



**PRELIMINARY (DESIGN-BUILD READY)
FOUNDATION INVESTIGATION AND DESIGN REPORT
for
NORTH CANAL BRIDGE NORTHBOUND REPLACEMENT
HIGHWAY 400, SITE No. 30-334/1
GWP 2005-11-00
TOWNSHIP OF WEST GWILLIMBURY, SIMCOE COUNTY, ONTARIO**

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PML Ref.: 14TF025-CN
Index No.: 021FIR and 022FDR
GEOCRES No.: 31D-604
June 8, 2015



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**PRELIMINARY (DESIGN-BUILD READY)
FOUNDATION INVESTIGATION REPORT**

for

North Canal Bridge Northbound Replacement

Highway 400, Site No. 30-334/1

GWP 2005-11-00

Township of West Gwillimbury, Simcoe County, Ontario

1. INTRODUCTION

This report summarises the results of a preliminary foundation investigation carried out for the replacement of the existing northbound bridge over the North Canal and Canal Road located on Highway 400 in the Township of West Gwillimbury, County of Simcoe, Ontario. The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

Information from "Foundation Investigation Report for Proposed Extensions to the Overpass Structures at the Crossing of Highway 400 and the North Canal Road and Drainage Canal, Township of West Gwillimbury, County of Simcoe" prepared by the Department of Highways Ontario and dated December 1970 (GEOCREs No. 31D-30) has been used in preparation of this report, with copies of the relevant borehole logs appended.

This report provides subsurface information pertaining to the proposed structure and approaches within about 20 m of the abutments.

All elevations in this report are expressed in meters.

2. SITE DESCRIPTION AND GEOLOGY

Highway 400 passes over the North Canal and Canal Road at approximate Station 10+561, Highway 400 chainage (ref. Drawing P-1 'Canal Road Bridge Replacement General Arrangement' prepared by MMM Group Limited in May 2013). The existing bridge constructed in 1948 and widened outwardly in 1971 is a six span structure having a total length of 69.5 m. The original bridge is supported on driven timber piles and steel H-piles were used for the widened portion. Its middle pier is situated within the canal channel.



The existing road grade on Highway 400 at the bridge location rises from elevation 226.7 at the south abutment to elevation 228.5 at the north abutment. The existing approach embankments are about 7 m high and have side slopes inclined at about 2.5H:1V. The water level in the canal was reported to be at elevation 218.8 on October 5, 2011. This water level is kept relatively constant for crop irrigation purposes.

The structure to be replaced carries three lanes of Highway 400 northbound traffic over the North Canal some 3 km north of the interchange with Highway 9 in the Township of West Gwillimbury, County of Simcoe. At the location of the bridge, Highway 400 runs approximately in the south-north direction.

At the existing bridge location, the North Canal flowing in the easterly direction is about 20 m wide and lined with steel sheet piling on both sides. It is understood that the existing sheet piles and tie-backs may have been weakened due to corrosion. The grade of Canal Road running under the bridge between its south abutment and the canal is at approximate elevation 223.5. There is a retaining wall at the toe of the south abutment foreslope immediately south of Canal Road.

The project site is in the Holland River valley that extends southwesterly from Cook Bay at the south end of Lake Simcoe. The land surface adjacent to the canal is flat to gently undulating.

The site is located on the southern lobe of the Simcoe Lowlands physiographic region. The topography is irregular in detail. Extensive deposits of clay and silt laid down on the floor of former Lake Algonquin are underlain by glacial till. These deposits are locally covered by sand of deltaic origin.

Bedrock predominantly comprising limestone is of the Trenton-Black Formation, Ordovician Period. The bedrock in the vicinity of the site is at depths exceeding 35 m.



3. INVESTIGATION PROCEDURES

The field work for this study was carried out during the period of October 29 to 31, 2014 and included borehole 102 advanced to a depth of 33.2 m at the north abutment of the replacement bridge. Borehole 101 was also drilled at the proposed north abutment of the southbound bridge location to a depth of 33.8 m. Boreholes 1, 3, 5 and 7 drilled along the alignment of the northbound structure during the previous investigation in October and November 1970 were put down to depths of 21.5 to 39.1 m. The borehole locations are shown on Drawing CN-1, attached.

The location of the current borehole was established in the field by Peto MacCallum Ltd. The ground surface elevations at the current borehole locations were provided by Callon Dietz.

The boreholes for the current investigation were advanced using continuous flight hollow stem augers, powered by a truck-mounted CME-95 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. A mud rotary technique was used to extend borehole 102 to 31.1 m depth, followed by a dynamic cone penetration test that was advanced to refusal at 33.2 m due to the presence of gravel and cobbles.

Representative soil samples were recovered at frequent depth intervals using a conventional split spoon sampler during drilling as well as Shelby tubes. Standard penetration tests (SPT) were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. The shear strength / consistency of the cohesive soils noted on the Record of Borehole sheet and in the subsequent sections of the report is primarily based on in situ vane shear testing (using the MTO 'N' vane according to the procedure described in the Northeastern Region Pavement Design Practices and Guidelines dated May 1997). Less consideration was given to SPT-'N' values since the shear strength indicated by this technique is less reliable in soft clayey soils.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of soil, the sampler and drill rods as the samples were retrieved. Because the boreholes were advanced using mud drilling techniques, direct observation of the water level after completion of drilling was not made. Artesian conditions were not observed during the current investigation. Upon



completion of drilling, the boreholes were backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. Atterberg limits testing (4) and grain size distribution analyses (5) were performed on selected soil samples from borehole 102. A consolidation test and one unconfined compressive strength test were conducted on a relatively undisturbed Shelby tube cohesive soil sample. The laboratory test results are presented in Figures CN-PC-1, CN-PC-2, CN-GS-1 to CN-GS-3, C-1 and on the corresponding Record of Borehole log.

4. SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard and dynamic cone penetration test data, in situ vane shear strength values as well as groundwater observations. The results of laboratory Atterberg limits testing, grain size distribution analyses, one unconfined compressive strength test and natural moisture content determinations are also shown on the Record of Borehole sheets.

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

The subsurface stratigraphy revealed in the boreholes drilled at the site is generally consistent with that in the previous boreholes and included embankment fill over silty/sandy soils overlying clayey silt interlayered with sand / silty sand and underlain by a glacial till. Cobbles were encountered in one borehole. The groundwater was at elevation 218.5 to 222.6, with artesian conditions reported in one borehole. The water level in the North Canal was at approximate elevation 218.8 at the time of the investigation in 2014.



The strata encountered in borehole 102, supplemented by previous boreholes 1, 3, 5 and 7 are summarised below.

4.1 Embankment Fill

Asphalt and granular material from the existing highway pavement was present surficially in borehole 102 put down at the north abutment. Covered with 225 mm of asphalt and composed of sand and gravel, the granular fill was dense, about 4% in moisture content and extended to a depth of 0.4 m (elevation 228.5).

Directly beneath the granular fill at 0.4 m depth (elevation 228.5) in borehole 102 was sand fill becoming silty sand fill. This unit was 1.3 m in thickness and compact in relative density, with a moisture content of 9 to 11%. The sand fill / silty sand fill was penetrated at a depth of 1.7 m (elevation 227.2).

Clayey silt fill was present surficially in boreholes 1, 3, 5, 7 and below the sand fill / silty sand fill at 1.7 m depth (elevation 227.2) in borehole 102. Having a thickness of 1.6 to 8.1 m, the clayey silt fill was soft to hard, typically firm to stiff in consistency and extended to the base of the embankment fill at depths of 1.6 to 9.8 m (elevation 219.1 to 219.9).

The results of Atterberg limits testing and grain size distribution analysis performed on a cohesive sample of the fill in borehole 102 are presented in respective Figures CN-PC-1 and CN-GS-1. The liquid and plastic limits of the clayey silt fill were 34 and 19 respectively, thus giving the plasticity index of 15. The moisture content of the cohesive soils varied between 12 and 27%.

4.2 Topsoil

Topsoil was buried under the embankment fill at 9.8 m depth (elevation 219.1) in borehole 102. The silty topsoil was 200 mm thick and penetrated at a depth of 10.0 m (elevation 218.9).



Topsoil and peat should be expected under the footprints of new approach embankment widenings. Copies of the relevant borehole logs from Stantec's pavement investigation are attached for reference.

4.3 Silty/Sandy Soils

Overlain by the fill at depths of 1.6 to 6.2 m (elevation 219.1 to 219.9) in boreholes 1, 3, 5, 7 or by the topsoil at 10.0 m depth (elevation 218.9) in borehole 102 was cohesionless sand / silty sand. Locally containing organics, this unit was loose to dense (SPT-'N' values of 6 to 34) and had a moisture content of 18 to 22%. The unit was 0.6 to 2.5 m in thickness and penetrated at depths of 4.0 to 10.6 m (elevation 217.0 to 218.3).

4.4 Clayey Silt

Underlying the silty/sandy soils at depths of 4.0 to 10.6 m (elevation 217.0 to 218.3) in all the boreholes was clayey silt. Interlayered with sand / silty sand, this deposit was 15.6 to 30.2 m thick where penetrated and soft to hard, typically firm to stiff in consistency. The results of in situ vane testing carried out in the clayey silt in borehole 102 yielded undisturbed shear strength values of 64 and 100 kPa (soil sensitivity of 3). An unconfined compression test on a Shelby tube sample of the deposit gave a shear strength value of 27 kPa (strain at failure of 13%). The results of the consolidation test indicated an initial void ratio of 0.79, unit weight of 19.5 kN/m³, preconsolidation pressure of 165 kPa, compression index of 0.20 and recompression index of 0.02. In boreholes 3, 5, 7 and 102, the deposit was penetrated at depths of 26.2 to 34.2 m (elevation 187.4 to 202.7). Borehole 1 was terminated in the clayey silt at a depth of 21.5 m (elevation 204.2).

The results of Atterberg limits testing and grain size distribution analyses conducted on 3 cohesive samples from borehole 102 are presented in respective Figures CN-PC-2 and CN-GS-2. The liquid and plastic limits of the clayey silt ranged from 24 to 32 and from 17 to 21 respectively, with the plasticity index of 6 to 15. The moisture content of the clayey silt varied between 16 and 30%.



4.5 Sand / Silty Sand

A layer of cohesionless sand / silty sand was identified within the clayey silt at depths of 10.8 to 17.4 m (elevation 210.0 to 211.8) in all the boreholes. This layer was compact to very dense (SPT-'N' values of 21 to 76) and 10% in moisture content. The sand / silty sand had a thickness of 2.1 to 5.6 m and was penetrated at depths of 13.9 to 19.5 m (elevation 206.2 to 209.4).

4.6 Glacial Till

Glacial till was revealed below the clayey silt at depths of 26.2 to 34.2 m (elevation 187.4 to 202.7) in boreholes 3, 5, 7 and 102. This stratum was compact to very dense / hard (SPT-'N' values of 27 to 256) and had a moisture content of 6 to 12%. With a thickness of at least 1.7 to 7.0 m, the glacial till was not penetrated upon termination of the boreholes at depths of 33.2 to 39.1 m (elevation 182.5 to 195.7). It is noteworthy that the stratum contained cobbles in borehole 102.

The results of grain size distribution analysis performed on the sand till in borehole 102 are presented in Figure CN-GS-3.

4.7 Groundwater

In the process of augering on October 29, 2014, water was detected at 10.3 m depth (elevation 218.6) in borehole 102. Upon completion of drilling, groundwater was measured in boreholes 1, 3 and 7 to be at depths of 3.0 to 3.1 m (elevation 218.5 to 222.6). It is noted that artesian water was encountered at a depth of 28.0 m (elevation 195.2) in borehole 5 during drilling, with a head of 0.2 m above the ground surface (elevation 223.4). No artesian water conditions were detected in borehole 102.

The water level in the North Canal was at approximate elevation 218.8. Although the groundwater levels at the site are somewhat subject to seasonal fluctuations and precipitation patterns, the water level in the canal is controlled to be relatively constant for crop irrigation purposes.



5. CLOSURE

The field work was carried out under the supervision of Mr. F. Portela, Senior Technician, and direction of Mr. Kyle Daly, EIT, and Mr. C.M.P. Nascimento, P.Eng., Project Manager. The equipment was supplied by Atcost Drilling Inc.

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

Yours very truly,

Peto MacCallum Ltd.



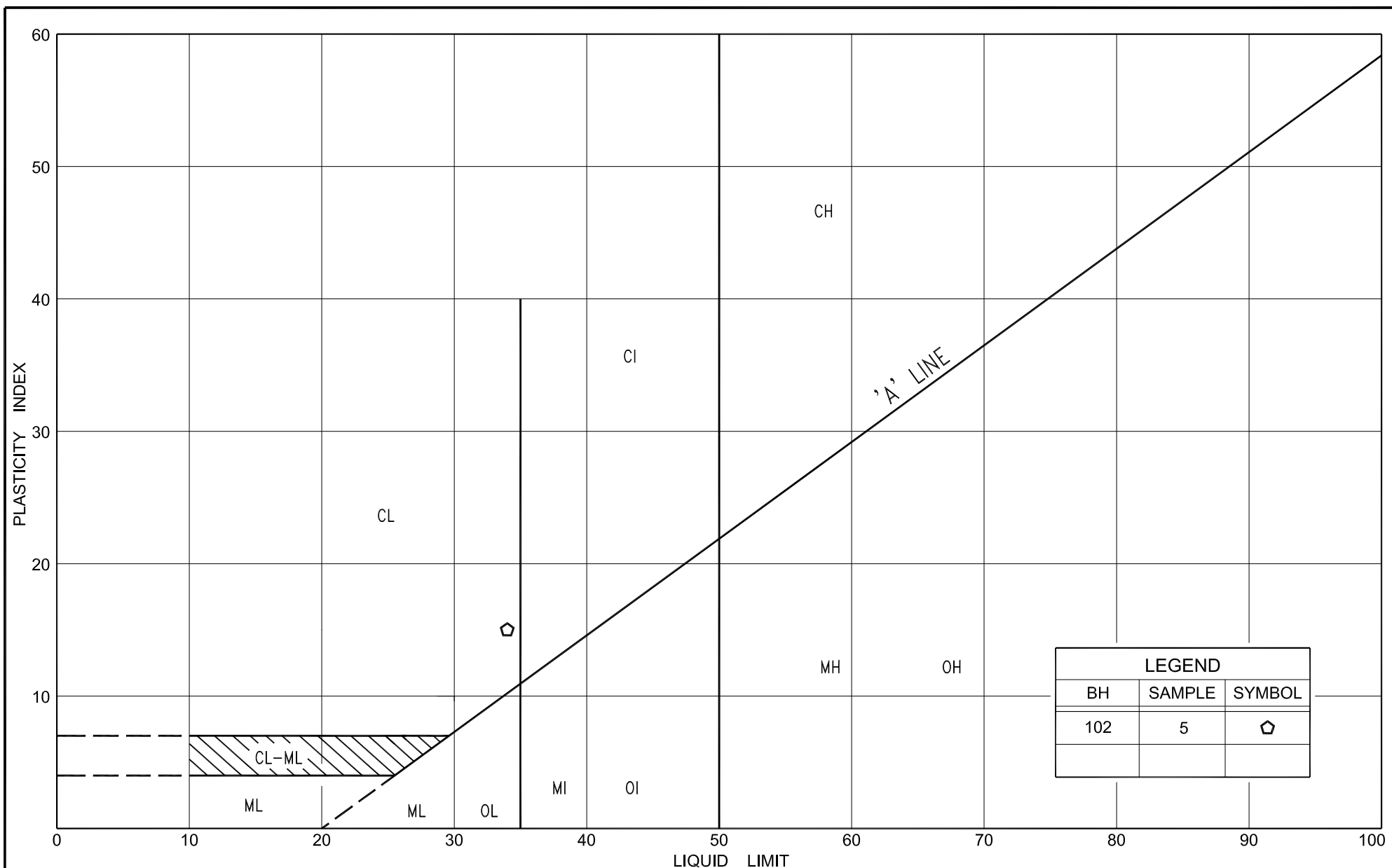
Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer



Brian R. Gray, MEng, P.Eng.
Principal Consultant

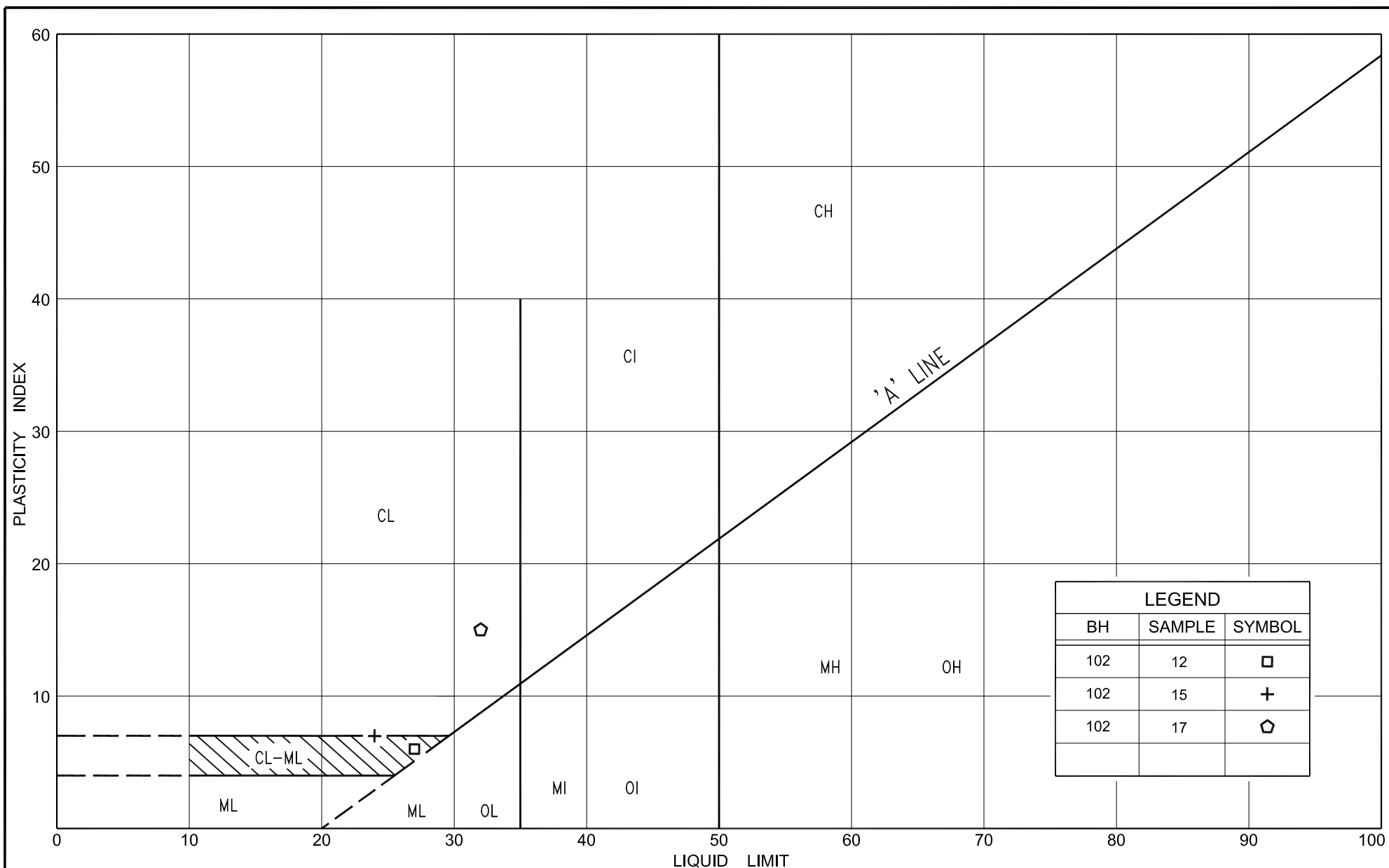


Carlos M.P. Nascimento, P.Eng.
MTO Designated Principal Contact



PLASTICITY CHART
 CLAYEY SILT, trace sand, trace gravel (CL)
 (FILL)

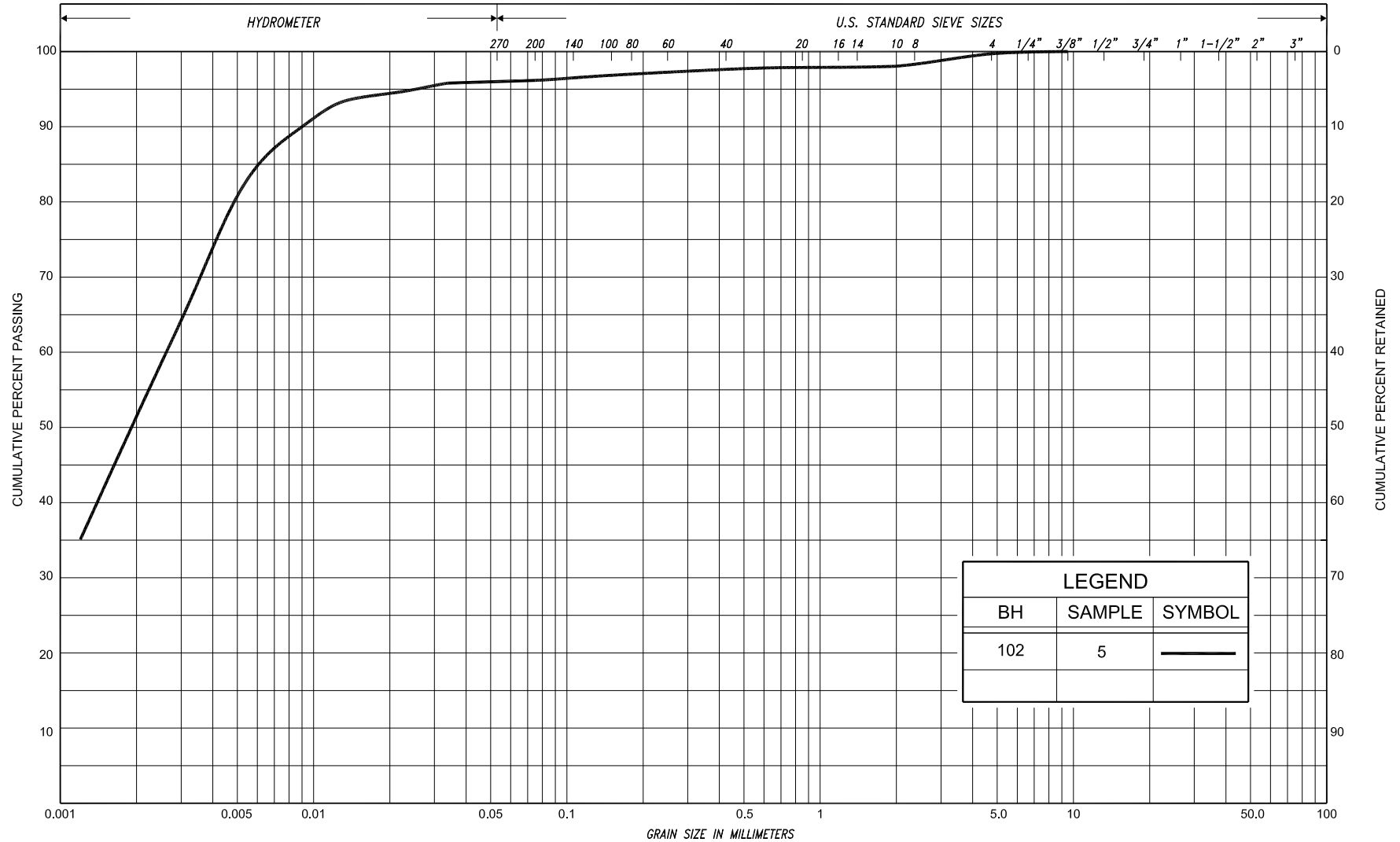
FIG No. CN-PC-1
 HWY: 400
 G.W.P. No. 2005-11-00



PLASTICITY CHART

CLAYEY SILT, trace sand, trace gravel (CL / CL-ML)

FIG No. CN-PC-2
 HWY: 400
 G.W.P. No. 2005-11-00



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													



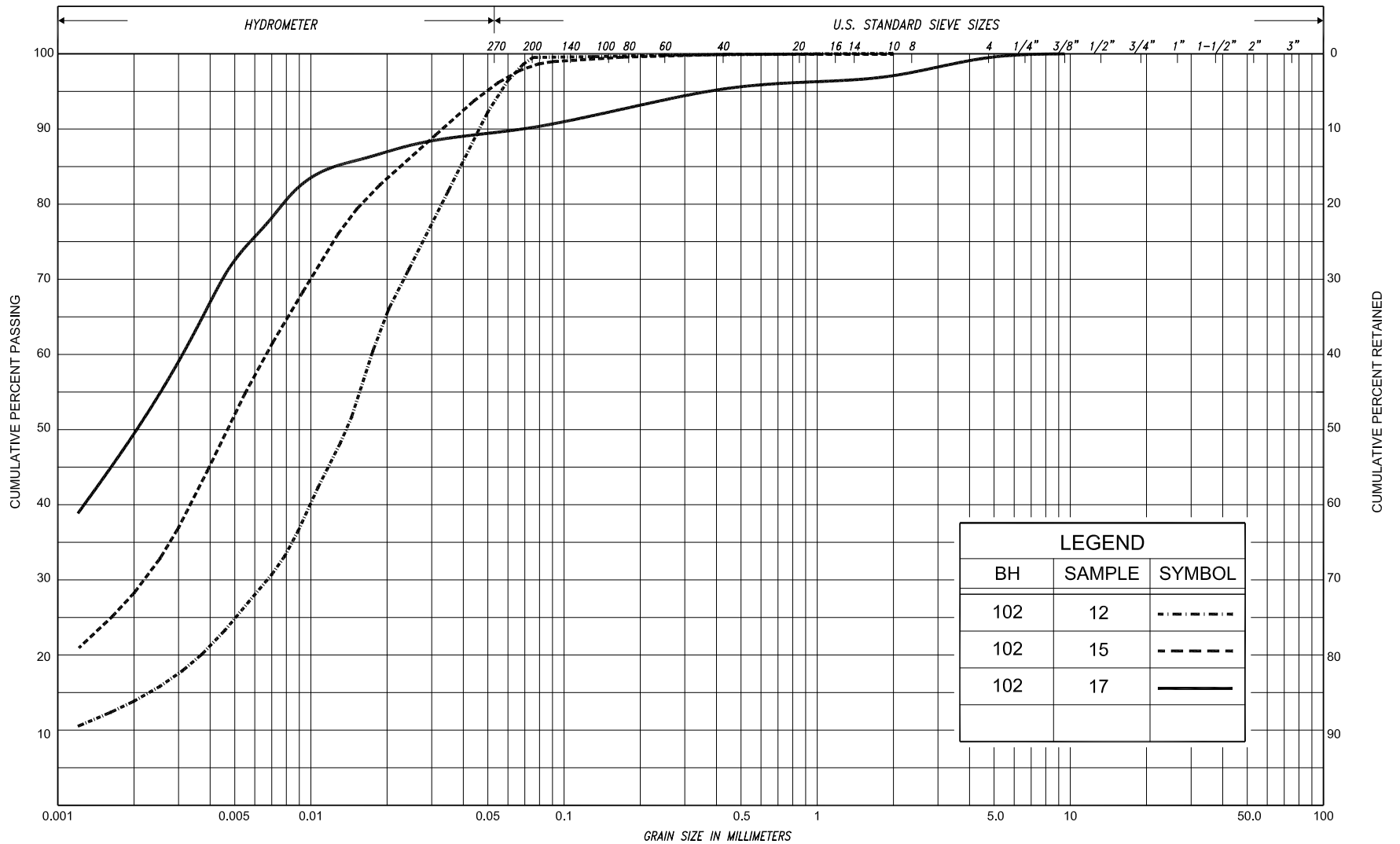
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand, trace gravel (CL)
(FILL)

FIG No. CN-GS-1

HWY:	400
------	-----

G.W.P. No. 2005-11-00



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 sand, trace gravel (CL-ML / ML)

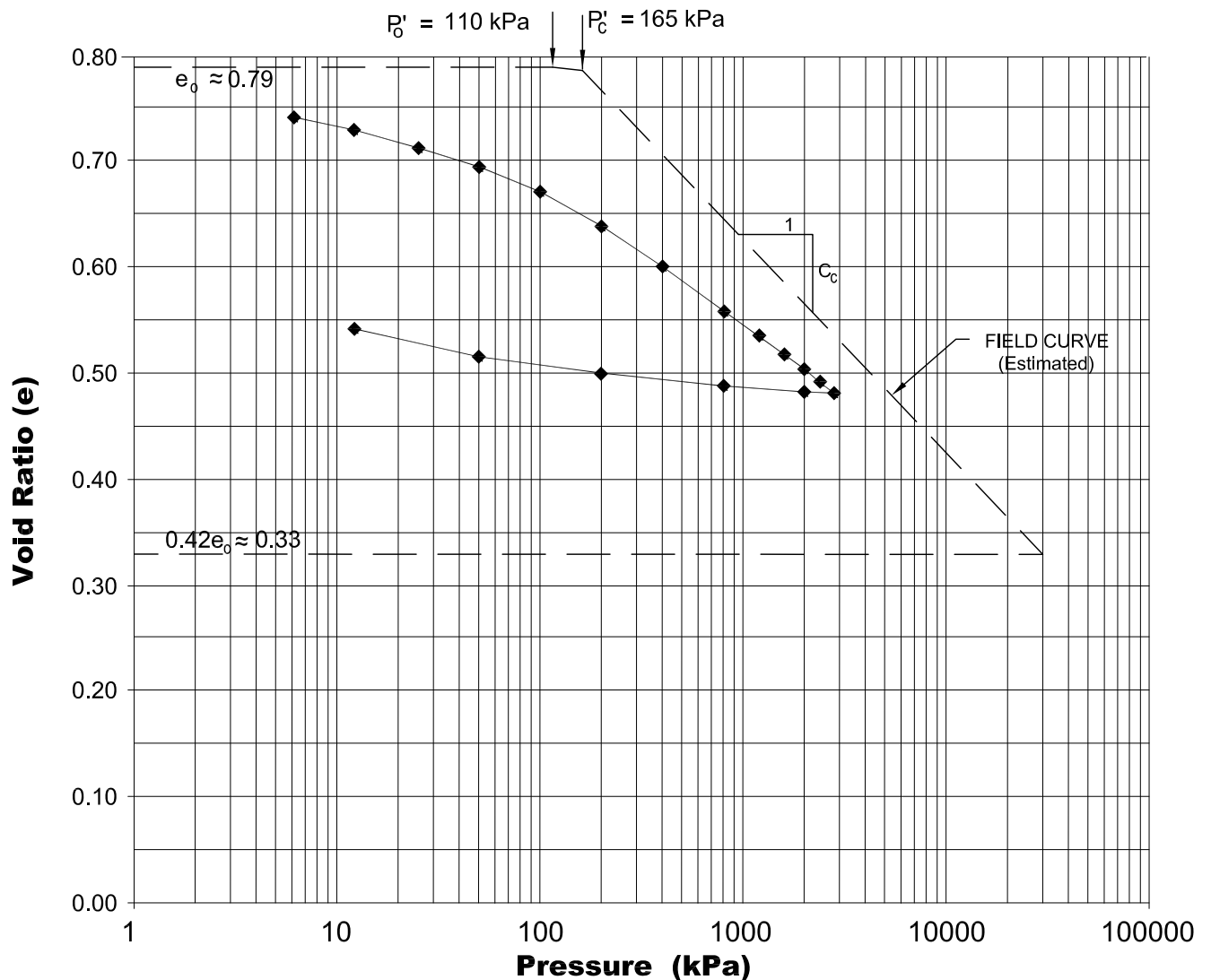
FIG No. CN-GS-2
 HWY: 400
 G.W.P. No. 2005-11-00

Laboratory Consolidation Test Results

Highway 400 / North Canal Road
Overpass Structure Replacement
Ontario

Borehole 102, Sample 12, Depth 12.2 - 12.6 m

Void Ratio versus Log of Pressure



SOIL TYPE: CLAYEY SILT, trace sand (CL-ML)

$e_o \approx 0.79$

$W_o = 30\%$

$\gamma = 19.5 \text{ kN/m}^3$

$P'_o = 110 \text{ kPa}$

$P'_c = 165 \text{ kPa}$

$C_c = 0.20$

$C_r = 0.02$

$W_L = 27$

$W_P = 21$

$PI = 6$

FIGURE No: C-1

HIGHWAY: 400

NORTH CANAL BRIDGE

G.W.P.: 5112-07-00

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/0.3m)	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 101

1 of 3

METRIC

G.W.P. 2005-11-00 **LOCATION** Co-ords: 4 879 222.9 N ; 295 971.0 E **ORIGINATED BY** F.P.
DIST Central **HWY** 400 **BOREHOLE TYPE** C.F.H.S.A. and Mud Rotary **COMPILED BY** G.D.
DATUM Geodetic **DATE** October 27 to 29, 2014 **CHECKED BY** B.R.G.

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									
							○ UNCONFINED + FIELD VANE									
							● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)											
229.1	Ground Surface					20	40	60	80	100	20	40	60			
0.0	230mm asphalt over sand and gravel															
228.7	(PAVEMENT FILL)		1	SS	15											
0.4	Sand, trace gravel asphaltic pieces															
228.2	Compact Brown Moist silty		2	SS	7											
0.9	Loose to dense		3	SS	34											
			4	SS	27											
226.2	Clayey silt trace sand, trace gravel															
2.9	Stiff Brown Moist sand seams		5	SS	11											
			6	SS	13											
			7	SS	16											
			8	SS	15											
	(FILL)															
			9	SS	7											
			10	SS	8											
219.2	Clayey silt, trace sand															
9.9	Firm to Grey Moist stiff		11	SS	5											
				FV												
			12	SS	2											
				FV												
	with sand, trace gravel															
	Firm to soft		13	SS	7											
				FV												
214.1	Cont'd															

RECORD OF BOREHOLE No 101

2 of 3

METRIC

G.W.P.	2005-11-00	LOCATION	Co-ords: 4 879 222.9 N ; 295 971.0 E	ORIGINATED BY	F.P.
DIST	Central	HWY	400	BOREHOLE TYPE	C.F.H.S.A. and Mud Rotary
				COMPILED BY	G.D.
DATUM	Geodetic	DATE	October 27 to 29, 2014	CHECKED BY	B.R.G.


