



## FOUNDATION INVESTIGATION AND DESIGN REPORT

for

**MAKYNEN ROAD SOUTHBOUND OVERPASS**

**WP 5046-00-01, SITE 46-493S**

**HIGHWAY 69, DISTRICT 54**

**SUDBURY**

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- PML Ref.: 03TF012C-S  
Index No.: 169FIR and 170FDR  
Geocres No.: 41I-191  
March 31, 2005



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for

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## **FOUNDATION INVESTIGATION REPORT**

for  
Makynen Road Southbound Overpass  
WP 5046-00-01, Site 46-493S  
Highway 69, District 54  
Sudbury

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### **1. INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed construction of an overpass to carry southbound traffic of Highway 69 on a new alignment over Makynen Road some 15 km south of Sudbury, Ontario. The investigation was conducted for Totten Sims Hubicki Associates (TSH) on behalf of the Ontario Ministry of Transportation.

The centreline of the overpass is at approximate Station 19+783, Highway 69 median chainage, in the Township of Burwash (ref. Drawing 1 'Hwy 69 (SBL). Makynen Road Overpass. Preliminary General Arrangement' prepared by TSH in February 2004).

The report provides subsurface information pertaining to the proposed structure and approach embankments within about 20 m of the abutments.

### **2. SITE DESCRIPTION AND GEOLOGY**

The site is situated on the new Highway 69 alignment at the crossing of Makynen Road approximately 15 km south of Sudbury. The proposed structure will carry Highway 69 southbound traffic over Makynen Road. Highway 69 is designated as a south-north road. The alignment of the overpass is therefore considered to extend south-north even though the highway is actually oriented in the southeast-northwest direction at the structure location.

The site is situated in the area of the Precambrian Laurentian peneplane. The topography is irregular in detail and dotted with areas of wet ground separated by steep rock ridges. Pleistocene lacustrine/fluvial deposits and recent swamp sediments have been laid down in depressions and are probably associated with the Nipissing post-glacial stage of the Great Lakes. The native soils are typically represented by sand/silt and/or clay deposits.



Metasedimentary rocks of the Huronian Supergroup and gneisses of the Grenville Province underlay the alignment. The area has undergone considerable folding, intrusive activity, regional metamorphism and faulting. The bedrock is at various depths ranging from surface to over 35 m, with the overburden/bedrock interface exhibiting sharp elevation differences along the alignment of Highway 69.

### 3. INVESTIGATION PROCEDURES

The field work for this study was carried out on July 13 and 14, 2004 (with two boreholes drilled on April 8 and 23, 2003) and comprised 10 boreholes advanced to depths of 1.2 to 7.6 m at the locations shown on Drawing 493S-1, appended. Further details are given in the following table:

LOCATION	BOREHOLE No.	DEPTH, m		
		AUGER / CONE	ROCK CORE <sup>(1)</sup>	TOTAL
South Approach	493S-1	4.4	—	4.4
South Abutment	493S-2	4.4	—	4.4
	493S-3	4.4	3.2	7.6
	493S-4	4.0	—	4.0
	493S-5	5.3	—	5.3
North Abutment	493S-6	6.7	—	6.7
	493S-7	6.0	—	6.0
	493S-8	1.3	3.1	4.4
	493S-9	1.2	—	1.2
North Approach	493S-10	1.2	—	1.2

<sup>(1)</sup> NQ diamond rock coring equipment

The alignment of Makynen Road at the structure location was staked in the field by TSH. The positions of the boreholes along the staked alignment were selected by Peto MacCallum Ltd. (PML). The locations of and ground surface elevations at the boreholes were also determined by PML. The ground surface elevations at the borehole locations were referenced to the ground surface at Station 19+750, Highway 69 median chainage, 19 m right of the Highway 69 alignment (elevation 230.6 m).



The test holes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. One borehole at the centre of each abutment was extended at least 3 m into bedrock using NQ diamond rock coring equipment supplemented by NW wash boring techniques.

Representative soil samples were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. Penetrometer testing was also performed to assess the shear strength of the cohesive soils.

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. Upon completion of drilling, the boreholes were backfilled with a bentonite/cement mixture in accordance with the MTO guidelines for borehole abandonment procedures.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. In addition, four Atterberg limits tests and six grain size distribution analyses were carried out on selected soil samples, with the results presented in respective Figures PC-1, PC-2 and GS-1, GS-2 as well as on the corresponding Record of Borehole sheets.

#### **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 penetration resistance and penetrometer test data, groundwater observations, the results of laboratory Atterberg limits tests, grain size distribution analyses and moisture content determinations.



The borehole locations and stratigraphic profiles prepared from the borehole data are shown on the appended Drawings 493S-1 and 493S-2.

The subsurface stratigraphy revealed in the boreholes drilled at the abutments and approach embankments to the structure generally comprised a surficial peat deposit underlain by sandy/silty strata containing a discontinuous layer of clayey silt. Boulders were identified in the north approach borehole. Bedrock/probable bedrock was encountered at depths of 4.0 to 5.3 m (elevation 225.0 to 226.2) and 1.2 to 6.7 m (elevation 225.1 to 232.9) south and north of Makynen Road, respectively. The strata encountered are summarised below.

#### **4.1 Peat**

A surficial deposit of peat was present in all boreholes except at the north approach location. The peat was fine to coarse fibrous in texture and had a thickness of 100 to 200 mm.

#### **4.2 Sand/Silty Sand**

Directly beneath the peat at elevation 229.4 to 232.3 in boreholes 493S-1 to 493S-9 was cohesionless silty sand. This unit was also encountered at depths of 3.4 and 5.6 m (elevation 226.8 and 226.2) in boreholes 493S-4 and 493S-6. Cohesionless sand was present surficially in the north approach borehole and identified below the silty sand at 6.4 m depth (elevation 225.4) in borehole 493S-6. The thickness of the sandy soil layers ranged from 0.3 to 1.1 m, increasing to 2.5 m in borehole 493S-3. The sand/silty sand was typically very loose to loose (SPT-'N' values of 1 to 6) but dense at a depth of about 6.4 m in borehole 493S-6. The moisture content of the sandy soils was 15 to 27%

It is noteworthy that boulders were contained within the sand in borehole 493S-10. The sandy soils were penetrated at depths of 0.4 to 6.7 m (elevation 225.1 to 232.9).



### **4.3 Clayey Silt**

A discontinuous cohesive deposit of clayey silt was revealed below the silty sand at depths of 0.4 to 1.2 m (elevation 228.4 to 231.6) in boreholes 493S-1, 493S-2, 493S-4, 493S-6, 493S-8 and below silt at 1.8 m depth (elevation 228.5) in borehole 493S-5. This deposit was 0.7 to 3.3 m thick and had a moisture content of about 27%. The clayey silt was very soft to very stiff, typically stiff in consistency. The unconfined compressive strength determined in two penetrometer tests varied between 150 and 325 kPa which corresponds to an undrained shear strength of 75 and about 160 kPa.

The results of Atterberg limits testing and grain size distribution analyses conducted on representative samples of this material are presented in Figures PC-1 and GS-1 respectively. The liquid limit of the clayey silt ranged from 23 to 34 and plastic limit from 16 to 18, with a corresponding range in the plasticity index of 5 to 18.

The cohesive deposit was penetrated at depths of 1.3 to 4.4 m (elevation 225.7 to 230.7).

### **4.4 Silt/Sandy Silt**

Underlying the silty sand or clayey silt at depths of 1.1 to 2.6 m (elevation 227.2 to 230.7) in boreholes 493S-1, 493S-3, 493S-5 to 493S-7 was cohesionless silt/sandy silt. The thickness of this unit varied between 0.7 and 4.9 m. The silt/sandy silt was typically compact in relative density (SPT-'N' values of 9 to 23) but very loose in boreholes 493S-1 and 493S-3. The moisture content of the unit was about 30%.

The results of one Atterberg limits test and three grain size distribution analyses conducted on this material are presented in Figures PC-2 and GS-2 respectively. The silty soils were penetrated at depths of 1.8 to 6.0 m (elevation 225.0 to 228.5).





#### **4.5 Bedrock**

Bedrock/probable bedrock was contacted below the native soils at depths of 1.2 to 6.7 m (elevation 225.0 to 232.9). The bedrock comprises a light grey to white, becoming light grey to pink meta-sedimentary rock. A detailed description of the rock cores retrieved from boreholes 493S-3 and 493S-8 is given in Table 1.

The measured core recovery varied between 97 and 100%. The RQD determined from the rock cores was in a range of 73 to 93%, thus indicating a fair to excellent quality rock.

#### **4.6 Groundwater**

Groundwater was observed in six boreholes during or upon completion of drilling. Water was detected in boreholes 493S-1, 493S-2, 493S-4 to 493S-7 at depths of 1.2 to 2.7 m (elevation 227.2 to 229.2) in the process of augering. Upon completion of drilling, groundwater was measured in boreholes 493S-2, 493S-4 to 493S-7 to be at depths of 3.7 to 5.6 m (elevation 226.1 to 226.6). No water was observed in the remaining boreholes in the course of the field work.

Groundwater levels may fluctuate subject to seasonal variations and precipitation patterns.

### **5. CLOSURE**

The field work was carried out under the supervision of Mr. F. Portela, C.E.T., and direction of Mr. D.W. Kerr, MEng, P.Eng., Chief Foundation Engineer. The equipment was supplied by Marathon Drilling Co. Ltd. The laboratory tests were conducted at the Toronto laboratory of Peto MacCallum Ltd.



The report was prepared by Mr. Grigory O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. Dennis W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

A handwritten signature in dark ink, appearing to read 'Grigory O. Degil'.

