



**FOUNDATION INVESTIGATION REPORT AND
PRELIMINARY FOUNDATION DESIGN REPORT
for
DRAINAGE PIPE REMEDIATION
DON RIVER BRIDGE NORTH WEST EMBANKMENT
HIGHWAY 401
CITY OF TORONTO, ONTARIO
AGREEMENT NO. 2013-E-0039
TASK NO. 2013-E-0039-006**

PREPARED FOR MINISTRY OF TRANSPORTATION OF ONTARIO

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

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PML Ref.: 14TF033
Index No.: 010FIR and 011FDR
GEOCRES No.: 30M14-412
May 10, 2017



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for
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TABLE OF CONTENTS

1. INTRODUCTION	1
2. INVESTIGATION PROCEDURES	1
3. SUMMARIZED SUBSURFACE CONDITIONS.....	3
3.1 Fill	3
3.1.1 Upper Silty/Sandy Fill.....	4
3.1.2 Lower Clayey Silty Fill.....	4
3.2 Silt / Silty Sand / Silt and Sand	4
3.3 Groundwater	5
4. CLOSURE	6

Appendix A

- Figure 1 – Key Map
- Figure DP-PC-1 – Plasticity Chart
- Figures DP-GS-1 to DP-GS-4 – Grain Size Distribution Charts
- Explanation of Terms Used in Report
- Record of Borehole Sheets
- Site Photographs from Field Reconnaissance

FOUNDATION INVESTIGATION REPORT
For
Drainage Pipe Remediation
Don River Bridge North West Embankment, Hwy 401
City of Toronto, Ontario
Task No. 2013-E-0039-006

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed remediation of the drainage pipe that carries water from the Highway 401 westbound collector lane, down the approximately 33 m high north side embankment and underneath the Don Valley Golf Course into the Don River in the City of Toronto, Ontario. The study was carried out by Peto MacCallum Ltd. (PML) for the Ministry of Transportation of Ontario (MTO).

This is a copy of the draft report that was submitted on February 4, 2015. The submission of the final report was deferred pending receipt of comments from MTO. Subsequent to the submission of the draft report, comments were not provided as the drainage pipe failed and was repaired using recommendations provided by others. This report has been submitted in final form for completion of the contract obligations.

2. INVESTIGATION PROCEDURES

A site inspection was carried out on August 2014 and January 14, 2015. Photographs 1 to 5 (Appendix A) illustrate the site condition at the time of the investigations.

The field work for this study was carried out on November 19 to 26, 2014. The field investigation consisted of two boreholes, with one borehole drilled at Highway 401, westbound collector shoulder (Borehole No. 3) and one borehole drilled at the bottom of the slope along the drainage pipe alignment (Borehole No. 1) as shown on Figure 1 (Key Map). The contingency planned borehole No. 2 was not drilled in view of accessibility concern and the results of borehole No. 1 and No. 3.

The borehole locations were established in the field by PML at locations previously agreed with MTO. The borehole locations and elevations were surveyed by J.D Barnes Ltd. All elevations in this report are expressed in metres.



The boreholes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-55 and truck-mounted CME-75, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field supervisor.

Soil samples were recovered at 0.75, 1.5 or 3.0 m depth intervals using the standard penetration test method. 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. Additionally, a dynamic cone penetration test was advanced from the bottom of the sampling termination depth in borehole No. 1.

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 groundwater levels in the boreholes following drilling were also obtained.

A piezometer consisting of 25 mm diameter PVC pipe slotted over the bottom 3.0 m and surrounded in filter sand was installed in borehole No. 1 to monitor groundwater conditions. The annular space around the pipe above the filter sand was backfilled with auger cuttings and a bentonite seal placed as illustrated on the respective borehole log. The water level in the piezometer was measured on November 19, 2014 and January 29, 2015.

The boreholes and piezometer 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 (22)
- Grain size distribution analyses (9)
- Atterberg Limit Test (3)
- Relative Density (1)



The results of the laboratory grain size distribution analysis and Atterberg Limit Test are presented in Figures DP-GS-1 to DP-GS-4 and DP-PC-1, respectively. All of the test results are summarized on the Record of Borehole Sheets.

3. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration tests, dynamic cone penetration tests and groundwater observations. The results of laboratory particle size distributions, Atterberg Limit Testing and moisture content determinations are also shown on the Record of Borehole Sheets. Refer to Appendix A.

The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the stratigraphic boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at site revealed pavement fill, 0.9 to minimum 10.0 m thick silty/sandy fill and 13.4 m of clayey silt fill underlain by dense to very dense cohesionless deposit (sand and silt to sand with silt, silt and silty sand) that extended to the 7.6 and 39.8 m borehole sampling depth. Dynamic cone penetration test was advanced from the 7.6 m sampling termination depth in one borehole and revealed probable dense to very dense sand, which extended to the 9.2 m borehole termination depth. The piezometric water level was at 5.4 m (elevation 132.5) on November 19, 2014 and at 4.3 m (elevation 133.6) on January 29, 2015.

The strata encountered are summarised below.

3.1 Fill

Fill was encountered in the boreholes located at the top of the highway embankment and at the toe of the slope. It is composed of cohesionless soils (sandy silt, sand and silty sand), clayey silt, organic inclusions and cobbles.



At borehole No. 1 (at the toe of the embankment), 0.9 m thick fill layer was encountered extending from the surface at elevation 137.9 to elevation 137.0.

At borehole No. 3 (at the top of the embankment), pavement structure consisting of 100 mm thick asphalt over gravelly sand was present at surface (elevation 169.8). A 23.4 m thick fill was present under the pavement structure extended from elevation 169.2 to 145.8.

3.1.1 Upper Silty/Sandy Fill

The fill composed of 0.9 and minimum 10.0 m thick silty/sandy soils in boreholes No. 1 and 3 respectively. The deposit was loose to dense (SPT-'N' values of 8 to 41) and typically compact.

The results of grain size distribution analyses conducted on a sample of the fill are shown in figure DP-GS-1. The moisture content of the silty/sandy fill varied between 3 and 20%.

3.1.2 Lower Clayey Silty Fill

Below the silty/sandy fill, a minimum 13.4 m thick clayey silt fill was contacted in borehole No. 3. The clayey silt fill was hard in consistency (SPT-'N' values of 34 to 60).

The results of grain size distribution analyses and Atterberg Limit tests conducted on 4 samples of the deposit are shown in figures DP-GS-2 and DP-PC-1, respectively. The clayey silt had liquid limits of 25 to 29, plastic limits of 18 to 20 and a plasticity index of 5 to 9. The moisture content of the cohesive fill varied between 11 and 28%.

3.2 Silt / Silty Sand / Silt and Sand

A minimum 6.7 and 15.8 m thick cohesionless soil (silt, silty sand, silt and sand) was encountered below the fills in boreholes No. 1 and 3 respectively. Boreholes were terminated within this soil layer. The deposit extended to a depth of 7.6 m (from elevation 137.0 to 130.3) in borehole No. 1 and to a depth of 39.8 m (from elevation 145.8 to 130.0) in borehole No. 3.



The deposit was compact to very dense (SPT-'N' values of 19 to 50). Loose (SPT-'N' value of 6) silt and sand layer was encountered locally in borehole No. 1. A dynamic cone penetration data (from elevation 130.3 to 128.7) indicated dense to very dense probable sand in borehole No. 1.

