



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
REPLACEMENT OF SHALLOW LAKE BRIDGE
OVER STONEY CREEK, HIGHWAY 6
SITE NO. 8-9
SHALLOW LAKE, ONTARIO
G.W.P. 43-00-00**

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PML Ref.: 11KF065A
Index No. 093FIR and 094FDR
GEOCRES No. 41A - 223
October 15, 2012



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FOUNDATION INVESTIGATION REPORT

For
Replacement of Shallow Lake Bridge over Stoney Creek
Site No. 8-9
Highway 6, G.W.P. 43-00-00
Shallow Lake, Ontario

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed replacement of Shallow Lake Bridge on Highway 6 in Shallow Lake. The investigation was conducted for McCormick Rankin (MRC), a member of MMM Group Ltd. on behalf of the Ministry of Transportation of Ontario (MTO).

The existing bridge over the Stoney Creek includes an approximately 10.0 m long, single span concrete structure. The bridge deck is at approximate elevation 223.8. The approach embankments are level with the bridge deck.

This report provides subsurface information pertaining to the foundation of the proposed new bridge and approach embankments within about 20 m of the abutments.

All elevations in this report are expressed in meters.

2. SITE DESCRIPTION AND GEOLOGY

The site is located on Highway 6 (Princess Street), about 170 m west of the intersection of Highway 170 and Highway 6 in Shallow Lake. The topography in the general area of the bridge is relatively level. The surrounding vegetation consists of landscaped areas and scattered mature trees. The surrounding area has commercial/residential land uses. Site photographs are shown in the Appendix A.

Physiographically, the site is located in the region referred to as the Bruce Peninsula. This region has a thin overburden scattered over grey dolomite bedrock.



3. INVESTIGATION PROCEDURES

The field work for the bridges was carried out on January 4, 5 and 10, 2012. The subsurface investigation comprised six (6) boreholes S1 to S6, that were advanced through the soil cover to bedrock depths of 2.6 to 4.1 m, elevations 219.6 to 221.1, at the locations shown on the appended Drawing SL-1. Boreholes S3 and S4 were cored 3.8 and 4.1 m into the bedrock to depths of 6.7 and 6.6 m, elevations 217.0 and 217.3, respectively.

Borehole locations were laid out in the field by PML by measuring distances from the existing bridge abutments and surveyed by MRC.

The boreholes were advanced using continuous flight hollow augers powered by a truck-mounted CME-45, equipped for rotary core (NQ size) drilling, supplied and operated by a specialist drilling contractor. The drilling crews worked under the full-time supervision of a member of our engineering staff. The rock core photographs are shown in Appendix B.

In order to determine foundation support conditions for the existing bridge abutments and wing walls, a geotechnical investigation program consisting of a series of auger probes was requested by MRC. Results of auger probe findings are included in Appendix C.

Representative samples of the soils encountered in the boreholes were recovered at 0.75 m intervals. Soil samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Where standard penetration tests were not carried out the consistency/relative density of the encountered soils was estimated from manual examination or the rate (ease) of advance of the augers.

The boreholes were backfilled in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes.



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 and soil classification. The laboratory test program comprised the following tests:

- Natural moisture content determinations (16)
- Grain size analyses (6)
- Atterberg limits tests (2)

The results of the laboratory tests are shown on the Record of Borehole sheets. The grain size distribution charts are presented in Figures SL-GS-1 to SL-GS-6 and the plasticity charts are presented in Figures SL-PC-1 and SL-PC-2.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including moisture conditions, soil classifications, bedrock descriptions, inferred stratigraphy, boundary elevations and groundwater observations.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are presented on the foundation Drawing SL-1.

The subsurface stratigraphy consisted of a pavement structure overlying fill and silty sand mantling dolostone bedrock. Localized silty clay, clayey silt and gravelly sand layers were contacted in some boreholes.

4.1 Pavement Structure

A pavement structure generally comprising 100 and 125 mm of asphalt underlain by 470 to 925 mm of sand and gravel base course was contacted in all the boreholes.



4.2 Fill

Below the pavement structure, a 0.4 to 1.4 m thick, mixed fill was contacted to 1.1 to 2.3 m depth, elevation 221.5 to 222.3 in all boreholes except borehole S6. The fill consisted of very soft to firm, clayey silt, loose to compact sandy silt and loose sand with some silt. SPT N values in the fill ranged from 2 to 27.

A grain size distribution chart of the recovered sand fill sample is presented in Figure SL-GS-1. The sample comprised 74% sand, 13% silt, 9% gravel and 4% clay size materials.

4.3 Silty Clay / Clayey Silt

Below the fill in borehole S-1, a localized 1.1 m thick, firm silty clay stratum was contacted to 2.2 m depth, elevation 221.2. Below the pavement structure in borehole S-6, a localized 1.5 m thick firm clayey silt stratum was contacted to 2.2 m depth, elevation 221.7. These deposits contained variable amounts of cobbles and boulders detected during the augering of the boreholes.

A grain size distribution chart of the recovered silty clay sample is presented in Figure SL-GS-2. The sample comprised 50% clay, 47% silt, and 3% sand size particles. A plasticity chart of the silty clay sample is presented in Figure SL-PC-1. The Atterberg liquid and plastic limit were 44 and 21 respectively, with a plasticity index of 23. The natural moisture content of the silty clay sample was 30%.

A grain size distribution chart of the recovered clayey silt sample is presented in Figure SL-GS-3. The sample comprised 68% silt, 27% clay and 5% sand size particles. A plasticity chart of the clayey silt sample is presented in Figure SL-PC-2. The Atterberg liquid and plastic limit were 27 and 17 respectively, with a plasticity index of 10. The natural moisture content of the clayey silt sample was 22%.



4.4 Silt

Below the fill in borehole S2, a localized 2.3 m thick, loose to compact silt stratum was contacted to 4.1 m depth, elevation 219.6.

A grain size distribution chart of the recovered silt sample is presented in Figure SL-GS-4. The sample comprised 83% silt, 13% clay and 4% sand size particles. The natural water content of the silt sample was 21%.

4.5 Silty Sand/ Sandy Silt

Silty sand was contacted in borehole S1, underlying the silty clay and underlying the pavement structure in boreholes S3, S4 and S5 at 1.4 to 2.3 m depth, elevation 221.5 to 223.3 and extended to termination on probable bedrock at 2.6 to 3.8 m depth, elevation 219.6 to 222.1. SPT N values in the 1.0 to 1.6 m thick silty sand layer ranged from 3 to 13 indicating a loose to compact relative density. Boulders and cobbles were encountered in the silty sand as evidenced by SPT N values of 50 blows over 100 mm in Borehole S3.

Sandy silt was contacted below the clayey silt at 2.2 m depth, elevation 221.7, in borehole S6. SPT N values in the sandy silt were 6 and 12. The natural moisture content of the sandy silt samples were 19 and 21%.

A grain size distribution chart of the recovered sandy silt sample is presented in Figure SL-GS-5. The sample comprised 56% silt, 34% sand, 5% clay and 5% gravel.



4.6 Sand with gravel

Below the silty sand in borehole S-4, a localized 0.5 m thick stratum of very dense sand with gravel was contacted from 2.3 to 2.8 m depth, elevation 221.6 to 221.1.

A grain size distribution chart of the recovered silty sand sample is presented in Figure SL-GS-6. The sample comprised 53% sand, 23% gravel, 20% silt and 4% clay. A moisture content determination obtained 10%.

4.7 Bedrock

Auger refusal on bedrock/probable bedrock was contacted in all boreholes at 2.6 to 4.1 m depth, elevation 219.6 to 221.1. Dolomite bedrock was cored for 4.1 and 3.8 m in boreholes S3 and S4, respectively.

A detailed description of the rock cores retrieved from the boreholes is provided in the attached Table 1 and included on the Record of Borehole logs.

The bedrock comprised light grey to blue grey dolomite of low to medium strength in slightly weathered to unweathered condition, and is described in further detail in the following sections.

4.7.1 South East Corner of Existing Bridge

Borehole S3 was advanced at the southeast corner of the existing bridge. The bedrock surface was encountered at 2.6 m depth, elevation 221.1 and cored 4.1m to elevation 217.0.

The measured core recovery varied from 90 to 98%. The Rock Quality Designation (RQD) for the recovered cores ranged from 45 to 98%, increasing with depth indicating poor to excellent rock quality.



4.7.2 North West Corner of Existing Bridge

Borehole S4 was advanced at the northwest corner of the existing bridge. The bedrock surface was encountered at 2.8 m depth, elevation 221.1 and cored 3.8 m to 217.3.

The measured core recovery varied from 67 to 90%. The RQD for the recovered cores ranged from 15 to 78%, indicating very poor to good rock quality.

Photographs of the rock cores taken from the current boreholes are included in Appendix B.

4.8 Groundwater

Groundwater was observed during augering in boreholes S1, S2, S5 and S6 at 2.1 to 2.3 m depth, elevation 221.1 to 221.6. Upon completion of augering, groundwater was established at 1.9 to 2.2 m depths, elevations 221.2 to 221.8, in boreholes S1, S2, S5 and S6.

Boreholes S3 and S4 were charged with water due to coring operations hence groundwater measurements were not taken in these boreholes.

The groundwater level in the Stoney Creek was indicated to be at approximately elevation 222.0 on the MRC preliminary drawing.

The groundwater is subject to fluctuations at the site due to seasonal conditions and rainfall patterns.



5. CLOSURE

The field work was carried out under the supervision of Mr. A. Lo, Senior Technician, and direction of Ms. N. Balakumaran, P.Eng and Mr. C. M. P. Nascimento, P.Eng., Project Manager and MTO Designated Principal Contact. Aardvark Drilling supplied the soil and rock drilling equipment.

This report was prepared by Mr. H. Gharegrat, P.Eng., and reviewed by Mr. B.R. Gray, MEng, P.Eng., Principal Consultant. Mr. C.M.P. Nascimento, P.Eng. carried out an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.



