

GEOCRETS No. 31E-124DIST. 11 REGION W.P. No. CONT. No. W. O. No. 92-11010STR. SITE No. 44-57HWY. No. Min.LOCATION Mackenzie local Roads
Board, South Maple IslandNo of PAGES - — Curvet

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

STABILIZATION MEASURES FOR EXISTING COFFERDAM

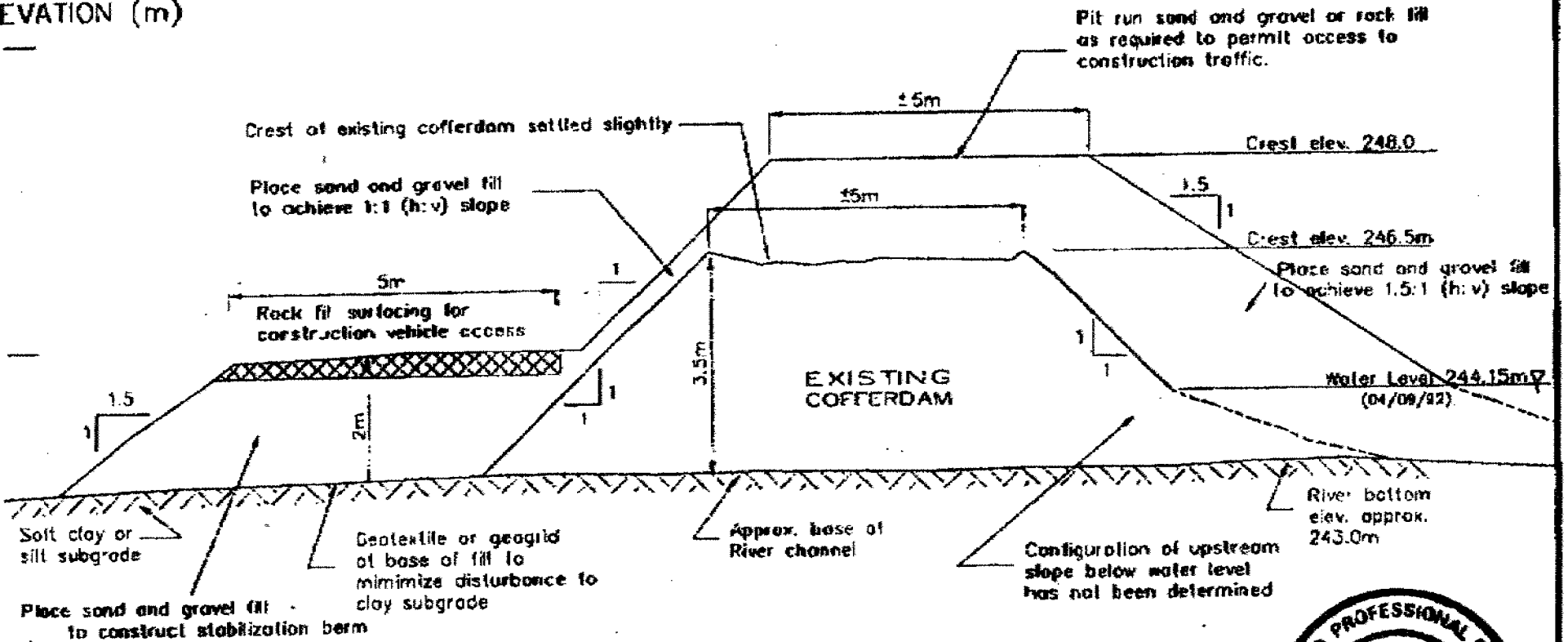
ELEVATION (m)

50 —

45 —

40 —

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MAPLE ISLAND BRIDGE REPLACEMENT

OUR FILE NO. 92217

TERRAPROBE Sep. 10, 1992



FIGURE 1

STABILITY ANALYSIS OF PROPOSED COFFERDAM

SOIL PARAMETERS						
LAYER	UNDRAINED			DRAINED		
	γ	ϕ	C_u	γ	ϕ	C
① SAND AND GRAVEL FILL	18 kN/m ³	30 degree	0 kPa	18 kN/m ³	30 degree	0 kPa
② SOFT CLAYEY SILT	19	0	6	19	25	0
③ FIRM CLAYEY SILT	19	0	30	19	25	0
④ DENSE GLACIAL SILT	21	35	0	21	35	0

RESULTS OF ANALYSIS		
FAILURE SURFACE	UNDRAINED	DRAINED
A	1.4	1.7
B	1.1	1.4
C	1.2	1.5
D	1.2	1.7

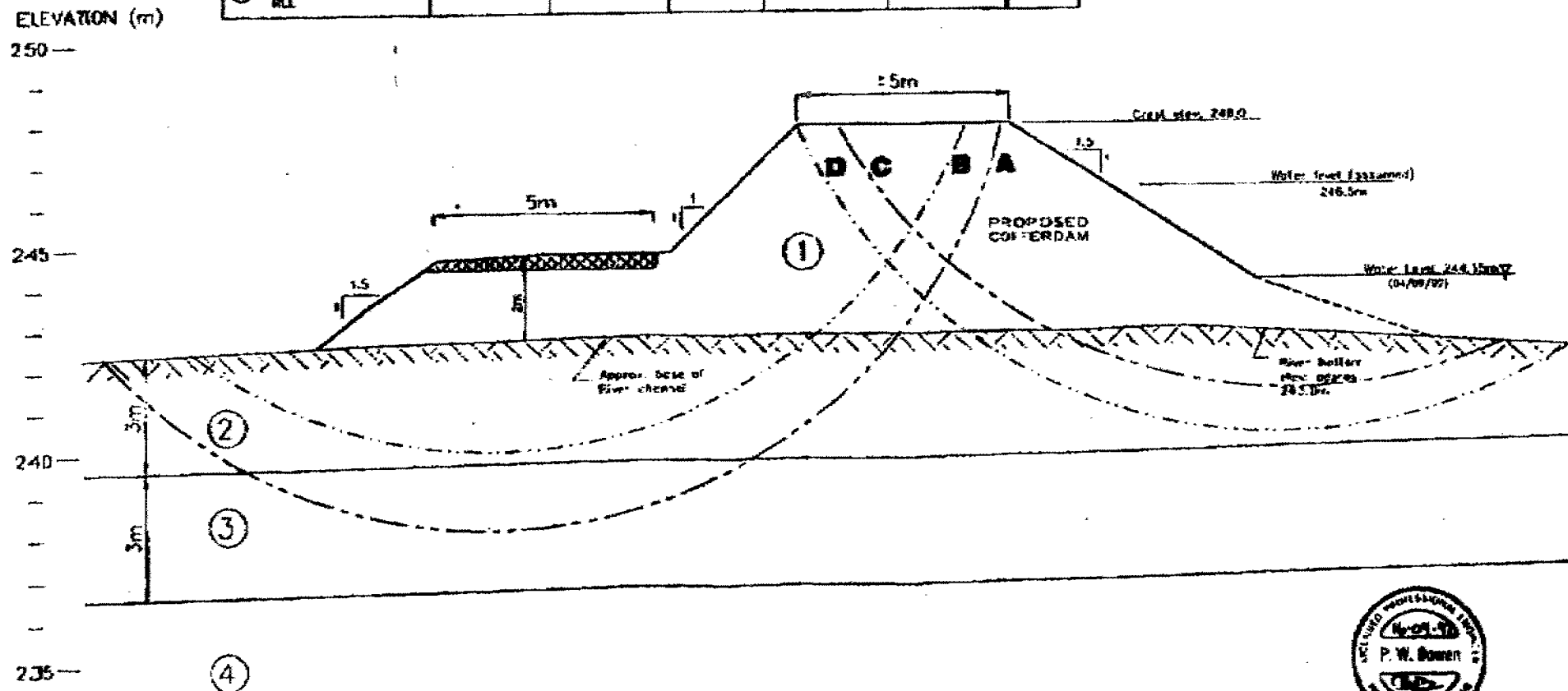
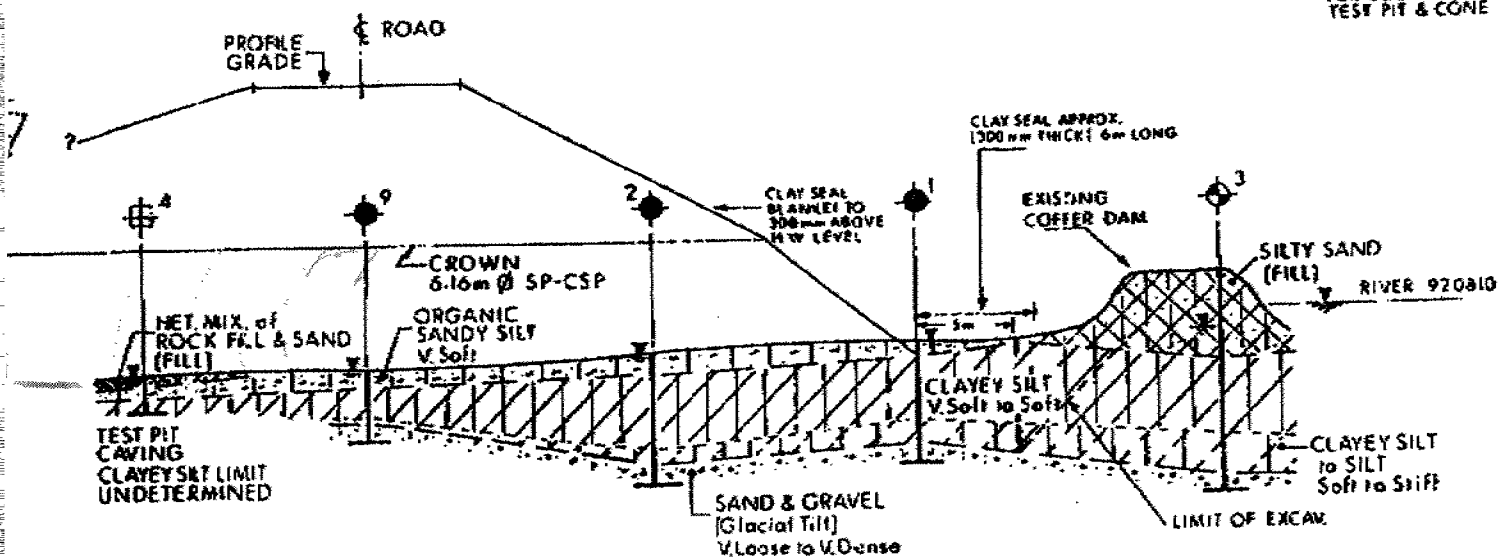


FIGURE 2

SEE ACCOMPANYING PLAN
FOR LOCATION OF BOREHOLES
TEST PIT & CONE TESTS.

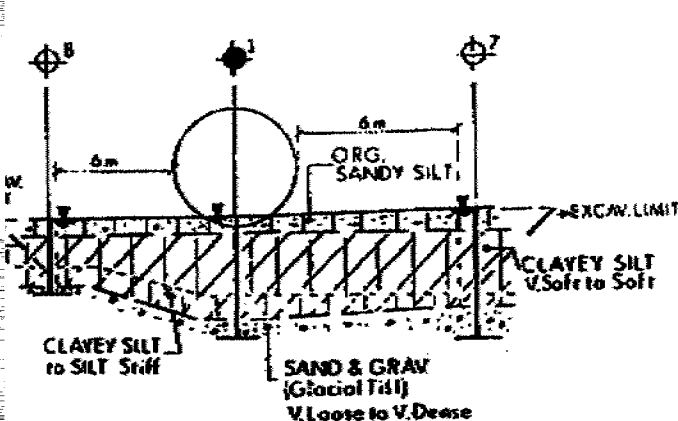


RECOMMENDATION:
Replace Excavated Material With Approved Shot
Blast Brn. Equivalent To Loose Shale, Down
To Sand & Gravel (Glacial Till)

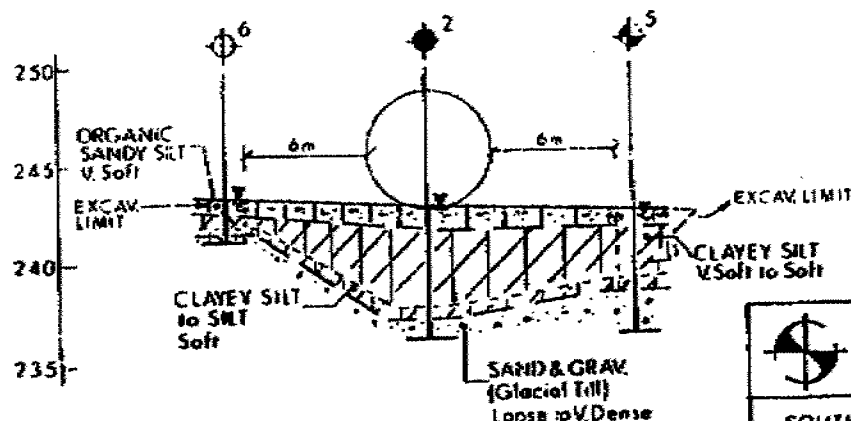
LEGEND

- DYNAMIC CONE PENETRATION TEST
- BOREHOLE
- BOREHOLE & CONE
- TEST PIT
- WATER LEVEL

PROFILE OF CULVERT



SECTION A-A



SECTION B-B



NOTE: STRATIGRAPHY B/W BOREHOLES
IS INTERPOLATED & NO GUARAN-
TEES ARE IMPLIED AS TO CONTI-
NUITY B/W OR BEYOND BOREH-
OLES.

STRATA ENGINEERING CORP.		
SOUTH MAPLE ISLAND SP-CSP FOUNDATION CONDITIONS		
DRAWN: AK	SCALE: AS SHOWN	DATE: 92 08 21
APPROVED:		SKETCH No. N90031A-SK-2

AUG 26 '92 08:59

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326 P10



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[illegible]



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RECORD OF BOREHOLE No 3

METRIC

PROJECT N-90-031A LOCATION South Maple Island Bridge ORIGINATED BY CN
 CLIENT McKenzie Local Rd BOREHOLE TYPE Solid Stem Auger/Cone Test COMPILED BY AK
 DATUM Concrete DATE 1992 08 10 CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			30	40	60	80	100					
246.4	Ground Surface																GR SA SI CL
0.0	Fill:																
	Silty Sand						246										
							245										
							244										
							243										
							242										
241.8	Brown						241										
4.6	Clayey Silt						240										
	Soft		1	SS	1		239										
							238										
	Gray Clayey Silt to Silt Firm		2	SS	12		237										
236.0	Gray						236										
10.4	Sand and Gravel Dense (Glacial Till)		3	SS	33												
235.4	Gray																
11.0	End of Borehole and Cone Test																

W.L. on
92 08 10

100/3 cm

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327 P04



STRATA ENGINEERING CORP.