The results of grain size distribution analyses conducted on 4 samples of the deposit are shown in Figures DP-GS-3 and DP-GS-4. The moisture content of the deposit varied between 6 and 24%.

3.3 Groundwater

In the process of augering, water was detected at 3.8 and 24.4 m (elevation 134.1 and 145.4) in boreholes No. 1 and 3, respectively. No water was observed in boreholes No. 1 and 3 upon completion of drilling. The water level measured in the piezometer installed in borehole No. 1 was at a depth of 5.4 m (elevation 132.5) on November 19, 2014 and at a depth of 4.3 m (elevation 133.6) on January 29, 2015.

The groundwater was encountered below the toe of the embankment. It is subject to fluctuations and precipitation patterns and will be controlled by the water level of the Don River.



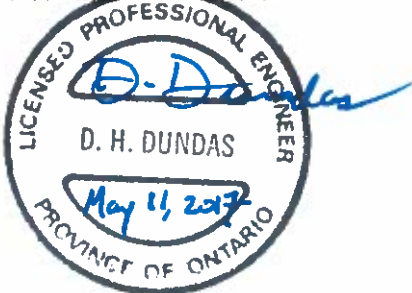
4. CLOSURE

Mr. S. Aziz carried out the field investigation for this study under the supervision of Mr. K. Daly, BEng, EIT., and Mr. C. M. P. Nascimento, P. Eng., Project Manager. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This Foundation Investigation Report was prepared by Ms. M. Kamranzadeh, MSc, EIT, and reviewed by Mr. David Dundas, P.Eng., Senior Engineer. Mr. C. M. P. Nascimento, P. Eng., MTO Designated Principal Contact conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



David Dundas, P.Eng.
Senior Engineer, Geotechnical Services



Carlos M.P. Nascimento, P.Eng.
Project Manager and
MTO Designated Principal Contact

