G.I.-30 SEPT. 1976

GEOCRES No. 31E-119

DIST. 52 REGION

W.P. No. 207-93-01

CONT. No.

W. O. No.

STR. SITE No. 42-09

HWY. No. 11

LOCATION Hwy 11 & Big East River

No of PAGES - —

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Ministry of Transportation and Communications

Ontario

FILE No.

DATE

REMARKS

Assistance 1 - area code - 555 - 1212

Clearance Appointment Ontario Hydro. ~~July 22/96~~ AUG. 06/96 1:00pm. voice mail not activated

MOBILE : 1-705-788-6870 GARY LANE

~~OFFICE (NANCY) : 1-705-789-4451~~

Clearance Appointment BELL AUG 06/96 @ 1:15 pm CONFIRMED FRI, AUG 02

@ HWY 11 & BIG EAST RIVER BRIDGE

Patrol Yard

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Ministry of
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Ontario

FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP	207-93-01	DIST	52
HWY	11	STR SITE	42-09

Detour Structure
Big East River Bridge

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GEOCRES 31E-119 DATE MAR 26 1997

FOUNDATION INVESTIGATION REPORT
For
Detour Structure
Big East River Bridge
W.P. 207-93-01, Site 42-09
Highway 11, District 52, Huntsville

INTRODUCTION

This report summarizes the results of a field investigation which was carried out for the construction of a detour structure over Big East River. The centreline of the proposed detour will be at an offset of about 18m on the east side from the centreline on Hwy 11.

The investigation was carried out at the request of Northern Region Structural Section. These recommendations apply to proposed detour structure and its approaches within 20m of the structure (Sta 18+180 to 18+265).

SITE DESCRIPTION

The site is located beside Hwy 11where it crosses Big East River, about 5 km North of Hwy 60. The site is in MTO District 52, Huntsville.

The area adjacent to the site (northeast and northwest) is parkland. The surrounding area at the site is undulating and covered with grass, shrubs and small trees. Further east of the proposed detour alignment, there are remains of an old bridge abutment and approaches. The river flows from east to west. At the time of the investigation, the water level in the river was about 285.5m

INVESTIGATION PROCEDURES

The field investigation for this project was conducted between 1996 08 07 and 1996 08 09. The field work for the Foundation Investigation consisted of drilling two boreholes (BH 1 and BH 2). The boreholes were put down at the south and north banks of the river respectively. These boreholes were terminated into the bedrock at depths 21.5m and 20.9m respectively. Bedrock was encountered at depth 19.1m in BH 1 and 19.3m in BH 2.

The boreholes were drilled using a track-mounted auger machine equipped with 82mm ID hollow stem augers and BX size coring equipment.

Soil samples were recovered by means of a 50mm OD Split Spoon sampler driven into the soil according to the specifications of the Standard Penetration Test (ASTM D 1586). Samples were retrieved at intervals ranging from 0.75m to 1.5m. Once practical refusal to auguring was encountered, BX-size bedrock cores were obtained from the boreholes. Groundwater was monitored during drilling and after the completion of the boreholes.

The Laboratory testing program for representative soil samples consisted of:

- Grain Size Analyses
- Natural Moisture Content, and
- Atterberg Limit

The results of the laboratory tests are plotted on the borehole logs. The bedrock core was logged by D.A. Williams, Petrographer of the Soils and Aggregates Section of MTO.

The boreholes were staked out by the Pavements and Foundations Section. Ground surface elevations and locations of the boreholes were interpolated from a plan E-625-11-1, dated 95-08 provided to us by the Northern Region Structural Section.

SUBSURFACE CONDITIONS

The Record of Borehole Sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes, along with stratigraphical profiles based on the borehole data are shown on Drawing No. 2079301-A.

The soil conditions in both boreholes were similar. Both boreholes encountered silty sand to sand as the surficial deposit. The silty sand to sand layer was underlain by a cohesive layer of silt with a trace of clay which was in turn underlain by silt to coarse sand. The silt to coarse sand was overlying the bedrock. The boreholes were terminated into bedrock. The details of the soil condition is shown on the individual borehole logs. Following are the detailed descriptions of the soil strata encountered

Silty Sand to Sand

This non-cohesive material was encountered in both boreholes as the surficial deposit. The thickness of this deposit ranged from 4.4m to 7.2m. The Standard Penetration N-values ranged from 2 to 10 blows/0.3m penetration, that indicated that the deposit is in very loose to compact state. The material was wet below elevation 285.3m to 285.6m (2.9m below ground surface).

Silt

This cohesive deposit was underlying the silty sand to sand deposit. This deposit was mainly silt but contained trace of clay that made it cohesive. The top elevation of this deposit ranged from 281.3 m (BH 1) to 283.8 m (BH 2). The thickness ranged from 6m to 11.8m. The Standard Penetration N-values ranged from 2 blows to 19 blows that suggest that the material is soft to very stiff.

Silt to Coarse Sand

This non cohesive silt to coarse sand deposit was underlying the cohesive silt deposit and was overlying the bedrock. The top elevation of this deposit ranged from 272.0m (BH 2) to 275.3 m (BH 1). The Standard Penetration N-values ranged from 6 to 88 blows/0.3m. A low N-value 0 blows/0.3m was also encountered but was thought to be disturbed and was not representative. The record of N-values indicated that the deposit is in compact to very dense state.