[illegible]

RECORD OF BOREHOLE No 101

3 of 3

METRIC

G.W.P. 2005-11-00 **LOCATION** Co-ords: 4 879 222.9 N ; 295 971.0 E **ORIGINATED BY** F.P.
DIST Central **HWY** 400 **BOREHOLE TYPE** C.F.H.S.A. and Mud Rotary **COMPILED BY** G.D.
DATUM Geodetic **DATE** October 27 to 29, 2014 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)												
199.1							20	40	60	80	100	20	40	60			
30.0	Clayey silt some sand, trace gravel					199											
	Hard Grey Moist (TILL)																

RECORD OF BOREHOLE No 102

1 of 3

METRIC

G.W.P. 2005-11-00 **LOCATION** Co-ords: 4 879 244.7 N ; 296 006.8 E **ORIGINATED BY** F.P.
DIST Central **HWY** 400 **BOREHOLE TYPE** C.F.H.S.A. and Mud Rotary + Dynamic Cone Penetration Test **COMPILED BY** G.D.
DATUM Geodetic **DATE** October 29 to 31, 2014 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							WATER CONTENT (%)										
228.9	Ground Surface							20	40	60	80	100					
0.0	225mm asphalt over sand and gravel																
228.5	(PAVEMENT FILL)		1	SS	43								○				
0.4	Sand, trace gravel																
228.0			2	SS	19		228						○				
0.9	silty																
	Compact Brown Moist																
227.2			3	SS	13		227						○				
1.7	Clayey silt trace sand, trace gravel																
	Stiff Brown Moist to firm																
	(FILL)		4	SS	5		226						○				
							225										
			5	SS	9		224						○				1 3 44 52
							223										
			6	SS	7		222						○				
							221						○				
			7	SS	13		220						○				
			8	SS	8								○				
	Grey		9	SS	6		219						○				
219.1																	
9.8	Topsoil						219						○				
218.9																	
10.0	Sand, trace silt		10	SS	22								○				
	Compact Grey Wet																
218.3																	
10.6	Clayey silt, trace sand		11	SS	4		218						○				
	Soft to Grey Moist very stiff			FV													
							217										
			12	TW	PH								○				0 1 85 14
				FV			216										
			13	TW	PH		215										
			14	SS	18												
				FV			214										
213.9																	

RECORD OF BOREHOLE No 102

2 of 3

METRIC

G.W.P. 2005-11-00 **LOCATION** Co-ords: 4 879 244.7 N ; 296 006.8 E **ORIGINATED BY** F.P.
DIST Central **HWY** 400 **BOREHOLE TYPE** C.F.H.S.A. and Mud Rotary + Dynamic Cone Penetration Test **COMPILED BY** G.D.
DATUM Geodetic **DATE** October 29 to 31, 2014 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							WATER CONTENT (%)										
213.9 15.0	Clayey silt, trace sand Firm to Grey Moist stiff (Cont'd.)		15	SS	7		213									0 2 70 28	
							212										
211.5 17.4	Sand with gravel, trace silt Dense Grey Moist silty trace clay, trace gravel		16	SS	41		211										
							210										
209.4 19.5	Clayey silt trace sand, trace gravel Stiff Grey Moist						209										
							208										
			17	SS	10		207									1 9 40 50	
							206										
							205										
			18	SS	12		204										
							203										
202.7 26.2	Sand, with silt some gravel, trace clay Compact Grey Moist to dense (TILL)		19	SS	27		202										
							201										
	sandy gravel layer cobbles						200										
198.9	Compact Cont'd						199										

RECORD OF BOREHOLE No 102

3 of 3

METRIC

G.W.P. <u>2005-11-00</u>	LOCATION <u>Co-ords: 4 879 244.7 N ; 296 006.8 E</u>	ORIGINATED BY <u>F.P.</u>
DIST <u>Central</u> HWY <u>400</u>	BOREHOLE TYPE <u>C.F.H.S.A. and Mud Rotary + Dynamic Cone Penetration Test</u>	COMPILED BY <u>G.D.</u>
DATUM <u>Geodetic</u>	DATE <u>October 29 to 31, 2014</u>	CHECKED BY <u>B.R.G.</u>

[illegible]

**DRAFT PAVEMENT DESIGN REPORT
HIGHWAY 400, NORTH CANAL OVERPASS STRUCTURE REPLACEMENT
GWP 2005-11-00**

Appendix B: Borehole Logs

		9.5 mm = 87%	250 µm = 56%
		4.75 mm = 76%	75 µm = 35%
		2.36 mm = 67%	5 µm = 13%
		300 µm = 40%	2 µm = 7%
		75 µm = 24%	LSFH
		LSFH	w = 8%
		Accep SSM	K-Factor 0.09
		w = 6%	
850	- 1.5	Si W Sa W Cl	
10+495, 16.5 Lt, SCL (Power Auger)			
0	- 295	Asph	
295	- 450	Gr and Sa Tr Si	
		%Passing	
		19.0 mm = 100%	
		13.2 mm = 89%	
		9.5 mm = 71%	
		4.75 mm = 46%	
		2.36 mm = 38%	
		300 µm = 13%	
		75 µm = 4%	
		LSFH	
		Accep Gran B Type I	
		w = 8%	
450	- 1.7	Sa W Si some Gr	
		%Passing	
		19.0 mm = 100%	
		13.2 mm = 98%	
		9.5 mm = 95%	
		4.75 mm = 89%	
		2.36 mm = 85%	
		300 µm = 57%	
		75 µm = 26%	
		LSFH	
		w = 6%	
1.7	- 4.5	Si Cl some Sa	
		W _L = 23%	
		W _P = 9%	
		I _P = 14%	
		w = 20%	
10+495, 19.9 Rt, EP (Power Auger)			
0	- 220	Asph	
220	- 450	Sa W Cr Gr Tr Si	
450	- 900	Sa some Si Tr Gr	
900	- 1.5	Dk Br Cl Si some Sa some Gr	
		%Passing	
		19.0 mm = 100%	
		13.2 mm = 98%	
		9.5 mm = 95%	
		4.75 mm = 88%	
		2.00 mm = 82%	
10+495, 19.9 Lt, EP (Power Auger)			
0	- 280	Asph	
280	- 460	Sa and Cr Gr Tr Si	
460	- 1.4	Si Sa Tr Gr	
1.4	- 4.5	Si Cl some Sa	
		1.5-1.8 Occ Cob	
10+495, 21.5 Rt, SHR (Curb) (Power Auger)			
0	- 210	Asph	
210	- 480	Sa Cr Gr Tr Si	
480	- 800	Br Sa some Si Tr Gr	
800	- 1.5	Dk Br Si Sa Tr Gr, Moist	
10+495, 21.7 Lt, MSH (Power Auger)			
0	- 190	Asph	
190	- 470	Sa and Cr Gr Tr Si	
470	- 600	Sa W Gr some Si	
600	- 1.5	Si Sa Tr Gr	
10+495, 35.0 Lt, D-5.2 (Hand Auger)			
0	- 240	Tps	
240	- 500	Gry Sa Si some Cl Tr Gr	
500	- 950	Si Cl some Sa	
950	- 3	Blk Org (Moist 1.7 down)	
10+495, 36.0 Lt, D-5.2 (Hand Auger)			
0	- 250	Tps	
250	- 650	Gry Sa Si some Cl Tr Gr	
650	- 950	Gry Si Cl some Sa	
950	- 3.1	Blk Org	
3.1	- 4.5	Sa Si some Cl	
10+495, 35.5 Rt, D-6.0 (Hand Auger)			
0	- 200	Tps	
200	- 400	Br Si Sa some Gr	
400	- 800	Gry Si Cl some Sa	
800	- 3.3	Blk Org	
3.3	- 4.5	Gry Sa Si some Cl	
10+495, 39.6 Rt, D-6.1 (Hand Auger)			
0	- 200	Tps	
200	- 320	Br Sa W Gr some Si	
320	- 870	Gry Sa Si Tr Gr Tr Cl	
870	- 1.5	Blk Org	

**DRAFT PAVEMENT DESIGN REPORT
HIGHWAY 400, NORTH CANAL OVERPASS STRUCTURE REPLACEMENT
GWP 2005-11-00**

Appendix B: Borehole Logs

10+500, 20.2 Rt, SHR (Power Auger)

0	-	150	Tps
150	-	1.5	Br Si Sa Tr Gr, Moist

10+515, 4.2 Lt, 0.3 Lt EP (Power Auger)

0	-	225	Asph
225	-	460	Sa W Cr Gr Tr Si
460	-	720	Sa W Gr some Si
720	-	1.5	Br Cl Si W Sa Tr Gr, Firm, Moist

10+595, 4.4 Lt, EP (Power Auger)

0	-	260	Asph
260	-	450	Sa W Cr Gr Tr Si
450	-	700	Sa W Gr some Si
700	-	1.5	Br Cl Si W Sa Tr Gr, Moist, Firm

10+595, 4.8 Rt, EP (Power Auger)

0	-	170	Asph
170	-	350	Sa W Cr Gr Tr Si
350	-	720	Sa W Gr some Si
720	-	1.5	Br Cl Si W Sa Tr Gr, Moist, Firm

10+595, 19.6 Rt, EP (Power Auger)

0	-	270	Asph
270	-	460	Sa W Cr Gr Tr Si
460	-	800	Br Sa W Si some Gr
800	-	1.5	Gry Si Sa Tr Gr

10+595, 19.8 Lt, EP (Power Auger)

0	-	410	Asph
410	-	650	Br Sa W Gr some Si
650	-	1.5	Br Sa W Si Tr Gr

10+595, 21.6 Rt, SHR(Guard Rail) (Power Auger)

0	-	140	Asph
140	-	390	Sa W Cr Gr Tr Si
390	-	770	Sa W Si some Gr
770	-	1.5	Br Sa Si Tr Gr, Moist

10+595, 25.0 Lt, MSH (Power Auger)

0	-	230	Asph
230	-	480	Br Sa W Gr some Si
480	-	850	Br Sa W Si some Gr
850	-	1.5	Br Si Sa Tr Gr

10+595, 38.0 Rt, D-8.2 (Hand Auger)

0	-	350	Tps
350	-	600	Br Sa W Gr W Si
600	-	1	Blk Org
1	-	1.5	Gry Sa W Si

10+595, 41.6 Lt, D-7.5 (Hand Auger)

0	-	250	Tps
250	-	600	Br Sa and Si Tr Gr
600	-	1.2	Blk Org, Muskeg
1.2	-	1.5	Gry Si Sa

10+620, 38.0 Rt, D-8.3 (Hand Auger)

0	-	280	Tps
280	-	750	Br Sa Si W Cl Tr Gr
750	-	1.35	Blk Org
1.35	-	1.5	Gry Si Sa Tr Cl

10+620, 41.5 Rt, D-7.8 (Hand Auger)

0	-	280	Tps
280	-	700	Br Sa Si some Gr
700	-	1.4	Blk Org
1.4	-	1.5	Gry Sa Si some Cl

10+645, 3.8 Lt, EP (Power Auger)

0	-	190	Asph
190	-	360	Sa W Cr Gr Tr Si
360	-	590	Sa W Gr some Si
590	-	1.5	Br Cl Si W Sa, Firm, Dry

10+645, 3.9 Rt, 0.5 Lt EP (Power Auger)

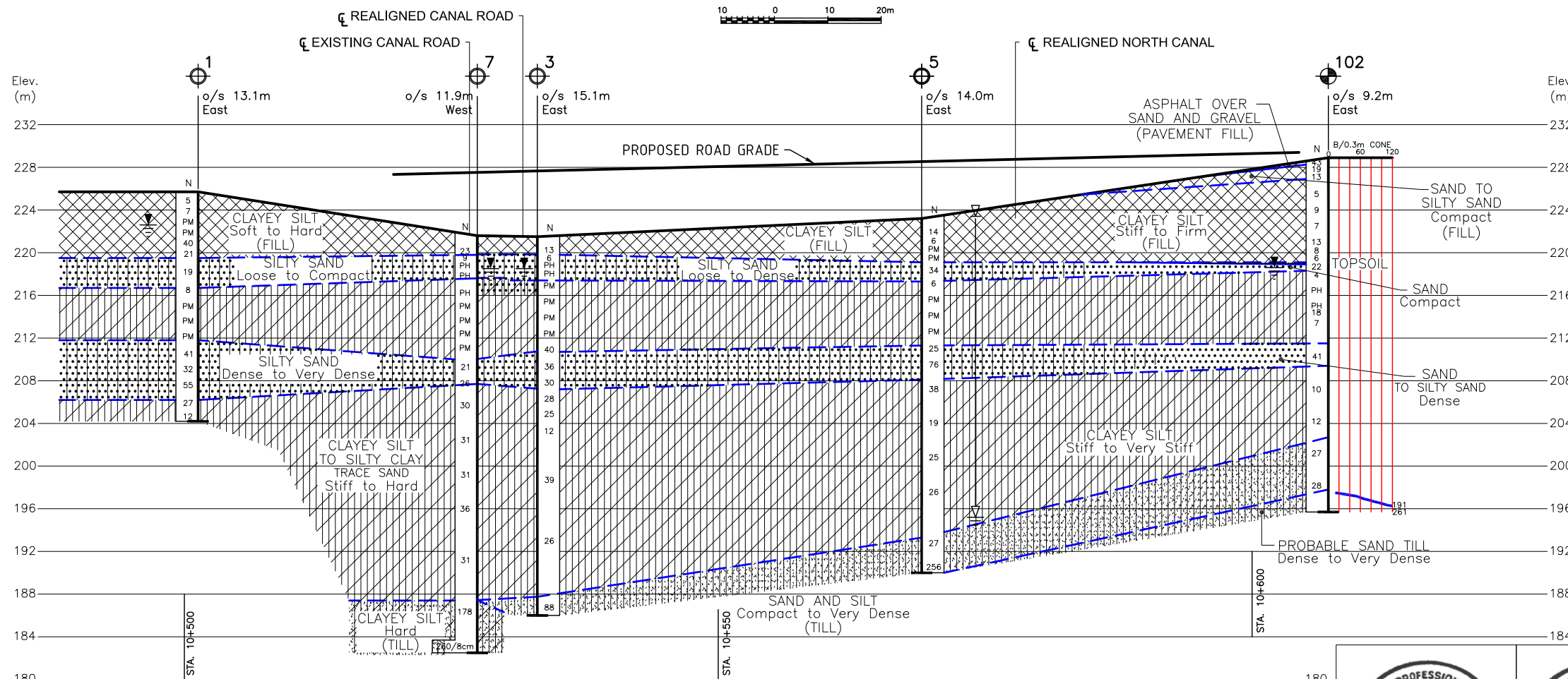
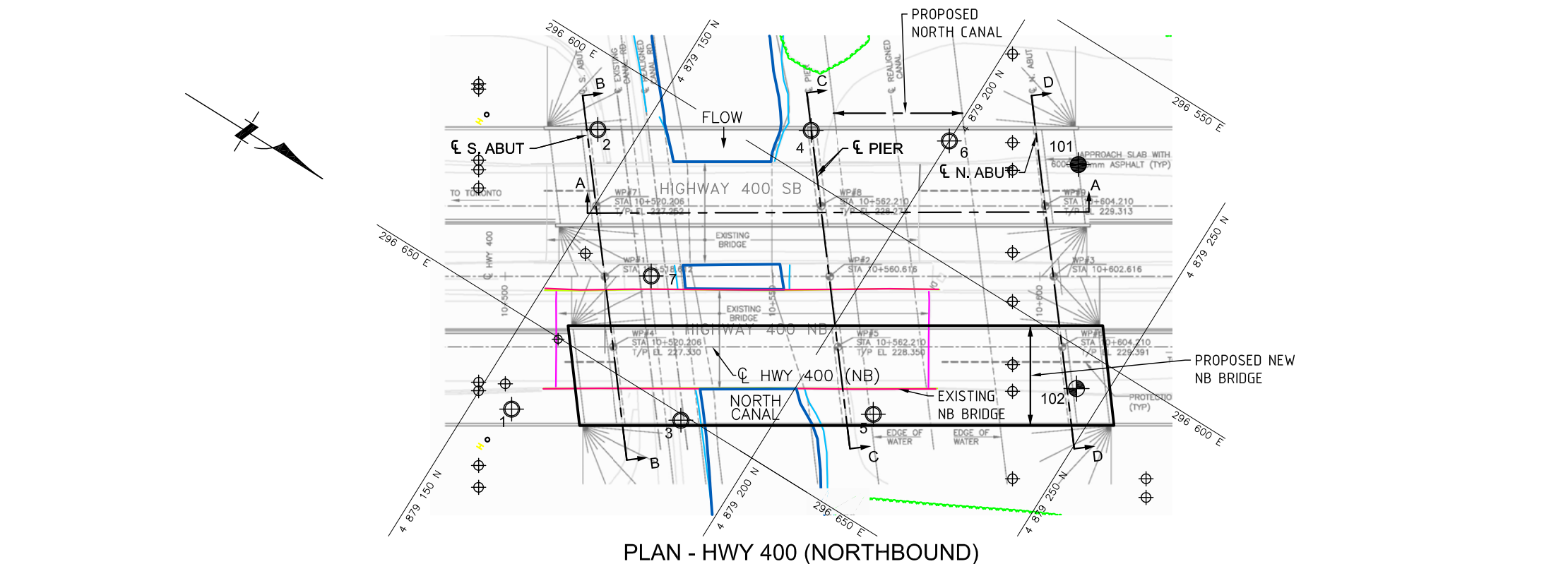
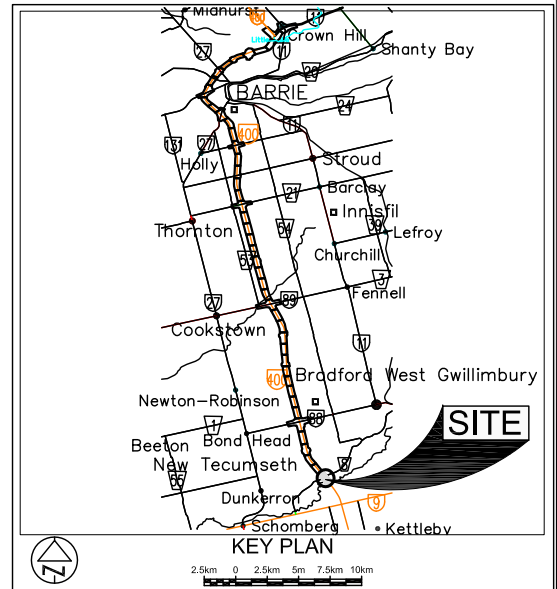
0	-	190	Asph
190	-	380	Sa W Cr Gr Tr Si
380	-	610	Sa W Gr some Si
610	-	1.5	Br Cl Si W Sa, Firm, Moist