Harry Gharegrat, MS, P.Eng.
Senior Engineer



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



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

HG/CN/BRG/hg-nk

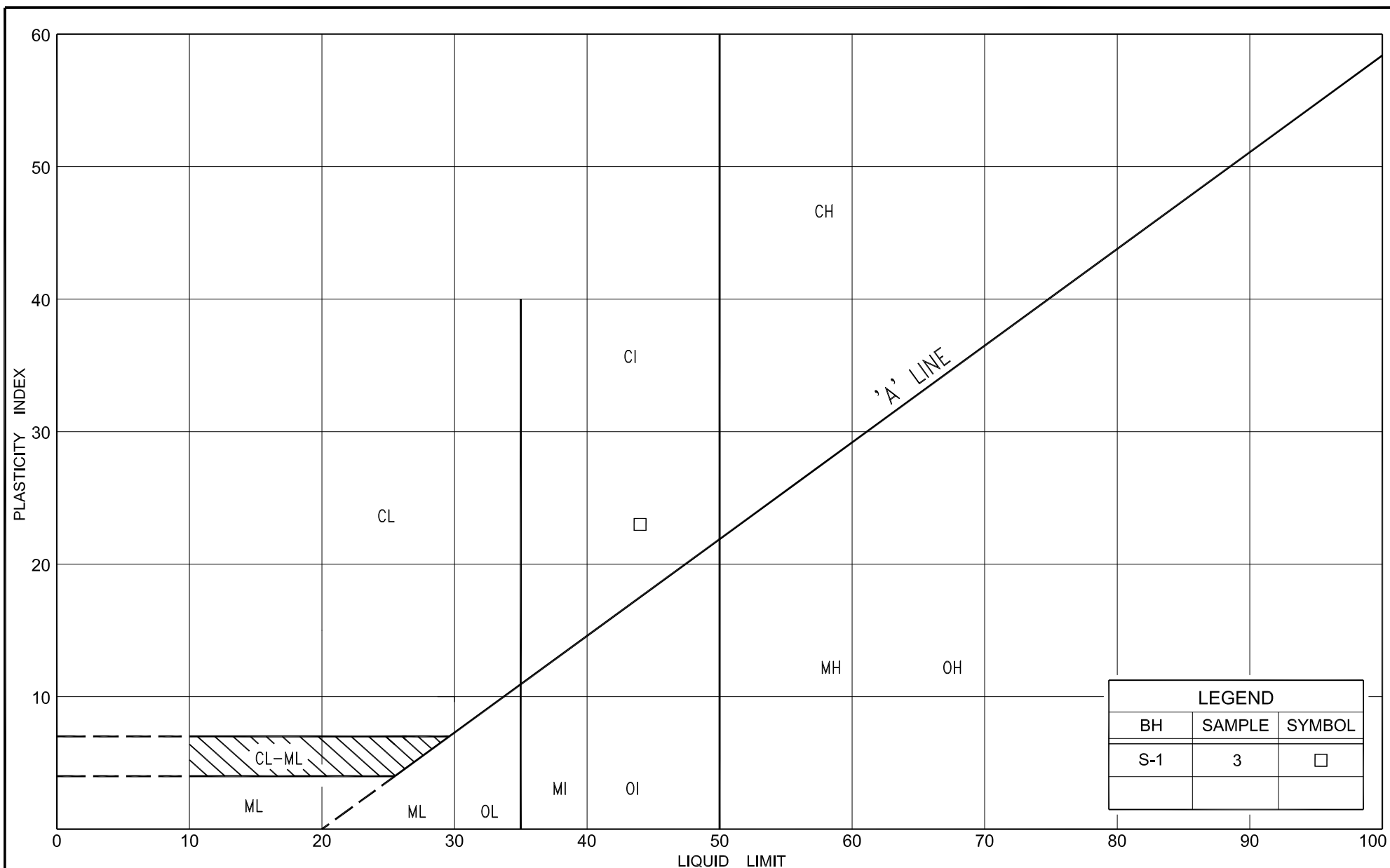


TABLE 1
ROCK CORE DESCRIPTIONS

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
S3	5	2.6 – 3.7	93	45	2.6 – 3.7	DOLOMITE: Light grey, fine crystalline, highly porous, fossiliferous, low to medium strength, slightly weathered, close spaced flat discontinuities, tight, poor quality.
	6	3.7 – 5.2	90	69		
	7	5.2 – 6.7	98	98	3.7 – 6.7	DOLOMITE: Blue grey to buff with dark bituminous or argillaceous laminations, aphanitic, with occasional vugs, 5 to 25 mm in diameter with calcite crystals, medium strength, slightly weathered, close to moderately spaced flat bedding layers, rough planar, tight to slightly altered with oxidation stains on partings, fair to excellent quality.
S4	5	2.8 – 3.6	80	37	2.8 – 6.6	DOLOMITE: Blue grey to buff with dark bituminous or argillaceous laminations, aphanitic, with occasional vugs to 50 mm diameter, with calcite crystals, medium strength, slightly weathered to unweathered with possible highly weathered zone, close to moderately spaced flat bedding layers, smooth to rough planar, tight to slightly altered, very poor to good quality. <u>Note:</u> Low core recovery in run 6 may reflect highly weathered material.
	6	3.6 – 5.1	67	15		
	7	5.1 – 6.6	90	78		

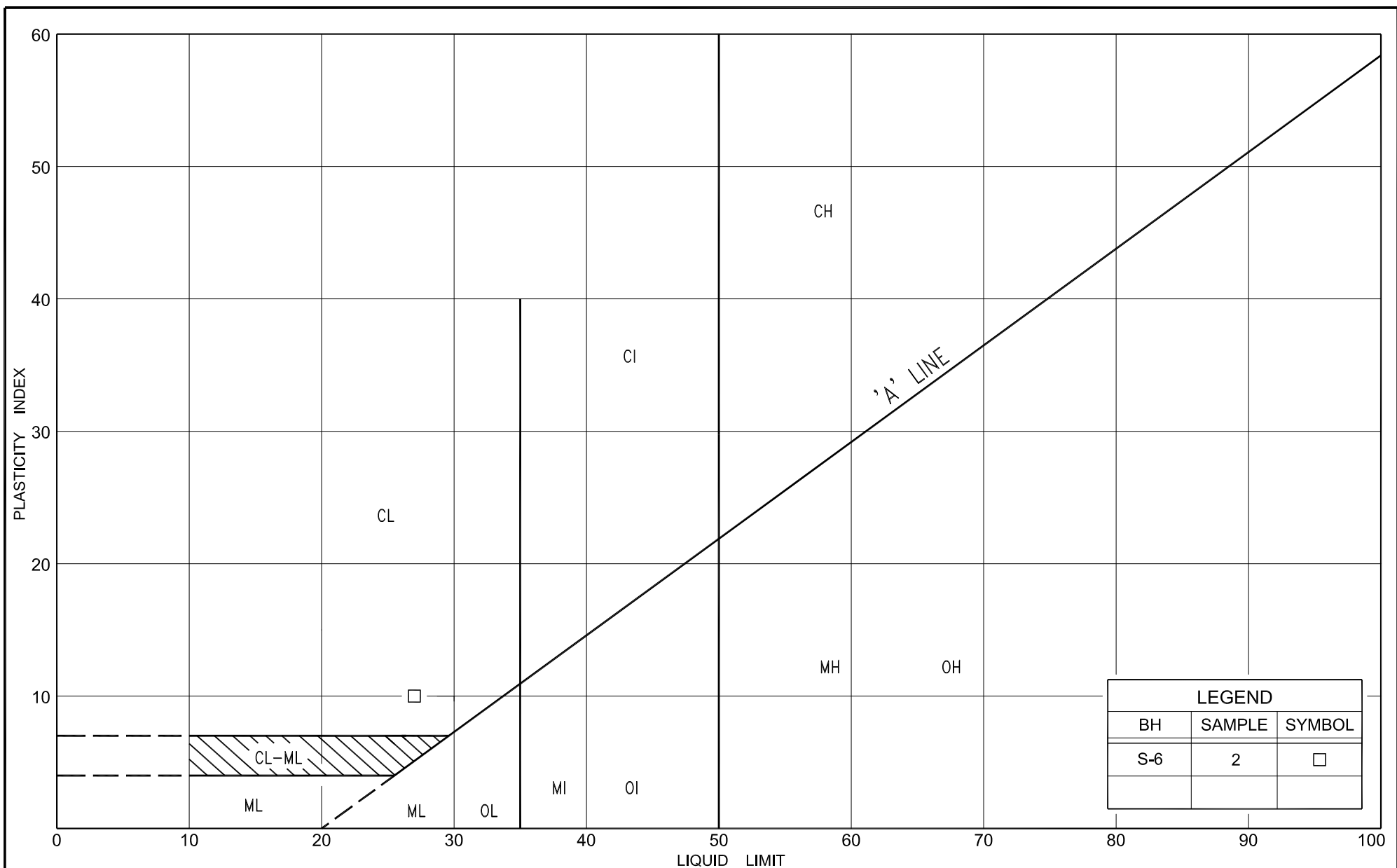
NOTE: RQD = Rock Quality Designation

Originated: JFW
 Compiled: FP
 Checked: NB / CN



PLASTICITY CHART
 SILTY CLAY, trace sand (CI)