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



A handwritten signature in blue ink, appearing to read 'Dennis W. Kerr'.

Dennis W. Kerr, MEng, P.Eng.  
Chief Foundation Engineer



A handwritten signature in blue ink, appearing to read 'Brian R. Gray'.

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



GD:gd-mm-mi



**TABLE 1**  
**ROCK CORE DESCRIPTION**

CORE RECOVERY					CORE DESCRIPTION	
HOLE No.	CORE No.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
493S-3	4	4.4 - 5.6	100	75	4.4 - 7.6	META-SEDIMENTARY: Mainly light grey to white with numerous dark bands, becoming light grey to pink with some dark bands, arkose-like, fine grained, high strength, unweathered, close to moderate spaced dipping partings, rough planar, tight, occ. with encrustation on parting, good to excellent quality.
	5	5.6 - 7.2	100	93		
	6	7.2 - 7.6	100	85		
493S-8	3	1.3 - 2.5	97	73	1.3 - 4.4	META-SEDIMENTARY: Light grey to white, fine grained, arkose-like, slight banding, occ. thin dipping layer of black biotite, high strength, unweathered, close to wide spaced flat to dipping partings, smooth to rough planar, tight to oxidized, fair to excellent quality.
	4	2.5 - 4.0	100	92		
	5	4.0 - 4.4	100	83		

RQD: Rock Quality Designation

Originated: FP  
 Compiled: JFW  
 Checked: CN



Photograph 1: Borehole 493S-3, rock cores 4 and 5 (top)



Photograph 2: Borehole 493S-3, rock cores 5 (bottom) and 6

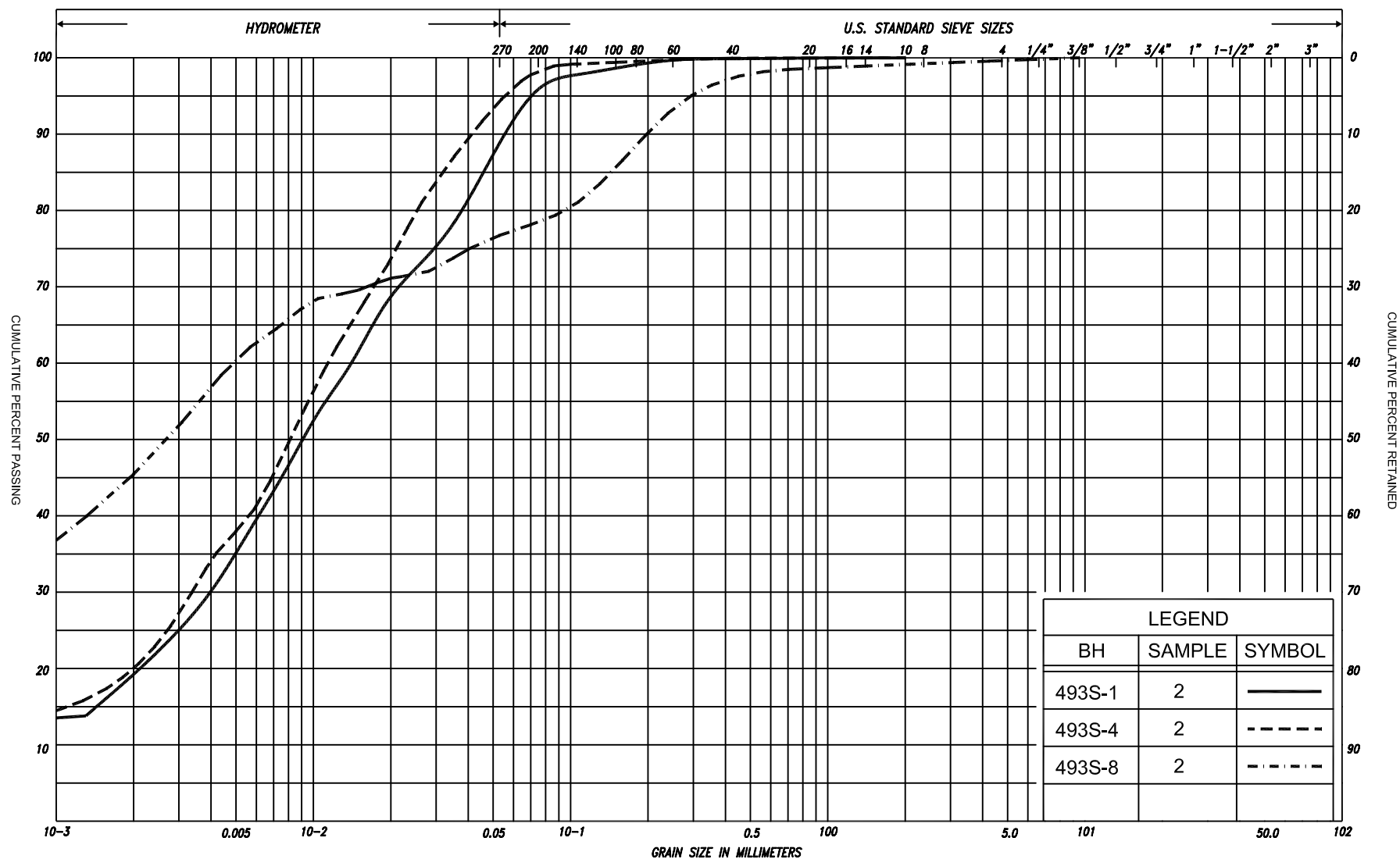




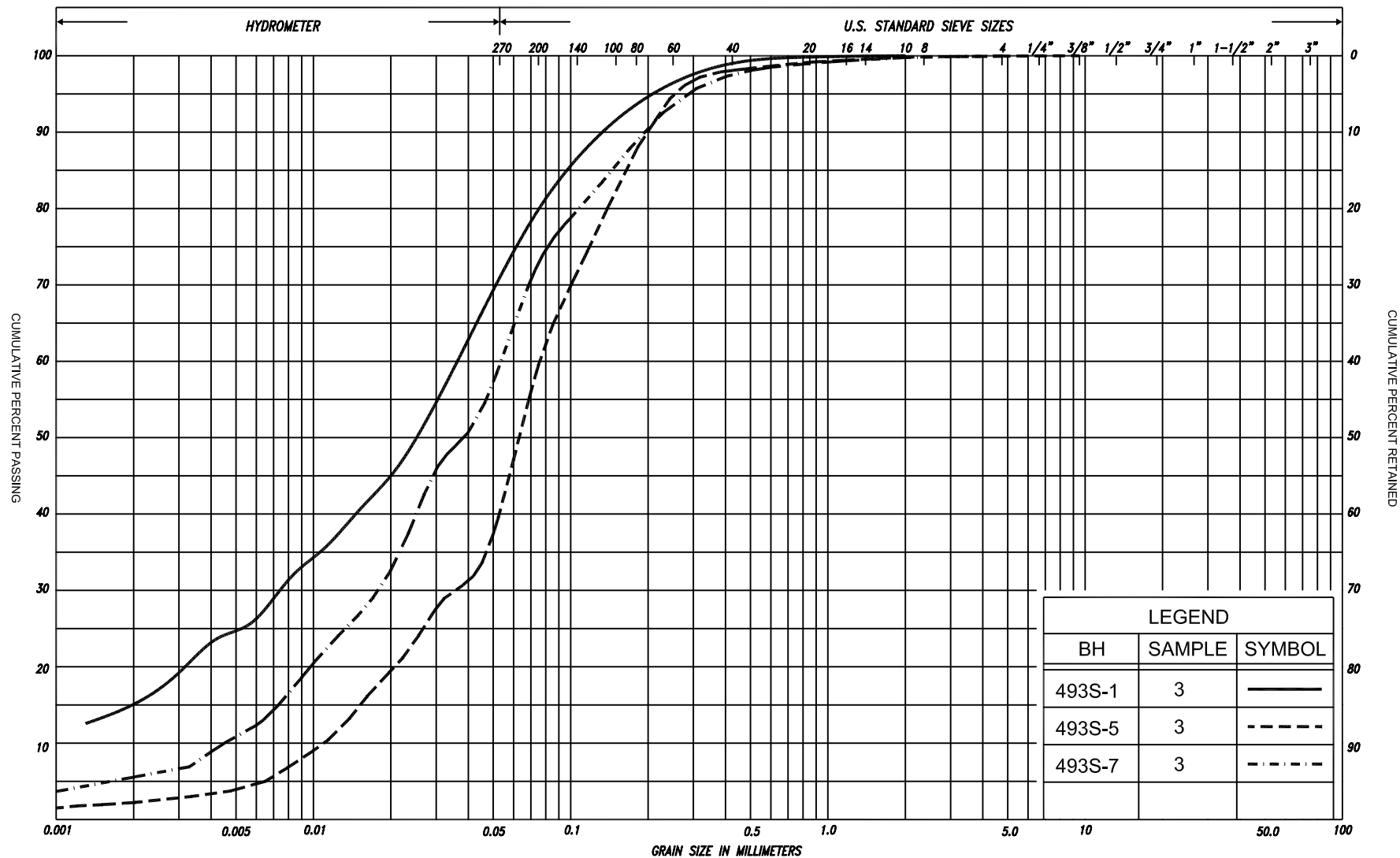
Photograph 3: Borehole 493S-8, rock cores 3 and 4 (top)