10+645, 6.3 Rt, N1 (Power Auger)

0	-	255	Asph
255	-	500	Sa and Cr Gr Tr Si
500	-	850	Sa W Gr some Si
850	-	1.5	Cl Si W Sa Tr Gr

%Passing

19.0 mm	=	100%
13.2 mm	=	9%
9.5 mm	=	97%
4.75 mm	=	95%
2.00 mm	=	94%
250 µm	=	83%
75 µm	=	75%
5 µm	=	48%
2 µm	=	26%
HSFH		
w	=	18%
K-Factor		0.27



- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
 - REFER TO DRAWING CN-2 FOR SECTIONS B-B, C-C & D-D.
 - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

PROFILE A - A



LEGEND			
	Borehole		
	Borehole and Cone		
	Geocres Borehole		
	Pavement Borehole by Stantec Consulting Ltd.		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation (Oct.-Nov. 1970 and Oct. 2014)		
PM	Pushed Mechanically		
PH	Pushed Hydraulically		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
BH No	ELEVATION	NORTHINGS	EASTINGS
101	229.1	4 879 223	295 971
102	228.9	4 879 245	296 007
1	225.7	4 879 157	296 066
2	221.6	4 879 143	296 013
3	221.5	4 879 185	296 051
4	220.9	4 879 177	295 992
5	223.2	4 879 215	296 031
6	227.7	4 879 200	295 980
7	221.6	4 879 166	296 031

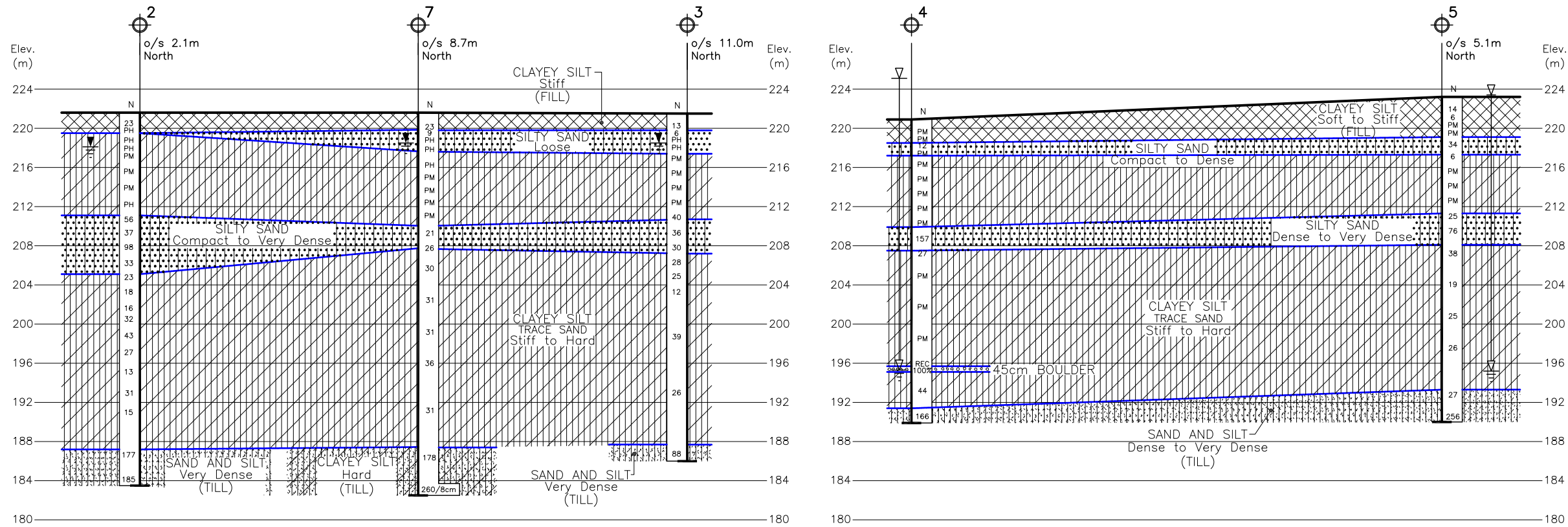
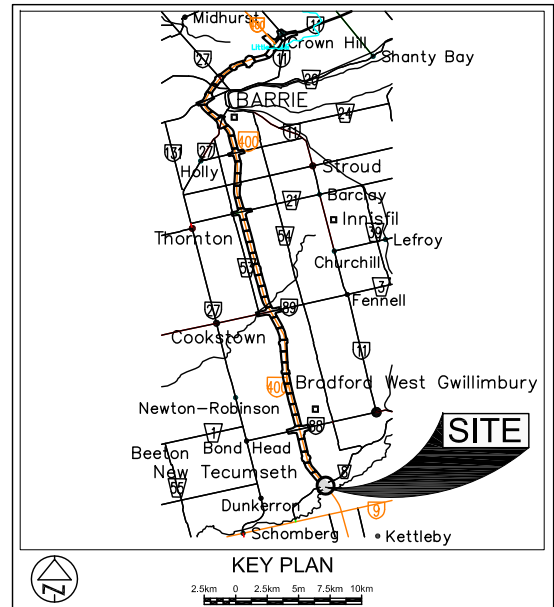
NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

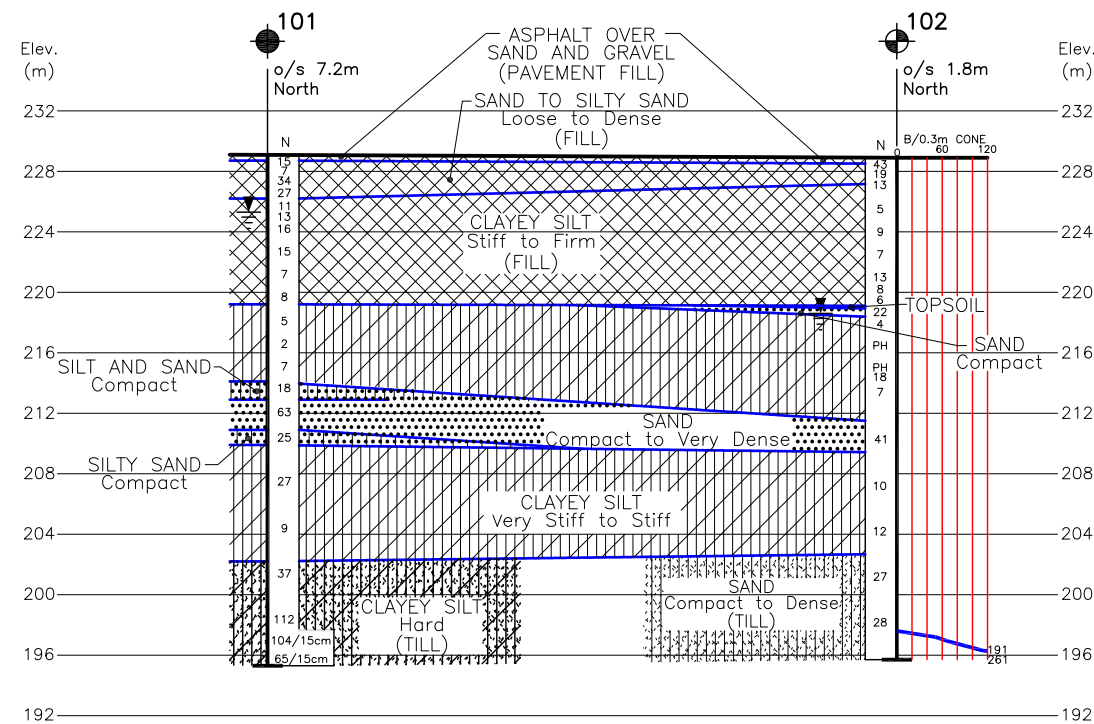
Geocres No. 31D-604			
HWY No	400	CHECKED	GD
SUBM'D	NA	DATE	JUNE 05, 2015
DRAWN	NL	APPROVED	CN
DIST	Central	SITE	30-334/1
DWG	CN-1		





SECTION B - B

SECTION C - C



SECTION D - D

NOTES:

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



LEGEND			
	Borehole		
	Borehole and Cone		
	Geocres Borehole		
	Pavement Borehole by Stantec Consulting Ltd.		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation (Oct.-Nov. 1970 and Oct. 2014)		
PM	Pushed Mechanically		
PH	Pushed Hydraulically		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
BH No	ELEVATION	NORTHINGS	EASTINGS
FOR DETAILS, REFER TO DRAWING CN-1			

- NOTE -

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

REVISIONS			
DATE	BY	DESCRIPTION	

Geocres No. 31D-604

HWY No	400	CHECKED	GD	DATE JUNE 05, 2015	DIST	Central
SUBM'D	NA	CHECKED	BRG	APPROVED	SITE	30-334/1
DRAWN	NL	CHECKED	CN	CN	DWG	CN-2



APPENDIX A

GEOCRES No. 31D-30

“Foundation Investigation Report for Proposed Extensions to the Overpass
Structures at the Crossing of Highway 400 and the North Canal Road and Drainage
Canal, Township of West Gwillimbury, County of Simcoe”

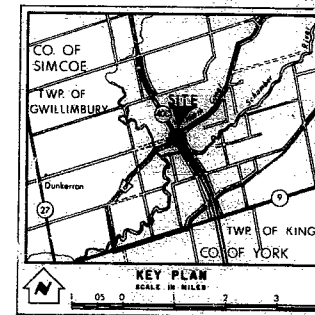
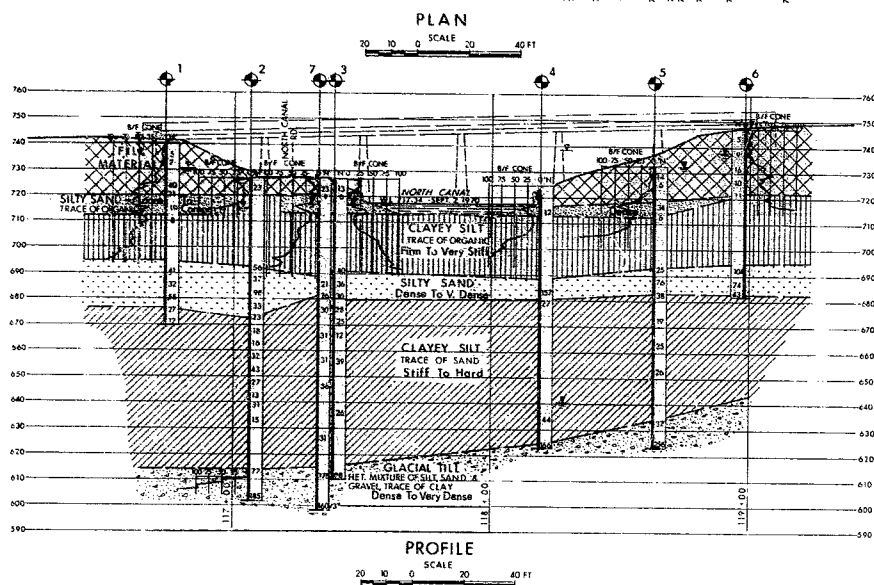
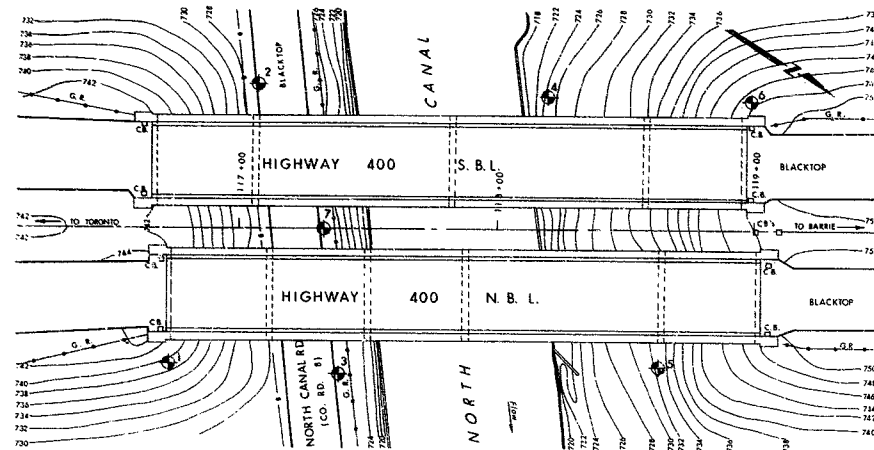
CONT. 71-12

HWY. 400 +

NORTH CANAL RD.

+ DRAINAGE CANAL

31 D-30



LEGEND				
	Bore Hole			
	Cone Penetration Hole			
	Bore & Cone Penetration Hole			
	Water Levels established at time of field investigation Oct.-Nov. 1970			
	Head of Artesian Water Encountered			
NO.	ELEVATION	STATION	OFFSET	
1	740.4	116+73	32.5' RT.	
2	726.9	117+07	56.5' LT.	
3	726.8	117+39	56.0' RT.	
4	724.6	118+20	52.5' LT.	
5	732.3	118+63	52.5' RT.	
6	747.1	118+98	51.0' LT.	
7	726.9	117+33	6	

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence and may be subject to considerable error.

SECTION	DATE	REVISIONS

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE - FOUNDATION SECTION

NORTH CANAL & COUNTY RD. 8

KING'S HIGHWAY NO. 400 DIST. NO. 5
CO. SIMCOE
TWP. KING LBY 8 CON. 2 & 3

BORE HOLE LOCATIONS & SOIL STRATA

DRAWN BY: [Signature] DATE: 10/25/70
CHECKED BY: [Signature] DATE: 10/25/70
APPROVED BY: [Signature] DATE: 10/25/70

REF No. E-4994-1

GEORES NO. 31D-30

MEMORANDUM

(RM. 110 CAB. Bldg.)

71-012

31D-30
GEOCRE No.

Mr. S. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. S. McCombie

DATE: December 4, 1970

OUR FILE REF.

IN REPLY TO **DEC 10 1970**

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For

Proposed Extensions to the Overpass
Structures at the Crossing of Hwy. #400
& the North Canal Rd. and Drainage Canal
Twp. of W. Gwillimbury, Co. of Simcoe
District No. 6 (Toronto)

W.O. 70-11090 -- W.P. 105-70-05

CONT 71-012 Site 30-334

Attached, we are forwarding to you our detailed
Foundation investigation report on the subsoil conditions
existing at the above structure site.

We believe that the factual data and recommendations
contained therein, will prove adequate for your design
requirements. Should additional information be required,
please do not hesitate to contact our Office.

AGJ/MdeF
Attach.

cc: Messrs. B. R. Davis
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
H. Greenland
G. C. E. Burkhardt (2)
I. J. Kovich
B. J. Giroux
B. A. Singh

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

Foundations Files
Gen. Files

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 - 4.3) Silty Sand, trace of Organics.
 - 4.4) Clayey Silt.
 - 4.5) Heterogeneous Mixture of Silt, Sand and Gravel - (Glacial Till).
 5. GROUNDWATER CONDITIONS.
 6. EXISTING STRUCTURES.
 7. DISCUSSION AND RECOMMENDATIONS:
 - 7.1) General.
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 - 7.3) Approach Embankments.
 - 7.4) Proposed Retaining Walls.
 8. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT
For

Proposed Extensions to the Overpass
Structures at the Crossing of Hwy. #400
& the North Canal Rd. and Drainage Canal
Twp. of W. Gwillimbury, Co. of Simcoe
District No. 6 (Toronto)
W.O. 70-11090 -- W.P. 105-70-05

1. INTRODUCTION:

Major reconstruction is proposed for Hwy. #400 from the northern limits of Metropolitan Toronto northerly to the outskirts of the City of Barrie. This will involve the extensions of existing structures at various locations. In conjunction with this project, the Foundation Section was requested, in a memo from Mr. G. C. E. Burkhardt, Regional Bridge Planning Engineer, Central Region, dated October 6, 1970, to carry out a subsurface investigation at the crossing of Hwy. #400 and the North Canal Rd. and Drainage Canal. An investigation was subsequently carried out by this Section to determine the subsoil and groundwater conditions at the site.