[illegible]

327 P05



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[illegible]



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RECORD OF BOREHOLE No 7

METRIC

PROJECT N-40-031A

LOCATION South Maple Island Bridge

ORIGINATED BY GN

CLIENT McKenzie Local

BOREHOLE TYPE CONC PENETRATION TEST

COMPILED BY AK

DATUM 060151C

DATE 1992 08 11

CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS 8 GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	W _p	W	W _L		
243.3 0.0	Riverbed Surface Probable Organic Sandy Silt						243						H.L. ON 92 08 11
242.1 1.2	Probable Clayey Silt						242						
							241						
							240						
	Probable Clayey Silt to Silt						239						
238.4 4.9	Probable Sand and Gravel (Glacial Till)						238						
236.9 6.4	End of Cone Test						237						

326 P13



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RECORD OF BOREHOLE No 8

METRIC

PROJECT N-90-031A LOCATION South Maple Island Bridge ORIGINATED BY GN
CLIENT McKenzie Local MB BOREHOLE TYPE Cone Penetration Test COMPILED BY AK
DATUM Geoidetic DATE 1992 08 11 CHECKED BY CM

[illegible]

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326 P14



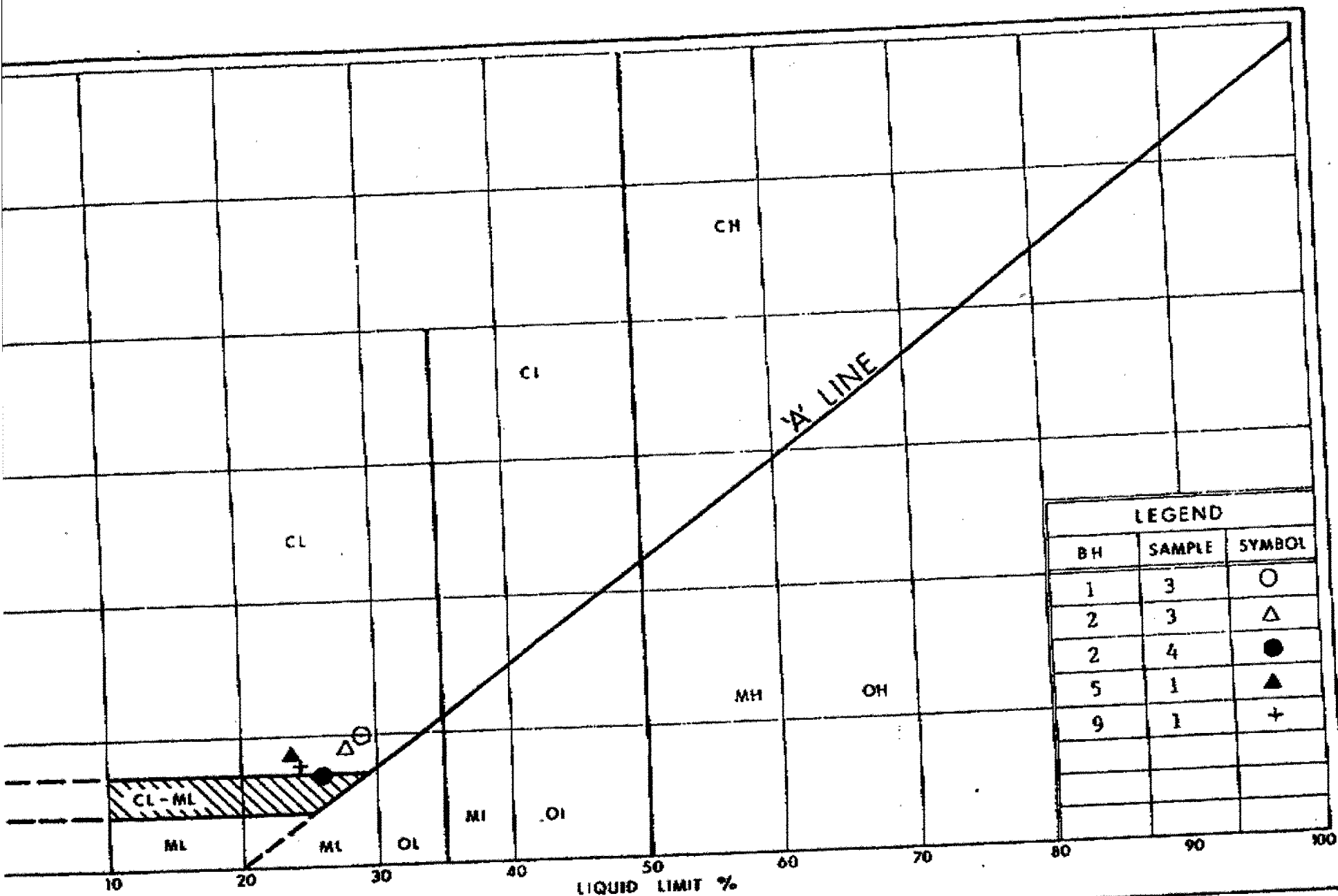
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AUG 26 '92 09:03

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326 P15



LEGEND		
BH	SAMPLE	SYMBOL
1	3	○
2	3	△
2	4	●
5	1	▲
9	1	+

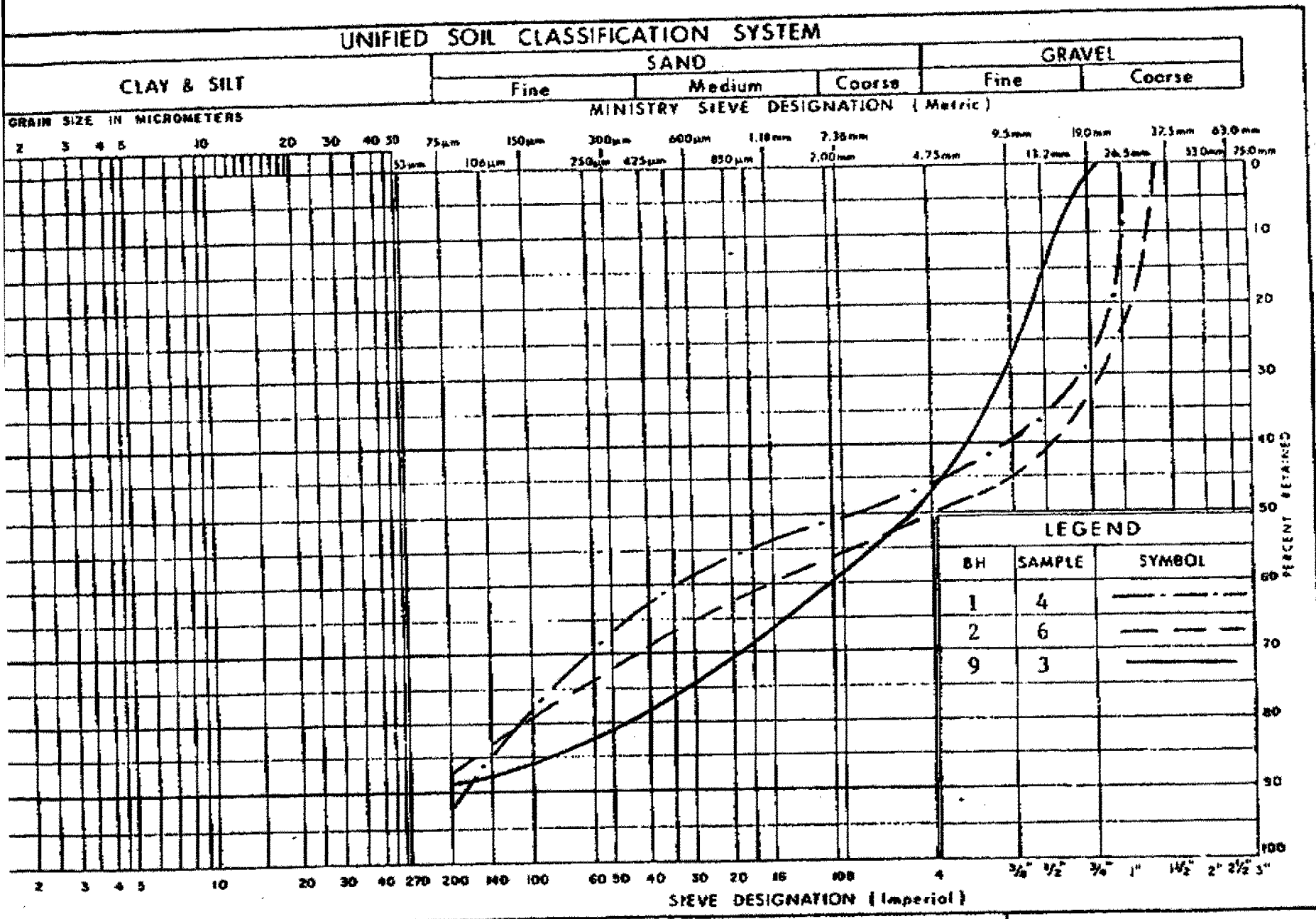
PLASTICITY CHART
Clayey Silt to Silt

FIG No 1
Project#: N-90-031A
South Maple Island

AUG 26 '92 09:03

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326 P16



	Gran. "A"	Gran. "B"
Angle of internal friction, ϕ'	35.0°	30.0°
Unit Weight (kN/m^3), γ	22.8	21.2

Surcharge effects, if any should be computed as per Clause 6-6.1.2.4 of OHBD Code.

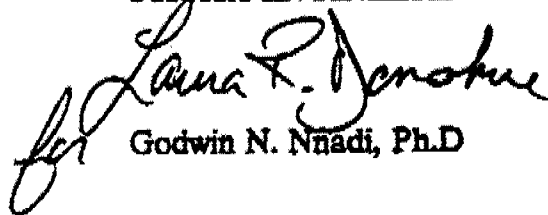
4. Dewatering may be accomplished by means of cofferdams, and to prevent ingress of water into the excavation by sheet piling cutoff driven a minimum of 1.0m below the river bed. Alternatively, interlocking steel sheet piling driven a minimum of 1.0m below the river bed could be used.

Dewatering for the concrete retaining wall requires a final sheeting penetration of a minimum of 1.0m below river bed.

5. Attached is the log sheet of borehole No. 7.

We trust this is the information you require but if we can be of further assistance, please contact us.

Yours very truly,
STRATA ENGINEERING CORP.


Godwin N. Nnadi, Ph.D



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No 7

METRIC

PROJECT N-90-031

CLIENT McKenzie Local RD

DATUM Geodetic

LOCATION Sta. 1 + 003 O/S 9m Rt.

BOREHOLE TYPE Hand Auger

DATE 1990 03 08

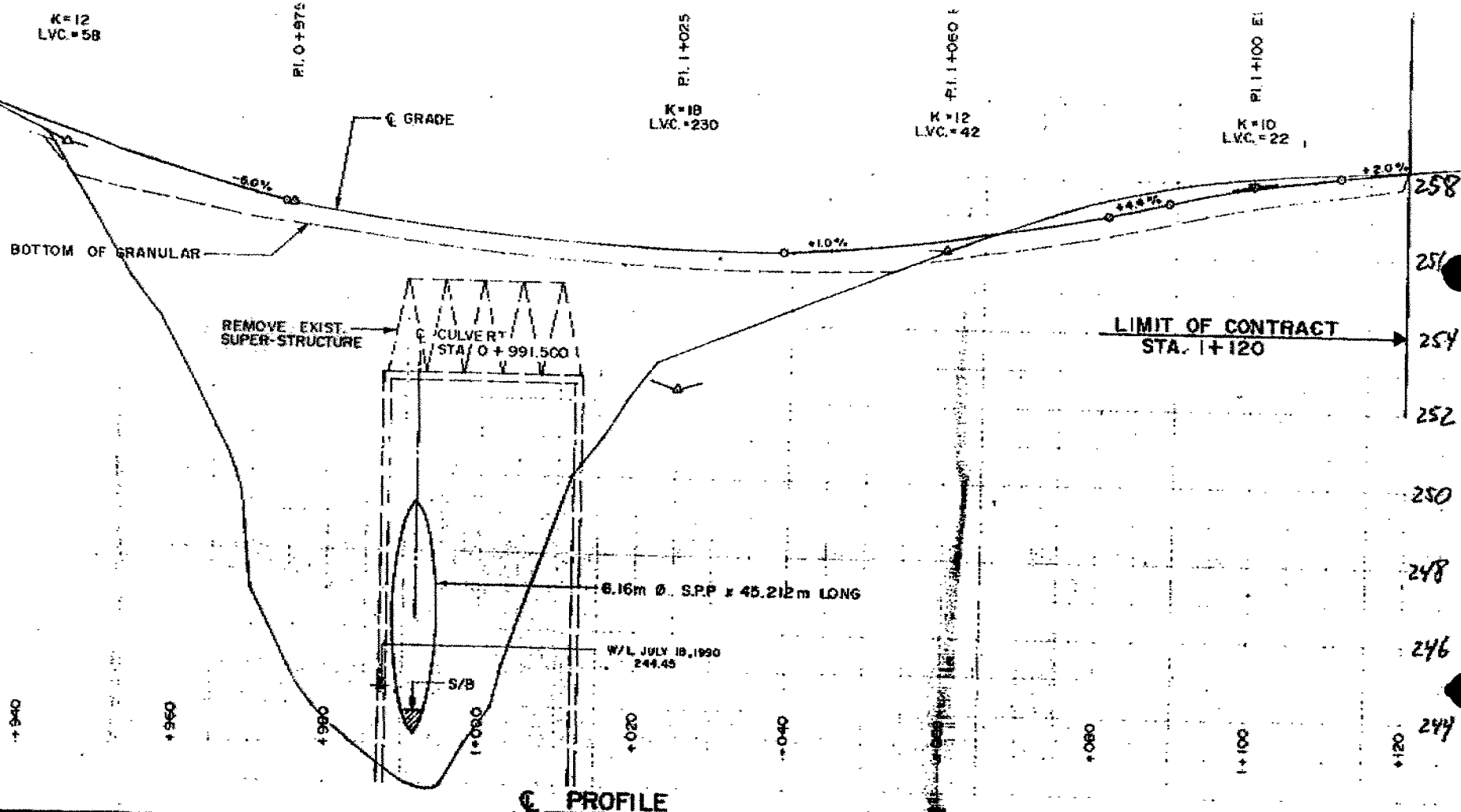
ORIGINATED BY CH

COMPILED BY AK

CHECKED BY CH

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STAT. PLOT	NUMBER	TYPE			W _N VALUES	20				
0.05	50 mm Ice Peat Soft											
0.25	Clayey Silt Sat Soft Reddish Brown Hard											
0.90	Return to hand Auger											

• 1, 2, 3: Numbers refer to
Sensitivity20
15 → 5 (%) STRAIN AT FAILURE
10



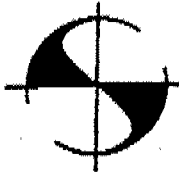
PROFILE



No.	DATE	BY	REVISIONS

DESIGNED: K.A.
 DRAWN: P.S.S.
 CHECKED: F.P.C.
 APPROVED: F.P.C.
 SCALE: 1:500 HOR.
 1:100 VERT.