MK/DD/CN:mk-mi-jk-az



APPENDIX A

Figure 1 – Key Map

Figure DP-PC-1 – Plasticity Chart

Figures DP-GS-1 to DP-GS-4 – Grain Size Distribution Charts

Explanation of Terms Used in Report

Record of Borehole Sheets

Site Photographs from Field Reconnaissance



GEOCREs No.: 30M14-412

MINISTRY OF TRANSPORTATION

**DRAINAGE PIPE REMEDIATION
HIGHWAY 401 - DON RIVER BRIDGE NW EMBANKMENT
TORONTO, ONTARIO**

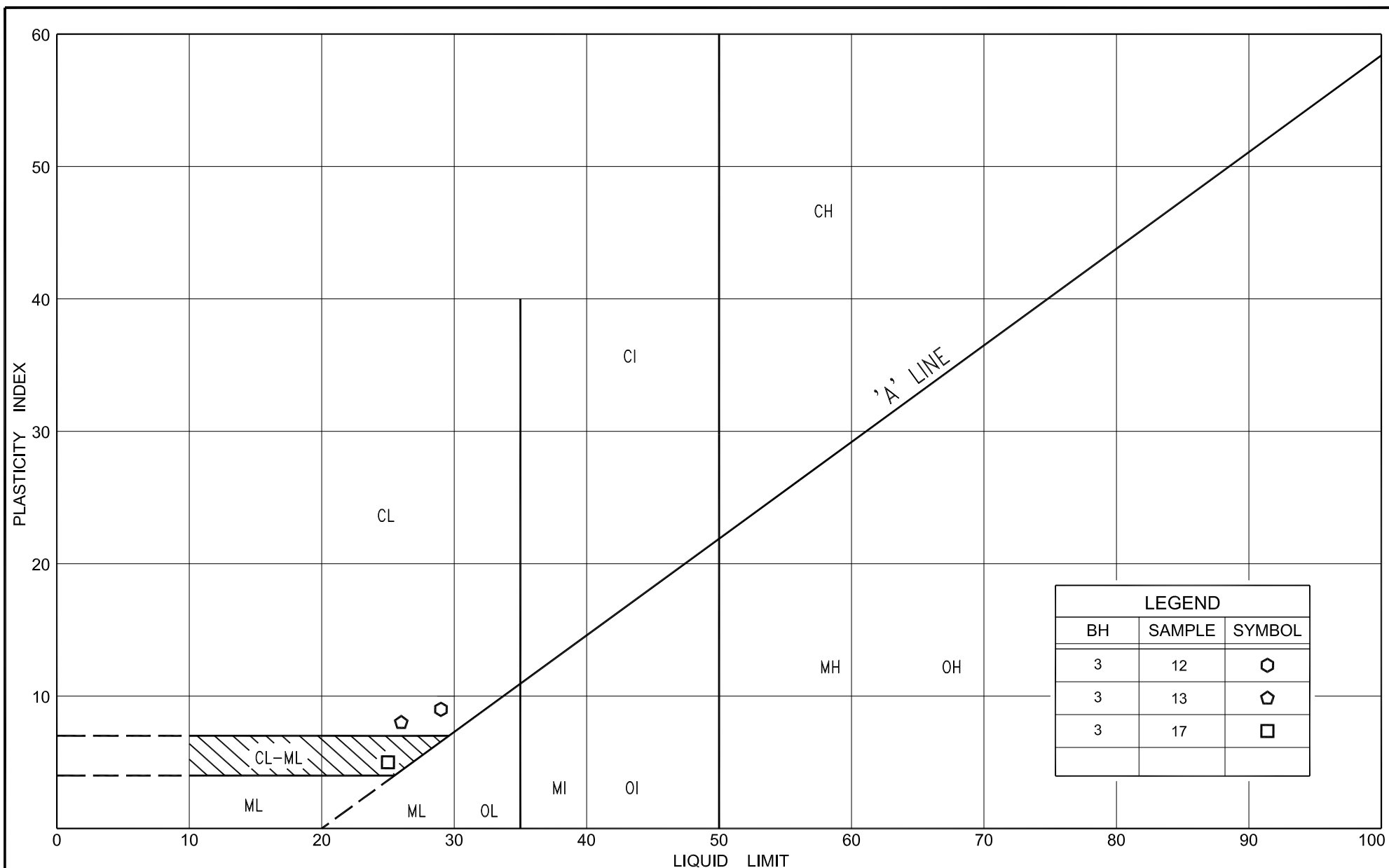
KEY MAP



Peto MacCallum Ltd.
CONSULTING ENGINEERS

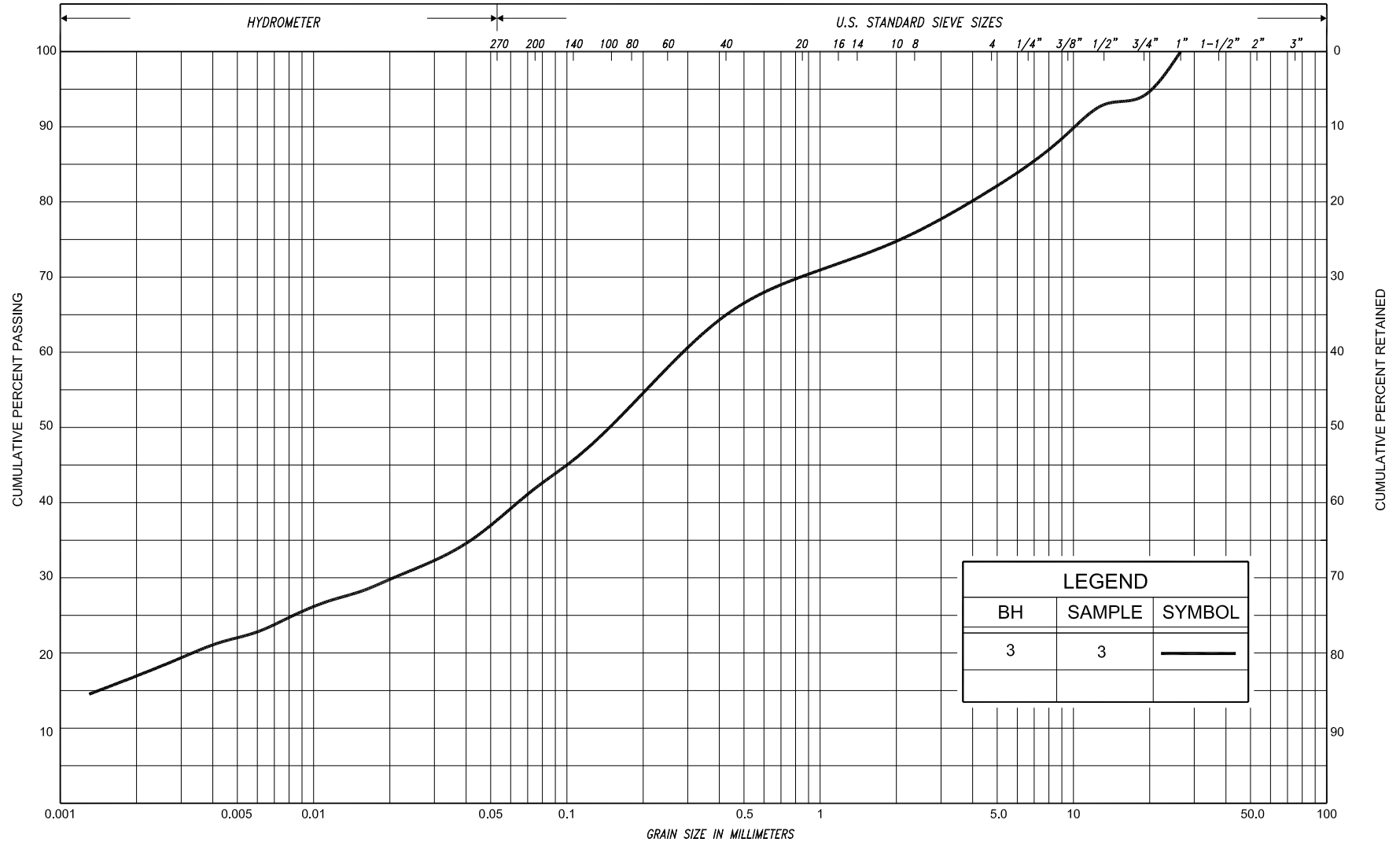
DRAWN:	N.A.	DATE	SCALE	JOB NO.	FIGURE NO.
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APPROVED:	C.N.				

REFERENCE:
THIS FIGURE WAS PREPARED FROM THE GOOGLE MAP - MAPDATA @ 2015 GOOGLE.



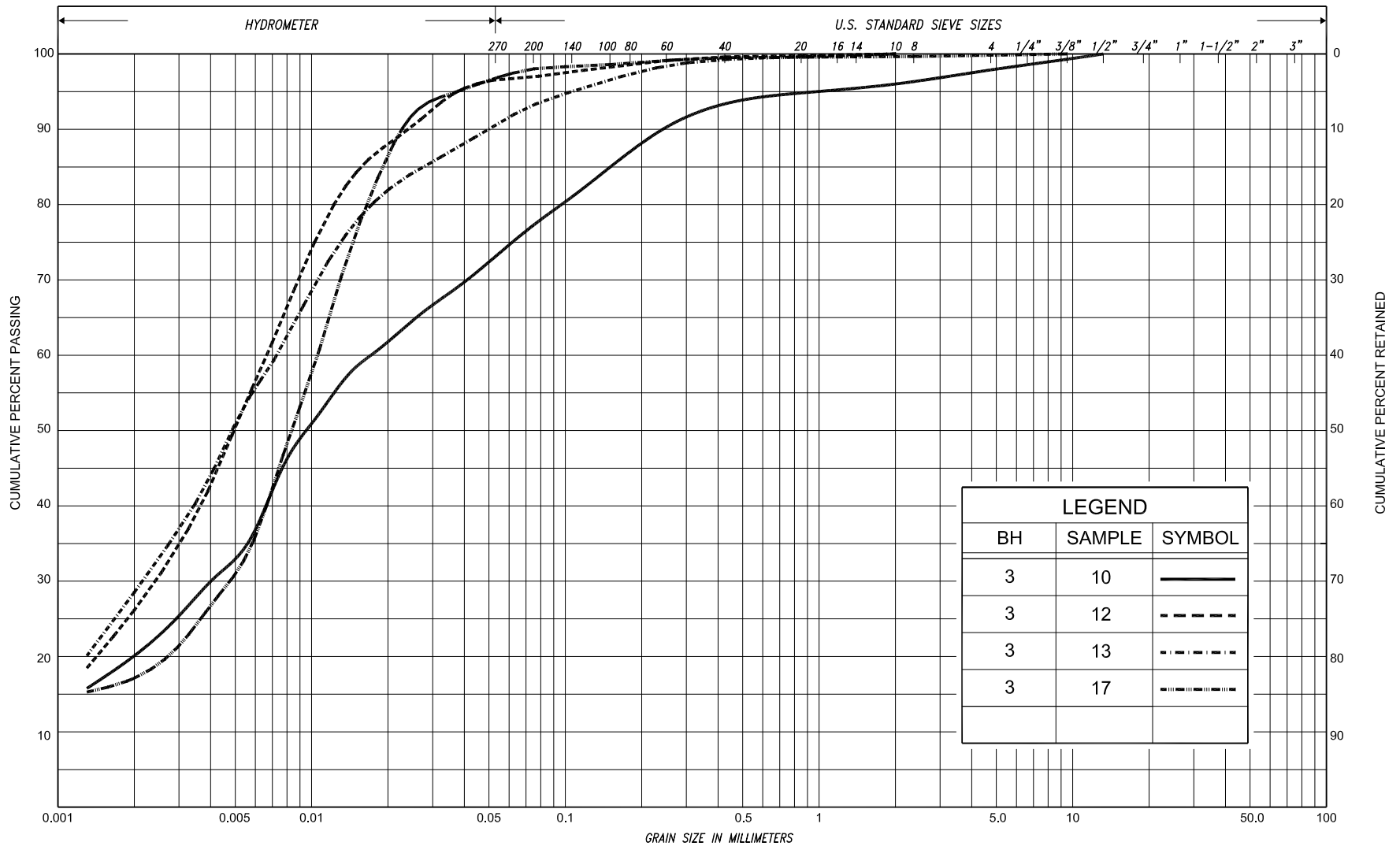
PLASTICITY CHART
 CLAYEY SILT, trace to with sand, trace gravel
 (FILL)