FIG No. SL-PC-1
 HWY: 6
 G.W.P. No. 43-00-00

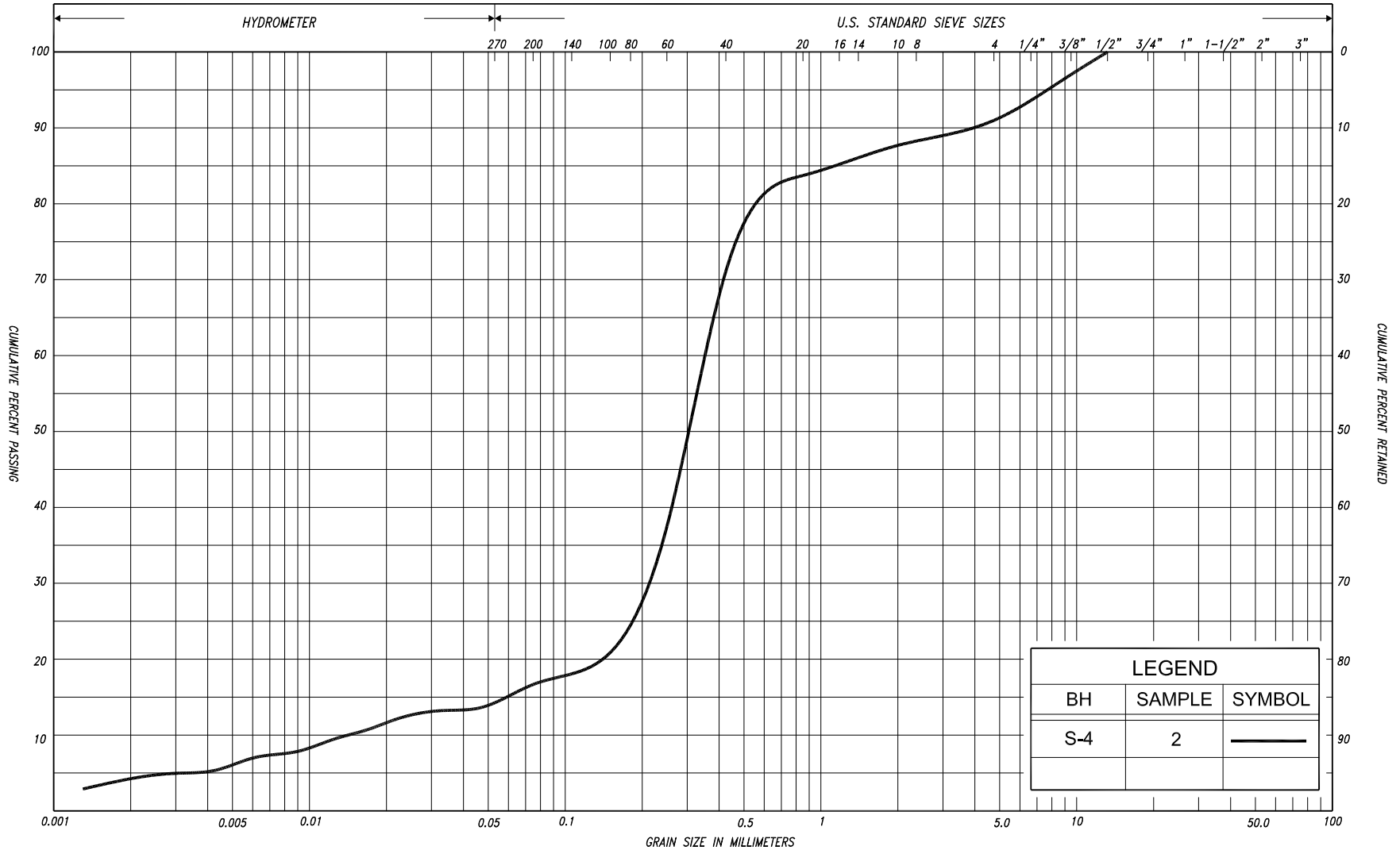


PLASTICITY CHART
CLAYEY SILT, trace sand (CL)

FIG No. SL-PC-2

HWY: 6

G.W.P. No. 43-00-00



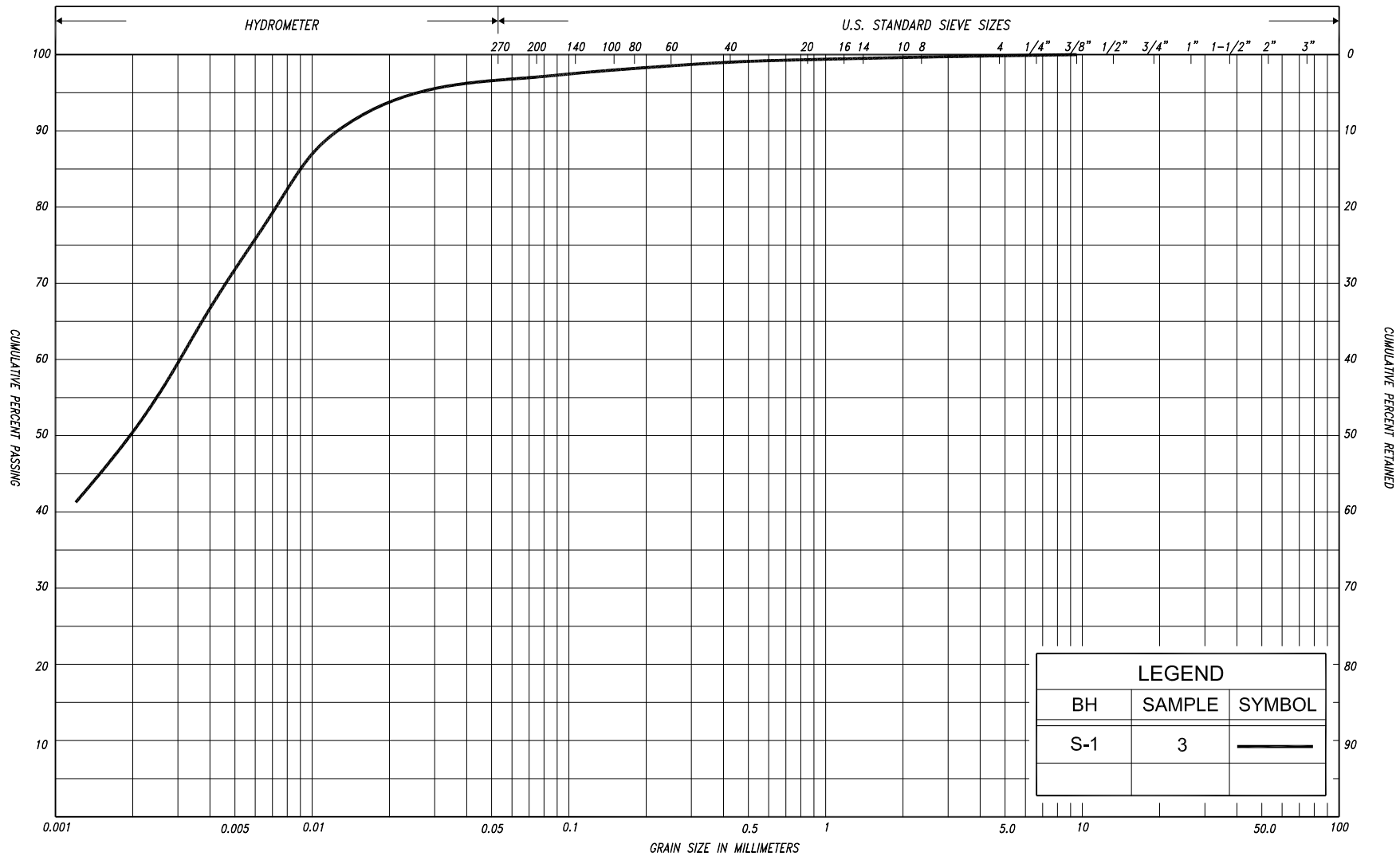
SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL				COB BLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL					COBBLES
	SILT																
CLAY			SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU
						SAND											



GRAIN SIZE DISTRIBUTION

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

FIG No. SL-GS-1
HWY: 6
G.W.P. No. 43-00-00



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												



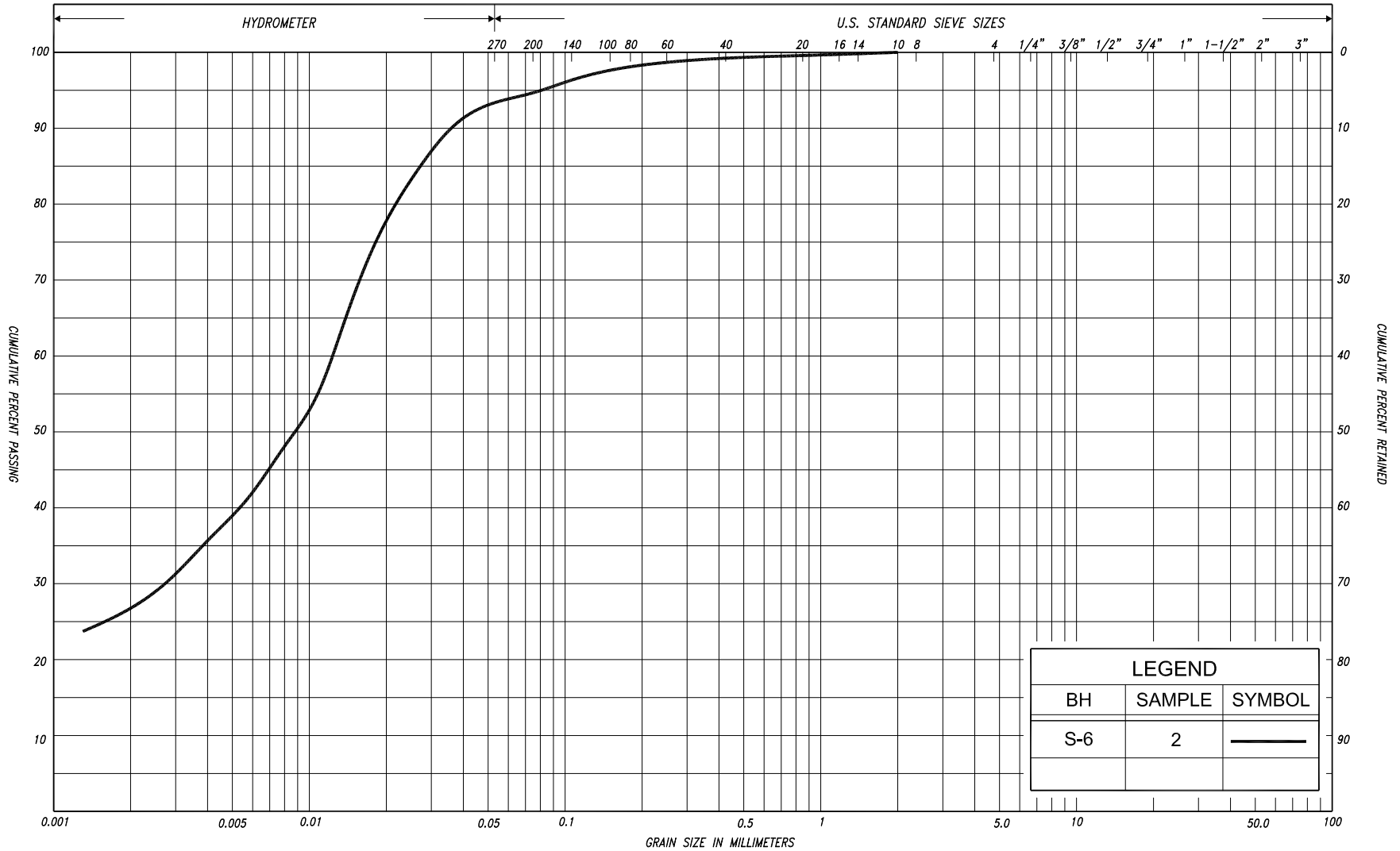
GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand (CI)