Bedrock

Both boreholes were terminated in to the bedrock. The bedrock was encountered at depths 19.1m (BH 1) and 19.3m (BH 2) respectively. The bedrock surface elevation ranged from 268.9 (BH 2) to 269.4m (BH 1). Bedrock cores were obtained from each locations. The core lengths were 1.6m and 2.4m. The bedrock was a Biotite-Hornblende Gneiss. The recovery of the bedrock ranged from 73% to 100 %. The RQD ranged from 21% to 93%.

Groundwater Condition

Groundwater was monitored in open boreholes. Groundwater in the boreholes was at the same elevation as water level in the river. The groundwater table was encountered in each borehole at a depth of 2.9m. The groundwater table in Borehole 1 was at elevation 285.6m and at 285.3m in Borehole 2. It should be noted that the groundwater is subject to fluctuation and will change as the water level in the river changes.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to replace the superstructure of the Big East River Bridge using a detour. The existing bridge is a 36.6 metre single span supported on concrete filled tube piles. The proposed detour bridge will be on the east side of the bridge with a centreline to centreline offset of 18m. The detour structure will be a two lane Double Wide Acrow bridge having a substructure width of approximately 10.2m. Three alternatives for the detour structure are under consideration, which are:

Alternative 1: three spans 12m, 24m, 12m
Alternative 2: 38m single span
Alternative 3: 42m single span.

Although, it is not known which option will be selected for the detour structure, we understand that the most preferred alternative is a 42m single span bridge. This report therefore, contains the Foundation recommendations for a proposed 42m single span Acrow bridge structure.

Structure Foundations

The proposed profile grade, at the detour crossing, will be at approximate elevation of 291.8m. The approach fills will be approximately 3.0m to 4.0m high.

Based on the subsoil conditions, which is mainly very loose to compact sand, the most cost-effective foundation is spread footing founded on a granular pad. However, other foundation types, such as deep foundations, are also feasible but should be assessed based on cost, construction and environmental considerations.

On the east side of the proposed detour alignment, there are abutments of an old bridge. The old abutments are approximately 35m to 45m from the centreline of Highway 11. There is no information available in our office on the old abutments. However, visually the old abutments appear to be in good condition. The Structural Section archive may have some information on the old bridge foundations. If possible, consideration should be given to using the old abutments for the detour structure.

Spread Footings on Granular Pad

The spread footings for the abutments can be founded on granular pad constructed above the groundwater level (elev. 285.6m). The thickness of the granular pad will depend on the footing elevation but it should be at least 2m thick. The granular pad will extend 1m beyond the plan limits of the abutment footing and will slope at 1H:1V as illustrated in Figure 1. The forward slope will be constructed at 2H:1V from the toe of the existing slope.

The recommended bearing resistance for the footings, on granular pad as per OHBDC are given below. The SLS values are given for 25mm and 50mm settlements.

Factored Bearing Resistance at ULS =	900 kPa
Bearing Resistance at SLS for 25mm=	175 kPa
Bearing Resistance at SLS for 50mm=	350 kPa

Deep Foundation

Alternatively, if higher bearing resistance is required, then the structure can be supported on steel H-piles driven to bedrock [bedrock depth 19.1m (BH 1) and 19.3m (BH 2)]. However, this alternative should be assessed based on cost comparison. The recommended resistance of H-piles founded on the bedrock are as follow:

	HP 310X110	HP 310X79
Factored Axial Resistance at ULS	1600 kN/pile	1150 kN/pile
Axial Resistance at SLS for 25mm	1150 kN/pile	825 kN/pile
Factored Horizontal Resistance at ULS	80 kN/pile	60 kN/pile
Horizontal Resistance at S.L.S.	60 kN/pile	40 kN/pile

In order to facilitate pile driving, particle sizes of any fill placed beneath the pile locations should be restricted to 75mm.

Embankment Stability

The height of the embankment will be approximately 3m to 4m. Prior to placing fill, all surficial topsoil or any organic material should be removed within the plan limits of the embankments. The embankment should be then constructed with rockfill or native soil. The rockfill embankment can be constructed at 1.25H:1V.

If native soil is used then permanent slopes should be maintained at 2H:1V. All slopes should be protected against surficial erosion e.g. by establishing vegetation cover. Slopes at abutments should be armoured with 600mm thick rock protection to prevent erosion. Such rock protection should extend horizontally 10m on each side of the abutments and vertically from the high water level to the base of the embankment and 2m along the river bottom.

There are no long term settlement concerns for the embankments. Settlement will be elastic in nature and should occur during construction.

No stability problems are anticipated for the proposed height of the permanent embankments.

Lateral Earth Pressure

If abutments are constructed, free draining granular material such as Granular 'A' or 'B', or rockfill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure build-up.

If rockfill is used for approaches, special care will be required to avoid damaging the abutment. It would be preferable to place a 0.3m cushion of Granular 'A' or smaller rockfill (with diameter of less than 300mm), between the structure and the main mass of rock fill. Granular material may also be used at the approaches.

For design purposes, the following properties for backfill are recommended:

Granular 'A'	$\Upsilon = 22.8$ kN/m^3	$\phi = 35°$
Granular 'B'	$\Upsilon = 21.2$ kN/m^3	$\phi = 30°$
Rockfill	$\Upsilon = 18.0$ kN/m^3	$\phi = 35°$

Active condition (K_a) may be assumed to apply for yielding structure.

Resistance to Horizontal Forces

For footings placed on compacted Granular 'A' pad, the sliding resistance between the concrete footing and Granular 'A' pad should be computed as per OHBDC 91. For abutments on piles lateral capacity may be supplemented by the horizontal component of battered piles.

