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DIST. 14 REGION

W.P. No. 314-85-01

CONT. No. 91-226

W. O. No.

STR. SITE No. 47-81

HWY. No. 560

LOCATION Hwy 560 & Englehart River

No. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 91-226



Ministry of
Transportation

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Note: For purposes of the contract, this report supersedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

EXPLANATION OF TERMS USED IN REPORT

2

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 (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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u_o	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
C_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{C_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

REPORT ON
FOUNDATION INVESTIGATION
FOR PROPOSED
REPLACEMENT OF ENGLEHART RIVER BRIDGE
WP-314-85-01: SITE 47-31
NEW LISKEARD DIST. 14 REG. NORTHERN

1. INTRODUCTION

B.P. Walker Associates Limited, Consulting Geotechnical, Inspection and Testing Engineers, was authorized by the Ministry of Transportation, Ontario to conduct a geotechnical investigation at the site of the existing Englehart River Bridge. The existing structure will be replaced with a new structure at the same location. Conceptual design data regarding the project were transmitted to us by M. Devata, P. Eng., Chief Foundation Engineer of the Ministry.

The purpose of the geotechnical investigation was to explore the subsurface conditions by means of boreholes. This report presents a brief account of the procedures followed in the investigation, the field and laboratory test results, and our interpretation of the findings.

2. THE SITE AND GEOLOGY

The site is located on the Englehart River at Highway 560 west of Town of Englehart. The area is part of the Canadian Shield and the rock formation is classified as Precambrian. The bedrock, except at outcrop locations, is mostly covered with a thin mantle of drift which includes ground moraine, silt and talus.

The topography of the area is gently rolling with infrequent bedrock outcrops. The glacio-lacustrine deposits of the area consist of silts and clays and have been deposited either during or following the last glaciation.

3. FIELD AND LABORATORY WORK

Four boreholes, numbered 1 to 4, were drilled as shown on the borehole location plan, drawing No 2.* These borehole locations were selected in agreement with MTO on a site plan transmitted to us.

The four boreholes were drilled in the interim between April 30, 1990 and May 3, 1990, to depths ranging from 3.1m to 12.8m. A bombardier mounted drilling rig, equipped with hollow stem augers, 75mm casing and BXC core for coring the bedrock were used for advancing the holes. The boreholes near the abutment were advanced to refusal using continuous flight augers. Below the refusal depth the borehole was advanced using diamond drilling. At the pier locations the casing was socketed into the bedrock and the bedrock cored. The drilling, sampling and the field testing procedures were supervised and the borings were logged by an experienced geotechnical engineer from our office.

Dynamic cone penetration tests were performed adjacent to the abutment boreholes.

Samples in the fill and the overburden were taken with a 51mm o.d. split spoon (SS) in accordance with ASTM D 1586-84, Standard Method for PENETRATION TEST AND SPLIT BARREL SAMPLING OF SOILS. Although the recovered samples are disturbed, they are representative of the stratum from which they were obtained and the STANDARD PENETRATION RESISTANCE (N-values) indicates the relative density or consistency of the sampled soil. In the dynamic cone penetration test a 50mm diameter cone is driven with the same driving energy as in the Standard Penetration Test.

* DWG NO 2 OF THE CONTRACT DWG'S

4. SUBSURFACE CONDITIONS

(a) Abutments

Fill was encountered at both abutment boreholes to depths of 4.4m below the existing ground. The natural soil under the fill is clayey silt, some to trace sand, trace gravel with silt and sand seams. Underlying the clayey silt is a layer of boulders varying in thickness from 0.6m at borehole 4 to 1.7m at borehole 1 over bedrock.

(b) Piers

No fill was encountered at the piers. At borehole 2 the bedrock was exposed at the surface. At borehole No. 3 overburden consisting of silty clay followed by boulders was encountered to depths of 0.9m and 1.2m respectively at which depth rock occurred.

A detailed description of the soil encountered in each borehole is given in the Record of Borehole drawings. The estimated stratigraphic profile shown on Drawing NO 2*, is based on this information. From ground level downwards, the subsurface conditions in detail are as follows:

Fill Material (Boreholes 1 and 4)

The boreholes carried out near the embankment encountered fill to a depth of 4.4m below the existing ground surface. The fill material is composed of gravelly sand, trace of silt and clay. Standard Penetration Tests gave "N" values in the range of 6 to 15 blows per 30cm indicating that the fill material has a loose to compact consistency.

* DWG NO 2 OF THE CONTRACT DWG'S

The results of grain size distribution testing performed on representative samples from the fill are shown on Figure 1.

Clayey Silt, some to trace sand, trace of gravel

This deposit was encountered at the boreholes near the bridge abutments.

Immediately under the fill at boreholes 1 and 4 is a deposit of clayey silt, some to trace sand, trace of gravel. This deposit contains frequent layers of silt and sand. The thickness of this deposit varied from 3.7m at borehole 1 to 2.8m at borehole 4. Shear strength, based on unconfined compressive tests and laboratory shear vane test results on a thin wall shelby tube sample gave a value of 75kPa. Based on the N-values and the shear strength, the clayey silt is soft to very stiff.

The physical properties of this deposit, as determined from laboratory testing, are summarized below:

	<u>Range</u>	<u>Average</u>
Liquid Limit (W _L) %	30 to 35	33
Plastic Limit (W _p) %	16 to 19	18
Plasticity Index (I _p) %	11 to 18	14
Moisture Content (W) %	22 to 35	27

The results of the Atterberg Limit Tests are shown on the Plasticity Chart on Figure 2. These results indicate that the deposit is inorganic, low to medium plasticity (CL to CI zone).

The results of grain size distribution testing performed on representative samples from the clayey silt deposit are shown on Figure 3.

Boulders

Bedrock was exposed at one of the four boreholes, i.e. at borehole 2. At other boreholes refusal to augers was encountered below the clayey silt/silty clay deposit. Core drilling was started from the refusal depth. Cuttings from the drilling showed a mixture of sand and clay with some rock powder and the core recovery was minimal indicating boulders mixed with sand and clay. The elevation of bedrock was estimated below the depth where no sand and clay was noticed in the core cuttings.

Bedrock

The quality of bedrock on the site was found to vary from poor at the east abutment to excellent at the west abutment. The bedrock type, recovery and the RQD values at each borehole are as follows:

Borehole 1 - East Abutment

The bedrock at this borehole is sound, greenish grey, basalt with serpentine and quartz filled fractures. Core recovery at this location was 100%. The RQD values varied from 35% to 45% indicating a poor quality bedrock.

Borehole 2 - East Pier

The bedrock at this borehole is sound, greenish grey, basalt with serpentine and quartz filled fractures. Core recovery was 100%. The RQD values varied from 66% to 85% indicating fair to good quality bedrock.

Borehole 3 - West Pier

The bedrock at this location is sound greenish black basalt. It is massive, aphanitic with granitic inclusions. The core recovery varied from 99% to 100% with an average RQD of 96 indicating an excellent bedrock quality.

Borehole 4 - West Abutment

The bedrock at this location is sound, greenish black basalt. It is massive, aphanitic with granitic inclusions. The core recovery was 100% with RQD varying from 97% to 100% indicating excellent bedrock quality.

5. CLOSURE

The soil stratigraphy and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations and subsurface conditions may become apparent during construction which could be detected or anticipated from the site investigation. Also, depending on seasonal factors, the groundwater table could be at a different level than at the time of the field work.

NOTE: The preceding report is a copy of the factual information from the Foundation Investigation Report prepared by B.P. Walker Associates Ltd. (consulting geotechnical engineers for this project), under the technical supervision of the MTO Foundation Design Section.

