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DIST. 18 REGION

W.P. No. 46-87-01

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W. O. No.

STR. SITE No. 388-340

HWY. No. 556

LOCATION Hwy 556 & Aubinadong River

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Ministry
of
Transportation
Ontario

FILE No. _____ **DATE** _____

REMARKS NEARBY JOB WP 52-86-01
GEOCRES # 41J-46

ADM-G-24 79-06

FOUNDATION INVESTIGATION REPORT

CONTRACT NO. 93-229

Ontario

Ministry of
Transportation

INDEX

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

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX = $w_L - w_p$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

FOUNDATION INVESTIGATION REPORT
FOR
AUBINADONG RIVER BRIDGE
WP 46-87-00; SITE 385-380
HIGHWAY 556, SAULT STE. MARIE
DIST. 18; REG. NORTHWEST

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 proposed Aubinadong River Bridge on Highway 556 in Sault Ste. Marie. The proposed structure will replace an existing structure located approximately 1.0km north of the site investigated. Conceptual design data regarding the project was transmitted to us by the Ministry.

2. SITE & GEOLOGY

The site is located on the Aubinadong River at Highway 556 approximately 100km Northeast of Sault Ste. Marie. The area is part of the Canadian Shield. In this area the Glacio-fluvial subsoils were deposited as outwash and consist of a thin surficial deposit of sand and gravel overlying sand and sandy silt. The sand and gravel deposit contains frequent cobbles and occasional boulders. However, the sandy silt contains a minimum of pebbles and boulders. Bedrock underlies the entire area at some depth but no outcrops occur.

3. FIELD AND LABORATORY WORK

Four boreholes, numbered 1 to 4* were drilled at the location of the proposed foundation options as shown on the borehole location plan, Drawing 468701-A**. Dynamic cone penetration tests were carried out next to the boreholes and at the west pier

* BORE HOLES 9&10 ADDED IN 1993 07
** DWG NO 2 OF THE CONTRACT DWG'S

foundation location for the proposed 3 span bridge. In addition shallow boreholes were carried out for the bridge approaches.

The boreholes 4, 5 and 6 east of the river were drilled on March 19th and 20th, 1991 to depths of 20.2m and 5.2m. Borehole #3 on the east bank of the river and boreholes 1, 2 and 8 and cone test #7, west of the river, could not be carried out during the above dates due to access problems. A temporary road had to be cut along the steep slope immediately west of the proposed west foundation location. These boreholes and the cone tests were carried out on June 26th and 27th, 1991. A bombardier mounted drilling rig, equipped with continuous flight hollow stem augers, was used for advancing the holes. The drilling, sampling and the field testing procedures were supervised and the borings were logged by an experienced geotechnical engineer from our office.

Samples were taken with a 51mm o.d. split spoon (SS) in accordance with ASTM D 1586-84, STANDARD 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.

4. SUBSURFACE CONDITIONS

4.1 East Bank

The subsoils east of the river are quite uniform and consist of a surficial, compact to very dense deposit of sand and gravel, trace silt over an average of 3.2m thick silty sand, trace clay and gravel deposit. Underlying the silty sand is a deposit of sandy silt, trace clay. This deposit continues to the maximum depth of boreholes.

4.2 West Bank

Peat varying in depth from 0.5m at borehole 2 to 1.3m at borehole 1 was encountered west of the river. Underlying the peat is a very loose deposit of silty sand which contains organic layers. Underlying the silty sand at borehole 1 is a deposit of sand with gravel and cobbles. The thickness of the deposit was approximately 2.5m. The sand and gravel layer was not encountered at borehole 2.

Underlying the sand and gravel deposit at borehole 1 and the silty sand at borehole 2 is a deposit of sandy silt trace clay. This deposit continues to the maximum depth of the boreholes.

The subsoils encountered at the boreholes are as follows:

4.3 Sand and gravel, trace of silt, frequent cobbles

This deposit was encountered from the surface at all boreholes on the east side of the river and under the very loose silty sand at borehole 1 on the west side of the river. The thickness of the deposit varies from 0.6m at borehole 3 to 5.2m at borehole 4.

The Standard Penetration tests gave N-values of 14 to 90 blows per 30cm. These Standard Penetration values indicate that the deposit is compact to very dense.

The grain size distribution performed on a representative sample from this deposit is shown in Figure 1.

4.4 Silty sand, trace clay and gravel

4.4.1 East side

Underlying the sand and gravel, trace silt is a deposit of silty sand trace clay and gravel. This material is fine to medium grained. The thickness of the deposit varies from 4.0m at borehole 3 to 3.3m at borehole 4.

The Standard Penetration test results gave N-values in the range of 6 to 21 blows per 30cm. These Standard Penetration tests indicate that the deposit is loose to compact.

The grain size distribution performed on a representative sample from the deposit is shown on Figure 2.

4.4.2 West side

The silty sand deposit underlying a deposit of peat varying in thickness from 1.3m to 0.5m, has very thin organic layers and rootlets and represents a thin mantle of talus derived from the periodic weathering of the steep slope immediately west of borehole 1.

The Standard Penetration tests gave N-values of 1 to 9 blows per 30cm indicating that the deposit is very loose to loose. The moisture content at the deposit was also found to be high, varying from 21% to 49%.

4.5 Sandy silt, trace of clay

This deposit was encountered at an average elevation of 333m. The lower boundary of the deposit was not established at any of the boreholes as this deposit continues to the maximum depth of all boreholes.

The Standard Penetration tests gave N-values ranging from 9 to 32 blows per 30cm. These Standard Penetration results indicate that the deposit is compact.

The results of grain size distribution tests performed on representative samples from this deposit are shown on Figure 3.

Atterburg Tests carried out on two representative samples from this deposit indicated that the sandy silt, trace of clay is non-plastic.

5. GROUND WATER CONDITIONS

The ground water levels recorded in the open boreholes on completion are shown on the record of borehole logs and Drawing 468701-A. The water level in Boreholes 3 and 4 was slightly higher than the water level in the river. The water level in borehole 1 probably represents water contained in the peat deposit. The water level at borehole 2 was not established.

Note: The preceding report is a copy of the factual information from the Foundation Report prepared by B.P. WALKER ASSOCIATES LTD. (consulting geotechnical engineers for this project), under the technical supervision of the M.T.O. Foundation Design Section.

B. Iyer, P. Eng.
Senior Foundation Engineer

M.S. Devata, P. Eng.
Chief Foundation Engineer

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 46-87-01 LOCATION STA. 13 + 733.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS & CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 27, 1991 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
338.2	GROUND SURFACE										
0.0 / 336.9	DEAD VEGETATION; Peat		1	SS	4	▽	338		45.9%		
1.3 / 335.2	SILTY SAND - fine to med. grained; sat.; very loose; dark brown occ. thin org. layers		2	SS	2		336				
			3	SS	3						
3.0 / 332.7	SAND WITH GRAVEL AND COBBLES; dense to very dense		4	SS	39		334				
			5	SS	51						
5.5	SANDY SILT - trace clay; compact		6	SS	16		332				
			7	SS	6*		330				
			8	SS	14						
			9	SS	14		328				
325.6			10	SS	16		326				
12.6 / 315.9	END OF BOREHOLE						324				
							322				
							320				
							318				
							316				
22.3	END OF CONE TEST										

* Low N value probably due to loosening of soil by water pressure.

+3, x5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 46-87-01 LOCATION STA. 13 + 753.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS AND CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 27, 1991 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPo: ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
337.5	GROUND SURFACE										
337.0	PEAT					*					
0.5	SILTY SAND WITH ORGANIC LAYERS - very fine grained; wet; very loose to loose		1	SS	2						
			2	SS	2		336				
			3	SS	1				49%		
333.5			4	SS	9		334				
4.0	SANDY SILT - trace clay; saturated; compact		5	SS	9						
			6	SS	10		332				0 8 85 7
			7	SS	10		330				
			8	SS	17		328				
			9	SS	14		326				
324.9			10	SS	18						
12.6	END OF BOREHOLE						324				
							322				
							320				
							318				
314.6							316				
22.9	END OF CONE TEST										

* W.L. NOT ESTABLISHED

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 46-87-01 | LOCATION STA. 13 + 798.0; CENTRELINE HWY. 556 | ORIGINATED BY C.N.

DIST 18 HWY 556 | BOREHOLE TYPE HOLLOW STEM AUGERS AND CONE TEST | COMPILED BY U.S.

DATUM GEODETIC | DATE JUNE 26, 1991 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, x LAB VANE)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
337.2	**GROUND SURFACE**										
0.0 / 336.6	**SAND AND GRAVEL**										
0.6	**SILTY SAND**- trace clay and gravel; loose to compact		1	SS	7		336		o		
			2	SS	6				o		
			3	SS	6		334		o		0 85 10 5
			4	SS	12				o		
332.6											
4.6	**SANDY SILT** - trace clay; compact		5	SS	11		332		o		0 10 82 8
			6	SS	14				o		
			7	SS	12		330		o		
			8	SS	10		328		o		
			9	SS	17		326		o N.P.		0 7 87 6
			10	SS	18				o		
323.0			11	SS	18		324		o		
14.2	**END OF BOREHOLE**										
317.4							322				
							320				
19.8	**END OF CONE TEST**						318				

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

$\frac{20}{15 \diamond 5}$ (%) STRAIN AT FAILURE
10

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

METRIC

W P 46-87-01 LOCATION STA. 13 + 816.5M; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS AND WASH BORING; CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE MARCH 19 & 20, 1991 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS
341.4	**GROUND SURFACE**					
0.0	**SAND AND GRAVEL** - trace silt; frequent cobbles; compact to very dense		1	SS	90	
			2	SS	25	
			3	SS	45	
			4	SS	41	
336.2			5	SS	43	
5.2	**SILTY SAND** - trace clay; compact		6	SS	16	▼
332.9			7	SS	21	
8.5	**SANDY SILT** - trace clay; grey; saturated; compact		8	SS	19	
			9	SS	18	
			10	SS	23	
			11	SS	11	
			12	SS	20	
			13	SS	25	
			14	SS	28	
321.1			15	SS	32	
20.3	**END OF BOREHOLE**					
320.1						
21.3	**END OF CONE TEST**					

ELEVATION SCALE: 340, 338, 336, 334, 332, 330, 328, 326, 324, 322

DYNAMIC CONE PENETRATION RESISTANCE PLOT — 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

N.P.

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

79 18 (3)

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 46-87-01 LOCATION STA. 13 + 826.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY U.S.

DATUM GEODETIC DATE MARCH 20, 1991 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
341.7	GROUND SURFACE										
0.0	SAND AND GRAVEL - trace silt; frequent cobbles; compact to very dense		1	SS	57	*			○		
			2	SS	40		340		○		
			3	SS	63				○		
			4	SS	27		338		○		
336.7			5	SS	14				○		
5.0	END OF BOREHOLE										

* BOREHOLE DRY ON COMPLETION

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

20
15 ⊹ 5 (%) STRAIN AT FAILURE
10

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 6

METRIC

W P 46-87-01 LOCATION STA. 13 + 836.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY U.S.

DATUM GEODETIC DATE MARCH 20, 1991 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
342.2	**GROUND SURFACE**										
0.0	**SAND AND GRAVEL** - trace silt; frequent cobbles; compact to very dense		1	SS	53	*	342		○		
			2	SS	30/15cm						
			3	SS	72		340		○		
			4	SS	19				○		
							338				
337.2			5	SS	19				○		
5.0	**END OF BOREHOLE**										

*** BOREHOLE DRY ON COMPLETION**

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 $\diamond$ 5 (%) STRAIN AT FAILURE (20 / 10)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 7

METRIC

W P 46-87-01 LOCATION STA. 13 + 756.3; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 27, 1991 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) SHEAR STRENGTH kPo ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
336.8	GROUND SURFACE						336				
							334				
							332				
							330				
							328				
							326				
							324				
							322				
							320				
319.4											
17.4	END OF CONE TEST										

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 (20, 10) 5 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 8

METRIC

W P 46-87-01 LOCATION STA. 13 + 740.0; O/S 10.0m R.T. CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE PROBE HOLE (H.S.A.) COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 26, 1991 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
338.2	GROUND SURFACE										
337.9	PEAT						338				
0.3	SILTY SAND - loose; brown; saturated										
							336				
335.2			1	AS	-				o		
3.0	END OF PROBE HOLE										

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 9 1 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta 13+750.4 CL ORIGINATED BY DK

DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

DATUM Geodetic DATE 93 07 07 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE 10 20 30 40 50	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
337.5	Ground Surface										
0.0						▼*	338				
							334				
			1	SS	8		332	AUGER			
	Silt						330				
	Trace Sand and Clay		2	SS	13		328				
	Grey, Loose to Dense						326				
							324				
			3	SS	32		322				
							320				
			4	SS	45		318				
317.7											
19.8								WASHBORING			
			5	SS	12		316				
	Sandy Silt, Trace Clay						314				
	Grey, Compact						312				
			6	WS	–						
310.1							310				
27.4			7	SS	15		308				

Continued

Continued

$+^3$, $\times^5$: Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 9 2 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta 13+750.4 CL ORIGINATED BY DK

DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

DATUM Geodetic DATE 93 07 07 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 10 20 30 40 50	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
	Continued										
30.5	Silty Sand Grey, Compact		8	SS	27		306	WASHBORING			
303.5			9	SS	26		304				
34.0	End of Borehole * 93 07 07										

+^{3}, ×^{5} : Numbers refer to Sensitivity

20
15-◇-5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 10 1 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta. 13+799 o/s Rt 1.5 m ORIGINATED BY DK

DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

DATUM Geodetic DATE 93 07 06 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR SA SI CL
337.3	Ground Surface										
0.0							336				
							334				
			1	SS	10		332				
	Silt						330	AUGER			
	Trace Sand and Clay		2	SS	8		328				
	Grey, Loose to Compact						326				
			3	SS	18		324				
							322				
317.5			4	SS	25		320				
19.8							318				
			5	SS	18		316				
	Sandy Silt, Trace Clay						314	WASHBORING			
			6	SS	8		312				
	Grey, Loose to Compact						310				
308.0			7	SS	27		308				
29.3											

Continued

+3, ×5 : Numbers refer to Sensitivity

20
15-◇-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 10 2 OF 2 METRIC

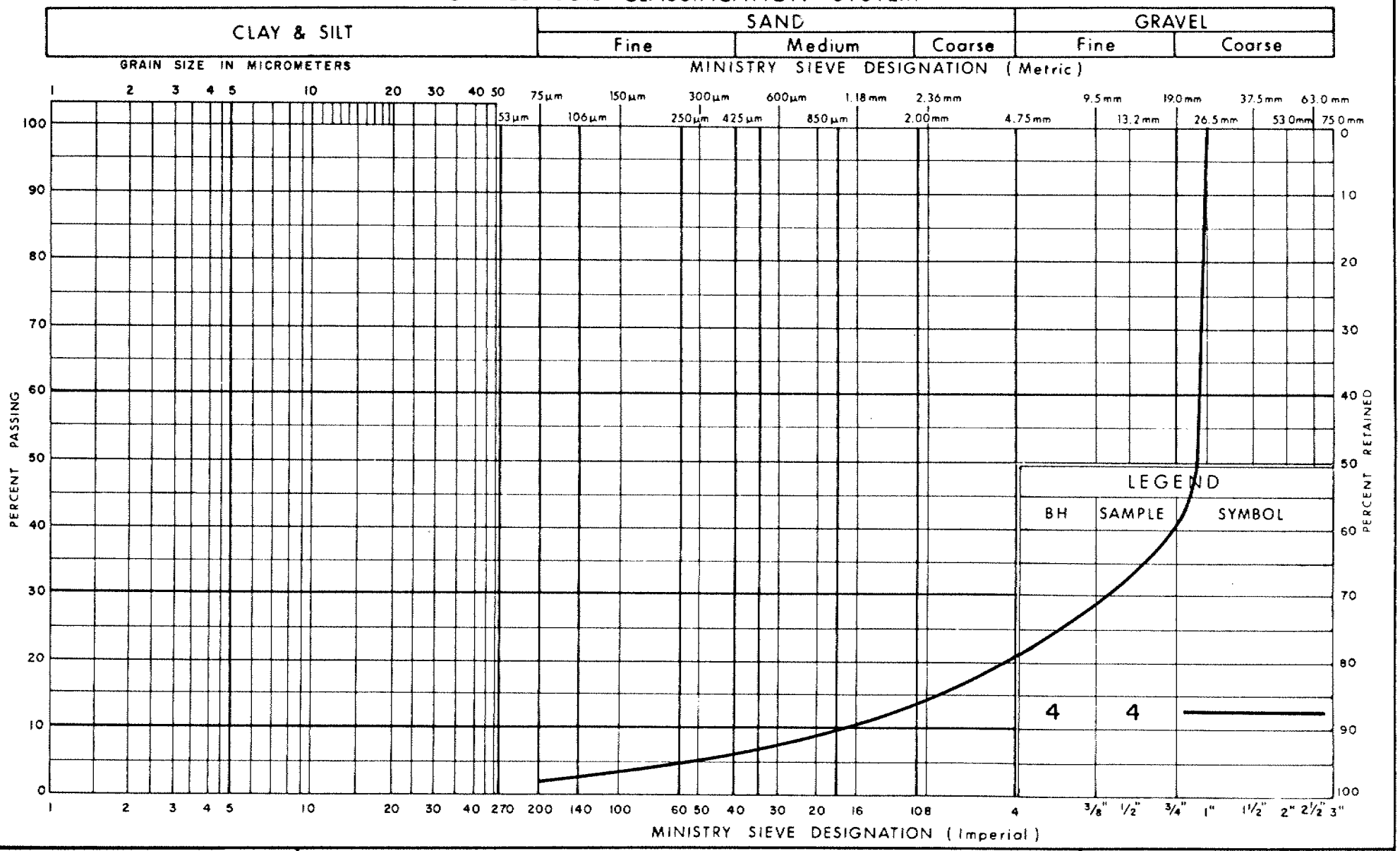
W.P. 46-87-01 LOCATION Sta. 13+799 o/s Rt 1.5 m ORIGINATED BY DK
DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK
DATUM Geodetic DATE 93 07 06 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, • QUICK TRIAXIAL, × LAB VANE; 10 20 30 40 50)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
30.5	Continued		8	SS	100	/15cm	306	WASHBORING			
	Silty Sand and Gravel Pinkish Grey, Very Dense										
303.3			9	WS	–		304				
34.0	End of Borehole * 93 07 07										

$+^3$, $\times^5$: Numbers refer to Sensitivity

$\frac{20}{15 \diamond 5}$ (%) STRAIN AT FAILURE
10

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of Transportation
Ontario

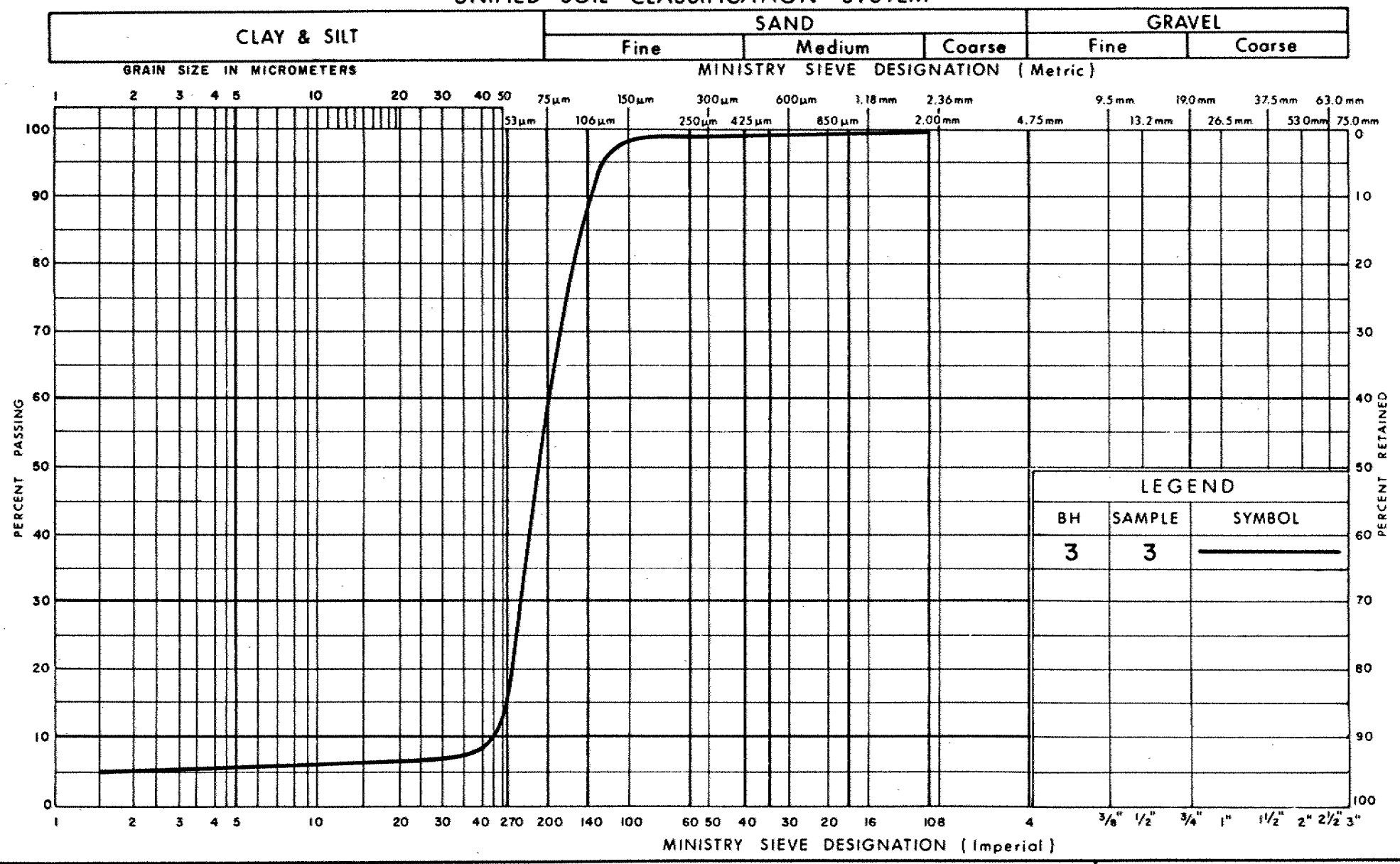
GRAIN SIZE DISTRIBUTION

SAND & GRAVEL, TRACE OF SILT

FIG No 1

W P 46-87-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of Transportation
Ontario

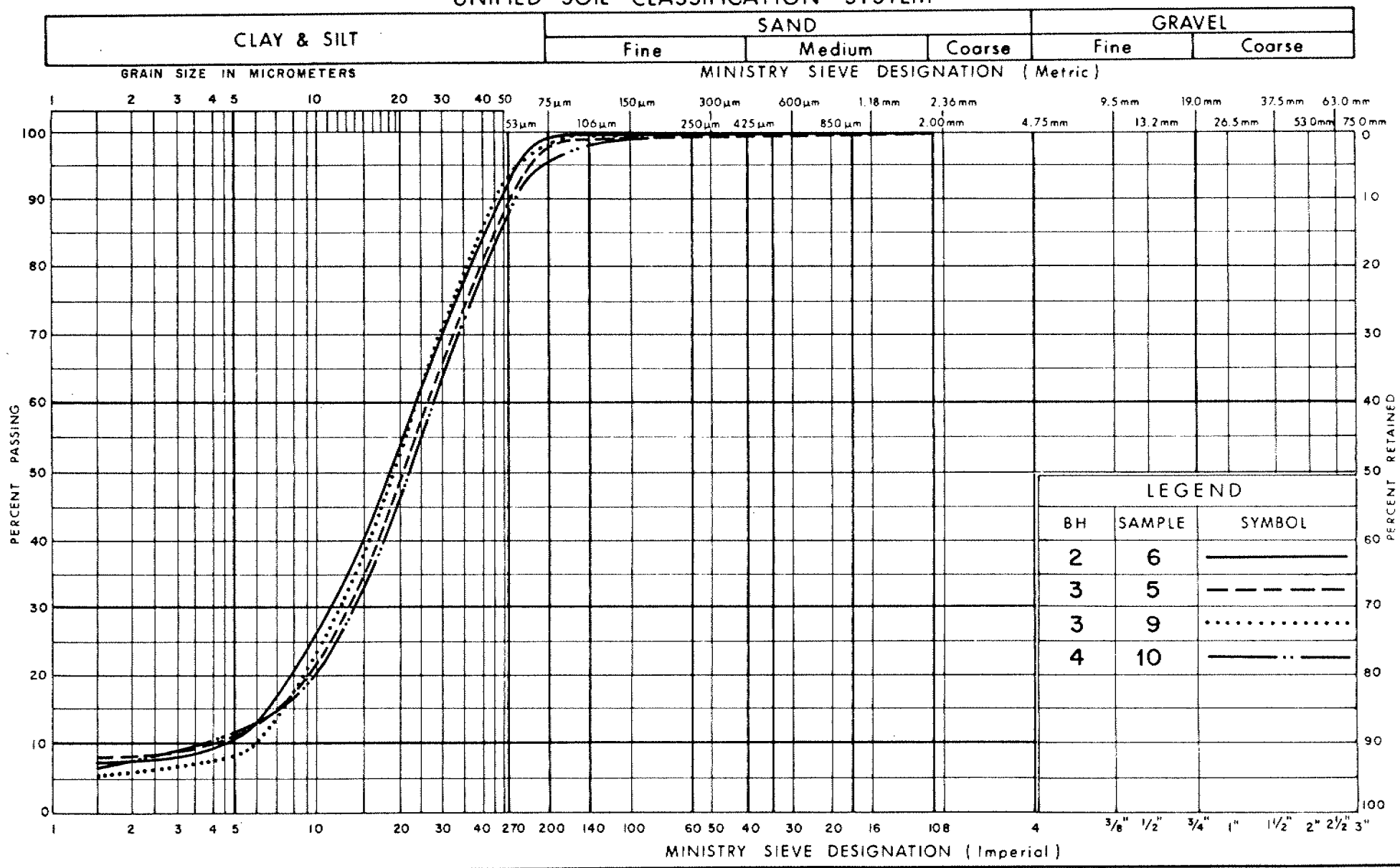
GRAIN SIZE DISTRIBUTION

SILTY SAND, TRACE OF CLAY

FIG No 2

WP 46-87-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT, TRACE OF CLAY

FIG No 3

W P 46-87-01

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B.P. WALKER
Associates Limited

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REPORT ON FOUNDATION INVESTIGATION

FOR PROPOSED REPLACEMENT OF

AUBINADONG RIVER BRIDGE

WP 46-87-01: SITE 38S-340

HIGHWAY 556, SAULT STE. MARIE DIST.18 REG. NORTHWEST

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 AUBINADONG RIVER BRIDGE

WP 46-87-00: SITE 38S-~~300~~ 340

HIGHWAY 556, SAULT STE. MARIE DIST. 18 REG. NORTHWEST

Prepared for:

Ministry of Transportation of Ontario

Foundation Design Section

1201 Wilson Avenue

Downsview, Ontario

M3M 1J8

Project No. 2398-0291

August 6, 1991

FOUNDATION INVESTIGATION REPORT
FOR
AUBINADONG RIVER BRIDGE
WP 46-87-00; SITE 385-30
HIGHWAY 556, SAULT STE. MARIE
DIST. 18; REG. NORTHWEST

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 proposed Aubinadong River Bridge on Highway 556 in Sault Ste. Marie. The proposed structure will replace an existing structure located approximately 1.0km north of the site investigated. Conceptual design data regarding the project was transmitted to us by the Ministry.