This report contains the results of the investigation, together with recommendations pertaining to the foundations for the widening of the existing overpass structures, as well as the stability and settlement considerations for the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is situated at the crossing of Hwy. #400 and the North Canal Rd. (Co. Rd. #18) and the Drainage Canal, in the Township of West Gwillimbury. Highway #400 has two paved lanes in both the Northbound and Southbound direction. There are existing structures at the crossing, the pertinent details of which will be discussed in Section #6.

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd.) ...

In the vicinity of the crossing the canal channel is approximately 66 feet wide and 13 feet deep. At the time of the investigation the canal water level was at about elevation 717.5 - i.e., the water is approximately 5 feet deep. The North Canal Road is located on the South bank of the canal; this roadway has 2 paved lanes with the profile grade being at about elevation 727.

The surrounding terrain, which is cultivated and being used for farming purposes, is flat to gently undulating in relief between about elevations 722 to 724.

Physiographically the site is situated in the region known as "Simcoe Lowlands", specifically in the sub-section known as the Lake Simcoe Basin. A broad, flat valley extends in a southwest direction from Cook Bay, located on the southern end of Lake Simcoe. This valley, which is approximately 15 miles in length, is surrounded by high morainic hills. The area under investigation is located within this valley. This basin was at one time part of the floor of glacial Lake Algonquin. Extensive deposits of silt and clay were laid down on the floor of this lake; these deposits are underlain by a glacial till sheet. In some localized areas the silts and clays are overlain by a thin, poorly graded sand which is of deltaic origin. The fact that the valley is a depressed, poorly drained area has led to the formation of a surficial cover of peat, which in the area under investigation, is known to be of the order of 4 to 7 feet in thickness. The Holland River meanders sluggishly along the floor of this valley; this valley is locally referred to as the Holland Marsh.

The overburden is underlain by limestone bedrock of the Trenton-Black formations, Ordovician Period.

3. FIELD AND LABORATORY WORK:

Seven sampled boreholes, each accompanied by a dynamic cone penetration test, were put down at this site. The borings were advanced by means of either a conventional diamond drill, or a power auger machine (Penndrill), both of which were adapted for soil sampling purposes.

Samples of the subsoil were recovered at required depths in a 2" O.D. split-spoon sampler which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. This testing was supplemented by obtaining some 2-inch I.D. Shelby tubes in the cohesive portion of the overburden and fill; these Shelby tubes were advanced by manually pushing them into the subsoil. In addition, in situ vane tests were carried out, wherever possible, in the cohesive deposits.

The groundwater level conditions across the site, during the period of the investigation, were determined by recording the levels in the open boreholes. Artesian groundwater pressure was encountered at B.H.'s #4 and 5. These borings were completely sealed with quick-setting cement following completion of the drilling operations.

The locations and elevations of all the borings were surveyed in the field by personnel from the Engineering Surveys Section, Central Region; they are shown on Drawing 70-11090A, together with an estimated stratigraphical section across the site. All elevations in the report are referenced to a geodetic datum.

All samples were visually examined and identified in the field and later in the laboratory. Following this, laboratory tests were carried out on selected representative samples to determine the following physical properties:

3. FIELD AND LABORATORY WORK: (cont'd.) ...

Atterberg Limits
Natural Moisture Content
Bulk Density
Grain-size Distribution
Undrained Shear Strength

The results of the laboratory testing are plotted on the Record of Borelog sheets and summarized on Figures #1 to 5, all of which are located in the Appendix of this report.

4. SUBSOIL CONDITIONS:

4.1) General:

The predominant stratum across the site is composed of a firm to hard clayey silt; the overall thickness of this stratum varies from 79 to 106 feet. An 8 to 19-foot thick layer of dense silty sand is sandwiched within the cohesive stratum. Further, in some localized areas the clayey silt is covered by a loose to compact deposit of silty sand. The clayey silt is underlain by a competent basal glacial till sheet.

The approach embankments, which have a maximum height of 27 feet, are composed of firm to stiff clayey silt fill.

The boundaries of the various deposits are shown on the accompanying borelog sheets. The stratigraphical profile plotted on Log. #70-11090A, is inferred from this boring data.

From ground surface downwards, the various soil types encountered are as follows:

4.2) Fill Material - Clayey Silt:

All the borings penetrated through the approach fills; the encountered thickness varied from 5-1/2 feet at B.H. #3 to 25 feet at B.H. #6. The fill is composed of a clayey silt of low plasticity with a trace of sand and gravel throughout. Grain-size distribution curves, for samples obtained from the fill are plotted on Figure #1.

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Fill Material - Clayey Silt: (cont'd.) ...

The undrained shear strength of the fill, as determined by field and laboratory testing, are plotted on the Borelog sheets. The values obtained range from 500 to greater than 2,000 p.s.f., being typically of the order of 800 p.s.f. Based on this testing, it is concluded that the fill has been compacted to a moderate degree.

4.3) Silty Sand, trace of Organics:

A surficial layer of loose to dense ('N' values 6 to 34 blows/ft.) silty sand was generally encountered below original ground surface. This layer was not present at B.H.'s #2 and 6. The thickness of this layer varies from 4 feet at B.H. #4 to 9 feet at B.H. #1. Traces of organic matter is present at random, throughout the silty sand; at B.H. #3 a 1-foot thick organic layer was encountered. Grain-size distribution curves for samples of the granular subsoil are plotted on Figure #3.

4.4) Clayey Silt:

The predominant stratum across the site underlies the silty sand deposit, where it exists, and original ground surface elsewhere. It is composed of a clayey silt with a trace of sand; numerous silty sand seams and layers, which vary from 1/4 inch to up to 1 foot in thickness, were encountered throughout the stratum. The overall thickness of the cohesive stratum, where fully penetrated, varies from 78.5 feet at B.H. #5 to 106 feet at B.H. #2. Grain-size distribution curves, for samples of the cohesive stratum, are plotted, in envelope form, on Figure #2.

A 7.5 to 19.5 foot thick deposit of compact to very dense ('N' values 19 to 104 blows/ft.) silty sand to sandy silt is sandwiched within the clayey silt stratum. Occasional clayey silt seams, up to 1/2 inch in thickness, are located throughout. Typical grain-size distribution curves for the material are plotted on Figure #3.

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.4) Clayey Silt: (cont'd.) ...

The engineering properties of the cohesive stratum, which are plotted on the "Record of Borelog sheets, are also summarized in the table to follow. The properties of the clayey silt above and below the granular deposit are quite different. These two portions are, therefore, differentiated in the table.

		<u>Above Granular Deposit</u>	<u>Below Granular Deposit</u>
<u>Identity Tests:</u>		<u>Range</u> <u>(Average)</u>	<u>Range</u> <u>(Average)</u>
Bulk Density	(γ)	126 - 134 (127)	126 and 129
(p.s.f.)			
Liquid Limit	(W_L)	22 - 29.5 (25)	22 - 38.5 (33)
(%)			
Plastic Limit	(W_P)	13.5 - 19.5 (15)	14.5 - 19.5 (18)
(%)			
Natural Moisture Content	(W)	18 - 24 (20)	16.5 - 27.5 (22)
(%)			
Liquidity Index	(I_L)	0.2 - 0.8 (0.4)	0.3 - 0.3 (0.4)
<u>Undrained Shear Strength (C_u)</u>			
(p.s.f.)			
In Situ Field Vane Tests		700 - > 2,000	-
Laboratory Tests		650 - > 2,000	-
Sensitivity	(S_t)	2 - 5 (3)	-
<u>Standard Penetration Resistance ('N')</u>			
(Blows/ft.)		6 - 11 (8)	12 - 43 (36)

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.4) Clayey Silt: (cont'd.) ...

The results of the Atterberg limit tests are summarized on Figure #5. The values indicate that the cohesive stratum is inorganic and of low plasticity. The natural moisture content is located between the liquid and plastic limit, as represented by liquidity indices.

The results of the undrained shear strength and standard penetration testing carried out, indicate that the consistency of the upper zone varies from firm to very stiff. The lower zone, however, is in the stiff to hard range.

4.5) Heterogeneous Mixture of Silt, Sand and Gravel - (Glacial Till):

The cohesive stratum is underlain by a dense to very dense ('N' values 37 blows/ft. to 160 blows/3 inches), basically granular glacial till. The borings were terminated in this stratum; the maximum penetration was 12 feet (B.H. #2). The glacial till is primarily composed of silt and sand binding gravel sizes; a trace of clay is present throughout. In some localized areas, however, the glacial till is cohesive in nature - i.e., it is composed of a matrix of clayey silt binding sand and gravel. Grain-size distribution curves, for samples obtained with 2-inch O.D. sampling equipment, are plotted on Figure #4.

5. GROUNDWATER CONDITIONS:

Groundwater level observations were carried out in the open boreholes, during the period of the field investigation. The results of these observations are plotted on the "Record of Borelog" sheets and on Drawing No. 70-110904. The observations indicate that the static groundwater level, in the vicinity of the canal, is at about the same level as the canal water level (717.5). From the banks of the canal back along the approaches, to the existing structures, however, the groundwater level rises until it reaches an elevation of about 738. This trend would

5. GROUNDWATER CONDITIONS: (cont'd.) ...

indicate that there is a natural hydraulic gradient towards the canal. These elevations correspond to depths of between 9 and 11.5 feet below existing ground surface.

At B.H.'s #4 and 5, an artesian groundwater pressure was encountered within the lower portion of the overburden. At B.H. #4 this condition was encountered once the casing penetrated to about elevation 640.5. At this point the water rose instantaneously in the casing, finally stabilizing itself at elevation 738.5 - i.e., some 14 feet above the ground surface. It is pertinent to note that a bouldery granular zone is present at the depth at which the artesian head was first encountered. It is inferred that this relatively pervious zone, which is confined on top and bottom by impervious subsoil, is acting as an aquifer. As such, it is probably being charged with groundwater from the surrounding terrain which is at a higher elevation. The artesian head encountered at B.H. #5 was small compared to that discussed above. Both holes were completely sealed with quick-setting cement following completion of the drilling operations.

6. EXISTING STRUCTURES:

The existing twin-parallel overpass structures, which are approximately 16 feet apart, have 6 spans (38'-38'-38'-38'-38'-38') and are 26 feet wide. The piers and 'perched' abutments are founded on 40 to 60 feet long timber piles, the tips of which are probably located within the lower sand deposit.

The profile grade of Hwy. #400, in the vicinity of the structures, varies from elevation 744 along the south approach to elevation 750 along the north. At this grade the heights of the existing south and north approach embankments are of the order of 20 and 24 feet, respectively. The slopes of the approaches in the transverse and longitudinal directions are of the order of 2-1/2:1 and 3:1, respectively.

6. EXISTING STRUCTURES: (cont'd.) ..

In general, the structures and approaches appear to be performing quite satisfactorily.

The canal banks have sloughed continuously since the construction period. In order to prevent this, 10-foot long steel sheet pile walls were driven along either bank in 1961. Some bulging of these walls toward the canal has occurred. This lateral movement is no doubt due to the surcharge loading effect of the approach fills.

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General:

In order to accommodate additional traffic lanes, it is proposed to widen the existing twin overpass structures at the crossing of Hwy. #400 and the North Canal Rd. and Drainage Canal, in the Township of West Gwillimbury, County of Simcoe. The Southbound and Northbound Lane structures will be widened in a westerly and easterly direction, respectively; the amount of widening in both cases is to be of the order of 25 feet. The existing alignment and profile grade of Hwy. #400 is to be maintained. The widened portion of the fills will, therefore, have a height comparable to the existing sections - i.e., be between 22 and 24 feet above the original ground surface.

The predominant overburden deposit across the site is composed of a firm to hard clayey silt, the overall thickness of which varies from 73.5 to 106 feet. A dense to very dense silty sand layer (7.5 to 19.5 feet thick) is sandwiched within the cohesive deposit. Further, a surficial cover of loose to compact silty sand with a trace of organic matter (4 to 9 feet thick) periodically overlies the cohesive subsoil.

The aforementioned subsoil sequence is underlain by a competent basically granular glacial till deposit.

The existing embankments are basically composed of a clayey silt material which has been subjected to a moderate degree of compaction.

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

7.2) Structure Foundations:

The presence of the upper firm zones of the cohesive stratum, at a shallow depth below original ground surface, precludes the use of shallow foundations for the proposed extensions. It is recommended, therefore, that the foundations for the abutment and pier extensions be supported on piles. One possibility would be to support the extensions in a manner similar to the existing foundations, that is, on timber piles driven into the dense granular layer sandwiched within the cohesive stratum. Alternatively, the extensions could be founded on end-bearing piles driven to practical refusal within the basal glacial till deposit. These two schemes will be discussed separately below.

1) Timber Piles Driven into the Dense Granular Layer -

The tips of the timber piles would be driven into the dense to very dense granular layer, the surface of which varies between elevations 689 and 695 (refer to Drawing W.O. 70-11090A). The pile lengths would be of the order of 35 to 40 feet at the pier location and up to 50 feet at the abutments. No. 14 timber pile sections could be designed using an allowable load of about 40 tons/pile. If this scheme is adopted this Section will provide the estimated elevations for the pile tips at the respective foundation extensions.

The foundation subsoil sequence, located beneath the pile groups, will settle due to the surcharge loading transferred by the piles. The subsoil at and below the pile tips will be composed of a dense granular deposit followed, in turn, by the lower overconsolidated portion of the cohesive stratum. Settlement computations were, therefore, carried out. Based on these computations, it is estimated that the settlement will likely be of the order of 1-1/2 to 2 inches, the majority of which will be due to recompression of the cohesive subsoil.

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

7.2) Structure Foundations: (cont'd.) ...

1) Timber Piles Driven into the Dense Granular Layer -
(cont'd.) ...

The differential settlement between the existing and the extended sections would be equivalent to the magnitude quoted in the previous paragraph. Therefore, if this scheme is adopted it would be advantageous to provide a construction joint between the existing and the widened portion of the foundations in order to accommodate this amount of differential settlement.

11) End-bearing Piles Driven to Refusal with Glacial Till:

As an alternative to the timber pile scheme, the extensions could be supported on end-bearing piles driven to practical refusal within the basal granular glacial till sheet; this would eliminate differential settlement between the existing and extended section. It is estimated that the pile tips would be located at about the following elevations:

	<u>Pile Tip Elev.</u>	
Southern Portion of Structure Site	620 - 624)	Refer to
)	Drawing
Northern Portion of Structure Site	608 - 612)	70-11090A

The allowable load would be dependent on the pile section chosen - for example, 14 BF 70 steel H-piles could be designed for 95 tons/pile. The pile driving in the field should be controlled by employing the Hiley Dynamic Pile Driving Formula, as per current D.E.C. practices.

Three of the piers are located within the confines of the canal channel. A dewatering scheme will be required at these locations in order to facilitate pile driving and the construction of the pile caps in the dry. No major dewatering problems are anticipated at the remaining extension locations. Any minor

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

7.2) Structure Foundations: (cont'd.) ...

ii) End-bearing Piles Driven to Refusal with Glacial Till:
(cont'd.) ...

seepage or surface run-off occurring in these excavations could be handled by employing conventional techniques, such as pumping from sumps.

The underside of the pile caps should be provided with at least 4 feet of earth cover for frost protection purposes.

No bouldery or rock fill should be placed in areas where piles are to be driven.

7.3) Approach Embankments:

No stability problems are anticipated for the proposed widening of the approach embankments, provided that:

- a) in the transverse as well as the forward direction the slopes be no steeper than the existing embankment slopes, and
- b) measures should be taken to protect the banks against the scour action of the canal.

The clayey silt subsoil, located beneath the widened portions of the embankments will be subjected to consolidation settlement due to the induced pressure. Settlement computations were, therefore, carried out. Based on the results of these computations, it is estimated that the maximum amount of settlement could be of the order of 2-1/2 to 3 inches. It is anticipated that the majority of the aforementioned settlement will take place within a period of 18 months.

In order to have a smooth transition from the existing to the new fill sections, it is recommended that:

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

7.3) Approach Embankments: (cont'd.) ...

i) all topsoil be stripped from the existing fill sections prior to placing future fill, and

ii) the future fill be 'keyed' into the existing approaches as per current D.H.O. methods.

7.4) Proposed Retaining Walls:

The existing steel sheet pile walls, which are approximately 10 feet deep, have showed signs of distress (refer to Section 6). It is understood they are to be replaced by new sheet piling. The new sheeting should be driven to sufficient penetration into the subsoil so that the resultant passive earth pressure developed in front of the wall is adequate in relation to the resultant active earth pressure tending to displace the wall; factor of safety of 1.2 is recommended in this regard.