Frost Protection

A soil cover of 1.8m or equivalent will be required for frost cover for footings or pile caps.

Dewatering

Since there will be no excavation below water table, no major dewatering will be required.

Miscellaneous

The field work for this project was carried out under the supervision of Lizette Viera, an engineering student. The equipment used was owned and operated by Mater Soil Investigation Ltd. This report was written by K. Ahmad, P. Eng. and reviewed and approved by T.C. Kim, P. Eng., Senior Foundation Engineer.

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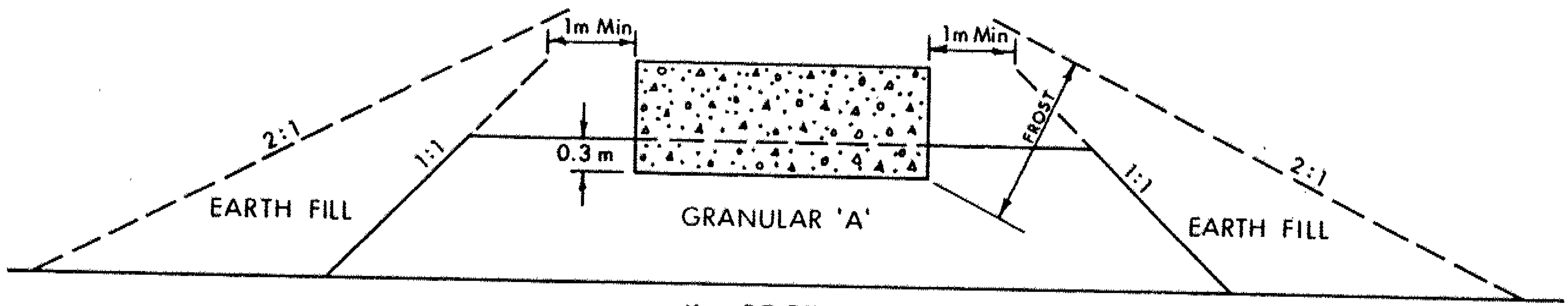
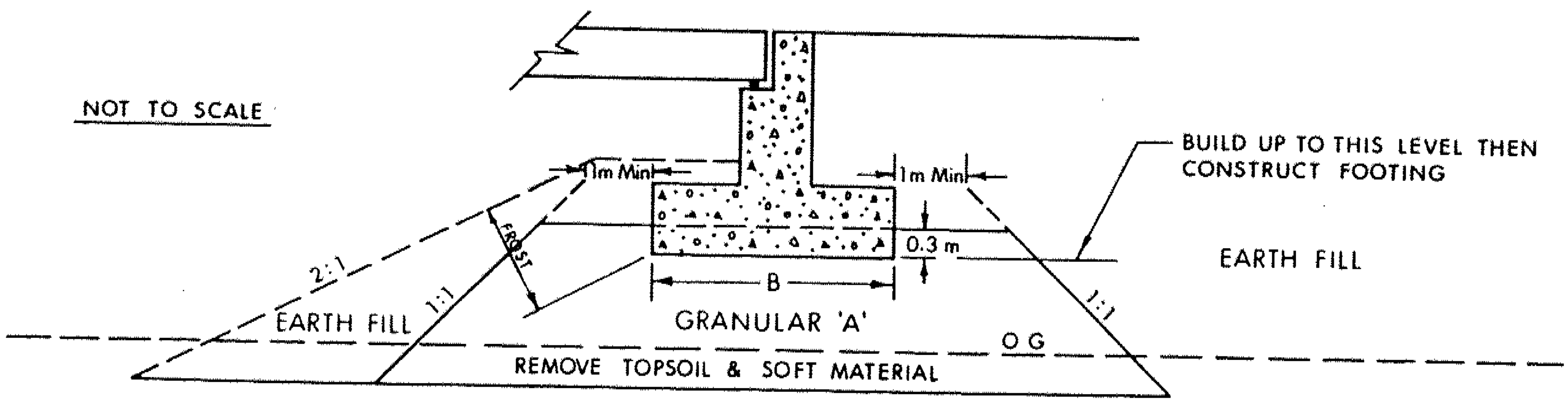
APPENDIX



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 BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.

3 - CONSTRUCT CONCRETE FOOTING.

4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

Ministry of Transportation Ontario	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE	FIG No 1 W P 207 - 93 - 01

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 1

METRIC

W.P. 207-93-01 LOCATION Co-ords.: N 5 026 632.4; E 326 556.4 ORIGINATED BY LV

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Auger, BX Core, Cone Test COMPILED BY KA

DATUM Geodetic DATE 1996 08 08,09 CHECKED BY TC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
288.5	Ground Surface					
0.0	Silty Sand to Sand, Very Loose to Loose, Brown, Moist to Wet	1	SS	2		0 84 14 2
		2	SS	4		
		3	SS	6	▼	0 90 (10)
		4	SS	7		
		5	SS	5		
		6	SS	2		0 98 (2)
		7	SS	3		
		8	SS	2		
281.3 / 7.2	Silt, with a trace of Clay, Soft to Stiff, Grey, Wet	9	SS	2		
		10	SS	10		
		11	SS	11		
		12	SS	11		
275.3 / 13.2	Silt to Coarse Sand, trace Gravel, Loose to Very Dense, Grey, Wet	13	SS	6		0 11 85 4
		14	SS	54		
		15	SS	88		
		16	RC	REC	100%	RQD %
269.4 / 19.1	Biotite-Hornblende Gneiss Bedrock	17	RC	REC	93%	RQD 21%
		18	RC	REC	73%	RQD 24%
		19	RC	REC	100%	RQD 76%
		20	RC	REC	100%	RQD 46%
267.0 / 21.5	End of Borehole					