FIG No. SL-GS-2

HWY: 6

G.W.P. No. 43-00-00



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

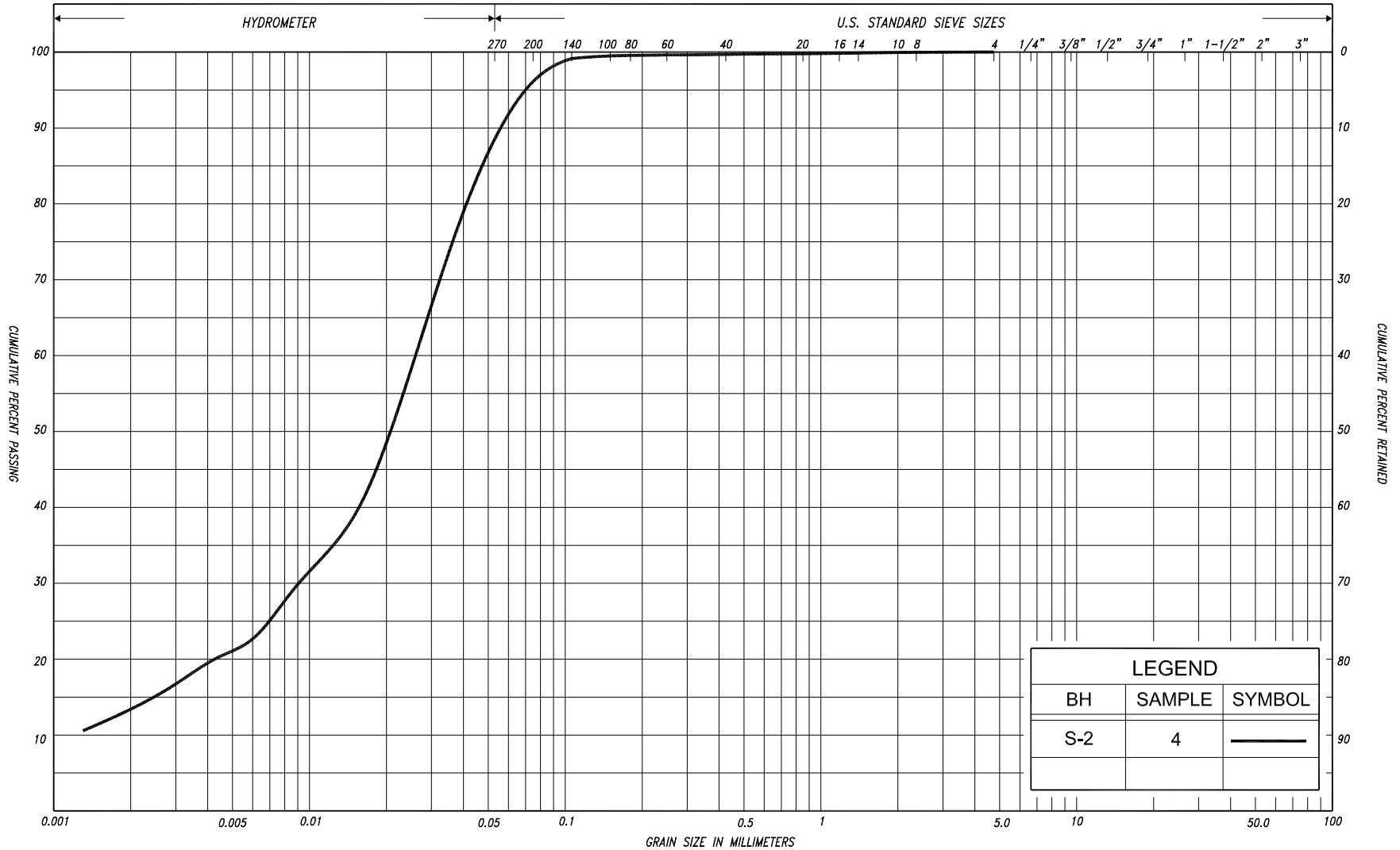


GRAIN SIZE DISTRIBUTION CLAYEY SILT, trace sand (CL)

FIG No. SL-GS-3

HWY: 6

G.W.P. No. 43-00-00



LEGEND		
BH	SAMPLE	SYMBOL
S-2	4	—

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



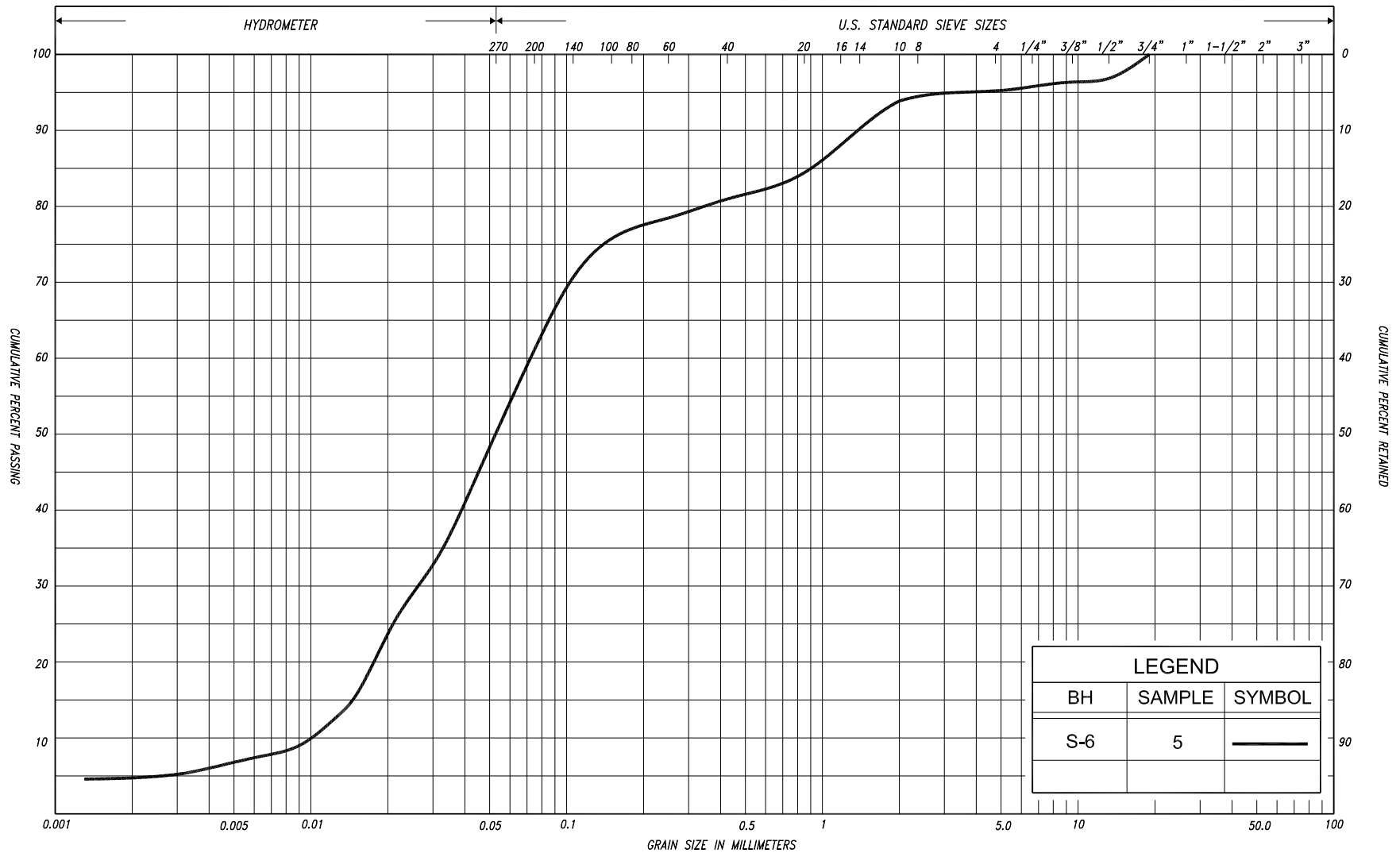
GRAIN SIZE DISTRIBUTION

SILT, some clay, trace sand

FIG No. SL-GS-4

HWY: 11

G.W.P. No. 43-00-00



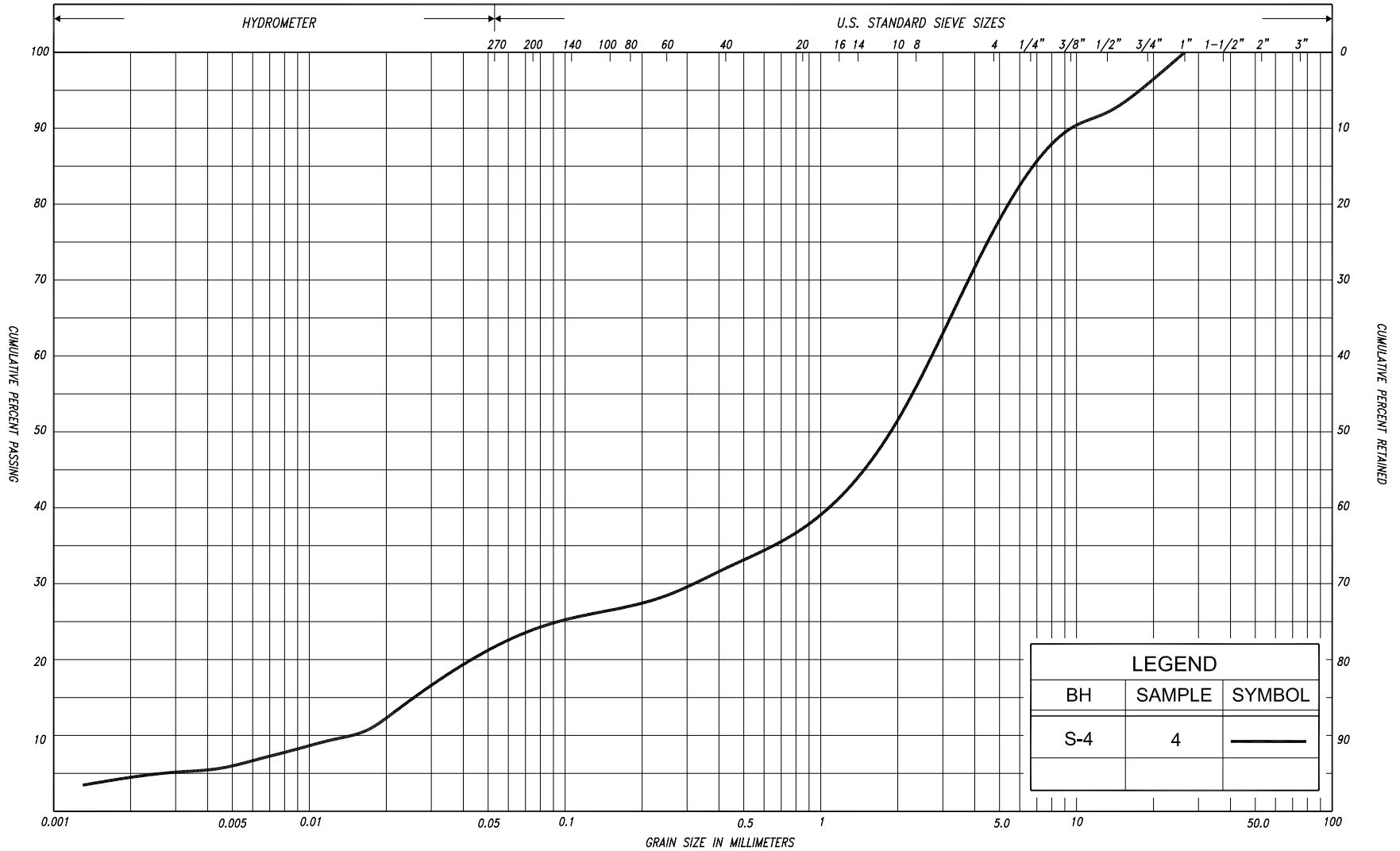
SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
					SAND											
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT															
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU
					SAND											



GRAIN SIZE DISTRIBUTION

SANDY SILT, trace clay, trace gravel

FIG No. SL-GS-5
 HWY: 6
 G.W.P. No. 43-00-00



LEGEND		
BH	SAMPLE	SYMBOL
S-4	4	—

SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COB BLES	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
SAND, with gravel, some silt, trace clay

FIG No. SL-GS-6
HWY: 6
G.W.P. No. 43-00-00

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	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No S1

1 of 1

METRIC

G.W.P. 43-00-00 **LOCATION** Co-ords: 4 942 658.6 N ; 416 957.4 E
Hwy 6, Sta. 20+397, o/s 4.5m Lt. **ORIGINATED BY** A.L.
DIST London **HWY** 6 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** H.G.
DATUM Geodetic **DATE** January 10, 2012 **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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										○ UNCONFINED + FIELD VANE		
								● QUICK TRIAXIAL × LAB VANE										○ UNCONFINED + FIELD VANE		
223.4	Ground Surface						20	40	60	80	100									
0.0	130mm asphalt over 600mm sand and gravel		1	SS	14		223													
	Compact Brown																			
222.3	Clayey silt, trace sand trace gravel, rootlets topsoil inclusions		2	SS	4															
1.1	Firm Brown Moist (FILL)						222													
	Silty clay, trace sand		3	SS	4															
221.2	Firm Greyish Moist brown					▽* ▼*	221									0 3 47 50				
2.2	Silty sand, trace clay cobbles and boulders		4	SS	3															
	Loose to Greyish Wet compact brown																			
			5	SS	10		220													
219.6	layer of gravel																			
3.8	End of borehole Refusal on probable bedrock																			
* 2012 01 10																				
▽ Water level observed during drilling																				
▼ Water level measured after drilling																				

RECORD OF BOREHOLE No S2

1 of 1

METRIC

G.W.P. 43-00-00

LOCATION

Co-ords: 4 942 669.4 N ; 416 938.2 E
Hwy 6, Sta. 20+419, o/s 5.0m Lt.

ORIGINATED BY A.L.

DIST London

HWY 6

BOREHOLE TYPE

Continuous Flight Hollow Stem Augers

COMPILED BY H.G.

DATUM Geodetic

DATE

January 05 and 10, 2012

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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
223.7	Ground Surface							20	40	60	80	100								
0.0	130mm asphalt over 680mm sand and gravel		1	SS	29	* ▽	223										0 4 83 13			
	Compact Brown																			
	Sandy silt trace clay, trace gravel rootlets, topsoil inclusions limestone fragments cobbles and boulders		2	SS	27															
221.9	Compact Brown Moist (FILL)		3	SS	10															
1.8	Silt some clay, trace sand cobbles and boulders		4	SS	3		221													
	Loose to Greyish Wet compact brown		5	SS	25		220													
219.6	layer of gravel																			
4.1	End of borehole Refusal on probable bedrock																			

RECORD OF BOREHOLE No S3

1 of 1

METRIC

G.W.P. 43-00-00

LOCATION

Co-ords: 4 942 677.0 N ; 416 945.1 E
Hwy 6, Sta. 20+417, o/s 5.0m Rt.

ORIGINATED BY A.L.

DIST London

HWY 6

BOREHOLE TYPE C.F.H.S.A. and NO Diamond Coring

COMPILED BY H.G.

DATUM Geodetic

DATE January 10, 2012

CHECKED BY B.R.G.

[illegible]

RECORD OF BOREHOLE No S4

1 of 1

METRIC

G.W.P. 43-00-00

LOCATION

Co-ords: 4 942 682.8 N ; 416 915.9 E
Hwy 6, Sta. 20+445, o/s 5.0m Lt.

ORIGINATED BY A.L.

DIST London

HWY 6

BOREHOLE TYPE C.F.H.S.A. and NQ Diamond Coring

COMPILED BY H.G.

DATUM Geodetic

DATE _____

January 04 and 05, 2012

CHECKED BY B.R.G.

[illegible]

RECORD OF BOREHOLE No S5

1 of 1

METRIC

G.W.P. 43-00-00 **LOCATION** Co-ords: 4 942 692.0 N ; 416 919.1 E
Hwy 6, Sta. 20+447, o/s 4.5m Rt. **ORIGINATED BY** A.L.
DIST London **HWY** 6 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** H.G.
DATUM Geodetic **DATE** January 05, 2012 **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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
223.8	Ground Surface							20	40	60	80	100								
0.0	130mm asphalt over 925mm sand and gravel		1	SS	20		223													
	Compact Brown to loose																			
	_____		2	SS	7															
	Clayey silt, trace sand rootlets, organics and topsoil inclusions																			
	Firm to Brown Moist very soft		3	SS	2															
221.5	(FILL)																			
2.3	Silty sand		4	SS	12		221													
	Compact Brown Wet																			

	cobbles		5	SS	13															
220.2	End of borehole																			
3.6	Refusal on probable bedrock																			
	* 2012 01 05																			
	▽ Water level observed during drilling																			
	▼ Water level measured after drilling																			

RECORD OF BOREHOLE No S6

1 of 1

METRIC

G.W.P. 43-00-00 **LOCATION** Co-ords: 4 942 702.3 N ; 416 902.0 E
DIST London **HWY** 6 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **ORIGINATED BY** A.L.
DATUM Geodetic **DATE** January 05, 2012 **COMPILED BY** H.G.
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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
223.9	Ground Surface																			
0.0	130mm asphalt over 550mm sand and gravel		1	SS	20															
223.2	Compact Brown																			
0.7	Clayey silt, trace sand		2	SS	5		223									0 5 68 27				
	Firm Brown Moist																			
	cobbles and boulders		3	SS	50/5cm															
	Greyish brown						222													
221.7	Sandy silt																			
2.2	trace clay, trace gravel		4	SS	6															
	Loose to Brown Wet compact						221													
220.4	seams/layer of sand and gravel		5	SS	12											5 34 56 5				
3.5	End of borehole																			
	Refusal on probable bedrock																			
	* 2012 01 05																			
	▽ Water level observed during drilling																			
	▼ Water level measured after drilling																			



KEY PLAN
NOT TO SCALE

LEGEND

- Borehole
- Auger Probe
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation Nov. and Dec. 2011
- * Water level not established
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
S1	223.4	4 942 658.6	416 957.4
S2	223.7	4 942 669.4	416 938.2
S3	223.7	4 942 677.0	416 945.1
S4	223.9	4 942 682.8	416 915.9
S5	223.8	4 942 692.0	416 919.1
S6	223.9	4 942 702.3	416 902.0