In computing the active components the following should be used:

a) for the surficial granular subsoil - a coefficient of active earth pressure (K_a) of 0.33.

b) for the underlying cohesive subsoil - an undrained shear strength (C_u) of 800 p.s.f., and

c) the full surcharge effect of the forward slope of the approach fills should be taken into consideration.

The passive components can be computed using a coefficient of passive earth pressure (K_p) of 3.0, for the granular subsoil. The undrained shear strength value given above can be used for the cohesive material.

7. MISCELLANEOUS:

The field work, performed during the period of October 26 to November 10, 1970, was carried out under the immediate supervision of Mr. V. Korlu, Project Foundation Engineer.

The equipment was owned and operated by Master Soil Investigation Ltd., Toronto.

This report was prepared by Mr. B. T. Darch, Senior Foundation Engineer and Mr. V. Korlu, Project Foundation Engineer.

This project was carried out under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who also reviewed this report.

December, 1970

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-11090 LOCATION Sta. 116+73 @ Hwy. 400 o/s 52.5' Rt. ORIGINATED BY VK
 W.P. 105-70-05 BORING DATE Nov. 2-4, 1970 COMPILED BY WH
 DATUM Geodetic BOREHOLE TYPE Diamond Drill-Washboring CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS						
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %					
							20	40	60	80	100						w_p	w	w_L			
												400	800	1200			1600	2000	10	20	30	
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE																
740.4	Ground Level																					
0.0	Clayey silt with trace of sand and gravel (Roadway Fill) Soft to Hard		1	SS	5																	
			2	SS	7																	
			3	TW	PM	730																
			4	TW	PM																	
			5	SS	40																	
720.0	Brown		6	SS	21	720																
20.4	Silty sand - trace of organics Compact Grey		7	SS	19																	
711.9			8	SS	8	710																
29.5	Clayey silt Occ. sandy silt layers up to 1/2" thick below el. 700. Firm to Very Stiff		9	TW	PM																	
			10	TW	PM	700																
694.9	Grey		11	TW	PM																	
45.5	Silty Sand - trace of gravel. Occ. clayey silt seams up to 1/2" thick throughout Dense to Very Dense		12	SS	41	690																
			13	SS	32																	
			14	SS	55	680																
676.4	Grey		15	SS	27																	
64.0	Clayey silt to silty clay, trace of sand Stiff to Very Stiff		16	SS	12	670																
669.9	Brown to Grey																					
70.5	End of Borehole																					

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 70-11090 LOCATION Sta. 117 + 07 & Hwy. 400 e/s 56.5' Lt. ORIGINATED BY VK
W.P. 105-70-05 BORING DATE Oct. 26, 1970 COMPILED BY MH
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w _L PLASTIC LIMIT — w _p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					WATER CONTENT %				
							20	40	60	80	100	10	20	30		
							SHEAR STRENGTH P.S.F.									
</																

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 70-11090 LOCATION Sta. 117 + 39 & Hwy. 400 o/s 56' Rt. ORIGINATED BY VK

W.P. 105-70-05 BORING DATE Nov. 3-4, 1970 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w _L PLASTIC LIMIT — w _P WATER CONTENT — w			BULK DENSITY Y P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT										
							20	40	60	80	100						
							SHEAR STRENGTH P.S.F.										
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					WATER CONTENT %					
726.8	Ground Level						400	800	1200	1600	2000						
0.0	Clayey silt, trace of sand & gravel (Roadway Fill)		1	SS	13												
721.3	stiff Brown		2	SS	6	720				+s3							
5.5	Silty sand																
717.8	Organic Material Black		3	TW	18												
10.5	Loose Grey & Brown		4	TW	18												
713.3																	
13.5	Clayey silt		5	TW	18	710											
	Trace of organics around Elev. 700.		6	TW	18												
	Firm to Very Stiff		7	TW	18	700											
			8	TW	18												
691.3	Grey		9	SS	40	690											
35.5	Silty sand		10	SS	36												
	Occ. clayey silt seams up to 1/2" thick throughout		11	SS	30	680											
679.8	Dense Grey		12	SS	28												
47.0	Clayey silt - trace of sand		13	SS	25	670											
	Stiff to Hard		14	SS	12												
			15	SS	39	660											
						650											
	Grey		16	SS	26	640											
						630											
						620											
615.8																	
111.0	Net. mix. of silt, sand & gravel, trace of clay (Glacial Till)		17	SS	88	610											
610.3	Very Dense Grey																
116.5	End of Borehole																

DEPARTMENT OF HIGHWAYS- ONTARIO
 MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 70-11090

LOCATION

Sta. 118 + 20 @ Hwy. 400 o/s 52.5' Lt.

ORIGINATED BY

VK

W.P. 105-70-05

BORING DATE

Nov. 12-13, 1970

COMPILED BY

WE

DATUM Geodetic

BOREHOLE TYPE

Diamond Drilling-Washboring

CHECKED BY

V 738.6

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.					WATER CONTENT %			
724.6	Ground Level							20	40	60	80	100			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE							
								400	800	1200	1600	2000	10	20	30
0.0	Clayey silt, trace of sand & gravel (Roadway Fill)		1	TW	PM	720									
716.6	Firm to Stiff		2	TW	PM										
8.0	Silty sand		3	SS	12										
712.6	Compact		4	TW	PM										
12.0	Clayey silt trace of organics		5	TW	PM	710									
			6	TW	PM										
	Occ. sandy silt seams up to 2" thick below el. 695		7	TW	PM	700									
	Firm to Stiff		8	TW	PM										
688.6	Silty sand		9	TW	PM	690									
36.0	Very Dense		10	SS	157										
680.6	Clayey silt Trace of sand		11	SS	27	680									
44.0	Stiff to Very Stiff		12	TW	PM	670									
			13	TW	PM	660									
			14	TW	PM	650									
642.1	Boulder 18" in size		15	BX	100%	640									
84.0	Grey		16	SS	144										
624.6			17	SS	166	630									
623.1	Glacial Till														
101.5	End of Borehole														

 20
 10-5 % STRAIN AT FAILURE
 10

FOUNDATION SECTION

W

2000

20
15- ϕ -5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 70-11090

LOCATION Sta. 118 + 98 @ Hwy. 400 o/s 51' Lt.

ORIGINATED BY VK

W.P. 105-70-05

BORING DATE Nov. 9-10, 1970

COMPILED BY WH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill-Washboring

CHECKED BY HK

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.					WATER CONTENT %					
747.1	Ground Level							20 40 60 80 100					w_p — w — w_L				
								400 800 1200 1600 2000					10 20 30				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE									
0.0	Clayey silt - trace of sand & gravel (Roadway Fill)		1	SS	5												
	Firm to Very Stiff		2	TW	PM	740		x 92				2000		○	126	738.1	
			3	SS	2							2000		○	126	0 9 75 16	
			4	TW	PM							q		○			
			5	SS	16	730											
			6	SS	10												
722.1	Brown		7	SS	11	720											
25.0	Clayey silt		8	TW	PM												
	Traces of organics		9	TW	PM	710		p					A	○	126		
	Occ. thin ($\frac{1}{8}$ ") silt seams below 702.		10	TW	PM			f _{sc} 4					B	○	129		
	Firm to Stiff		11	TW	PM	700							○		130	0 3 79 18 0.8% Org.	
			12	TW	PM												
695.1	Grey		13	SS	104	690											
52.0	Silty sand, occ. clayey		14	SS	74												
	silt seams up to $\frac{1}{2}$ " thick throughout		15	SS	43												
681.6	Clayey silt Grey																
65.5	End of Borehole					680											

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 7U-11090

LOCATION Sta. 117 + 33 @ Hwy. 400

ORIGINATED BY VK

W.P. 105-70-05

BORING DATE Oct. 29, 1970

COMPILED BY WH

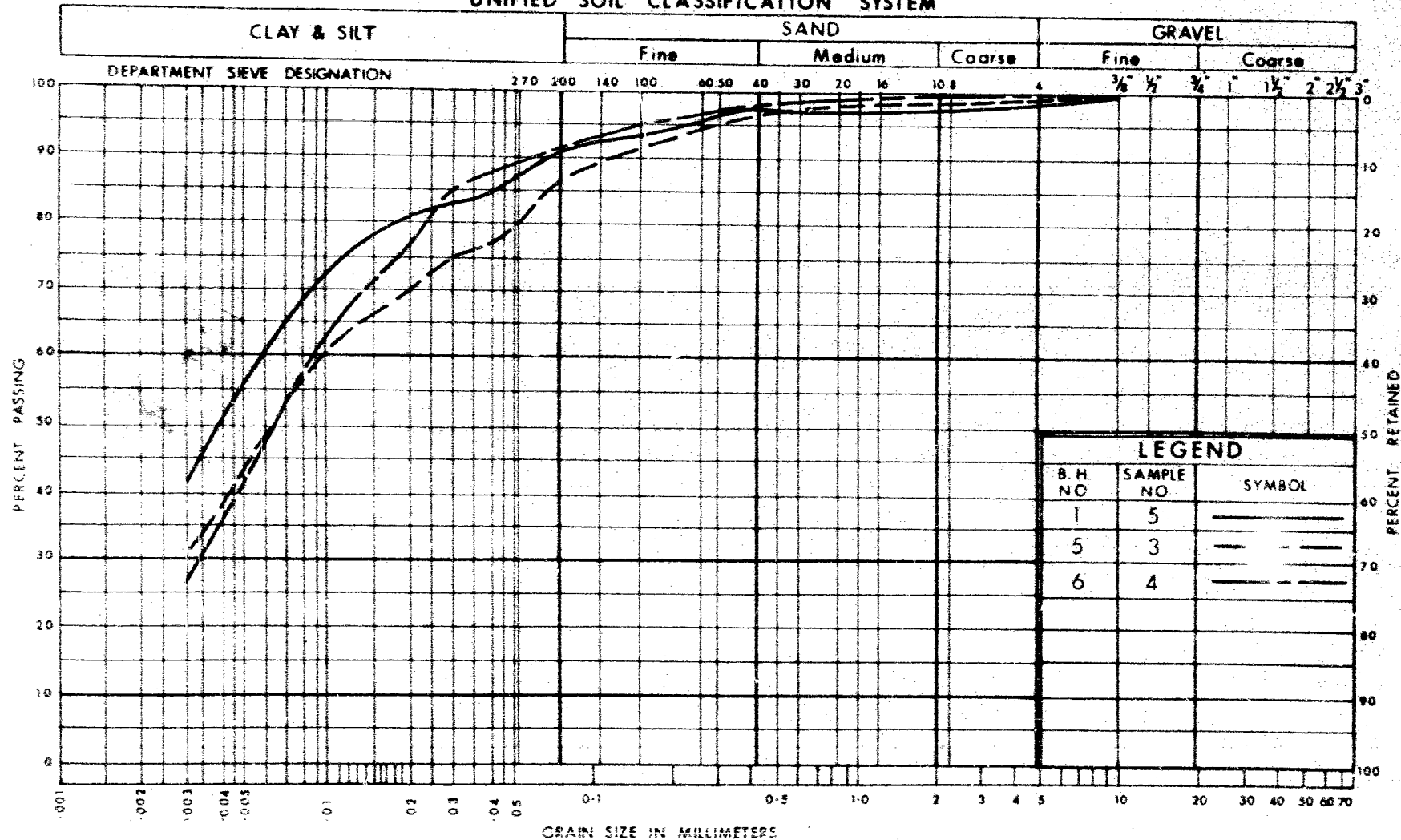
DATUM Geodetic

BOREHOLE TYPE Cont. Flight Auger

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w _L PLASTIC LIMIT ——— w _p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.					WATER CONTENT %				
							20	40	60	80	100					
							UNCONFINED + FIELD VANE QUICK TRIAXIAL x LAB. VANE					w _p ——— w _L				
							400	800	1200	1600	2000	10 20 30				
726.9	Ground Level															
0.0	Clayey silt, trace of sand & gravel (Roadway Fill)		1	SS	23											
720.9	Very Stiff Brown		2	SS	9	720										1.0% Org.
6.0	Silty sand, trace of organics.		3	TW	PH											716.9
713.9	Loose Brown & Grey		4	TW	PH											0 10 82
13.0	Clayey silt trace of organics		5	TW	PH	710										0 31 65
			6	TW	PH											
	Occ. thin (1/4") silty sand seams below 697.		7	TW	PH	700										
	Firm to Very Stiff		8	TW	PH											0 3 75 22
688.9	Grey		9	TW	PH	690										
38.0	Silty sand		10	SS	21											
681.4	Compact Grey		11	SS	26	680										0 12 58 30
45.5	Clayey silt Trace of sand		12	SS	32											
			13	SS	31	670										0 6 43 51
	Very Stiff to Hard		14	SS	31	660										
			15	SS	36	650										
	Grey		16	SS	31	640										
						630										
						620										
614.9	Het. mix. of clayey silt, sand & gravel (Glacial Till)		17	SS	178	610										4 47 40 9
598.6	Hard		18	SS	160/3"	600										
128.3	End of Borehole					590										

UNIFIED SOIL CLASSIFICATION SYSTEM



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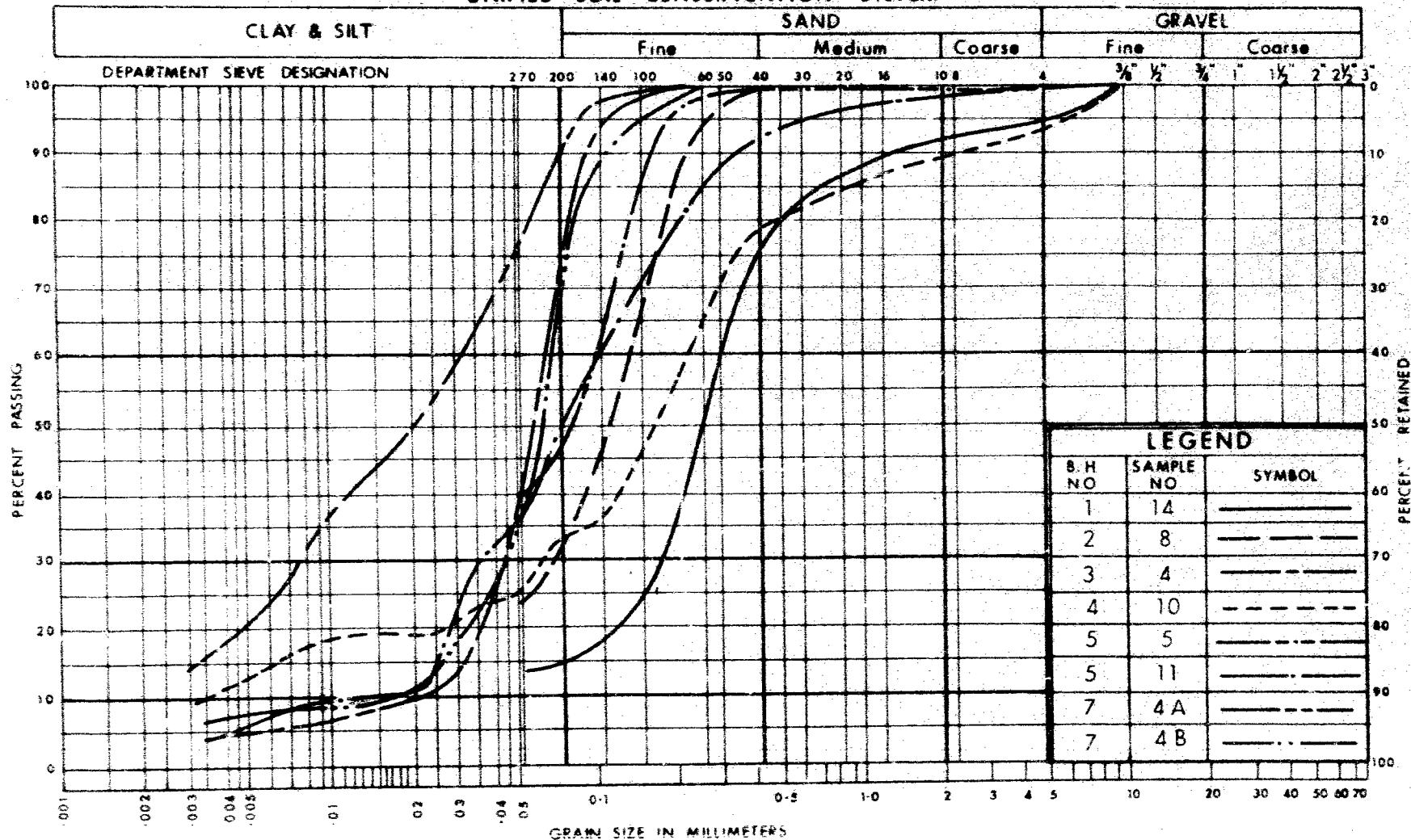
GRAIN SIZE DISTRIBUTION
CLAYEY SILT (FILL)
TRACE OF SAND & GRAVEL