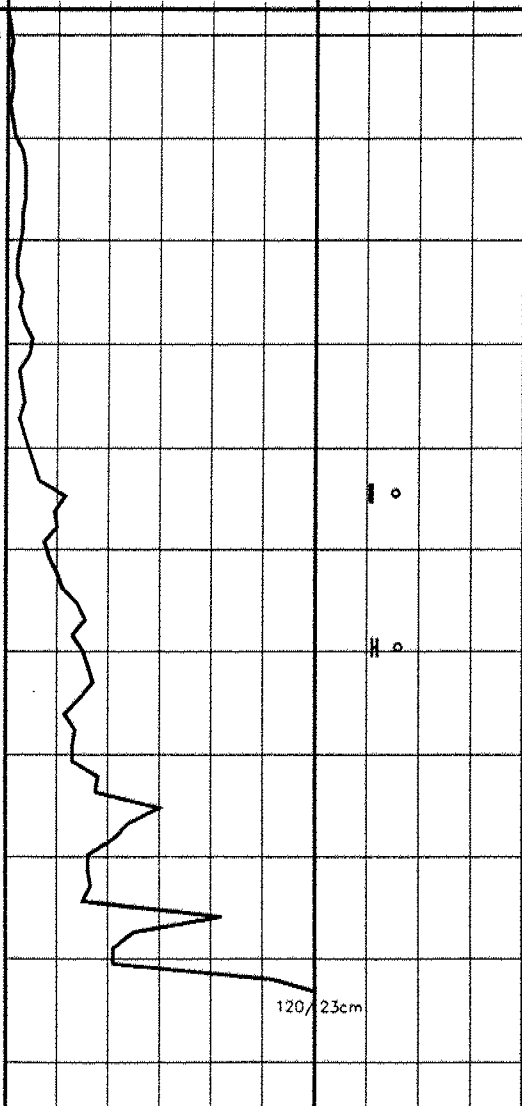
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

ELEVATION SCALE: 288, 286, 284, 282, 280, 278, 276, 274, 272, 270, 268



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

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 2

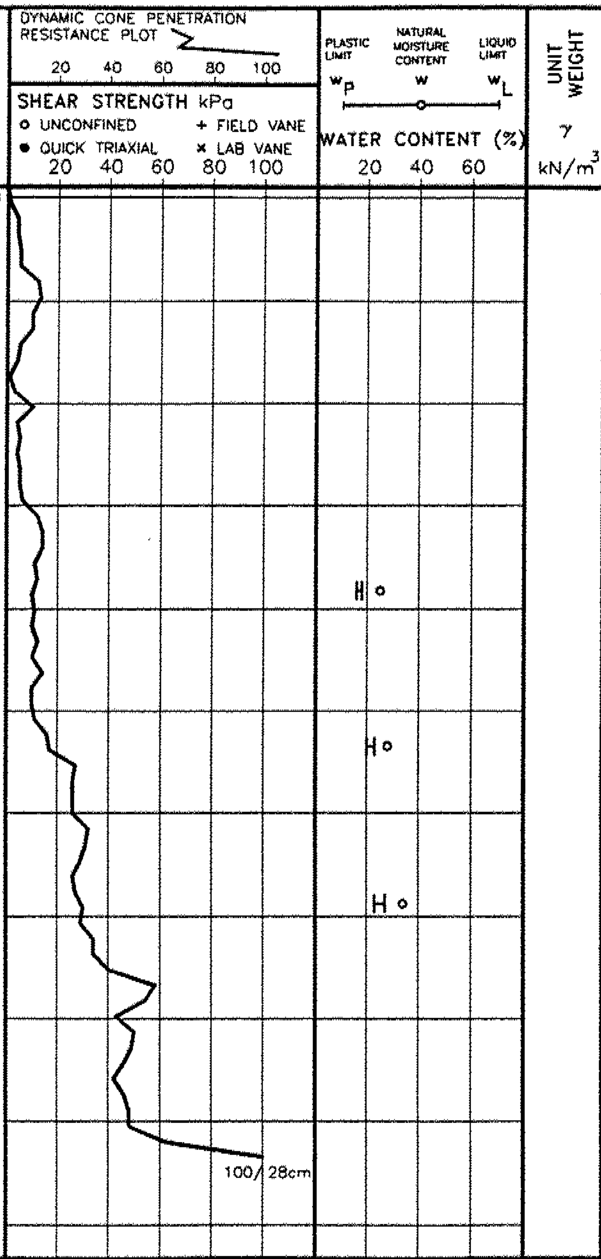
METRIC

W.P. 207-93-01 LOCATION Co-ords.: N 5 026 673.8; E 326 535.0 ORIGINATED BY LV

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Auger, BX Core, Cone Test COMPILED BY KA

DATUM Geodetic DATE 1996 08 07 CHECKED BY TC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
288.2	Ground Surface					288	
0.0	Silty Sand to Sand Very Loose to Compact Brown, Moist to Wet	1	SS	5			
		2	SS	10		286	0 82 16 2
		3	SS	9	▼		
		4	SS	3			0 95 (5)
283.8		5	SS	2		284	
4.4	Silt with a trace of Clay Soft to Very Stiff Grey, Wet	6	SS	2			
		7	SS	2			0 2 95 3
		8	SS	5		282	
		9	SS	2		280	
		10	SS	3		278	
		11	SS	12			
		12	SS	11		276	
		13	SS	12		274	
272.0		14	SS	19		272	
16.2	Silt, some Sand, Tr. Clay Trace Gravel Compact, Grey, Wet	15	SS	0			0 8 86 6
268.9		16	SS	16		270	
19.3	Biotite-Hornblende Gneiss Bedrock	17	RC	REC	96%	268	RQD 93%
267.3							
20.9	End of Borehole						