Photograph 4: Borehole 493S-8, rock cores 4 (bottom) and 5



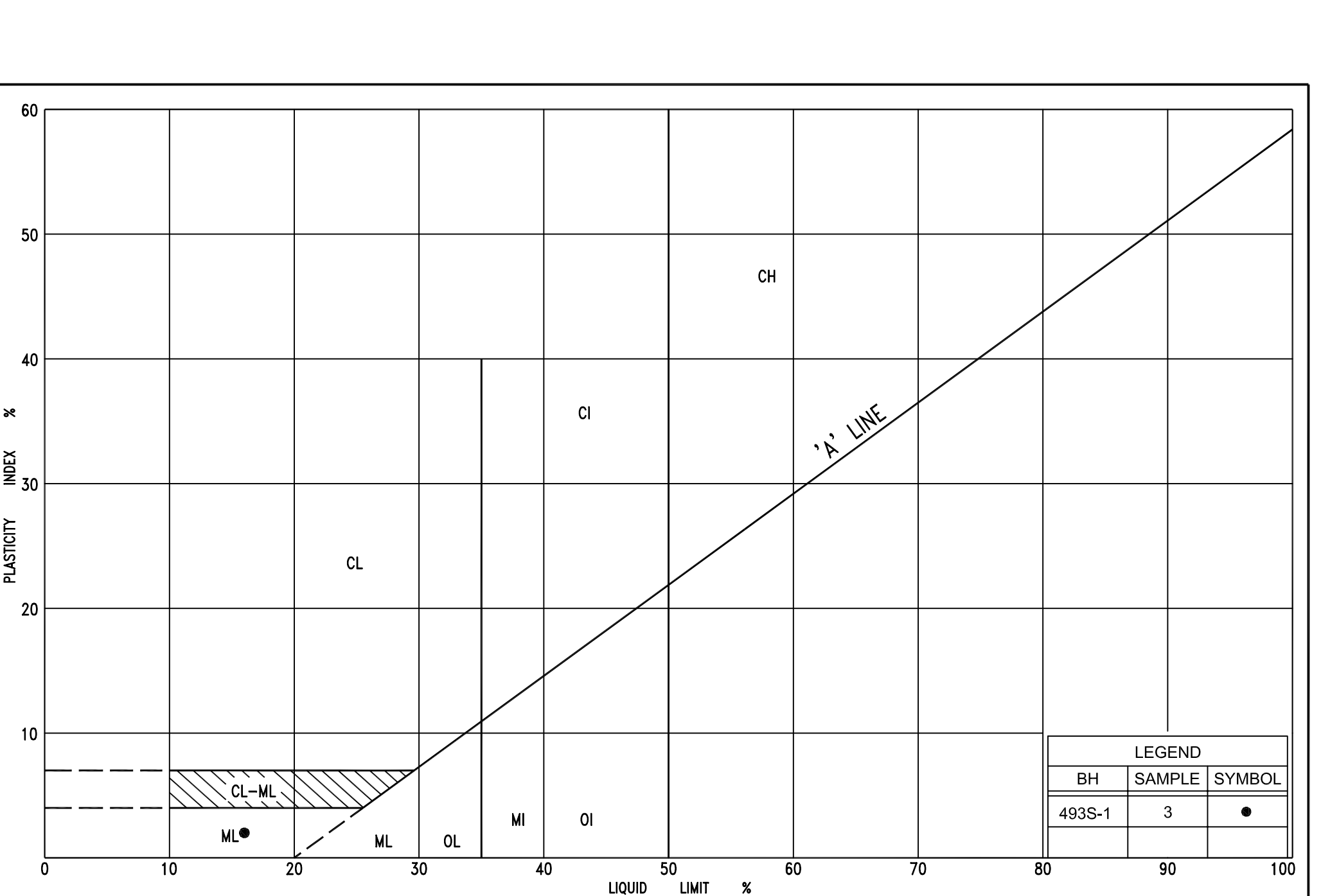
SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL				COB BLES	UNIFIED
CLAY	FINE		MEDIUM		COARSE		SAND					GRAVEL			COBBLES	
	SILT					FINE		MEDIUM		COARSE						
	CLAY			SILT			V. FINE	FINE	MED.	COARSE	GRAVEL					U.S. BUREAU
						SAND										



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







PLASTICITY CHART  
SILT

## 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.

**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:

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

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{\min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{KN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
$\rho_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{KN}/\text{m}^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$\text{KN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{KN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{\text{sat}}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{\text{sat}}$	$\text{KN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	$e_{\max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$\text{KN}/\text{m}^2$	SEEPAGE FORCE
$\gamma'$	$\text{KN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						

**METRIC**

(%) STRAIN AT FAILURE

**RECORD OF BOREHOLE No 493S-2**

1 of 1

**METRIC**

W.P. 5046-00-01 LOCATION Co-ord. 5 135 074 N; 318 937 E  
Hwy 69, Sta. 19+759, o/s 8m Lt. (CL SBL) ORIGINATED BY FP  
 DIST 54 HWY 69 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD  
 DATUM Geodetic DATE July 14, 2004 CHECKED BY \_\_\_\_\_

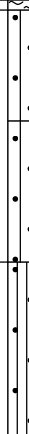

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE												
230.1	Ground Surface							20	40	60	80	100								
0.0	Peat, fine fibrous						230													
0.1	Dark brown Silty sand		1	SS	2															
	Very loose      Brown      Moist																			
229.0							229													
1.1	Clayey silt, with sandy silt and silt seams																			
	Stiff      Brown      Moist to very soft		2	SS	9		228													
	_____																			
	Grey      Wet						227													
			3	SS	1															
225.7							226													
4.4	End of borehole																			
	Refusal on probable bedrock																			

**RECORD OF BOREHOLE No 493S-3**

1 of 1

**METRIC**

W.P. 5046-00-01 LOCATION Co-ord. 5 135 079 N; 318 942 E ORIGINATED BY FP  
 DIST 54 HWY 69 BOREHOLE TYPE C.F.H.S.A. + Rock Coring COMPILED BY GD  
 DATUM Geodetic DATE July 14, 2004 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			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					w <sub>p</sub>	w	w <sub>L</sub>							
230.0	Ground Surface																					
0.0	Peat, fine fibrous		1	SS	1		229															
0.1	Dark brown Silty sand																					
	Very loose Brown Moist to loose																					
	with clayey silt lenses																					
227.4			2	SS	6				228													
2.6	Sandy silt, trace clay with clayey silt lenses										227											
	Very loose Grey Wet																					
225.6							226															
4.4	Bedrock						225															
	Meta-sedimentary																					
	High strength																					
	Good to excellent quality																					
			4	RC NQ	REC 100%				224													

**METRIC**

CHECKED BY \_\_\_\_\_

20  
15 — 5 (%) STRAIN AT FAILURE  
10

**METRIC**

20  
15 — 5 (%) STRAIN AT FAILURE  
10

**RECORD OF BOREHOLE No 493S-6**

1 of 1

**METRIC**

W.P. 5046-00-01 LOCATION Co-ord. 5 135 104 N; 318 918 E ORIGINATED BY FP  
 DIST 54 HWY 69 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD  
 DATUM Geodetic DATE July 14, 2004 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	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												
231.8	Ground Surface																			
0.0	Peat, coarse fibrous																			
0.1	Dark brown Silty sand		1	SS	2															
230.7	Very loose Brown Moist						231													
1.1	Clayey silt																			
	Very stiff Brown Moist to wet		2	SS	21		230				162									
229.2	Sandy silt																			
2.6	Loose Brown Wet to compact		3	SS	9		229													
							228													
			4	SS	23		227													
226.2																				
5.6	Silty fine sand						226													
	Dense Brown Wet																			
225.4			5	SS	37															
6.4	Sand, trace silt																			
225.1	Dense Brown Wet to grey																			
6.7	End of borehole																			
	Refusal on probable bedrock																			



**METRIC**

20  
15 — 5 (%) STRAIN AT FAILURE  
10

**METRIC**[illegible]

**RECORD OF BOREHOLE No 493S-9**

1 of 1

**METRIC**

W.P. 5046-00-01 LOCATION Co-ord. 5 135 116 N; 318 916 E  
Hwy 69, Sta. 19+803, o/s 8m Rt. (CL SBL) ORIGINATED BY FP  
 DIST 54 HWY 69 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD  
 DATUM Geodetic DATE July 14, 2004 CHECKED BY \_\_\_\_\_


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
232.4	Ground Surface																
0.0	Peat, fine fibrous		1	SS	1		232										
0.1	Dark brown Silty sand, with rootlets																
231.2	Very Brown Moist loose to loose																
1.2	End of borehole Refusal on probable bedrock																
	* Borehole dry on completion of drilling																

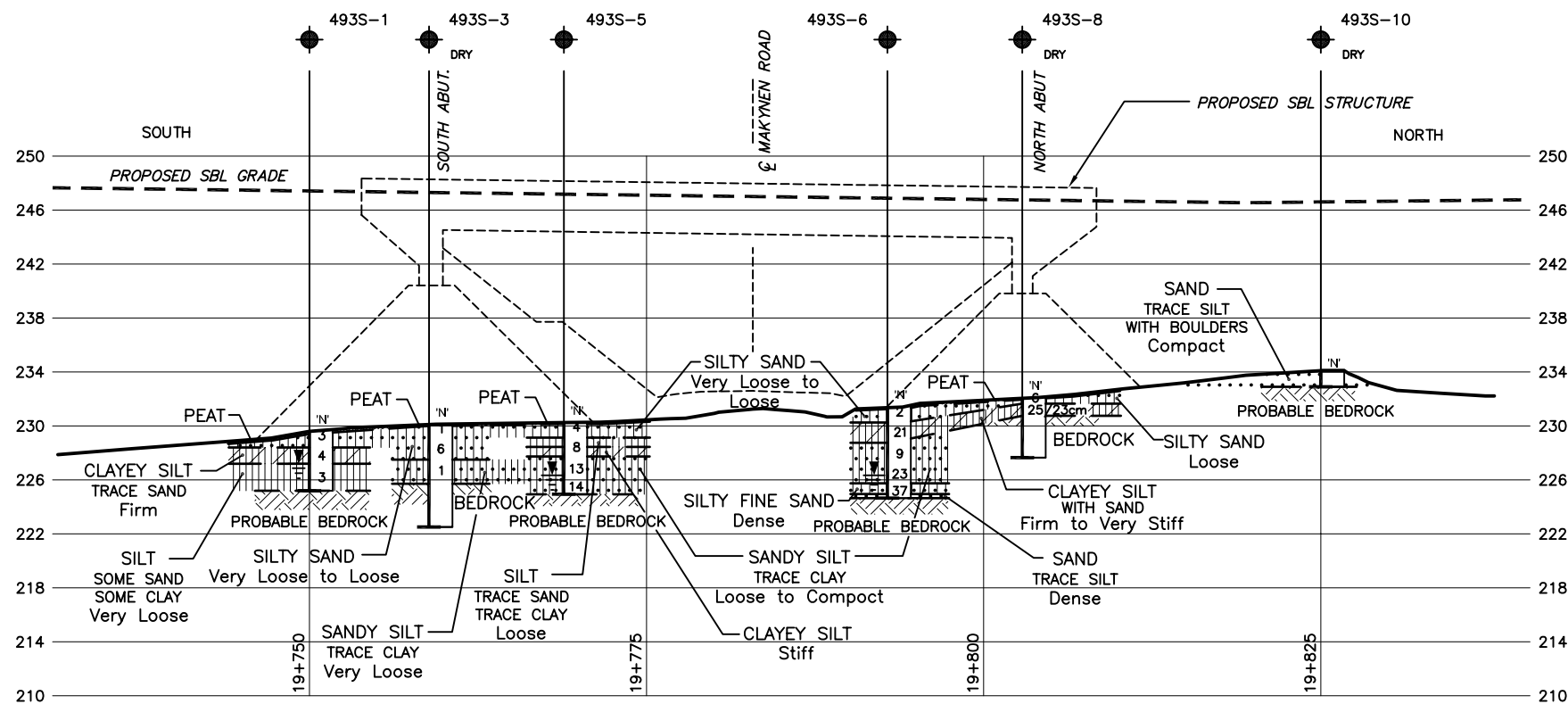
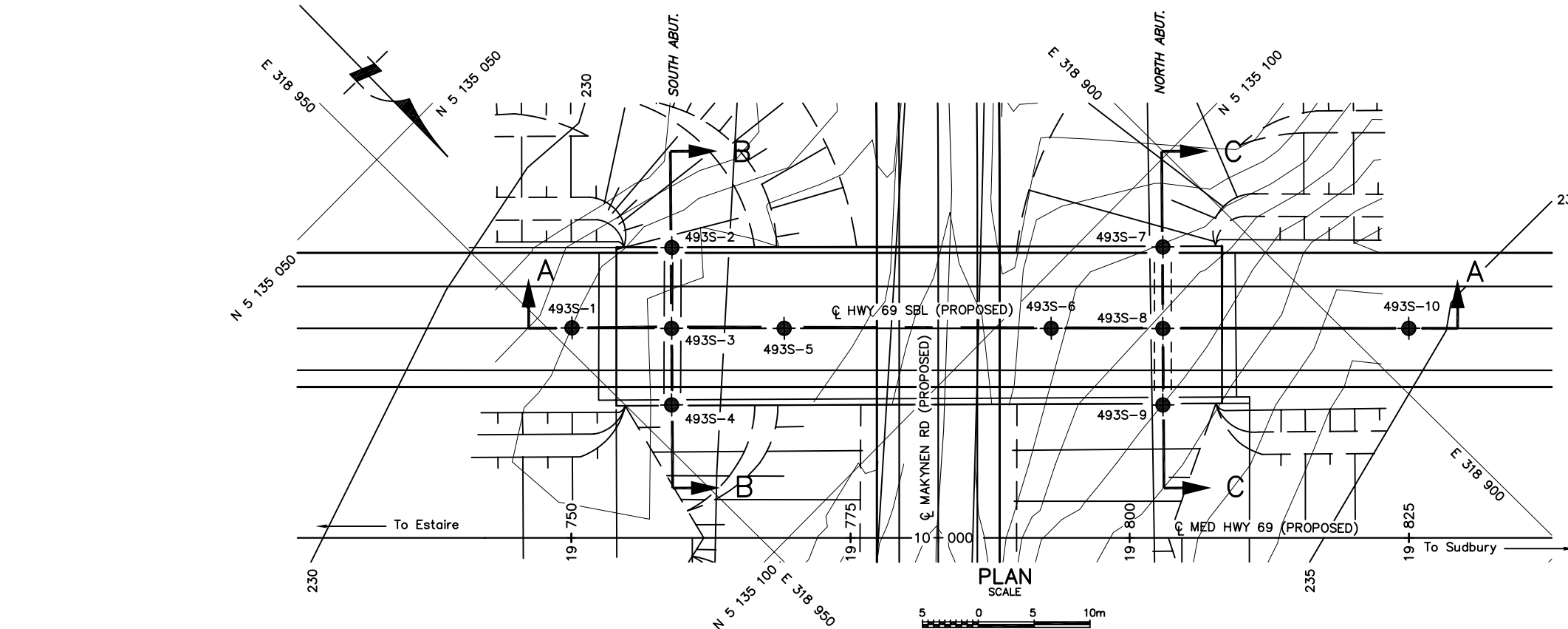
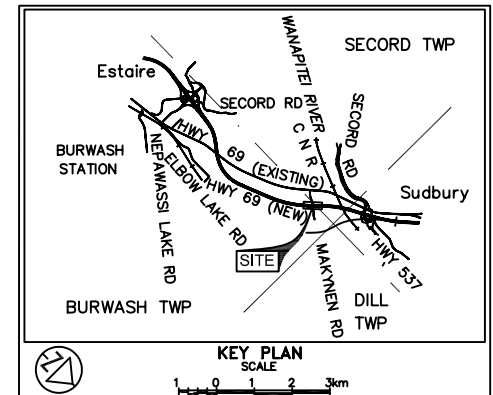
**RECORD OF BOREHOLE No 493S-10**

1 of 1

**METRIC**

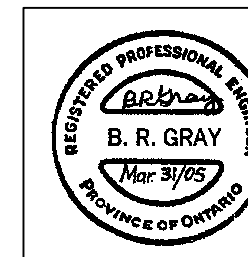
W.P. 5046-00-01 LOCATION Co-ord. 5 135 126 N; 318 896 E ORIGINATED BY CB  
 DIST 54 HWY 69 BOREHOLE TYPE Excavator COMPILED BY PC  
 DATUM Geodetic DATE April 08, 2003 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	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									
234.1	Ground Surface						234										
0.0	Sand, trace silt																
	Compact <u>Brown</u> <u>Moist</u> _____ boulders																
232.9							233										
1.2	End of borehole																
	Refusal on probable bedrock																



NOTES:

1. REFER TO DRAWING 493S-2 FOR SECTIONS B-B AND C-C.
2. SECTIONS ARE PROVIDED SOLELY FOR ILLUSTRATIVE PURPOSES. REFER TO RECORD OF BOREHOLES FOR DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS, IN-SITU TEST DATA AND LABORATORY TEST RESULTS.



— 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		
	DATE	BY		DATE	BY		DATE	BY	

Geocres No.

HWY No	69	CHECKED	GD	DATE	MAR 31, 2005	SITE	54
SUBM'D	FP	CHECKED	BRG	DATE	MAR 31, 2005	SITE	54
DRAWN	MM	CHECKED	BRG	DATE	MAR 31, 2005	SITE	54

REF No E-Makynen Rd-ga Feb 2004

METRIC

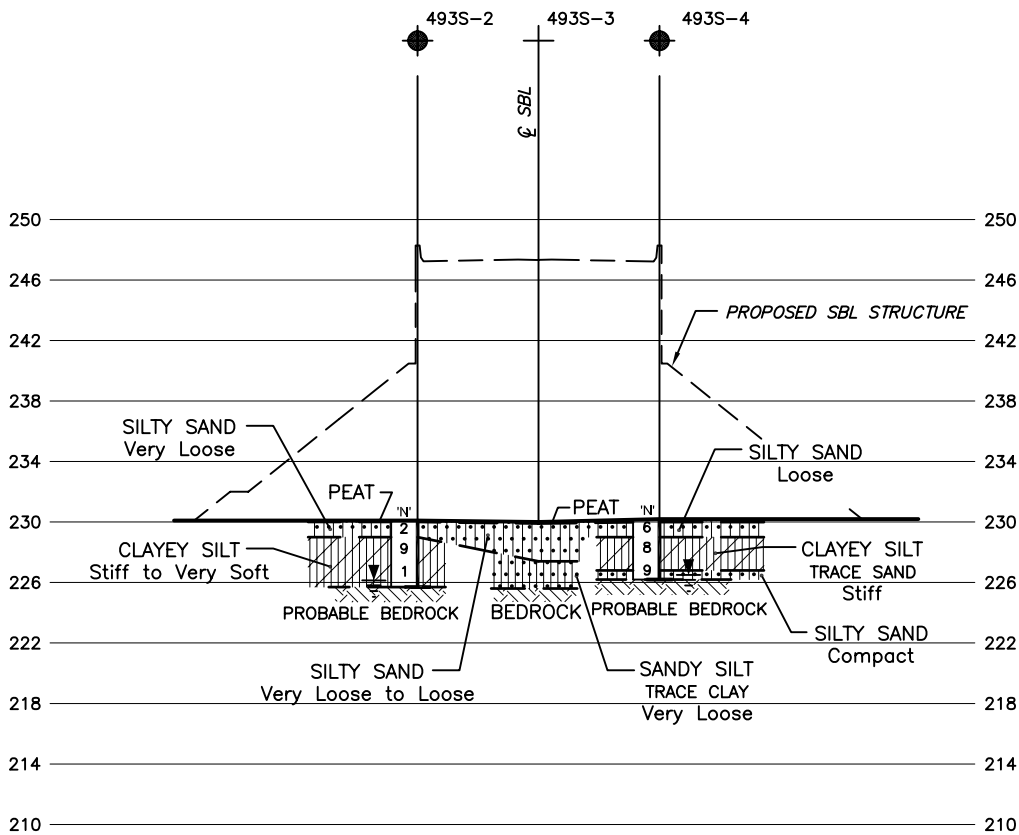
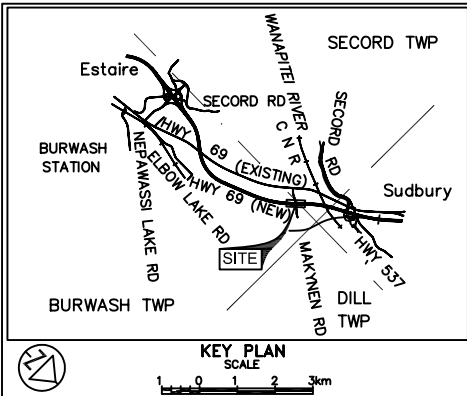
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES

CONT No  
WP No 5046-00-01

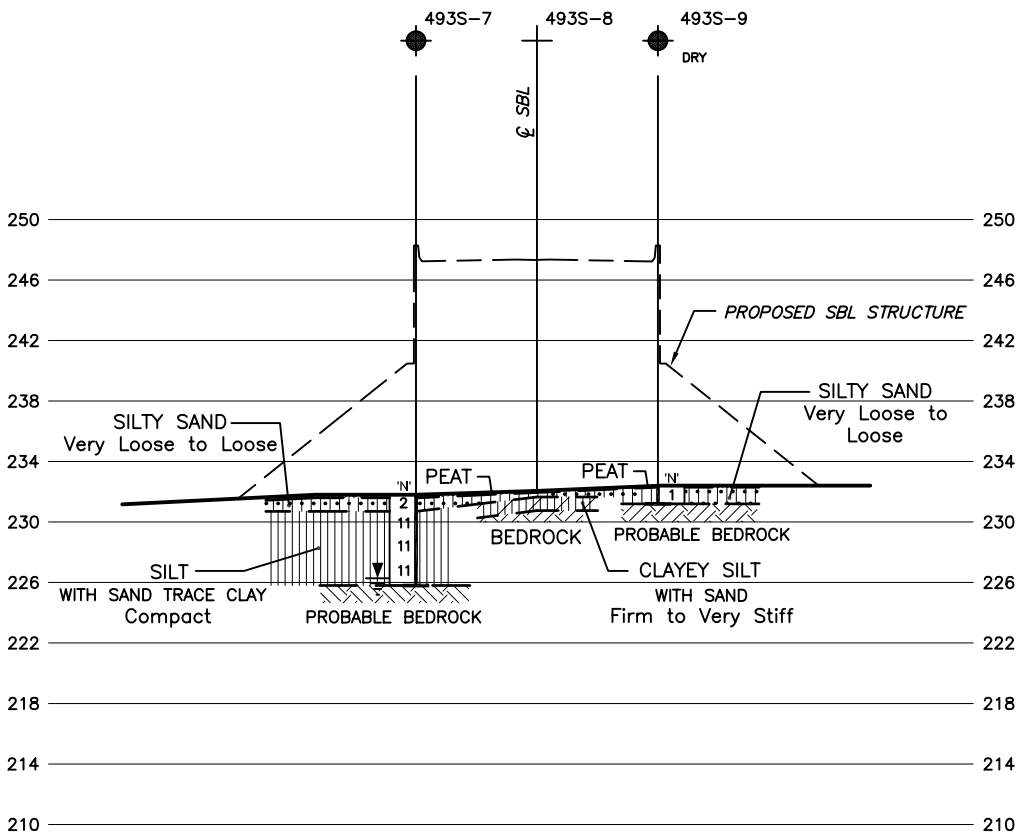
MAKYNEN ROAD OVERPASS  
HIGHWAY 69 SBL  
HIGHWAY 69 FOUR-LANING FOR 12 km  
From 4 km South of Estaire to 1 km North of Hwy 537  
SOIL STRATA

SHEET

PML Peto MacCallum Ltd.  
CONSULTING ENGINEERS



B-B



C-C

NOTES:

- REFER TO DRAWING 493S-1 FOR PLAN AND SECTION A-A.
- SECTIONS ARE PROVIDED SOLELY FOR ILLUSTRATIVE PURPOSES. REFER TO RECORD OF BOREHOLES FOR DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS, IN-SITU TEST DATA AND LABORATORY TEST RESULTS.

SECTIONS  
SCALE



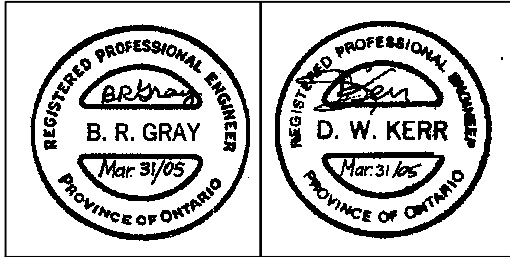
LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- 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 July 2004
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
(Refer to Drawing 493S-1 for co-ordinates)			

NOTE

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



REF No E-Makynen Road-ga Feb 2004

REVISIONS			
DATE	BY	DESCRIPTION	
Geocres No.			
HWY No	69	CHECKED	GD
SUBM'D	FP	DATE	MAR 31, 2005
DRAWN	MM	CHECKED	BRG
		APPROVED	DWK
		DWG	493S-2



**FOUNDATION DESIGN REPORT**

for

**MAKYNEN ROAD SOUTHBOUND OVERPASS  
WP 5046-00-01, SITE 46-493S  
HIGHWAY 69, DISTRICT 54  
SUDBURY**

PETO MacCALLUM LTD.  
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March 31, 2005



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Table 1 - Summary of Advantages, Disadvantages and Recommended Foundations

Table 2 - List of MTO Documents Used 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

Figure 2 - Rockfill Drainage in Slope Flattened Areas



**FOUNDATION DESIGN REPORT**  
for  
Makynen Road Southbound Overpass  
WP 5046-00-01, Site 46-493S  
Highway 69, District 54  
Sudbury

---

**1. INTRODUCTION**

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations, abutments and approach embankments for the proposed construction of an overpass to carry southbound traffic on a new alignment of Highway 69 over Makynen Road some 15 km south of Sudbury, Ontario. The investigation was conducted for Totten Sims Hubicki Associates (TSH) on behalf of the Ontario Ministry of Transportation (MTO).

The centreline of the overpass is at approximate Station 19+783, Highway 69 median chainage. The overpass is proposed to be a single span structure with a span of 44.0 m and width of 14.1 m (ref. Drawing 1 'Hwy 69 (SBL). Makynen Road Overpass. Preliminary General Arrangement' prepared by TSH in February 2004).

The road grade on Highway 69 at the overpass location is planned to be at elevation 247.5 at the south abutment and elevation 246.7 at the north abutment, with the Makynen Road grade at elevation 232.1 to 232.5. The approach embankments to the structure are envisaged to be about 17 and 16 m high at the south and north abutments, respectively (interpolated from ground surface elevations at the borehole locations and the road grade shown on the TSH drawing referred to above).

The subsurface stratigraphy revealed in the boreholes drilled at the abutments and approach embankments to the structure generally comprised a surficial peat deposit underlain by sandy/silty strata containing a discontinuous layer of clayey silt. Boulders were identified in the north approach borehole. Bedrock/probable bedrock was encountered at depths of 4.0 to 4.4 m at the south abutment and 1.2 to 6.0 m at the north abutment.



The depth to and surface elevation of the bedrock identified in the boreholes drilled at this site is summarised in the following table:

Location	Borehole No.	Depth to Rock (m)	Bedrock Elevation
South Approach	493S-1	4.4	225.2
South Abutment	493S-2	4.4	225.7
	493S-3	4.4*	225.6*
	493S-4	4.0	226.2
	493S-5	5.3	225.0
North Abutment	493S-6	6.7	225.1
	493S-7	6.0	225.8
	493S-8	1.3*	230.7*
	493S-9	1.2	231.2
North Approach	493S-10	1.2	232.9

\* confirmed by rock coring

## 2. FOUNDATIONS

### 2.1 General

#### 2.1.1 Abutments

The design road grade at the south and north abutments is near elevation 247.5 and 246.7 respectively or about 21 and 16 m above the bedrock surface. Consequently, use of end-bearing piles driven to bedrock is considered to be the preferred means of supporting the abutment loads from a foundation engineering perspective. Further, construction of integral abutments supported on steel H-piles is considered to be feasible.

Conventional spread footings founded on a pad of engineered fill could also be considered.



### 2.1.2 Seismic Analysis and Liquefaction Potential

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), CAN/CSA-S6-00). The zonal acceleration ratio is 0.05.

The site is located in Seismic Performance Zone 1. The liquefaction potential of the silty/sandy soils at the site was evaluated using the procedure suggested by Seed and Idriss (1971) and, on this basis, it is considered that liquefaction of the cohesionless soils is unlikely to occur (clause 4.6.2 of the CHBDC).

### 2.1.3 Other Comments

The preferred system employed to support the structure foundations will be dictated by structural design considerations, economic considerations and construction constraints.

Further comments and recommendations for design of the foundations are provided in the following sections. A summary of the advantages, disadvantages and the preferred foundation scheme from a foundation engineering perspective is provided in Table 1, attached.

A list of MTO documents used in subsequent sections of the report is given in Table 2 for ease of reference.

## 2.2 Piles

The H-piles should be driven to refusal on bedrock anticipated at depths of 4.0 to 4.4 m below existing grade (elevation 225.6 to 226.2) at the south abutment and 1.2 to 6.0 m (elevation 225.8 to 231.2) at the north abutment.

The recommended factored axial resistance at ultimate limit states (ULS) for four pile sections is as follows (refer to notes 5 and 6 in Section 3.3.3 of the Pile Driving Notes in the Structural Manual, June 2002).



<u>Pile Section</u>	<u>Factored Axial Resistance at ULS (kN)</u>
HP 310 x 110	2000
HP 310 x 152	2800
HP 360 x 108	2000
HP 360 x 152	2800

Boulders were identified within the surficial sand deposit in the north approach borehole. Since they were not identified in the boreholes drilled at the abutments and the native soils are typically loose to compact, it is considered that damage during driving is unlikely and, as a consequence, application of a reduction factor to account for potential damage during driving is not necessary.

The resistance at serviceability limit states (SLS) normally allows for 25 mm compression of the pile and founding medium. Considering the bedrock to be non-yielding and the pile length required, the design is not expected to be governed by settlement criteria since the loading necessary to produce 25 mm axial deformation of the pile and bedrock would be larger than the factored resistance at ULS.

The approach fill embankments within the limits of the pile foundation should comprise Granular A to enable driving and minimise the potential for damage during pile installation.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with granular material meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 3 may be used. Refer to MTO Report SO-96-01 for further details.

The piles are assessed to be about 15 m long at the south abutment and 9 to 14 m long at the north abutment. The piles will be driven through 7 to 10 m of compacted granular fill and the underlying native soils that comprise very loose to dense cohesionless sandy/silty strata containing a discontinuous cohesive deposit of stiff clayey silt. It is considered, based on our



extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed. Since the piles will be driven to bedrock, a specific set is not provided.

The bedrock surface elevation revealed in the boreholes drilled south of Makynen Road is relatively uniform. At the north abutment, however, the bedrock surface elevation revealed in boreholes 493S-8 and 493S-9 is also relatively uniform but some 5 to 6 m higher than that in boreholes 493S-6 and 493S-7 put down 10 m south of the abutment and at the west limit of the abutment footing, respectively. The bedrock surface in borehole 493S-10 located 22 m north of the abutment centreline is about 2 m higher than in borehole 493S-8 drilled at the centre of the abutment. It appears therefore that a northwest/southeast trending rock ridge crosses the central part of the north abutment.

The piles will set on or into bedrock. Therefore, they should be equipped with Oslo Points (OPSD 3304) or Titus "H" Bearing Pile Points, Rock Injector model (SP 903S01, clause 3.1.2 and 3.3.1-6 of the Structural Manual (Division 1 – Exceptions to the CHBDC) dated June 2002).

The piles should be installed and monitored in accordance with the requirements of MTO Special Provision No. 903S01. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 1.7 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 mobilisation of passive resistance along the pile below the annular space. The recommended lateral resistance is as follows:

	<u>Native Silt/Sand</u>		<u>Granular Backfill</u>	
Pile Section	310	360	310	360
At ULS, kN	100	130	120	170
At SLS, kN	25	40	50	70

If greater resistance is required, batter piles should be installed. Since the bedrock surface in the west half of the north abutment slopes down to the southwest at an inclination at least 30°, the batter in piles driven at this foundation should not exceed 1H:6V. The bedrock surface at the south abutment is reasonably level and the maximum batter will be dictated by the equipment employed to install the piles.

The coefficient of horizontal subgrade reaction  $k_s$  (MN/m<sup>3</sup>) should be computed using the following equation to evaluate the point of contraflexure:

$$k_s = n_h z/b$$

where  $n_h$  = coefficient related to soil density

= 10 MN/m<sup>3</sup> for granular backfill

= 2 MN/m<sup>3</sup> for native sandy/silty soils above the groundwater level (elevation 229.5)

= 1 MN/m<sup>3</sup> for native sandy/silty soils below the groundwater level

$z$  = depth, m

$b$  = pile width, m

## 2.3 Spread Footings

### 2.3.1 General

Supporting the overpass structure on conventional spread footings founded on structural fill placed on the native soils or directly on bedrock is considered to be feasible.



Supporting both abutments of the proposed structure on conventional spread footings founded in the native soils is not considered feasible at this site due to the low bearing resistance available.

All footings subject to frost action should be provided with the normal 1.7 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover. Footings bearing directly on bedrock do not require protection from frost.

Construction of the footings should be performed and monitored in accordance with SP 902S01 to verify the competency of the founding surface. In addition, a rock engineering specialist should be retained to examine the integrity and/or impact on bedrock below the footings should blasting be required near the structure foundations.

### 2.3.2 Footings Constructed on Structural Fill

Footings constructed on structural fill placed in the approach embankments could also be employed to support the foundation loads. The structural fill should comprise OPSS Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the standard Proctor maximum dry density and extended laterally to a line inclined downwards at 45° to the horizontal originating at least 1 m from the top of the footing. The limits of the fill should be defined by a site specific survey. This scheme is illustrated in Figure 1, appended.

Footings should not be constructed on rock fill. However, rock fill may be placed adjacent to the Granular 'A' core noted in Figure 1.

The recommended bearing resistance for 2.5 m wide footings constructed on a minimum 3.5 m thick pad of structural fill (founded near elevation 229.0 at the south abutment and near elevation 230.7 at the west end or on the bedrock surface at the north abutment) is as follows:

Factored Bearing Resistance at ULS	= 900 kPa
Bearing Resistance at SLS	= 350 kPa



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. A footing embedment depth of 1.7 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 granular fill.

### **3. ABUTMENT WALLS**

#### **3.1 General**

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)

$\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$

$h$  = 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  $\phi$  = angle of internal friction of retained soil ( $35^\circ$  for Granular A or B Type II)

$\delta$  = angle of friction between soil and wall ( $23.5^\circ$  for Granular A or B Type II)

The seismic site coefficient and zonal acceleration ratio for the conditions at this site were provided in Section 2.1.2.





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

<u>Parameters</u>	<u>Granular A</u>	<u>Granular B Type II</u>	<u>Rock Fill</u>
Angle of Internal Friction, degrees	35	35	42
Unit Weight, kN/m <sup>3</sup>	22.8	22.8	18.0
Coefficient of Active Earth Pressure $K_a$	0.27	0.27	0.20
Coefficient of Earth Pressure At-Rest $K_o$	0.43	0.43	0.33
Coefficient of Passive Earth Pressure $K_p$	3.69	3.69	5.04

Refer to MTO Report SO-96-01 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 and/or weep holes 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. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to the structure should be performed in conformance with Ontario Provincial Standards specifications for granular or rock backfill at abutments (OPSD 3501 or 3505). As noted earlier, Granular A should be employed within the limits of driven piles.

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.



### **3.2 Retained Soil System**

A retained soil system (RSS) designed in accordance with SP 599S22 could also be employed. A high performance, high appearance rated RSS wall will be required.

The founding material for the RSS is expected to comprise bedrock or granular engineered fill. The footings should not be constructed on the native sandy soils due to the low bearing resistance and potential for significant total and differential settlement.

The earth pressure coefficients provided in Section 3.1 as well as the bearing resistance and settlement previously recommended for footings (anticipated width of 0.5 to 1.0 m) founded on structural fill are considered to be appropriate for the RSS wall.

The geotechnical parameters employed to design the RSS system will be dependent upon the type of backfill required for internal stability of the proprietary system as well as the adjacent soil that will govern global stability, overturning and/or sliding of the base. The following parameters are provided for preliminary design purposes and are subject to review:

	<u>Granular A</u>	<u>Granular B Type II</u>	<u>Sand/Silt</u>
Friction Angle, degrees	35	35	28
Cohesion, kPa	0	0	0
Unit Weight, kN/m <sup>3</sup>	22.8	22.8	18.5

The horizontal force at the base of the RSS will be resisted in part by the friction force developed through the granular backfill or along the interface between the granular backfill and the founding soil or bedrock, subject to site specific design details. An unfactored friction factor of 0.7 is considered to be appropriate for the granular material and 0.5 for the sandy/silty soil. The analysis should consider both situations and the worst case scenario used for design.

The RSS supplier should be responsible for specifying the type of backfill material employed, taking into account the engineering properties of the proprietary product, the design life of the



structure, the pull-out resistance required, drainage requirements and the predicted settlements noted in the next section.

The supplier of the RSS should also be responsible for design of the structure (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance etc.

Since the RSS wall, if employed, will be constructed on cohesionless soil or bedrock, it is considered that an adequate margin of safety for global stability will exist ( $F > 1.5$ ).

#### **4. APPROACH EMBANKMENTS**

It is anticipated that the approach embankments will be constructed with earth borrow, granular material or rock fill. The south and north approach fill embankments will be about 17 and 16 m high respectively. Construction of the fill on the very loose to dense sandy/silty soils and/or bedrock is considered to be feasible, subject to the comments provided in the following paragraphs.

The peat identified at the abutment locations and along the alignment of the approach fills within 20 m of the abutments should be stripped and the exposed sandy/silty soils within 10 m of the abutments proof rolled with at least six passes of a heavy roller to improve the density of the subgrade soil prior to placement of the embankment fill.

The embankments should be constructed in accordance with OPSD 201.010, 201.020, 202.010, 203.010, 208.010 and SP 206S03. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rock fill. For erosion control and slope maintenance purposes, a 2 m wide mid-height berm should be provided on the sides of the embankment so that no uninterrupted slope is greater than 6 m high as specified in the Northeastern Region Pavement Design Practices and Guidelines.

The height of the embankment fill on the foreslopes of this structure will be about 10 m. We understand from discussions with MTO Pavements and Foundations that the requirement for mid-height berms was established to improve the stability of rockfill embankment slopes before implementation of SP 206S03. The requirement did not consider the type of founding material



(bedrock, granular, clayey). Following implementation of SP 206S03, construction of rockfill embankments founded on bedrock and/or good quality granular material up to a height of 10 m without a mid-height berm is considered to be appropriate.

The full depth of the soil overlying the bedrock in boreholes 493S-1 and 493S-4 drilled at the south abutment consists of very soft/very loose clayey silt/sandy silt. Consequently, it is recommended that a mid-height berm be constructed in the foreslope of the embankment at the south abutment unless this poor quality soil is excavated to the bedrock surface and replaced with structural fill.

Prior to construction of the rockfill embankment within 20 m of the south abutment, the existing soil should be excavated to elevation 229.0, the exposed subgrade proof rolled with at least six passes of a large smooth drum roller and the excavation backfilled with structural fill.

The structural fill should comprise granular material (Granular B Type II) placed in 200 mm thick lifts compacted to 100% of the standard Proctor maximum dry density. The limits of the excavation should be determined by projecting the line defined by the rockfill slope (1.25H:1V) down to elevation 229.0; the back slope of the excavation should be cut at an inclination of 1H:1V. The limits of the excavation should be established in the field by a surveyor.

At the north abutment, compact clayey silt, silt and sandy silt soils were revealed below the surficial loose material in boreholes 493S-6 and 493S-7 (where the bedrock surface is some 5 to 6 m lower than that in adjacent boreholes). Construction of the foreslope of the rockfill embankment without a mid-height berm is considered to be suitable if the subgrade soil to elevation 229.75 is excavated and replaced with structural fill as noted previously for the south abutment.

It is recommended that the work be carried out in the late summer period when the groundwater level is normally lowest.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. Where slopes are flattened to eliminate the need for a guide rail, a granular infilled drainage gap should be provided in accordance with the Northeastern Region Pavement



Design Practices and Guidelines as shown in Figure 2, appended. OPSS Granular B Type II should be used for the drainage gaps.

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

Some settlement of the road surface should be expected, however, resulting from two mechanisms – consolidation of the soil below the recently placed fill and 'consolidation' of the new fill (rockfill remote from the abutments and backfill adjacent to the abutments).

- Settlement of the embankment surface due to consolidation of the subgrade soil is expected to be less than 10 mm and completed within two months following placement of the fill.
- The backfill placed adjacent to the south and north abutments will be about 17 and 16 m high respectively. Settlement of the embankment surface due to consolidation of the backfill is computed to be about 40 mm.

Consequently, the total settlement of the approach fill surface near the abutments should be in the order of 50 mm and be essentially complete within 4 months after placement of the fill.

The embankments remote from the abutments will be of similar height. Settlement of the rockfill embankments is computed to be about 40 mm in the first year and 50 mm in the following 10 year period.

The settlement assessment for the granular backfill and the rockfill was based on the following criteria/considerations:

#### **Consolidation' of the Granular Backfill Placed Adjacent to the Abutments**

Settlement of the road surface due to 'consolidation' of the granular backfill placed adjacent to the abutments will be primarily dictated by the height of the embankment, the quality of workmanship employed by the contractor and the diligence of the quality control program (to ensure that the



material is placed in accordance with the requirements of SP 902S01 and SP 105S10 and is considered to be about 0.25% of the embankment height.

### **Consolidation of the Rockfill**

Assessment of the magnitude of settlement resulting from consolidation of the rockfill was based on the following criteria established from review of research documents prepared by MTO (RR229 dated March 1983) and discussions with the Pavement and Foundation Section of MTO.

- **Rockfill Above Grade**

Total settlement is about 0.5% of the rockfill height considering that it will be placed in accordance with SP 206S03.

- **Rockfill Below Grade**

Total settlement is up to 2% of the rockfill thickness since it will be end dumped and placed with minimal compactive effort.

- **Rate of Settlement**

About 50% of the total settlement occurs during the first year following placement of the rockfill and the remainder at a progressively decreasing rate during the following 10 year period.

Since the total settlement of the road surface is computed to be about 50 mm, the embankment platform width should be increased by 0.5 to 1.5 m on each side for embankments constructed on bedrock and 2.5 m for embankments constructed on earth fill in accordance with the requirements of the Northeastern Region Engineering Directive (NRE 98-200) dated October 28, 1998.

Fill slopes should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. Where slopes are flattened to eliminate the need for a guide rail, a granular



infilled drainage gap should be provided (refer to Figure 2). OPSS Granular B Type II should be used for the drainage gaps.

## **5. EXCAVATION AND GROUNDWATER CONTROL**

The Preliminary General Arrangement drawing referred to in the introduction indicates that the bottom of the south and north abutments will be constructed on a compacted granular pad near elevation 240.5 and 239.5 respectively, some 10 and 7 m above existing grade. It is expected that construction of the pile caps will be carried out before the fill is placed to raise the grade to the design road level and therefore excavation will not be required.

Excavation of the peat and surficial sandy/silty deposits along the alignment of the embankment will extend to depths of up to 3 m; comments in this regard were presented previously.

The stiff to firm clayey silt and loose to compact sand/silt are classified as Type 3 soil according to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes 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 not be overlooked.

Excavation of bedrock if required will be more difficult and necessitate conventional rock excavation techniques such as blasting (OPSS 120) and jack-hammering. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of cut and relative depth of excavation into the bedrock. The need for preshearing and presplitting to overbreak should not be overlooked.



It is important that blasting/excavation of the rock is controlled to prevent fracturing and/or disturbance of the bedrock surface on which footings will be founded. In this regard, reduced charges to minimise overbreak should be considered. Any overblasting/overexcavation should be made the sole responsibility of the contractor and all loosened rock resulting from blasting operations is to be removed by mechanical means. A large excavator equipped with a tiger-toothed bucket in conjunction with a jack-hammer or hoe ram is the preferred method of excavation to shallow depths in rock at foundation locations.

Mechanical means should be employed to excavate the loosened rock at the footing. Mass concrete could be employed to level minor variations in the bedrock surface.

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

Groundwater was observed in six boreholes in the course of the field work. Upon completion of drilling, groundwater was measured at depths of 3.7 to 5.6 m (elevation 226.1 to 226.6). It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations at both abutments. Groundwater levels are subject to seasonal fluctuations and precipitation patterns.

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





## 6. CLOSURE

The report was prepared by Mr. Grigory O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. Dennis W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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GD:gd-mm-mi



**TABLE 1**  
**SUMMARY OF ADVANTAGES, DISADVANTAGES AND**  
**RECOMMENDED FOUNDATIONS**

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RECOMMENDED FOUNDATION SCHEME
South Abutment			
Spread footings on rock	Not appropriate		Driven piles
Spread footings on engineered fill pad	<ul style="list-style-type: none"><li>• Ease of construction</li><li>• No need for rock excavation</li></ul>	<ul style="list-style-type: none"><li>• Difficulty in placing engineered fill on undulating rock surface</li><li>• Lower bearing resistance than for piles</li><li>• Need for erosion protection</li></ul>	
Driven piles	<ul style="list-style-type: none"><li>• High bearing resistance</li><li>• Construction of integral abutment possible</li></ul>	<ul style="list-style-type: none"><li>• Higher cost than footings</li><li>• Difficult access</li></ul>	
Caissons	<ul style="list-style-type: none"><li>• High bearing resistance</li></ul>	<ul style="list-style-type: none"><li>• High cost relative to other alternatives</li><li>• Difficult access</li></ul>	
North Abutment			
Spread footings on rock	Not appropriate		Driven piles
Spread footings on engineered fill pad	<ul style="list-style-type: none"><li>• Ease of construction</li><li>• No need for rock excavation</li></ul>	<ul style="list-style-type: none"><li>• Difficulty in placing engineered fill on undulating rock surface</li><li>• Low bearing resistance</li><li>• Need for erosion protection</li></ul>	
Driven piles	<ul style="list-style-type: none"><li>• High bearing resistance</li><li>• Construction of integral abutment possible</li></ul>	<ul style="list-style-type: none"><li>• Higher cost than footings</li><li>• Difficult access</li></ul>	
Caissons	<ul style="list-style-type: none"><li>• High bearing resistance</li></ul>	<ul style="list-style-type: none"><li>• High cost relative to other alternatives</li><li>• Difficult access</li></ul>	



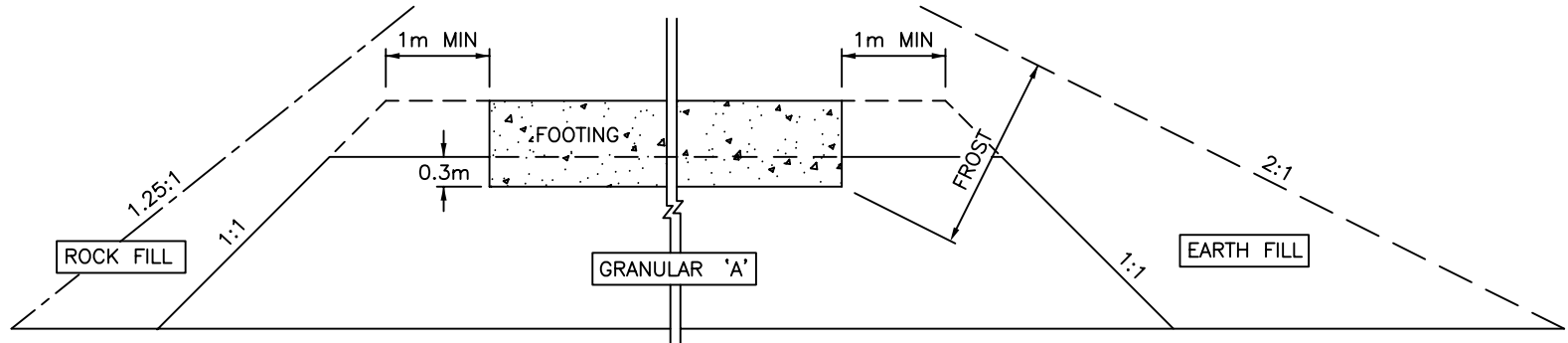
**TABLE 2**  
**LIST OF MTO DOCUMENTS USED IN REPORT**

<b>NO.</b>	<b>TITLE</b>	<b>DATE</b>
OPSD 201.010	Rock Grading. Undivided Highway	April 1999
OPSD 201.020	Rock Grading. Divided Highway	April 1999
OPSD 202.010	Embankment Construction Using Excess Material Outside of Earth or Rock Fill	March 1, 1998
OPSD 202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment	March 1, 1998
OPSD 203.010	Embankments over Swamp. New Construction	November 2004
OPSD 208.010	Benching of Earth Slopes	November 2003
OPSD 3304.000	Oslo Points for HP310 H-Piles	November 2001
OPSD 3501.000	Minimum Granular Backfill Requirements. Abutments	April 1999
OPSD 3505.000	Rock Backfill Requirements. Abutments	November 2001
OPSS 120	General Specification for the Use of Explosives	November 2003
OPSS 571	Construction Specification for Sodding	November 2001
OPSS 572	Construction Specification for Seed and Cover	November 2003
SP 105S10	Soils Compaction - Quality Assurance and Quality Control	November 2004
SP 206S03	Earth and Rock Excavation	January 2004
SP 599S22	Retained Soil Systems	March 2001
SP 902S01	Earth and Rock Excavation for Structure	September 2003
SP 903S01	Piling	September 2004
SP 999S26	Design, Installation and Testing of Pre-Stressed Anchors in Soil and Rock	July 2004
SO-96-01	Integral Abutment Bridges	July 1996
NRE 98-200	Embankment Platform Widening	October 28, 1998



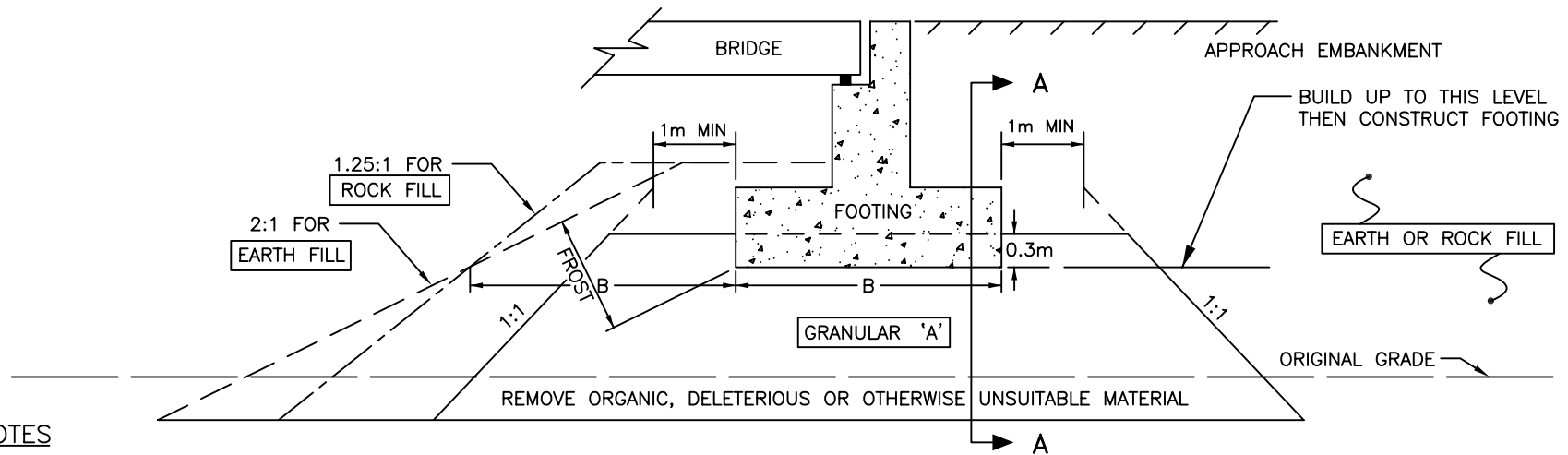
**TABLE 3**  
**GRADATION SPECIFICATION FOR SAND FILL IN**  
**PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS**

<b>MTO SIEVE DESIGNATION</b>		<b>PERCENTAGE PASSING BY MASS</b>
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



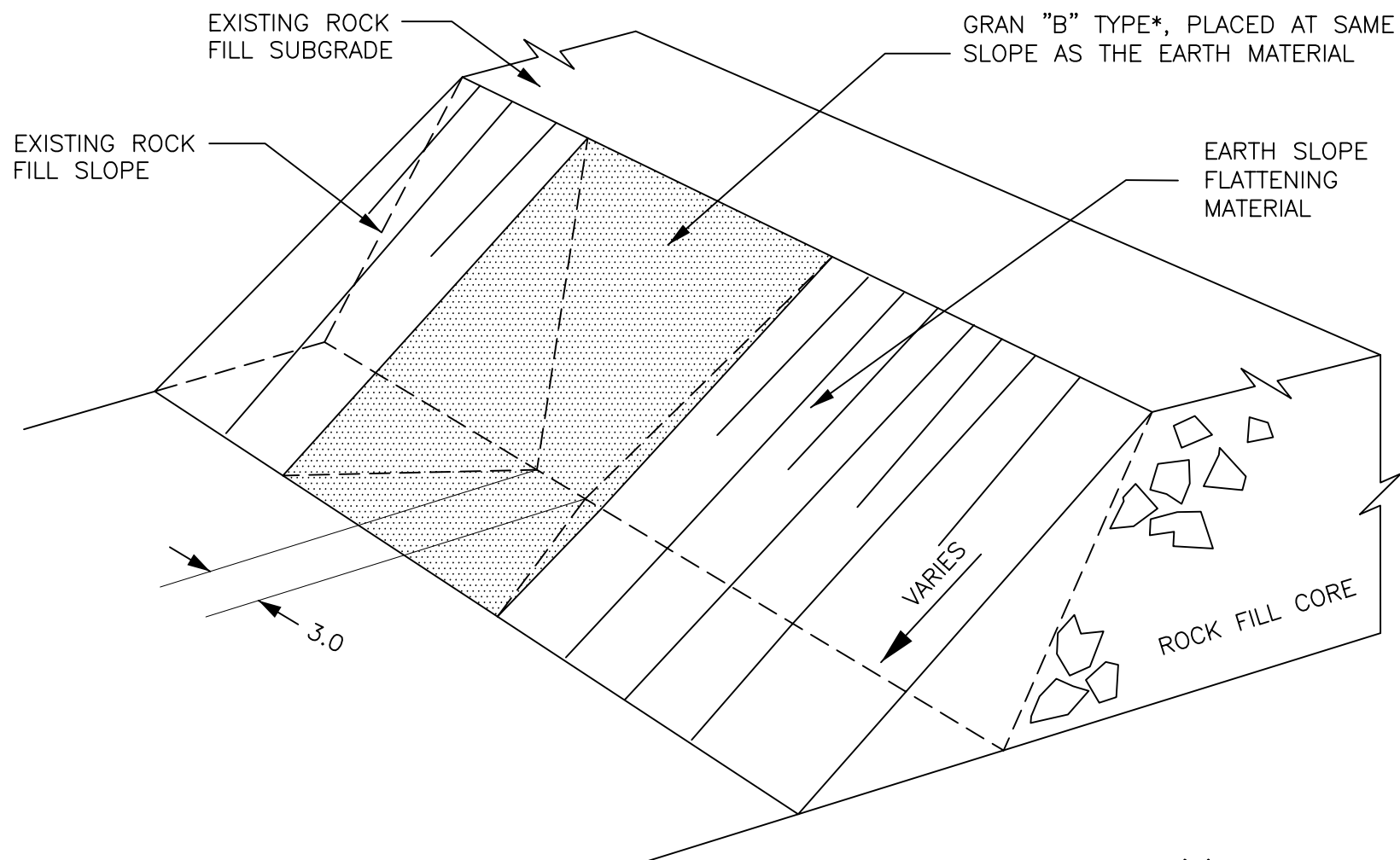
**LONGITUDINAL SECTION**

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**



\* GRAN 'B' TYPE I OR TYPE II AS  
RECOMMENDED FOR PROJECT.

FIGURE 2: ROCK FILL DRAINAGE IN SLOPE FLATTENED AREAS

NOT TO SCALE