W.P. No. 105-70-05

JOB No. 70-11090

FIG. 1

UNIFIED SOIL CLASSIFICATION SYSTEM



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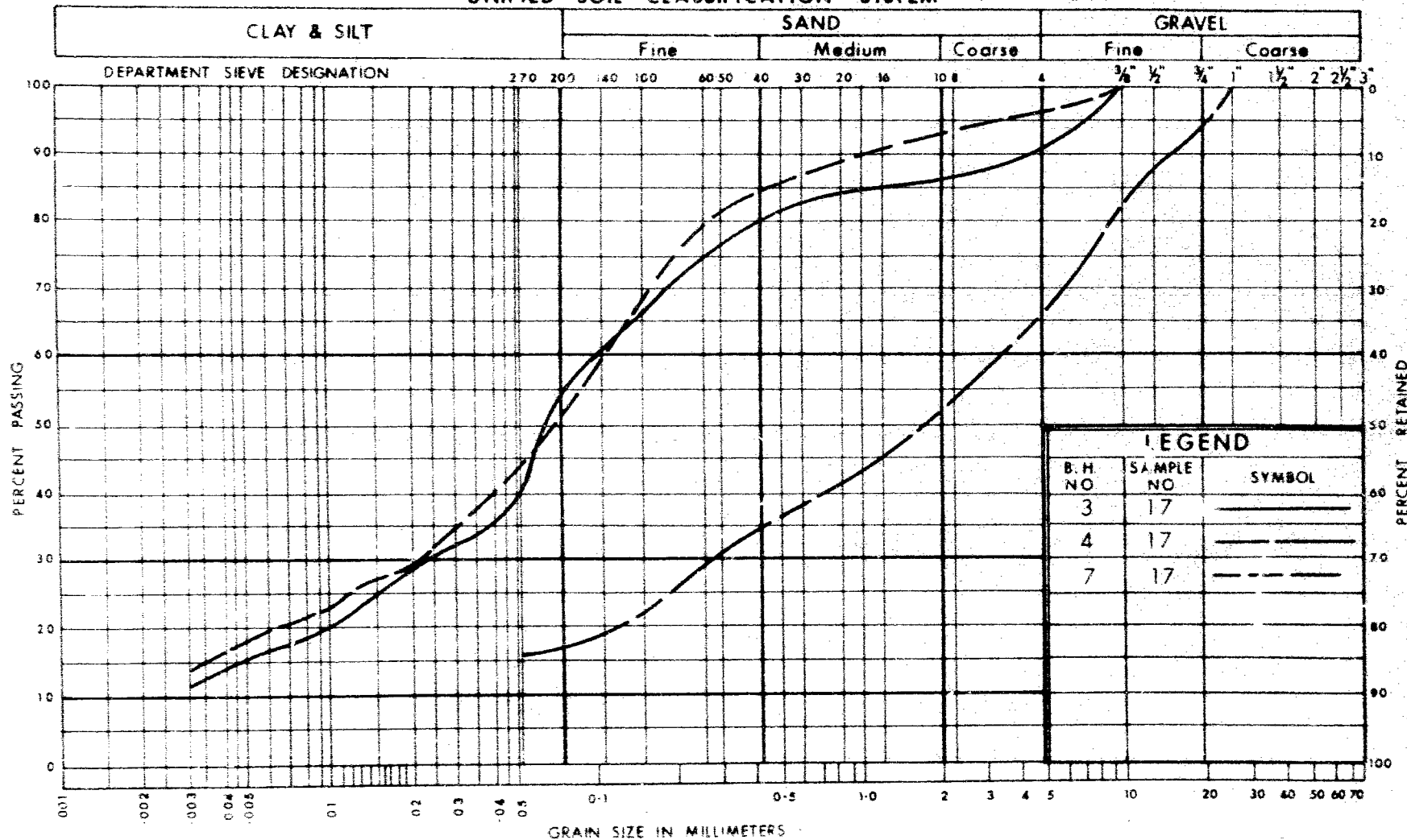
GRAIN SIZE DISTRIBUTION SILTY SAND TO SANDY SILT

W.P. No. 105-70-05

JOB No. 70-11090

FIG. 3

UNIFIED SOIL CLASSIFICATION SYSTEM



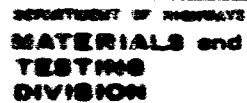
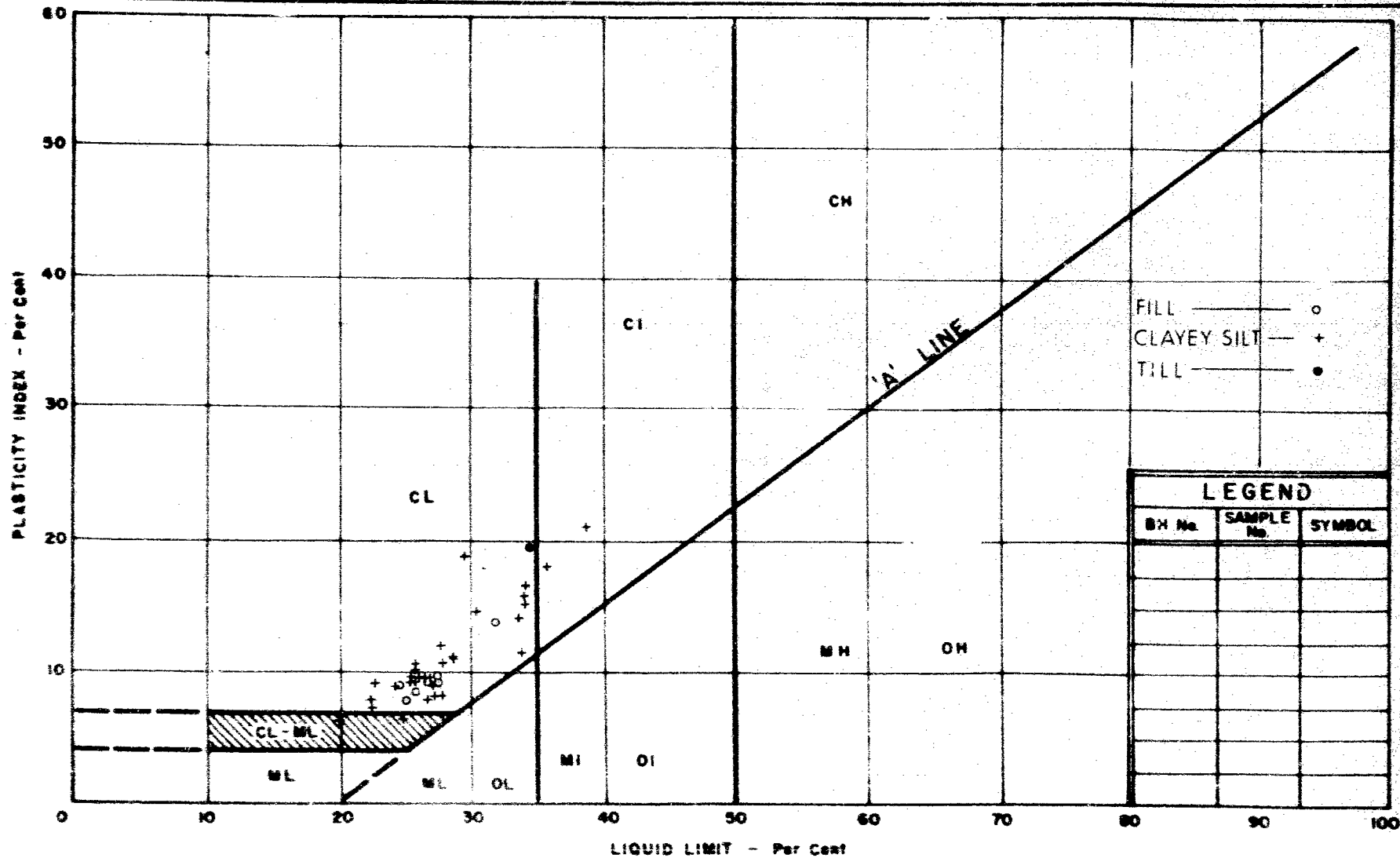
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DIVISION

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
HET MIXTURE OF SILT, SAND & GRAVEL, SOME CLAY

W.P. No. 105-70-05

JOB No. 70-11090

FIG. 4



PLASTICITY CHART

W.F. No. 105 - 70 - 05

JOB NO. 70-11090

FIG. 5

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

FROM: C.S. Grebski,
Bridge Office

ATTENTION:

DATE: January 12, 1971

OUR FILE REF.

IN REPLY TO

SUBJECT: North Canal Twin Bridge Widening
W.P. 105-70-05, Site No. 30-334
Highway 400, District No. 5

70-11090

Attached herewith we are submitting the final
bridge drawings which show the foundation design for
this structure.

Kindly give us your comments at your earliest
convenience.

C.S. Grebski
C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.s. Foundation Office

see comments
BTD
Jan 25/71
sf
Feb 11

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac

~~Mr. G. Burkhardt.~~

Reg. Bridge Planning Engineer,
Central Region

C.S. Grebski,
Bridge Office

December 21, 1970

North Canal Twin Bridge Widening
W.P. 105-70-05, Site No. 30-334
Highway 400, District No. 6

70-11680

Attached herewith are prints of the Preliminary Bridge Plan
Drawing D-6906-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$160,000.
This cost includes tender, materials, engineering, and sundry con-
struction.

Any comments or revisions you may have should be submitted
within three weeks.

C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

C.C. B. Davis

1. Stermac (2)

2. Anderson

the comments
p. 7 D. see 28/10

A48

MEMORANDUM

Telephone: 248-3097

Mr. A.G. Stermac,
Pain. Foundation Engineer,
Room 107,
Lab. Building.

FROM: G.C.E. Burkhardt,
Bridge Office,
Central Region.

ATTENTION:

DATE: October 6th, 1970.

OUR FILE REF.

IN REPLY TO

SUBJECT: King Township Overpass, *no problem*
 70-11088 Site 37-59, W.P. 105-70-10, *Site 52*
 70-11089 South Canal Road Bridge, *Bad soil conditions*
 70-11090 Site 37-34, W.P. 105-70-04, *Disturbance 62*
 70-11091 North Canal Road Bridge, *Disturbance 62*
 Site 30-334, W.P. 105-70-05, *no problem*
 Overpass at Hwy. 27, *no problem*
 Site 30-178, W.P. 105-70-09,
Districts 5 and 6, Hwy. 400.

The centre gaps of the above noted twin structures (Site 37-34, 30-334 and 30-178) are being joined to satisfy roadway requirements for the design concept of widening the existing Hwy. 400 from 4 lanes to 6 lanes. In all four structures, outside widening may be required on both sides of the structure for an approximate distance of 10 feet.

We would appreciate having your opinion and direction on driving piles, assessing possible differential settlement and any other related conditions which could arise from the proposed work. The boring data and existing bridge drawings are enclosed for your information with the understanding that any further borings will be done at your discretion.

As this information is required as early as possible to comply with the tight schedule of the project we would appreciate any priority you may be able to extend to it.

JSR/ed
 Encl.
 cc G. Grebski
 (Attn: W. Lin)

[Signature]
 S.S. Robertson,
 BRIDGE PLANNING SUPERVISOR,
 for:
 G.C.E. Burkhardt,
 REGIONAL BRIDGE PLANNING ENGINEER.



**PRELIMINARY (DESIGN-BUILD READY)
FOUNDATION DESIGN REPORT
for
NORTH CANAL BRIDGE NORTHBOUND REPLACEMENT
HIGHWAY 400, SITE No. 30-334/1
GWP 2005-11-00
TOWNSHIP OF WEST GWILLIMBURY, SIMCOE COUNTY, ONTARIO**

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

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GEOCRES No.: 31D-604
June 8, 2015



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Table 1 – Summary of Advantages, Disadvantages and Recommended Foundations

Table 2 – List of Standard Specifications Referenced in Report

Table 3 – Gradation Specification for Sand Fill in Pre-Augered Holes at Integral Abutments

Figure 1 – Abutment on Compacted Fill Showing Granular A Core

**PRELIMINARY (DESIGN-BUILD READY)
FOUNDATION DESIGN REPORT**

for

North Canal Bridge Northbound Replacement

Highway 400, Site No. 30-334/1

GWP 2005-11-00

Township of West Gwillimbury, Simcoe County, Ontario

1. INTRODUCTION

This report provides preliminary foundation engineering comments and recommendations for Design-Build purposes regarding design and construction of the foundations and approach embankments for a replacement of the existing northbound bridge over the North Canal and Canal Road located on Highway 400 in the Township of West Gwillimbury, County of Simcoe, Ontario. The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

Highway 400 passes over the North Canal and Canal Road at approximate Station 10+561, Highway 400 chainage. The replacement bridge is proposed to be a two span structure with a total length of 84.0 m and width of 17.8 m (ref. Drawing P-1 'Canal Road Bridge Replacement. General Arrangement' prepared by MMM Group Limited in May 2013).

The road grade on Highway 400 at the bridge location is planned to be at elevation 227.3 at the south abutment, elevation 228.4 at the pier and elevation 229.4 at the north abutment. The approach embankments to the structure are envisaged to be about 7 m high at the abutments (interpolated from ground surface elevations and the road grade on Highway 400). The intended construction location of the new south and north abutments is some 10 and 25 m north of the corresponding abutments of the existing structure, with the new pier situated between existing piers 4 and 5. The North Canal will be realigned to the north of the new pier. Canal Road running under the existing bridge is planned to be relocated 5 m north of its current alignment. The approach embankments will be widened on the east side to accommodate the proposed shift of the centreline of the replacement structure by about 7 m.

The replacement of the bridge will require staged construction and use of roadway protection systems to maintain traffic over Highway 400. Refer to section 6 of this report for a detailed discussion.



In summary, the subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised embankment fill over silty/sandy soils overlying clayey silt interlayered with sand / silty sand and underlain by a compact to very dense / hard glacial till. Cobbles were encountered in borehole 102. The groundwater was at elevation 218.5 to 222.6, with artesian conditions reported in borehole 5. The water level in the North Canal was at approximate elevation 218.8 at the time of the investigation in 2014.

2. FOUNDATIONS

2.1 General Considerations

Based on the available information, design and construction of foundations to support the replacement bridge is considered feasible at the site.

A summary of advantages, disadvantages of shallow and deep foundation options and the recommended foundation type is provided in the attached Table 1.

Cognisant of the relatively low bearing resistance of the native soil and the presence of cohesionless soils below the canal / groundwater level and possible cobbles and boulders at depth, it is not considered feasible to employ either spread footings on native soils or caissons to support the proposed structure foundations. In addition, the difficulties associated with construction of the caissons in artesian conditions and potential consequences such as necking do not warrant the use of caissons at this site. Construction of spread footings on engineered fill would also require excavation below the water table and necessitate extensive groundwater control measures.

Use of end-bearing piles driven into the hard / very dense glacial till is considered to be the preferred foundation system from a foundation engineering perspective. Further, construction of integral abutments supported on steel H-piles is considered to be feasible. It is noteworthy that installation of piles may encounter some difficulty due to the presence of cobbles / boulders. Considering that the existing bridge is supported on driven timber piles (steel H-piles under the portion widened in 1971), pre-augered holes may be required to reduce impact of pile driving on the existing structure.



Since the Highway 400 road grade may be raised slightly (about 0.5 m), the existing approach embankments will not settle significantly. For the widened section of the embankment, however, the 7 m high fill will induce consolidation of the clayey silt and, as a consequence, negative skin friction on new piles should be considered.

The upper zones of the subsoil near the structure as well as along the canal bed are made up of silty/sandy soils that are considered to be susceptible to potential erosion and scour. The detail design and temporary works should provide for adequate measures to protect the foundations of the new structure from erosion and scour.

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

The foundation frost penetration depth at this site is 1.5 m according to OPSD 3090.101. The seismic site coefficient for the conditions at the site is 1.0 – Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) 2006 Edition – for the anticipated foundation conditions. The zonal acceleration ratio is 0.05. The bridge site is located in Seismic Performance Zone 1.

Further comments and recommendations for design of the foundations are provided in the following sections. The standard specifications referenced in the report are listed in Table 2.

2.2 Shallow Foundations

Taking into account the presence of extensive weak deposits across the site, it is not considered feasible to employ conventional spread footings placed on the native soils to support the proposed structure foundations. Spread footings constructed on engineered fill would require removal of the silty/sandy soils with organics to some 1.5 m below the canal level (elevation 217.0 to 217.5) and necessitate appropriate groundwater control measures including cofferdams which may not be



economically viable. In addition, carefully designed erosion and scour control such as using interlocking steel sheet piles would be required to protect the engineered fill.

If employed, structural fill should comprise OPSS.PROV. 1010 Granular A material or Granular B Type II below the water table placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D-689 (standard Proctor) maximum dry density, and extend laterally to a line inclined downwards at 45° to the horizontal originating at least 1 m from the top of the footing. This scheme is illustrated in Figure 1, appended.

The recommended bearing resistance at ultimate and serviceability limit states (ULS and SLS) for a minimum 2.5 m wide footing constructed on a structural fill pad at least 4 m thick is as follows:

$$\begin{aligned}\text{Factored Bearing Resistance at ULS} &= 900 \text{ kPa} \\ \text{Bearing Reaction at SLS} &= 350 \text{ kPa}\end{aligned}$$

The reaction at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.5 m was assumed for computation of the ULS resistance.

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

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

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding surface.