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

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

ROCK CORE DESCRIPTION
WP 207-93-01

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	17	19.03-19.76	93	21	19.03-21.46	**BIOTITE-HORNBLENDE GNEISS**, greyish black to light grey to moderate orange pink; medium to coarse grained; strong; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
	18	19.76-20.70	73	24		
	19	20.70-21.13	100	76		
	20	21.13-21.46	100	46		
2	17	19.33-20.85	96	93	19.33-20.85	**BIOTITE-HORNBLENDE GNEISS**, greyish black to light greenish grey; medium to coarse grained; strong; unweathered to slightly weathered; fractures wide to close spaced, flat to dipping, undulating to planar, smooth to rough.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

Note: Depths are approximated where core recovery is less than 100%
Logged by: DAW, Soils and Aggregates Section

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/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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

METRIC

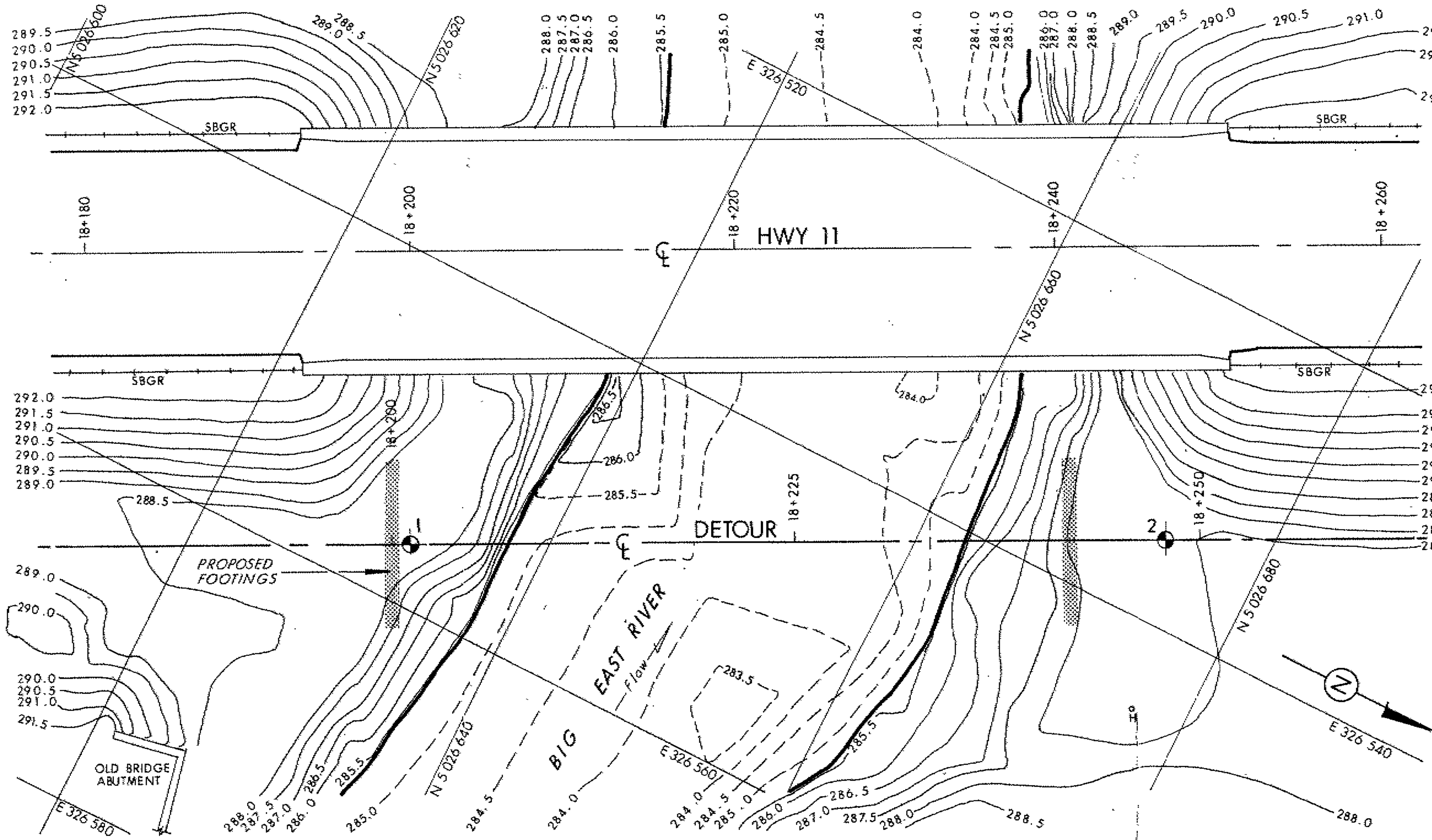
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 207-93-01