APPENDIX



METRIC

OFFICE, REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 2										METRIC				
W P 314-85-01		LOCATION STA 16+813.6, O/S 2.0m RT of HWY 560				ORIGINATED BY T.O.								
DIST N.L. HWY 560		BOREHOLE TYPE BXC ROCK CORE				COMPILED BY U.S.S.								
DATUM		DATE May 4, 1990				CHECKED BY U.S.S.								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
242.1	GROUND SURFACE													
0.0	BEDROCK-basalt with serpentine, greenish grey, sound, quartz filled fractures		1	BXC RC	REC 100%									RQD 66%
			2	BXC RC	REC 100%									RQD 87%
			3	BXC RC	REC 100%									RQD 85%
239.0	END OF BOREHOLE													
3.1	Water level not established.													

OFFICE REPORT ON SOIL EXPLORATION

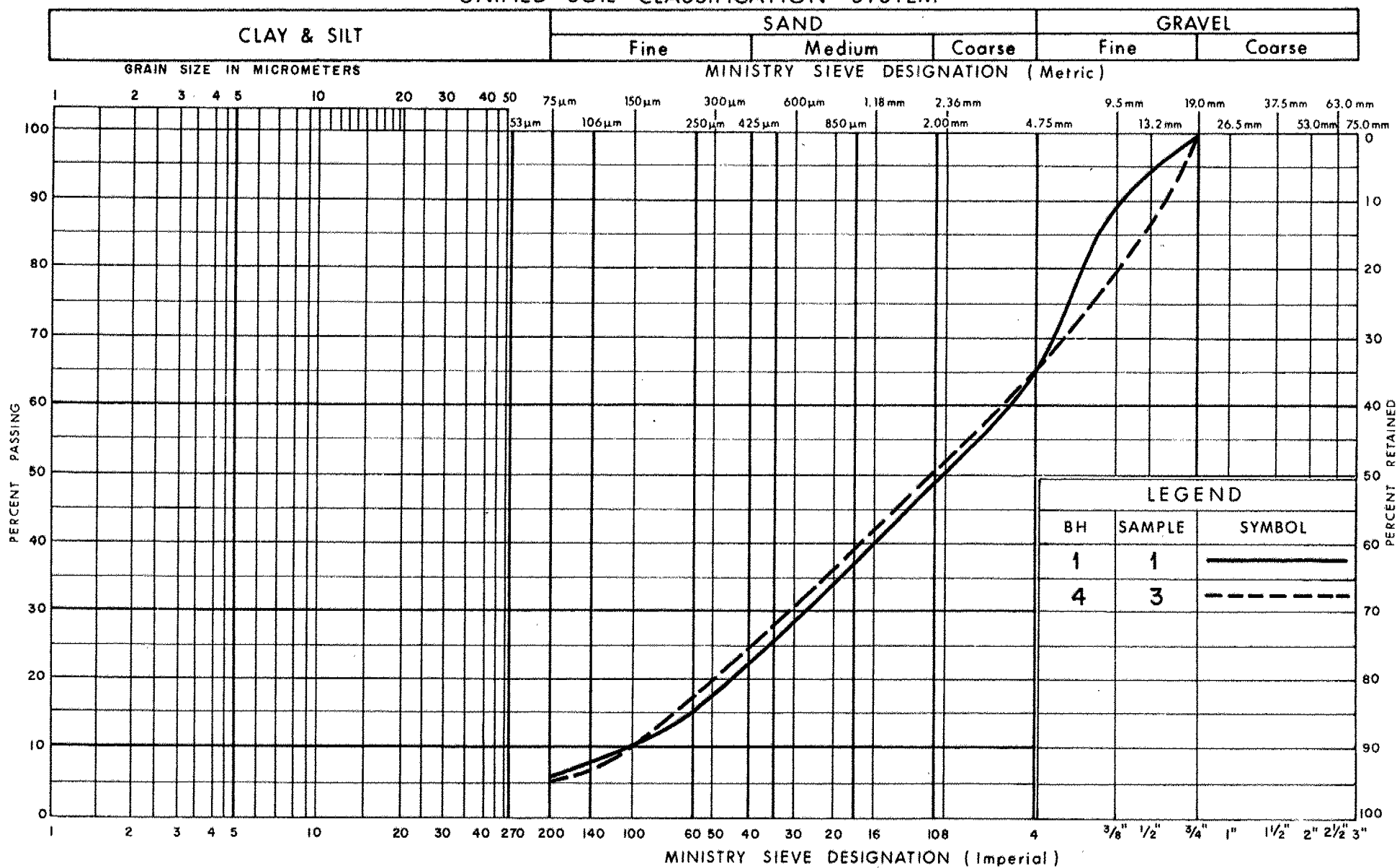
RECORD OF BOREHOLE No 3										METRIC			
W P <u>314-85-01</u>			LOCATION <u>STA 16+790.8; 0/S 2.0m LT of HWY 560</u>				ORIGINATED BY <u>T.O.</u>						
DIST <u>N.L.</u> HWY <u>560</u>			BOREHOLE TYPE <u>WASH BORING AND BXC ROCK CORE</u>				COMPILED BY <u>U.S.S.</u>						
DATUM _____			DATE <u>May 3, 1990</u>				CHECKED BY <u>USS</u>						
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)			
242.5	GROUND SURFACE												
0.0	SILTY CLAY-some sand, trace gravel												
241.6													
0.9	BOULDERS												
241.3													
1.2	BEDROCK-basalt, greenish black, massive, sound, aphanitic with granitic inclusions		1	BXC RC	REC 99.1%								RQD 95%
			2	BXC RC	REC 100%								RQD 96%
238.3													
4.2	END OF BOREHOLE												
	Water level not established.												

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 4										METRIC			
W P 314-85-01		LOCATION STA 16+772.3; O/S 2.0m RT of HWY 560				ORIGINATED BY T.O.							
DIST N.L. HWY 560		BOREHOLE TYPE HOLLOW STEM AUGER, BXC ROCK CORE & CONE TEST				COMPILED BY U.S.S.							
DATUM		DATE May 2, 1990				CHECKED BY USS							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
252.0	GROUND SURFACE												
0.0	ASPHALT												
	FILL-gravelly sand, trace of silt and clay, brown, damp, loose to compact		1	SS	15								
			2	SS	6								
			3	SS	13								
			4	SS	7								
247.6													
4.4	CLAYEY SILT-some to trace sand, soft to very stiff		5	SS	7								
			6	TW	PUSH								
			7	SS	16								
244.8	BOULDERS @ 6.7m												
7.2	BOULDERS												
244.2													
7.8	BEDROCK-basalt, greenish black, massive, aphanitic with granitic inclusions		1	BXC RC	REC 100%								
			2	BXC RC	REC 100%								
241.5													
10.5	END OF BOREHOLE												
	Water level not established.												