MCKENZIE LOCAL ROADS BOARD
 SOUTH MAPLE ISLAND BRIDGE
 LOT 1 SOUTH CHANNEL OF MAGNETAWAN RIVER
 SITE PLAN



STRATA ENGINEERING CORP.

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RESEARCH . ENGINEERING . SCIENCE

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FOUNDATION INVESTIGATION REPORT

South Maple Island Bridge

McKenzie Local Roads Board

Totten Sims Hubicki Associates

Strata Project: N-90-031

Date of Submission: 1991 04 16

Report Distribution:

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Strata File: N-90-031	- 1 copy

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FOUNDATION INVESTIGATION REPORT

South Maple Island Bridge

McKenzie Local Roads Board

Totten Sims Hubicki Associates

1.0 INTRODUCTION

Strata Engineering Corp. has been retained by Totten Sims Hubicki Associates of Bracebridge, Ontario, on behalf of the McKenzie Local Roads Board, to conduct a foundation investigation for a replacement bridge across an overflow (south) channel of the Magnetewan River. The terms of reference originally provided were to investigate the site by means of sampled boreholes with one borehole cored into bedrock, and to provide a full geotechnical report for the bridge structure and for road pavement. Subsequently, a request was made to investigate the site for its feasibility to accommodate an SP-CSP type of soil-steel structure.

This report is submitted in compliance with these terms of reference and contains recommendations for the design and construction of: the foundations for the proposed replacement structure, retaining walls, grade revisions, and approaches, as well as recommendations for the bedding and backfilling of an SP-CSP alternative.

The subsurface information provided in this report is based on a total of six power auger drilled boreholes and local geological evidence, complemented by hand auger borings in the upstream section of the river. The findings in this report are intended for use in design only. The liability of the factual data given in this report is limited only to the information at borehole locations.

2.0 SITE AND GEOLOGY

Maple Island is located on the Magnetewan River, just north of Highway 520 and about 12km downstream from Ahmic Lake. The island splits Magnetewan River into two channels. The south channel (this site) serves as an overflow route. Main flows occur through the north channel (Totten Sims Hubicki Hydrology Report dated October 29, 1990).

The site location is shown on the Key Plan, appended. The existing bridge provides access to approximately 14 homes and numerous cottages on the Island.

The geology of the area is predominantly precambrian with shield rocks being prominent. There is evidence of ice-contact drift from the most recent glaciation.

The surrounding area has a gently rolling to undulating topography. Several denuded slopes in the general area south of Maple Island show the presence of outwash type materials. The present road carrying traffic to the Island dips steeply towards the existing single lane steel truss bridge (from the south towards the north) and curves away and up after the bridge (towards the north).

Minor shoreline undercutting is evident in some localized areas where the river channel becomes narrow near the bridge itself. The slopes bordering the river are stable and often as steep as 1.75:1. Numerous boulders are evident within the valley and in the river channel downstream of the bridge.

3.0 FIELD AND LABORATORY WORK

The power auger field investigation commenced on 1990 12 05 and was completed on 1990 12 06. Six boreholes were drilled at locations as shown on appended Sketch N90031-SK-1, with a CME 55 machine mounted on a bombardier tracked vehicle. Hollow stem augers were used throughout to advance these boreholes. Bedrock was cored at Borehole 4 in BQ size.

The ground elevations at the borehole locations are referenced to Geodetic datum, and have been inferred from the profile and plan provided by Totten Sims Hubicki, Preliminary Drawing P1 dated Oct. 1990 under their Project No. 36-0483.

In the machine drilled boreholes (Boreholes 1 to 6 inclusive) samples of the overburden were obtained in the split-barrel sampler, the N values being noted for the Standard Penetration Test in blows/0.3m.

On 1991 03 08, hand auger borings were attempted upstream of the bridge to determine streambed level soil conditions for a proposal to consider an SP-CSP structure in lieu of a bridge. Only one borehole, marked as Borehole 7 on Sketch N90031-SK-1, was successfully completed to competent materials. The other boreholes are not shown since they encountered boulders below surficial peat and organic soils. Due to the prevailing depth of water and thin ice conditions, no borings could be made in the river channel proper. On 1991 03 08, the upstream portion of the river and flood plain was iced over with 25 to 50mm of ice cover, whereas the downstream portion was open flowing water.

In the hand augered Borehole 7 location, soil samples were obtained off the auger flight.

Groundwater was not encountered in any of the machine drilled boreholes. Borehole 7 was drilled below ice level and therefore, no groundwater level information could be obtained for this location.

In the laboratory, all soil samples were examined visually. Index property tests, such as moisture contents, grain size distributions and Atterberg limits, were conducted on selected specimens. The laboratory test results are shown on Figures 1 to 3 and on the Borehole Log Sheets in the Appendix. A Borehole log sheet for Borehole 7 has not been prepared.

4.0 SUBSURFACE CONDITIONS

4.1 Shoreline Organics

In the vicinity of Borehole 7, surficial organics, consisting of soft peat and muck, were encountered to depths of 600mm below ice level. In many locations tested with the hand auger, refusal was encountered on presumably boulders in the streambed below the peat. The moisture content of the peat was found to be in excess of 200 per cent.

4.2 Sand and Gravel (Road Fill)

In Boreholes 1, 5 and 6, up to 500mm of sand and gravel was found. This material is relatively well graded and may be re-used for road structure design purposes as a Granular B.

4.3 Silty Sand

In Borehole 2, located off the existing roadway, a silty sand was encountered from the surface down to a depth of 0.9m where the borehole was ended. This material is non-cohesive, and contains some gravel. One grain size distribution curve is shown in Figure 1.

4.4 Clayey Silt (Glacial Till)

In Boreholes 1, 4, 5 and 6, a deposit of clayey silt was encountered. The thickness of this cohesive deposit ranged between 0.7m and 2.3m, being thickest at Borehole 4 where it was encountered from the ground surface down. The clayey silt contains some sand and occasional gravel, suggesting a glacial origin. However, in some samples, isolated zones of uniform texture, with little sand or gravel typical of the till fabric, were also found.

The moisture content of the deposit ranged between 18 and 28 per cent, averaging about 20 per cent. Atterberg limits (Figure 2) show the soil to be a clayey silt. The moisture content lies at or close to the plastic limit of the soil.

N values of generally over 10 blows/ft. and tactile examinations in the laboratory suggest the soil to be firm to very stiff, being generally stiff.

A reddish-brown clayey silt was encountered below about 300mm of peat in Borehole 7, to 900mm below ice level. The upper 200mm of the soil was soft and saturated. However, below this upper zone, the material was hard in consistency, and exhibited a moisture content of about 15 per cent. The hand auger could not be turned past the 900mm depth.

4.5 Silty Sand to Sandy Silt

All boreholes encountered a silty sand to sandy silt stratum, generally below the clayey silt stratum or at ground surface. In Borehole 6, the material was more a silt (see Figure 3).

The moisture content in this stratum ranged between 11 and 20 per cent, averaging about 13 per cent. N values of as low as 5 blows/0.3m to over 50 blows/0.3m suggest the soil is loose to very dense.

In several cases, cobbles and boulders made an evaluation of the in situ density difficult to assess.

4.6 Bedrock

A white to light grey carbonate metasediment with dark sills was encountered and proven by diamond coring at Borehole 4. Core recoveries were excellent and the RQD values were found to be in excess of 80 per cent. The upper 1m of the rock shows slightly more fractures. However, there is no gouge present and the fractures are fairly tight. Slight discolouration suggests water flow and iron oxide deposition.

4.7 Groundwater

Groundwater was not encountered in any of the machine drilled boreholes (BH 1-6). In March 1991, the river water level was 7.7m below deck level.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

Two alternatives are being considered for the replacement of the existing single span steel single lane steel truss bridge. One alternative is to replace the bridge at approximately the same location with a new structure. The other alternative is to consider deployment of a 6m diameter circular shaped SP-CSP (Structural Plate Corrugated Steel Pipe) conduit, located just upstream of the existing bridge, in order to utilize the existing concrete abutments as wing walls for the downstream end of the conduit.

5.2 Foundation Design - Bridge Structure

Abutments founded within the overburden at a depth of 2.0m for frost protection purposes will be underlain by either compact to dense silty sand to sandy silt or by a slightly compressible clayey silt deposit. It is therefore recommended that the replacement bridge be supported on spread footings taken down an additional metre or so to the bedrock. Footings on bedrock may be designed for the following factored bearing capacities:

In ULS	5,000kPa
In SLS Type II	Not applicable

The rock surface should be examined, at time of construction, to review any deleterious jointing patterns that may affect the stability of the rock mass under full loading from the abutments. Such jointing patterns can not be detected by one vertical core drilled hole.

5.3 Foundation Design - SP-CSP Structure

The hand auger borings attempted along the north shoreline upstream of the existing bridge show the likely presence of competent glacial till at streambed level. Therefore, no foundation problems are anticipated for the bedding of an SP-CSP structure.

It is presumed the structure will be built under fully dewatered conditions.

For design and quantity estimating purposes, assume the average depth of organic removal to be 450mm and that of softened mineral soil (glacial till) to be 200mm.

5.4 Conduit Bedding

All organics and softened clayey silt should first be stripped to competent native mineral soil below the bedding and to a distance of not less than 3m to either side of the spring line of the conduit.

Provide 300mm of Granular B bedding below the conduit invert, extending laterally, as a minimum to the spring lines. The upper 150mm of the bedding should not be compacted, so as to allow proper seating of the invert plate corrugations into the bedding. Provide a camber of 25mm at the centre of the longitudinal axis of the structure to accommodate potential invert gradient settlement.

5.5 Conduit Backfilling

The "engineered zone" of backfill around the conduit should extend transversely 3m beyond the conduit spring lines (assuming as 6m diameter conduit). The backfill material should consist of clean (less than 10% material finer than 75 μ m) non-cohesive, granular material, placed in maximum 200mm loose lifts, each lift being compacted to 100% Std. Proctor density before the placement and compaction of the next lift. The imbalance between the two sides in backfill height should not exceed 300mm.

Beyond the engineered fill zone any local earth material is acceptable, provided it is placed and compacted to 95% Std. Proctor density.

The engineered fill zone should extend to 1.0m above the crown of the conduit before native fill is permitted. Also, the 1.0m minimum height of cover should be maintained before allowing construction equipment to cross the conduit transversely.

Heavy vibratory compaction equipment should be kept at a distance of 1.0m minimum from the sidewalls of the conduit. Compaction equipment should be run parallel to the longitudinal axis of the conduit.

5.6 Hydraulic Cut-Offs and End Treatments

In order to prevent hydraulic uplift and potential buckling of the base plates of the SP-CSP structure, provisions should be made as per current OHBD Clause 12 (revised version Clause 7) for cut-offs at both ends as necessary. Corrugated plates could be used for this purpose. Use the OHBD Code provisions for end treatments and for ice ride-up, should that be seen to be necessary based on hydrological considerations.

5.7 Deformation Control during Backfilling

Upward crown deflection should be monitored to maintain the deformation, during backfilling, to OHBD Code permitted values.

5.8 Approach Fills

No stability problems are anticipated for approach fills to either type of structure, if they are built with standard 2:1 slopes and any underlying peat and organics are stripped to competent mineral soil.

Approach fill settlements will be elastic in nature and will occur almost immediately upon application of load. Hence, normal 6m length approach slabs should be adequate for a conventional bridge alternative.

5.9 Erosion Protection

Erosion protection measures will be required for sand fills encroaching upon the river banks. Protection may be provided with a geotextile covering held in place with appropriate sized rip-rap and armour stone to high water level plus freeboard.

5.10 Earth Pressures

Earth pressures to abutments and wing walls of a replacement bridge structure should be computed as per subsection 6-6.1.2.2 of the OHBD Code. A yielding foundation condition may be assumed. Surcharge effects, if any, should be computed as per Clause 6-6.1.2.4 of the OHBD Code. The following design parameters are recommended for Granular "A" or "B" backfill:

	Gran. "A"	Gran. "B"
Angle of internal friction, ϕ'	35.0°	30.0°
Unit Weight (kN/m ³), γ	22.8	21.2

For the conduit option (SP-CSP) we should be consulted to provide guidance with respect to the selection of an appropriate modulus of subgrade reaction value, E' , to use in designing the steel shell. This value will depend on the type of material selected for the engineered backfill zone.

5.12 Pavement Structure

Provide 300mm Granular B sub-base, 150mm Granular A base and 80mm hot mix (HL 4) in two equal lifts, as per current OPSD standards for the type of road cross-section chosen. Carry the granulars full width. The existing roadway granular material may be re-used as Granular B.

5.11 Quality Control/Assurance

It is recommended that preliminary design drawings and the final contract documents be sent to us for review and comments from a geotechnical engineering point of view. A program of quality assurance and control should be implemented for the SP-CSP option, as per OHBD Code requirements.

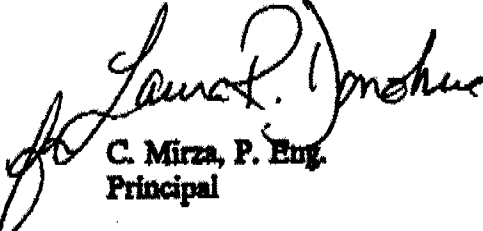
6.0 CLOSURE

The power auger field work for this investigation was supervised by Mr. Gordon Lo, P. Eng., working under the direction of Mr. C. Mirza, P. Eng. Hand auger borings were made by Mr. C. Mirza, P. Eng.

The drilling equipment was rented from Master Soil Investigation Limited of Weston, Ontario.

This report supersedes all previous correspondence dealing with factual data and recommendations for this site.

Respectfully submitted:
STRATA ENGINEERING CORP.