FIG No. DP-PC-1
 HWY: 401
 TASK No. 2013-E-0039-006



LEGEND		
BH	SAMPLE	SYMBOL
3	3	—

SILT & CLAY										FINE			MEDIUM			COARSE		GRAVEL				COBBLES	UNIFIED	
										SAND														
CLAY	FINE			MEDIUM			COARSE			FINE			MEDIUM			COARSE			GRAVEL				COBBLES	
	SILT															GRAVEL				COBBLES				
CLAY				SILT				V. FINE		FINE		MED.		COARSE		GRAVEL						U.S. BUREAU		
										SAND														



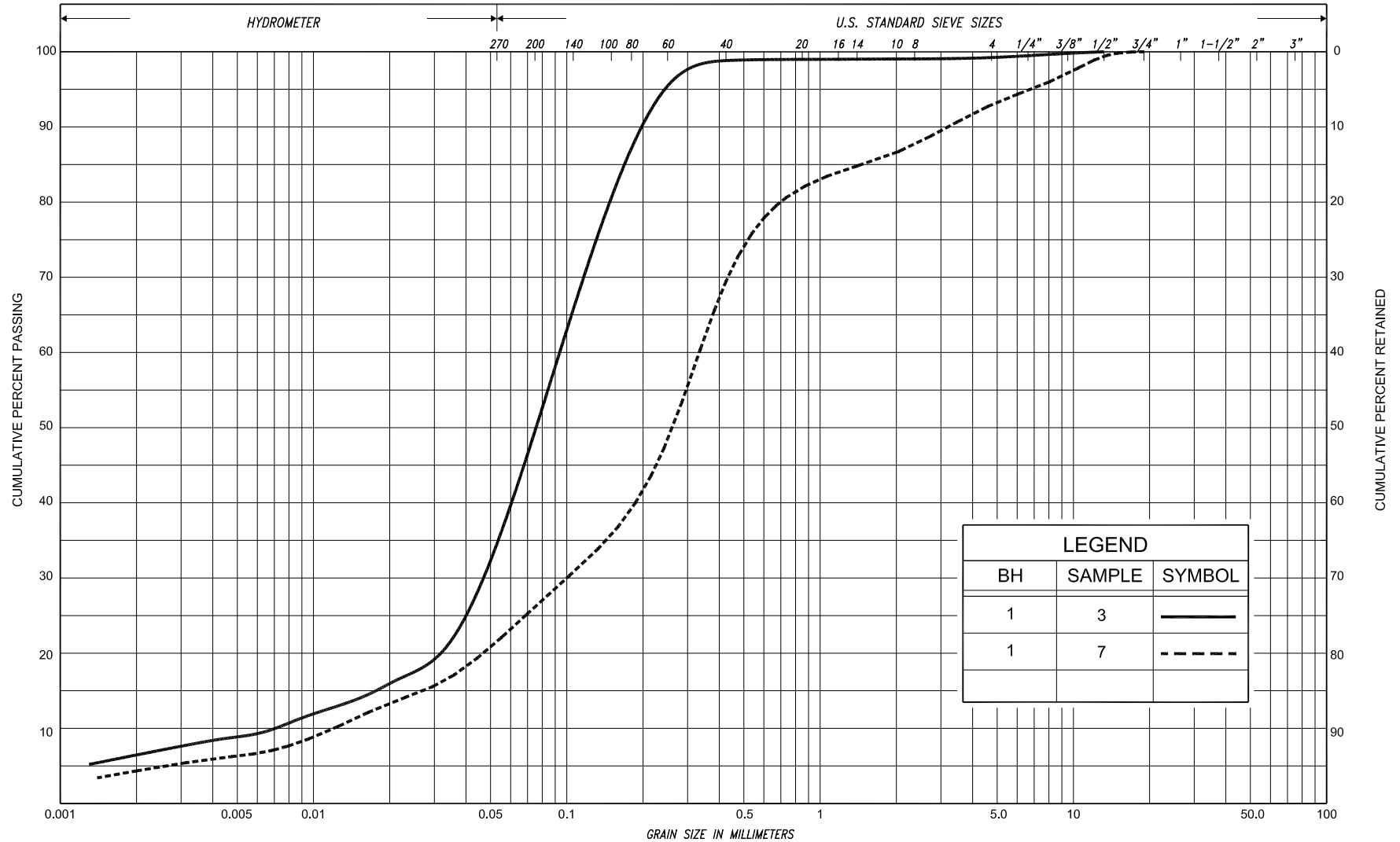
SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
				SAND											
CLAY	FINE		MEDIUM	COARSE	FINE		MEDIUM	COARSE		GRAVEL			COBBLES	M.I.T.	
				SILT											
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL						U.S. BUREAU	
				SAND											



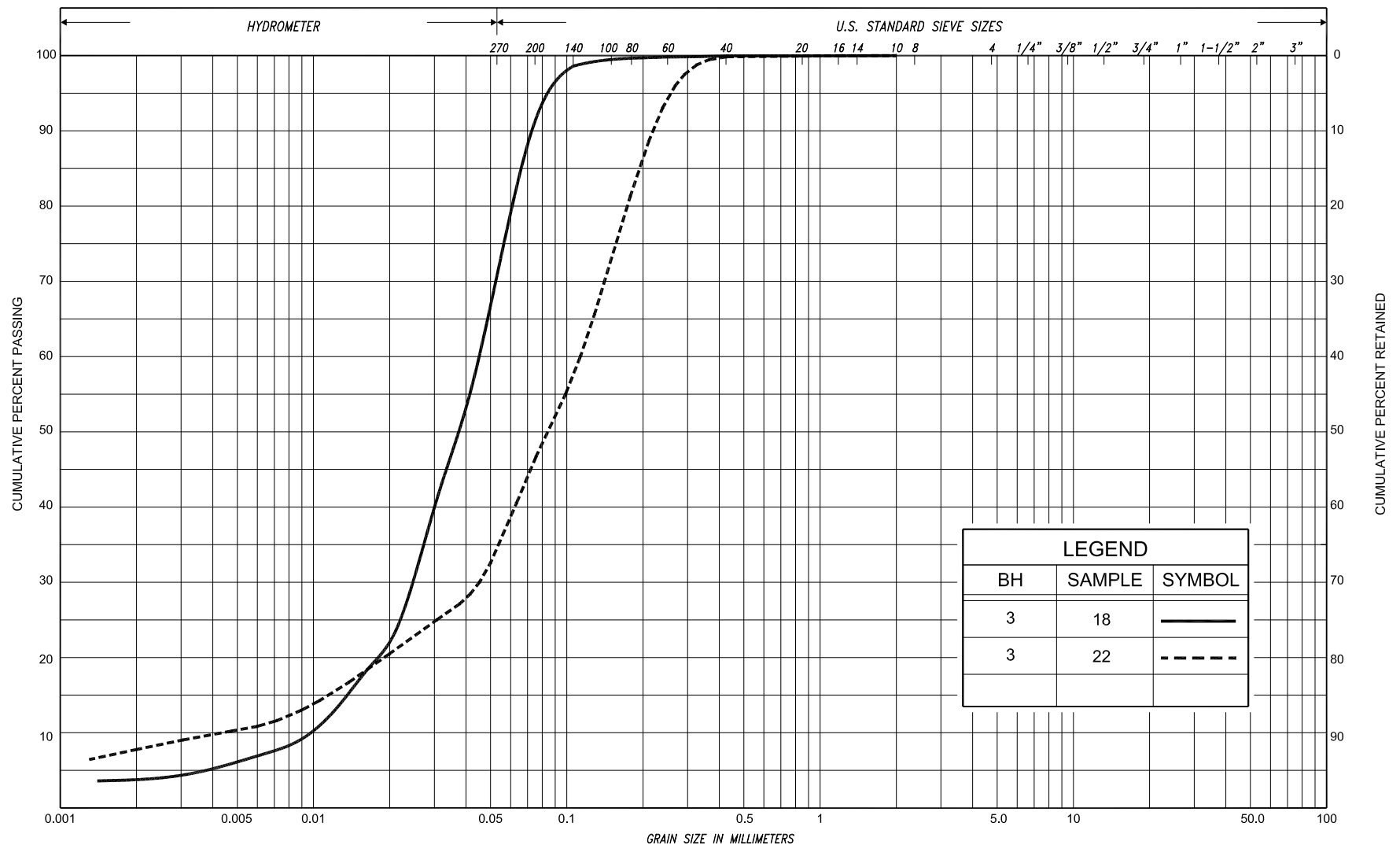
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace to with sand, trace gravel
(FILL)