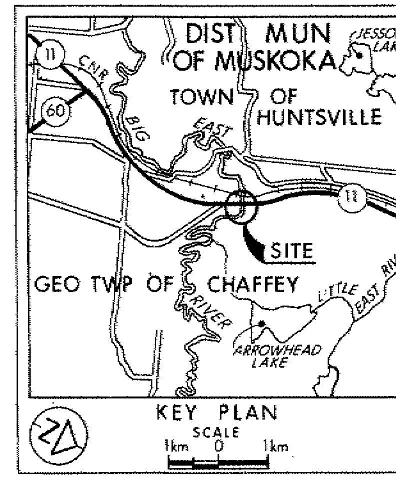
SHEET

BIG EAST RIVER

BORE HOLE LOCATIONS & SOIL STRATA



PLAN

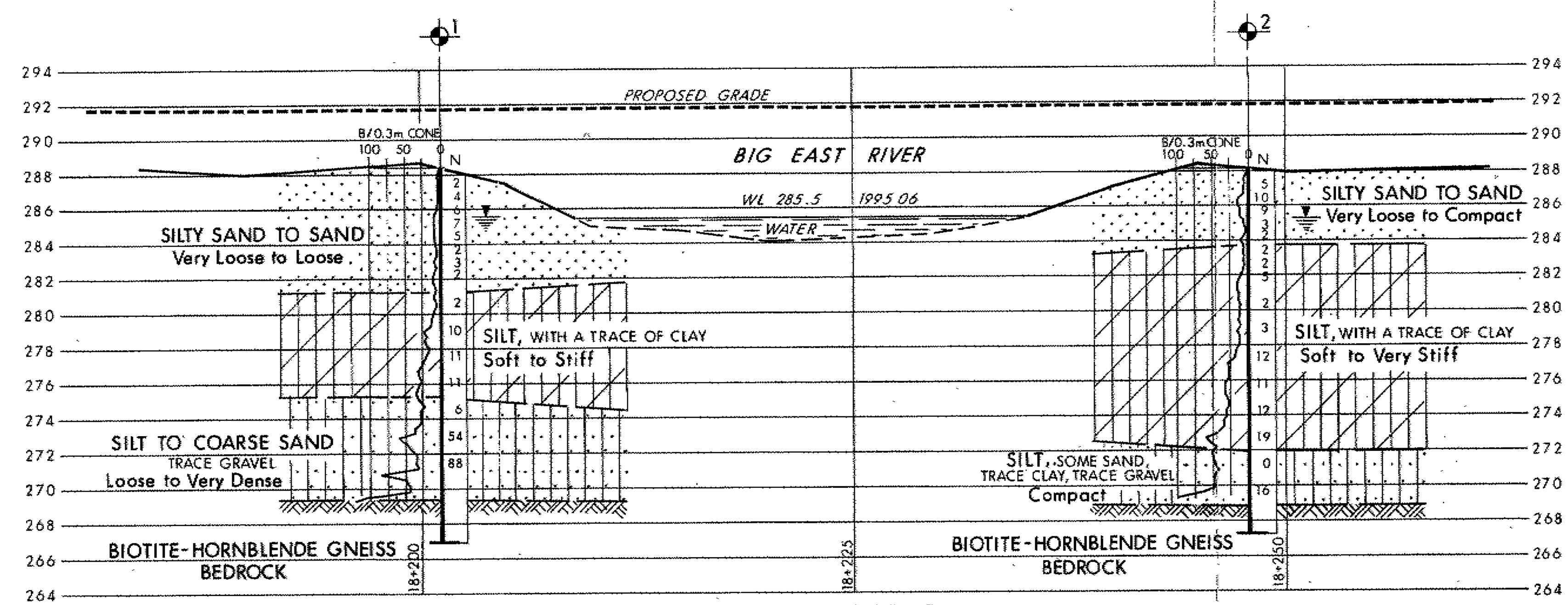


KEY PLAN

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 1996 08

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	288.5	5 026 632.4	326 556.4
2	288.2	5 026 673.8	326 535.0



℄ PROFILE DETOUR

=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 GC 2.01 of OPS Gen.Cond.

LICENSED PROFESSIONAL ENGINEER S. Q. AHMAD PROVINCE OF ONTARIO

REV	DATE	BY	DESCRIPTION

Geocres No 31E-119

HWY No 11		DIST 52
SUBM'D KA	CHECKED SQA DATE 1996 11 22	SITE 42-09
DRAWN RS	CHECKED APPROVED	DWG 2079301-A

REF No E-625-11-01, Sheet 1 of 2, 1995 08

Ontario

memorandum

To: P. Furst, P. Eng.
Head, Structural Section
Northern Region
North Bay, Ontario P1B 8L2

1996 12 12

Attn: Salah Ismail, P. Eng.
Structural Engineer

From: Pavement & Foundation Design Section
Room 315, Central Building, Downsview, Ontario

Re: Foundation Recommendations
W.P. 207-93-01, Detour For Big East River Bridge
Site 42-09, Highway 11
District 52, Huntsville

The field investigation for the above project has been completed. The fieldwork was carried out for the proposed detour structure and approach embankments.

This memorandum outlines the preliminary foundation recommendations that should provide sufficient information for you to carry out the structural design. Although, we do not anticipate any major changes in the final recommendations, there may be minor changes in the final report due to further analyses. The final report will be provided before the due date of April 1, 1997.

Introduction

This report summarizes the results of a field investigation which was carried out for the construction of a detour structure over Big East River, on the east side of existing Hwy 11.

The investigation was carried out at the request of Northern Region Structural Section. These recommendations apply to proposed detour structure and its approaches within 20m of the structure.