C. Mirza, P. Eng.
Principal



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N-90-031

APPENDIX

Explanation of Terms Used in Report

Key Plan

Borehole Location Plan

Record of Boreholes 1 to 6

Figures 1 to 3



STRATA ENGINEERING CORP.

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 31mm 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 (31mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 1" 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 (LBS)	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 - 1	2 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

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

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

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

JOINTING AND BEDDING:

SPACING	50mm	50 - 100mm	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 OSIERBERG SAMPLE
S I SIGHTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H P W ADVANCED HYDRAULICALLY
C S CHURN SAMPLE	P M P W ADVANCED MANUALLY
I W THINWALL OPEN	F S FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

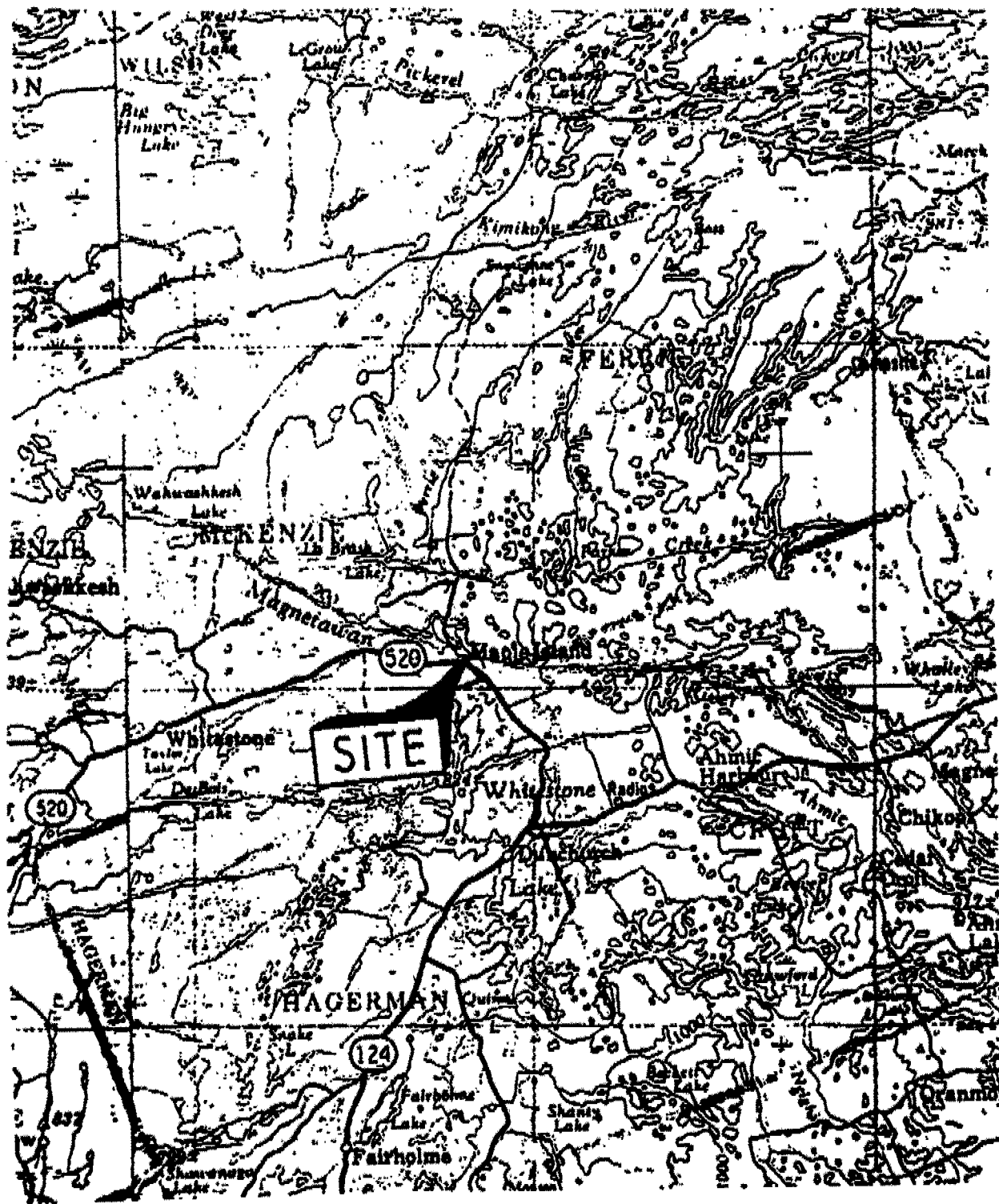
m_v	$1/m^3$	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m^2/s	COEFFICIENT OF CONSOLIDATION
m		DRAINAGE PATH
t_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	lps	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	lps	PRECONSOLIDATION PRESSURE
τ_f	lps	SHEAR STRENGTH
c'	lps	EFFECTIVE COHESION INTERCEPT
ϕ'	-	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	lps	APPARENT COHESION INTERCEPT
ϕ_u	-	APPARENT ANGLE OF INTERNAL FRICTION
τ_h	lps	RESIDUAL SHEAR STRENGTH
τ_s	lps	REMOULDED SHEAR STRENGTH
s	1	SENSITIVITY = $\frac{c_u}{\tau_s}$

STRESS AND STRAIN

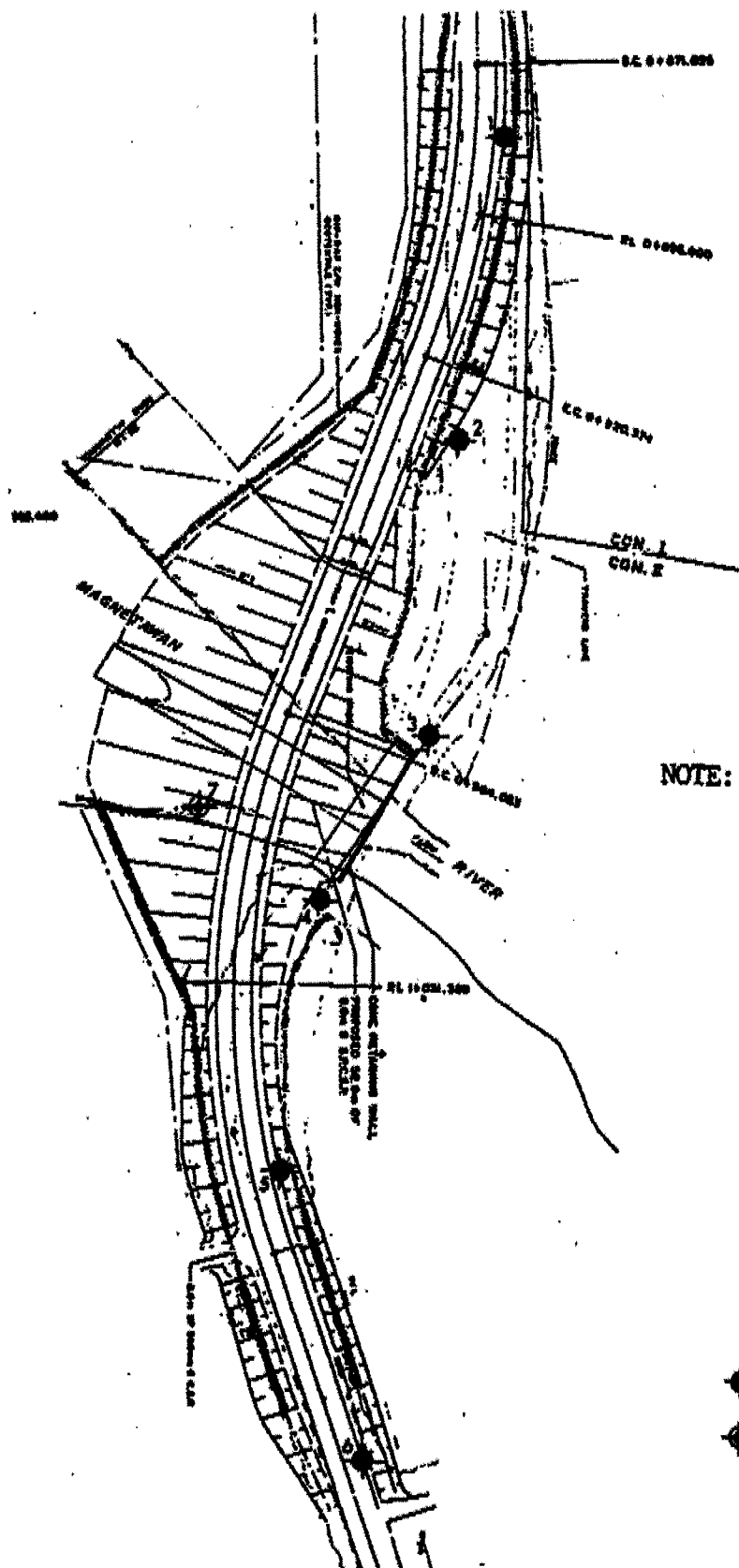
u_o	lps	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	lps	TOTAL NORMAL STRESS
σ'	lps	EFFECTIVE NORMAL STRESS
τ	lps	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	lps	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	lps	MODULUS OF LINEAR DEFORMATION
G	lps	MODULUS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

PHYSICAL PROPERTIES OF SOIL

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



KEY PLAN



NOTE: Boreholes drilled in December 1990 were for a proposed replacement bridge near existing alignment. Since then, the alignment has been revised, as shown here, for a proposed SP-CSP 6m diameter conduit.

- Power Auger Holes
- ⊗ Hand Auger Hole

SCALE
12.5m 0 12.5m



STRATA ENGINEERING CORP.

BOREHOLE LOCATION PLAN FOR
SOUTH MAPLE ISLAND BRIDGE



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No1

METRIC

PROJECT N-90-031

LOCATION Sta. 0+884 5m L.L.

CLIENT McKensia Local RB

BOREHOLE TYPE Hollow Stem Auger

ORIGINATED BY GL

DATUM Geodetic

DATE 1990 12 05

COMPILED BY JK

CHECKED BY CH

SOX PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
LEV DEPTH	DESCRIPTION	STRAIT PLOT	NUMBER	TYPE	"N" VALUES			75	15	40	80	100		
264.5	Ground Surface													
0.0	Sand & Gravel - Rd. Fill		1	SS	10		254							
0.2														
	Clayey Silt with sand, occ. grav. (Glacial Till)		2	SS	20									
263.2	V. Stiff- Hard. Brown													
1.2	Silty Sand, some Gravel. Dense						253							
262.9			3	SS	36									
2.0	End of Borehole													
	*Borehole dry upon completion													

*3, *5, Numbers refer to
Sensitivity20
15
10
+5 (%) STRAIN AT FAILURE



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No 2

METRIC

PROJECT N-90-031

LOCATION Sta. 0+931 o/s 10m Lt.

CLIENT McKenzie Local AB

BOREHOLE TYPE Hollow Stem Auger

DATUM Geodetic

DATE 1990 12 06

ORIGINATED BY CL

COMPILED BY AK

CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
DEPTH	DESCRIPTION	TEST POINT	NUMBER	TYPE	VALUES			20	40	60	80				
260.0 0.0	Ground Surface														
	Silty Sand, some gravel.		1	SS	22	*									21 61 (18)
259.1 0.9	Compact. Brown		2	SS	77										
	End of Borehole * Borehole dry upon completion														

*S, N: Numbers refer to
Sensitivity20
15
10
5
0
5
10
15
20
(%) STRAIN AT FAILURE



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No 3

METRIC

PROJECT H-90-031

LOCATION Sta. 0+979 o/a 23rd Lt.

CLIENT McKenzia Local RD

BOREHOLE TYPE Hollow Stem Auger

ORIGINATED BY CL

DATUM Geodetic

DATE 1990 12 06

COMPILED BY AK

CHECKED BY CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	LIQUID LIMIT W _L	SHRINKAGE LIMIT W _s	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
DEPTH	DESCRIPTION	NUMBER	TYPE	VALUES			70 80 90 100	110 120					
251.5	Ground Surface												
0.0	Silty Sand, some gravel. Brown occ. cobbles & boulders. V. Dense	1	SS	8/20cm									
		2	SS	62/20cm									
249.2	End of Borehole												
2.0	* Borehole dry upon completion												

* 1, 2 : Numbers refer to
Sensitivity20
10 10 10 (%) STRAIN AT FAILURE
10



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No 4

METRIC

PROJECT N-90-031

LOCATION Sta. 1+016 o/s 13m Lt

CLIENT McKenzie Local RS

BOREHOLE TYPE Hollow Stem Auger & Diamond Drilling (Rock)

ORIGINATED BY GL

DATUM Geodetic

DATE 1990 12 05 & 06

COMPILED BY AK

CHECKED BY CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	LIQUID LIMIT W _L	SHRINKAGE LIMIT W _s	UNIT WEIGHT Y	REMARKS A GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	TEST PILOT	NUMBER	TYPE	VALUES		30	40	50	60	70	80	90	100	
251.5 0.0	Ground Surface														
	Clayey Silt with some sand, Tr. Grav (Glacial Till)		1	SS	11										
	Reddish Brown to Brown		2	SS	13										
	Very Stiff-Hard														
248.7 2.8	occ. mottling		3	SS	17										
248.0	Sandy Silt occ. cobbles		4	SS	30										
	Dense, Brown														
245.7 3.3	Bedrock		5	BQ	Rec- RC 75%										
	White - Lt. Grey Carbonate Meta-sedi- ment, some intrusive sills of metamorph. type.														
	Tight fractures. No infilling.		6	BQ	Rec- RC 100%										
245.7 5.8	End of Borehole • Borehole dry upon completion														

• 3, 4, 5 : Numbers refer to
Sensitivity

30
15
10
+ 6 (%) STRAIN AT FAILURE



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No 5

METRIC

PROJECT N-90-031

LOCATION Sta. 1+061 a/s 5m Lt.

CLIENT McKenzie Local RD

BOREHOLE TYPE Hollow Stem Auger

DATUM Geodetic

DATE 1990 12 05

ORIGINATED BY: CL

COMPILED BY: AK

CHECKED BY: CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) OR SA SI CL
SLV DEPTH	DESCRIPTION	SLV - POI	NUMBER	TYPE	W VALUES			30	40	60	80	100	W _p		
255.5	Ground Surface														
0.0	Sand & Gravel		1	SS	21	*									
255.0	(Md. Fill) Brown-Gray														
0.5	Clayey Silt with some sand, occ. grav. (Gl. Till). Stiff. Brown.		2	SS	7										
254.5															
1.2	Sandy Silt														
253.6	V. Dense		3	SS	52										
1.9	End of Borehole														
	* Borehole dry upon completion														

*3, 4, 5: Numbers refer to
Sensitivity30
15-30 (%) STRAIN AT FAILURE
10



STRATA ENGINEERING CORP.