FIG No. DP-GS-2
 HWY: 401
 TASK No. 2013-E-0039-006



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL				COBBLES	UNIFIED		
					SAND													
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL				COBBLES	M.I.T.
	SILT																	
CLAY			SILT			V. FINE		FINE		MED.		COARSE		GRAVEL				U.S. BUREAU
						SAND												



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT																
CLAY		SILT			V. FINE		FINE		MED.		COARSE		GRAVEL			U.S. BUREAU	
					SAND												



GRAIN SIZE DISTRIBUTION

SILTY SAND TO SILT, trace sand, trace clay

FIG No. DP-GS-4

HWY: 401

TASK No. 2013-E-0039-006

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/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
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_i	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	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL				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			

RECORD OF BOREHOLE No. 1

1 of 1

METRIC

TASK No. 2013-E-0039-006 **LOCATION** Coords: 4 845 374 N; 627 563 E **ORIGINATED BY** S.A.

DIST Central **HWY** 401 **BOREHOLE TYPE** C.F.H.S.A. and Dynamic Cone Penetration Test **COMPILED BY** M.K.

DATUM Geodetic **DATE** November 19, 2014 **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa										WATER CONTENT (%)		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
137.9	Ground Surface						20	40	60	80	100								
0.0	Sandy silt rootlets, organics topsoil inclusions		1	SS	9														
137.0	Loose Dark Moist brown/brown (FILL)		2	SS	6							○							
0.9	Silt and sand trace clay, trace gravel																		
	Loose to Brown Moist compact /grey		3	SS	19							○				1 49 43 7			
135.7																			
2.2	Sand, with silt trace clay, trace gravel		4	SS	34							○							
	Dense Mottled Moist brown/grey		5	SS	43							○							
	with _____, _____, _____ gravel, cobbles		6	SS	50/13cm														
	Very dense Wet																		
	trace _____ gravel																		
	Dense to compact		7	SS	39							○				7 67 22 4			
			8	SS	29								○						
130.3	End of borehole																		
7.6	Switched to Dynamic cone penetration test																		
	Probable sand																		
	Dense to very dense																		
128.7	End of dynamic cone penetration test																		
9.2	Refusal on probable boulder																		

RECORD OF BOREHOLE No. 3

1 of 3

METRIC

TASK No. 2013-E-0039-006

LOCATION

Coords: 4 845 306.9 N; 627 628.1 E

ORIGINATED BY S.A.

DIST Central

HWY 401

BOREHOLE TYPEC.F.H.S.A. and Mud Rotary

COMPILED BY M.K.

DATUM Geodetic

DATE _____

November 24 to 26, 2014

CHECKED BY C.N.

[illegible]

RECORD OF BOREHOLE No. 3

2 of 3

METRIC

TASK No. 2013-E-0039-006

LOCATION

Coords: 4 845 306.9 N; 627 628.1 E

ORIGINATED BY S.A.

DIST Central HWY 401

BOREHOLE TYPEC.F.H.S.A. and Mud Rotary

COMPILED BY M.K.

DATUM Geodetic

DATE November 24 to 26, 2014

CHECKED BY C.N.

[illegible]

RECORD OF BOREHOLE No. 3

3 of 3

METRIC

TASK No. <u>2013-E-0039-006</u>	LOCATION <u>Coords: 4 845 306.9 N; 627 628.1 E</u>	ORIGINATED BY <u>S.A.</u>
DIST <u>Central</u> HWY <u>401</u>	BOREHOLE TYPE <u>C.F.H.S.A. and Mud Rotary</u>	COMPILED BY <u>M.K.</u>
DATUM <u>Geodetic</u>	DATE <u>November 24 to 26, 2014</u>	CHECKED BY <u>C.N.</u>

[illegible]



Photograph P1: Looking northwest from top of the slope. Light vegetation is visible along the embankment (August 8, 2014).



Photograph P2: Looking at the pipe outlet. The CSP pipe and HDPE liner are visible. The HDPE liner sheared off and slipped down the slope to the Don River (August 8, 2014).



Photograph P3: Looking north at the location of the pipe outlet. The HDPE liner slipped down the slope to the Don River (August 8, 2014).



Photograph P4: Looking down from top of the slope. No significant surficial erosion is visible along the slope (January 14, 2015).



Photograph P5: Looking at the pipe outlet. The HDPE liner is cut off and temporary braced with blocks and bolts at the outlet (January 14, 2015).



**PRELIMINARY FOUNDATION DESIGN REPORT
for
DRAINAGE PIPE REMEDIATION
DON RIVER BRIDGE NORTH WEST EMBANKMENT
HIGHWAY 401
CITY OF TORONTO, ONTARIO
AGREEMENT NO. 2013-E-0039
TASK NO. 2013-E-0039-006**

PREPARED FOR MINISTRY OF TRANSPORTATION OF ONTARIO

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

1 cc: MTO Geotechnical Section
1 cc: MTO Foundations (Danny Tari, Darren Berwick)
+ 1 digital copy (PDF)
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Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page TOC 1 of 1



TABLE OF CONTENTS

5. DISCUSSION	7
6. BACKGROUND	7
7. FOUNDATION RECOMMENDATIONS AND DISCUSSION.....	8
7.1 General	8
7.1.1 Armoured Open Channel	11
7.1.2 Rehabilitation of Existing Pipe	11
7.1.3 Installation of New Pipe	11
7.1.4 Drop Structures With Manholes.....	11
7.1.5 Embankment Stability	12
8. FUTURE STUDIES.....	13
9. CLOSURE	14

Appendix B

- Slope Stability Analysis Result
- Drawing 1 – Conceptual Illustration for Alternative Drop Structure Geometry

PRELIMINARY FOUNDATION DESIGN REPORT

For
Drainage Pipe Remediation
Don River Bridge NW Embankment, Hwy 401
City of Toronto, Ontario
Task No. 2013-E-0039-006

5. DISCUSSION

This Preliminary Foundation Design Report provides preliminary foundation engineering recommendations to permit the design team to select a preferred option for future detail design of remedial measures or replacement of the existing drainage pipe that carries water from Highway 401 Westbound Lane Collector drainage system to the base of the 33 m high embankment. The site is located at northwest quadrant of the Highway 401 embankment of the Don River Bridge, just west of Yonge Street in the City of Toronto. This Preliminary Foundation Design Report was prepared for the Ministry of Transportation (MTO).