OFFICE REPORT ON SOIL EXPLORATION

UNIFIED SOIL CLASSIFICATION SYSTEM

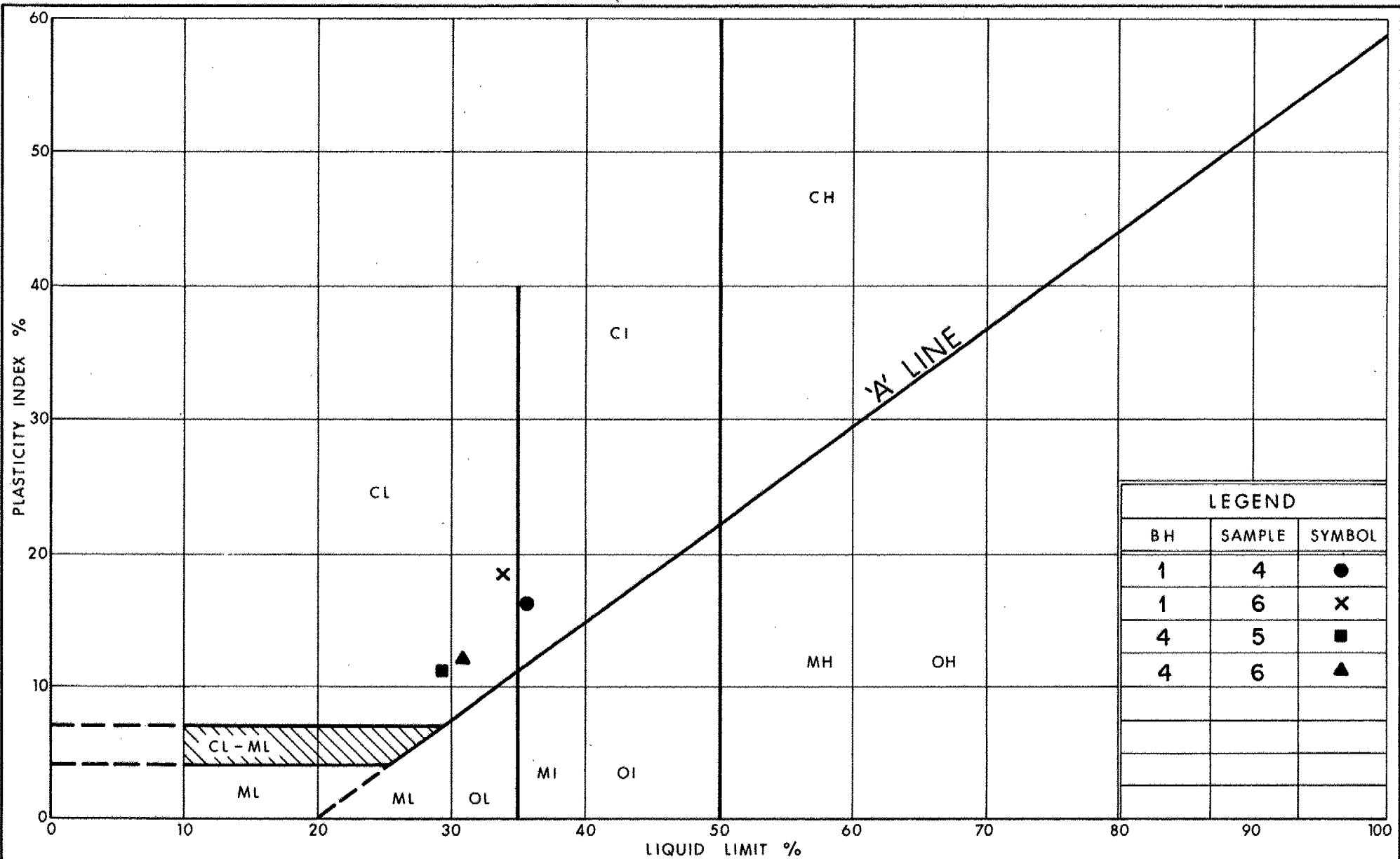


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
GRAVELLY SAND, TRACE SILT, CLAY (Fill)

FIG No 1

W P 314-85-01



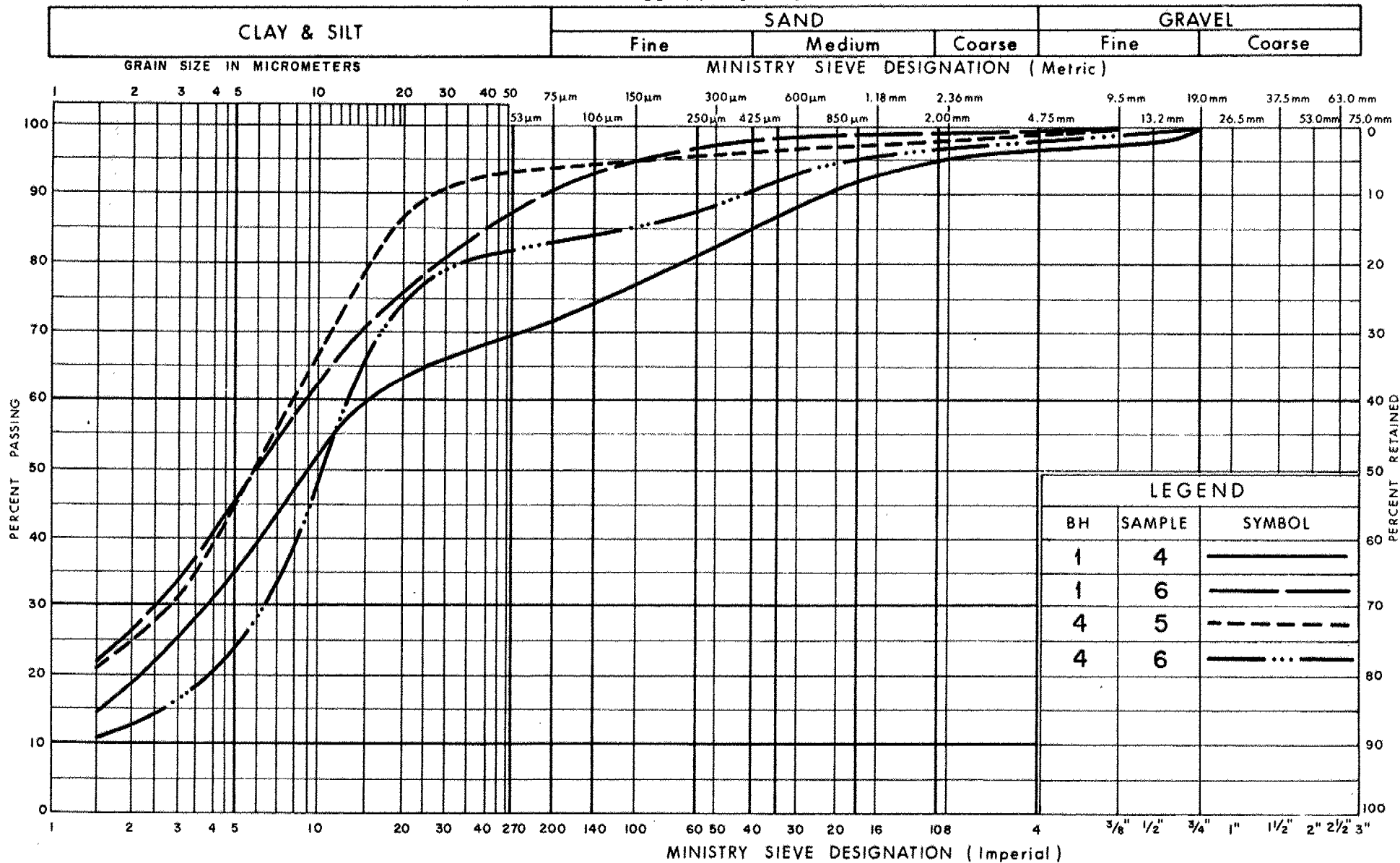
Ministry of
Transportation
Ontario

PLASTICITY CHART
CLAYEY SILT, SOME TO TRACE SAND, TRACE GRAVEL

FIG No 2

W P 314-85-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME TO TRACE SAND, TRACE GRAVEL

FIG No 3

W P 314-85-01

B.P. WALKER

Associates Limited

REPORT ON
FOUNDATION INVESTIGATION
FOR PROPOSED
REPLACEMENT OF ENGLEHART RIVERBRIDGE
WP-314-85-01: SITE 47-01
NEW LISKEARD DIST. 14 REG. NORTHERN

CONT 91-226

Consulting Geotechnical
Inspection and
Testing Engineers

B.P.Walker Associates Ltd.

Consulting Geotechnical, Inspection and Testing Engineers

101 Amber Street, Suite 2, Markham, Ontario, L3R 3B2 (416) 491-4075 Fax. # 475-5376

REPORT ON
FOUNDATION INVESTIGATION
FOR PROPOSED
REPLACEMENT OF ENGLEHART RIVER BRIDGE
WP-314-85-01: SITE 47-31
NEW LISKEARD DIST. 14 REG. NORTHERN

GEOCRES # 31M-53

Prepared for:
Ministry of Transportation
Foundation Design Section
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Project No. 2339-0590

June 21, 1990

Project No. 2339-0590

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4. SUBSURFACE CONDITIONS	3
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DRAWINGS

	<u>Drawing No.</u>
Record Of Boreholes	1 to 4
Grain Size Distributions	Fig. 1 & 3
Plasticity Chart	Fig. 2

1. INTRODUCTION

B.P. Walker Associates Limited, Consulting Geotechnical, Inspection and Testing Engineers, was authorized by the Ministry of Transportation, Ontario to conduct a geotechnical investigation at the site of the existing Englehart River Bridge. The existing structure will be replaced with a new structure at the same location. Conceptual design data regarding the project were transmitted to us by M. Devata, P. Eng., Chief Foundation Engineer of the Ministry.