RECORD OF BOREHOLE No 6

METRIC

PROJECT N-90-031

LOCATION Sta. 1+111 o/s 3.5m Lt.

ORIGINATED BY CL

CLIENT McKenzie Local RB

BOREHOLE TYPE Hollow Stem Auger

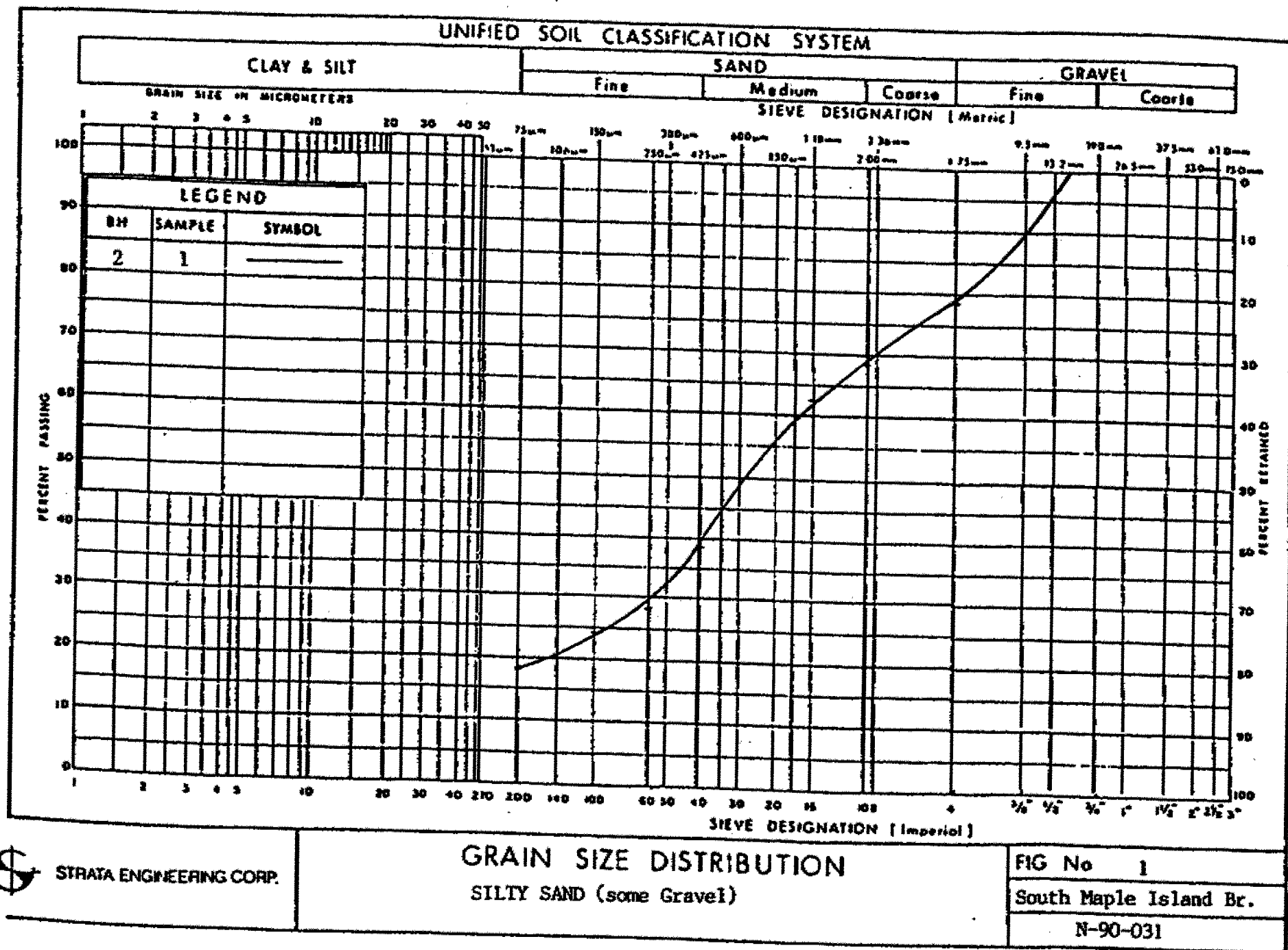
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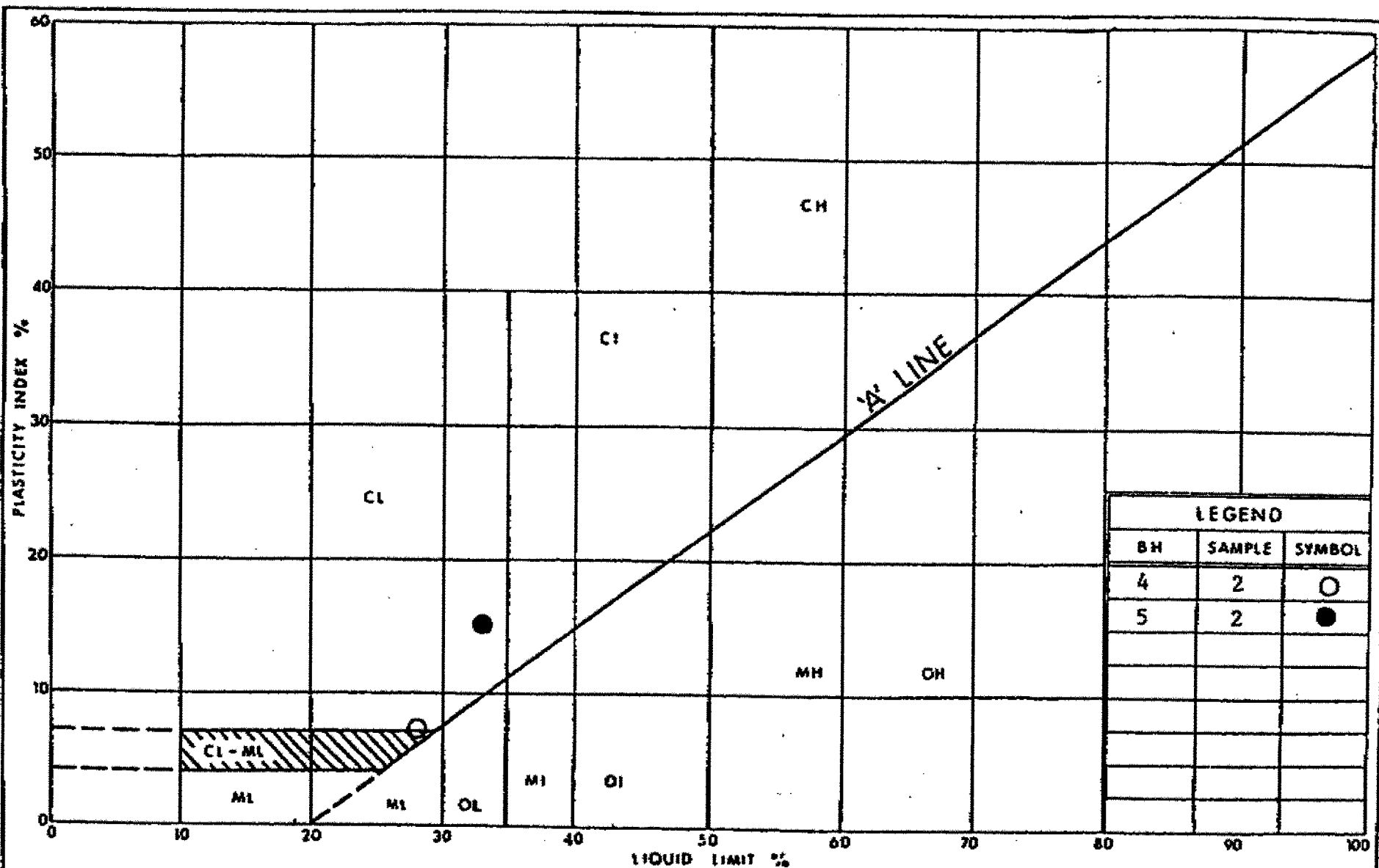
DATUM Geodetic

DATE 1990 12 05

CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			30	40	60	80	100		
258.2	Ground Surface.													
0.0	Sand & Gravel		1	SS	17		258							
257.7	(Bd. Fill) Compact													
0.5	Clayey Silt with some sand, occ. grav.		2	SS	13									
257.0	(Glacial Till). Stiff						257							
1.0	Silt, some sand													
256.5	Gray-Brown, Loose		3	SS	5									
2.0	End of Borehole													
	Borehole dry upon completion													

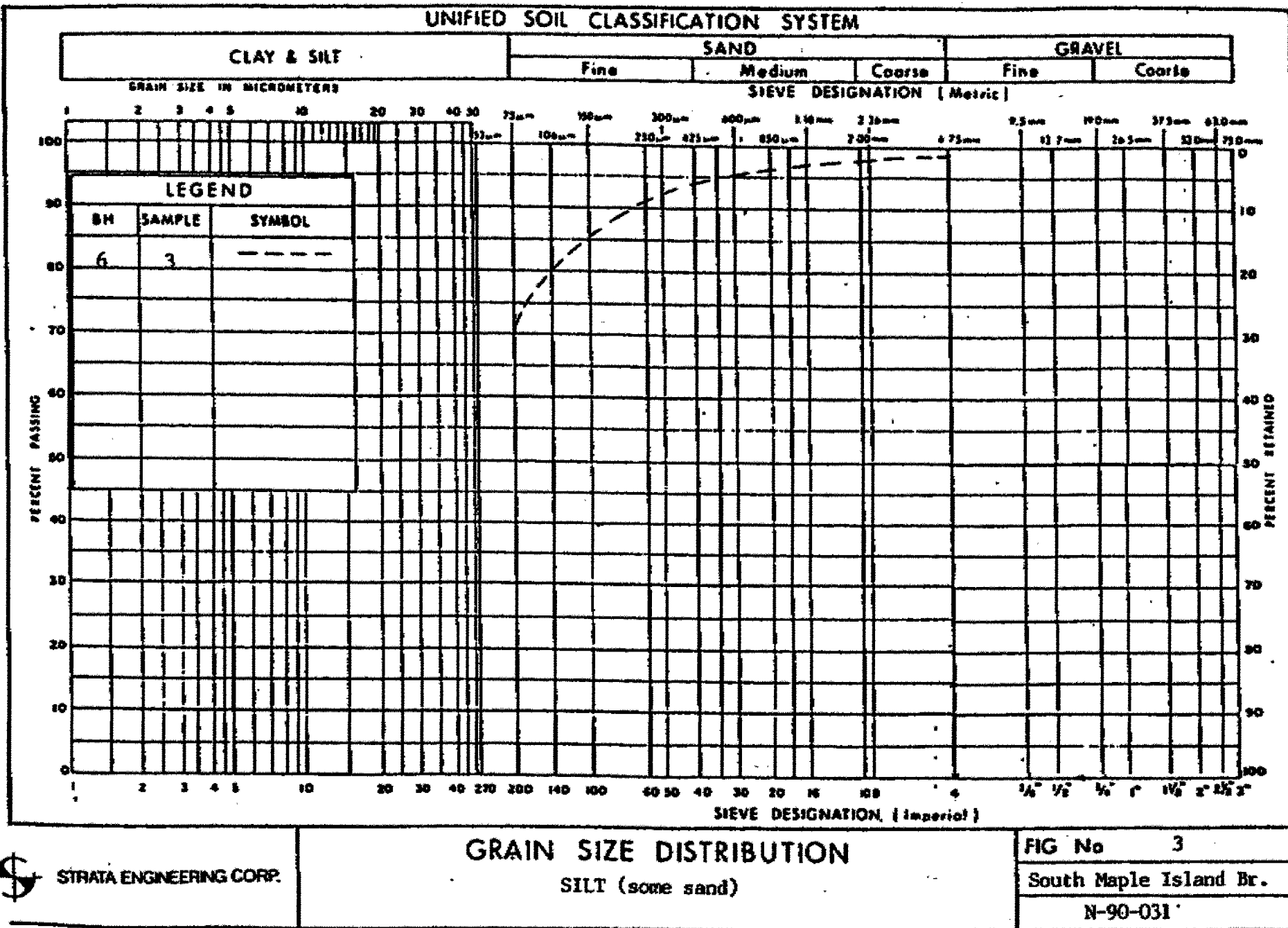




STRATA ENGINEERING CORP.

PLASTICITY CHART
CLAYEY SILT (Glacial Till)

FIG NO	2
PROJECT	N-90-031
SITE	South Maple Isl.



STRATA ENGINEERING CORP.

GEOCRES No. 31E-124DIST. 52 REGION W.P. No. CONT. No. W. O. No. 98-11002STR. SITE No. 44-057HWY. No. LOCATION North Maple Island Bridge
ReplacementNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



**FOUNDATION INVESTIGATION REPORT
FOR
NORTH MAPLE ISLAND BRIDGE
MTO AGREEMENT NO. 9750-7813-5270, MTO SITE 44-57
McKENZIE TOWNSHIP, DISTRICT OF PARRY SOUND**

PREPARED FOR:

K. SMART ASSOCIATES LIMITED

**TROW CONSULTING ENGINEERS LTD.
Brampton, Cambridge, Hamilton, London, Markham,
North Bay, Ottawa, Sudbury, Thunder Bay, Winnipeg**

**Project: SO7455G
Date: Dec. 3, 1997**

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Fax: (705) 674-8271**

**FOUNDATION INVESTIGATION REPORT
FOR
NORTH MAPLE ISLAND BRIDGE
MTO AGREEMENT NO. 9750-7813-5270, MTO SITE 44-57
McKENZIE TOWNSHIP, DISTRICT OF PARRY SOUND**

SO7455G

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SO7455G

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BOREHOLES 3, 4, 7 & 8	Approaches
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BOREHOLES 1 & 3	Previous Terraprobe Information

PART 1

FOUNDATION INVESTIGATION

1.1 INTRODUCTION

This submission presents the results of a geotechnical investigation completed by Trow Consulting Engineers Ltd., Sudbury office, for the North Maple Island Bridge replacement (MTO Site No. 44-57). The project consists of the design and construction of a new, single span bridge, with a new alignment, some 10 m to the north (downstream) from the present structure. The project will also incorporate new east and west approach embankments and the general arrangement is shown schematically on the attached Drawing 1A.