This Preliminary Foundation Design Report is based on the foundation investigation carried out in November 2014 and the site inspection carried out on January 14, 2015.

This is a amended copy of the draft report that was submitted on February 4, 2015. The submission of the final report was deferred pending receipt of comments from MTO. Subsequent to the submission of the draft report, comments were not provided as the drainage pipe failed and was repaired using recommendations provided by others. This report has been submitted in final form for completion of the contract obligations.

6. BACKGROUND

The drainage pipe carries water from the Highway 401 westbound collector lane, down an embankment, underneath the Don Valley Golf Course into the Don River. A key plan based on Google map is shown in Figure 1. The overall geometry of the existing embankment at the location of the drainage pipe is approximately 33 m high with an estimated inclination of 1.75 Horizontal to 1 Vertical (1.75H:1V). The geometry of the slope varies along the pipe alignment, with a steeper portion in the upper 1/3 of the slope and an approximately 10 m wide bench semi

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 8



horizontal berm at the base of the steepened portion. The embankment was constructed in its current configuration in 1964 in order to add the westbound collector lanes. Trees along the slope directly above the drainage pipe have been removed and it is covered with the light vegetation. The embankment is stable and no significant surficial erosion has been notified during the site inspection carried out on January 2015.

The initial drainage pipe was a 1200 mm diameter corrugated steel pipe. In 2012, a pipe failure occurred near the top of the pipe that caused erosion and gully in the order of 1.2 m deep that extended from the pipe break to the bottom of the slope. As an interim remedial measure, a 900 mm diameter HDPE pipe was slip-lined inside the 1200 mm diameter CSP with the annular space partially grouted near the break point with unshrinkable fill. In 2014, the HDPE pipe sheared off at the previous pipe break location and slipped down the slope some 25 m. The HDPE liner was cut off and removed at the outlet of the CSP pipe and temporarily braced with blocks and bolts at the outlet.

7. FOUNDATION RECOMMENDATIONS AND DISCUSSION

7.1 General

The construction operations such as excavation, shoring, groundwater control, backfill and bedding should be considered in the selection of the replacement/remedial option.

Based on discussions with the MTO, the following alternatives are proposed for consideration for remediation of the drainage pipe:

1. Armoured open channel
2. Rehabilitation of existing pipe
3. Installation of new pipe
4. Drop structures with manholes

The following table compares the advantages, disadvantages, risks / consequences and relative costs of each alternative from the foundation perspective:

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 9



ALTERNATIVE DRAINAGE PIPE REMEDATION METHOD	ADVANTAGES	DISADVANTAGES	RISKS/ CONSEQUENCES	RELATIVE COSTS
1. Armoured open channel	<ul style="list-style-type: none">- Less complex design and short construction time	<ul style="list-style-type: none">- Less reliable and permanent drainage function- Temporary drainage provisions may be required during the construction	<ul style="list-style-type: none">- Seepage into slope with associated reduction in Safety Factor for surficial slope stability- Destabilization of slope through erosion- Destabilization of the highway embankment if erosion occurs- Risk of runoff damage to the golf course during or after the construction	<ul style="list-style-type: none">- Least cost due to less complex construction and shorter construction time- Cost of temporary drainage installation, if needed, has to be considered
2. Rehabilitation of existing pipe <ul style="list-style-type: none">- for either temporary or permanent use including installation of thrust blocks or pipe anchors to improve connection integrity of pipe and liner	<ul style="list-style-type: none">- More reliable performance than open channel- Relatively short time required for construction- Temporary drainage probably not a concern	<ul style="list-style-type: none">- A repair strategy including installation of a liner has already failed once.	<ul style="list-style-type: none">- General deterioration of an old pipe- Breaking at joints of pipe and in connections between liner and pipe	<ul style="list-style-type: none">- Relatively low cost- Cost of camera pipe inspection has to be considered
3. Installation of new pipe <ul style="list-style-type: none">- including installation of thrust blocks or pipe anchors to improve connection integrity of pipe	<ul style="list-style-type: none">- More reliable performance and longer service life than open channel or repaired pipe- No need for pipe liner	<ul style="list-style-type: none">- Requires installation along new alignment to permit temporary drainage through existing pipe or installation of temporary drainage channel while new pipe is installed along existing alignment	<ul style="list-style-type: none">- Slope destabilization during construction	<ul style="list-style-type: none">- Moderately high cost

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 10



ALTERNATIVE DRAINAGE PIPE REMEDATION METHOD	ADVANTAGES	DISADVANTAGES	RISKS/ CONSEQUENCES	RELATIVE COSTS
4. Drop structures with manholes	<ul style="list-style-type: none">- Superior longevity due to use of concrete pipes- Permits engineered energy dissipation	<ul style="list-style-type: none">- Requires installation along new alignment to permit temporary drainage through existing pipe or installation of temporary drainage channel while new pipe is installed along existing alignment- Requires superior settlement performance to prevent distress to concrete structures- More time required for construction- Needs complex design to determine optimum drop structure layout and hydrology and structural engineering design input- Requires extensive excavations or shoring for installation with excavation geometry dependent on the selected geometry of the drop structures	<ul style="list-style-type: none">- Destabilization of slope during construction- Maintenance required to periodically check manholes for debris accumulation	<ul style="list-style-type: none">- Much more costly than other methods

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 11



7.1.1 Armoured Open Channel

In general, armoured open channel requires less complex design compared to other methods. This will result in a shorter construction time and the least cost. However, there is a risk that installation of an open channel could cause the destabilization of the highway embankment and slope as well as damaging the golf course during or after the construction. Moreover, temporary drainage may be required during the construction.

7.1.2 Rehabilitation of Existing Pipe

Rehabilitation of the existing pipe is another alternative that could be considered for the drainage pipe remediation. In this method, thrust blocks or pipe anchors should be installed to improve the connection integrity of the pipe and liner. Video inspection of the pipe would be required to assess the extent of the damages on the pipe.

7.1.3 Installation of New Pipe

New pipe installation has more reliable performance and longer service life than open channel and existing pipe rehabilitation. The new pipe could be installed along a new alignment to permit temporary drainage through the existing pipe or alternatively could be installed along the existing alignment. If installed along the existing alignment, the installation of a temporary drainage channel along the slope would be required.

7.1.4 Drop Structures With Manholes

Drop structure permits engineered energy dissipation and has superior longevity due to use of concrete pipes. It requires hydrology and structural engineering design input and complex design to determine optimum drop structure layout. It is also require superior settlement performance to prevent distress to concrete structures.

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 12



Different slope configuration could be used for the drop structure. They can be either designed vertically or sloped at 1H:1V. The sloped drops could minimize the volume of excavation. Refer to Appendix B, Drawing 1 for the corresponding configurations, which provide an illustration of the differences in volume of excavation depending on the selected slope of the drop structures. The drop structure is much more costly than other method.

7.1.5 Embankment Stability

The stability of the embankment was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd.

A 24.0 m thick fill was contacted in borehole 3 (located at the top of the embankment). The fill was composed of loose to dense (SPT-'N' values of 8 to 41) silty/sandy soils and hard (SPT-'N' values of 34 to 60) clayey silt. Below the fill, a minimum 15.8 m thick very dense silty/sandy soils (SPT-'N' value of 50) was encountered.