The purpose of the geotechnical investigation was to explore the subsurface conditions by means of boreholes for the design and construction of the foundations for the bridge. This report presents a brief account of the procedures followed in the investigation, the field and laboratory test results, and our interpretation of the findings. Included herein are our recommendations for the geotechnical design of the project.

2. THE SITE AND GEOLOGY

The site is located on the Englehart River at Highway 560 west of Town of Englehart. The area is part of the Canadian Shield and the rock formation is classified as Precambrian. The bedrock, except at outcrop locations, is mostly covered with a thin mantle of drift which includes ground moraine, silt and talus.

The topography of the area is gently rolling with infrequent bedrock outcrops. The glacio-lacustrine deposits of the area consist of silts and clays and have been deposited either during or following the last glaciation.

3. FIELD AND LABORATORY WORK

Four boreholes, numbered 1 to 4, were drilled as shown on the borehole location plan, drawings 3148501-A. These borehole locations were selected in agreement with MTO on a site plan transmitted to us.

The four boreholes were drilled in the interim between April 30, 1990 and May 3, 1990, to depths ranging from 3.1m to 12.8m. A bombardier mounted drilling rig, equipped with hollow stem augers, 75mm casing and BXC core for coring the bedrock were used for advancing the holes. The boreholes near the abutment were advanced to refusal using continuous flight augers. Below the refusal depth the borehole was advanced using diamond drilling. At the pier locations the casing was socketed into the bedrock and the bedrock cored. The drilling, sampling and the field testing procedures were supervised and the borings were logged by an experienced geotechnical engineer from our office.

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4. SUBSURFACE CONDITIONS

(a) Abutments

Fill was encountered at both abutment boreholes to depths of 4.4m below the existing ground. The natural soil under the fill is clayey silt, some to trace sand, trace gravel with silt and sand seams. Underlying the clayey silt is a layer of boulders varying in thickness from 0.6m at borehole 4 to 1.7m at borehole 1 over bedrock.

(b) Piers

No fill was encountered at the piers. At borehole 2 the bedrock was exposed at the surface. At borehole No. 3 overburden consisting of silty clay followed by boulders was encountered to depths of 0.9m and 1.2m respectively at which depth rock occurred.

A detailed description of the soil encountered in each borehole is given in the Record of Borehole drawings. The estimated stratigraphic profile shown on Drawing 3148501-A is based on this information. From ground level downwards, the subsurface conditions in detail are as follows:

Fill Material (Boreholes 1 and 4)

The boreholes carried out near the embankment encountered fill to a depth of 4.4m below the existing ground surface. The fill material is composed of gravelly sand, trace of silt and clay. Standard Penetration Tests gave "N" values in the range of 6 to 15 blows per 30cm indicating that the fill material has a loose to compact consistency.

The results of grain size distribution testing performed on representative samples from the fill are shown on Figure 1.

Clayey Silt, some to trace sand, trace of gravel

This deposit was encountered at the boreholes near the bridge abutments.

Immediately under the fill at boreholes 1 and 4 is a deposit of clayey silt, some to trace sand, trace of gravel. This deposit contains frequent layers of silt and sand. The thickness of this deposit varied from 3.7m at borehole 1 to 2.8m at borehole 4. Shear strength, based on unconfined compressive tests and laboratory shear vane test results on a thin wall shelby tube sample gave a value of 75kPa. Based on the N-values and the shear strength, the clayey silt is soft to very stiff.

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The results of grain size distribution testing performed on representative samples from the clayey silt deposit are shown on Figure 3.

Boulders

Bedrock was exposed at one of the four boreholes, i.e. at borehole 2. At other boreholes refusal to augers was encountered below the clayey silt/silty clay deposit. Core drilling was started from the refusal depth. Cuttings from the drilling showed a mixture of sand and clay with some rock powder and the core recovery was minimal indicating boulders mixed with sand and clay. The elevation of bedrock was estimated below the depth where no sand and clay was noticed in the core cuttings.

Bedrock

The quality of bedrock on the site was found to vary from poor at the east abutment to excellent at the west abutment. The bedrock type, recovery and the RQD values at each borehole are as follows:

Borehole 1 - East Abutment

The bedrock at this borehole is sound, greenish grey, basalt with serpentine and quartz filled fractures. Core recovery at this location was 100%. The RQD values varied from 35% to 45% indicating a poor quality bedrock.

Borehole 2 - East Pier

The bedrock at this borehole is sound, greenish grey, basalt with serpentine and quartz filled fractures. Core recovery was 100%. The RQD values varied from 66% to 85% indicating fair to good quality bedrock.

Borehole 3 - West Pier

The bedrock at this location is sound greenish black basalt. It is massive, aphanitic with granitic inclusions. The core recovery varied from 99% to 100% with an average RQD of 96 indicating an excellent bedrock quality.

Borehole 4 - West Abutment

The bedrock at this location is sound, greenish black basalt. It is massive, aphanitic with granitic inclusions. The core recovery was 100% with RQD varying from 97% to 100% indicating excellent bedrock quality.

5. DISCUSSION AND RECOMMENDATIONS

5.1 Project

The proposed structure will replace the existing bridge at the same location. The present road alignment will be temporarily rerouted north of the existing bridge and the bridge will be removed. A new bridge will be located at the same location as the existing bridge. According to the conceptual layout, Drawings E-701-560-1 obtained from MTO, the top of the bridge will be at an elevation of 252.0m.

The proposed structure will be of three span construction with side spans of 16.0m and centre span of 24.0m. The abutment foundation base level is estimated to be about El. 249.0m whereas the pier foundation will be founded on the bedrock.

Our recommendations are based on this information.

1. Abutment Foundations

The subsoil at the above elevation of 249.0m at both abutments is a fill consisting of gravelly sand, trace of silt and clay. This material, based on the N-values is loose to compact only and not suitable for spread foundations for the bridge.

The clayey silt, some to trace sand, trace gravel, underlying the gravelly sand fill is stiff to very stiff at the east abutment and soft to very stiff under the west abutment and therefore not suitable for spread footings. In view of these variable conditions we have considered different options for the abutment foundations for the proposed structure and it is our opinion that the following foundation alternatives will be the most suitable for this site.

(a) Spread Foundations on Engineered Fill

Spread foundations on engineered fill will require excavation to an elevation of 247.0m at the east abutment. At the west abutment, Borehole 4, slightly deeper excavation to elevation 246.5m will be required to remove the soft clayey silt material. The advantage of placing footings on engineered fill is to ensure the continuity and uniformity of the soil immediately beneath the footings. This can be done with greater certainty on compacted granular fill than on the clayey silt deposit. This design will require an engineered fill thickness of at least 2.0m. The engineered fill should conform to MTO Standard Form 1010 and should be placed in 150mm thick lifts and each lift should be uniformly compacted at the optimum moisture content to at least 100% of its Standard Proctor Maximum Dry Density.

For footings on engineered fill, placed in accordance with the above requirements, the Factored Bearing Capacity at Ultimate Limit

State is 775 kPa. The Bearing Capacity at Serviceability Limit State Type II (qs) is 300 kPa. Assuming a 4m X 12m footing and a 45 degree load distribution through the 2m Granular 'A' core, the stress on the clayey silt will be 150 kPa at the Serviceability Limit State Type II (qs). The clayey silt is capable of safely supporting this bearing pressure.