2. SITE & GEOLOGY

The site is located on the Aubinadong River at Highway 556 approximately 100km Northeast of Sault Ste. Marie. The area is part of the Canadian Shield. In this area the Glacio-fluvial subsoils were deposited as outwash and consist of a thin surficial deposit of sand and gravel overlying sand and sandy silt. The sand and gravel deposit contains frequent cobbles and occasional boulders. However, the sandy silt contains a minimum of pebbles and boulders. Bedrock underlies the entire area at some depth but no outcrops occur.

3. FIELD AND LABORATORY WORK

Four boreholes, numbered 1 to 4 were drilled at the location of the proposed foundation options as shown on the borehole location plan, Drawing 468701-A. Dynamic cone penetration tests were carried out next to the boreholes and at the west pier

foundation location for the proposed 3 span bridge. In addition shallow boreholes were carried out for the bridge approaches.

The boreholes 4, 5 and 6 east of the river were drilled on March 19th and 20th, 1991 to depths of 20.2m and 5.2m. Borehole #3 on the east bank of the river and boreholes 1, 2 and 8 and cone test #7, west of the river, could not be carried out during the above dates due to access problems. A temporary road had to be cut along the steep slope immediately west of the proposed west foundation location. These boreholes and the cone tests were carried out on June 26th and 27th, 1991. A bombardier mounted drilling rig, equipped with continuous flight hollow stem augers, was used for advancing the holes. The drilling, sampling and the field testing procedures were supervised and the borings were logged by an experienced geotechnical engineer from our office.

Samples were taken with a 51mm o.d. split spoon (SS) in accordance with ASTM D 1586-84, STANDARD 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.

4. SUBSURFACE CONDITIONS

4.1 East Bank

The subsoils east of the river are quite uniform and consist of a surficial, compact to very dense deposit of sand and gravel, trace silt over an average of 3.2m thick silty sand, trace clay and gravel deposit. Underlying the silty sand is a deposit of sandy silt, trace clay. This deposit continues to the maximum depth of boreholes.

4.2 West Bank

Peat varying in depth from 0.5m at borehole 2 to 1.3m at borehole 1 was encountered west of the river. Underlying the peat is a very loose deposit of silty sand which contains organic layers. Underlying the silty sand at borehole 1 is a deposit of sand with gravel and cobbles. The thickness of the deposit was approximately 2.5m. The sand and gravel layer was not encountered at borehole 2.

Underlying the sand and gravel deposit at borehole 1 and the silty sand at borehole 2 is a deposit of sandy silt trace clay. This deposit continues to the maximum depth of the boreholes.

The subsoils encountered at the boreholes are as follows:

4.3 Sand and gravel, trace of silt, frequent cobbles

This deposit was encountered from the surface at all boreholes on the east side of the river and under the very loose silty sand at borehole 1 on the west side of the river. The thickness of the deposit varies from 0.6m at borehole 3 to 5.2m at borehole 4.

The Standard Penetration tests gave N-values of 14 to 90 blows per 30cm. These Standard Penetration values indicate that the deposit is compact to very dense.

The grain size distribution performed on a representative sample from this deposit is shown in Figure 1.

4.4 Silty sand, trace clay and gravel

4.4.1 East side

Underlying the sand and gravel, trace silt is a deposit of silty sand trace clay and gravel. This material is fine to medium grained. The thickness of the deposit varies from 4.0m at borehole 3 to 3.3m at borehole 4.

The Standard Penetration test results gave N-values in the range of 6 to 21 blows per 30cm. These Standard Penetration tests indicate that the deposit is loose to compact.

The grain size distribution performed on a representative sample from the deposit is shown on Figure 2.

4.4.2 West side

The silty sand deposit underlying a deposit of peat varying in thickness from 1.3m to 0.5m, has very thin organic layers and rootlets and represents a thin mantle of talus derived from the periodic weathering of the steep slope immediately west of borehole 1.

The Standard Penetration tests gave N-values of 1 to 9 blows per 30cm indicating that the deposit is very loose to loose. The moisture content at the deposit was also found to be high, varying from 21% to 49%.

4.5 Sandy silt, trace of clay

This deposit was encountered at an average elevation of 333m. The lower boundary of the deposit was not established at any of the boreholes as this deposit continues to the maximum depth of all boreholes.

The Standard Penetration tests gave N-values ranging from 9 to 32 blows per 30cm. These Standard Penetration results indicate that the deposit is compact.

The results of grain size distribution tests performed on representative samples from this deposit are shown on Figure 3.

Atterburg Tests carried out on two representative samples from this deposit indicated that the sandy silt, trace of clay is non-plastic.

5. GROUND WATER CONDITIONS

The ground water levels recorded in the open boreholes on completion are shown on the record of borehole logs and Drawing 468701-A. The water level in Boreholes 3 and 4 was slightly higher than the water level in the river. The water level in borehole 1 probably represents water contained in the peat deposit. The water level at borehole 2 was not established.

6. DISCUSSIONS AND RECOMMENDATIONS

6.1 Project

A new bridge is proposed at this location, approximately 1.0km downstream from an existing bridge on Highway 556, as a part of realigning Highway 556. Preliminary information provided by the Ministry of Transportation of Ontario indicates that the proposed bridge will be either a 42m long single span bridge (option 1) or a 3 span bridge with side spans of 24m each and a 35m middle span (option 2).

The proposed grade for the new highway indicates that additional fill on top of the present ground level on the east side of the river will vary from approximately 7.0m at borehole 3 to approximately 2.4m at borehole 4. On the west side of the river the height of the embankment will vary from approximately 6.0m at borehole 1 to approximately 7.5m at borehole 2.

6.2 Foundations

6.2.1 Option 1

Single span structure (Boreholes 2 and 3). Closed type of abutments are contemplated for this option.

Three alternatives are considered as foundation alternatives for this option: timber piles, spread foundations on natural soil and spread footings on engineered fill.

6.2.1.1. Timber Piles

The thickness of the sand and gravel layer with frequent cobbles at borehole 3 was very small and was not encountered at borehole 2. Therefore it will be possible to drive piles without pre-augering.

No end bearing stratum was encountered within the borehole and the dynamic cone penetration test depths. Therefore friction piles will have to be used as one of the foundation alternatives. We recommend driven timber piles.

For a timber pile with an embedded depth of approximately 14.0m and Butt and Tip diameters of approximately 360mm and 240mm respectively, the Factored Axial load capacity at Ultimate Limit StateS is estimated at 400kN.

The Axial load capacity at Serviceability Limit States type II (qs) is estimated at 200kN.

These estimated axial loads are based on load tests carried out by Ministry of Transportation in similar subsoils (Ref. to M.T.O. report EM-48; sites 12 and 13). However, we recommend that a load test be carried out to confirm these load capacities.

Battered timber piles should be used to resist the horizontal earth pressures.

6.2.1.2 Spread Footings on Natural Soil

Shallow spread footings placed at an elevation of approximately 334m is another foundation alternative.

The Bearing capacity of spread footings at the above elevation was calculated using the following data and assumptions:

Average N-value below the founding elevation - 10
Footing Width - 6.0m
Footing Length - 9.0m
Footing Depth - 3.0m
Submerged unit weight of soil - 7.5kN/m^3

The Factored Bearing capacity at Ultimate Limit States, based on the above parameters, is 340kPa.

The Bearing capacity at Serviceability Limit States Type II (qs) is 100kPa.

The total and differential settlement between abutments of the structure, using the above recommended design bearing pressures will be less than 25mm and 15mm respectively.

For the evaluation of the sliding resistance of footings on the silty sand, the angle of friction between the concrete and the silty sand should be taken as 30 degrees, unfactored coefficient of friction of 0.58.

6.2.1.3 Spread Footings on Engineered Fill

Placing the footings on engineered fill will require excavation to an elevation of 334.0m, approximately 0.6m below the river water level recorded on September 5th, 1990. In order to avoid any expensive dewatering scheme, the fill to the water level should consist of well graded crushed rock fill. This type of fill will not require any compactive efforts and due to the thickness of the fill, less than 1.0m, the post construction settlement will be negligible. Above the water table the engineered fill should consist of Granular 'A' conforming to M.T.O. Standard Form 1010. The advantage of placing footings on the 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 silty sand.

The engineered fill 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 States is 600kPa. The Bearing Capacity at Serviceability Limit States Type II (qs) is 350kPa. Assuming a 4.0m X 9m footing and a 45 degree load distribution through approximately 3.0m Granular 'A' core, the bearing pressure on the silty sand will be approximately 100kPa at the Serviceability Limit State Types II (qs). The silty sand at the elevation of 334.0m is capable of safely supporting this bearing pressure.

The geometry of the fill is shown on Figure 4.

For the evaluation of the sliding resistance of footings on engineered fill consisting of Granular 'A', the friction between the concrete and the Granular 'A' should be taken as 35 degrees, unfactored coefficient of friction of 0.70..

Scour protection should be provided for all footings placed above the water table or within the scour depth.

6.2.2 Option 2

Three span structure with 2 side spans of 24m and a middle span of 35m.

Closed type of abutments and open piers are contemplated for this option.

Three alternatives are considered as foundation alternatives for this option; timber piles, spread footings on natural soil and spread footings on engineered fill.

6.2.2.1 Abutment Footings

6.2.2.1.1 Spread Footings on Natural Soil

The Bearing capacity of spread footings in the sand and gravel deposit at an elevation of 335.2m for west abutment and 340.0m for east abutment was calculated using the following data and assumptions:

Average N-value below the footing elevation - 40
Footing Depth - 1.5m for east abutment
- 3.0m for west abutment
Footing Width - 3.0m
Footing Length - 9.0m
Unit weight of foundation soil - 21.0kN/m^3
Thickness of sand and gravel layer
over a compact silty sand deposit - 3.0m (average)

The Factored Bearing capacity at Ultimate Limit States, based on the above parameters, is 770kPa. The bearing capacity at Serviceability Limit States Type II (qs) is 440kPa.

The bearing capacity at Serviceability Limit States Type II (qs) of 440kPa will result in a bearing pressure of approximately 160kPa on the lower deposit of compact silty sand which will be satisfactory.

6.2.2.1.2 Driven Piles

Driven piles are not recommended for the abutments due to the presence of cobbles and possible boulders. In order to drive timber piles, the boulders might have to be pre-augered using expensive rock coring techniques.

6.2.2.1.3 Spread Footings on Engineered Fill

To reduce the abutment height of the west abutment spread footings for the abutment can be placed on engineered fill. For engineered fill, the excavation should be carried to an elevation of 335.2m.

For footings on engineered fill, compacted to at least 100% of its Standard Proctor Maximum Dry Density in lifts of 150mm, the Factored Bearing Capacity at Ultimate Limit States is 600kPa. The Bearing Capacity at Serviceability Limit States Type II (qs) is 350kPa.

For the evaluation of sliding resistance of footings on sand and gravel, the angle of friction between the concrete and the silty sand should be taken as 35 degrees, unfactored coefficient of friction of 0.70.

6.2.2.2 Pier Footings

A dynamic cone test carried out at the proposed west pier gave similar penetration resistances to the dynamic cone test at borehole 2.

A dynamic cone test could not be carried out at the east pier as the location was under water during investigation and was not accessible. Based on the subsoil stratigraphy we assume that the subsoil at the founding elevation of the east pier will be similar to borehole 3.

Three foundation alternatives are considered for the pier footings: timber piles, spread foundations on natural soil and spread footings on engineered fill.

6.2.2.2.1. Timber Piles

No end bearing stratum was encountered within the borehole and the dynamic cone penetration test depths. Therefore friction piles will have to be used as one of the foundation alternatives. We recommend driven timber piles.

For a timber pile with an embedded depth of approximately 14.0m and Butt and Tip diameters of approximately 360mm and 240mm respectively, the Factored Axial load capacity at Ultimate Limit States is estimated at 400kN.

The Axial load capacity at Serviceability Limit States type II (qs) is estimated at 200kN.

These estimated axial loads are based on load tests carried out by Ministry of Transportation in similar subsoils (Ref. to M.T.O. report EM-48; sites 12 and 13). However, we recommend that a load test be carried out to confirm these load capacities.

Battered timber piles should be used to resist the horizontal earth pressures.

6.2.2.2.2 Spread Footings on Natural Soil

Shallow spread footings placed at an elevation of approximately 334m is another foundation alternative.

The Bearing capacity of spread footings at the above elevation was calculated using the following data and assumptions:

Average N-value below the founding elevation - 10
Footing Width - 6.0m
Footing Length - 9.0m
Footing Depth - 3.0m
Submerged unit weight of soil - 7.5kN/m^3

The Factored Bearing capacity at Ultimate Limit States, based on the above parameters, is 340kPa.

The Bearing capacity at Serviceability Limit States Type II (qs) is 100kPa.

The total and differential settlement between abutments of the structure, using the above recommended design bearing pressures will be less than 25mm and 15mm respectively.

For the evaluation of the sliding resistance of footings on the silty sand, the angle of friction between the concrete and the silty sand should be taken as 30 degrees, unfactored coefficient of friction of 0.58.

6.2.2.2.3 Spread Footings on Engineered Fill

Placing the footings on engineered fill will require excavation to an elevation of 334.0m, approximately 0.6m below the river water level recorded on September 5th, 1990. In order to avoid any expensive dewatering scheme, the fill to the water level should consist of well graded crushed rock fill. This type of fill will not require any compactive efforts and due to the thickness of the fill, less than 1.0m, the post construction settlement will be negligible. Above the water table the engineered fill should consist of Granular 'A' conforming to M.T.O. Standard Form 1010. The advantage of placing footings on the 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 silty sand.

The engineered fill 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 States is 600kPa. The Bearing Capacity at Serviceability Limit States Type II (qs) is 350kPa. Assuming a 4.0m X 9m footing and a 45 degree load distribution through approximately 3.0m Granular 'A' core, the bearing pressure on the silty sand will be approximately 100kPa at the Serviceability Limit States Type II (qs). The silty sand at the elevation of 334.0m is capable of safely supporting this bearing pressure.

The geometry of the fill is shown on Figure 4.

For the evaluation of the sliding resistance of footings on engineered fill consisting of Granular 'A', the friction between the concrete and the Granular 'A' should be taken as 35 degrees, unfactored coefficient of friction of 0.70..

Scour protection should be provided for pier footings placed above the water table or within the scour depth.

6.3 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 States	10 kPa/m
At Serviceability Limit States, 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 the backfill to the abutments will consist of Granular 'A' or 'B' type aggregate.

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" - $\phi = 35°$, $\gamma = 22.8$ kN/m^3, $k_o = 0.43$
Granular "B" - $\phi = 30°$, $\gamma = 21.2$ kN/m^3, $k_o = 0.50$

If wing walls are provided on either side of the proposed bridge, construction joints should be provided between the wing walls which can yield and the main bridge structure which is rigidly restrained.

To avoid the development of large horizontal pressures the compaction and backfilling behind the abutments should be carried out simultaneously.

6.4 Excavation

Excavation for the bridge abutment footings for option 1 or the piers for option II will be slightly below the water table (approximately 0.6m below water level recorded on September 5, 1990). In order to avoid any expensive de-watering schemes, the work should be scheduled during the dry season with a low water condition. A mud slab should be placed to the water level using tremie concrete techniques and the foundation should be constructed above the mud slab. If the excavation cannot be carried out during the dry season, consideration should be given to de-watering the excavation if spread footings on natural soil are considered as a foundation alternative.

7. BRIDGE APPROACHES

7.1 East Approach

The depth of fill for the east approach will vary from 7.0m at borehole 3 to approximately 2.4m at borehole 4 decreasing to less than 2m at borehole 6.

The subsoils at these boreholes are quite competent and we do not anticipate any settlement problems. Embankment slopes of 2:1 should be satisfactory.

7.2 West Approach

The depth of fill for the west approach will vary from approximately 6.0m at borehole 1 to approximately 7.5m at borehole 2.

Peat varying from 0.5m to 1.3m was encountered at boreholes 2 and 1 respectively over a very loose deposit of silty sand with thin organic layers. The thickness of the very loose silty sand deposit varies from 1.7m at borehole 1 to 3.5m at borehole 2.

We recommend that the peat be removed from underneath the proposed embankment from toe to toe. Most of the settlement of the very loose silty sand deposit due to the embankment loading is anticipated to be completed within the construction period. The horizontal organic layers within the silty sand were very thin and we anticipate only minor settlement of the deposit due to consolidation of these layers. However, we recommend a flexible approach slab be designed at the west abutment to guard against any minor post construction settlement that might occur due to consolidation of the thin organic layers.

8. 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 not be detected or anticipated from the site investigation. Also, depending on seasonal factors, the water level in the river could be different than at the time of the field work.

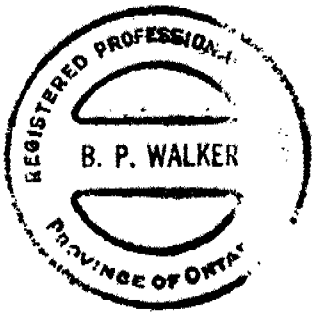
If you have any questions regarding the contents of this report, please contact this office.

Yours very truly,
B. P. WALKER ASSOCIATES LTD.

U. S. Sappal, P. Eng.

per **B. P. Walker, Ph.D., P. Eng.**

USS:lb



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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $w_L - w_P$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 46-87-01 LOCATION STA. 13 + 733.5: CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS & CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 27, 1991 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS
338.2	GROUND SURFACE					
0.0 / 336.9	DEAD VEGETATION; Peat		1	SS	4	
1.3 / 335.2	SILTY SAND - fine to med. grained; sat.; very loose; dark brown occ. thin org. layers		2	SS	2	
			3	SS	3	
3.0 / 332.7	SAND WITH GRAVEL AND COBBLES; dense to very dense		4	SS	39	
			5	SS	51	
5.5 / 325.6	SANDY SILT - trace clay; compact		6	SS	16	
			7	SS	6*	
			8	SS	14	
			9	SS	14	
			10	SS	16	
12.6 / 315.9	END OF BOREHOLE					
22.3	END OF CONE TEST					

* Low N value probably due to loosening of soil by water pressure.

ELEVATION SCALE: 338, 336, 334, 332, 330, 328, 326, 324, 322, 320, 318, 316

DYNAMIC CONE PENETRATION RESISTANCE PLOT — 20 40 60 80 100

SHEAR STRENGTH kPa — ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) — 10 20 30

45.9%

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) — GR SA SI CL

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 46-87-01 LOCATION STA. 13 + 753.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS AND CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 27, 1991 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
337.5	GROUND SURFACE										
337.0	PEAT					*					
0.5	SILTY SAND WITH ORGANIC LAYERS - very fine grained; wet; very loose to loose		1	SS	2		336				
			2	SS	2				49%		
			3	SS	1						
333.5			4	SS	9		334				
4.0	SANDY SILT - trace clay; saturated; compact		5	SS	9						
			6	SS	10		332				0 8 85 7
			7	SS	10		330				
			8	SS	17		328				
			9	SS	14		326				
324.9			10	SS	18						
12.6	END OF BOREHOLE						324				
							322				
							320				
							318				
314.6							316				
22.9	END OF CONE TEST										

* W.L. NOT ESTABLISHED

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 46-87-01 LOCATION STA. 13 + 798.0; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS AND CONE TEST COMPILED BY D.S.

DATUM GEODETIC DATE JUNE 26, 1991 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
337.2	GROUND SURFACE						
0.0 / 336.6	SAND AND GRAVEL						
0.6	SILTY SAND- trace clay and gravel; loose to compact		1	SS	7		336
			2	SS	6		
			3	SS	6		334
			4	SS	12		
332.6							
4.6	SANDY SILT - trace clay; compact		5	SS	11		332
			6	SS	14		
			7	SS	12		330
			8	SS	10		328
			9	SS	17		326
			10	SS	18		324
323.0			11	SS	18		
14.2	END OF BOREHOLE						322
317.4							320
19.8	END OF CONE TEST						318

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W_L

WATER CONTENT (%)
10 20 30

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR	SA	SI	CL
	0	85	10	5
	0	10	82	8
	0	7	87	6

○ N.P.

$+^3$, $\times^5$: Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 4

METRIC

W P 46-87-01 LOCATION STA. 13 + 816.5M; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS AND WASH BORING; CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE MARCH 19 & 20, 1991 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
341.4	GROUND SURFACE						
0.0	SAND AND GRAVEL - trace silt; frequent cobbles; compact to very dense		1	SS	90		340
			2	SS	25		
			3	SS	45		338
			4	SS	41		
336.2			5	SS	43		336
5.2	SILTY SAND - trace clay; compact		6	SS	16	▼	
332.9			7	SS	21		334
8.5	SANDY SILT - trace clay; grey; saturated; compact		8	SS	19		332
			9	SS	18		
			10	SS	23		330
			11	SS	11		328
			12	SS	20		326
			13	SS	25		
			14	SS	28		324
321.1			15	SS	32		322
20.3 / 320.1	END OF BOREHOLE						
21.3	END OF CONE TEST						

DYNAMIC CONE PENETRATION RESISTANCE PLOT — 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR	SA	SI	CL
	79	18	(3)	

N.P.

$+^3$, $\times^5$: Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 46-87-01 LOCATION STA. 13 + 826.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY U.S.

DATUM GEODETIC DATE MARCH 20, 1991 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
341.7	GROUND SURFACE										
0.0	SAND AND GRAVEL - trace silt; frequent cobbles; compact to very dense		1	SS	57	*	340		○		
			2	SS	40				○		
			3	SS	63		338		○		
			4	SS	27				○		
336.7			5	SS	14				○		
5.0	END OF BOREHOLE										

* BOREHOLE DRY ON COMPLETION

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 6

METRIC

W P 46-87-01 LOCATION STA. 13 + 836.5; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY U.S.

DATUM GEODETIC DATE MARCH 20, 1991 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT Wl; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
342.2	GROUND SURFACE										
0.0	SAND AND GRAVEL - trace silt; frequent cobbles; compact to very dense		1	SS	53	*	342		○		
			2	SS	30/15cm						
			3	SS	72		340		○		
			4	SS	19				○		
							338				
337.2			5	SS	19				○		
5.0	END OF BOREHOLE										

* BOREHOLE DRY ON COMPLETION

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 7

METRIC

W P 46-87-01 LOCATION STA. 13 + 756.3; CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE CONE TEST COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 27, 1991 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) SHEAR STRENGTH kPo ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
336.8	GROUND SURFACE						336				
							334				
							332				
							330				
							328				
							326				
							324				
							322				
319.4							320				
17.4	END OF CONE TEST										

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 8 METRIC

W P 46-87-01 LOCATION STA. 13 + 740.0; O/S 10.0m R.T. CENTRELINE HWY. 556 ORIGINATED BY C.N.

DIST 18 HWY 556 BOREHOLE TYPE PROBE HOLE (B.S.A.) COMPILED BY U.S.

DATUM GEODETIC DATE JUNE 26, 1991 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
338.2	GROUND SURFACE										
337.9	PEAT					▽	338				
0.3	SILTY SAND - loose; brown; saturated										
							336				
335.2			1	AS	-				o		
3.0	END OF PROBE HOLE										

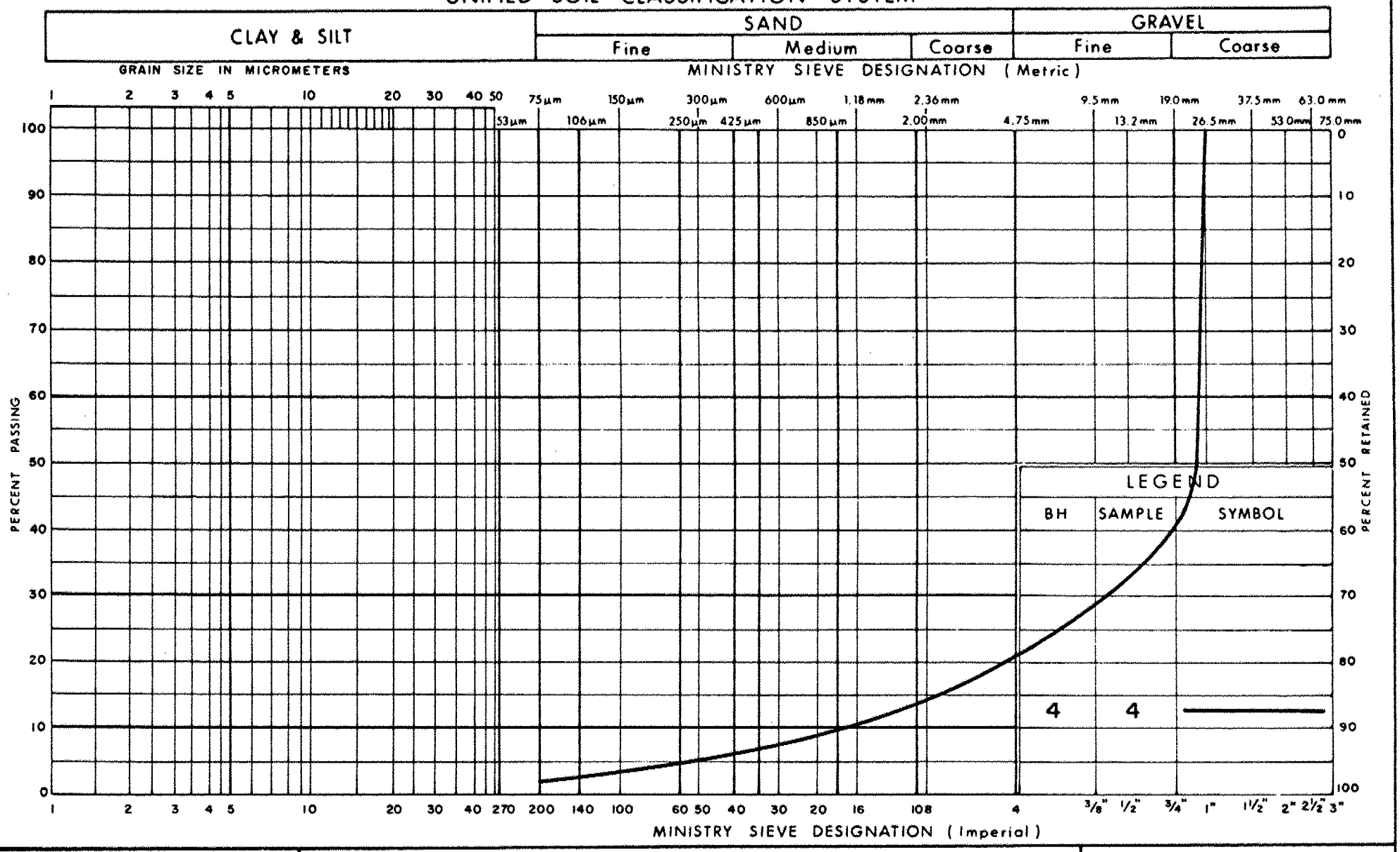
OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

78 12 M

UNIFIED SOIL CLASSIFICATION SYSTEM



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Ontario

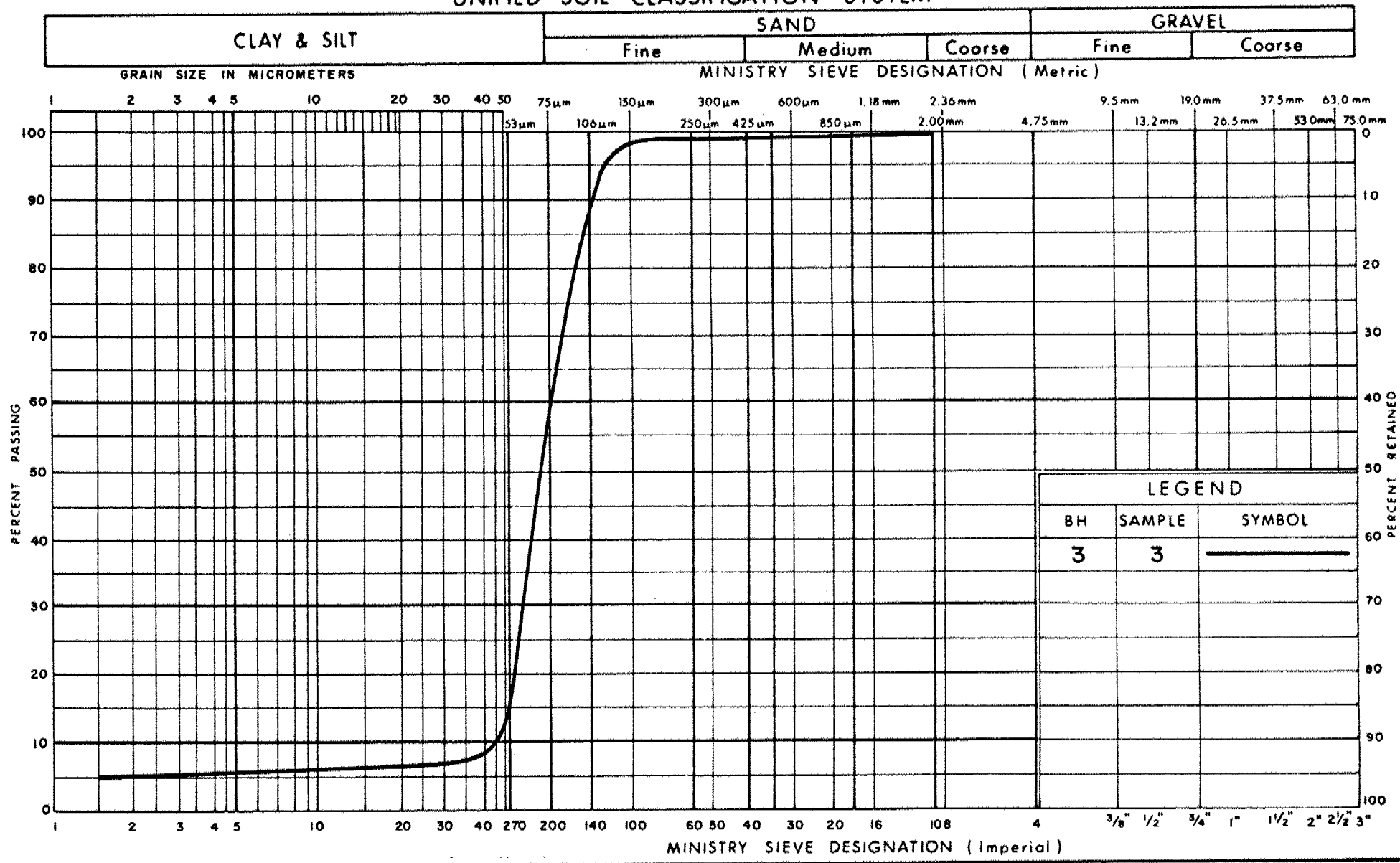
GRAIN SIZE DISTRIBUTION

SAND & GRAVEL, TRACE OF SILT

FIG No 1

WP 46-87-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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Ontario

GRAIN SIZE DISTRIBUTION

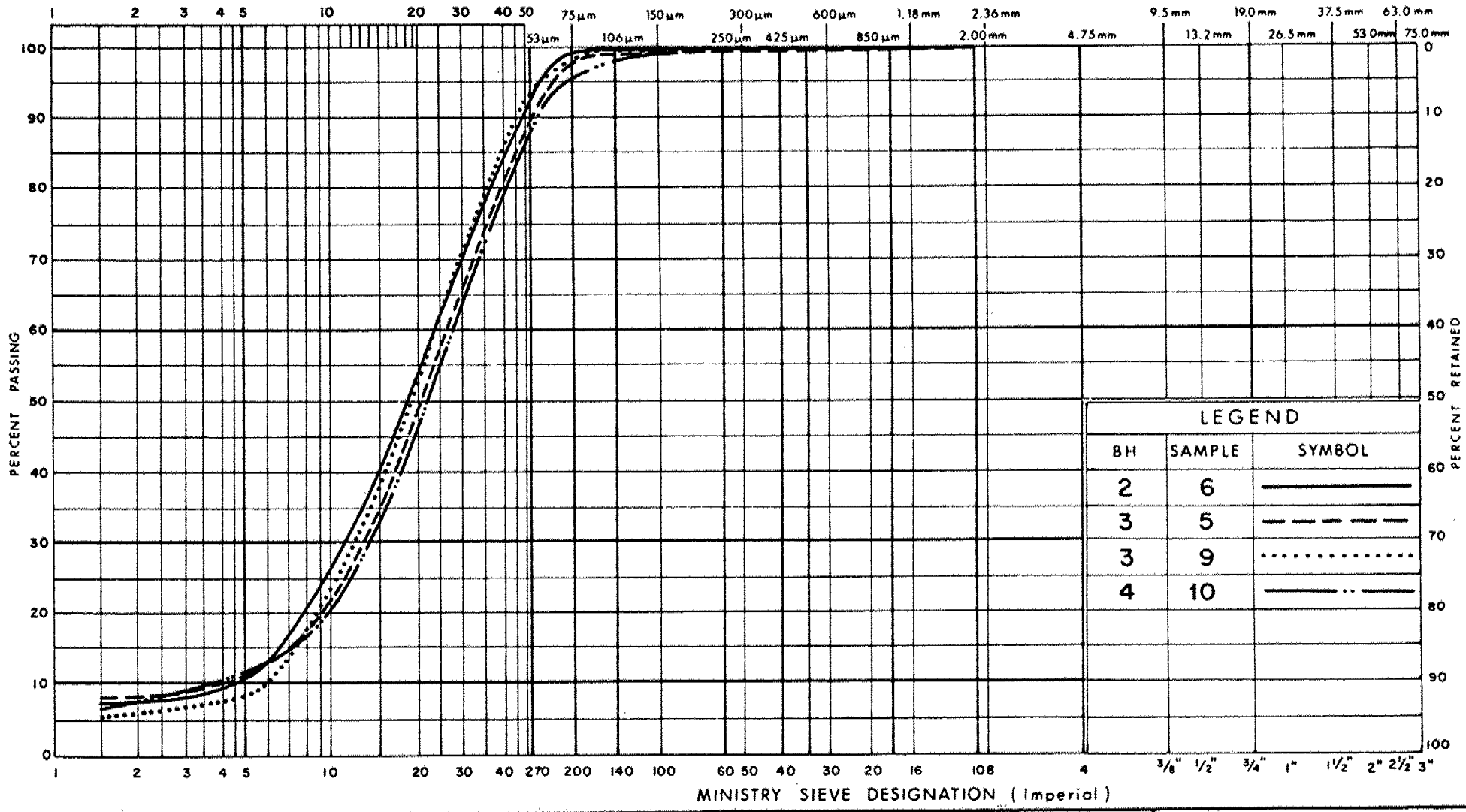
SILTY SAND, TRACE OF CLAY

FIG No 2

WP 46-87-01

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



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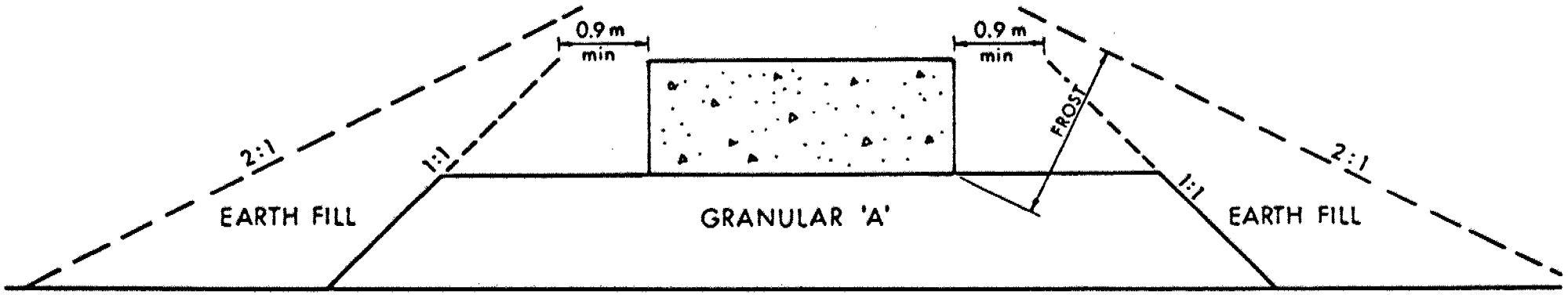
GRAIN SIZE DISTRIBUTION

SANDY SILT, TRACE OF CLAY

FIG No 3

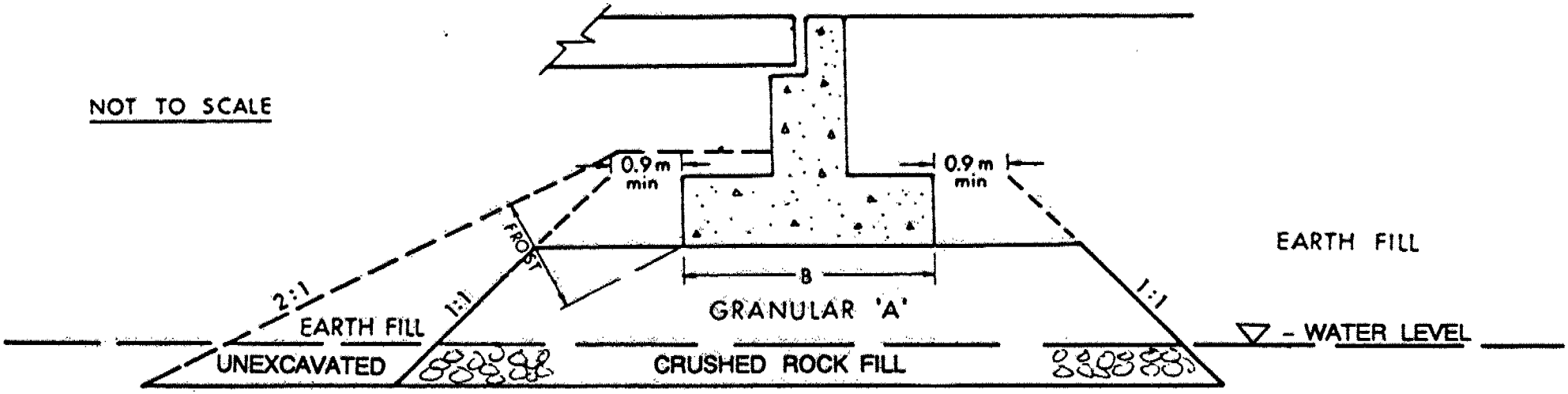
WP 46-87-01

FOOTINGS ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



X SECTION

NOT TO SCALE



LONGITUDINAL SECTION

NOTES:

1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.

2 - PLACE GRANULAR 'A' & EARTH FILL TO BASE OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.

3 - CONSTRUCT FOOTINGS.

4 - PLACE REMAINDER OF FILL.

FIGURE 4

WP 46-87-00

memorandum

Ontario

To: R.J. Krisciunas (3)
Head, Structural Section
Northwestern Region

Date: 1991 08 23

From: Foundation Design Section
Room 315, Central Building

Re: Foundation Investigation and Design Report For
Proposed Replacement of Aubinadong River Bridge
W.P. 46-87-00, Site 38S-380, Hwy. 556
District 18, Sault Ste Marie

The Foundation Design Section retained B.P. Walker Associates Ltd., to carry out a foundation investigation for the above-noted project. The Foundation Investigation and Design Report is forwarded under cover of this memo.

After preparing the consultant agreement, this office provided technical supervision including the establishment of terms of reference and careful review of the consultant's proposals and progress at all stages of the project. Several meetings were held with the Consultant during the course of the project to discuss progress findings of the foundation investigation and their influence on geotechnical recommendations for the proposed structures. The Foundation Investigation (factual) portion of the report was reviewed only for format, and its accuracy and completeness are the responsibility of the Consultant. The Foundation Design (recommendation) portion of the report has been carefully reviewed by this office based on the subsurface information provided by the Consultant.

Comments are given in the attached report regarding design and construction of foundations for the proposed two options - single span bridge and three span bridge - for the replacement structure. Comments are also given in the attached report regarding the design of approach embankments. We consider that the report contains sufficient information for your design needs.

..../2

Difficulties were encountered by our consultant to get access to the west abutment area. This resulted in the delay in completing this assignment.

If there are any questions regarding the report or during the design please contact this office.

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

BI/jb

Distribution

c.c. - J.R. Morgenroth
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E.A. Joseph
File

OVERSIZE DRAWING

P.O. Box 1177
615 South James Street
Thunder Bay, Ontario
P7C 4X9

(807) 473-2129

19 September 1994

David Holla, P.Eng.
Project Engineer
Ministry of Natural Resources
Ontario Government Building
199 Larch Street
Sudbury, Ontario
P3E 5P9

Dear Mr. Holla:

RE: Interim Slope Stability of River Bank on Highway 556 at the Abinadong River, Contract 93-229

Rather than rehash the minutes of our field meeting of August 24, 1994, I have included copies of memoranda produced by Messrs. Tae Kim and David Shaw of this Ministry.

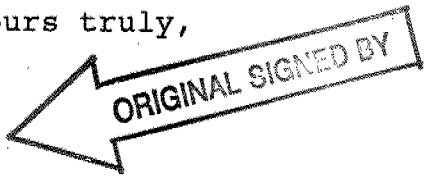
As agreed the Contractor has abandoned the site of the failure and has been directed not to perform any further work adjacent to the river bank in the area of instability.

Mr. Tim Bellrose will be recording the existing conditions with his video camera, once the total station survey of the river bank in this area has been completed this fall. He will return to the site next spring/summer once the spring flooding has subsided in order to record the same area on his video camera. This action should document any erosion in the area of the slope failure.

Planning and Design has submitted a survey request to our Surveys and Plans Section. The request is for a total station survey of the river banks throughout a 500 metre stretch immediately upstream and downstream of the location of the slope failure. We have requested the establishment of permanent reference points. The survey will be redone next spring/summer in order to document erosion in the area of the survey.

Once the video and survey are complete, we will forward the results to your office and discuss our findings. We can jointly consider an appropriate course of action at that time. Please advise this office if you have any concerns with our plan of action.

Yours truly,



B. Buffington
Project Manger
Planning & Design
Northwestern Region

BB/ca
Enclosure

cc. T. Mills
T. Kim
P. Bound

u:\ca\letters\Holla.BB

MEMORANDUM

Ontario

To: D. Siczkar
Area Manager (East)
Planning and Design Section
Northwestern Region

Date: September 13, 1994

Atten: Mr. B. Buffington, Project Manager

From: Pavements & Foundations Section
Room 315, Central Building

Tel: 235-3731
Fax: 235-5240

Re: Interim Slope Stability of River Bank on
Highway 556 at the Abinadong River
Contract 93-229
District 18, Sault Ste. Marie

Further to an on-site meeting on August 24, 1994 between representatives of the Ministries of Natural Resources and Transportation, this memo summarizes our observations and discussion made during the site visits.

Background

The contractor had placed the geotextile from Stations 14 + 800 to 15 + 050. The backhoe was placing the rock protection along the river bank in the vicinity of Station 14 + 950 when the rock fill and geotextile sank down and slipped slightly into the river causing silty material of the bottom of river to flow upward between the rock protection. This silty material was very susceptible to boiling and was boiled up out of the area. The area affected was approximately from Stations 14 + 930 to 14 + 960. A mud wave approximately 20 cm high and 10 m long was observed on the river botttom at Station 14 + 930. The contractor stopped work immediately after the slip had occurred.

The initial on-site meeting took place on Thursday, August 4, 1994 with an attendance of the following MTO people. Tae C. Kim, Senior Foundation Engineer, Downsview, Tom Mills, Area Construction Supervisor, Sault Ste. Marie, David Shaw, Pavement Design & Evaluation Officer, Thunder Bay, Ronald Arsenault, Superintendent, Leo Alarie and Sons Ltd. The site seemed to be stable at the time of the meeting.

At the meeting, it was agreed that the contractor would bring the backhoe to the area and try to pull some of the rock protection back into place and tidy up the slopes. All the geotextile that had not been covered with rock protection should be removed. Remedial work was completed based on the above recommendations.

DISCUSSION AND RECOMMENDATIONS

Subseqent on-site meeting was held on August 24, 1994 between representatives of the Ministries of Natural Resources and Transportation.

A lengthy discussion took place as to the effect of the rock protection placed into the river on the erosion of the river bank. The Ministry of Natural Resources concern is not only that the existing interim slope at the toe is stable or not, but also that the flow of the river being constricted through the oxbow area will flow faster and consequently will create a more rapid rate of erosion farther down the embankment.

It is our opinion that the rock protection has been placed at the most critical erosion area and that this protection may alleviate the erosion problem. Based on our observation, it appears that the existing interim slope is stable for the short term. Therefore, it was agreed that the Ministry of Transportation would do the following:

1. Do not disturb the rock protection placed any further.
2. Set up a total station survey to monitor the erosion of the toe of the embankment for one year.
3. Take photos and continue to video record the area once this summer and again next spring (1995).

For a long term solution to Highway 556, it is recommended that the highway should be moved to the east approximately 30-40 m away from the river bank as early as possible. There is an existing low laying area approximately 80 m to the east, however, a new alignment could be built in this area with no difficulty.

We believe that this memorandum meets with your present requirements. However, should you have any questions regarding this memo, please do not hesitate to contact this office.

Taedul Kim

Tae C. Kim, P. Eng.
Sr. Foundation Engineer

TCK/mmj

c.c. - Tom Mills
Peter Bound

M E M O R A N D U M

TO: M.T.O. Attendees

DATE: September 7, 1994

FROM: Geotechnical Section
Northwestern Region
(807)473-2092

RE: Site Meeting - Contract 93-229, Highway 556, on August 24, 1994

An on-site meeting took place on August 24, 1994, between representatives of the Ministries of Transportation and Natural Resources. The following people were present:

Tae Kim, Ministry of Transportation, Foundations Office
Barry Buffington, Ministry of Transportation, Planning and Design Section
Tom Mills, Ministry of Transportation, Construction Office
Tim Bellrose, Ministry of Transportation, Construction Office
David Shaw, Ministry of Transportation, Geotechnical Section
David Holla, Ministry of Natural Resources, Regional Office
Chuck Anderson, Ministry of Natural Resources, Area Office

A discussion took place as to the effect of the rock protection placed into the river on the erosion of the river bank. The Ministry of Natural Resources' concern is that with the flow of the river being constricted through the Oxbow area will flow faster and will create a more rapid rate of erosion farther down the embankment.

It was agreed that the Ministry of Transportation would do the following:

1. Do not disturb the rock protection any further.

2. Continue to video record the area once more this summer and again next spring (1995).

3. Set up a total station survey to monitor the erosion of the embankment for one year.

Tae Kim is to send to Barry Buffington a letter regarding how stable the placed rock protection is now and a copy is to be forwarded to David Holla.

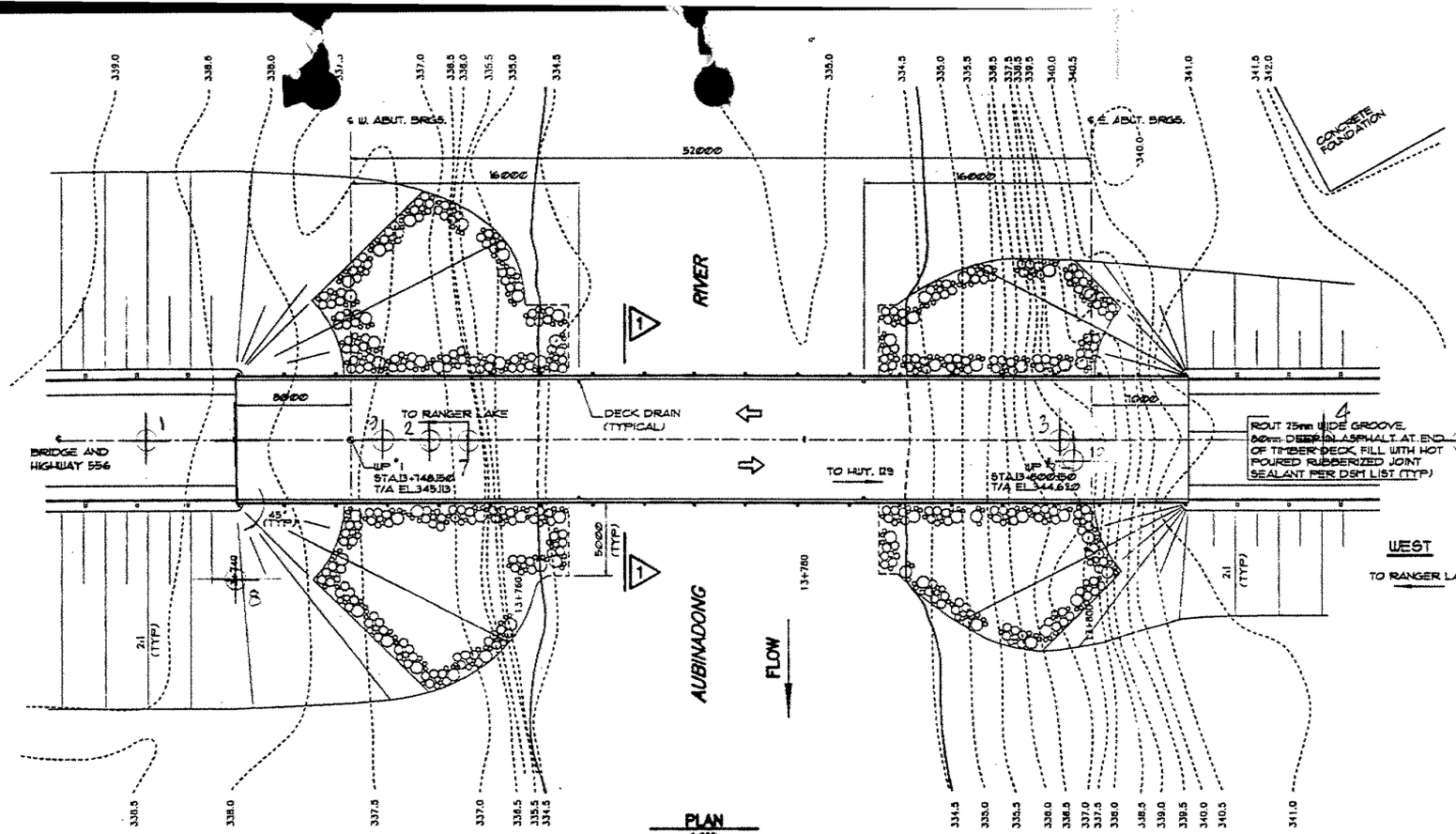
Barry Buffington is to initiate a survey request for this area and will generate a letter to David Holla, Ministry of Natural Resources in Sudbury.

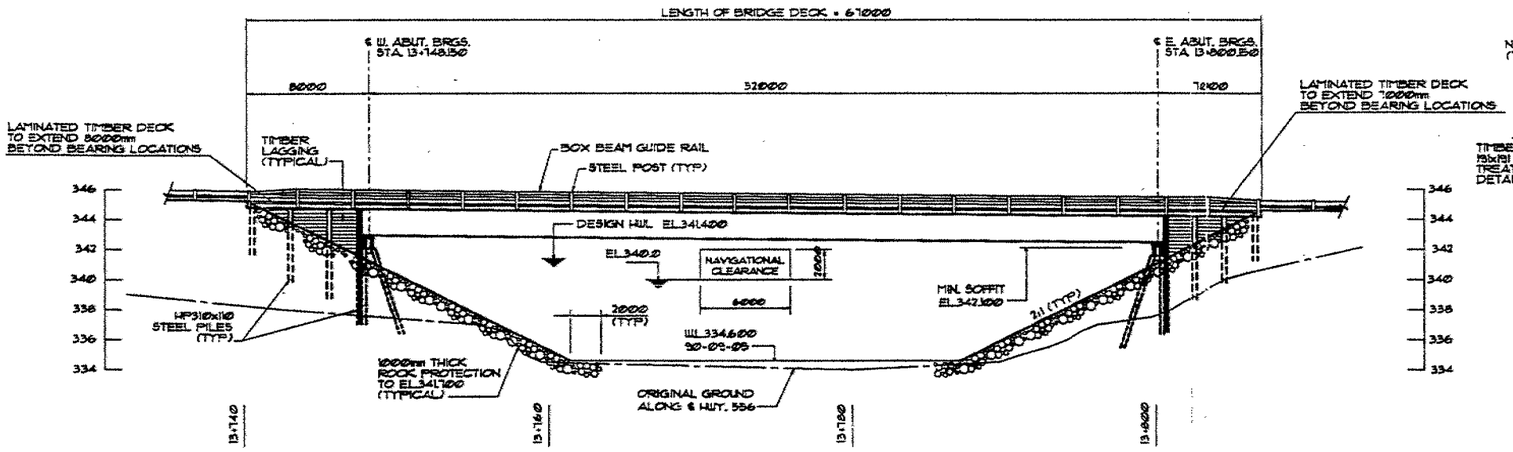
If you have any questions, please contact me at the Geotechnical Section in Thunder Bay.

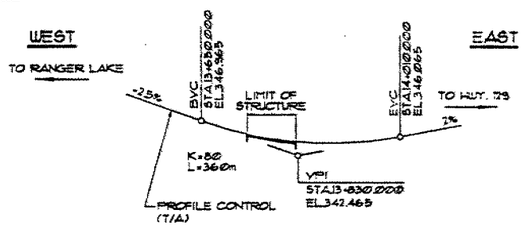
J. David Shaw
Acting Pavement Design &
Evaluation Officer

P. J. Bound
Acting Head, Geotechnical Section

JDS/ric

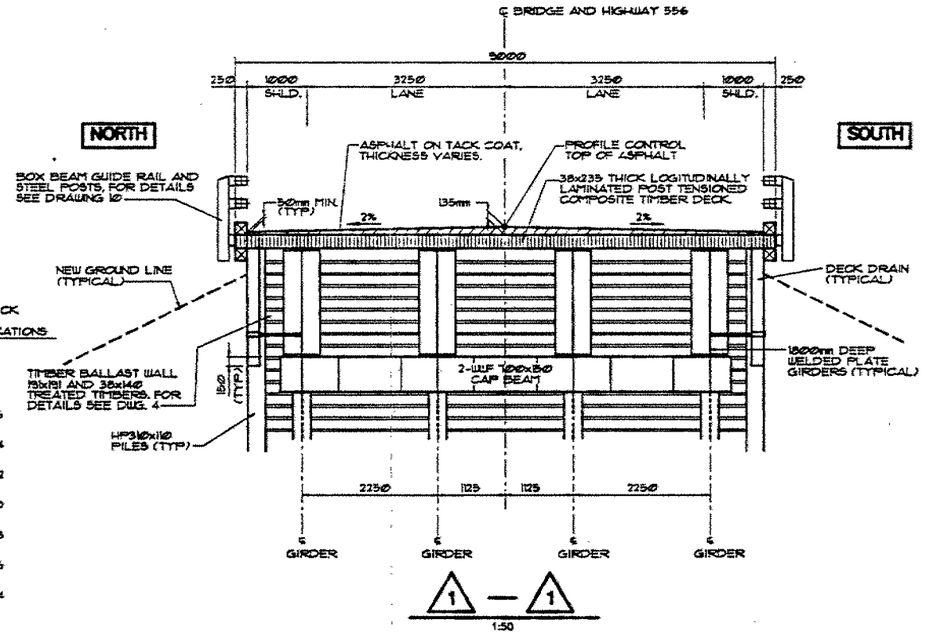






PROFILE OF HIGHWAY 556

NOT TO SCALE



METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NORTH FOR CONSTRUCTION

DIST. No. 18 HWY. 556
CONT. No.
WP No. 46-87-01

SHEET

AUBINADONG RIVER BRIDGE

PRELIMINARY GENERAL ARRANGEMENT

DILLON
Consulting Engineers + Planners
Environmental Scientists

LIST OF ABBREVIATIONS

T/A	DENOTES TOP OF ASPHALT
HWL	DENOTES HIGH WATER LEVEL
WL	DENOTES WATER LEVEL
WP	DENOTES WORKING POINT

NOTES

1. NO CHAINS, HOOKS OR PEAVIES SHALL BE USED IN HANDLING TREATED WOOD. SURFACES EXPOSED BY FIELD CUTTING OR DRILLING SHALL BE TREATED BY SOAKING WITH AN APPROVED PRESERVATIVE EQUAL IN TOXICITY TO THE ORIGINAL PRESERVATIVE. THREE APPLICATIONS ARE REQUIRED, EACH TO AN AIR DIRED SURFACE, EXCEPT WHERE NOTED.
2. PRESERVATIVE TREATMENT FOR ALL TIMBER IN NEW STRUCTURE SHALL BE CREOSOTE.
3. NO CREOSOTE SHALL BE PERMITTED TO DRIP FROM THE STRUCTURE OR ENTER THE WATERCOURSE DURING FIELD APPLICATION.
4. ALL TIMBER SHALL BE #2 JACK PINE OR BETTER, EXCEPT AS NOTED.

LIST OF DRAWINGS

385-340	1. GENERAL ARRANGEMENT
385-340	2. BOREHOLE LOCATIONS + SOIL STRATA
385-340	3. PILING DETAILS
385-340	4. ABUTMENT DETAILS
385-340	5. STRUCTURAL STEEL 1
385-340	6. STRUCTURAL STEEL 2
385-340	7. BEARING DETAILS
385-340	8. DECK DETAILS 1
385-340	9. DECK DETAILS 2
385-340	10. RAILING DETAILS
385-340	11. QUANTITIES - STRUCTURE

BM 342.406
GEODETIC DATUM
S & W in Root of 0.10 Birch
19.6 Lt. 13+823.1

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN P.T.	CHK. M.R.	CODE OHBDC-83	LOAD CLASS A	DATE OCT. 1992	
DRAWN A.K.L.	CHK. P.T.	SITE 385-340	STRUCT.	SCHEME	DWG. P1

FILE:

M E M O R A N D U M

TO: Mr. Tom Mills
Area Construction Supervisor
Sault Ste. Marie

DATE: August 9, 1994

FROM: Geotechnical Section
Northwestern Region
(807)473-2092

Re: Slip Area at the Aubinadong River, Contract 93-229

In attendance at the on-site meeting on Thursday, August 4, 1994 were Tae Kim, Senior Foundation Engineer, M.T.O., Downsview, Tom Mills, Area Construction Supervisor, M.T.O., Sault Ste. Marie, Rolald Arsenault, Construction Superintendent, Leo Alarie and Sons Ltd. and myself, J. David Shaw, Acting Pavement Design and Evaluation Officer, M.T.O., Thunder Bay.

Background:

The contractor had placed the geotextile from Station 14+800 to 15+050 and had a backhoe positioned at the water's edge at approximately Station 14+970 on Wednesday, August 3rd. The backhoe was placing the rock protection along the river bank in the vicinity of Station 14+950 when the rock and geotextile sank down and slipped slightly into the river causing silty material to flow upwards between the rock protection and the river embankment. This silt material was very liquid and the water was bubbling and flowing up out of the area. The area affected was approximately from Station 14+930 to 14+960. A mud wave approximately 200 mm high and 10 m long was observed on the river bottom at Station 14+930. The contractor stopped work immediately after the slip had occurred and the site seemed to be stable at the time of the meeting.

It was agreed that the contractor would bring the backhoe back to the area and try to pull some of the rock protection back into place and tidy up the slopes. All the geotextile that had not been covered with rock protection should be removed.

It is felt that the rock protection that has been placed already had been placed at the most critical erosion area and that this protection may alleviate the erosion problem for the short term.

For a long term solution to the erosion problem, a recommendation similar to the one attached will have to be built. Tae Kim and the Foundations Design Section will have to be asked to investigate the site and make specific site proposals.

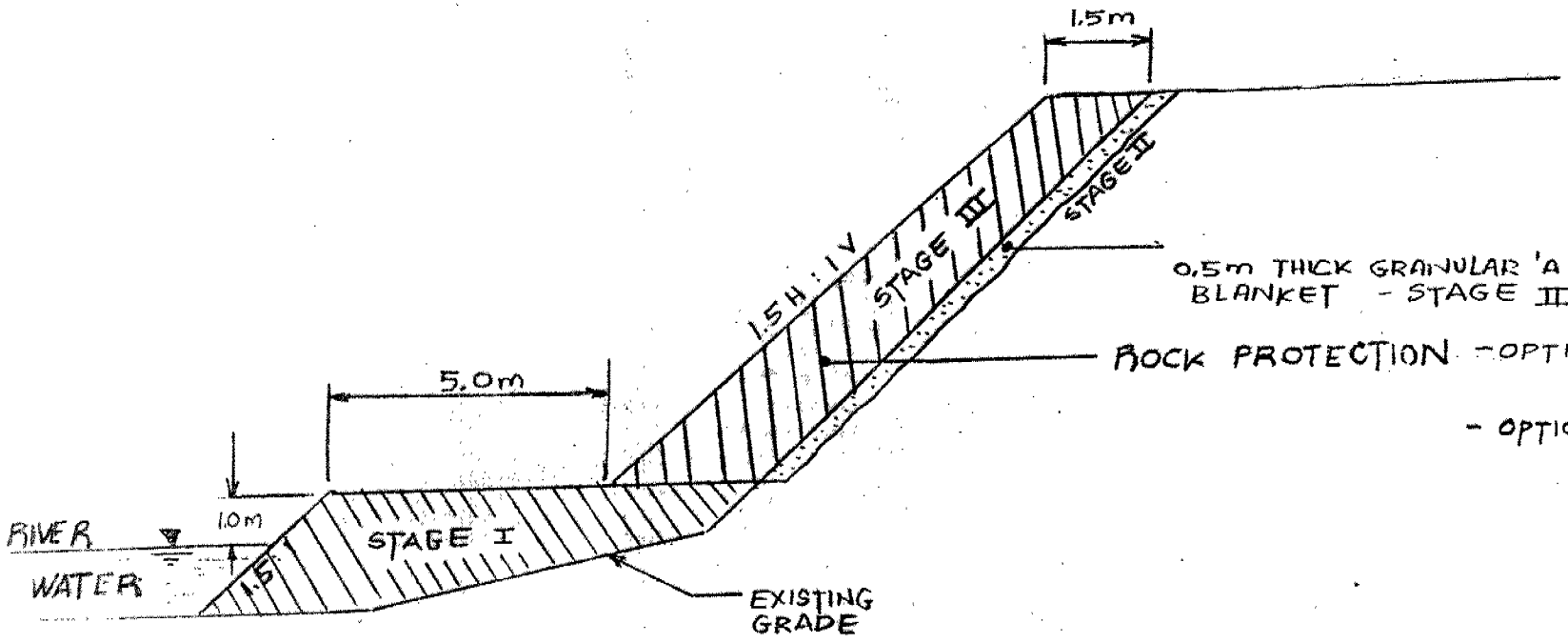
Another option that may not be as costly would be to move the highway to the left approximately 30 m away from the river embankment. There is an existing low laying area approximately 80 m to the left, but I am sure a new alignment could be built up to this area with no difficulty.

If you have any more concerns, please call.

David Shaw
Acting Pavement Design and
Evaluation Officer

P. J. Bound
Acting Head, Geotechnical Section

DS/ric
Attach
cc Tae Kim



NTS

TREATMENT GEOMETRY - ONLY

NOT FOR CONSTRUCTION PURPOSES:

FOR PRECISE DIMENSIONS, MTO FOUNDATIONS OFFICE IS TO BE NOTIFIED AND THIER SITE SPECIFIC RECOMMENDATIONS ARE TO BE USED

Ministry of Transportation
Ontario

FAXGRAM

PLEASE TYPE

DATE August 4, 1994 PAGE ___ OF ___

TO:	FROM:
Mr. Tae Rim Foundation Design Section 3rd Floor, Central Building, Downsview **FAX NUMBER: 1-416-235-5240**	Geotechnical Section Northwestern Region (807) 473-2132

SUBJECT: **Contract 93-229, Highway 556, Aubinadong River**

While placing rock protection along the river bank, the existing bank started to fail, resulting in a tension crack on the bank.

I request that you come to Sault Ste. Marie and travel to the site for your thoughts and recommendation.

P. J. Bound
Acting Head, Geotechnical Section

PJB/ric

ADM-S-481 91-05

memorandum

Ontario

To: Mr. P. Bound
Acting Head, Geotechnical Section
Northwestern Region

Date: 93/05/13

From: Foundation Design Section
Room 315, Central Building

Re: Hwy. 17/Gravel River -Slope stability due to river erosion
WP 909-76-01(A)
District 19, Thunder Bay

This memo was written in response to a request by the Northwestern Region Geotechnical Section concerning river bank stabilization problems along Hwy. 17 at the Gravel River. Erosion of the existing bank is taking place at four locations between stations 13+800 along Hwy. 17 where the Gravel River winds, nearing and crossing the existing highway (Site 1), 14+000 to 14+150 (Site 2), 14+300 to 14+450 (Site 3) and 14+600 to 14+700 (Site 4). We are providing in this memo a brief description of the subsurface conditions encountered at the site together with recommendation. This information is provided to you in advance of our final report, in order to facilitate you to proceed with your design work. The final report would be submitted at a later date upon the completion of laboratory testing and drafting.

The foundation investigation was carried out between 92/12/16 to 92/12/19 at the above site consisting of seven 18.75 m boreholes and five cone penetration tests at the locations listed below.

Borehole	Site No.	Location	Station
1	1	13+800 N/W	13+787 24.5 m Left
2	2	14+000 to	14+102 13 m Right
3	2	14+150	14+170.5 15.5 m Right
4	3	14+300 to	14+350 16.5 m Left
5	3	14+450	14+400 20 m Left
6	4	14+600 to	14+600 24.5 m Left
7	4	14+750	14+675 24 m Left

Site Description

The site is located approximately 50 km east of the Town of Nipigon along Hwy. 17 at the Gravel River Bridge easterly 0.5 km in the Township of Lett, District of Thunder Bay. The existing Highway is an asphalt covered 2 lane road with a third passing lane just east of the Gravel River bridge.

The topography is generally flat with a medium growth of birch, spruce and poplar trees. Bedrock outcrops were encountered approximately 5 Km along Hwy. 17 east and west from the site location. The Gravel River winds north and south of Hwy. 17 crossing at one location. The course of the river is irregular with several meander loops in the immediate vicinity of the bridge location. The water level of the river at the time of the investigation was relatively low (at an elevation of 83.15 m) enclosed by steep embankments composed of fine sands of up to 8 m high. Generally along the course of the river there is evidence of embankment failures, as up rooted trees and shrubs are found at the toe of the slopes. This process appears to be continuing as the river embankment is slowly moving closer to the existing Hwy. 17 at the four locations investigated. Inspection of the embankment appears to indicate surficial failures with in the sandy materials observed. The two span two lane Gravel River bridge is situated at the apex of a horse shoe river bend with active erosion taking place along the west-ward shore immediately north of the existing bridge, while deposition is evident on the easterly side of the river. In the design of the existing bridge sheet piles were utilized around footing locations to prevent scouring, however at the north-west shore these measures appeared to of failed as the sheet piles have failed being disturbed and relocated.

Subsoil conditions

The stratigraphy throughout the four sites was generally consistent with three distinct deposits encountered, however their thickness varied considerably as the deeper deposits are not present in some boreholes. A 5.6 m to 18.7 m thick deposit of Fine Sands with Pockets of Medium Sand, trace/some Silt which contained a thin surficial Organic Topsoil was encountered immediately below the surface. This layer was generally loose to compact with pockets of very loose and dense material. It comprises the material within the natural slopes of the existing river embankments. Underlying the above is a Silty Clay with Clayey Silt seams, slightly varved with Silt and fine Sand layers. Its thickness was not established in some boreholes (BH 1,2,4,5) as they were terminated at 18.7 m depths, not at all in another (BH 3) and with a thickness of 4.6 m and 1.5 m in two boreholes (BH 6,7 respectively). This layer contained a large percentage of silt with a very dense consistency. Encountered only at site 4 and only to a small extent in BH 2, the most eastern location the above layer became a Silt, with Seams of Clay extending beyond the scope of the boreholes. This deposit had a compact state of denseness. The previous investigation to the west at the Gravel River bridge indicated the probability that this deposit is also present at greater depths at all locations. Bedrock was confirmed

during the course of the bridge investigation classified as a granite pegmatite at a depth of approximately 49 m, however was not encountered in this investigation due to relatively shallow termination depths.

Groundwater Conditions

Observations of the groundwater level was carried out by measuring the water levels in open boreholes during the course of the investigation. Groundwater levels determined at the time of the investigation are shown in the table below.

BH No.	SITE No.	Depth (m)	Elevation (m)
BH 1	Site 1	6.1	184.4
BH 2	Site 2	7.6	180.9
BH 3	Site 2	6.7	182.2
BH 4	Site 3	5.2	184.9
BH 5	Site 3	7.6	183.5
BH 6	Site 4	6.1	183.9
BH 7	Site 4	6.7	183.4

The water table levels were found to correspond to those observed in the Gravel River.

It should be noted that the groundwater levels are subject to seasonal fluctuations and may therefore change as the water level in the river changes.

Discussions and Recommendations

GENERAL

Slope failures of the existing river embankments at four locations necessitated remedial measures to prevent any damage to existing structure and highways.

Based on our observations at the site, the slope instability of all four sites appears to be surficial in nature due to the erosion of the natural embankments by the flow of the Gravel River and rapid drawdown of the water level producing excess pore pressures. This assessment has been verified by the observation of site conditions and by the stability analysis of generalized embankment conditions. The stability analysis illustrates that the critical failure surface occurs above the toe of the embankment and is not a deep seated failure. It should be noted that this type of failure is of recurring nature occurring during the spring thaw period or period of heavy precipitation.

An effective stress analysis was applied for calculations of slope stability of the river bank at four separate locations using the limit equilibrium method of stability developed by

Sarma (Sarma, S.K., 1973 Stability Analysis of Embankment and Slopes, Geotechnique, Vol. 23, No. 3). Results are shown in the appendix, figures 1 to 8.

The stability analysis carried out on existing as shown on Figures 1,3,5 and 7 slopes indicates that the slopes at all four sites are unstable with a factor of safety near unity.

DESCRIPTION OF REMEDIAL SITES

All of the existing instabilities occur at river bends where the erosive power of the river is at its maximum. This report considers remedial treatment of these problem areas. The longitudinal limits for each site are listed below:

Site No.	Rock Fill Limits
1	North/West bank from the existing Gravel River structure, 150m north
2	Sta. 14+000 to 14+200 parallel to Hwy. 17 along Gravel River -North Bank
3	Sta. 14+300 to 14+500 parallel to Hwy. 17 along Gravel River -South Bank
4	Sta. 14+600 to 14+800 parallel to Hwy. 17 along Gravel River -South Bank

Special considerations should be given adjacent to the Gravel River Bridge as strong currents have caused the collapse of sheet piling previously installed to prevent erosion of the surrounding embankments in the vicinity of the West Abutment. The fine sand material which comprises the river bank is eroding on the north/west side and deposited on the south/east bank. Failure of these embankments behind the sheet piling have caused their disturbance.

EMBANKMENT TREATMENT

As shown in Figures 2,4,6 and 8 the stability analysis of generalized embankment conditions indicate that this type of failure can be controlled by rock protection. A Factor of Safety of 1.3 was employed in the analysis to ensure stability.

It is therefore recommended that the following remedial measures be carried out as soon as possible in order to stabilize the embankment slope in the distressed areas. Slope treatment will depend upon the type of rockfill available. Thus two options are described below.

OPTION A

All failed and/or loose material in the affected areas should be removed and replaced with a suitable locally available rockfill together with a minimum 0.5 m thick blanket of Granular 'A' fill placed between the rock fill and the native material to act as a filter. The recommended configurations are shown at each site in Figures 9 through 12.

OPTION B

All the failed and/or loose material in the affected areas should be removed and replaced with rockfill composed of a select quarry stone. The recommended configurations for each site are shown in Figures 9 through 12 armoured into the native material.

The following gradation requirement for the quarried stone is acceptable for the embankment protection.

Stone Size (mm)	Range (percent smaller)
200	100
150	60-100
106	50-70
75	25-45
37.5	10-25
16	5-15
9.5	0-10
4.75	0-5
75 micro mm	0-2

The material specified above has previously been utilized for W.P. 280-85-01, Mississagi River Remedial Measures (June 10/86) project with acceptable results.

This gradation will act as a filter to prevent internal erosion (piping) when high hydraulic gradients exist. A comparison of existing treatments with and without a geotextile filter revealed no appreciable differences in performance. Due to the above together with foreseeable difficulties which may arise with their placement we do not recommend geotextile application.

While the above material is required to be placed on the slopes of the embankments the apron formed along the river bed may be composed of a cheaper locally available rockfill.

Construction Considerations

Placement of slope protection is recommended to be accomplished in three stages as shown on the rockfill configuration drawings. Stage one would involve removing any loose or failed material at the toe and along the slope of the embankments and placing the apron along the river bed above the water level. The second stage would involve placing the Granular 'A' blanket if Option A was chosen. The third stage would involve placing the rockfill material along the length of the slope.

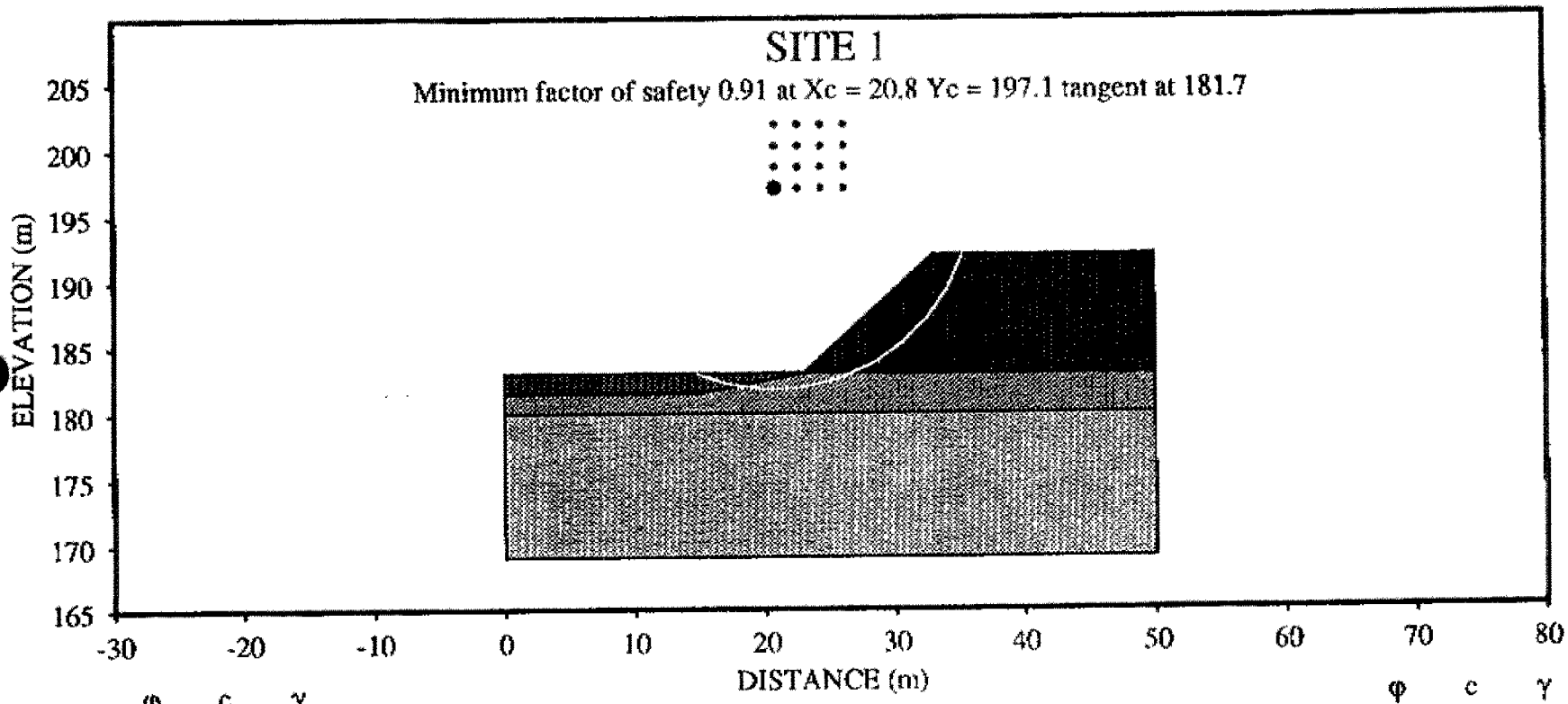
The longitudinal limits for each site are defined by the survey chainages as previously indicated. The minimum slope for the rock protection is 1.5H:1V. The minimum thicknesses are 1.5 m at the crest of the slope.

At the bridge location (Site 1) the displaced sheet piles should be removed and replaced by the rock fill configuration specified.

The rock fill placement should be in accordance with current MTO standards and practices.

We hope that these recommendations will be adequate for your purposes. If you have any questions please do not hesitate to contact this office.

M. Michalek, P. Eng.
Jr. Foundation Engineer
For:
T. Kim, P, Eng.
Sr. Foundation Engineer

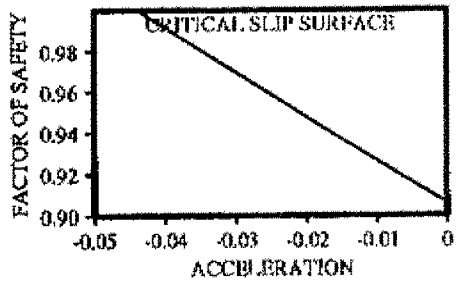


φ	c	γ	
32.0	0.0	18.9	Fine Sands
0.0	0.1	10.0	Water

φ	c	γ	
32.0	0.0	18.9	Fine Sands
28.0	3.0	18.1	Clayey Silt

CRITICAL ACCELERATIONS

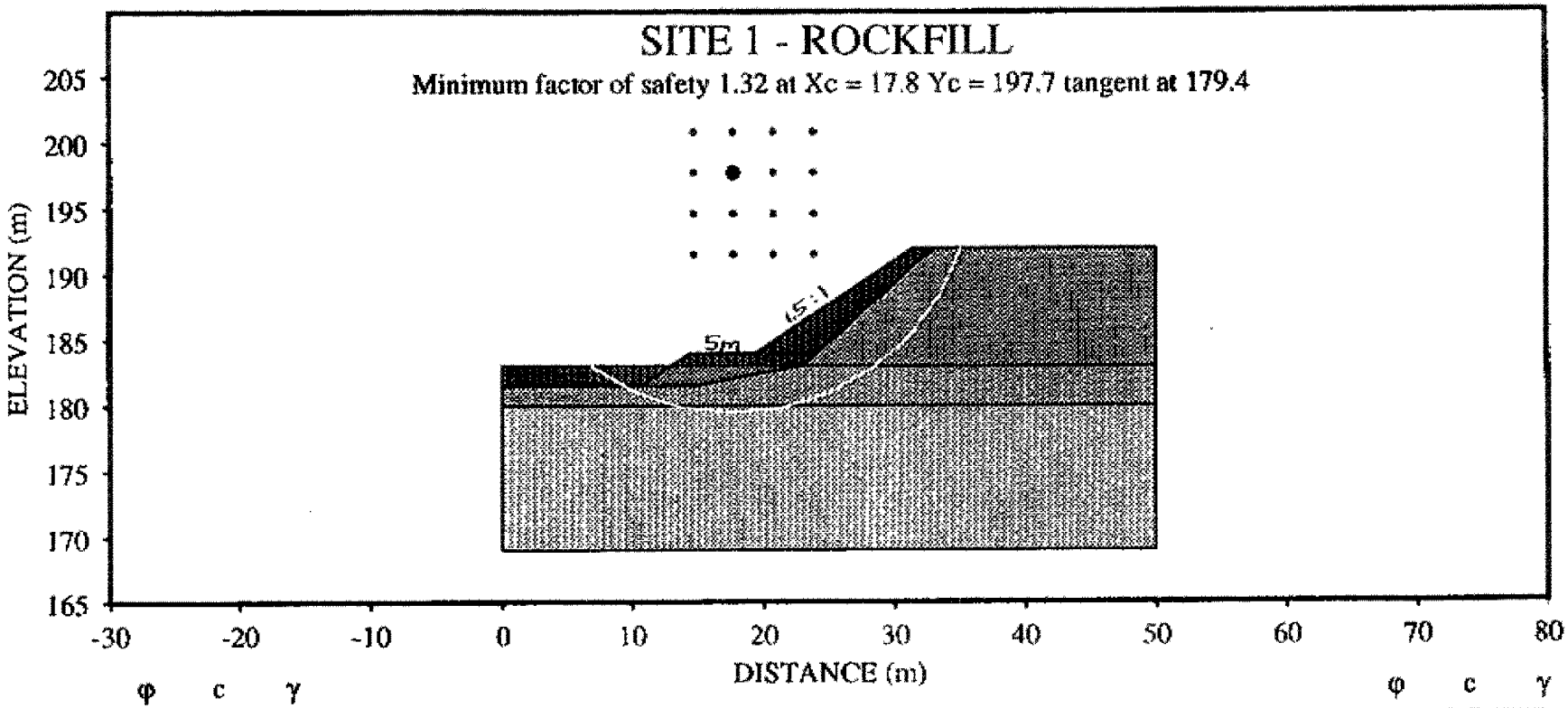
0.031	0.079	0.127	0.176
0.008	0.058	0.109	0.162
-.017	0.035	0.090	0.146
-.044	0.010	0.068	0.129



FACTORS OF SAFETY

1.074	1.204	1.352	1.528
1.018	1.145	1.288	1.467
0.960	1.081	1.227	1.405
0.906	1.022	1.164	1.341

FIG. 1 - EXISTING GEOMETRY

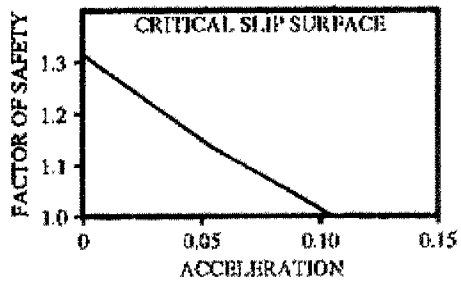


φ	c	γ	
45.0	0.0	18.1	Rock Fill
45.0	0.0	18.1	Rock Fill
0.0	0.1	10.0	Water

	φ	c	γ
Rock Fill	45.0	0.0	18.1
Fine Sands	32.0	0.0	18.9
Fine Sand	32.0	0.0	18.9
Clayey Silt	28.0	3.0	18.1

CRITICAL ACCELERATIONS

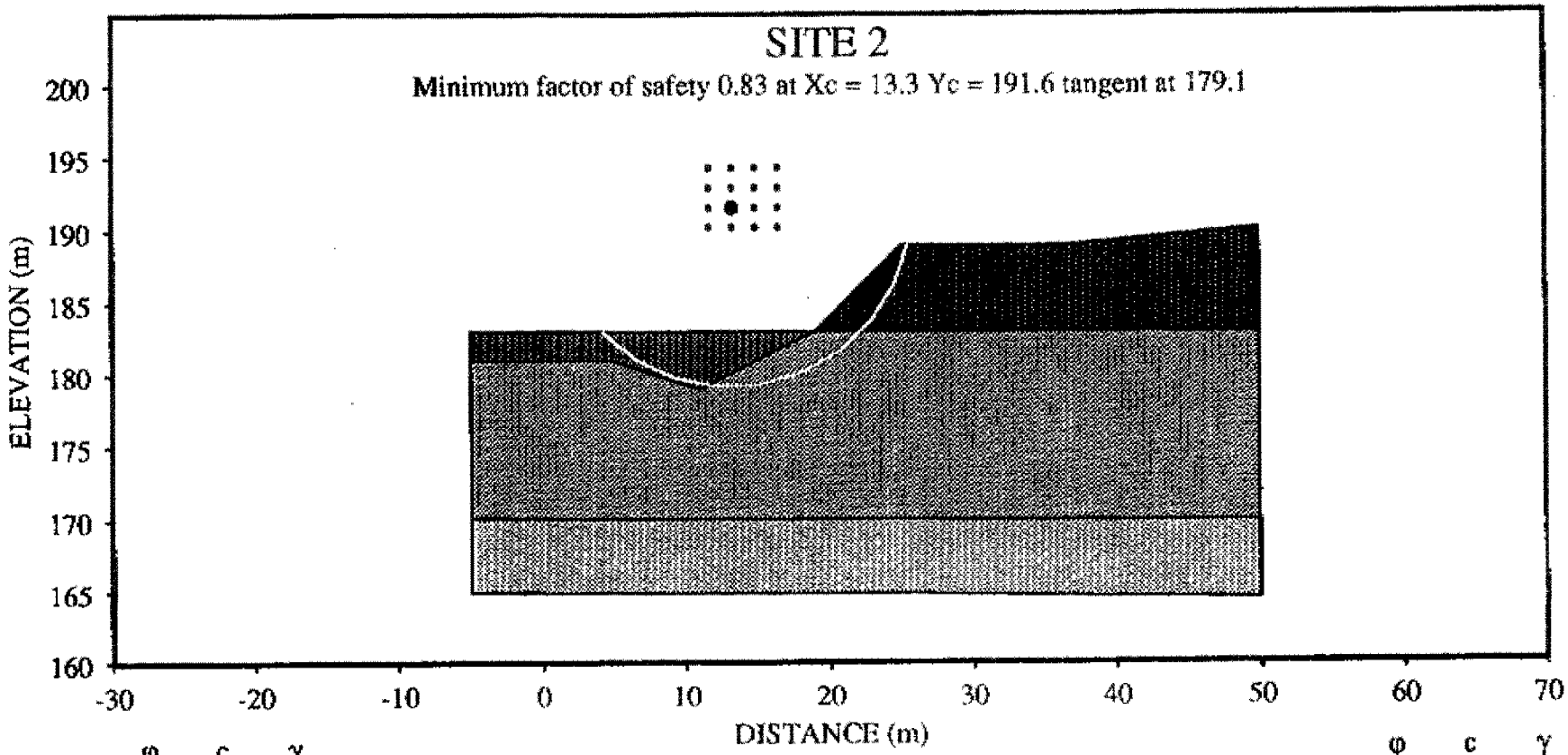
0.107	0.116	0.157	0.187
0.125	0.106	0.150	0.184
0.144	0.126	0.147	0.188
0.176	0.141	0.157	0.211



FACTORS OF SAFETY

1.345	1.364	1.535	1.765
1.417	1.316	1.467	1.654
1.537	1.376	1.428	1.573
1.647	1.440	1.493	1.697

FIG. 2 - PROPOSED STABILITY PROTECTION

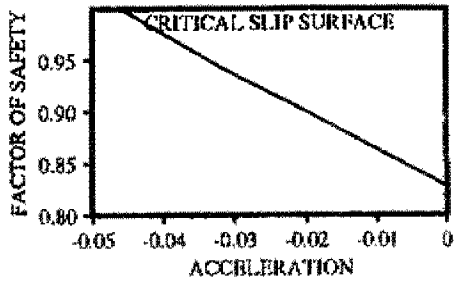


φ	c	γ	
32.0	0.0	18.9	Fine Sand
0.0	0.1	10.0	Water

φ	c	γ	
32.0	0.0	18.9	Fine Sand
28.0	3.0	18.1	Clayey Silt

CRITICAL ACCELERATIONS

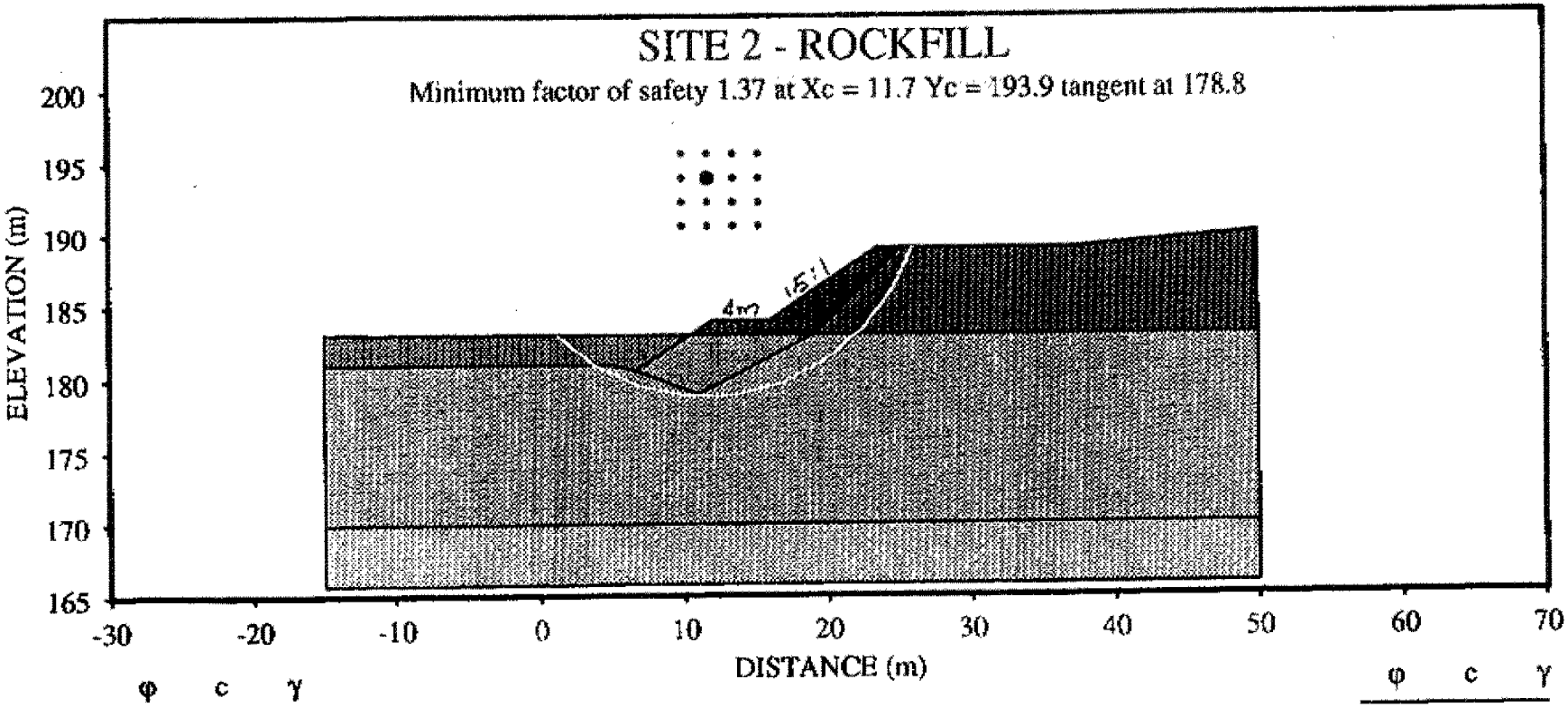
-.019	-.036	-.010	0.026
-.009	-.045	-.025	0.010
0.009	-.046	-.039	-.006
0.025	-.028	-.047	-.022



FACTORS OF SAFETY

0.918	0.875	0.968	1.081
0.956	0.841	0.918	1.031
1.056	0.829	0.879	0.982
1.245	0.875	0.854	0.936

FIG. 3 - EXISTING GEOMETRY

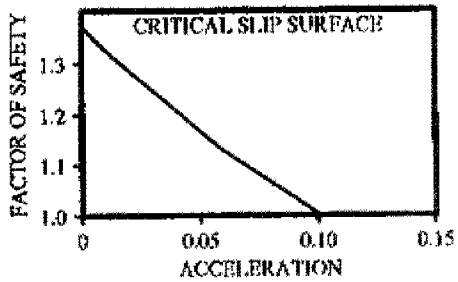


φ	c	γ	
45.0	0.0	18.1	Rock Fill
45.0	0.0	18.1	Rockfill
32.0	0.0	18.9	Fine Sand

	φ	c	γ
Water	0.0	0.1	10.0
Rock Fill	45.0	0.0	18.1
Fine Sand	32.0	0.0	18.9
Clayey Silt	28.0	3.0	18.1

CRITICAL ACCELERATIONS

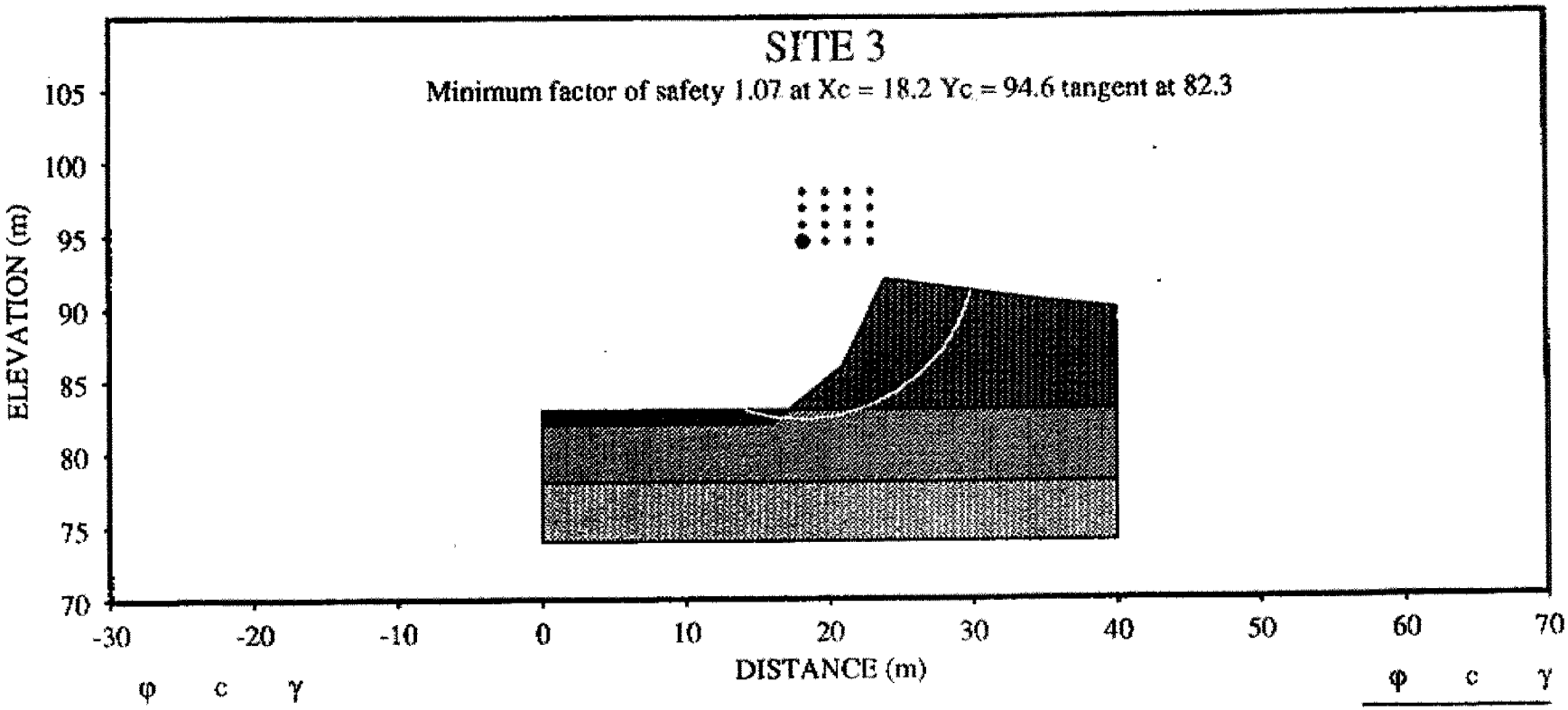
0.107	0.104	0.128	0.192
0.113	0.102	0.124	0.192
0.126	0.107	0.128	0.197
0.142	0.120	0.163	0.226



FACTORS OF SAFETY

1.427	1.388	1.467	1.678
1.470	1.371	1.434	1.638
1.574	1.392	1.428	1.612
1.727	1.467	1.515	1.612

FIG. 4 - PROPOSED STABILITY PROTECTION

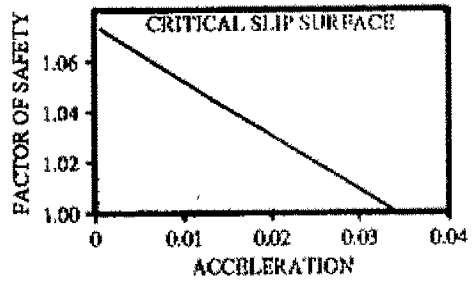


φ	c	γ	
0.0	0.1	10.0	WATER
32.0	0.0	18.9	FINE SAND

	φ	c	γ
FINE SAND	32.0	0.0	18.9
CLAYEY SILT	28.0	3.0	18.1

CRITICAL ACCELERATIONS

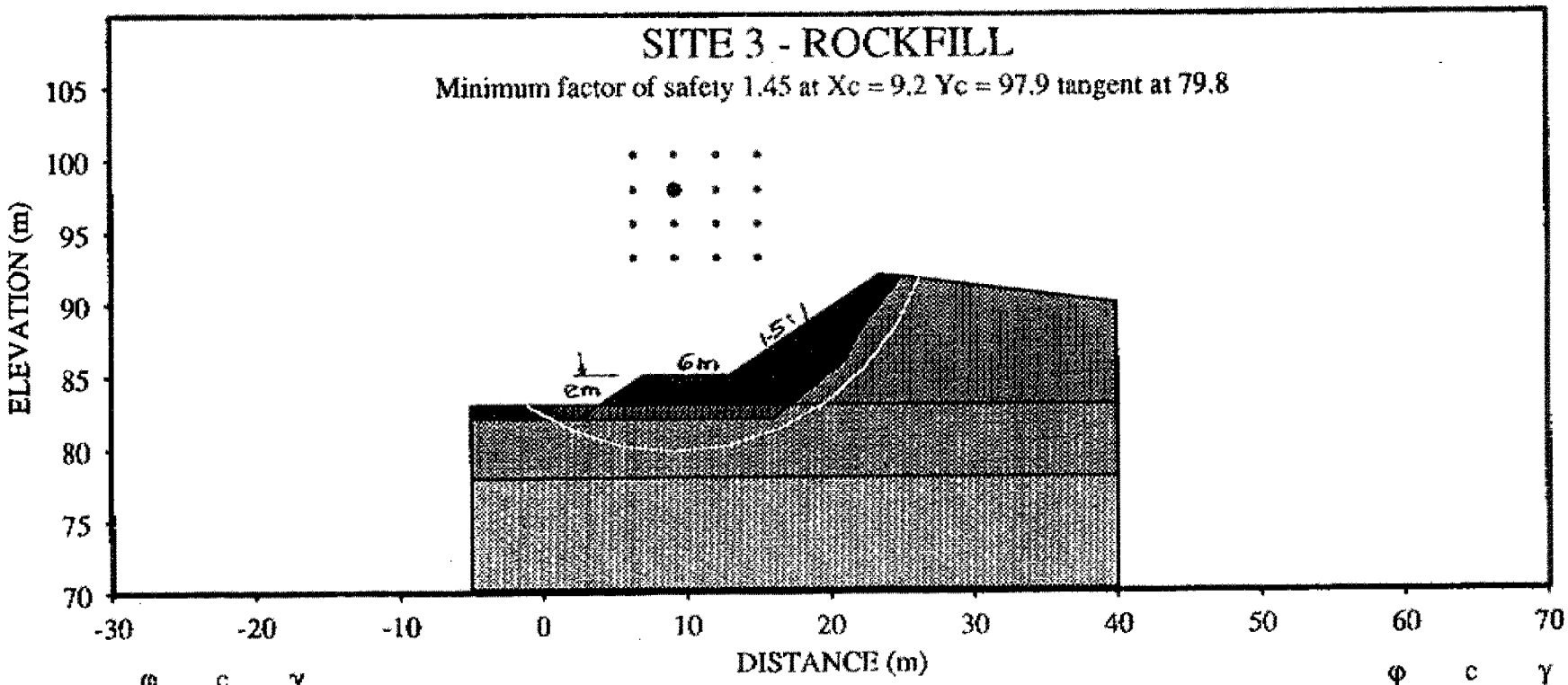
0.105	0.175	0.244	0.313
0.084	0.159	0.232	0.306
0.060	0.141	0.220	0.299
0.034	0.122	0.206	0.292



FACTORS OF SAFETY

1.260	1.492	1.776	2.160
1.200	1.429	1.711	2.095
1.138	1.364	1.646	2.030
1.073	1.296	1.580	1.965

FIG. 5 - EXISTING GEOMETRY

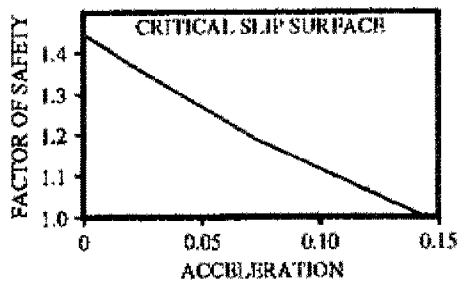


φ	c	γ	
45.0	0.0	18.1	ROCKFILL
0.0	0.1	10.0	WATER
45.0	0.0	18.1	ROCKFILL

	φ	c	γ
FINE SAND	32.0	0.0	18.9
FINE SAND	32.0	0.0	18.9
CLAYEY SILT	28.0	3.0	18.1

CRITICAL ACCELERATIONS

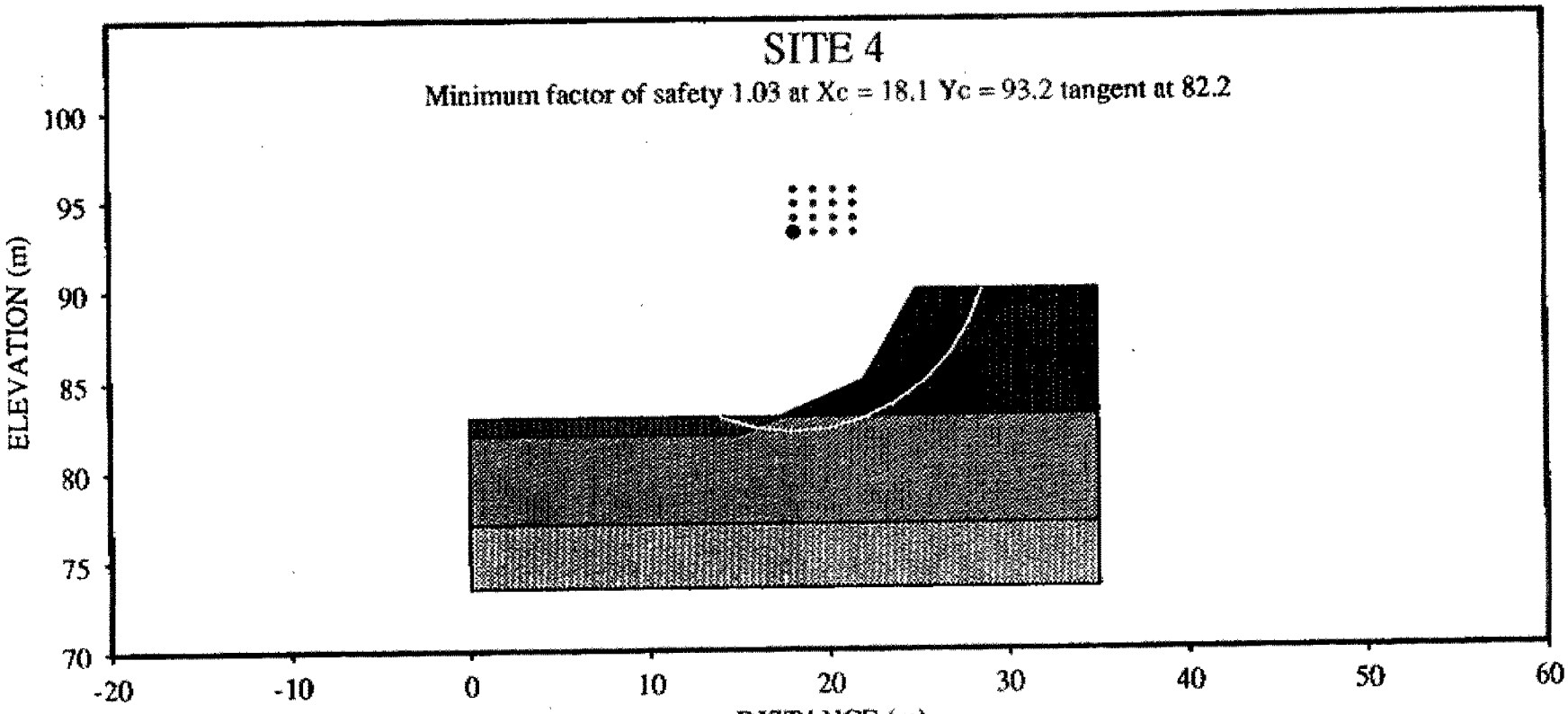
0.147	0.151	0.203	0.282
0.214	0.145	0.198	0.293
0.221	0.148	0.199	0.326
0.229	0.199	0.222	0.428



FACTORS OF SAFETY

1.500	1.468	1.695	2.055
1.848	1.451	1.638	2.000
1.945	1.455	1.600	1.950
2.065	1.684	1.615	1.892

FIG. 6 - PROPOSED STABILITY PROTECTION

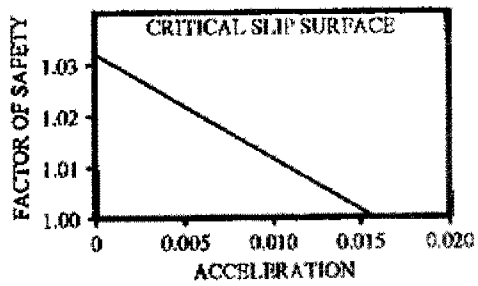


φ	c	γ	
32.0	0.0	18.9	FINE SAND
0.0	0.1	10.0	WATER

φ	c	γ	
28.0	3.0	18.1	CLAYEY SILT
28.0	3.0	18.1	SILT-SEAMS OF CLAY

CRITICAL ACCELERATIONS

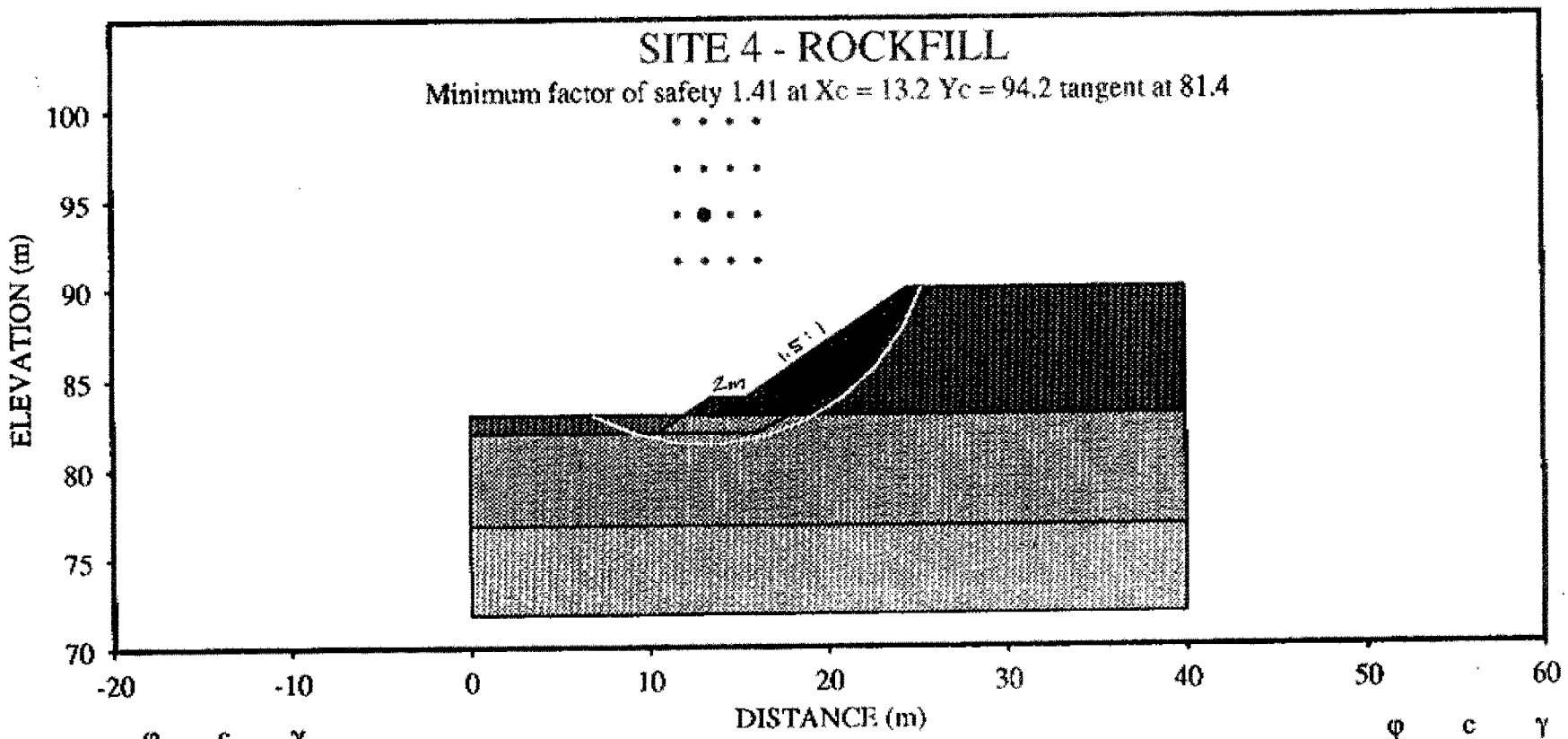
0.063	0.103	0.140	0.178
0.048	0.090	0.129	0.169
0.032	0.076	0.118	0.160
0.016	0.061	0.105	0.149



FACTORS OF SAFETY

1.143	1.250	1.365	1.501
1.106	1.211	1.326	1.462
1.067	1.172	1.285	1.423
1.032	1.133	1.246	1.383

FIG. 7 – EXISTING GEOMETRY

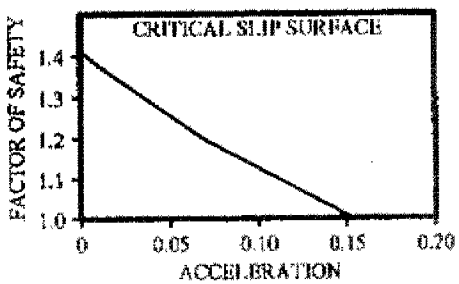


φ	c	γ	
45.0	0.0	18.1	ROCKFILL
32.0	0.0	18.9	FINE SAND
0.0	0.1	10.0	WATER

	φ	c	γ
ROCKFILL	45.0	0.0	18.1
CLAYEY SILT	28.0	3.0	18.1
SILT'-SEAMS OF CLAY	28.0	3.0	18.1

CRITICAL ACCELERATIONS

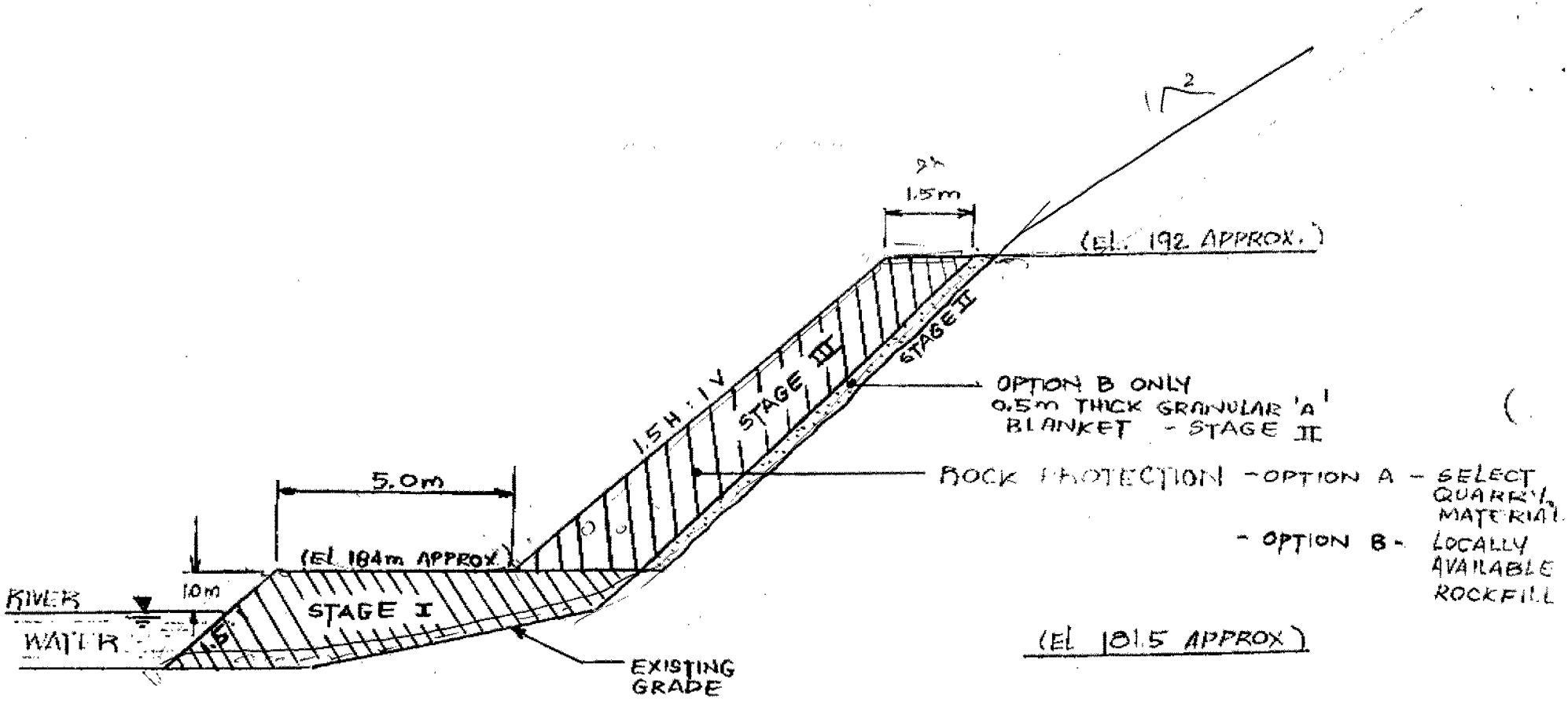
0.170	0.161	0.169	0.195
0.189	0.164	0.159	0.181
0.229	0.154	0.168	0.174
0.243	0.173	0.166	0.203



FACTORS OF SAFETY

1.485	1.449	1.481	1.609
1.547	1.448	1.441	1.527
1.684	1.412	1.446	1.471
1.832	1.514	1.432	1.533

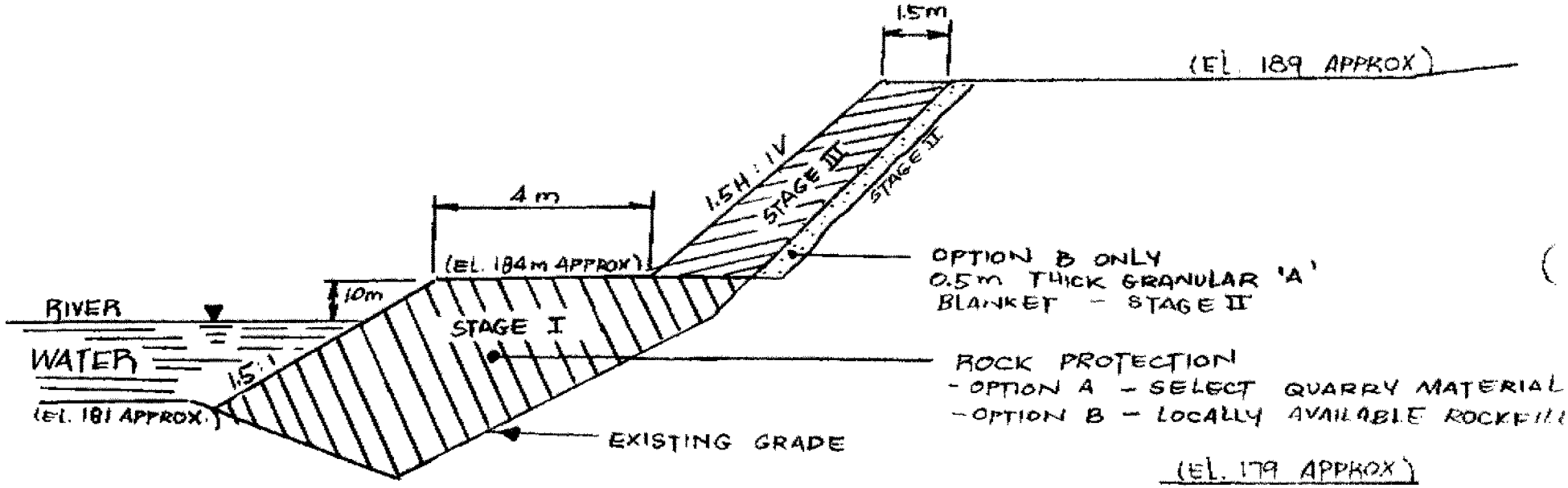
FIG. 8 - PROPOSED STABILITY PROTECTION



NTS

TREATMENT GEOMETRY - SITE 1
FIG. 9

WP 909-76-01 (A)