1.2 SITE DESCRIPTION

The existing bridge was reportedly constructed in the 1930's and is located approximately 2 km northeast of Highway 520 on the local road that crosses the main channel of the Magnetewan River, between the communities of Whitestone and Dunchurch. The site is located in the Township of McKenzie, Lots 1 and 2, Concession 2, in the District of Parry Sound, Ontario.

The existing structure is a steel truss on concrete abutments, with a concrete deck. It has a single, 24.4 m long span with a width of 3.96 m between the curbs. The selected replacement alternative will be a one lane, single span, concrete deck and concrete girder structure constructed about 10 m north (downstream) of the existing bridge. The proposed design will also upgrade the vertical and horizontal alignments, entrances and drainage.

The river is approximately 27 m wide at the proposed bridge crossing. The slopes of the shallow creek banks vary from as shallow as approximately 3H:1V to as steep as 1H:1V. There is exposed bedrock at the bridge location, both in the river bed and on the banks. At the time of the field work, the depth of water was approximately 1 m. The level of the water level in the river at the time of the field work was approximately 244 m, as recorded

by K. Smart and Associates Limited. The creek bed was near elevation 242.8 m at the mid point of the river. The height of the existing approach embankments, above the river bed level in the immediate vicinity of the abutments was about 4.5 m. The grade of the proposed new bridge deck will be approximately 0.8 m higher than the existing structure, i.e. elevation 249.8 m.

1.3 GEOLOGICAL SETTING

Based on published geological information, the predominant overburden soil at the site is anticipated to consist of glacial till, which is predominantly "sandy" in texture. The till, however, is expected to be shallow and may be absent, thus exposing areas of outcropping bedrock. Intervening deposits of fluvioglacial soils, i.e. silts and clays, can also occur in relatively level sections of terrain with a potential for surficial organic deposits in any low-lying, poorly drained sections.

The underlying bedrock is from the Central Gneiss Belt which generally consists of a mafic, layered, hornblende, biotite gneiss, that is black to dark grey in colour with secondary minerals including garnet and sulphides. Other rock types found in the Central Gneiss Belt in this locality, which may be present include white phaneritic marble, as well as breccia.

1.4 INVESTIGATIVE PROCEDURES

1.4.1 General:

The investigation procedures adopted for the geotechnical assessment, at the abutments for the new structure, as well as along the realigned approaches, are described below in Section 1.4.2 (structures) and 1.4.3 (embankment approaches). Properties of the shallow overburden soils were obtained by in-situ testing in the field and with standard laboratory testing, i.e. moisture content and grain size analyses.

Details of the soil and bedrock conditions encountered in all the boreholes are included in the attached Appendix. Locations of the boreholes are plotted on the attached plan,

Drawing 1B. The field work was completed in the period November 20 to November 25, 1997, inclusive.

1.4.2 Bridge Structure:

The field work consisted of four(4) boreholes (boreholes 1 and 2, as well as boreholes 5 and 6) which were advanced to depths ranging from 0.6 m to 5.77 m below grade. The boreholes were drilled in the vicinity of the proposed abutments as shown on the enclosed plan, Drawing 1B.

The boreholes were advanced at the west abutment (boreholes 5 and 6) using cased wash boring and diamond drilling techniques, using a hydraulic drill placed on a raft. At the east abutment (boreholes 1 and 2), the boreholes were drilled through the overburden with a skid mounted unit, equipped with solid stem flight auger equipment with capabilities for diamond drilling into the bedrock.

At one of the two boreholes at each proposed abutment location (boreholes 1 and 5), rock coring techniques were used to advance the boreholes approximately 3 m into the underlying bedrock. The casing also had to be advanced by diamond drilling techniques through the bouldery fill over the rock. Standard "B" sized casing was used to extract BQ size core of the bedrock, which was retrieved for rock quality designation and classification purposes.

1.4.3 Approaches:

Along the realigned east approach centreline, the two boreholes (boreholes 3 and 4) were drilled to machine auger refusal at depths of 0.7 m to 2.4 m below grade with a skid-mounted drill equipped with standard solid stem flight augers. Along the proposed west approaches, however, (boreholes 7 and 8) access was not feasible for the skid-mounted machine auger drill related to steep embankment slopes and dense bush. The two boreholes advanced on this side of the river were completed using hand power auger equipment.

1.5 SUBSURFACE CONDITIONS

1.5.1 Bridge Location:

As previously discussed, the location of the four(4) boreholes at the proposed two new bridge abutments are shown on the enclosed site plan, Drawing 1B. Details of the soil conditions in the boreholes, as well as relevant previous borehole data at this bridge (Terraprobe report No. 96-1063 dated March 3, 1997, prepared for Sutcliffe Limited) are included in the attached appendix. Based on the borehole information, a soil stratigraphy has been prepared, approximately perpendicular to the new alignment at each of the two abutments and a soil profile has been drawn along the centreline of the proposed alignment as shown on Drawings 1D and 1C, respectively.

Referring to this information, it is evident that the soil stratigraphy at the abutments consists of dense bouldery rock fill (likely blast rock) directly overlying shallow bedrock. As noted previously, it was not possible to auger through this bouldery fill layer, hence the hole was advanced by wash boring techniques and advancing the casing by diamond drilling. Standard split spoon samples were attempted (two at borehole 5 and one at borehole 1) inside the casing in the bouldery fill layer; however, zero penetration was established after 60 blows, i.e. the spoon was "bouncing" on the boulders. Based on this data and site observations during the field work, it is concluded that the bouldery fill layer is dense.

The thickness of bouldery rock fill overlying the bedrock will depend on the final size, location and configuration of the proposed abutments, since the south side of the new abutments will encroach into the embankment fill along and around the abutments of the existing bridge (see Drawing 1D).

The bedrock at the west abutment is anticipated to be at approximate elevation 241.4 m and at the east abutment at approximate elevation 244 m. We should caution, however, that this type of bedrock is known to have sharp irregularities in its surface profile. As such, the level of rock may be different between the test locations.

The bedrock was established in borehole 1 (east abutment) and borehole 5 (west abutment) by obtaining rock cores which were used for subsequent classification of the rock. Detailed descriptions of the rock is included in Table 1 in the enclosed Appendix. The bedrock at both abutments is similar, consisting of a slightly weathered to unweathered, strong, Biotite-Hornblende Gneiss. In the lower, approximately 400 mm at borehole 5 (west abutment), the rock type changes to marble. The Rock Quality Designation (RQD) values were generally in excess of 75 percent, indicating a sound rock of good quality.

1.5.2 Approaches:

The locations of the four(4) boreholes advanced along the new approach embankments are shown on the site plan, Drawing 1B. Details of the soil conditions in these boreholes are included in the Appendix. A soil stratigraphy along the centreline profile of the approaches is shown on Drawing 1C. Drawings 2A and 2B provide additional information of soil descriptions.

At the approaches along the east side (boreholes 3 and 4), the subsoil, beneath a thin organic veneer of 75 mm to 150 mm of topsoil, consists of essentially competent granular soils. At borehole 3, the subsoil comprises approximately 660 mm of dense sand, gravel and cobbles. At borehole 4, the subsoil consists of 2.36 m of compact to dense, silty sand. Refusal to the machine augers occurred at depths of 0.66 m and 2.36 m in boreholes 3 and 4, respectively, on assumed bedrock or cobble/boulder obstructions within the overburden. A typical grading of the sand from borehole 4 is included on Drawing 3.

Along the west approaches (boreholes 7 and 8), the subsoil is similar, consisting, beneath a thin veneer of topsoil (100 mm to 150 mm thick), of compact sand with some silt. Refusal to the hand power auger drilling unit occurred at a depth of 1.6 m to 2.2 m in boreholes 7 and 8, respectively, on assumed bedrock or a cobble/boulder obstruction within the overburden. A typical grading of the sand from borehole 8 is included on the attached Drawing 3.

1.6 GROUNDWATER CONDITIONS

At the time of the investigation (November 1997), the water level in the Magnetewan River was recorded by K. Smart and Associates Limited survey crew to be at approximate elevation 244 m. Information regarding the groundwater levels at the site was observed by measuring the water levels, if any, in open boreholes after the completion of drilling. Other than the two boreholes which were completed from the raft on the river, (boreholes 5 and 6), no groundwater of any consequence was intercepted in any of the remaining six(6) boreholes.

1.7 CHEMICAL TESTS

Chemical tests were completed on a sample of the water obtained at borehole 5. This sample is essentially the Magnetewan River water.

The sample has a pH of 7.2 with negligible concentration (mg/ml) of sulphates.

PART 2

ENGINEERING DISCUSSION AND RECOMMENDATIONS

2.1 INTRODUCTION

The following subsections address geotechnical considerations pertaining to the North Maple Island Bridge replacement. It is understood that the replacement option includes a single span structure, located some 10 m to the north (downstream). New approaches, up to 4 m high, will be required on each side of the new bridge.

2.2 FOUNDATIONS

2.2.1 General:

Geotechnical conditions at the proposed abutments are similar, consisting of varying thicknesses of existing embankment fill (associated with the present road and bridge abutments) overlying bedrock. The thickness of the overlying fill will vary, depending on the final size and location of the new abutments. Based on the borings and sections presented on Drawing 1D, the fill may be as thick as 6 m at the west abutment (rock level ~ El. 241.4 m) and somewhat shallower, about 2 m at the east abutment (rock level ~ El. 244 m). The river level was established, during the field work, at elevation 244 m. As such, excavation design and construction of the east abutment should be reasonably straightforward, since conventional footings on shallow rock above the water table are envisaged. At the west abutment, however, because of the deeper rock level and the fact that the proposed abutment is located in the river, design and construction will be more complex. The design of the two abutments are discussed below.

2.2.2 East Abutment

Given the fact that the geotechnical conditions consist of about 2 m of bouldery rock fill over bedrock (see Drawing 1D - Section A.A), it is recommended that the new foundations be designed and constructed directly on bedrock. For the purpose of design, based on the Ontario Highway Bridge Design Code, the following bearing capacity can be used for

Section B-B -

spread footings placed directly on bedrock, subject to inspection by a qualified geotechnical engineer.

Factored Bearing Resistance At ULS = 5000 kPa

The above Factored Bearing Resistance at ULS applies to spread footings subjected to vertical loads, and placed directly on rock. The footing base(s) must be cleared of all loose materials and shatter prior to placement of concrete, and inspected by a qualified geotechnical engineer to verify the competency of the bedrock. The bedrock may be considered to be a non-yielding stratum; hence no serviceability limit state (SLS) capacity has been provided

Foundations must be designed in conformance with Sections 6 to 8 of the Ontario Highway Bridge Design Code, in particular adjustments may have to be incorporated to the factored bearing resistance (ULS) to account for inclined loads (Section 6 - 8.4.2) and/or eccentric loading conditions (Section 6 - 8.5.3).

2.2.3 West Abutment:

2.2.3.1 Foundations on Bedrock:

As noted above, conditions at the west abutment are more complex. The level of the bedrock was established at approximate elevation 241.5 m, requiring excavations of at least 6 m through the existing road embankment fill (Drawing 1D - Section B-B). The existing embankment fill appears to consist of blast rock fill in the immediate vicinity of the present concrete abutments, grading into silty sand. The river level, at the time of the field work, was recorded at elevation 244 m. This level could also fluctuate seasonally. Open cut excavations, therefore, down to the bedrock will likely not be feasible. The existing bouldery rock or sand fill embankments should be temporarily stable if cut back at 45 degrees to the horizontal above the river level, during the anticipated short construction stage. On the other hand, below the water level (El. 244 m), the rock fill may "slough" back to a shallower slope of about 2H:1V, and any sandy embankment fill will "slough" back to even flatter slopes of about 3H:1V. As such, there is insufficient

Section A-A

distance available to open cut the proposed abutment excavation, unless the present road can be closed.

Based on the above, therefore, in order to establish foundations on bedrock, using the design parameters discussed in Section 2.2.2, above, excavations will probably have to be undertaken within the confines of a properly designed, "drop" caisson. The soil exposed in the excavations must be supported. The bouldery rock and sand fill will be highly permeable, hence, it will not be feasible to seal the caisson at the rock surface. In this regard, the bedrock will have to be cleaned and inspected "underwater" and the foundation concrete tremied below river level.

In order to reduce the extent of excavations and possibly enable open cut techniques to be used, consideration could be given to relocating the abutment either further to the north (away from the existing road embankment) and/or further back (requiring a longer span). If the abutment location is moved, additional borings will be necessary to confirm the level and competency of the rock at the new abutment location.

2.2.3.2 Foundations on Blast Rock Fill:

As an alternate concept, consideration may be given to supporting the abutment (as a "perched" foundation) on an engineered mat of blast rock fill above the water level, the rock fill can be placed at the proposed abutment location directly over the existing dense bouldery fill. Once the rock fill reaches the river level, the surface and remaining height must be thoroughly compacted to the underside of the foundations.

With this concept, the following bearing pressure can be considered:

Factored Bearing Resistance at ULS = 400 kPa

Factored Bearing Resistance at SLS = 300 kPa

It is difficult to predict accurate settlement values. Based on previous experience, it is probable that the settlements will be in the order of 0.5 to 1.0 percent of the height of rock fill. If the rock fill is carefully selected, as a well-graded crushed rock (maximum size 450 mm), thoroughly compacted with heavy vibratory equipment, and the placement

supervised on a full time basis by a qualified geotechnical engineer, then the settlements will probably be in the lower range, i.e. 0.5 percent of the height, using the recommended SLS bearing pressure of 300 kPa. Assuming a rock fill thickness of about 3 m, settlements are likely to be in the order of 15 mm to 30 mm. This settlement, however, will tend to be elastic, i.e. occur more or less as the loading is applied, hence most of the movement should occur during construction.

The engineered rock fill pad must extend well beyond the edges of the abutment footing to ensure both adequate load spread (1.5H:1V) and to prevent scouring at the foundation.

2.3 FROST PROTECTION

Frost cover is not required for footings placed directly on bedrock. Footings on blast rock fill should be provided with a minimum of 2 m of earth/rock fill cover or equivalent rigid insulation.

2.4 SLIDING RESISTANCE

The computation of the sliding resistance of the foundation shall be carried out in accordance with the Ontario Highway Bridge Design Code (Section 6 - 6.2.2). A friction angle, ϕ' , of 32 degrees and 38 degrees can be used for sliding along discontinuities within the bedrock and blast rock fill, respectively, at the interface at the footing base. If the factored resistance against sliding failure, based on friction alone is inadequate, then a passive resistance key should be excavated into the bedrock or rock fill, or the foundation anchored into the rock to resist shear.

2.5 ABUTMENT BACKFILL

Backfill to abutments or retaining walls should consist of free-draining granular materials such as Granular "A" and Granular "B" or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following table.

Material Types and Unfactored Properties					
Material	Friction Angle, ϕ'	$\gamma(\text{kN/m}^3)$	K_a	K_p	K_o
Granular A	35 degrees	22.5	0.27	3.7	0.43
Granular B	30 degrees	21.5	0.33	3.0	0.50
Clean Rock Fill	38 degrees	19.0	0.24	4.2	0.38

Note: K_a is the earth pressure coefficient corresponding to the active state.

K_o is the earth pressure coefficient at rest

K_p is the earth pressure coefficient corresponding to the passive state.

2.6 EXCAVATIONS

2.6.1 General:

As outlined previously, excavations to bedrock for foundations at the east abutment should be reasonably straightforward.

At the west abutment, however, excavation conditions to bedrock are expected to be more complex, since the bedrock is anticipated to be deeper and the structure is located in the present river channel adjacent to the existing road embankment fill. Our comments related to excavation at both abutments are summarized below.

2.6.2 East Abutment:

Bedrock is expected to occur at an approximate elevation of 244 m and hence excavation of some 2 m of overburden (mostly bouldery fill) will be required. It is unlikely that the bedrock surface will be level, hence areas of lean concrete ("dental" infilling) may be necessary to provide a reasonably horizontal surface for the abutment foundations. Alternatively, bedrock removal may be necessary to provide a reasonably level bearing surface.

Excavation and stripping of the overburden should be straightforward. The overburden can be classified as a Type 3 soil, related to the Occupational Health and Safety Act.

Any bedrock excavations, i.e. to level the rock or to provide a shear key, will require drilling and blasting techniques. From previous experience, the type of bedrock expected at this site is known to be brittle. It is often difficult to blast and hence excavate to "neat" lines using conventional drilling and blasting procedures, since problems with "overbreak" are common. This potential problem may affect quantities claimed by the contractor for rock excavations, as well as the amount of imported dental concrete required to compensate for rock excavations, as well as the compensate for "overbreak". The contractor should, therefore, make adequate allowances for these conditions. Some consideration may have to be given to pre-splitting techniques in critical areas in order to reduce potential problems. Due consideration must also be given to controlling blasting procedures, in order to prevent potential damage to the adjacent, nearby properties. Limiting the depth of subdrilling to control overbreak beneath the required foundation grade, while still achieving the desired break, is also an important factor that must be considered by the contractor. Overbreak conditions, i.e. rock shatter, under footing bases should be assessed by the geotechnical engineer prior to decisions either to continue over excavation in the rock, or alternatively to prepare footing bases on the specified grade.

2.6.2 West Abutment:

In order to establish this abutment on bedrock, excavation of up to 6 m will be required through the existing bouldery rock or sand fill road embankment (see Drawing 1D). The lower approximately 2.5 m will be below river level.

As discussed in Section 2.2.3.1, above, it is unlikely that open excavation through the embankment fill, down to bedrock, will be feasible using open cut techniques. The bouldery rock fill, in the immediately vicinity of the existing abutments and sand fill beneath the road embankment can both be classified as a Type 4 soil in the Occupational Health and Safety Act. If the abutment is to be located at the presently proposed position, then excavation to bedrock through the fill and below the water table will have to be undertaken within the confines of properly designed, temporary shoring (drop caisson), in order to ensure the integrity of the existing road.

The bedrock surface will probably not be level and the overlying embankment fill is permeable. It is unlikely that construction of an earth cofferdam to cut off water seepages below river level will be practical. As such, since it will not be possible to seal the temporary shoring at the bedrock contact, the final cleaning and inspection of the bedrock surface will have to be completed underwater.

Any excavation of the bedrock will require drilling and blasting techniques as outlined in Section 2.6.2, above.

2.7 EROSION AND SCOUR PROTECTION

The abutment foundations and embankment approaches must be protected from potential scour and erosion. For foundations on bedrock, scour and erosion procedures will not be necessary; however, the embankments will need erosion protection. If the embankments are constructed with blast rock fill, then no special protection will be required. The extent and sizing of scour and erosion protection will depend extensively on design parameters related to hydrological effects and current velocities, etc.

The abutments must also be protected from the effects of ice, particularly the west abutment, if it is supported on the rock fill mat alternate.

2.8 APPROACH EMBANKMENTS

2.8.1 Stability and Settlement:

As shown on the general site plan, Drawing 1A, and the proposed profile on Drawing 1C, approach embankment heights of up to 4 m will be required.

The subsoils encountered along the approach embankments (boreholes 3, 4, 7 and 8) consist of compact and/or dense, granular deposits, i.e. sand and/or sand and gravel. As such, we do not foresee any stability nor settlement problems along the new approaches.

Depending on the material selected for construction of the approaches, the following safe side slopes are recommended:

Poorly graded blast rock fill	1.5H:1V
Well graded blast rock fill	1.25H:1V
Sand or poorly graded sand and gravel	2.5H:1V
Well graded sand and gravel	2H:1V

2.8.2 Road Base:

At present, the approach embankments have a gravel travelling surface. It is anticipated that a gravel surface will be retained and hence a pavement design has not been selected.

Depending on the type of soil used to construct the embankments, the following road base composition is recommended:

Blast Rock Fill Embankment

Granular Surface	150 mm OPSS Granular "A"
Granular Subbase	150 mm OPSS Granular "B"

Sand/Sand and Gravel Fill Embankments

Granular Surface	150 mm of OPSS Granular "A"
Granular Subbase	300 mm of OPSS Granular "B"

A non-woven geotextile should be placed to separate the subbase granular material from the rock fill and to prevent potential migration of fines. Assuming the rock fill can be properly "blinded" and "chinked", and the maximum rock size is limited to 300 mm, then the geotextile may consist of Terrafix 270R, or equivalent. If the rock size is larger, then a stronger geotextile, Terrafix 400R or equivalent should be used.

For the bridge deck, the following asphalt thickness may be considered:

Asphalt	50 mm HL-4 (surface course)
	50 mm HL-4 (Binder)

It is noted that the road is gravel surfaced; hence, salt is generally not used. In this case, the asphalt layer(s) on the bridge deck may not be necessary unless there is a possibility of surfacing the road in the future.

Organic (topsoil) or soft subsoils should be removed from beneath the proposed embankments prior to fill placement. All common earth fill materials placed beneath the roadway should be compacted to a minimum of 95 percent of the Standard Proctor Maximum Dry Density in lifts not exceeding 150 mm. The granular subbase and base fill materials should be compacted to a minimum of 100 percent SPMDD in lifts of 150 mm or less. Asphalt materials should be rolled and compacted to a minimum of 97 percent Marshall Bulk Density (MBD).

Immediately prior to placement of the road base granular courses, the subgrade should be proof-rolled and any loose, soft or unstable areas should be subexcavated and backfilled with compacted select subgrade materials.

The above road base design thicknesses are considered adequate for this local road. However, if construction occurs in wet, inclement weather, it may be necessary to provide additional subgrade support for heavy construction traffic by increasing the thickness of the granular subbase or base. Further, main traffic access areas for construction equipment may experience unstable subgrade conditions. These areas may be stabilized utilizing additional thicknesses of granular materials.

It is recommended that inspection and testing be carried out by a competent material testing firm during construction to confirm material quality, thickness and to ensure adequate compaction.

2.9 CHEMICAL TESTS

The river water has negligible concentrations of sulphates in a neutral environment. There should not, therefore, be any serious deterioration, by sulphates in the water, of a good quality, dense, foundation concrete made from ordinary Portland cement.

2.10 CLOSURE

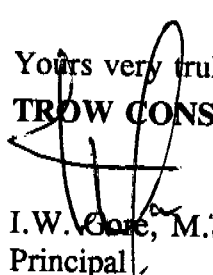
The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed bridge replacement. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the borehole logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

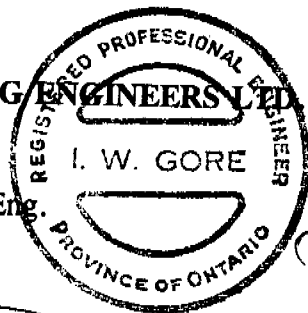
This report has been prepared by I.W. Gore and reviewed by S.E. Gonsalves and E.A. Gonneau.

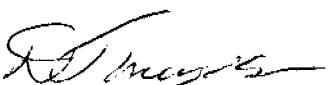
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

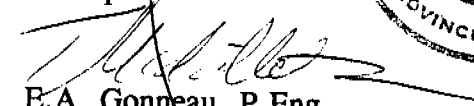
Yours very truly,

TROW CONSULTING ENGINEERS LTD


I.W. Gore, M.Sc., P.Eng.
Principal




S.E. Gonsalves, P.Eng.
Vice President


E.A. Gonneau, P.Eng.
Project Engineer

IWG:gm52

Encl.

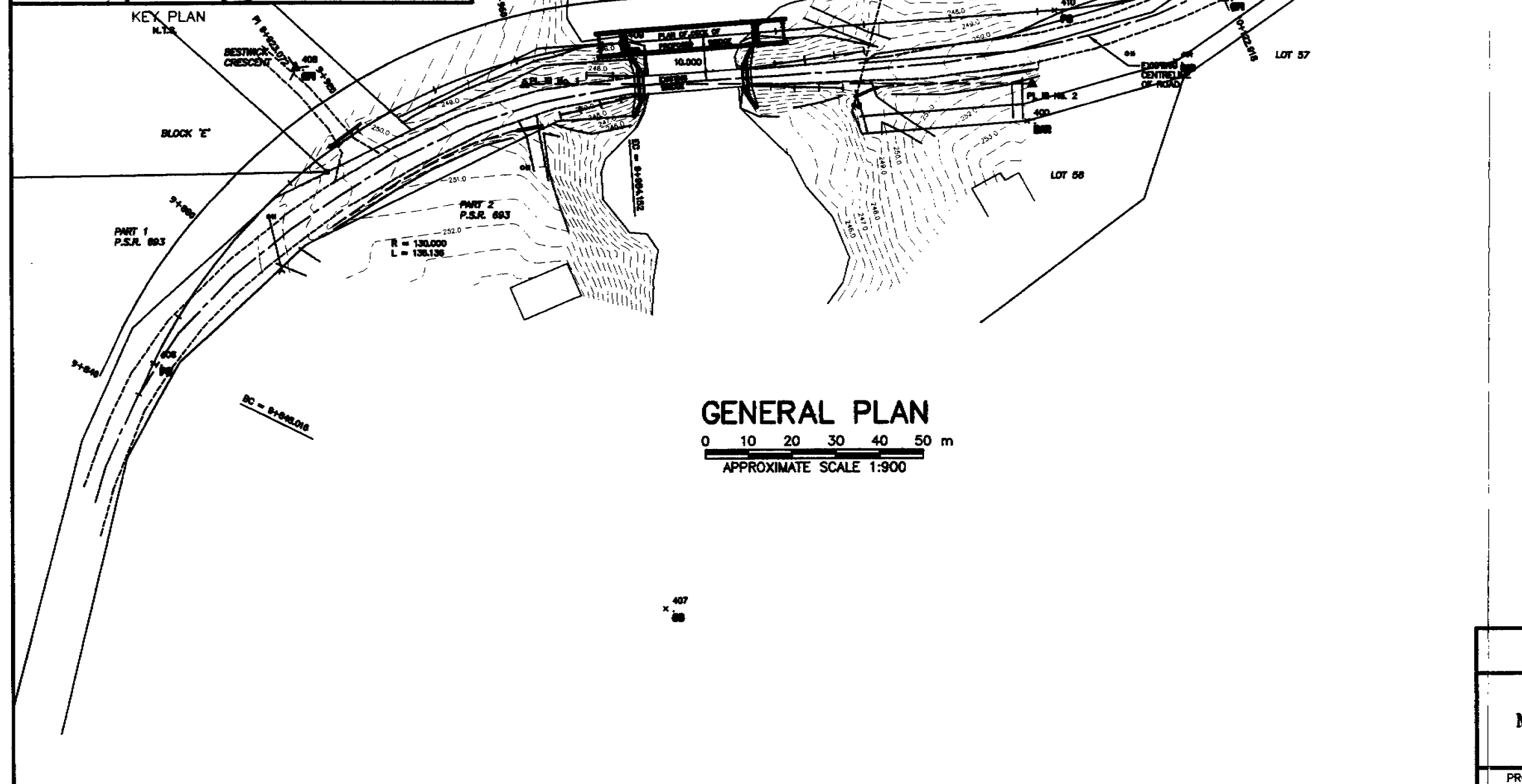
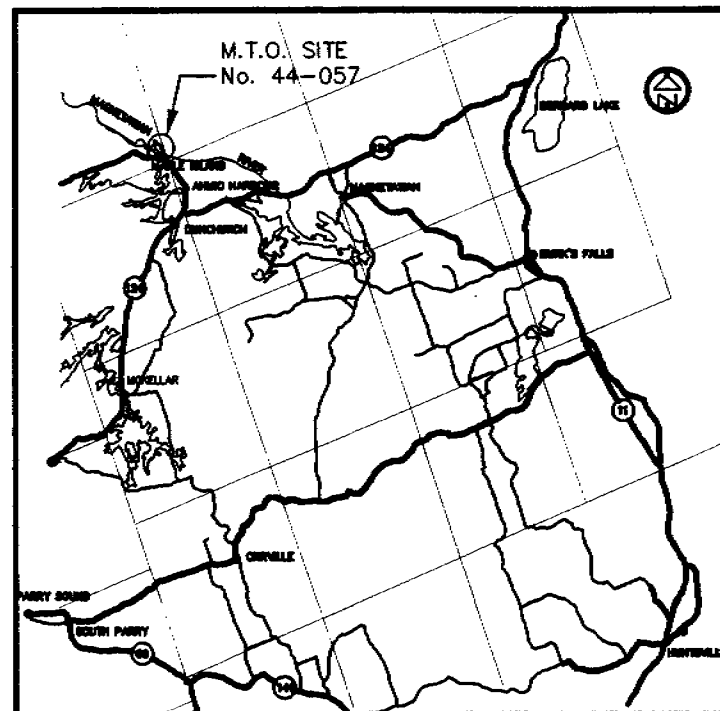
Dist: K. Smart Associates Ltd. (3)
Mr. E.M. Abraham, P.Eng.