The soil parameters used in the analyses were based on the results of both field and laboratory testing and adjusted by applying engineering judgement in case of layered soil deposits. A summary of the engineering parameters and their values assumed in the calculations is as follows:

SOIL TYPE	UNIT WEIGHT (kN/m ³)	SHORT TERM	
		SHEAR STRENGTH (kPa)	INTERNAL FRICTION (degree)
Silty/Sandy Fill (Loose to Dense, Typically Compact)	19.5	0	32
Clayey Silt Fill (Hard)	18	60	0
Silty/Sandy Soils (Very Dense)	21	0	35

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 13



The slope has performed for over 40 years and there are no deep seated stability issues. The result of the short term slope stability analyses is enclosed in Appendix B and considered embankment geometry with side slope of 1.75H:1V without excavation of cohesive soils. The result of the stability analyses is 1.1 which indicated that there are surficial stability concerns along the slope. This marginal surficial slope stability is due to the height and steepness of the slope and would be exacerbated by high velocity runoff.

8. FUTURE STUDIES

The recommendations in this report are for preliminary foundation engineering purposes only and are based on interpretations of an existing site condition and the foundation investigation. Detail information such as existing drainage plan of the highway and embankment will be required during the Detail Design phase of the project.

Aspects that will require further investigation are as follows:

- Camera inspection of the existing pipe: this information is needed to evaluate and select the potential remedial measures.
- Design water volume and thrust block pressures
- Hydrology and structural design inputs such as determination of the drop structure geometry and energy dissipation requirements (if drop structure has been considered as a preferred option).
- Energy dissipation requirements (if drop structure has been considered as a preferred option).

It will be necessary to review the size and condition of the existing drainage pipe including its liner, the size of feeder pipes and the volume of water to be drained and the integrity of details and integrity of pipe connections. This information was not available at the time of preparation of this preliminary foundation recommendations memo, but will be required for preparation of the detail design foundation design report for the selected option.

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017, Page 14



9. CLOSURE

This Foundation Design Report was prepared by Ms. M. Kamranzadeh, MSc, EIT, and reviewed by D. Dundas, P.Eng., Senior Engineer. Mr. C. M. P. Nascimento, P. Eng., Project Manager and MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



David Dundas, P.Eng.
Senior Engineer, Geotechnical Services



Carlos M.P. Nascimento, P.Eng
Project Manager and
MTO Designated Principal Contact

MK/DD/CN:mk-mi-jk-az

Preliminary Foundation Design Report

Drainage Pipe Remediation, Don River Bridge NW Embankment, Hwy 401, City of Toronto
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR
PML Ref.: 14TF033, May 10, 2017



APPENDIX B

- Slope Stability Analysis Result
- Conceptual Illustration for Alternative Drop Structure Geometry

Preliminary Foundation Design Report

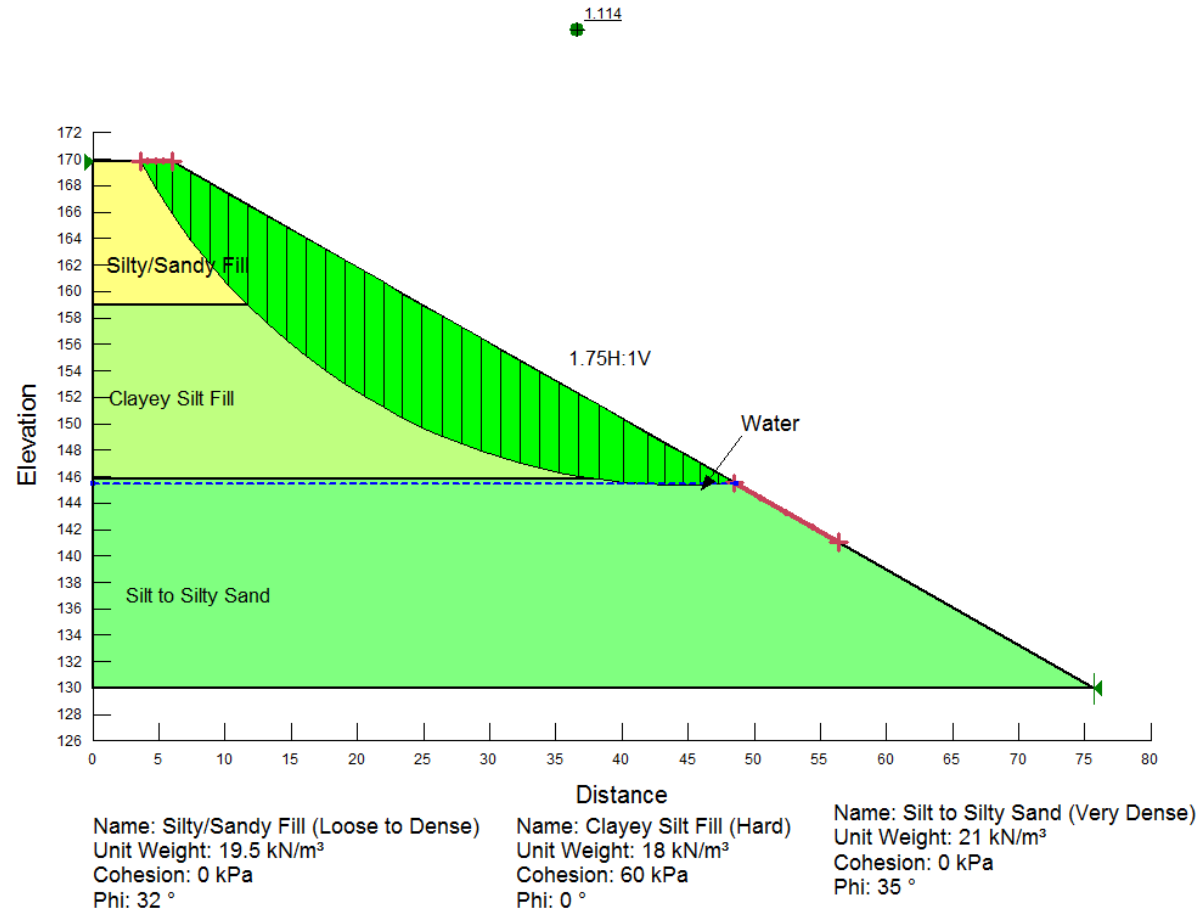
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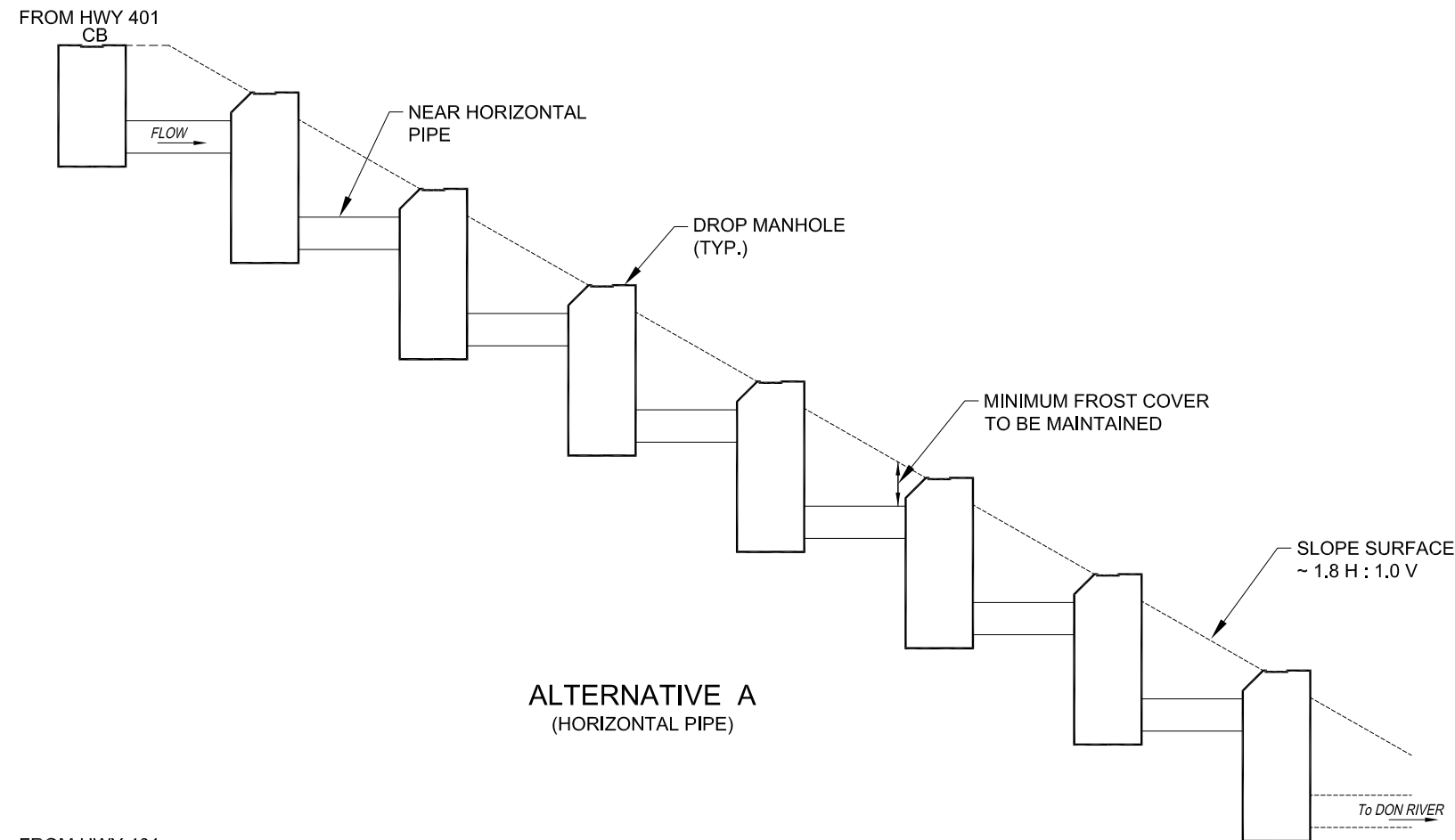
Agreement No. 2013-E-0039, Task No. 2013-E-0039-006, Index No.: 011FDR