Two boreholes (BH 1 and BH 2) were drilled for the Foundation Investigation. The boreholes were put down at the south and north banks of the river respectively. These boreholes were terminated in bedrock at depths 21.5m and 20.9m respectively. Bedrock was encountered at depth 19.1m at BH 1 and 19.3m at BH 2 locations.

Subsurface Conditions

The soil conditions in both boreholes were similar. Both boreholes encountered silty sand to sand as the surficial deposit. The silty sand to sand layer was underlain by a cohesive layer of silt with a trace of clay which was in turn underlain by silt to coarse sand. The silt to coarse sand was overlying the bedrock. The boreholes were terminated into bedrock. The details of the soil condition is shown on the individual borehole logs. However, brief summary of each soil layer is given below:

Silty Sand to Sand

This non-cohesive material was encountered in both boreholes as the surficial deposit. The thickness of this deposit ranged from 4.4m to 7.2m. The Standard Penetration N-values ranged from 2 to 10 blows/0.3m penetration, that indicated that the deposit is in very loose to compact state. The material was wet below elevation 285.3m to 285.6m (2.9m below ground surface).

Silt, with clay

This cohesive deposit was underlying the silty sand to sand deposit. The top elevation of this deposit ranged from 281.3 m (BH 1) to 283.8 m (BH 2). The thickness ranged from 6m to 11.8m. The Standard Penetration N-values ranged from 2 blows to 19 blows that suggest that the material is soft to very stiff.

Silt to Coarse Sand

This non cohesive silt to coarse sand deposit was underlying the cohesive silt deposit and was overlying the bedrock. The top elevation of this deposit ranged from 272.0m (BH 2) to 275.3 m (BH 1). The Standard Penetration N-values ranged from 6 to 88 blows/0.3m. A low N-value 0 blows/0.3m was also encountered but was thought to be disturbed and was not representative. The record of N-values indicated that the deposit is in compact to very dense state.

Bedrock

Both boreholes were terminated in to the bedrock. The bedrock was encountered at depths 19.1m and 19.3m respectively. The bedrock was proved by coring 2.4m and 1.6m into the bedrock. The bedrock was a Biotite-Hornblende Gneiss Bedrock. The recovery of the bedrock ranged from 73% to 100 %. The RQD ranged from 21% to 93%.

Groundwater Condition

Groundwater was monitored in open boreholes. Groundwater in the boreholes was at the same elevation as water level in the river. The groundwater table was encountered in each borehole at a depth of 2.9m. The groundwater table in Borehole 1 was at elevation 285.6m and at 285.3m in Borehole 2. It should be noted that the groundwater is subject to fluctuation and will change as the water level in the river changes.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to replace the superstructure of the Big East River Bridge using a detour. The existing bridge structure is a 36.6 metre single span bridge supported on concrete filled tube piles. The proposed detour will be on the east side of the bridge with a centreline to centreline offset of 18m. The structure will be a two lane Double Wide Acrow bridge having a substructure width of approximately 10.2m. Three alternatives for the detour structure is under consideration. Alternative 1 will have three spans 12m, 24m, 12m, Alternative 2 will have 38m single span and Alternative 3 will have 42 m single span. Although, it is not yet decided which option will be selected for the detour structure, the most preferred alternative is a single span bridge.

There are abutments of an old bridge on the east side of the highway. The old abutments are approximately 35m to 45m from the centreline of the highway. Although, we could not find any information on the old abutments, visually it appears to be in good condition. May be the structural section archive have some information on the old bridge foundations. If possible, consideration should be given to using the old abutments for the detour structure.

Structure Foundations

The proposed profile grade, at the detour crossing, will be at approximate elevation of 291.8m. The approach fills will be approximately 3.0m to 4.0m high.

Based on the subsoil conditions, which is mainly very loose to compact sand, the most suitable structure from a cost point of view appears to be spread footing founded on granular pad. However, alternatives should be assessed based on cost, as well as construction and environmental considerations.

Spread Footings on Granular Pad

The spread footing for the abutments could be founded on granular pad built up above the groundwater level (Elev. 285.6m). The thickness of the granular pad will be dictated by the required footing elevation but at least it should be 2m thick. The granular pad will extend 1m beyond the plan limits of the abutment footing and will slope at 1H:1V. The forward slope will be constructed at 2H:1V from the toe of the existing slope.

The recommended bearing capacities for the footings, on granular pad as per OHBDC are given below. The SLS values are given for 25mm and 50mm settlements.

Factored Bearing Capacity at ULS = 900 kPa
Bearing Capacity at SLS for 25mm= 175 kPa
Bearing Capacity at SLS for 50mm= 350 kPa

Deep Foundation

Alternatively, if higher bearing capacity is required then the structure can be supported on steel H-piles driven to bedrock [bedrock depth 19.1m (BH 1) and 19.3m (BH 2)]. However, this alternative should be assessed based on cost comparison. The recommended bearing capacities of H-piles founded on bedrock are as follow:

	HP 310X110	HP 310X79
Factored Axial Capacity @ ULS	1600 kN/pile	1150 kN/pile
Axial Capacity @ SLS for 25mm	1150 kN/pile	825 kN/pile
Factored Lateral Capacity at ULS	80 kN/pile	60 kN/pile
Lateral Capacity at S.L.S.	60 kN/pile	40 kN/pile

In order to facilitate pile driving, particle sizes of any fill placed beneath the pile locations should be restricted to 75mm.

Embankment Stability

The height of the embankment will be approximately 3m to 4m. Prior to placement of fill, all surficial topsoil or any organic material should be removed within the plan limits of the embankments. The embankment should be then constructed with rockfill or native sand. The rockfill embankment can be constructed at 1.5H:1V.