2.3 Piles

A foundation system consisting of steel H-piles driven to refusal into the hard / very dense glacial till is recommended. Taking account of the anticipated foundation loads and depth to a competent



bearing stratum, construction of integral abutments supported on end-bearing piles is considered to be feasible at this site.

It is anticipated that driven piles will encounter practical refusal in the hard / very dense glacial till at or below elevation 184 at the south abutment, elevation 188 at the pier and elevation 195 at the north abutment. A pile penetration of 1 to 2 m into the bearing stratum was assumed for adequate refusal. The founding levels for the H-piles should be confirmed during detail design. The HP 310x110 piles should be designed using the following geotechnical axial resistances:

$$\begin{aligned}\text{Factored Geotechnical Axial Resistance at ULS} &= 1600 \text{ kN} \\ \text{Geotechnical Axial Reaction at SLS} &= 1400 \text{ kN}\end{aligned}$$

The geotechnical reaction at SLS allows for 25 mm compression of the founding medium. Considering the estimated levels for the underside of pile caps and the anticipated founding levels, the pile length is expected to vary between 28 and 36 m.

The piles will have to be driven through the native soils containing relatively compressible clayey soils. Taking into account that the abutment locations have been preloaded by the approach embankments for a considerable period of time, settlements under the existing fill are assessed to be negligible. Under the widened section of the embankment, however, the anticipated consolidation settlements will bring about the development of negative skin friction. A downdrag load due to negative skin friction is estimated to be 150 kN per pile. To minimise the effect of negative skin friction, fill for the embankment widening should be placed and preloaded for 2 years prior to pile installation. To reduce the waiting time, consideration could be given to surcharging the new fill and/or using lightweight fill. This should be further analysed and discussed during detail design, including the details of surcharging such as its height, the period of surcharge application and instrumentation to be used for settlement monitoring.

The approach embankment fill within the limits of the pile foundation including fill placed below grade to replace any excavated unsuitable/compressible soils should include Granular A or Granular B Type II with a maximum nominal size of 75 mm to enable driving of the piles and



minimise the potential for damage during pile installation. Granular B Type II is recommended below the water table if required.

The native soil adjacent to the upper portion of the abutment piles is expected to comprise the soft to stiff clayey silt or loose to compact silty/sandy soils or embankment fill materials. To accommodate movement of the integral abutment, it is recommended that two concentric CSPs that extend at least 3 m below the bottom of the abutment be placed around the pile to create an annular space. The inner CSP of 600 mm diameter should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements given in the attached Table 3 may be used. Refer to MTO Report SO-96-01 for further details.

The piles will be founded into the hard / very dense glacial till with cobbles / boulders and should be driven in accordance with SS103-11. This should be confirmed by dynamic analysis (such as the Hiley formula) in the process of pile installation.

All piles should be equipped with driving shoes as per OPSD 3000.100. The piles should be installed and monitored in accordance with the requirements of OPSS 903.

Pile caps should be provided with at least 1.5 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. For preliminary design, the lateral resistance for the HP 310x110 pile section is tabulated below. The horizontal displacement associated with the lateral reaction at SLS should be verified during detail design.

PARAMETERS	LOOSE TO COMPACT SILT / SAND	FIRM TO STIFF CLAYEY SILT	GRANULAR FILL
Factored Lateral Resistance at ULS, kN	130	140	150
Lateral Reaction at SLS, kN	40	50	50



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

The coefficient of horizontal subgrade reaction, k_s (kN/m^3), should be computed using the following equations to evaluate the point of contraflexure:

Cohesionless:

$$\begin{aligned}k_s &= n_h z/b \\n_h &= \text{coefficient related to soil density} \\&= 12 \text{ MN/m}^3 \text{ for granular fill} \\&= 1 \text{ MN/m}^3 \text{ for native silty/sandy soils} \\z &= \text{depth, m} \\b &= \text{pile width, m}\end{aligned}$$

Cohesive:

$$\begin{aligned}k_s &= \frac{67c_u}{b} \\c_u &= \text{undrained shear strength of cohesive material} \\&= 50 \text{ kPa for firm to stiff clayey silt} \\b &= \text{pile width, m}\end{aligned}$$

It is noteworthy that artesian water was encountered at 28.0 m depth (elevation 195.2) in borehole 5 located near the pier of the replacement structure, with a head of 0.2 m above the previous ground surface (elevation 223.4). Since H-Piles are anticipated to be driven to approximate elevation 188 at the pier, the soil strata under artesian pressure are likely to be penetrated resulting in flowing of water along the pile-soil interface. It is recommended, therefore, that a filter layer consisting of clean stone wrapped in a non-woven geotextile and designed to retain soil particles be placed under the pile cap in order to collect rising water. If considered to be required, the design of this filter mat should be carried out during detail design.



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:

$$p = K(\gamma h + q) + C_p + C_s$$

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
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
 where ϕ = angle of internal friction of retained soil (35° for Granular B Type II)
 δ = angle of friction between the soil and wall (23.5° for Granular B Type II)

The seismic site coefficient and zonal acceleration ratio for the conditions at this site were provided in section 2.1.

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

PARAMETERS	GRANULAR A	GRANULAR B TYPE II
Angle of Internal Friction, degrees	35	35
Unit Weight, kN/m^3	22.8	22.8
Coefficient of Active Earth Pressure K_a	0.27	0.27
Coefficient of Earth Pressure At-Rest K_o	0.43	0.43
Coefficient of Passive Earth Pressure K_p	3.69	3.69

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained



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

A weeping tile system (OPSS 405 and OPSD 3190.100) should be installed to minimise 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 to Ontario Provincial Standards specifications 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.PROV. 501 for additional information in this regard.

4. APPROACH EMBANKMENTS

The existing approach embankments at the abutments are presently some 4.5 and 7.0 m high at the south and north approaches and have side slopes inclined at about 2.5H:1V. It is likely that the grade raise, if any, over the existing approach embankments will be constructed with granular material.

4.1 Embankment Stability

It is considered that the approach embankments constructed in accordance with the following recommendations will be stable.

The topsoil or other deleterious soils encountered during construction at the abutment locations and along the shoulders and toes of the alignment of the approach fills at least within 20 m of the abutments should be stripped prior to placement of the embankment fill.



Backfilling adjacent to the structure abutments should be carried out in conformance to Ontario Provincial Standards specifications for granular backfill at abutments (OPSD 3101.150).

The embankment widenings and grade raises should be constructed in accordance with OPSD 201.010, 202.010, 203.030, 208.010 and OPSS.PROV. 206. For preliminary design purposes, the side slopes of the widened embankments as well as the foreslopes at both abutments should be inclined no steeper than 2.5H:1V for earth fill. The foreslopes could be inclined at 2H:1V if constructed using granular material. This recommendation should be reviewed during detail design once the configuration of the structure and embankments has been established. A 2 m wide mid-height berm should be provided for earth or granular material embankments higher than 8 m, in accordance with OPSD 202.010.

4.2 Embankment Settlements

In case of a grade raise, some settlement of the road surface over the existing approach embankments should be expected as a result of two mechanisms – consolidation of the native soil below the recently placed fill and self-compaction of the new fill.

Settlement of the existing embankment fill away from the abutments due to consolidation of the subgrade soil at both embankments is computed to be less than 25 mm and completed within 3 to 4 months following fill placement. Settlement of the material required for the grade raise is considered to be negligible.

The granular backfill placed adjacent to the south abutment will be about 5 m thick. The magnitude of self-compaction of this fill will be dependent on the workmanship employed by the contractor and, if placed in 200 to 300 mm thick lifts compacted to 100% of the standard Proctor maximum dry density in accordance with the requirements of OPSS 902 and OPSS.PROV. 501 (Method A), should be some 15 mm.

Consequently, the total estimated settlement of the existing approach fill near the abutments should be up to 40 mm. The total settlement will be essentially complete within 3 to 4 months after fill placement.



For the embankment widening sections, the fill will induce consolidation settlements of the cohesive deposits present at the site. It is assessed that settlements in the order of 100 mm will occur, with 90% consolidation taking place during a period of about 2 years. The estimated 15 to 20 mm settlement of the fill itself should be complete during construction. To eliminate negative skin friction on driven steel piles and reduce the longitudinal and transverse differential settlement across the embankment, it is recommended to preload the new fill for 24 months. This mitigation measure should be reconsidered for detail design including surcharging and lightweight fill alternatives.

Earth fill slopes where employed should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 803 or OPSS.PROV. 804 for time constraints and the type of seed and mulch required. Erosion protection against scour action from the canal, especially the toe of the north embankment foreslope that will be submerged in the relocated canal alignment should also be provided using rip-rap in accordance with OPSS 511.

5. EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of the abutments is expected to extend through the fill and native soils to depths of 2 to 6 m below existing grade. Excavation of these soils should be relatively straightforward.

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

The fill and typically firm to stiff / loose to compact soils are classified as Type 3 soils according to OHSA criteria. Temporary cut slopes in earth over the full depth of excavation should therefore be inclined at an angle of 45° to the horizontal. The need to excavate flatter sideslopes if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should be considered.

The groundwater was at elevation 218.5 to 222.6, with artesian conditions reported in the previously drilled borehole 5 put down near the pier of the replacement bridge. At the abutment



locations, it is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the foundation excavations for driven pile installations. If the excavation at the pier extends more than 0.6 m below the water level, a dewatering scheme will be needed to control the water flow and allow for pile installation and pile cap construction in the dry. A steel sheeting cofferdam may be used in such a case with the base sealed using a layer of tremie concrete.

6. CONSTRUCTION STAGING AND ROADWAY PROTECTION

The current plans call for the centreline of the proposed three-lane northbound bridge to be some 7 m to the east of the existing centreline. After the eastern portion of the new NBL structure is constructed, traffic will be diverted to it and the existing structure will be demolished. The remaining part of the replacement NBL bridge will then be constructed.

It is anticipated that a suitable roadway protection system following OPSS.PROV 539 will be necessary behind the abutments to support the walls of excavation and adjacent traffic lanes during staged construction.

Several alternative protection systems such as sheet piling, sheeting supported by rakers or bracing, cantilever or anchored soldier piles and lagging may be considered. It is noted, however, that a soldier pile and lagging scheme is not considered adequate where the excavation will extend through sandy soils, particularly below the water table due to potentially excessive loss of the retained soils during installation. A road protection system designed for performance level 2 in accordance with OPSS.PROV 539 is recommended to prevent excessive movement of the existing embankment. The contractor is responsible for the selection and preparation of a detailed design and performance of the road protection system.

Staged construction will involve installation of H-piles some 5 to 25 m from the foundation elements of the existing bridge that is supported on driven timber piles and steel H-piles. To be confirmed during detail design, mitigation measures such as pre-augered holes may be required to reduce impact of pile driving on the existing structure. It is also recommended that monitoring of the existing bridge be implemented.



7. CLOSURE

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

Yours very truly,

Peto MacCallum Ltd.



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TABLE 1
SUMMARY OF ADVANTAGES, DISADVANTAGES AND RECOMMENDED FOUNDATION TYPE

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES	RECOMMENDED FOUNDATION TYPE
Spread footings on native soil	Ease of construction relative to piles / caissons	Low bearing resistance necessitates large footings Large differential settlements anticipated Dewatering required due to high groundwater level	Lower cost than for piles / caissons	Large footings may not be feasible Post-construction differential settlements	Not recommended
Spread footings on engineered fill pad	Ease of construction relative to piles / caissons	Low bearing resistance relative to piles / caissons Large differential settlements anticipated under widening portions of bridge Dewatering required due to high groundwater level	Similar cost as for piles due to extensive dewatering required	Extensive groundwater control measures needed for excavation below water level Post-construction settlements	Not recommended
Driven piles	Higher capacity than for footings Allows construction of integral abutments	Difficulty of installation due to presence of cobbles / boulders	Higher cost than for footings	Special care required during pile installation near existing bridge	Preferred foundation type
Caissons	Higher capacity than for footings	Need to advance through cohesionless soils and cobbles / boulders below groundwater level Will interfere with existing piles Need to employ tremie techniques to place concrete	Higher cost relative to other alternatives	Installation likely not feasible due to groundwater conditions and loose silty/sandy soils layers	Not recommended



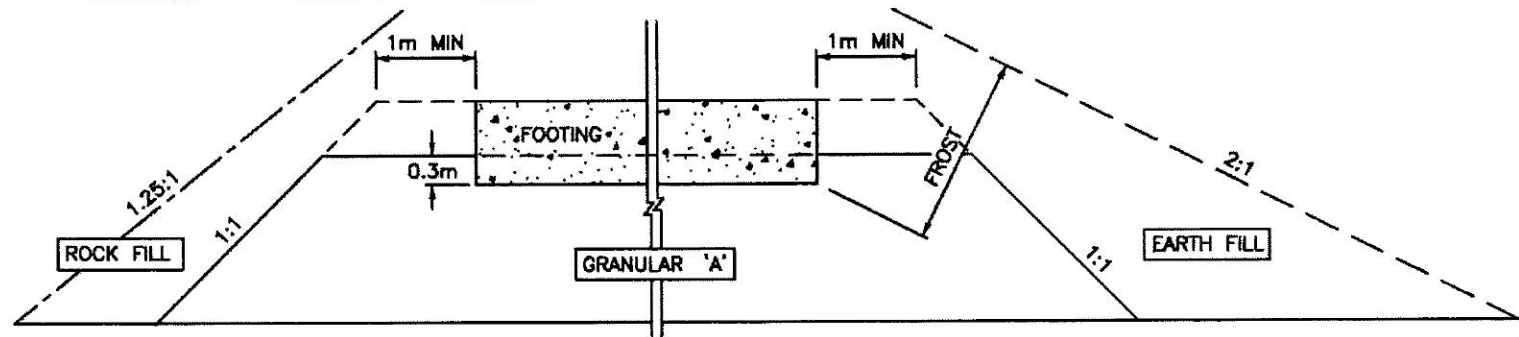
TABLE 2
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS.PROV. 206	Construction Specification for Grading
OPSS 405	Construction Specification for Pipe Subdrains
OPSS.PROV. 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV. 539	Construction Specification For Temporary Protection Systems
OPSS 803	Construction specification for Sodding
OPSS.PROV. 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification For Excavating and Backfilling –Structures
OPSS 903	Construction Specification For Deep Foundations
OPSS.PROV. 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSD 201.010	Rock Grading – Undivided Rural
OPSD 202.010	Slope Flattening Using surplus Excavated Material on Earth or Rock Embankment
OPSD 203.030	Embankments Over Swamp – Existing Slopes Maintained
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls Retaining and Abutment Wall Drain



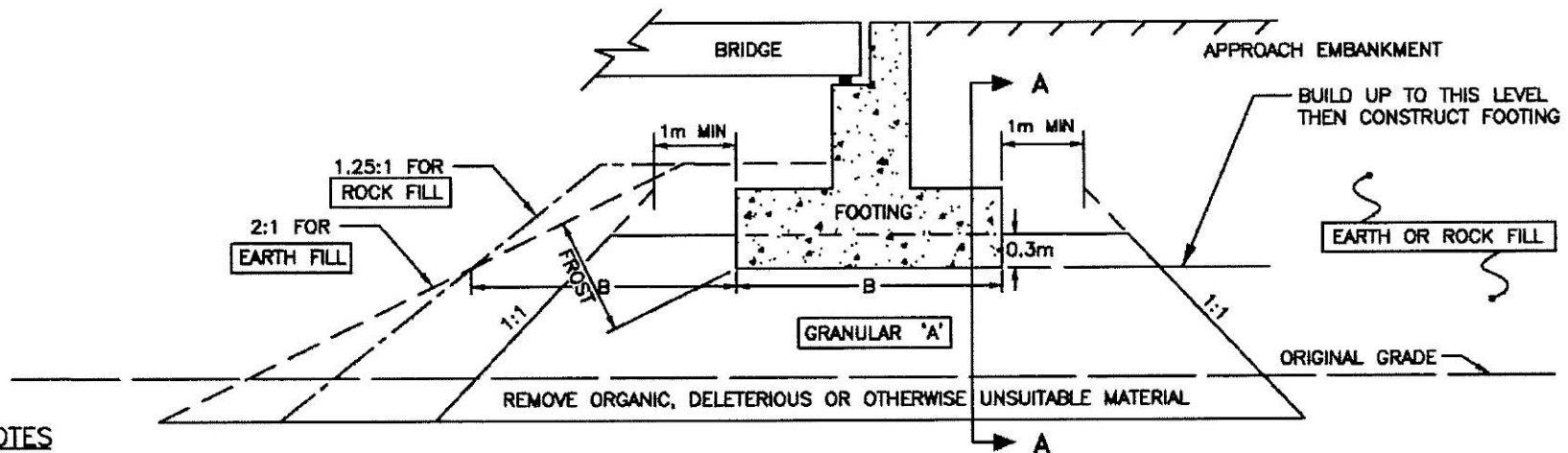
TABLE 3
GRADATION SPECIFICATION FOR SAND FILL
IN PRE-AUGERED HOLES
AT INTEGRAL ABUTMENTS

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 µm	#30	80 – 100
425 µm	#40	40 – 80
250 µm	#60	5 – 25
150 µm	#100	0 – 6



CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

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

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE