



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for
HIGHWAY 4, KIPPEN RIVER BRIDGE #2
MTO WEST REGION 59 STRUCTURE REHABILITATIONS
CONTRACT 2
GWP 3125-03-01
TOWNSHIP OF TUCKERSMITH, HURON COUNTY, ONTARIO**

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PML Ref.: 13KF006B
Index No.: 141FIR and 142FDR
GEOCRES No: 40P5-20
November 26, 2014



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PRELIMINARY FOUNDATION INVESTIGATION REPORT

For
Highway 4, Kippen River Bridge #2
MTO West Region 59 Structure Rehabilitations
Contract 2, GWP 3125-03-01
Township of Tuckersmith, Huron County, Ontario

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed rehabilitation of the Highway 4 Kippen River Bridge #2 over the Bannockburn River near the Town of Kippen, Ontario. The proposed rehabilitation is a part of the assignment for the rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. The study was carried out by Peto MacCallum Ltd. (PML) for MMM Group on behalf of the Ministry of Transportation of Ontario (MTO).

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

2. SITE DESCRIPTION AND GEOLOGY

The existing structure was constructed in 1947 and is a single span rigid frame reinforced concrete bridge with a span length of about 10 m. The bridge is located on Highway 4 approximately 550 m south of Highway 12 in Huron County, Ontario.

Based on the book *"The Physiography of Southern Ontario"*, the project site is located within the physiographic region known as the Horseshoe Moraines. The Horseshoe Moraines are mainly composed of irregular stony knobs and ridges comprised of tills (with some sand and gravel deposits) and pitted sand and gravel terraces and swampy valley floors. The typical bedrock types in this region are limestone, dolomite, shale and gypsum of the Dundee formation.



3. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, site reconnaissance of the Kippen River Bridge #2 was carried out on May 10, 2014. A photographic record of the site visit is attached in Appendix A.

At the time of the site reconnaissance, the depth of the river was about 0.5 to 0.7 m. Concrete cracking and spalling were observed on the wingwalls adjacent to the abutment walls (Photographs 1 to 4). Surface cracks with wall surface deterioration and visible cracks were observed on the north and south abutment walls above the water level (Photographs 5 and 6). Concrete spalling at water level was observed on the south abutment wall (Photograph 6).

The embankments on either side of the bridge are steep. Erosion of adjacent earth slopes to the north and south abutments was observed. On the west slope adjacent to the north abutment, rock protection was placed to prevent further erosion of the slope surface (Photograph 7). In addition, scouring of the slope toe edges due to river water course were also observed near the abutment walls (Photographs 1 to 4). It is possible that the surface water running along the face of the slopes may have caused erosion of the slope surface. Observation below the water level could not been made and thus, potential river scouring effect below the water at the abutment walls by visual observations could not been made (Photographs 5 and 6). No visible signs of foundation settlement were observed.

No weep holes out of the abutment walls and no drainage pipes adjacent to the abutments were visible at the site. If any perforated pipes were installed behind the abutment walls, they were not visible to verify their conditions. Drainage condition at the bridge structure is considered poor.



4. INVESTIGATION PROCEDURES

The field work for this study was carried out on November 13, 2013. The investigation included two boreholes, with one borehole drilled at both the south and north abutment areas as shown on Drawing 1, appended.

The borehole locations were established in consultation with MMM. The borehole locations and elevations were surveyed in the field by MMM. All elevations in this report are expressed in metres.

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

Soil samples were recovered from the boreholes at regular 0.75 or 1.5 m depth intervals using the standard penetration test method. Standard penetration tests were conducted to assess the strength characteristics of the substrata. Soils classifications were identified in accordance with the MTO soil classification manual procedures.

The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved. The groundwater levels in the open boreholes during and following drilling were also obtained.

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

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

- Natural moisture content determinations (16)
- Grain size distribution analyses (6)
- Atterberg Limit Tests (3)



The results of the laboratory grain size distribution analysis and Atterberg Limit Tests are presented in Figures KR-GS-1 to KR-GS-5 and KR-PC-1 to KR-PC-2, respectively. All of the test results are summarized on the Record of Borehole sheets.

5. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test data and groundwater observations. The results of laboratory grain size distributions and 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 1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised existing Highway 4 pavement structure over cohesive fill to 3.4 to 4.3 m (locally the fill was non-cohesive in the lower portions of borehole 1), over a gravelly sand deposit to about 10 m, which was underlain by a very dense silt till that extended to the 12.3 m borehole termination depth. Cobbles and boulders were encountered in both boreholes within the gravelly deposit.

A summary of the findings is given below.

5.1 Fill

A 3.4 and 4.3 m thick layer of embankment fill was encountered surficially in boreholes 1 and 2 respectively that extended to elevation 266.0 and 267.0. Surficially the fill comprised the Highway 4 pavement structure, which included 150 and 360 mm of asphaltic concrete in boreholes 2 and 1 respectively, locally over 180 mm of Portland cement concrete in borehole 1, underlain by 100 and 300 mm of granular fill composed of sand and gravel. Beneath the pavement structure, clayey silt fill which transitioned to a sandy silt fill and silty clay fill were



encountered in boreholes 1 and 2, respectively. The fill was soft to firm and loose to compact, (SPT-'N' values of 3 to 26) and moist (moisture contents of 19 to 37%). Organic and topsoil inclusions were noted within the fill in both boreholes.

The results of three grain size distribution analyses and two Atterberg Limit Tests performed on samples of the fill are presented in Figures KR-GS-1 to KR-GS-3, and KR-PC-1 to KR-PC-2, respectively.

5.2 Sandy Silt

A 1.0 m thick sandy silt layer was contacted beneath the fill in borehole 1 at 4.3 m (elevation 266.0) that extended to 5.3 m (elevation 265.0). The sandy silt was compact (SPT 'N' value of 19) and moist (moisture content of 10%). The results of a grain size distribution analysis and Atterberg Limit Test completed on a sample this layer is presented on Figures KR-GS-4 and KR-PC-3, respectively.

5.3 Gravelly Sand

A 5.1 and 6.7 m thick gravelly sand deposit was contacted beneath sandy silt and fill at 5.3 and 3.4 m (elevation 265.0 and 267.0) in boreholes 1 and 2 respectively. The deposit extended to the silt till at 10.1 and 10.4 m (elevation 260.3 and 259.9) in boreholes 2 and 1, respectively. The deposit was compact to very dense (SPT-'N' values of 26 to 50 blows for 10 cm) and moist (moisture contents of 7 to 19%). Cobbles and boulders were noted within the deposit in both boreholes. The results of grain size distribution analyses completed on two samples of the deposit are shown on Figure KR-GS-5.



5.4 Silt Till

A 1.9 and 2.2 m thick silt till deposit was contacted underlying the gravelly sand to sand and gravel at 10.4 and 10.1 m (elevation 259.9 and 260.3) in boreholes 1 and 2, respectively. The silt till extended to the 12.3 m borehole termination depth in both boreholes, elevation 258.0 and 258.1. The material was very dense (SPT-'N' values of 65 blows for 15 cm to 100 blows for 10 cm) and moist (based on visual and tactile evidence).

5.5 Groundwater

In the process of augering, water strikes were observed at depths of 3.4 and 3.7 m (elevation 267.0 and 266.6) in boreholes 2 and 1 respectively. Upon completion of drilling groundwater was measured at 3.7 m in both boreholes. The groundwater levels at the site are subject to seasonal fluctuation and rainfall patterns.

The water level in the Bannockburn River was at the time of the investigation (November 13, 2013) was at elevation 267.8.



6. CLOSURE

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

This Foundation Investigation Report was prepared by Mr. A. DeSira, MEng, P.Eng., and reviewed by R. Ng, MBA, PhD, P.Eng. Mr. B. R. Gray, MEng, P. Eng., MTO Designated Principal Contact conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

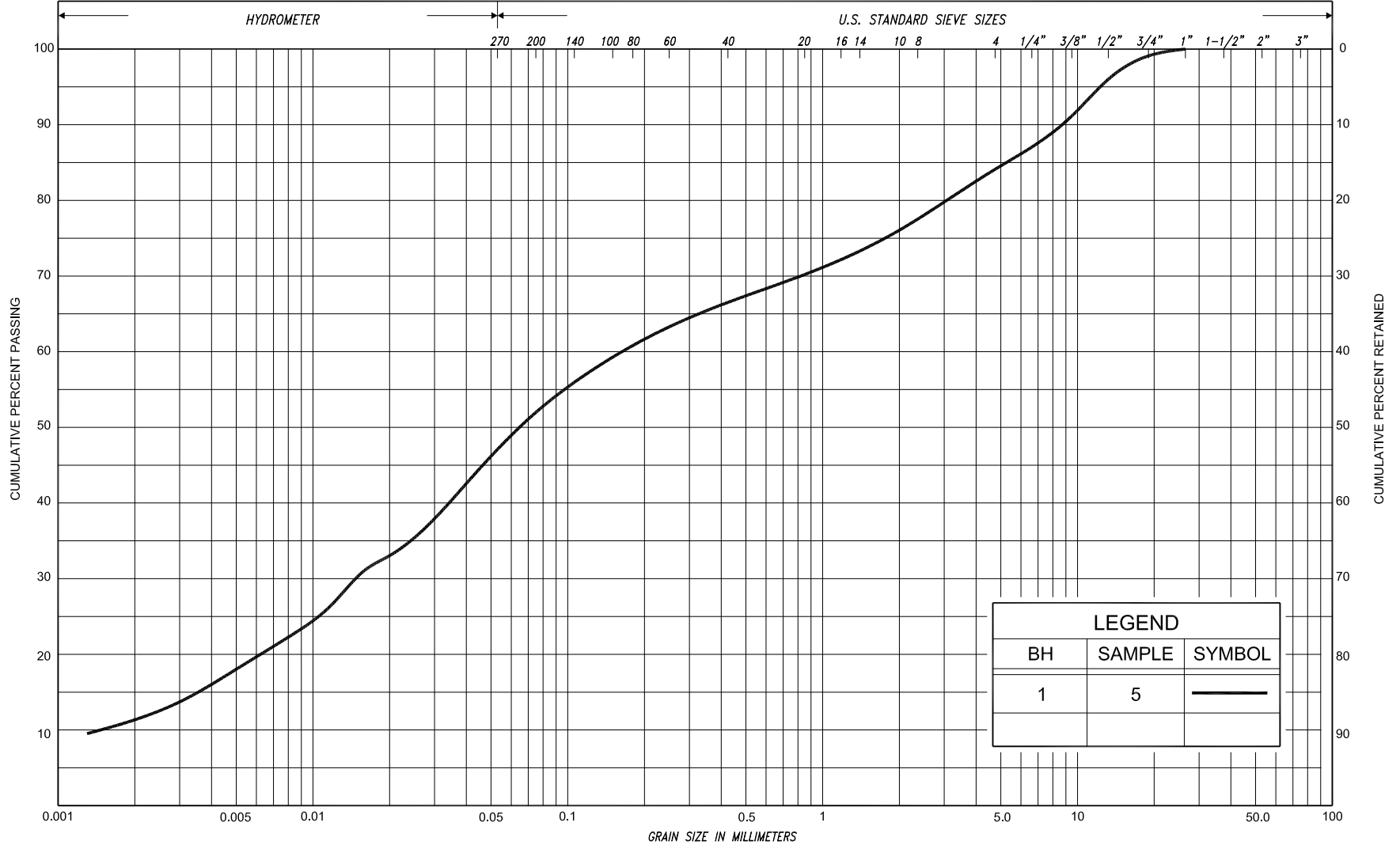


Robert Ng, MBA, PhD, P.Eng.
Senior Project Engineer



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

AD/CN/BRG:ad-mi

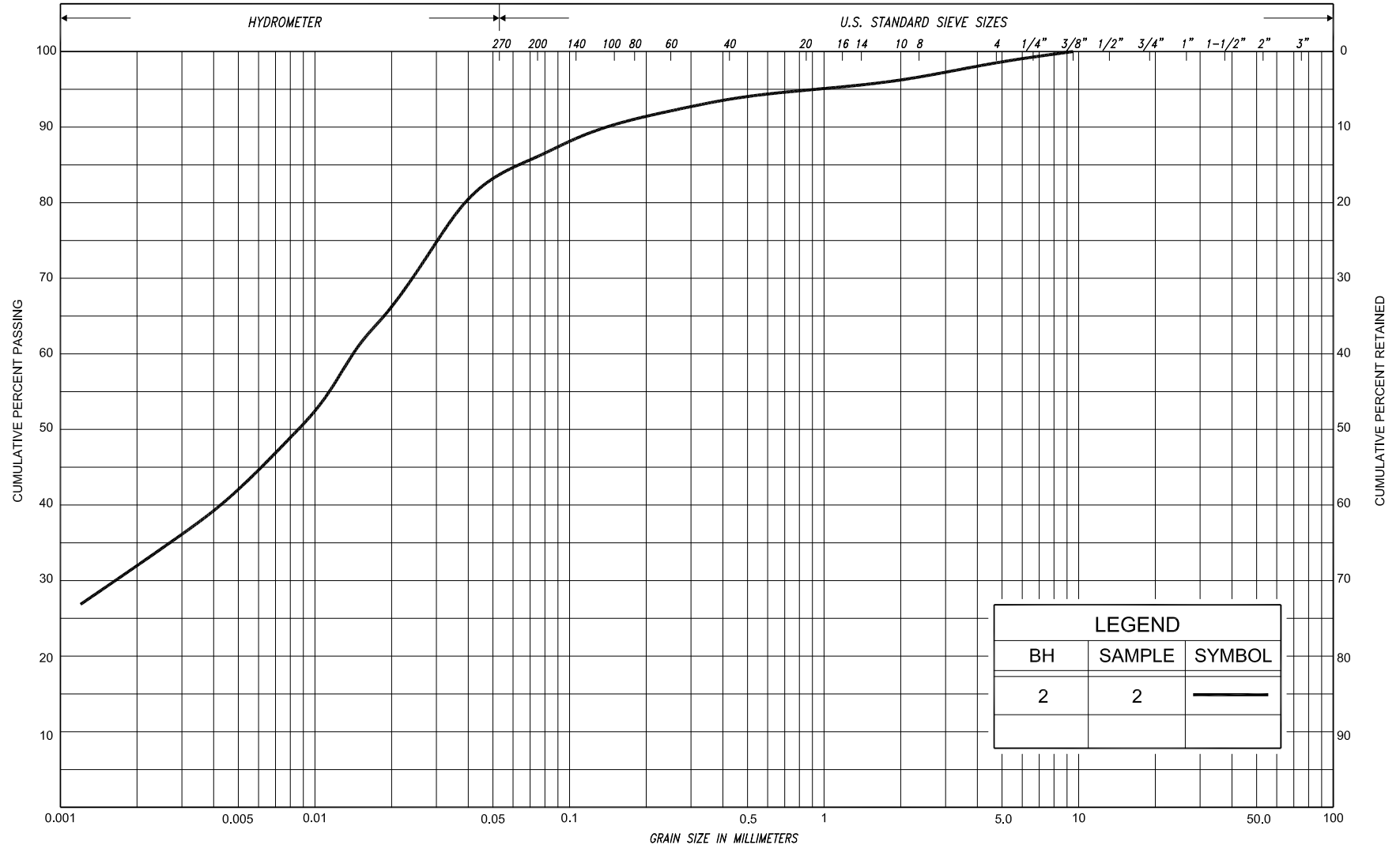


SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED
				SAND								
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL				COBBLES	M.I.T.
	SILT											
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL				U.S. BUREAU
				SAND								



GRAIN SIZE DISTRIBUTION
SANDY SILT, some clay, some gravel
(FILL)

FIG No.	KR-GS-2
HWY:	4
W.P. No.	3125-03-01



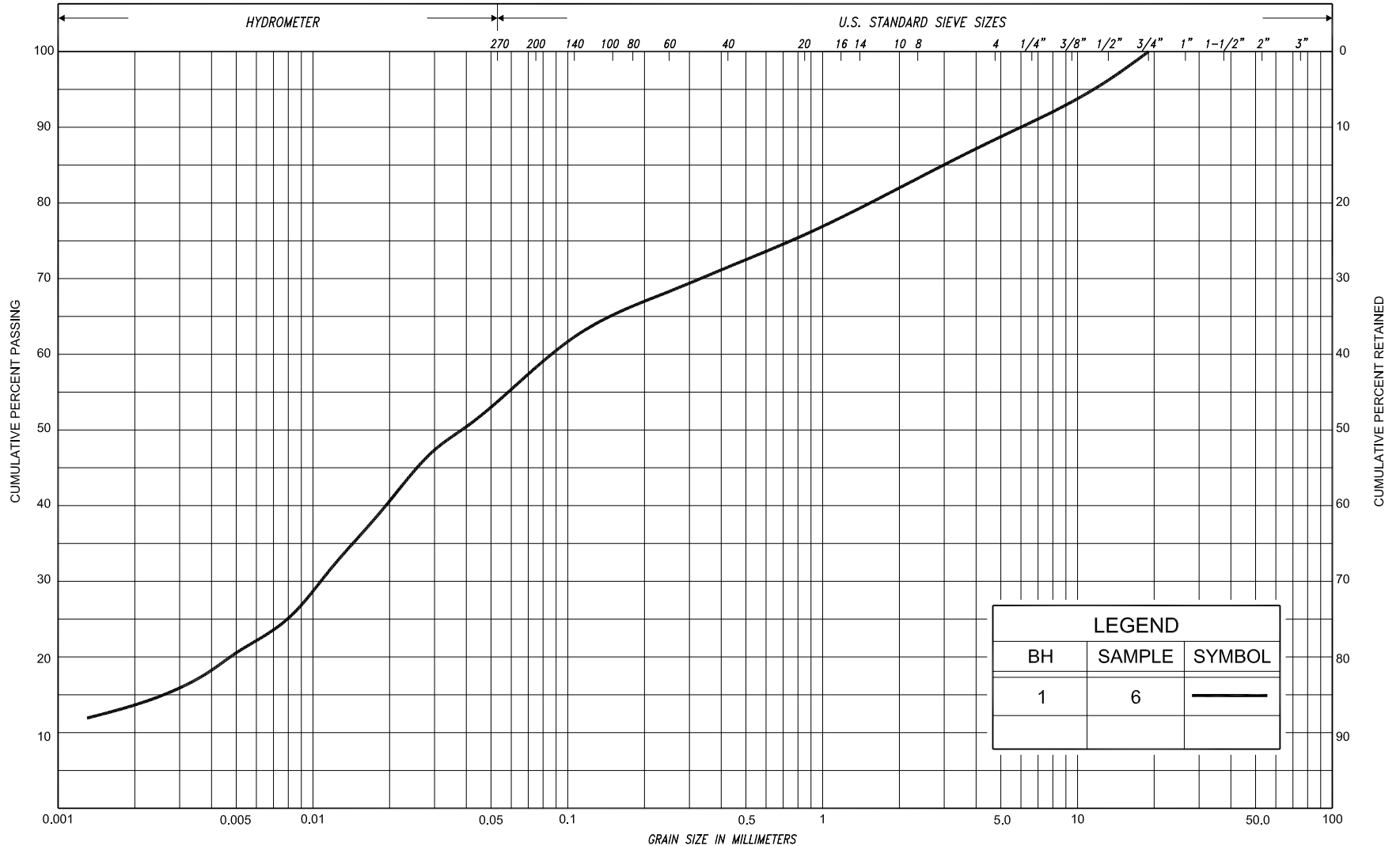
LEGEND		
BH	SAMPLE	SYMBOL
2	2	—

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



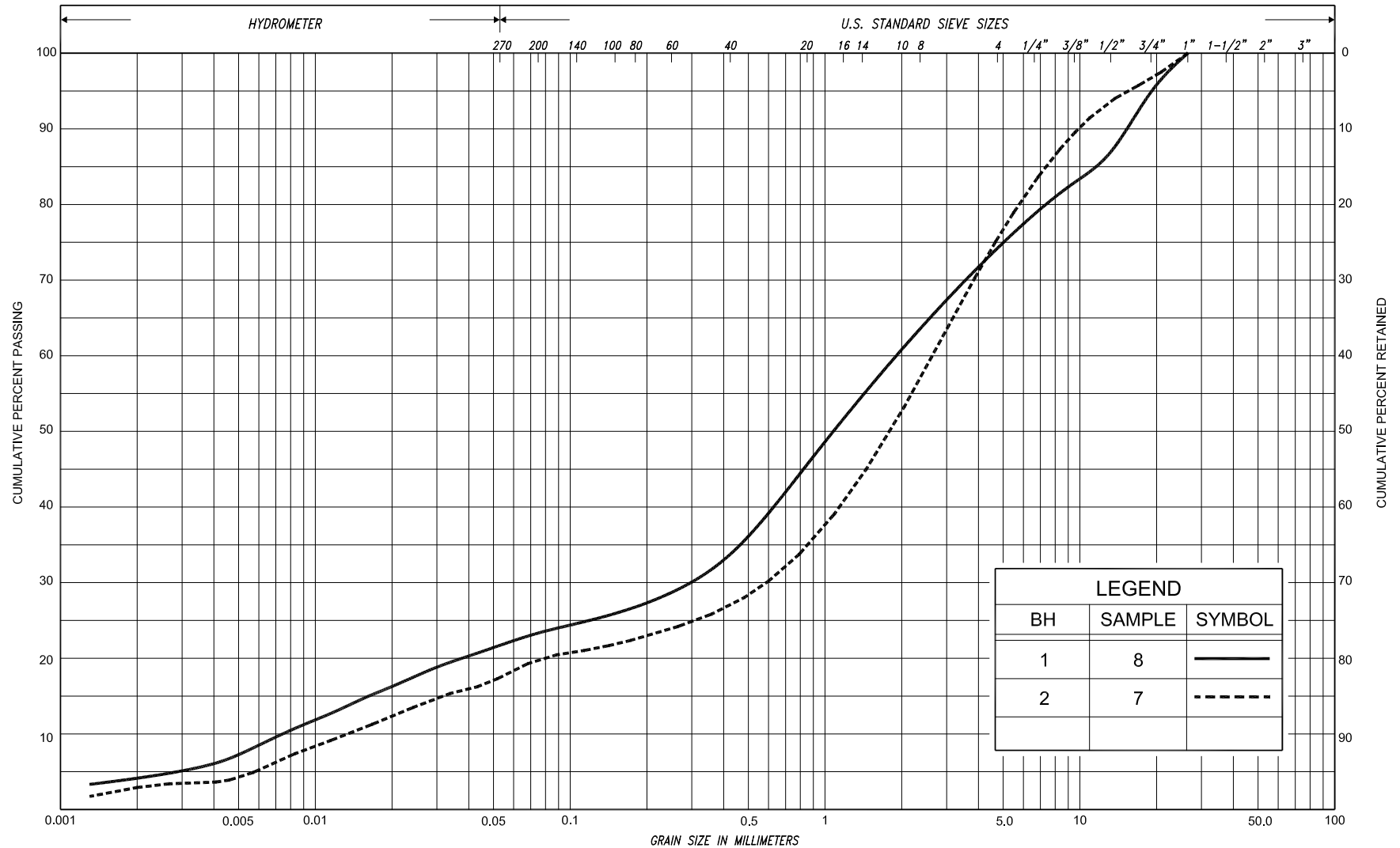
GRAIN SIZE DISTRIBUTION **SILTY CLAY, some sand, trace gravel (CI)** **(FILL)**

FIG No.	KR-GS-3
HWY:	4
W.P. No.	3125-03-01



LEGEND		
BH	SAMPLE	SYMBOL
1	6	

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									



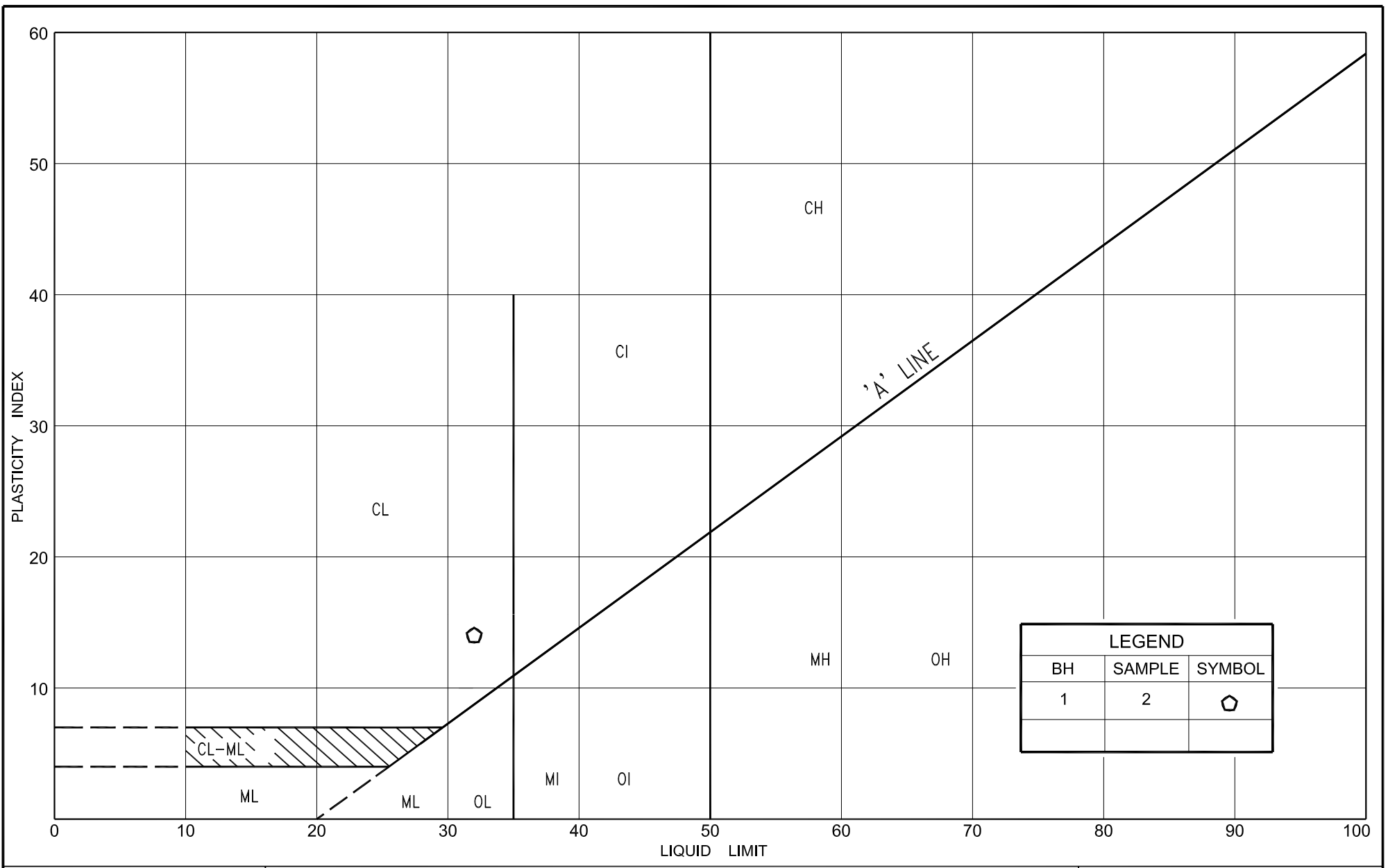
SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT																
CLAY			SILT			V. FINE		FINE		MED.		COARSE		GRAVEL			U.S. BUREAU
						SAND											



GRAIN SIZE DISTRIBUTION

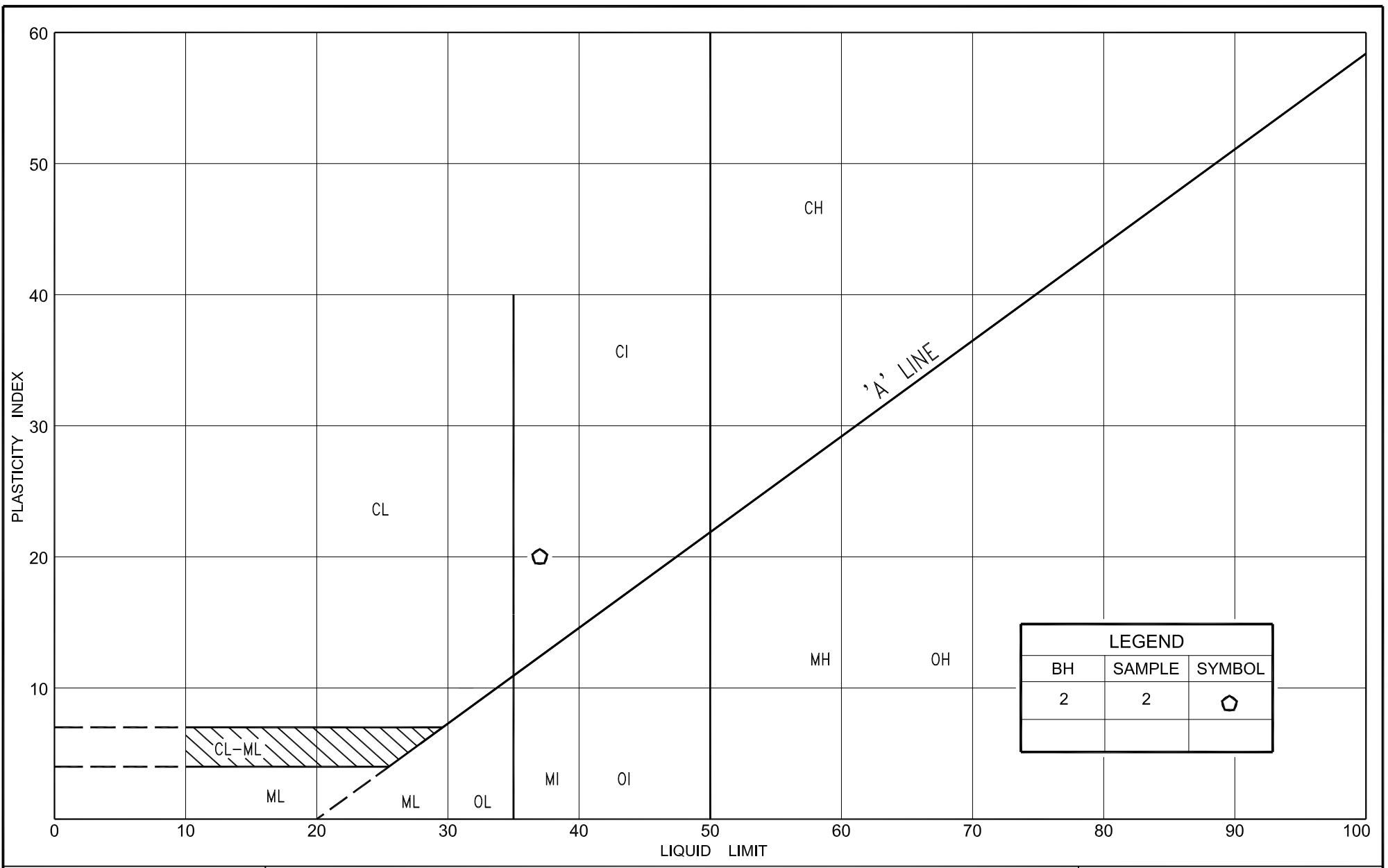
GRAVELLY SAND, some silt, trace clay

FIG No. KR-GS-5
 HWY: 4
 W.P. No. 3125-03-01



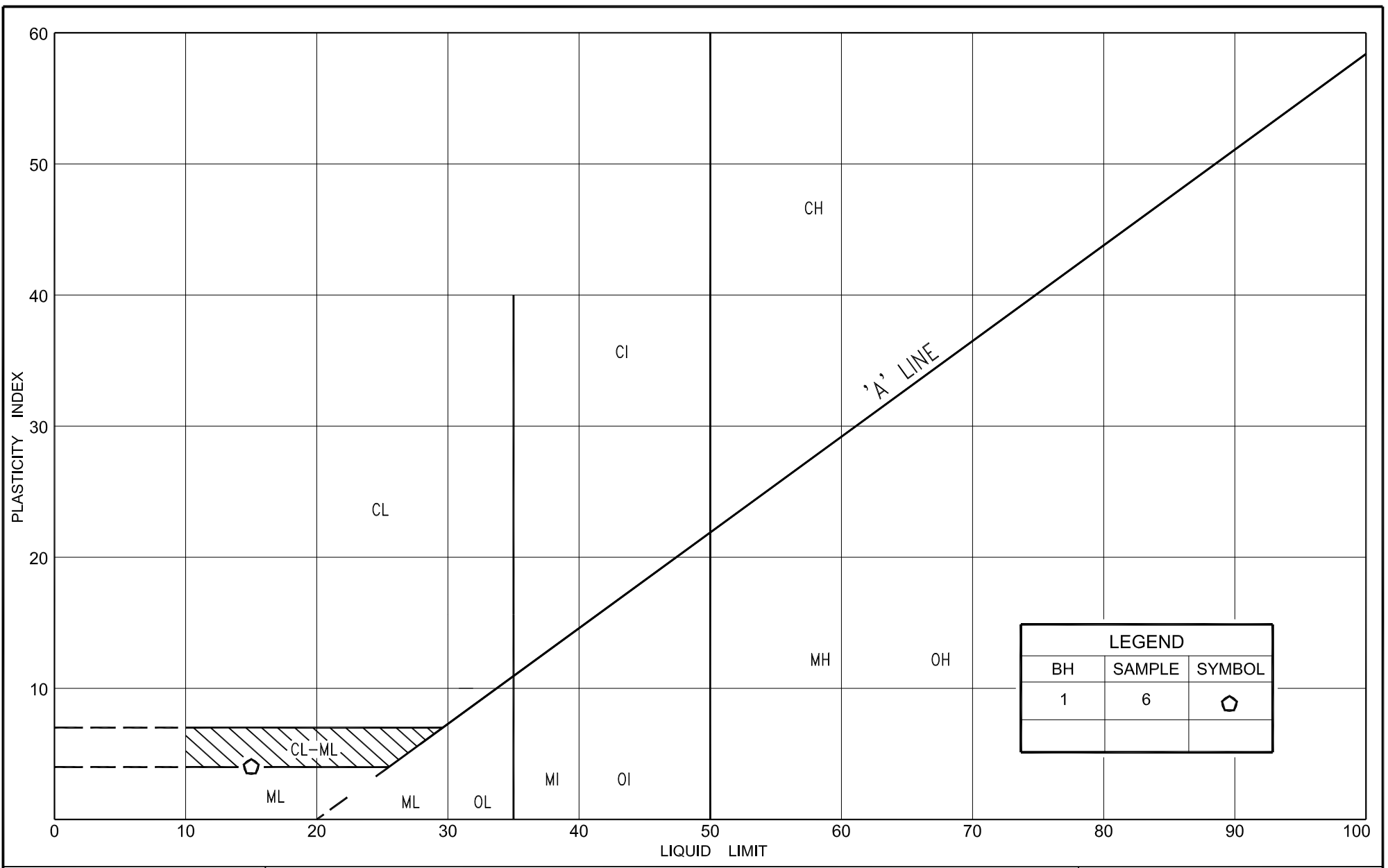
PLASTICITY CHART
 CLAYEY SILT, some sand, trace gravel (CL)
 (FILL)

FIG No.	KR-PC-1
HWY:	4
W.P. No.	3125-03-01



PLASTICITY CHART
 SILTY CLAY, some sand, trace gravel (CI)
 (FILL)

FIG No.	KR-PC-2
HWY:	4
W.P. No.	3125-03-01



PLASTICITY CHART

SANDY SILT, some clay, some gravel (ML)

FIG No.	KR-PC-3
HWY:	4
W.P. No.	3125-03-01

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 (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
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'_{v0}	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. 1

1 of 1

METRIC

W.P. 3125-03-01 **LOCATION** Coords: 4 814 521.5 N; 385 305.7 E **ORIGINATED BY** F.P.
DIST London **HWY** 4 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** A.D.
DATUM Geodetic **DATE** November 13, 2013 **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									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)												
270.3	Ground Surface						20	40	60	80	100						
0.0	360 mm asphalt over 180 mm concrete over 100 mm sand and gravel																
269.7	(PAVEMENT FILL)																
0.6	Clayey silt some sand, trace gravel organic inclusions		1	SS	3												
	Soft to Brown Moist firm		2	SS	5											5 19 49 27	
	Sandy silt some clay, some gravel		3	SS	9												
	Loose to Brown Moist compact (FILL)		4	SS	26												
			5	SS	19											16 32 40 12	
266.0	Sandy silt some clay, some gravel																
4.3	Compact Brown Moist		6	SS	19											12 30 45 13	
265.0	Gravelly sand some silt, trace clay cobbles and boulders		7	SS	39												
5.3	Compact to Grey Moist very dense		8	SS	28											26 51 19 4	
			9	SS	62												
			10	SS	50/15cm												
259.9	Silt, some sand some clay, trace gravel		11	SS	50/8cm												
10.4	Very dense Grey Moist (TILL)																
			12	SS	100/10cm												
258.0	End of borehole																
12.3																	
	* 2013 11 13																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																

* 2013 11 13

▽ Water level observed
during drilling

▼ Water level measured
after drilling

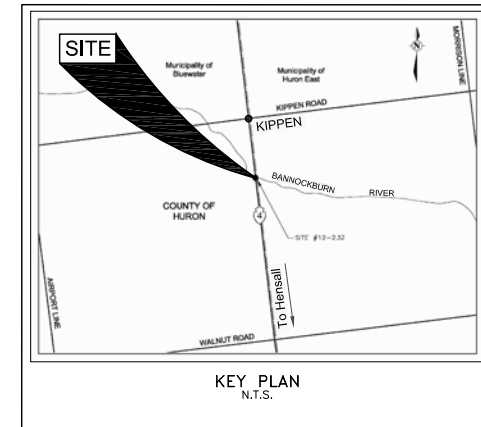
RECORD OF BOREHOLE No. 2

1 of 1

METRIC

W.P. 3125-03-01 **LOCATION** Coords: 4 814 488.4 N; 385 316.7 E **ORIGINATED BY** F.P.
DIST London **HWY** 4 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** A.D.
DATUM Geodetic **DATE** November 13, 2013 **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 (%)										
270.4	Ground Surface						20	40	60	80	100						
0.0	150 mm asphalt over 300 mm sand and gravel																
269.9	(PAVEMENT FILL)																
0.5	Silty clay some sand, trace gravel topsoil inclusions		1	SS	5												
	Soft to Dark Moist frim brown (FILL)		2	SS	4											2 12 53 33	
			3	SS	4												
			4	SS	11												
267.0	Gravelly sand some silt, trace clay		5	SS	26												
3.4	Compact to Grey Moist very dense		6	SS	33												
	wet sand layers		7	SS	40											25 55 17 3	
	cobbles and boulders		8	SS	43												
			9	SS	43												
			10	SS	50/10cm												



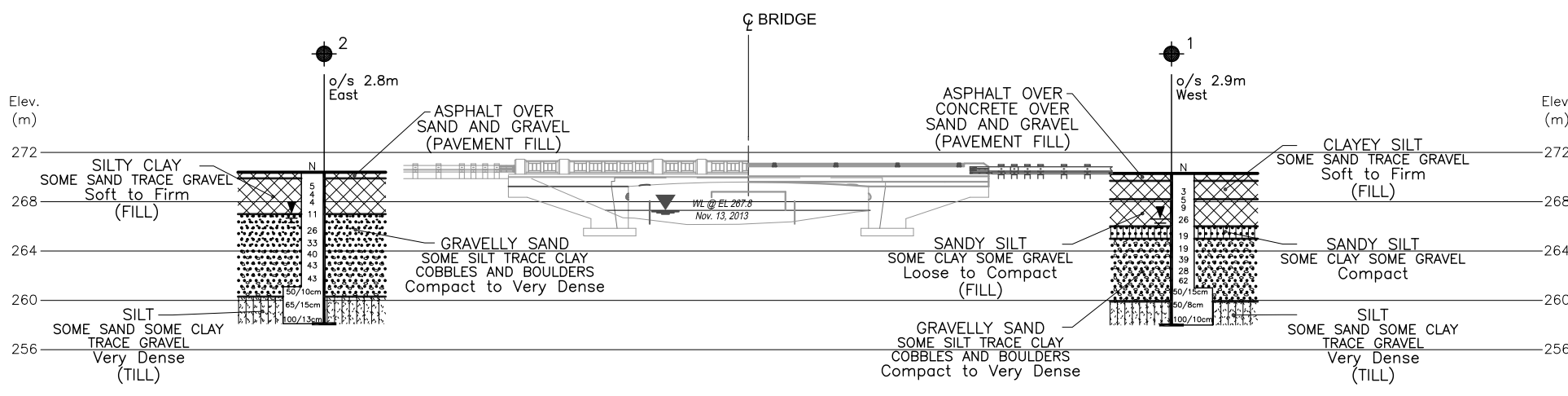
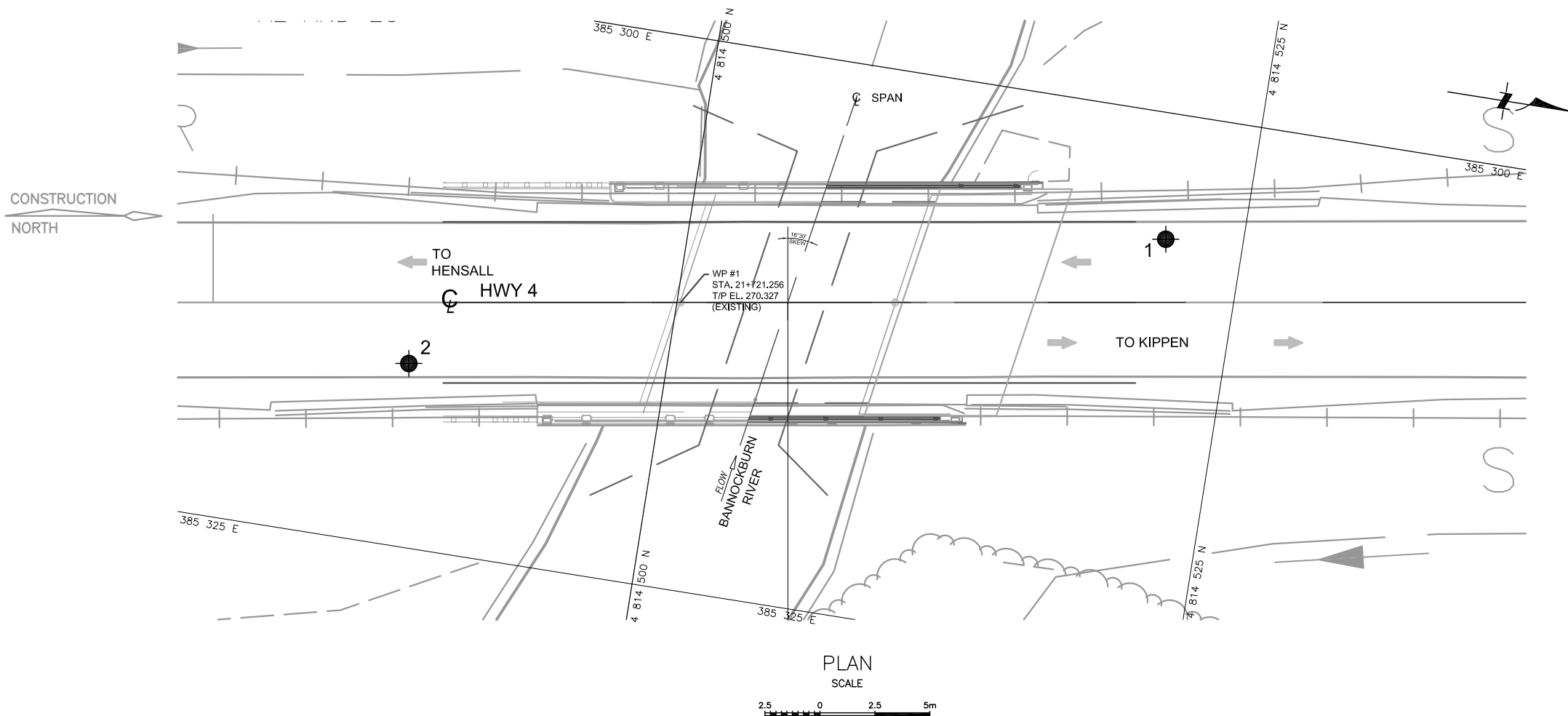
LEGEND				
	Borehole			
	Borehole and cone			
	Cone penetration test			
N	Blows/0.3m (Std. Pen Test, 475 J/blow)			
CONE	Blows/0.3m (60° Cone, 475 J/blow)			
	W L at time of investigation Nov. 2013			
	Head			
	ARTESIAN WATER			
	Encountered			
	PIEZOMETER			
BH No		ELEVATION	COORDINATES	
			NORTHINGS	EASTINGS
1		270.3	4 814 521.5	385 305.7
2		270.4	4 814 488.4	385 316.7

— 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. 40P5-20

HWY No	4			DIST	LONDON
SUBMD	NA	CHECKED	AD	DATE	NOV. 26, 2014
DRAWN	NA	CHECKED	RN	APPROVED	BRG
					DWG 1



PROFILE ALONG Q HIGHWAY 4
SCALE
HORIZONTAL
VERTICAL

- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH RECORD OF BOREHOLES AND REPORT
 - 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 No. MRC Drawing: S3813-001-001GA.dwg; dated April 2014



APPENDIX A

Site Photographs



Photograph 1: Looking north at the adjacent east slope of the north abutment from the east slope of the south side of the bridge. Cracks were observed on the wingwall. Slight scouring of the slope toe edge was observed. (May 10, 2014)



Photograph 2: Looking at the east slope adjacent to the south abutment. Scouring of the slope toe edge and erosion of the slope surface were observed. (May 10, 2014)



Photograph 3: Looking at the west slope adjacent to the north abutment. Scouring of the slope toe edge was observed. Concrete deterioration on wingwall was also observed. (May 10, 2014)



Photograph 4: Looking at the east slope adjacent to the south abutment. Scouring of the slope toe edge was observed. Concrete deterioration on east wingwall of the south abutment was also observed. (May 10, 2014)



Photograph 5: Looking north from the west slope of the south side of the bridge. Minor surface cracks, wall surface deterioration and a visible crack on the north abutment wall were observed. Potential scouring effect could not be verified under the water at the abutment wall. (May 10, 2014)



Photograph 6: Looking south from the west slope of the north side of the bridge. Minor surface cracks, wall surface deterioration and a visible crack on the south abutment wall were observed. Concrete spillings off the wall at the water level was also observed. Potential scouring effect could not be verified under the water at the abutment wall. (May 10, 2014)



Photograph 7: Looking north at the west slope adjacent to the north abutment. Rock protection placed on the slope to prevent slope surface erosion. (May 10, 2014)



**PRELIMINARY FOUNDATION DESIGN REPORT
for
HIGHWAY 4, KIPPEN RIVER BRIDGE #2
MTO WEST REGION 59 STRUCTURE REHABILITATIONS
CONTRACT 2
GWP 3125-03-01
TOWNSHIP OF TUCKERSMITH, HURON COUNTY, ONTARIO**

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- 3 cc: MMM Group Limited (MMM) for distribution to MTO
Project Manager – West Region (London)
+ 1 digital copy (pdf)
- 3 cc: Foundation Investigation Report only to MMM for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 1 cc: MMM for distribution to MTO, Pavements and
Foundations Section + digital copy (pdf) and
Drawing (AutoCAD)
- 1 cc: Foundation Investigation Report only to MMM for
distribution to MTO, Pavements and Foundations
Section + 1 digital copy (pdf) and Drawing (AutoCAD)
- 2 cc: MMM + 1 digital copy
- 1 cc: PML Toronto
- 1 cc: PML Kitchener

PML Ref.: 13KF006B
Index No.: 142FDR
GEOCRES No: 40P5-20
November 26, 2014



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Table 1 – List of Standard Specifications Referenced in Report
Non-Standard Specific Provision (NSSP)

General Arrangement Drawing May 28, 1947

PRELIMINARY FOUNDATION DESIGN REPORT

for

Highway 4, Kippen River Bridge #2
MTO West Region 59 Structure Rehabilitations
Contract 2, GWP 3125-03-01
Township of Tuckersmith, Huron County, Ontario

1. INTRODUCTION

This report provides preliminary foundation engineering analysis and recommendations for the proposed rehabilitation of the Highway 4 Kippen River Bridge #2 over the Bannockburn River near the Town of Kippen, Ontario. The proposed rehabilitation is a part of the assignment for the rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. The study was carried out by Peto MacCallum Ltd. (PML) for MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation of Ontario (MTO).

The purpose of this report is to provide foundation design recommendations for the existing structure based on the subsurface conditions revealed in the boreholes and available reports at the site. Further, this report will provide recommendations pertaining to proposed rehabilitations referenced in the General Arrangement drawing received from MMM in the email dated April 1, 2014.

In summary, the two boreholes at the site revealed existing Highway 4 pavement structure over a cohesive fill to 3.4 to 4.3 m (locally the fill was non-cohesive in the lower portions of borehole 1), over a compact to very dense gravelly sand to sand and gravel deposit to about 10 m, which was underlain by a very dense silt till that extended to the 12.3 m borehole termination depth. Cobbles and boulders were encountered in both boreholes within the gravelly deposit.

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



2. ENGINEERING DISCUSSION

2.1 Existing Foundations

Based on the general arrangement drawing "*Proposed Bridge*" from May 28, 1947 (attached) it is understood that the existing structure is supported on spread footings that are approximately 2.1 m wide and 0.6 m thick. These footings are founded approximately 5.0 m below the existing bridge deck, near elevation 265.3.

Based on the subsurface conditions revealed within the boreholes, it is considered that the spread footings are likely founded on dense gravelly sand.

Footings founded on the dense gravelly sand, which was found at the approximate founding depth of the existing structure, elevation 265.3, are considered to be capable of providing the following resistance values:

Factored Geotechnical Resistance at ULS, kPa	= 400
Geotechnical Reaction at SLS, kPa	= 300

The resistance at SLS normally allows for 25 mm of compression of the founding medium. Differential settlement is expected to be less than 75% of this value.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the Canadian Highway Bridge Design Code.

2.2 Rehabilitations

Based on the General Arrangement drawing of the proposed rehabilitation it is understood that the rehabilitation will typically be localized to the bridge superstructure (placement of new asphalt, construction of new parapet walls, modifications to the existing guard rails, removal and replacement of deteriorated concrete etc.) with the exception being the placement of a 450 mm



thick layer of class I rock protection in the river that will extend 2.0 m from the face of the abutments.

As per the Minutes of Meeting Report, dated May 5, 2012, it is understood that an assessment for a Permit to Take water (PTTW) to place in-water rock protection will be evaluated by the Prime Consultant.

Although only minimal excavations are anticipated, should temporary roadway protection be required during the rehabilitation of the Kippen River Bridge, the temporary protection should be designed and constructed in accordance with OPSS 539. A minimum performance level of 2, according to OPSS 539 is recommended. The contractor is responsible for selection, preparation of a detailed design and performance for the roadway protection system. The presence of cobbles and boulders within the gravelly sand and the possibility of the granular fill migrating through the temporary protection should be considered during detailed design. A NSSP shall be included in the contract documents advising the Contractor that presence of cobbles and boulders should be considered during the selection, design and performance for the roadway protection system.

The contractor should be responsible for lowering the prevailing groundwater elevation at the time of construction a minimum of 0.3 m below excavation bases. Since only minimal excavations are expected, dewatering should be relatively straight forward and it is anticipated that groundwater can be controlled through conventional sump pumping.

Provided adequate groundwater control is achieved, the existing fill material at the site is classified as Type 3 soils according to Occupational Health and Safety Act, 1990 and Regulation 213/1991 for Construction Projects. Excavations in Type 3 soil should be cut back at an inclination of 1H:1V from the base of the excavation.



3. CLOSURE

This Foundation Design Report was prepared by Mr. A. DeSira, MEng, and reviewed by R. Ng, MBA, PhD, P.Eng. Mr. B. R. Gray, MEng, P. Eng., MTO Designated Principal Contact conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



Robert Ng, MBA, PhD, P.Eng.
Senior Project Engineer



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

AD/CN/BRG:ad-mi



TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 539	Construction Specification for Temporary Protection Systems

NON-STANDARD SPECIAL PROVISION (NSSP)

NSSP - Construction Specification for Temporary Protection Systems (Addition of OPSS 539)

The Contractor shall be advised that cobbles and boulders were encountered within the native soil and that the Contractor shall use methods and equipment that will enable the design and construction of the temporary roadway protection system. The Contractor is responsible for selection, preparation of a detailed design and performance for the roadway protection system.

GENERAL ARRANGEMENT DRAWING MAY 28, 1947