Another alternative will be to excavate to the sound bedrock and place the engineered fill on top of the bedrock. The Factored Bearing Capacity at Ultimate Limit State is 775 kPa and the Bearing Capacity at Serviceability Limit State Type II (qs) is 300 kPa.

For the evaluation of the sliding resistance of footings on engineered fill consisting of granular 'A', the friction angle between the concrete and the granular 'A' should be taken as 35 degrees.

The clayey silt and the engineered fill are susceptible to scouring. If the abutment foundations are placed on the engineered fill, slope protection consisting of rip rap or equivalent will be needed.

(b) Spread Footings on Bedrock

Spread footings on bedrock will require excavation to an elevation of 242.2m at the east abutment, Borehole 1, and 244.2m at the west abutment, Borehole 4. For footings placed on sound bedrock, the Factored Bearing Capacity at Ultimate Limit State (qf) is 4000 kPa. The bearing capacity at Serviceability Limit State Type II (qs) need not be considered.

We recommend that all lateral forces along the footing base be resisted by a key cut into the sound bedrock below the footing.

The minimum depth of the key should be 0.5m. Provided that concrete is placed against the 'undisturbed' rock face the key should provide a resisting pressure of 2000 kPa against lateral forces. An alternative would be to use dowels, grouted into the bedrock.

Caissons founded on sound bedrock or driven H piles are other foundation alternatives. However, it is our opinion that these alternatives would be expensive and difficult to install due to a layer of boulders immediately above the bedrock.

2. Pier Foundations

Pier foundations founded on the sound bedrock may be designed for Factored Bearing Capacity at Ultimate Limit State (q_f) of 4000 kPa. Bearing capacity at Serviceability Limit State Type II (q_s) need not be considered.

We recommend that the lateral forces along the footing base be resisted by a key cut into the sound bedrock below the footing. The minimum depth of the key should be 0.5m. Provided that concrete is placed against the 'undisturbed' rock face the key should provide a resisting pressure of 2000 kPa against lateral forces. An alternative would be to use dowels, grouted into the bedrock.

5.2 Backfill

Rigid walls of the bridge abutments should be designed to withstand the at-rest earth pressures which can be approximated using the following equivalent fluid pressures:

At Ultimate Limit State	10 kPa/m
At Serviceability Limit State, Type II	8.5 kPa/m

When using the above values, it is assumed that the slope of the backfill behind the retaining structure is approximately level.

As an alternative to the "equivalent fluid pressure method" the earth pressures can also be calculated using the analytical approach, assuming that backfill to the abutments will consist of Granular 'A' or 'B' type aggregates.

In this case, backfill for the structures should consist of granular materials, in accordance with MTO Standard Special Provision No. 121, dated October, 1983. Earth pressures acting on the wall may be computed in accordance with Section 6.6.1.2.1 of the O.H.B.D.C. assuming a non-yielding foundation where the "at rest" condition applies. The physical properties to be assumed for the backfill are as follows:

Granular "A" - $\phi = 35$, $\gamma = 22.8 \text{ kN/m}^3$ $K_o = 0.43$

Granular "B" - $\phi = 30$, $\gamma = 21.2 \text{ kN/m}^3$ $K_o = 0.50$

Construction joints should be provided between those portions of structure which can yield and those which are rigidly restrained.

Care should be given to avoid the development of large horizontal pressures when compacting the backfill behind the abutments. Vibratory compaction equipment, for use behind retaining structures, must be restricted in size as per current M.T.O. specifications.

6. CLOSURE

The conclusions and recommendations given in this report are based on information determined at the borehole locations. The soil stratigraphy and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations and subsurface conditions may become apparent during construction which could be detected or anticipated from the site investigation. Also, depending on seasonal factors, the groundwater table could be at a different level than at the time of the field work.

The recommendations given in this report are applicable only to the project described in the text and then only if constructed in accordance with the general principles stated in the report.

Yours very truly,

B.P. WALKER ASSOCIATES LTD.

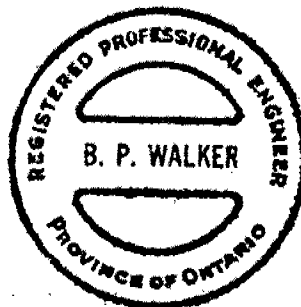
U.S. Sappal

U.S. Sappal, P.Eng.

B.P. Walker

Dr. B.P. Walker, P.Eng.

/jl





RECORD OF BOREHOLE No 1

METRIC

W P 314-85-01 LOCATION STA 16+832.4, O/S 2.0m LT # HWY 500 ORIGINATED BY T.O.
DIST N.L. HWY 560 BOREHOLE TYPE HOLLOW STEM AUGER, BXC ROCK CORE & CONE TESTS COMPILED BY U.S.S.
DATUM DATE April 30, and May 1, 1990 CHECKED BY U.S.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							W _p	W	W _L
								SHEAR STRENGTH kPa									

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION



Ministry
of
Transportation
Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 314-85-01 LOCATION STA 16+813.6, O/S 2.0m RT of HWY 560 ORIGINATED BY T.O.
DIST N.L. HWY 560 BOREHOLE TYPE BXC ROCK CORE COMPILED BY U.S.S.
DATUM _____ DATE May 4, 1990 CHECKED BY USS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT Y KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	W _p	W	W _L	WATER CONTENT (%)	10 20 30			
242.1	GROUND SURFACE															
0.0	BEDROCK-basalt with serpentine, greenish grey, sound, quartz filled fractures		1	BXC RC	REC 100%										RQD 66%	
			2	BXC RC	REC 100%										RQD 87%	
			3	BXC RC	REC 100%										RQD 85%	
239.0	END OF BOREHOLE															
3.1	Water level not established.															

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to
Sensitivity

20
15 \pm 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 3

METRIC

W P 314-85-01 LOCATION STA 16+790.8; O/S 2.0m LT of HWY 560 ORIGINATED BY T.O.
DIST W.L. HWY 560 BOREHOLE TYPE WASH BORING AND BXC ROCK CORE COMPILED BY U.S.B.
DATUM _____ DATE May 3, 1990 CHECKED BY USB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										10 20 30		
242.5	GROUND SURFACE																			
0.0	SILTY CLAY-some sand, trace gravel						242													
241.6																				
0.9	BOULDERS																			
241.3																				
1.2	BEDROCK-basalt, greenish black, massive, sound, aphanitic with granitic inclusions		1	BXC RC	REC 99.1%		241									RQD 95%				
			2	BXC RC	REC 100%		240													
							239									RQD 96%				
238.3																				
4.2	END OF BOREHOLE																			
	Water level not established.																			