PML Ref.: 14TF033, May 10, 2017

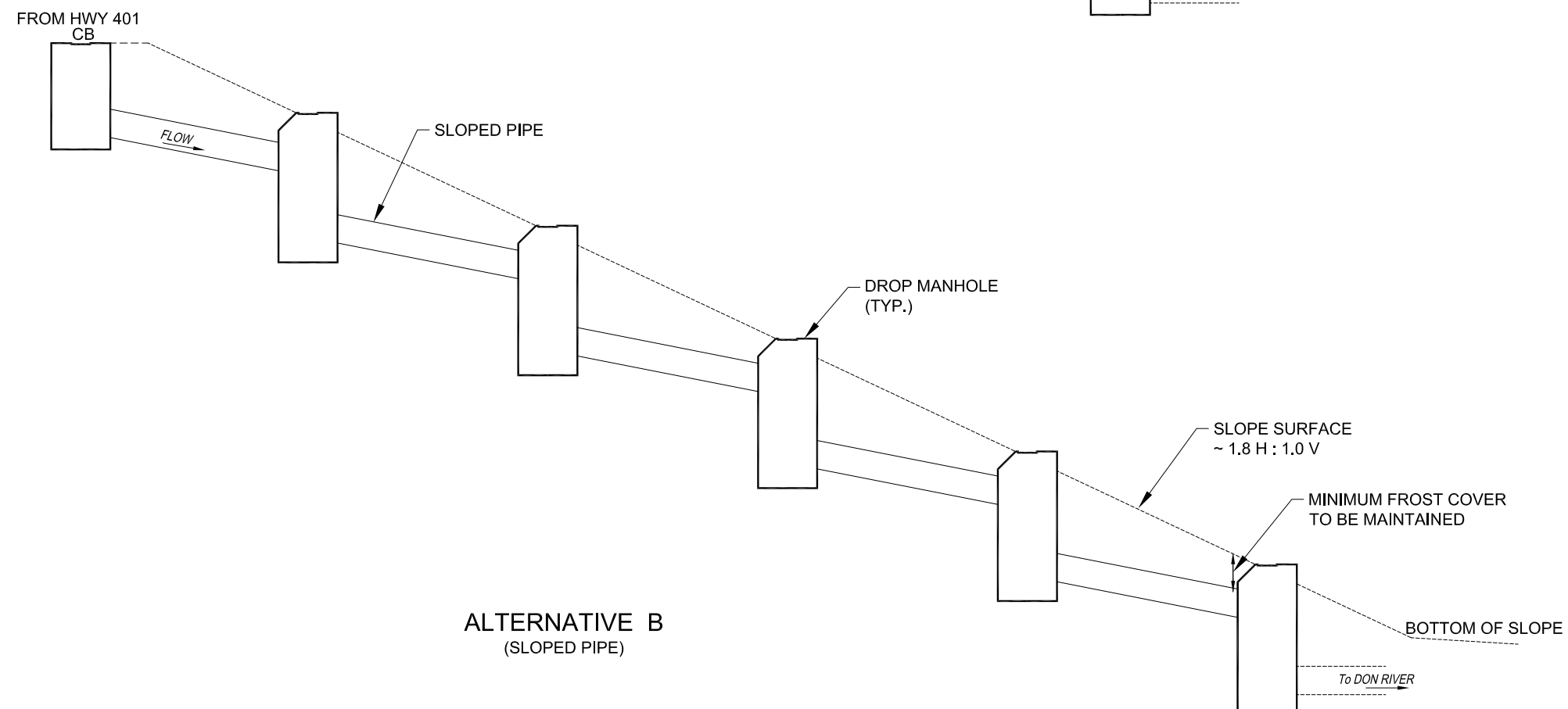


Embankment Stability Analysis





ALTERNATIVE A
(HORIZONTAL PIPE)



ALTERNATIVE B
(SLOPED PIPE)

NOTE:

THESE ILLUSTRATIONS ARE FOR CONCEPTUAL DEPECTION PURPOSES ONLY.
THE DESIGN MUST BE CARRIED OUT BY THE SPECIALIST ENGINEERS.

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MINISTRY OF TRANSPORTATION

**DRAINAGE PIPE REMEDIATION
HIGHWAY 401 - DON RIVER BRIDGE NW EMBANKMENT
TORONTO, ONTARIO**

**CONCEPTUAL ILLUSTRATION FOR
ALTERNATIVE DROP STRUCTURE GEOMETRY**



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