Sand is also suitable as fill material for embankment construction. If sand is used, the side slope of the embankment should be constructed at 2.5H:1V or flatter. If sand fill is used, the lower slopes must be protected from erosion which can consist of vegetation, or 600mm rock protection to the potential high water level.

There are no settlement concerns for the embankment constructed with the above mentioned materials. Settlement will be elastic in nature and should occur during construction.

No stability problems are anticipated for the proposed height of permanent embankments.

Lateral Earth Pressure

If abutments are constructed, free draining granular material such as Granular 'A' or 'B', or rockfill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure build-up.

If rockfill is used for approaches, special care will be required to avoid damaging the abutment. It would be preferable to place a 0.3m cushion of Granular 'A' or smaller rockfill (with diameter of less than 300mm), between the structure and the main mass of rock fill. Granular material may also be used at the approaches.

For design purposes, the following properties for backfill are recommended:

Granular 'A'	$\Upsilon = 22.8$ kN/m^3	$\phi = 35°$
Granular 'B'	$\Upsilon = 21.2$ kN/m^3	$\phi = 30°$
Rockfill	$\Upsilon = 18.0$ kN/m^3	$\phi = 35°$

Active condition (K_a) may be assumed to apply for yielding structure.

Resistance to Lateral Forces

For footings placed on compacted Granular 'A' pad, the sliding resistance between the concrete footing and Granular 'A' pad should be computed as per OHBDC 91. For abutments on piles lateral capacity may be supplemented by the horizontal component of battered piles.

Frost Protection

A soil cover of 1.8m or equivalent will be required for frost cover for footings or pile caps.

Dewatering

Since there will be no excavtion below water table, no major dewatering will be involved in this project, no major dewatering will be required.

Miscellaneous

The field work for this project was carried out under the supervision of Lizette Viera, an engineering student. The equipment used was owned and operated by Mater Soil Investigation Ltd. This report was written by K. Ahmad, P. Eng. and reviewed and approved by T.C. Kim, P. Eng., Senior Foundtion Engineer.

S.Q. (Ken) Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

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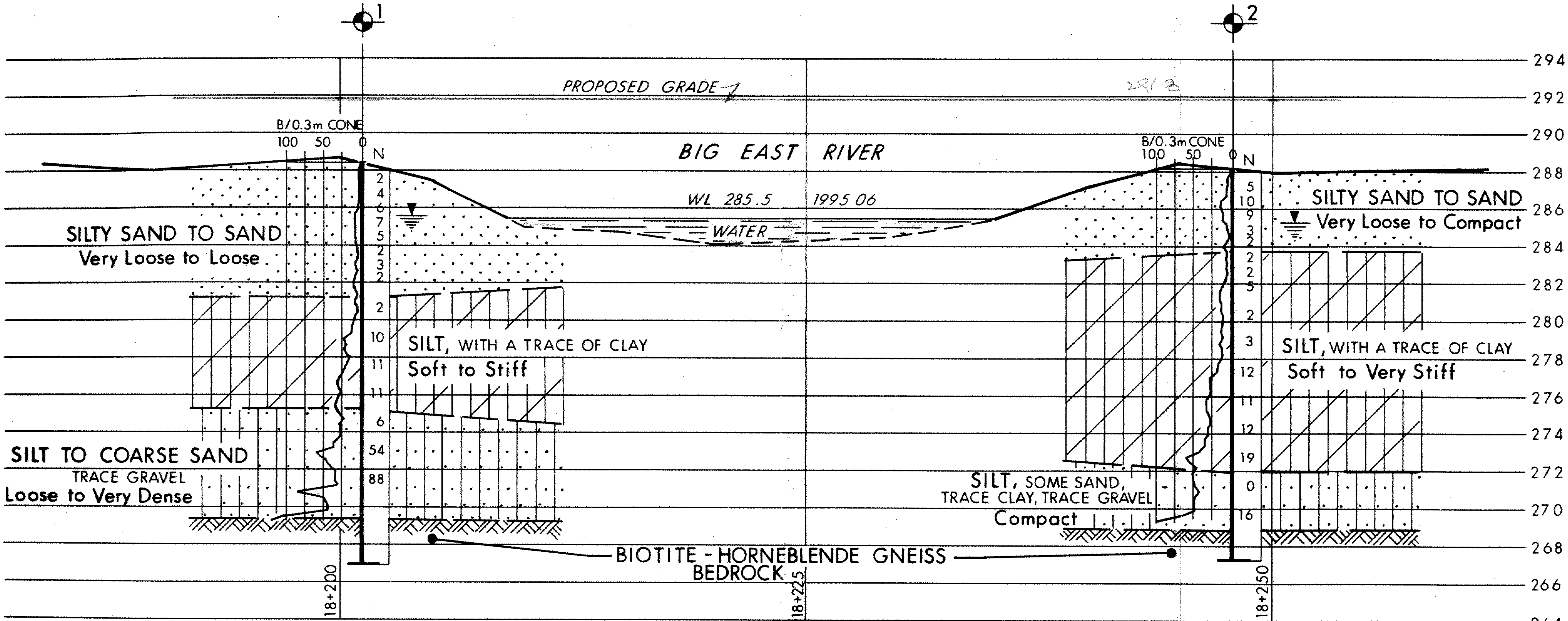

℄ PROFILE DETOUR

SCALE

4 m 0 4 m

REF No E-625-11-01, Sheet

WP 207-93-01