OFFICE REPORT ON SOIL EXPLORATION



METRIC

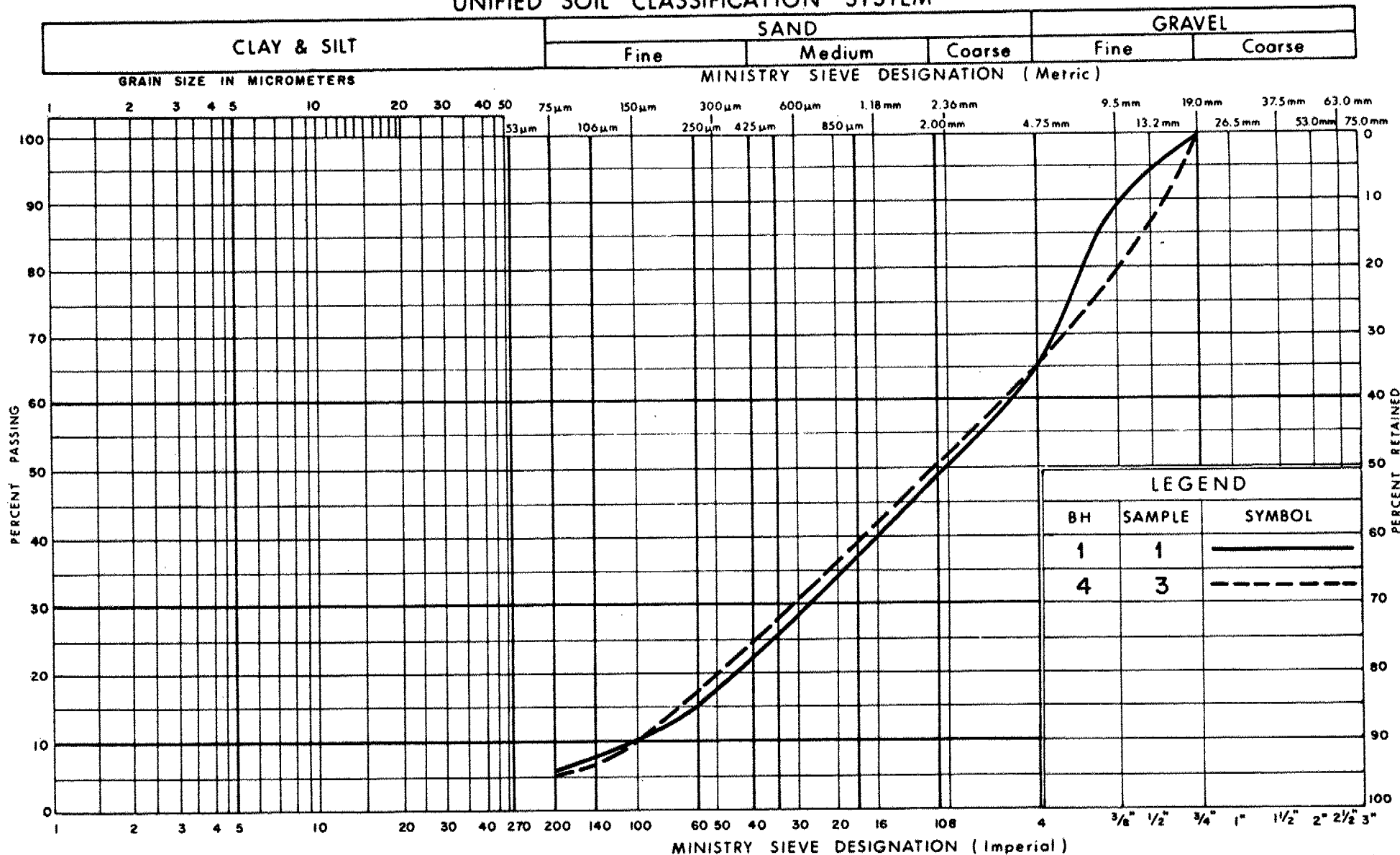
W P 314-85-01 LOCATION STA 16+772.3; D/S 2.0m RT of HWY 560 ORIGINATED BY T.O.
DIST N.L. HWY 560 BOREHOLE TYPE HOLLOW STEM AUGER, BXC ROCK CORE & CONE TEST COMPILED BY D.S.S.
DATUM _____ DATE May 2, 1990 CHECKED BY USS

[illegible]

+3, x5: Numbers refer to Sensitivity

20
✦
10

UNIFIED SOIL CLASSIFICATION SYSTEM

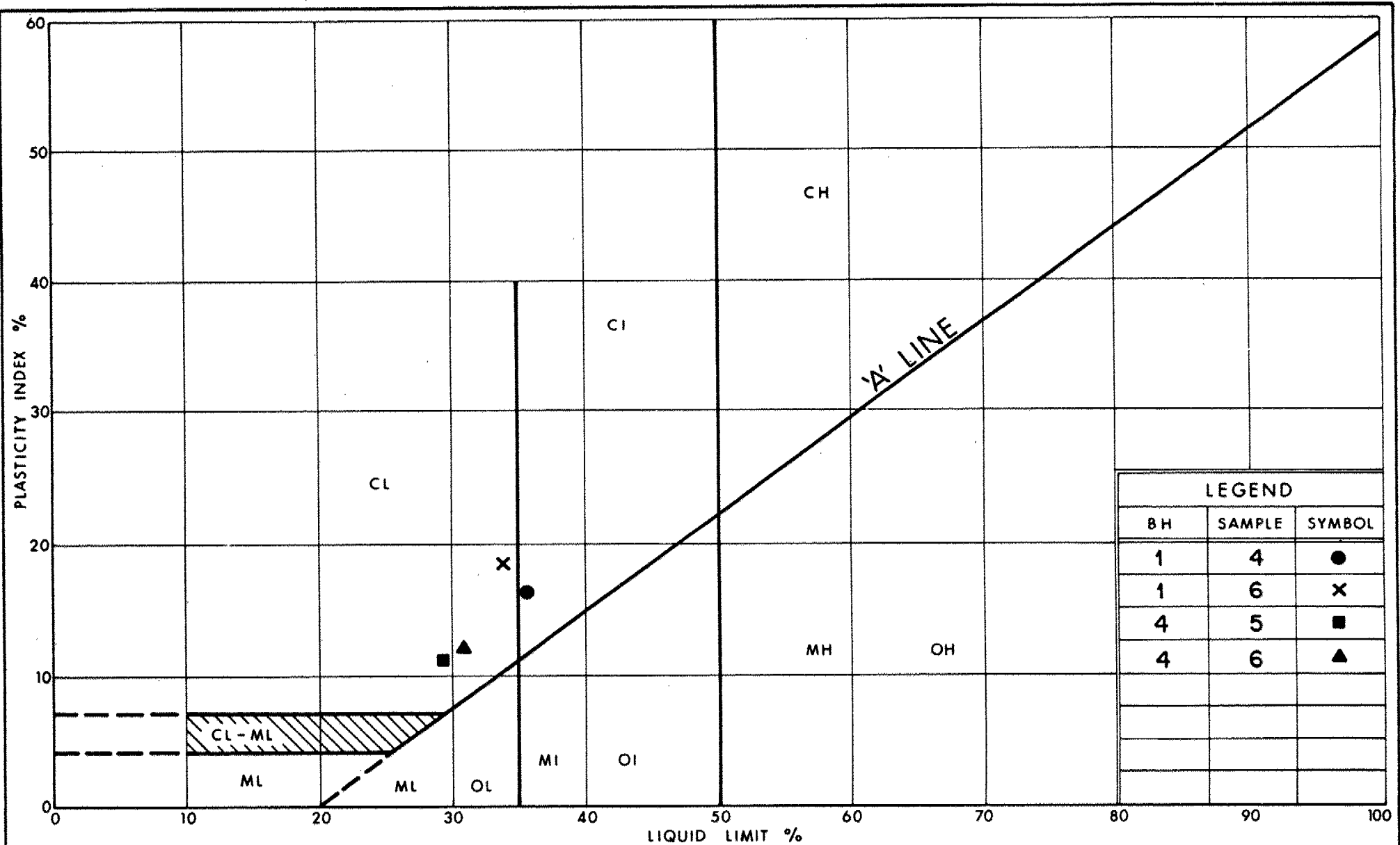


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
GRAVELLY SAND, TRACE SILT, CLAY (Fill)

FIG No 1

W P 314-85-01



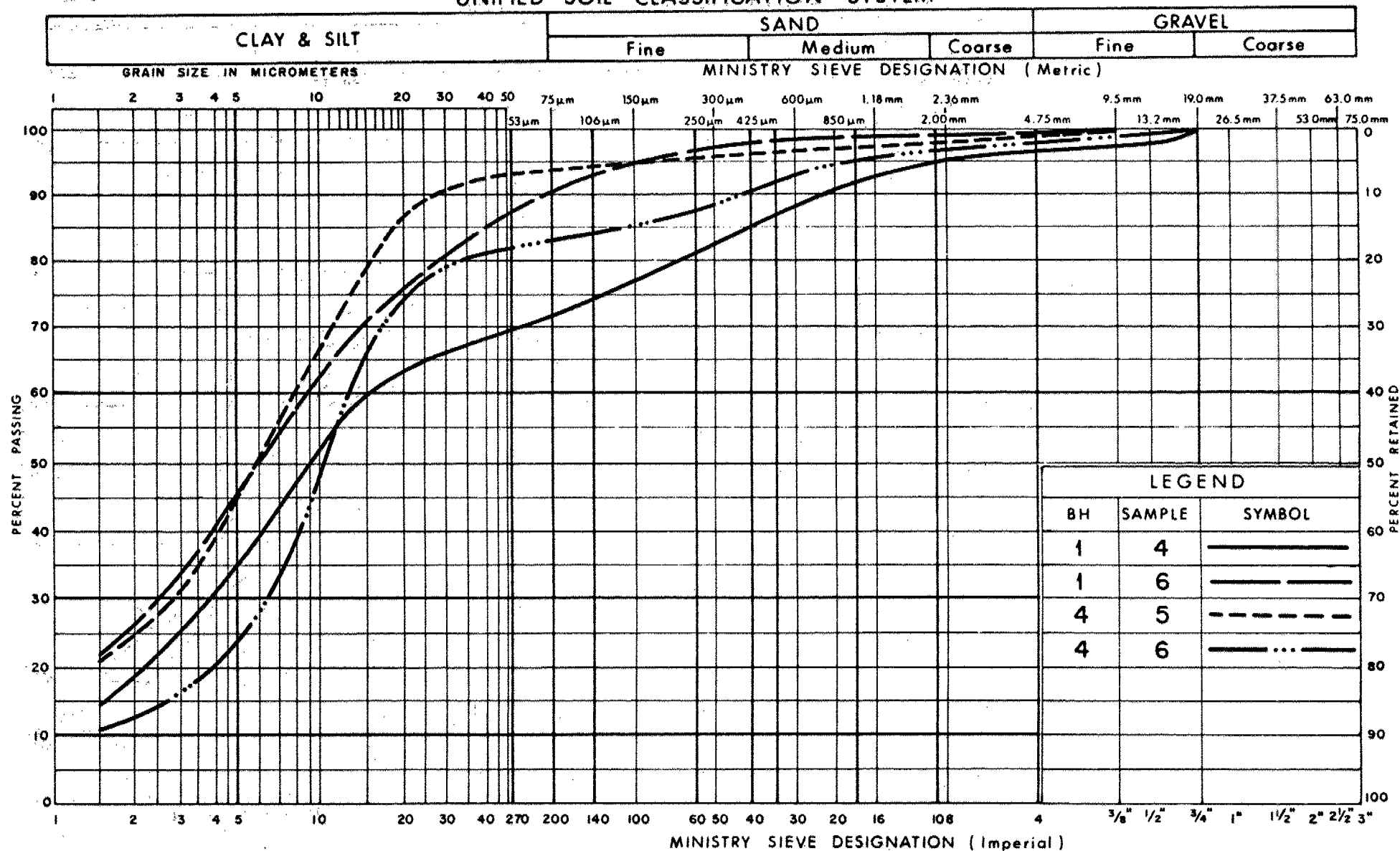
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Ontario

PLASTICITY CHART
CLAYEY SILT, SOME TO TRACE SAND, TRACE GRAVEL

FIG No 2

W P 314-85-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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Transportation

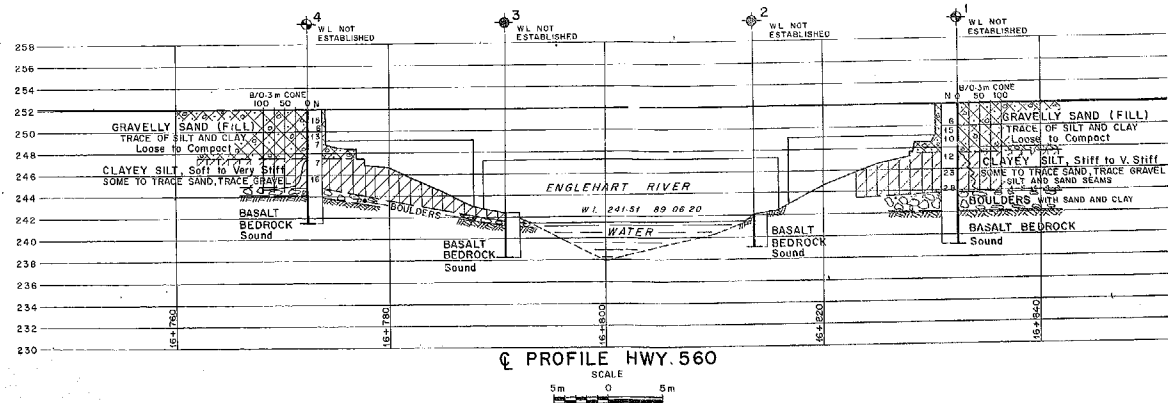
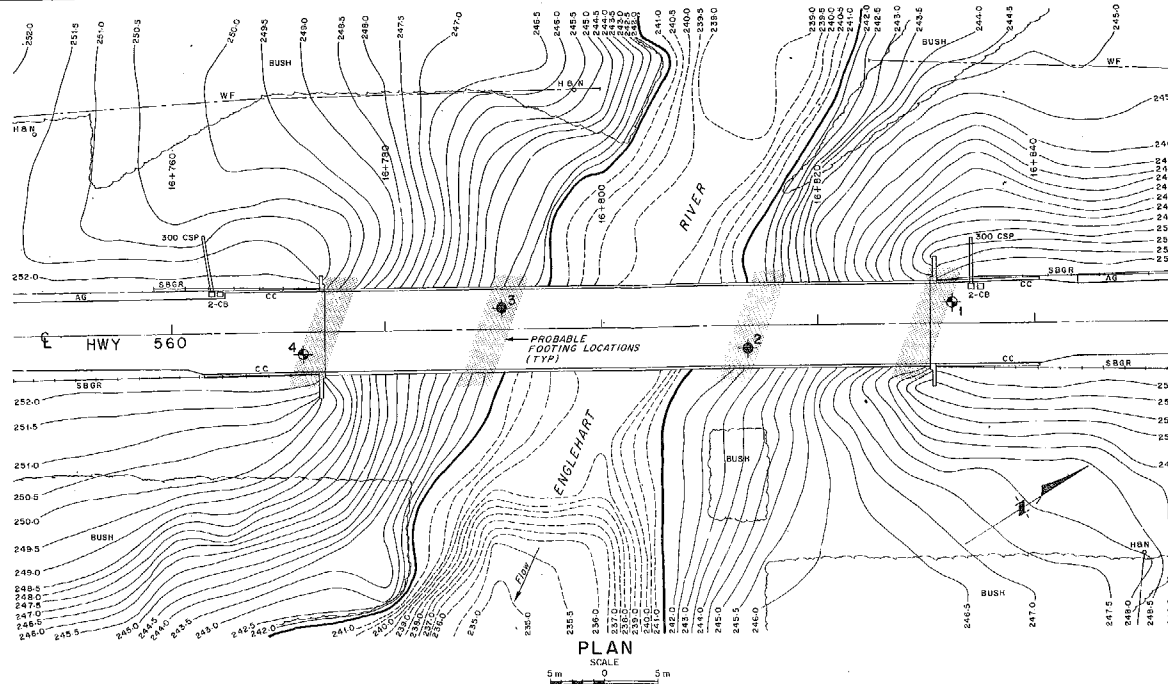
GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME TO TRACE SAND, TRACE GRAVEL

FIG No 3

W P 314-85-01

OVERSIZE DRAWING

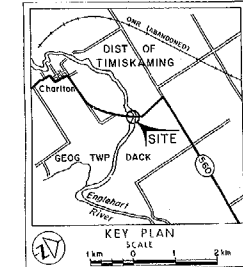
MINUTE OF TRANSFORMATION OBTAINED FROM 137-84-1



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN BRACKETS - METRES.

