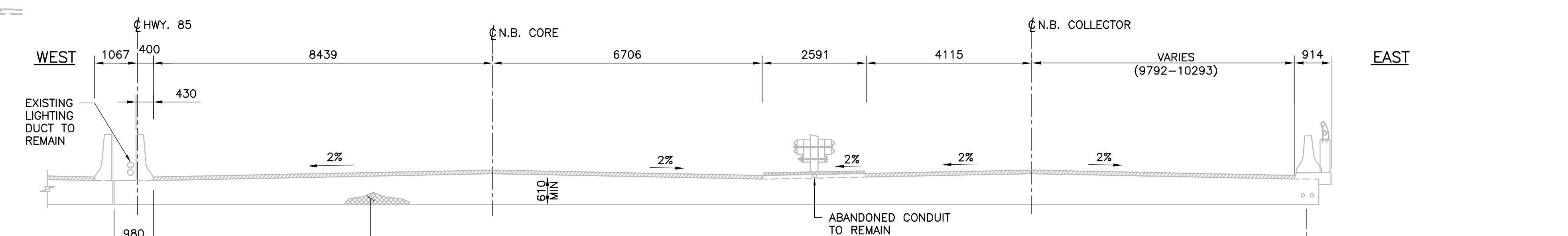
RETAINED SOIL SYSTEM WALLS SHALL HAVE THE FOLLOWING ATTRIBUTES:

APPLICATION:	FALSE ABUTMENTS/RETAINING WALLS
GEOMETRY:	VERTICAL
PERFORMANCE:	HIGH
APPEARANCE:	HIGH

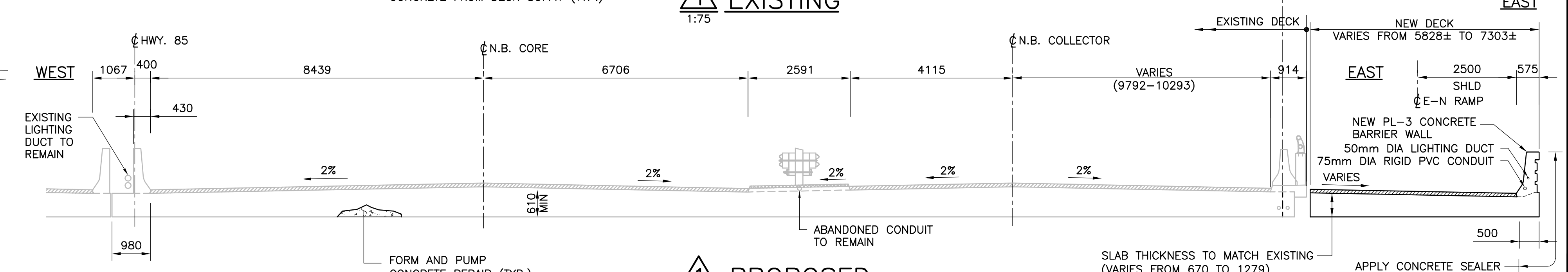
ALL PANELS SHALL HAVE ARCHITECTURAL FINISH TEXTURE.

APPLICABLE STANDARD DRAWINGS:

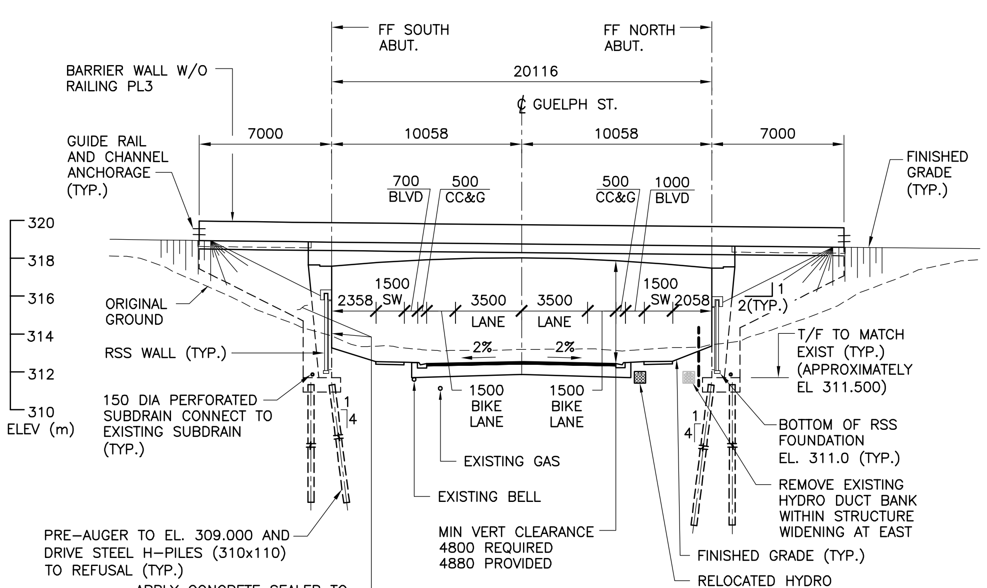
OPSD – 3101.150	WALLS, ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT
OPSD – 3370.100	BRIDGE DECK WATERPROOFING
OPSD – 3370.101	BRIDGE DECK WATERPROOFING DETAILS
OPSD – 3419.100	GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD – 3941.200	LOCATION OF SITE No. AND DATE FIGURES



EXISTING
1:75



PROPOSED
1:75



ELEVATION
1:200

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			DESCRIPTION	
DESIGN AY	CHK RSS	CODE CHBDC 2010	LOAD CL–625–ONT	DATE NOV/14
DRAWN RYR	CHK AY	SITE 33–238/1	STRUCT / SCHEME	DWG R3–1

MINISTRY OF TRANSPORTATION, ONTARIO PR–D–707 88–05

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DATE PLOTTED: 11/17/2014 9:15:05 AM BY: GREG PENNY