NOTE

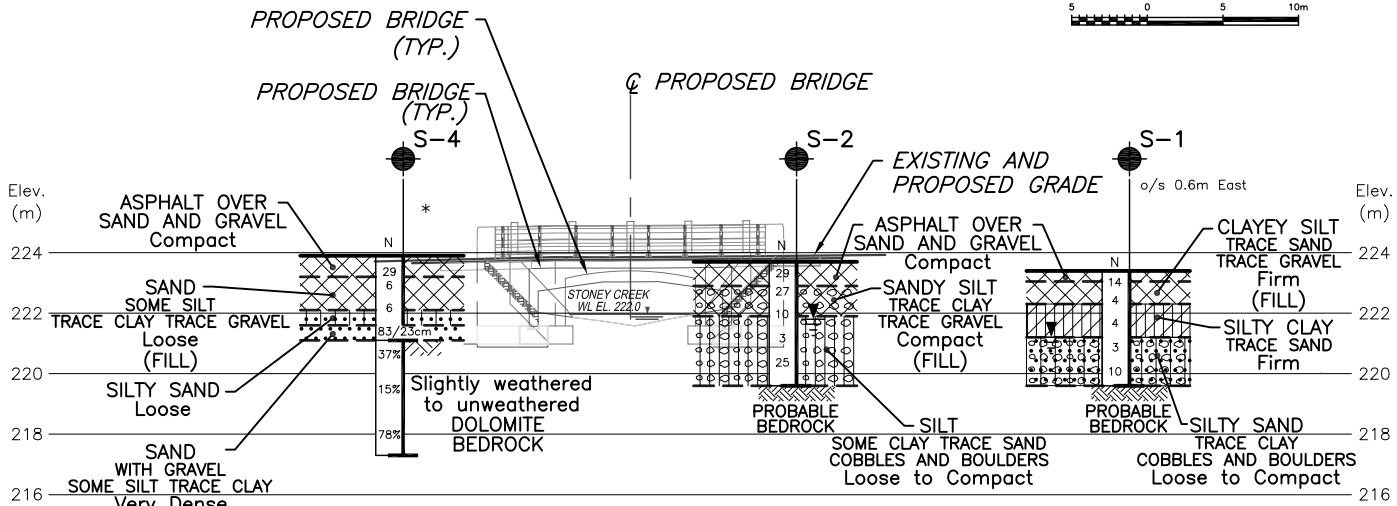
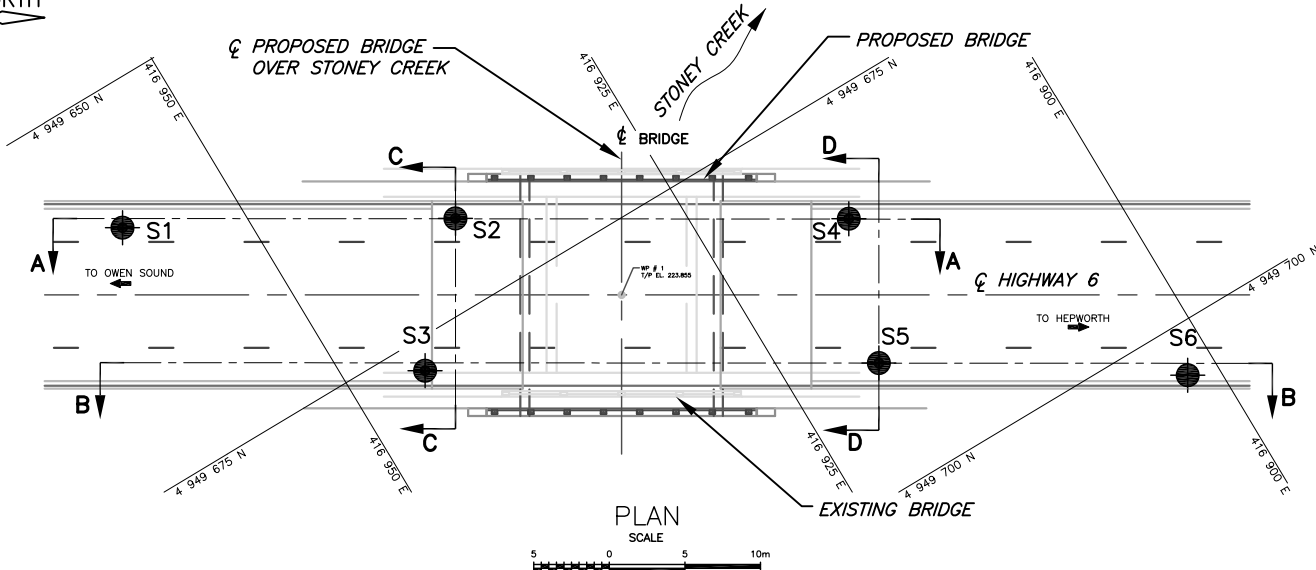
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

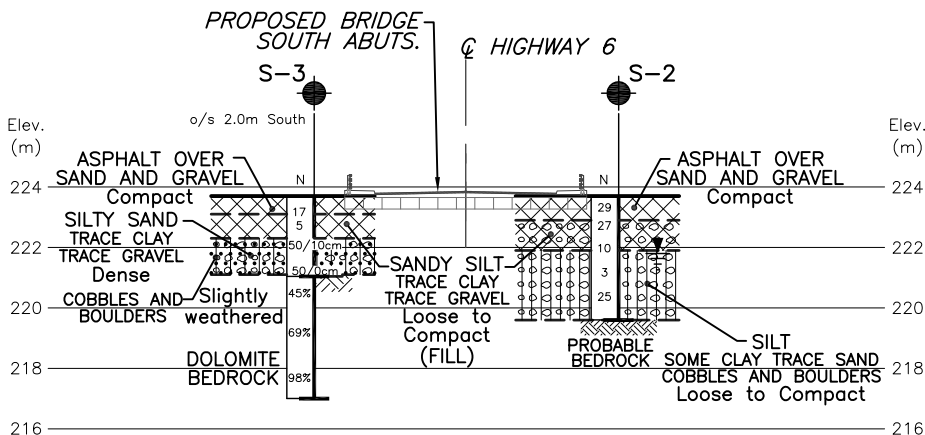
Geocres No. 41A-223

HWY No	6	CHECKED	HG	DATE	OCT. 12, 2012	DIST	London
SUBM'D	NA	CHECKED	BRG	APPROVED	CN	SITE	8-9
DRAWN	NA	CHECKED	BRG	APPROVED	CN	DWG	SL-1

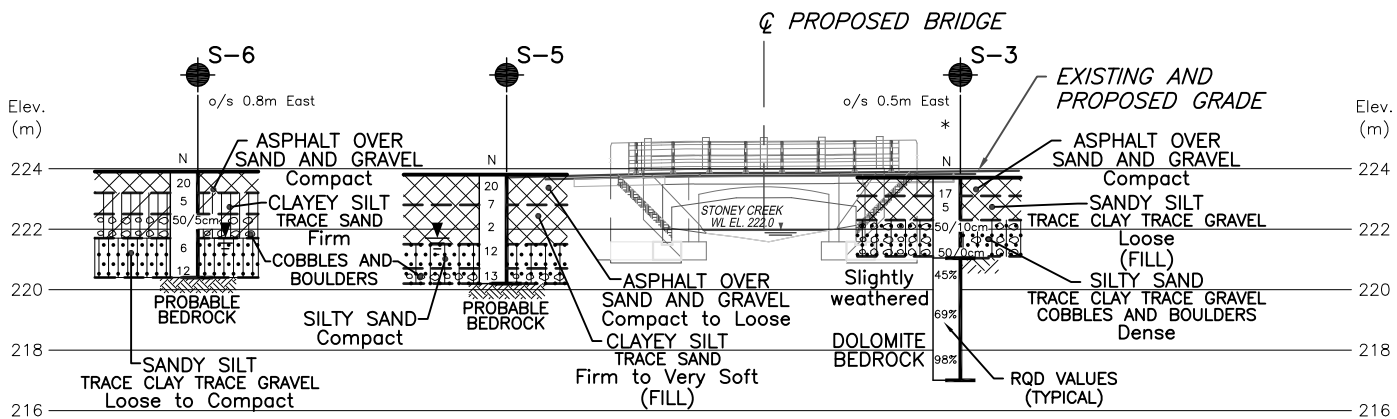
CONSTRUCTION NORTH



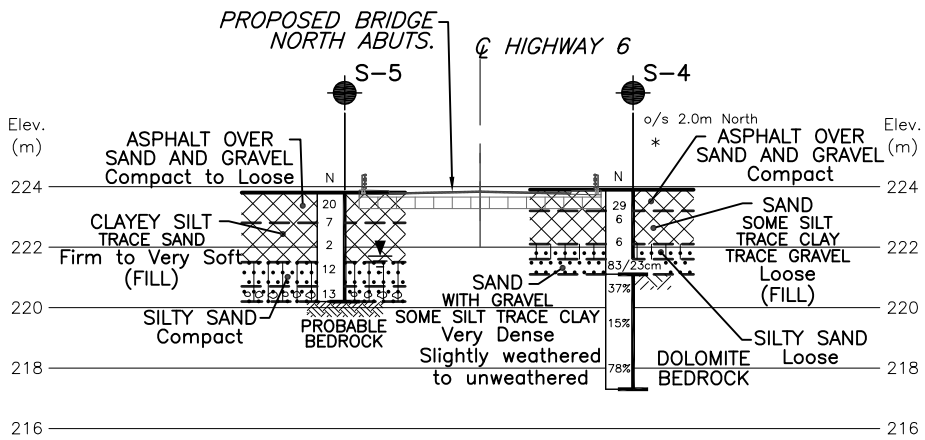
PROFILE A-A



SECTION C-C

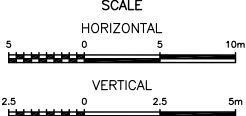


PROFILE B-B



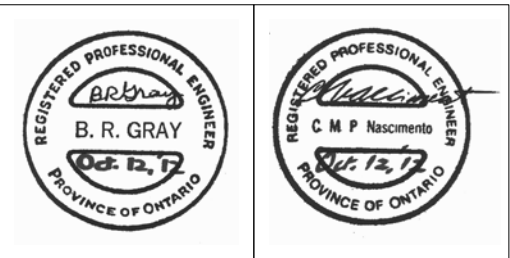
SECTION D-D

PROFILES AND SECTIONS



NOTES:

- THIS DRAWING SL-1 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- 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.



REF MRC Drawing:
3811011-310-001GA_D.dwg dated March 2012



APPENDIX A

Site Photographs



Photograph 1: Looking north from the east side of Highway 6 towards the bridge. (January 10, 2012)



Photograph 2: Looking north from the west side of Highway 6. (January 10, 2012)

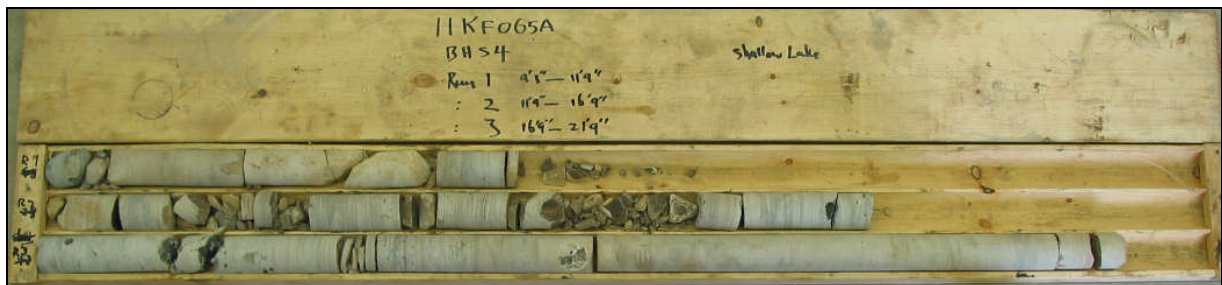


APPENDIX B

Rock Core Photographs



Photograph 1: Cores retrieved from borehole S3. Cores 5 to 7 from 2.6 to 6.7 m depth. RQD values ranged from 45 to 98%, indicating poor, fair to excellent rock quality.



Photograph 2: Cores retrieved from borehole S4. Cores 5 to 7 from 2.8 to 6.6 m depth. RQD values ranged from 15 to 78%, indicating very poor to good rock quality.



APPENDIX C

Auger Probe Findings

April 4, 2012

PML Ref.: 11KF065A
Index No.: 049LET

Mr. Scott Leitch, M.E.Sc., P.Eng.
Project Manager – Associate
Bridge Engineering
McCormick Rankin Corporation
72 Victoria Street South, Suite 100
Kitchener, Ontario
N2G 4Y9

Dear Mr. Leitch

Auger Probe Findings
Existing Shallow Lake Bridge Over Stoney Creek
Highway 6
Site No. 8-9, W.P. 43-00-00
Shallow Lake, Ontario

Peto MacCallum Ltd. (PML) is pleased to present our findings for the subsurface investigation conducted at the above-referenced site. Authorization to proceed with this assignment was provided by Mr. Leitch via email dated March 1, 2012.

Project Background

A geotechnical investigation report is currently being prepared for a bridge which will replace the existing Shallow Lake Bridge over Stoney Creek on Highway 6 in Shallow Lake, Ontario. In order to determine foundation support conditions for the existing bridge abutments and wing walls, a subsurface investigation program was requested by McCormick Rankin Corporation (MRC).

Investigation Procedures

The investigation program recommended by McCormick Rankin consisted of a series of auger probes at the southwest and northeast corners of the bridge, near the abutments and wing walls to determine the extent and thickness of the shallow spread footings used for bridge support.

Due to an existing overhead electrical utility which traverses the bridge at the southwest and northeast corners the probes were advanced at the southeast and northwest corners. The boreholes were advanced without sampling to auger refusal and the refusal materials sampled by coring methods in four probes.

The field work was carried out on March 20 and 21, 2012. The subsurface investigation comprised seven auger probes, three at the northwest corner (Probes 1S, 2S, 3S) and four at the southeast corner (Probes 1N to 4N).

Probes locations were laid out by PML based on distances from the existing bridge abutment specified by MRC.

The probes were advanced using continuous flight solid stem augers powered by a truck-mounted D-25, equipped for rotary core (NQ size) drilling, supplied and operated by a specialist drilling contractor. The drilling crews worked under the full-time supervision of a member of our engineering staff.



The refusal materials were cored in four auger probes locations. After completion of augering and coring, the holes were backfilled in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

Encountered Conditions

A summary of findings is provided in the Table below:

Borehole No.	Location	Depth to Auger Refusal (m)	Core Length (m)	Material Recovered in Core
1S (NW corner)	0.5 m north of north abutment, at approximate centreline of southbound shoulder lane	4.6	0.075 m	Bedrock
2S (NW corner)	2.85 m north of north abutment, 0.55 m east of wing wall pedestrian rail Probes drilled on a slight angle towards the wing wall. Due to this, the footing projection may be about 0.12 m less than indicated.	3.2	0.46	Concrete with 0.15 m of loose aggregate in middle of core
3S (NW Corner)	2.85 m north of north abutment, 1.1 east of wing wall pedestrian rail	3.05	1.07	Granular and cobble fragments
1N (SE corner)	0.5 m south of south abutment, 1.2 m west of wing wall pedestrian rail	3.66	---	---
2N (SE corner)	1.5 m south of south abutment, 1.2 m west of wing wall pedestrian rail	3.66	----	----
3 N (SE corner)	2.5 m south of south abutment, 1.2 m west of wing wall pedestrian rail	3.66	----	----
4 N (SE corner)	2.4 south of south abutment, 0.4 m west of wing wall pedestrian rail	1.5	2.6	1.67 m of cobbles underlain by 0.93 m of concrete. Top 0.12 m of concrete flat sided.



The auger probe findings indicate that a concrete footing was not encountered beyond 0.5 m of the abutment at the southeast and northwest corners of the bridge. The probe drilled at the wing wall at the northwest corner of the bridge contacted concrete 0.55 m east of the wing wall. This dimension is approximately 0.12 m smaller due to a small angle of drilling towards the wall that was required to set-up the drill rig. The borehole drilled at the southeast corner of the bridge contacted concrete 0.4 m west of the wing wall.

We trust the information presented in this report is sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



Harry Gharegrat, MS, P.Eng.
Senior Engineer



Carlos M.P. Nascimento, P.Eng.
Project Manager

HG/CN:mm

Enclosure(s):

Sketch 1 – Auger Probe Location Plan

Distribution:

- 1 cc: Mr. Scott Leitch, McCormick Rankin Corporation
- 1 cc: Mr. Dan Green, McCormick Rankin Corporation
- 1 cc: PML Toronto
- 1 cc: PML Kitchener



1



**FOUNDATION DESIGN REPORT
for
REPLACEMENT OF SHALLOW LAKE BRIDGE
OVER STONEY CREEK, HIGHWAY 6
SITE NO. 8-9
SHALLOW LAKE, ONTARIO
G.W.P. 43-00-00**

PETO MacCALLUM LTD.
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Distribution:

- 3 cc: McCormick Rankin (MRC) for distribution to
MTO, Project Manager – West Region
(London) + 1 digital copy (pdf)
- 3 cc: Foundation Investigation Report only to MRC
for distribution to MTO, Project Manager –
West Region (London) + 1 digital copy (pdf)
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PML Ref.: 11KF065A
Index No. 094FDR
GEOCRES No. 41A - 223
October 15, 2012



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Table 1 – List of Standard Specifications Referenced in Report

FOUNDATION DESIGN REPORT

For
Replacement of Shallow Lake Bridge over Stoney Creek
Site No. 8-9
Highway 6, G.W.P. 43-00-00
Shallow Lake, Ontario

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and approach embankments for the proposed replacement of Shallow Lake Bridge on Highway 6 in Shallow Lake. The investigation was conducted for McCormick Rankin (MRC), a member of MMM Group Ltd. on behalf of the Ministry of Transportation of Ontario (MTO).

The site is located on Highway 6 (Princess Street), about 170 m west of the intersection of Highway 170 and Highway 6 in Shallow Lake.

The replacement bridge will include a 12.2 m long, single span structure. Based on the Preliminary General Arrangement Drawings dated February 2012, provided by MRC, the proposed east and west abutment footing elevations are planned at elevation 220.98. The planned bridge deck will be at elevation 223.85. The approach embankments will be level with the bridge deck. The high water elevation (50 yr storm) is indicated as 223.94 on the provided drawing.

In summary, the depth of the soil cover mantling dolostone bedrock, revealed in the boreholes varied from 2.6 to 4.1 m, elevation 219.6 to 221.1. Generally the soil stratigraphy included a pavement structure consisting of asphalt and sand and gravel base overlying fill and silty sand mantling dolostone bedrock. Discontinuous layers of silty clay, clayey silt and sand with gravel were contacted in the boreholes below the fill. Cobbles and boulders were encountered during drilling of most of the boreholes.



Groundwater was observed during augering in boreholes S1, S2, S5 and S6 at 2.1 to 2.3 m depth, elevation 221.1 to 221.6. Upon completion of augering, groundwater was established at 1.9 to 2.2 m depths, elevations 221.2 to 221.8, in boreholes S1, S2, S5 and S6. It is inferred that the groundwater is influenced by the water level in the Stoney Creek that was previously recorded at approximately elevation 222.0. Groundwater levels in the creek may vary significantly during/following weather events.

Bedrock was contacted near the proposed foundation elevation of 220.98 at the abutments. The bedrock surface is undulating varying from 219.6 to 221.1 at the bridge abutment boreholes. It should be feasible to support the bridge abutments on shallow spread footings bearing on bedrock.

Piles and caissons are considered to be technically and economically unfeasible due to the shallow depth to bedrock and due to the presence of boulders and cobbles within the fill and native materials.

When excavating the soil cover to expose the bedrock surface, the presence of cobbles and boulders should be considered as this will affect excavation productivity.