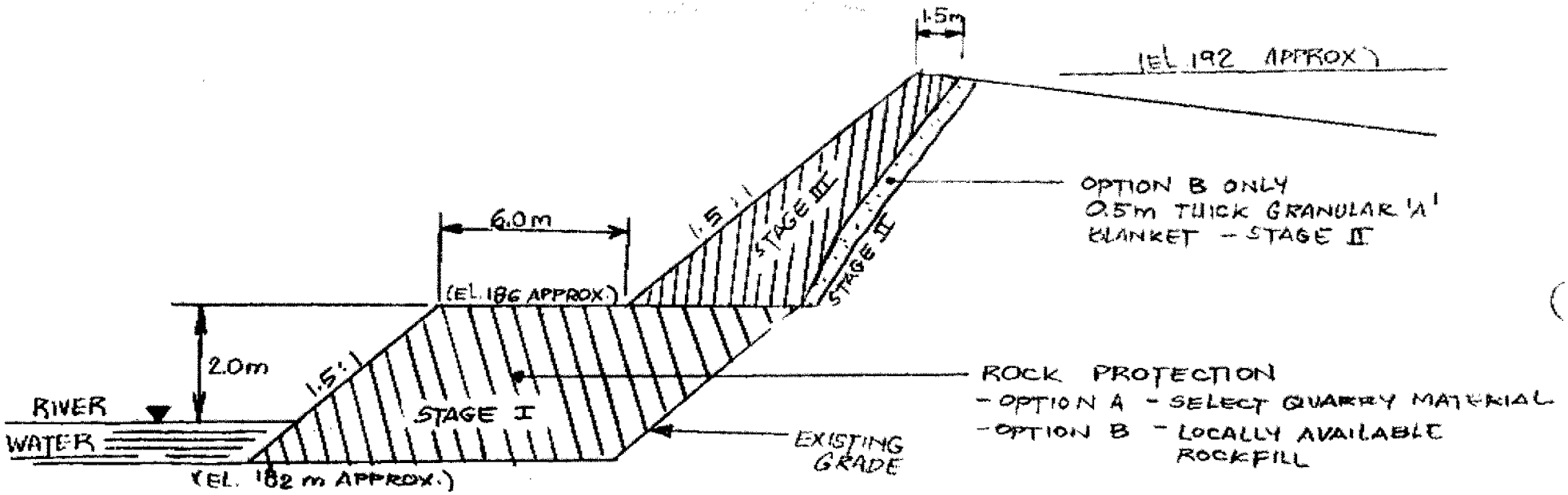


NTS

TREATMENT GEOMETRY - SITE 2

FIG. 10

WP 909-76-01 (A)

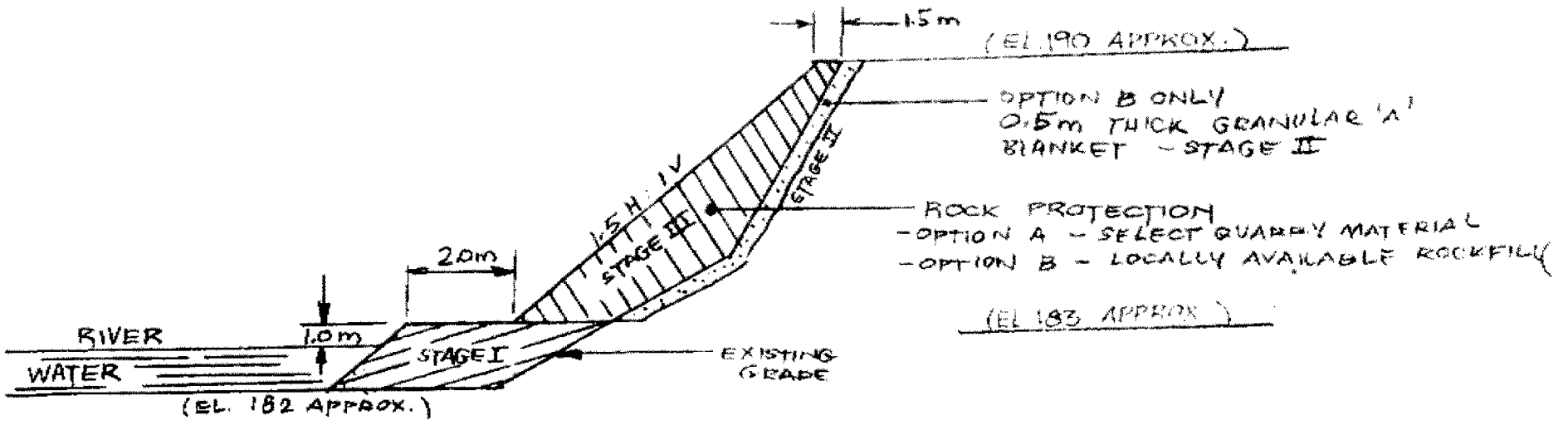


NTS

TREATMENT GEOMETRY - SITE 3

FIG. 11

WP 909-76-01(A)



NTS

TREATMENT GEOMETRY SITE 4

FIG. 12

WP 909-76-01(A)

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Northwestern Region

Date: 93 07 28

Attn.: D.C. Dykstra

From: Foundation Design Section
Room 315, Central Building
Downsview

Subject: Aubinadong River Bridge
Hwy 556, Site 38S-340
W.P. 46-87-01
District 18, Sault Ste Marie

Further to our memorandum dated 91 12 09, we summarize below the results of a supplementary foundation investigation recently carried out at the above noted site.

Two (2) boreholes were drilled, one at each abutment location during the period 93 07 06 to 93 07 07. Details of the subsoil conditions encountered are summarized in the attached Record of Borehole sheets. The subsurface stratigraphy typically comprises silt overlying sandy silt which in turn overlies a silty sand deposit at depth. The silt stratum contains trace sand and clay and is expected to physically behave as the sandy silt material described in the previous investigation.

With reference to your memorandum dated 92 12 03, design pile tip elevations at the west and east abutments are 310.0 m and 309.0 m respectively. The new boreholes were advanced to 34.0 m below the existing ground surface, to El. 303.5 m at the west abutment location (BH 9) and 303.3 m at the east abutment location (BH 10), some 6 $\pm$ m below the proposed pile tip elevations. The soil strata encountered in these boreholes are similar to those encountered during the earlier investigation.

The locations of the present boreholes are shown on the attached marked-up plan. During the present drilling operation, 4.6 m long hollow stem auger was lost in BH 9 location at a depth of 13.7 $\pm$ m. This should be shown on the contract drawings to advise the contractor of possible buried obstruction in his area of work.

HP 310x79 steel piles should be adopted as they provide the same surface area as 310x110 piles. Based on our estimation of the piling cost alone (pile cap not included), the timber pile alternative recommended before is more cost-effective than steel piles. In the original consultant's report, the SLS Type II capacity recommended for Size 36 timber

pile is 200 kN. In our opinion, the capacity can be increased to 250 kN, for your information.

Should there be any questions, please contact us.

David Kwok, P. Eng.
Project Foundation Engineer
for
Balu Iyer, P. Eng.
Senior Foundation Engineer

RECORD OF BOREHOLE No 9 1 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta 13+750.4 CL ORIGINATED BY DK

DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

DATUM Geodetic DATE 93 07 07 CHECKED BY BI

SOIL PROFILE		STRAT PLOT	SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB VANE 10 20 30 40 50	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION		NUMBER	TYPE	'N' VALUES						
337.5	Ground Surface										
0.0							336				
							334				
			1	SS	8		332	AUGER			
	Silt						330				
	Trace Sand and Clay		2	SS	13		328				
	Grey, Loose to Dense						326				
			3	SS	32		324				
							322				
			4	SS	45		320				
317.7							318				
19.8								WASHBORING			
			5	SS	12		316				
	Sandy Silt, Trace Clay						314				
	Grey, Compact										
			6	WS	–		312				
310.1											
27.4			7	SS	15		310				
							308				

Continued

Continued

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 9 2 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta 13+750.4 CL ORIGINATED BY DK

DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

DATUM Geodetic DATE 93 07 07 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 10 20 30 40 50	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
30.5	Continued										
	Silty Sand Grey, Compact		8	SS	27		306	WASHBORING			
303.5			9	SS	26		304				
34.0	End of Borehole ▪ 93 07 07										

+³, ×⁵ : Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 10 1 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta. 13+799 o/s Rt 1.5 m ORIGINATED BY DK

DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

DATUM Geodetic DATE 93 07 06 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 10 20 30 40 50	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
337.3	Ground Surface										
0.0						▼*	336				
							334				
			1	SS	10		332				
	Silt						330	AUGER			
	Trace Sand and Clay		2	SS	8		328				
	Grey, Loose to Compact						326				
							324				
			3	SS	18		322				
							320				
			4	SS	25		318				
317.5											
19.8							316				
			5	SS	18			WASHBORING			
	Sandy Silt, Trace Clay						314				
			6	SS	8		312				
	Grey, Loose to Compact										
			7	SS	27		310				
308.0											
29.3							308				

Continued

Continued

$+^3$, $\times^5$: Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 10 2 OF 2 METRIC

W.P. 46-87-01 LOCATION Sta. 13+799 o/s Rt 1.5 m ORIGINATED BY DK

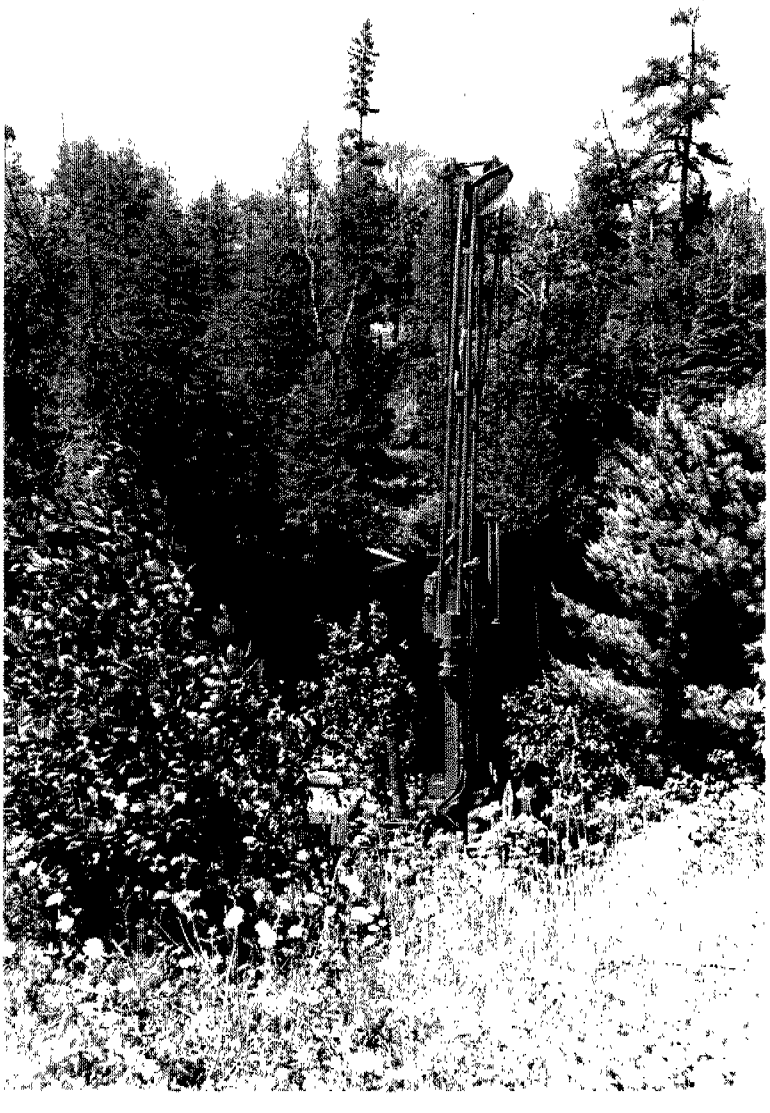
DIST 18 HWY 556 BOREHOLE TYPE H.S. Auger, Washboring COMPILED BY DK

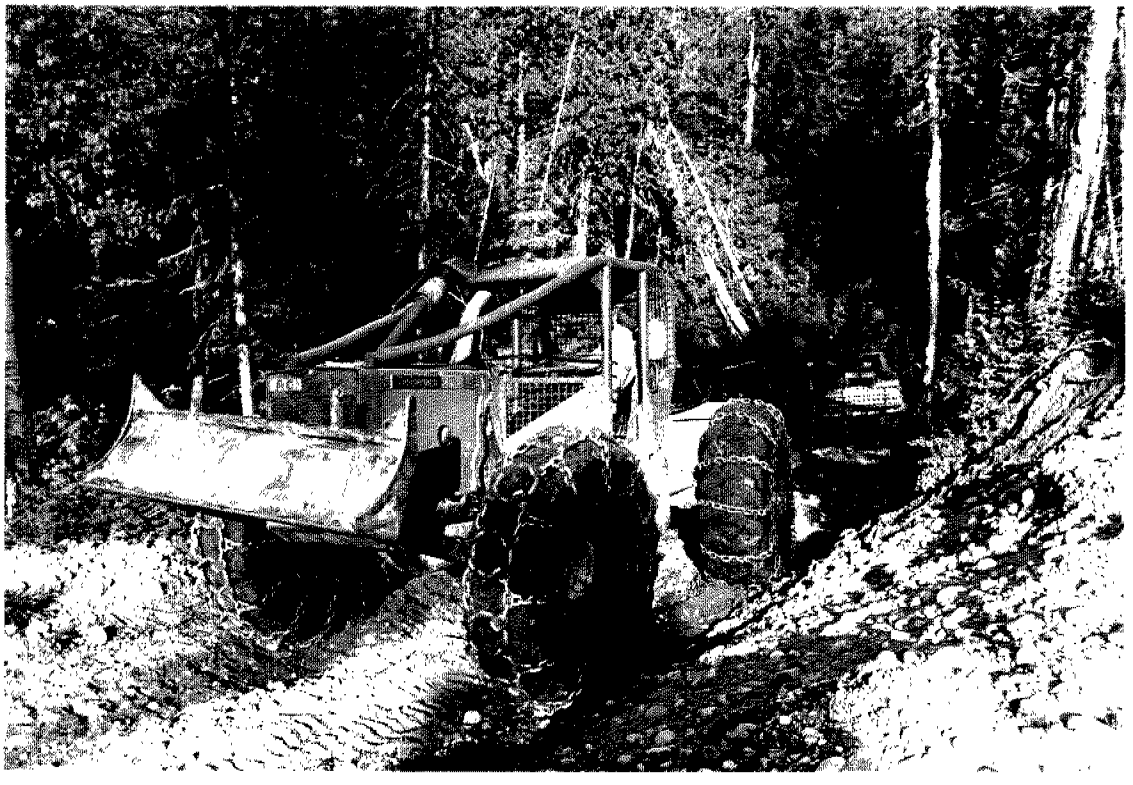
DATUM Geodetic DATE 93 07 06 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 10 20 30 40 50	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
30.5	Continued										
	Silty Sand and Gravel Pinkish Grey, Very Dense		8	SS	100	/15cm	306	WASHBORING			
303.3			9	WS	~		304				
34.0	End of Borehole * 93 07 07										

$+^5$, $\times^5$: Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10





memorandum

Ontario

To: R. Krisciunas ✓
Head, Structural Section
Northwestern Region

Date: 1993 05 21

Attn: D. Dykstra

From: Foundation Design Section
Room 315, Central Building

Re: Additional Foundation Investigations
W.P. 46-87-01, Aubinadong River Bridge
W.P. 185-90-01, White River Farm Culvert
District 18, Sault Ste. Marie

We were planning to undertake additional foundation investigations at the Aubinadong River Bridge site and the White River Farm Culvert site during the week of 1993 05 31. However, based on recent discussions with you and others in your office, we understand that the MNR may have concern related to trout hatching in the water bodies adjacent to these two sites especially if any work is carried out before mid-June.

To avoid unnecessary concerns related to our investigations, we propose to postpone our field work to later half of June. This should not cause any problem to you, since we have provided adequate information for the design and contract preparation activities to proceed without any delay.

Please call us if you have queries on these two projects.

Balu Iyer, P.Eng.
Sr. Foundation Engineer

cc. P. Bound

MEMORANDUM

TO: Mr. Balu Iyer
Senior Foundation Engineer
Foundation Design Section
Downsview
Fax (416)235-5240

FROM: Structural Section
Northwestern Region
807/473-2149
807/473-2167 (fax)

Attn: D. Kwok

DATE:December 3, 1992

RE: **Aubinadong River Bridge**
Site #38S-340, Highway 556
District 18, Sault Ste. Marie
W.P. 46-87-01

We have reviewed your memorandum dated November 23, 1992. Based on "theoretical" pile capacities and lengths supplied to us in your December 9, 1991 memo, please be advised that the steel H-pile tip elevations for the proposed Aubinadong River Bridge will be as follows:

LOCATION	PILE TIP ELEVATION
West Abutment	310.000
East Abutment	309.000

Since the depth of the boreholes of the original foundation investigation are above our proposed pile tip elevations, we assume you will be initiating a supplementary field investigation. Please advise us of the results when they become available.

D. C. Dykstra, P. Eng.
Structural Engineer

DCD/bh

cc: R. Radolli

P.S. Per my discussion with David, the supplementary results must be available by June 4, 1993.

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Northwestern Region

Date: 1992 11 23

Attn: D.C. Dykstra

From: Foundation Design Section
Room 315, Central Bldg.
Downsview

Re: Aubinadong River Bridge
Hwy. 556, Site 38S-340
W.P. 46-87-01
District 18, Sault Ste Marie

We refer to your memorandum dated 92 11 03 and the Preliminary General Arrangement Drawing P1 attached therein, for the above-noted project and have the following comments;

- The foundation investigation was carried out based on a proposed three span structure or a 42 m single span structure. The Preliminary General Arrangement Drawing shows a 52 m single span bridge. The foundation report contains sufficient information for the structure provided that friction piles to depths shallower than the boreholes are used.

- The design pile lengths are not shown on the drawing. As mentioned in our memorandum dated 91 12 09, additional investigation will be required before the design is finalized, if piles are driven to an elevation lower than 315 m at the west abutment and 318 m at the east abutment locations. Please let us know in advance so that we can schedule for the supplementary investigation.

  Steel H-piles should be equipped with reinforced tips (OPSD-3301.00) to facilitate pile penetration through the surficial layer of sand and gravel with cobbles.

- We believe that the H-piles for the timber lagging wall would extend into the native soils.

Should there be further questions, please contact our office.

D. Kwok, P. Eng.
Project Foundation Engineer
for
B. Iyer, P. Eng.
Senior Foundation Engineer

BI/DK/jb

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Northwestern Region

Date: 1992 07 20

Attn: W. Prystanski

From: Foundation Design Section
Room 315, Central Building, Downsview

Re: Aubinadong River Bridge
Hwy 556 - Site 38S - 340
W.P. 46-87-01
District 18, Sault Ste Marie

This memo is in reply to the letter dated 1992 07 13 from M.M. Dillon, your design consultant.

1. For lightweight fill, an active lateral earth pressure coefficient of 0.33 shall be used.

2. For the details shown on two sketches which accompanied the reference letter, the embedded pile lengths shall be calculated from below El 337.77 m at the east abutment and El 338.25 m at the west abutment.

We will provide further comments when the general arrangement drawing is forwarded to this office for review. In the meantime, please contact this office if you require further elaboration on items discussed in this memo.

Balu Iyer, P.Eng.
Sr. Foundation Engineer

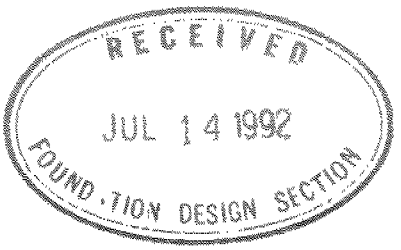
cc. B. Farago, Structural Office

A:468701.207

DILLON
Consulting Engineers • Planners
Environmental Scientists

Our File: 3400-03

13 July 1992



Ministry of Transportation, Ontario
Foundation Design Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

ATTENTION: Dr. B. Iyer, P.Eng.
Senior Foundation Engineer

Aubinadong River Bridge
Highway 556 - Site 38S - 340
W.P. 46-87-01

Dear Sirs:

On 9 December 1991 you provided us with theoretical pile lengths and capacities for steel H-piles varying in length from 20 to 30 m for the above noted site. As you may recall, the foundation investigation at this site extended only to a depth of 23 m below existing grade, and it was necessary for your office to assess the theoretical capacities of piles extended beyond the 23 m level. The data that was provided (attached for your reference) did not give a pile tip elevation, but rather an embedded length of pile, with corresponding capacities.

At the west abutment of the Aubinadong River bridge, it will be necessary to excavate a layer of peat before the bridge abutment is constructed. We anticipate that after the peat excavation is complete, the excavation will be backfilled, prior to the start of pile drivings. Cross-sections at both the east and west abutments are enclosed for your reference. In our design of the abutment, we are assuming that the length of embedded pile contributing to individual pile capacity, is only that length embedded in the original, undisturbed soil; i.e. the length of pile above the peat excavation does not add to the frictional resistance of the pile. Can you please confirm if this assumption is valid, or should we consider that some of the backfill material contributes to the pile capacity?

. . . continued

As you can see from the sections, we are proposing to use lightweight slag backfill at the abutments. Your office has previously supplied us with an in-situ density for this material. Can you please advise what angle of internal friction (Ø) we should assume for this material.

Should you require any clarification of the enclosed, please contact our office.

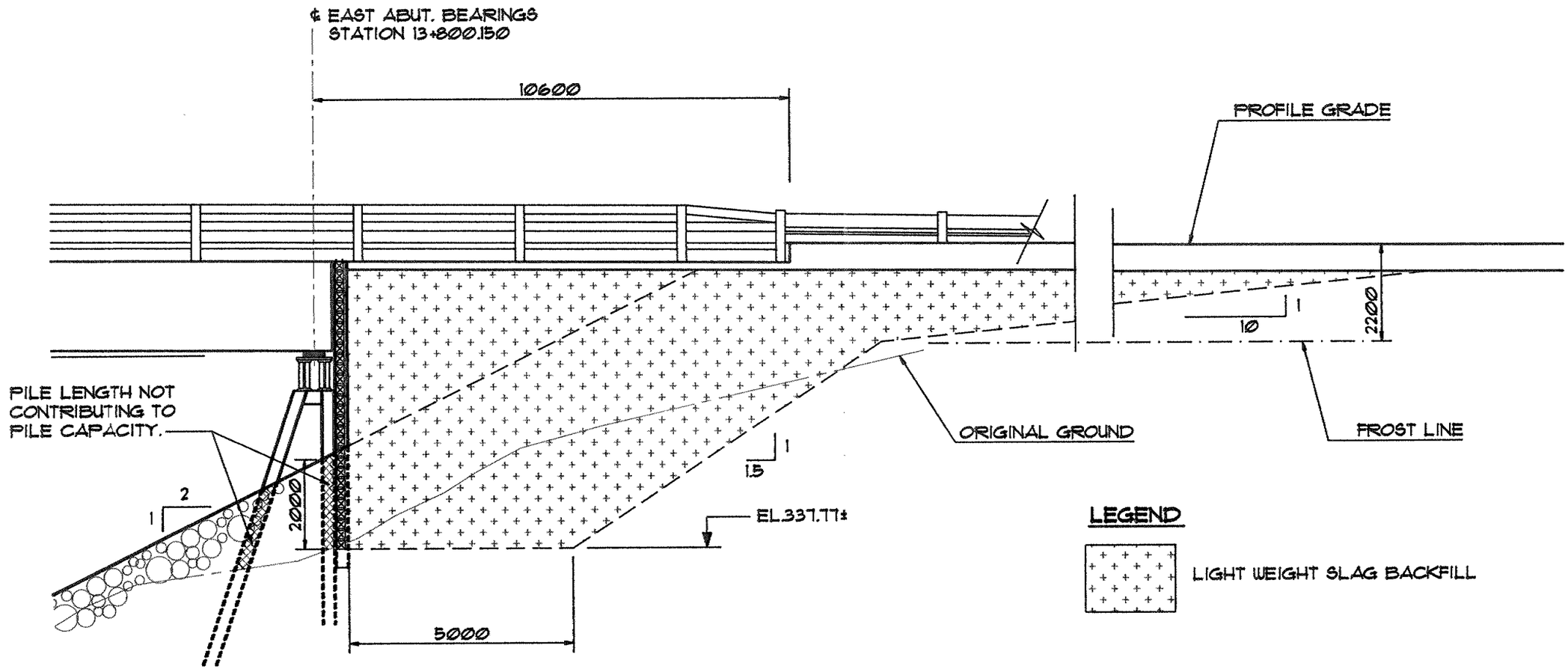
Yours truly,

M.M. DILLON LIMITED

R. Radolli, P.Eng.
Project Manager

RR:gjc

cc: W. Prystanski, MTO - N/W Region
P. Tam, M.M. Dillon Ltd.



DRAWING NOT TO BE SCALED
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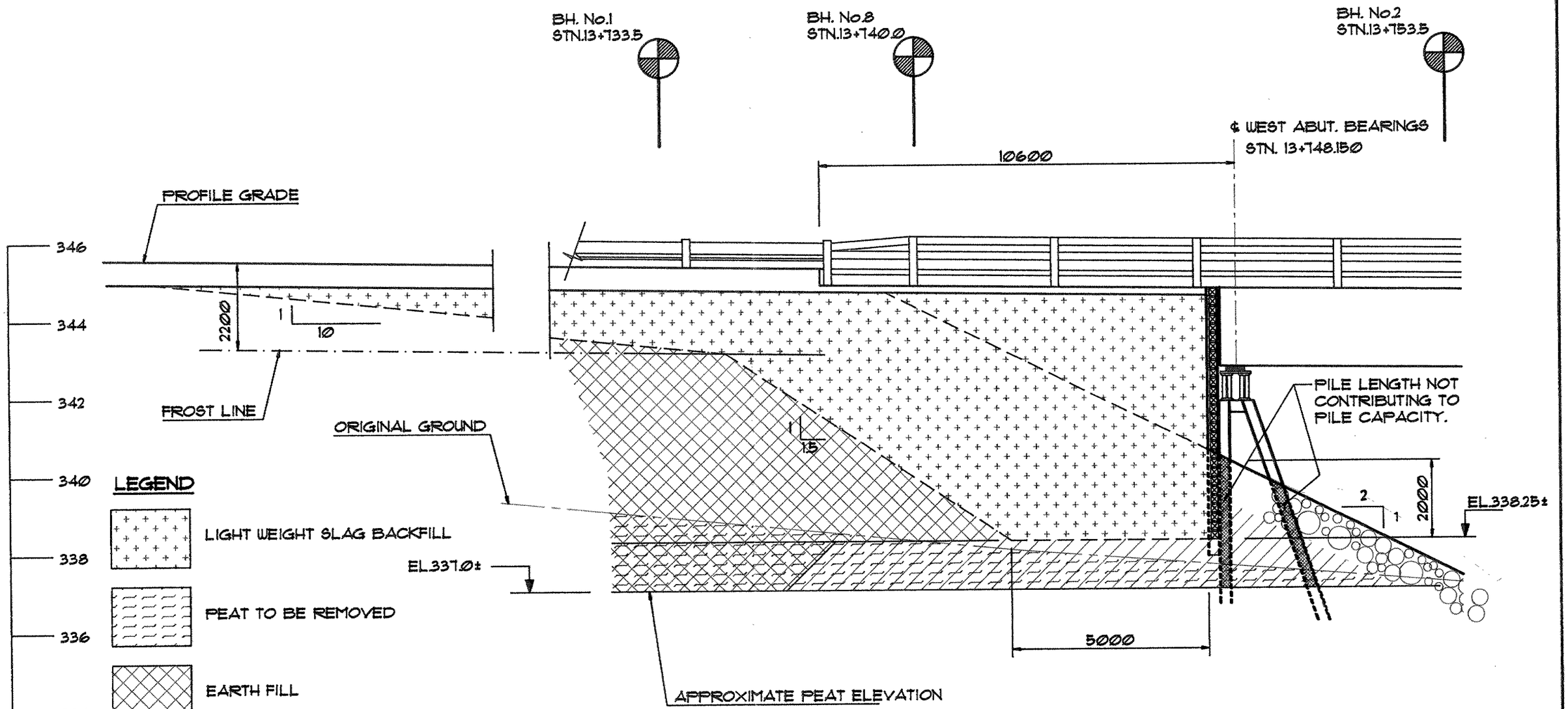
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DILLON
Consulting Engineers • Planners
Environmental Scientists

HIGHWAY 556 LINE 'A'
OVER AUBINADONG RIVER
EAST ABUTMENT
MINIMUM BACKFILL REQUIREMENT

DIST. No. 18

WP. No. 46-87-00



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

METRIC
DIMENSIONS ARE IN METRES
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UNLESS OTHERWISE SHOWN

DILLON
Consulting Engineers • Planners
Environmental Scientists

HIGHWAY 556 LINE 'A' OVER AUBINADONG RIVER
WEST ABUTMENT
MINIMUM BACKFILL REQUIREMENT

DIST. No. 18

WP. No. 46-87-00

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Northwestern Region

Date: 1992 07 03

Attn: W. C. Prystanski

From: Foundation Design Section
Room 315, Central Bldg.
Downsview

Re: Aubinadong River Bridge
Hwy. 556 - Site 38S-340
W.P. 46-87-01
District 18, Sault Ste. Marie

This is further to your memo of 1992 06 08 and our telephone conversation earlier today.

1. NSSP for Lightweight Fill

   We are now in the process of producing a "generic" NSSP for lightweight fill. As soon as this is ready, we will forward a copy to you.

2a. For design purposes, you may use an in-situ density of 13 kN/m^3, assuming that the lightweight fill consists of Algoma Steel's iron blast furnace slag obtained from Reiss Lime Company, Sault Ste. Marie, Ontario.

2b. It is not clear to us why there is concern regarding the elevation below which pile friction is assumed in our calculations. Please explain your concern in more detail and we will assist you in whatever way we can.

3. If the abutment walls can accommodate movements as specified in the OHBDC to make it a flexible wall, then active earth pressures can be used in your design.

Please feel free to contact us if you need elaboration on items 2 or 3 above.

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/jb

Ministry of Transportation
Ontario

FAXGRAM

PLEASE TYPE

DATE June 8, 1992

PAGE ____ OF ____

TO: Mr. M. Devata Chief Foundation Engineer Foundation Design Section	FROM: W.C. Prystanski Structural Section Northwestern Region 807 - 473 - 2063

SUBJECT: W.P. 46-87-01, Aubinadong River Bridge
Site 38S-340, Hwy. 556
District 18, Sault Ste. Marie

As we discussed by telephone on June 4, 1992, our consultant, M.M. Dillon Ltd. would like a few issues clarified with respect to the Foundation Report for the above noted structure before proceeding with the design. I have listed the issues below and would appreciate your comments.

1. The use of lightweight is an integral part of the design. Please provide properties of the lightweight fill, source and a copy of the Non-Standard Special Provision against this item presently being developed by Dave Dundas from your section.

2. The Foundation Design Report recommends removal of some peat at the west abutment. Friction piles will be used and therefore the following information will be required:

   a. the nature of the backfill to be used;
   b. at what elevation can we assume that pile friction is starting to be developed.

3. The Foundation Design Report recommends that rigid walls of the bridge abutments be designed to withstand "at-rest" earth pressures. However, the abutments will be designed similar to that used at the North Pagwachuan River Bridge. The bridge girders will be supported on steel friction piles topped with steel cap beams. This foundation will be flexible and therefore would not the "active" condition be more appropriate.

Your comments on the above would be greatly appreciated.

W. Prystanski

W.C. Prystanski
Senior Structural Engineer

WCP:gk
copy via fax to M.M. Dillon

RECEIVED
JUN - 9 1992
FOUNDATION DESIGN SECTION

B.I

DILLON
Consulting Engineers • Planners
Environmental Scientists

Our File: 3400-02

13 December 1991

Ministry of Transportation, Ontario
Central Building
Room 315
Foundation Design Section
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

ATTENTION: Mr. Balu Iyer, P.Eng.
Senior Foundation Engineer

Aubinadong River Bridge
W.P. 46-87-00 - Site 38S-380
Use of Lightweight Fill

Dear Sirs:

During our recent meeting, there was some discussion regarding the use of lightweight blast furnace slag as granular backfill to the bridge abutments for the Aubinadong River Bridge. The structure is located near Sault Ste. Marie and, it is realistic to assume that any lightweight fill will come from Algoma Steel.

In previous projects in the Sault area (Echo Bay Bridge), your section has recommended the following properties for lightweight blast furnace slag material:

- unit weight of 12.7 kN/m^3
- $\emptyset = 35°$

Additionally, we understand that MTO Foundation Section has developed a Special Provision for use with lightweight fill that, among other things, stipulates the gradation and compactive effort requirements of the lightweight material.

At our meeting, it was agreed that your office will provide theoretical pile lengths and allowable pile loads for steel H-piles at the Aubinadong River Bridge site. As part of the information that is provided, can you also supply the following:

. . . continued

i) confirmation of properties of lightweight fill obtained in the Sault area.

ii) a copy of the Special Provision to be used when lightweight blast furnace slag is specified to be used.

We thank you for your attention in this matter. Please call if you have any questions.

Yours truly,

M.M. DILLON LIMITED

R. Radolli, P.Eng.
Project Manager

RR:gjc

cc: W. Prystanski - MTO, Thunder Bay

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Northwestern Region

Date: 1991 12 09

Attn: W. Prystanski

From: Foundation Design Section
Room 315, Central Bldg.

Re: Theoretical Pile Lengths
and Capacities
Aubinadong River Bridge
W.P. 46-87-01
District 18, Sault Ste. Marie

At a meeting held on 1991 12 05 and attended by R. Radolli of M.M. Dillon and the undersigned, it was agreed that we would provide some theoretical pile lengths and capacities for the design of foundations for the above structure. This memo contains this information in Table 1.

Please note that the foundation investigation at this site extended only to a depth of 23 m below existing grade. Should the selected pile capacities are such that the theoretical pile lengths are greater than 23 m, then there may be a need to carry out an additional subsurface investigation at this site. The scope and extent of the future investigation will be defined by this section, based on the final design of the structure foundations. If an additional field investigation is not considered necessary, then this office will prepare an S.P. to be included in the contract package to handle any construction related problems.

Please contact this office should you require elaboration on any item discussed herein.

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/jb

cc: R. Radolli, M.M. Dillon

TABLE 1

Theoretical Pile Lengths and Capacities of 310 X 110 Steel H-pile

Length m	Factored ULS * Kn	SLS * Kn
20	600	300
25	900	450
26	1000	500
30	1350	675

* The pile capacities are computed assuming that the contribution from end bearing is negligible.

Memo To	File 3400-02
cc	B. Iyer - MTO, Foundation Office, Toronto W. Prystanski - MTO, Northwestern Region, Thunder Bay
From	R. Radolli
Subject	**Aubinadong River Bridge** **W.P. 46-87-00 - Site 38S-340**
File No	3400-02
Date	10 December 1991

A meeting was held on 5 December 1991 at the offices of MTO Foundation Design Section to review the use of steel H-Piles at the above noted bridge site. Those in attendance were:

B. Iyer - MTO Foundation Design Office
R. Radolli - M.M. Dillon Ltd.

The following summarizes the discussion:

- R. Radolli noted that the original Foundation Design Report was based on two structural options; a 42 m single span and a three span structure. In fact, analyses have indicated that the most economically feasible alternative is a 52 m span.

- Additionally, because of the difficulty in obtaining concrete at this site, an abutment type consisting of steel H-piles and timber lagging is being considered. The original Foundation report did not give recommendations for steel H-piles.

- Recommendations for steel H-piles were given in a subsequent memo from Foundation Office. The recommendations were given for 20 m long H-piles, simply because the boreholes at this site were only extended to this depth.

- The initial recommendation for the steel H-piles would require the use of 16 piles at each abutment (4 per girder). The purpose of the meeting was to see if a longer pile could be used, thereby reducing the number of piles at each abutment to eight (2 per girder). It was noted by R. Radolli that the use of eight piles would require pile capacities of 900-1000 kN at ULS and approximately 600 kN at SLS, based on structural requirements.

- It was noted by B. Iyer that using a longer pile may cause contractual difficulties during construction if refusal is reached before the specified length. With the information available, this cannot be confirmed.

To file

- After some discussion, B. Iyer recommended as follows:

  a) design can proceed with 8 piles/abutment (2 piles per girder)

  b) MTO Foundation Office will provide

     i) theoretical pile length
     ii) allowable loads for the theoretical pile length for ULS and SLS

  c) MTO Foundation Office will confirm the theoretical pile length by additional boreholes at the site.

  d) timing for additional boreholes will be coordinated between MTO Foundation Office and Structural Section, Northwestern Region.

  e) Foundation Office will provide appropriate wording for an SP, in the event the theoretical pile lengths are not achieved.

- It was also noted by R. Radolli that with the use of a 52 m span, piles will be required to be driven through a layer of sand and gravel at both abutments. The original foundation report recommended that timber piles not be driven through this layer.

- B. Iyer noted that with steel H-piles, there should be no difficulty in driving through this layer. Piles should be equipped with driving shoes.

Discussed this w. Wayne P. Asked him to refer to my notes to him on this topic
BA

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Thunder Bay

Date: 1991 10 29

Atten: W. Prystanski

From: Foundation Design Section
Room 315, Central Building

Re: Foundation Recommendations
Aubinadong River Bridge
W.P. 46-87-00, Site 38S-340
Highway 556
District 18, Sault Ste. Marie

Further to your memo dated 1991 09 30, we have reviewed the foundation recommendations for the above structure.

Single Span Bridge

For a single span option, it was recommended that the abutments could be supported on timber piles. Factored ULS and SLS capacities for a 14 m long (embedded length) x 360 mm butt diameter x 240 mm tip diameter timber piles were given as 400 kN and 200 kN respectively. Alternate recommendations were given in the foundation report for footings on natural ground and footings on engineered fill.

Three Span Bridge

For the three span option, it was recommended that the pier footings may be supported on timber piles as discussed above. Driven piles were not recommended for the abutment footings, because of the presence of cobbles and boulders encountered in the sand and gravel layers. Instead, spread footings on natural soil or on engineered fill were considered viable for the abutment foundations.

If timber piles are used as discussed above, no problems will be encountered during their installation, provided the piles supplied are straight and not knotted.

To File

Use of Steel H piles

Steel H piles could be used at the site instead of timber piles at the abutment locations for the single span option and for the pier footings for the three span option. The boreholes and associated penetration tests done at these locations were about 20 to 23 m deep and did not encounter an "end bearing" stratum. Thus the recommendations given below are for steel H piles approximately 20 m long (embedded length), designed as friction piles. The factored ULS capacity of a 20 m long hP 310 x 79 pile would be about 600 kN.

Closure

The recommendations given in the foundation report are for 14 m long timber piles. The factored ULS value of a longer pile, say 20 m embedded length, would be about the same as that of a 310 x 79 steel H pile given in this memo. From consideration of the foundations soil conditions, timber and steel H piles could be installed at all locations suggested herein. The closure would therefore be based on local availability of the suitable pile and cost.

Please call us if you need further input from us on this subject.

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

BI/MD/mmj

MEMORANDUM

TO: Foundation Design Section
3rd Floor Central Building
Downsview

DATE: September 30, 1991

ATTENTION: **Mr. B. Iyer**

FROM: Structural Section
Northwestern Region
807/473-2063

Re: **W.P. 46-87-00, Aubinadong River Bridge**
Site No. 38S-340, Hwy. 556
District 18, Sault Ste. Marie

Further to my memo of September 14, 1991, we feel that longer steel H-piles is a viable option for the replacement structure foundation. This is based on the low capacities given for the relatively short timber piles and low bearing capacities for the natural soil. Also, the three-span alternative, if chosen, will preclude the use of timber piles due to the presence of cobbles and boulders at the abutment locations.

Therefore, would you please provide design criteria for HP310 pile sections (HP 310 x 79 or HP 310 x 110) at embedment lengths of 20, 25 and 30 metres. To aid driving, the piles will be equipped with reinforced driving tips.

A prompt reply would be greatly appreciated and should you have any questions please contact me.

W.C. Prystanski
Senior Structural Engineer

WCP:gk

MEMORANDUM

TO: Foundation Design Section
3rd Floor, Central Building
Downsview

DATE: September 4, 1991

ATTENTION: **Mr. B. Iyer**

FROM: Structural Section
Northwestern Region
807/473-2063

Re: **W.P. 46-87-00 Aubinadong River Bridge**
Site 38S-380, Hwy. 556
Dist. 18, Sault Ste. Marie

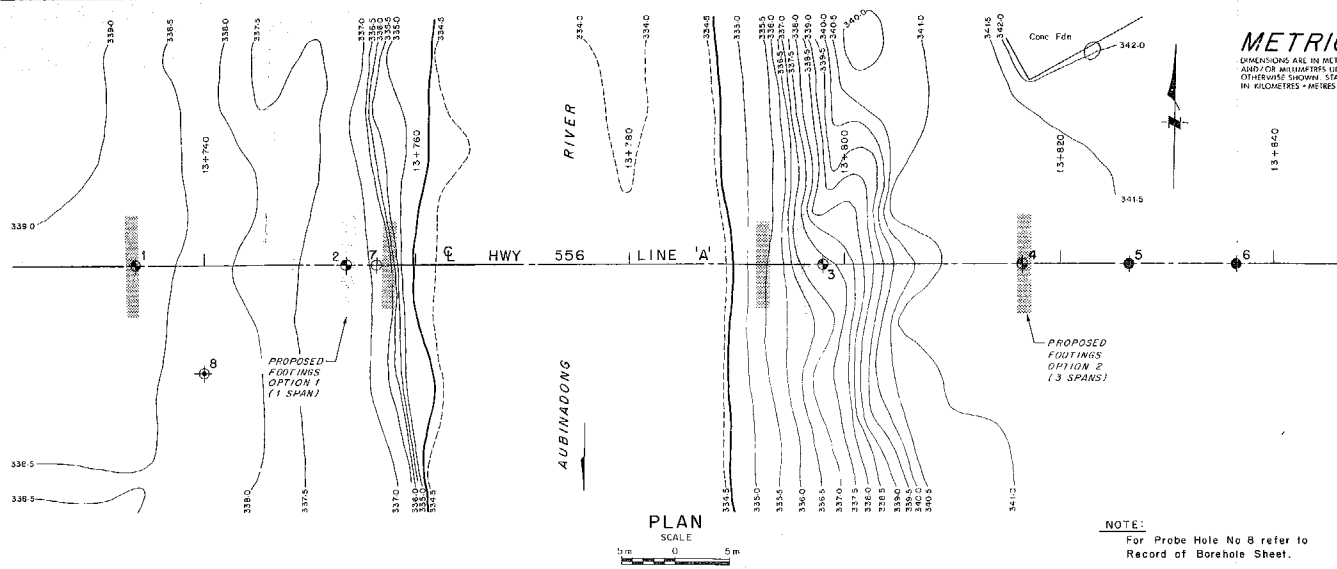
With reference to the timber piles as one of the considered foundation alternatives for the single span option, we are concerned that we may encounter difficult driving conditions with slender timber piles similar to that which has recently occurred at the Echo Bay Bridge (Cont. 91-218). Therefore as an option to the timber piles could you please provide design criteria for steel H-Piles (HP 310 x 79 and HP 310 x 110) for embedded lengths of 15 metres, 20 metres and 25 metres.

Should you have any questions, please contact me.

W.C. Prystanski, P. Eng.
Senior Structural Engineer

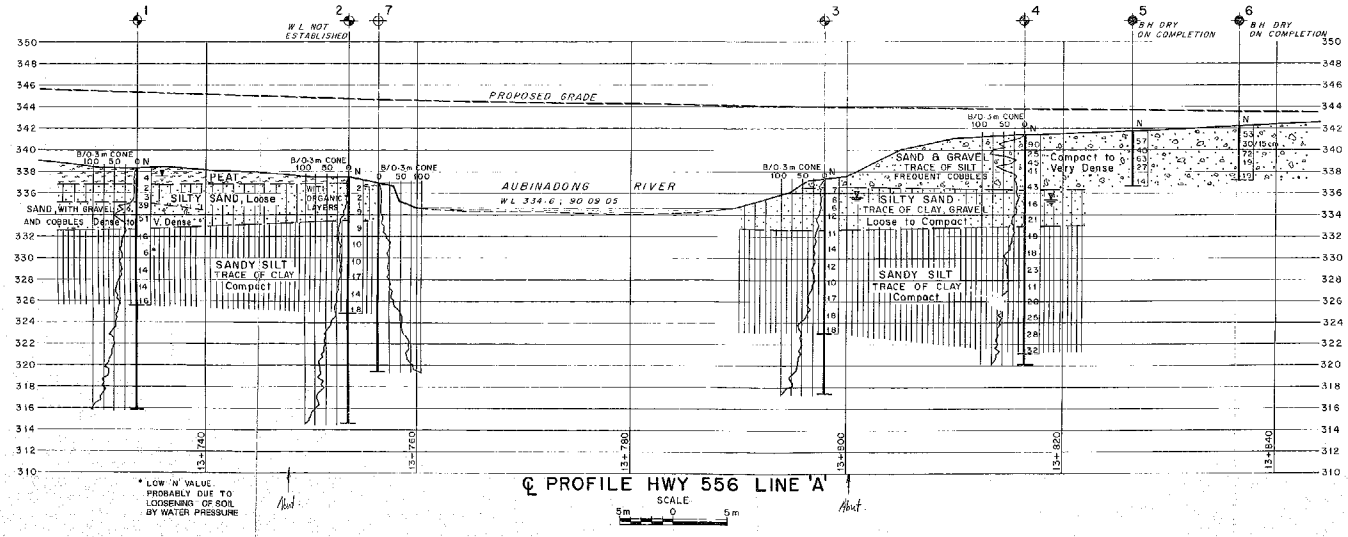
WCP:pi

MINISTRY OF TRANSPORTATION, ONTARIO FH-2-297 8810



PLAN
SCALE
5m 0 5m

NOTE:
For Probe Hole No 8 refer to Record of Borehole Sheet.



REF No E-1503-556-1 9010

METRIC

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

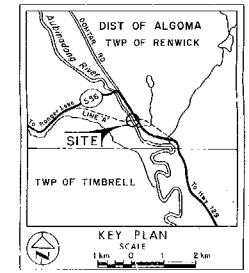
CONT No
WP No 46-87-01

AUBINADONG RIVER

BORE HOLE LOCATIONS & SOIL STRATA

SHEET

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 March and June 1991
- Probe Hole

No	ELEVATION	STATION	OFFSET
1	338·2	13+733·5	℄
2	337·5	13+753·5	℄
3	337·2	13+798·0	℄
4	341·4	13+816·5	℄
5	341·7	13+826·5	℄
6	342·2	13+836·5	℄
7	336·8	13+756·3	℄
8	338·2	13+740·0	10·0m Rt

=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, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION

Geocres No 41J-46A

HWY No 556			DIST 18
SUBMD D.S	CHECKED	DATE Aug 7, 1991	SITE 383-340
DRAWN P.S	CHECKED	APPROVED	DWG 468701-A