CONT No
WP No 314-85-01
ENGLEHART RIVER
SHEET
BORE HOLE LOCATIONS & SOIL STRATA

B. P. Walker Associates Ltd.



- LEGEND**
- Bore Hole
 - ⊕ Dynamic Cone Penetration Test (Cone)
 - ⊕ Bore Hole & 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

No	ELEVATION	STATION	OFFSET
1	252.0	16+832.4	2.0 m LI
2	242.1	16+813.6	2.0 m RI
3	242.5	16+790.8	2.0 m LI
4	252.0	16+772.3	2.0 m RI

=NOTE=
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office. However, information disclosed in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION
Geocres No 31M-53		
HWY No 560	CHECKED	DATE May 26, 1990
STANDARD U.S.	CHECKED	FORTE May 26, 1990
DRAWN P.S.	CHECKED	OFFICE

REF No E-701-560-1

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST 14
CONT No
WP No 314-85-01



SHEET

ENGLEHART RIVER BRIDGE
HWY 560 CROSSING

GENERAL ARRANGEMENT

PLANNAC CONSULTANTS LTD.
CONSULTING ENGINEERS & PLANNERS

GENERAL NOTES:

CLASS OF CONCRETE:

ALL CONCRETE 30 MPa

REINFORCING STEEL:

GRADE 400 UNLESS OTHERWISE SPECIFIED.
BAR NAMES WITH SUFFIX "C" DENOTE COATED BARS.

CLEAR COVER TO REINFORCING STEEL:

POSTTENSIONING 50-100
ABUTMENTS & PIERS 50-100
PIERS 50-100
DECK - TOP 70-100
- BOTTOM 40-100
REINFORCER (UNLESS OTHERWISE NOTED) 70-100

CONSTRUCTION NOTE:

IF THE ACTUAL BEARING HEIGHTS DIFFER FROM THE ASSUMED HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS.

LIST OF DRAWINGS:

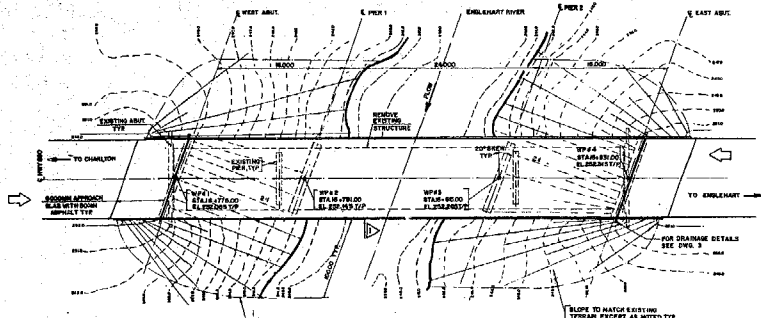
- 1 GENERAL ARRANGEMENT
- 2 BONE HOLE LOCATIONS & HOLE STRATA
- 3 FOOTING LAYOUT & DETAILS
- 4 WEST ABUTMENT
- 5 EAST ABUTMENT
- 6 PIERS
- 7 STRUCTURAL STEEL I
- 8 STRUCTURAL STEEL II
- 9 BEARINGS
- 10 DECK DETAILS & BORED ELEVATIONS
- 11 BARRIER WALL
- 12 BORDING APPROACH PLANS
- 13 BORDING APPROACH & BRIDGING
- 14 AS CONSTRUCTED ELEVATIONS & CONSIDERING
- 15 STANDARD DETAILS
- 16 QUANTITIES - STRUCTURE I
- 17 QUANTITIES - STRUCTURE II

APPLICABLE STANDARD DRAWINGS:

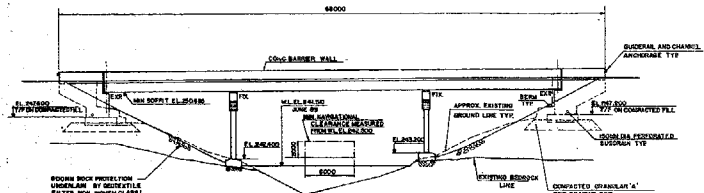
TO SUIT NEAREST AVAILABLE SCHEDULE REQUIREMENT.



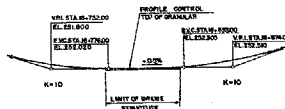
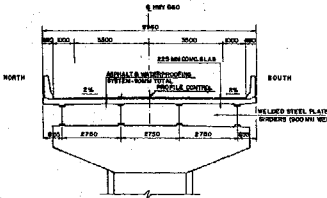
DATE	BY	DESCRIPTION
1985	PL	PRELIMINARY DESIGN
1985	PL	FINAL DESIGN
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION
1985	PL	FOR CONSTRUCTION



PLAN
1:200



ELEVATION
1:200



PROFILE OF HWY 560
N.T.S.

BM 258.253
TOP OF 818
U.S. N 4-122.8



DRAWING NOT TO BE SCALED

100 mm ON CHANNEL DRAWING

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST.
CONT No
WP No 514-85-01

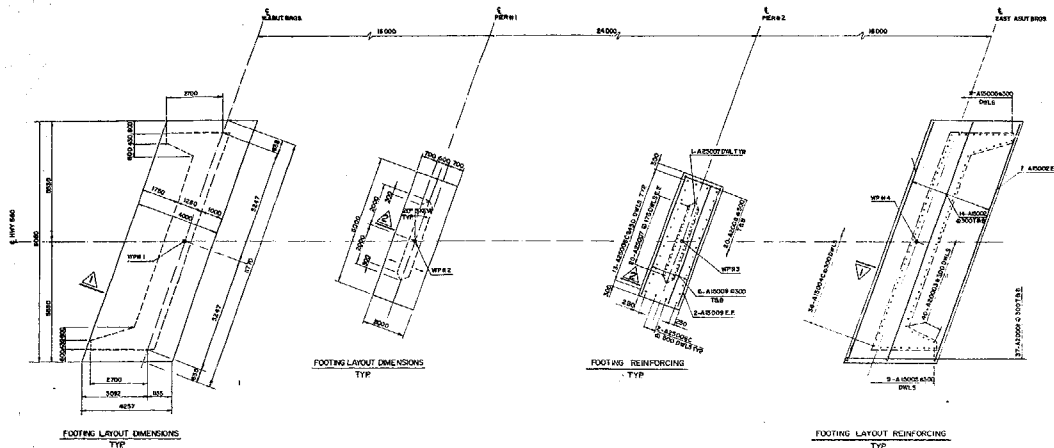


ENGLHART RIVER BRIDGE
HWY 550 CROSSING

SHEET

FOOTING LAYOUT & DETAILS

PLANMAC CONSULTANTS LTD.
CONSULTING ENGINEERS & PLANNERS

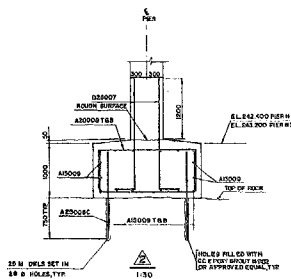
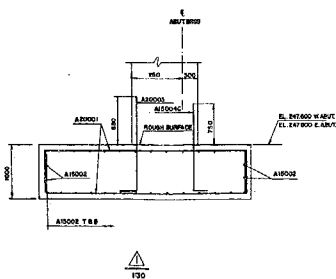


PLAN

1:75

NOTE:

16M BARS IN ROCK TO HAVE AN ULTIMATE PULL-OUT
CAPACITY OF 200kN.



DRAWING NOT TO BE SCALE
FOR ANY CIVIL ENGINEERING

NO.	DATE	BY	DESCRIPTION
1	1985	PLANMAC	ENGLHART RIVER BRIDGE
2	1985	PLANMAC	FOOTING LAYOUT & DETAILS
3	1985	PLANMAC	FOOTING LAYOUT & DETAILS
4	1985	PLANMAC	FOOTING LAYOUT & DETAILS
5	1985	PLANMAC	FOOTING LAYOUT & DETAILS