The extent and sophistication of the groundwater control measures will be dependent on the specific conditions at the time of the excavation. Groundwater control in the form of sandbagging the existing creek around the bridge abutments and pumping the water to a sump pit or a sheet pile wall cofferdam will be required to conduct foundation excavations in the dry. Alternatively foundation concrete will need to be placed by the tremie method.

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



2. FOUNDATIONS

2.1 General

It is considered feasible to support the abutment foundations on a spread footing placed directly on bedrock. The depth below grade and the surface elevation of the bedrock proved or inferred in the boreholes drilled at this site are summarized in the following Table.

LOCATION		BOREHOLE NO.	BEDROCK DEPTH (m)	BEDROCK ELEVATION
South Abutment	West Side	S2	4.1	219.6
	East Side	S3	2.6	221.1
North Abutment	West Side	S4	2.8	221.1
	East Side	S5	3.6	220.2

For identification of bedrock elevation in the contract documents, a general bedrock level at elevation 221.1 \pm 0.5 m should be used to account for variability of the bedrock surface at the footing locations.

Total and differential settlements of footings placed on the bedrock will be negligible.

Footings bearing directly on bedrock do not require foundation frost protection.

The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

2.2 Spread Footings on Bedrock.

A factored geotechnical bearing resistance at Ultimate Limit State (ULS) of 3,000 kPa should be used for the design of the spread footings bearing on the dolomite bedrock. A reduced bearing pressure than would normally be available for dolomite bedrock is provided considering the low to medium strength of the bedrock and the lower local RQD of the bedrock in Borehole S4.



The geotechnical resistance at Serviceability Limit State (SLS) is not applicable for footings founded on bedrock, since the bedrock is considered to be non-yielding. For structural computation purposes the geotechnical resistance at SLS may be taken as 3,000 kPa. The geotechnical bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

Mass concrete may be required to provide a level founding surface for the footings and/or to also raise the subgrade to a higher founding level of the footings, if required. The borehole findings indicate that 0.8 to 1.4 m of mass concrete fill may be required at the bridge abutment footings for this purpose when the footings are placed at an approximately elevation 221.0. The need to expand the plan area at the base of the mass concrete to provide for stress distribution (2V:1H), place reinforcing steel in the mass concrete and/or use high strength concrete to prevent overstressing of the mass concrete will be dictated by the actual thickness of the mass concrete and structural design considerations.

Subject to these comments, the bearing resistance provided for footings bearing on bedrock is considered to be appropriate for mass concrete with an unconfined compressive strength of at least 25 MPa.

Comments concerning excavation of the bedrock, where required to found the footings at an adequate level, are provided in section 5 of this report. Boulders and cobbles should be anticipated when excavating the soil cover to expose the bedrock surface.

The horizontal force imposed on the foundations will be resisted in part by the friction developed between the underside of the footing and the founding bedrock.

An unfactored friction factor of 0.7 is considered to be suitable at this site on the "rough bedrock surfaces" (asperity height of at least 25 mm). The factored horizontal resistance at ULS of the bedrock is considered to be 1,500 kPa.

The horizontal resistance of footings founded on bedrock could be increased, if required, by installing shear keys, sockets or anchors into the bedrock (SP 999S26). The increased lateral



resistance will be provided by the shear strength of the steel dowels, the horizontal resistance of the bedrock and the horizontal component of tensile forces developed in any inclined anchors. A greater frictional resistance between the footing and rock may be achieved if the anchors are prestressed to increase the vertical pressure.

If dowels are employed, a NSSP should be included in the tender documents to provide specific direction for the contractor during installation and testing of the dowels. Fractured rock should be removed from these areas. A NSSP should also be prepared for the shear keys.

Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 (CHBDC). If anchors are installed, a factored bond stress at the rock/grout interface of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) is recommended for design. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length.

The relative advantages and disadvantages of supporting the bridge abutments on shallow spread footings are as below:

SPREAD FOOTINGS ON BEDROCK	
Advantages	Disadvantages
Ease of Installation	Requires removal of all boulders from the foundation footprint
Lower cost than deep foundations	Removal of fractured rock
May be used with semi integral abutments	Requires mass concrete to provide level bearing surface
High bearing resistance	
DRIVEN PILES	
Advantages	Disadvantages
Negligible settlement of foundations	Not feasible due to shallow bedrock
	Specialized equipment required for driving piles
	Difficult driving due to presence of cobbles and boulders in the fill and native soil



SPREAD FOOTINGS ON BEDROCK	
AUGERED CAST IN PLACE CAISSONS	
Advantages	Disadvantages
Negligible settlement of foundations	Not economically feasible due to numerous boulders bedrock
Allows embedment into the bedrock for improved fixity	Presence of boulders and cobbles may impede caisson installation
	Specialized equipment required for drilling into bedrock

3. LATERAL EARTH PRESSURE ON ABUTMENTS AND WING WALLS

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

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

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 for the conditions at this site was provided in Section 2.1.



Hydrostatic pressures were not included in the equation since free-draining granular material or rockfill will be used as backfill behind the wall. The following parameters are recommended for design.

PARAMETER	GRANULAR A, GRANULAR B TYPE II or TYPE III	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m ³	22.8	18.0
Coefficient of Active Earth Pressure, K_a	0.27	0.20
Coefficient of Earth Pressure At Rest, K_o	0.43	0.33
Coefficient Passive Earth Pressure, K_p	3.69	5.04

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 and revised 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).

The magnitude of the passive resistance and active pressure is dependent on the actual lateral movement of the structure toward and away from the adjacent soil, respectively. We refer to Figure C6.16 (Clause C6.9.1) of the CHBDC for these computations. The backfill should be considered as medium dense sand for this project.

Weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. For integral abutments, a weeping tile system (OPSS 405 and OPSD 3190.100) should be installed. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to retaining structures should be carried out in conformance with OPSD 3101.150 and 3101.200 for granular or rock backfill at abutments. Backfill placement



should be conducted simultaneously at the north and south abutment and wing walls to prevent build up of unbalanced hydrostatic pressure on one abutment.

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 SP 105S10 for additional information in this regard.

4. APPROACH EMBANKMENTS

It is understood that the approach embankments will be level with the bridge deck and no new fill placement is anticipated to raise the grade.

Backfilling adjacent to the structure abutments should be carried out in conformance to Ontario Provincial Standards Drawings for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200).

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Settlement of the road surface will only be governed by consolidation of the newly placed fill near the abutments. The estimated settlement from the newly placed fill is anticipated to be within tolerable limits provided the fill is placed in accordance with OPSS 501.

4.1 Road Protection

The proposed structure will replace the existing bridge on the same alignment. It is anticipated, therefore, that a suitable roadway protection scheme following OPSS 404 and 539 will be necessary to support the walls of excavation and adjacent traffic lanes during staged construction.

A road protection scheme designed for performance level 1b system is recommended to prevent excessive movement of the existing embankment. The contractor is responsible for selection, preparation of a detailed design and performance for the road protection scheme.



The presence of cobbles and boulders in the fill and/or native soils must be considered when selecting the roadway protection system. Anchored or braced soldier piles and lagging may be considered. It is noted, however, that the contractor shall evaluate the method of advancement for soldier piles and propose means of protecting against potentially excessive loss of the retained soils during installation of a soldier piles and lagging system where the excavation will extend through sandy soils, particularly below the water table.

5. EXCAVATION CONSIDERATIONS

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

According to OHSA criteria, loose to compact cohesionless and firm to stiff cohesive soils are classified as Type 3 soils. The very stiff cohesive and dense to very dense cohesionless soils are considered as Type 2 soils. The bedrock is classified as Type 1 soil. Since open cut procedures are governed by soils with the highest soil type number, temporary cut slopes over the full depth of excavation inclined at 1 horizontal to 1 vertical should be provided assuming adequate drainage measures are in place. Cobbles and boulders should be expected in the excavations.

A large excavator equipped with a tiger-toothed bucket in conjunction with a jackhammer or hoe ram is the preferred method of excavation to shallow depths in rock scaling (SP 299F03). The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of the cut and relative depth of excavation into the bedrock. Mass concrete could be employed to level minor variations in the bedrock surface, as mentioned previously.

It is anticipated that blasting of the bedrock will not be required due to the relatively level formation encountered at the site.

Near vertical sidewalls may be utilized in excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workers.



6. GROUNDWATER CONTROL CONSIDERATIONS

Groundwater was observed during augering in boreholes S1, S2, S5 and S6 at 2.1 to 2.3 m depth, elevation 221.1 to 221.6. Upon completion of augering, groundwater was established at 1.9 to 2.2 m depths, elevations 221.2 to 221.8, in boreholes S1, S2, S5 and S6. The water level was indicated to be at approximately elevation 222.0 on the MRC preliminary drawing.

Excavations for foundations will experience heavy groundwater inflow considering the permeable cohesionless soil encountered above the bedrock, and the relatively high groundwater levels encountered in the boreholes. For construction to be conducted in the dry a steel sheet pile wall cofferdam with sheet piles pinned into the bedrock will be required.

Alternatively sandbags can be used to dam the construction area and pumps and sumps be used to dewater the excavation within the sandbagged area.

Use of tremie concreting method could also be considered for this project.

The requirement for a permit to take water (PTTW) will depend on the water tightness of the contractor's selected type of dewatering system. The PTTW requirement will also depend on the groundwater levels at the time of construction since these are subject to seasonal fluctuations and precipitation patterns. The PTTW may be required to address hydrological considerations.



7. CLOSURE

This report was prepared by Mr. H. Gharegrat, P.Eng., and reviewed by Mr. B.R. Gray, MEng, P.Eng., Principal Consultant. Mr. C.M.P. Nascimento, P.Eng., Project Manager and MTO Designated Principal Contact carried out an independent review of the report.

Yours very truly,

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HG/CN/BRG/hg-nk



TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 404	Construction Specification for Support System
OPSS 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
SP 105S10	Construction Specification for Compaction
SP 299F03	Rock Excavation (Machine Scaling)
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls Abutment, Backfill Rock
OPSD 3190.100	Walls Retaining and Abutment Wall Drain