APPENDIX

SO7455G		TABLE 1 ROCK CORE DESCRIPTION				
BH #	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
1	1	1.22 to 2.74	100	98	1.22 to 4.17	Biotite-Hornblende Gneiss , greyish black to dark grey, some brownish metasedimentary inclusions, medium to coarse grained, strong, unweathered, fractures widely spaced, dipping at 50° from vertical, planar, smooth
	2	2.74 to 3.57	100	90		
	3	3.57 to 4.17	100	100		
2	1	2.54 to 3.66	100	75	2.54 to 5.35	Biotite-Hornblende Gneiss (Garnetiferous) , greyish black to very light grey, medium to coarse grained, strong, unweathered to slightly weathered, fractures moderate to very close spaced, dipping to near vertical, planar, smooth
	2	3.66 to 4.11	100	78		
	3	4.11 to 4.81	100	91		
	4	4.81 to 5.77	100	70		
					5.35 to 5.77	Marble , pinkish white to white, medium to very coarse grained, strong, unweathered, no fractures

*CR - Core Recovery

**RQD - Rock Quality Designation



DRIVING TITLE	BOREHOLE LOCATIONS	REVISED	A5
ORIENTATION			
© 1997 H. SUTCLIFFE LIMITED PROTECTED BY COPYRIGHT - DO NOT COPY			

CONTROL POINT No. 1 - L.S. - ELEV. 248.037m
 STA: 9+482.805 - EXISTING ALIGNMENT
 OFFSET: 5.200 L
 STA: 9+480.843 - PROPOSED ALIGNMENT
 OFFSET: 2.827 R
 N 6082.842
 E 4975.886

CONTROL POINT No. 2 - L.S. - ELEV. 251.918m
 STA: 10+076.808 - EXISTING ALIGNMENT
 OFFSET: 8.004 R
 STA: 10+077.204 - PROPOSED ALIGNMENT
 OFFSET: 12.347 R
 N 6186.075
 E 5057.043

10+000 - MEASURED MIDPOINT OF BRIDGE

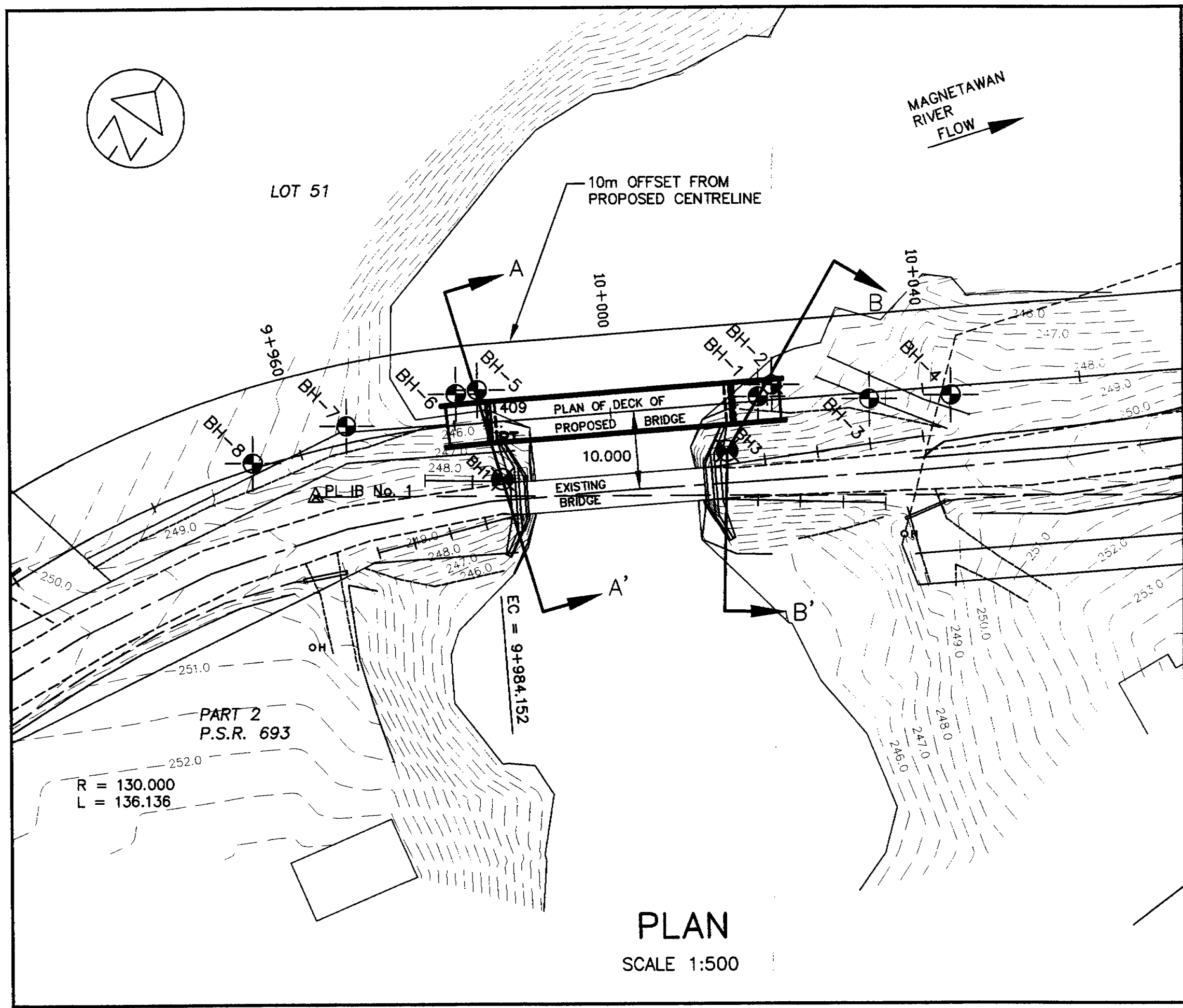
NOTE

Plan provided by the client.

Trow
 GEOTECHNICAL EVALUATION



**SITE LOCATION & GENERAL PLAN
 MAPLE ISLAND BRIDGE REPLACEMENT
 M.T.O. BRIDGE SITE No. 44-57**

PROJ. No. S07455G | DATE: DECEMBER 1997 | DWG. No. 1A



PLAN
SCALE 1:500

LEGEND

-  TROW BOREHOLE (NOV/97)
-  TERRAPROBE BOREHOLE (MAR/97)

NOTES

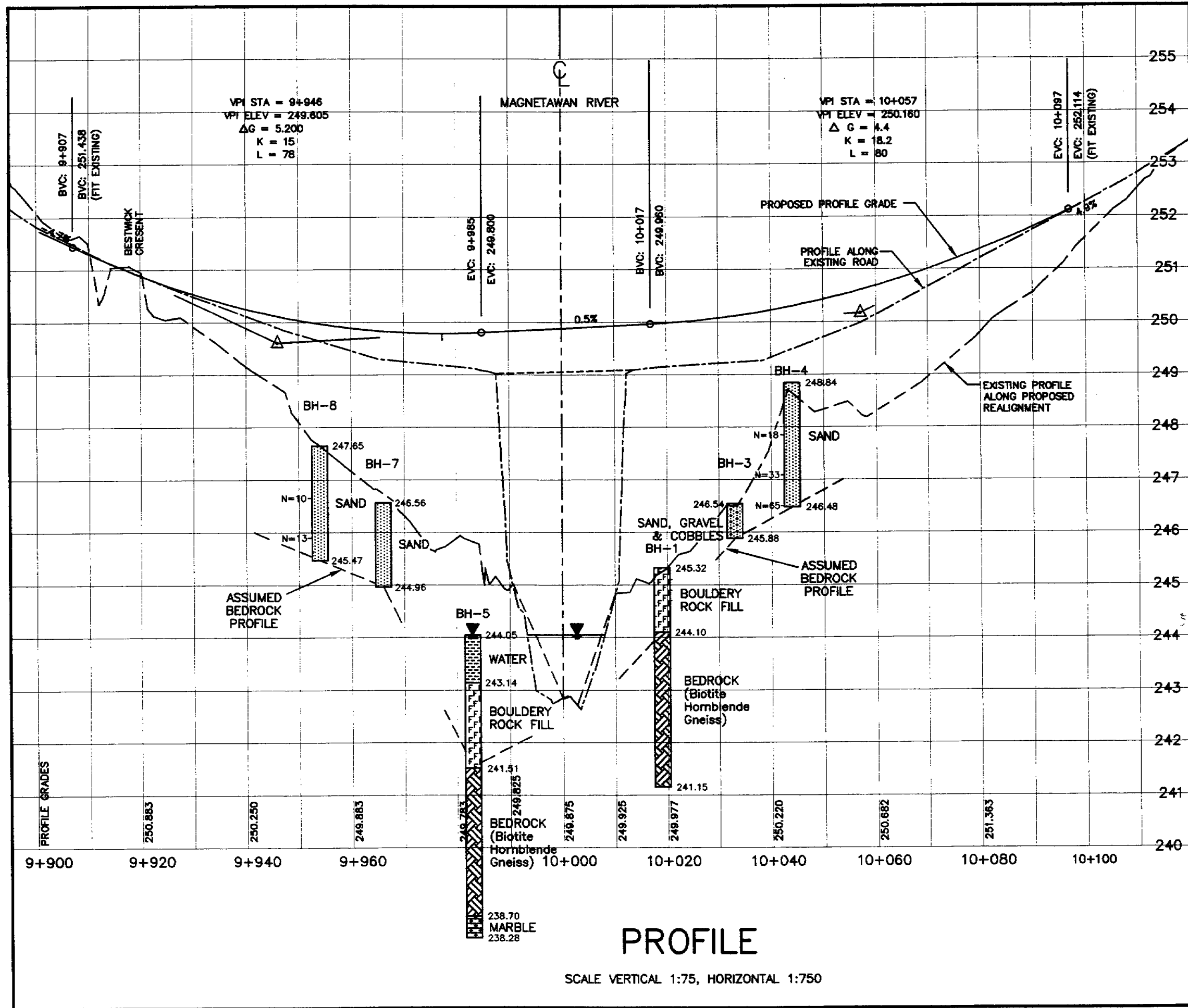
- 1) Plan supplied by Sutcliffe Limited.
- 2) Borehole locations and elevation supplied by K. Smart Associates.

- NOTES -

- 1) The boundaries and soil types have been established only at Test Hole locations. Between Test Holes they are assumed and may be subject to considerable error.
- 2) Do not use Test Hole elevations for design purposes.
- 3) Soil samples will be retained in storage for 1 year and then destroyed unless client advises that an extended time period is required.
- 4) Quantities should not be established from the information provided at the Test Hole locations.
- 5) This drawing forms part of the report, project number as referenced, and should be used only in conjunction with this report.

Trow
GEOTECHNICAL EVALUATION

**BOREHOLE LOCATION PLAN
MAPLE ISLAND BRIDGE REPLACEMENT
M.T.O. BRIDGE SITE No. 44-57**



— NOTES —

- 1) The boundaries and soil types have been established only at Test Hole locations. Between Test Holes they are assumed and may be subject to considerable error.
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- 5) This drawing forms part of the report, project number as referenced, and should be used only in conjunction with this report.

Trow

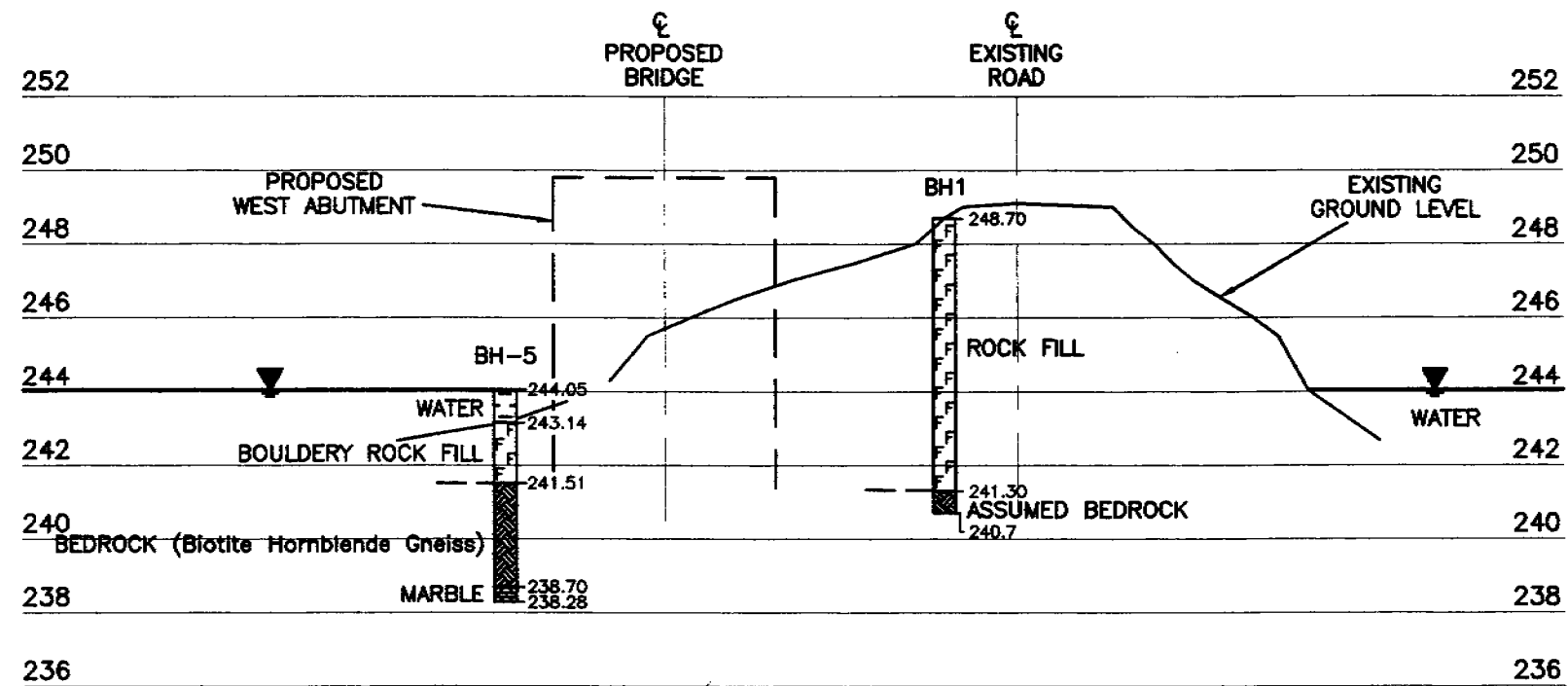
GEOTECHNICAL EVALUATION

CENTRELINE PROFILE
MAPLE ISLAND BRIDGE REPLACEMENT
M.T.O. BRIDGE SITE No. 44-57

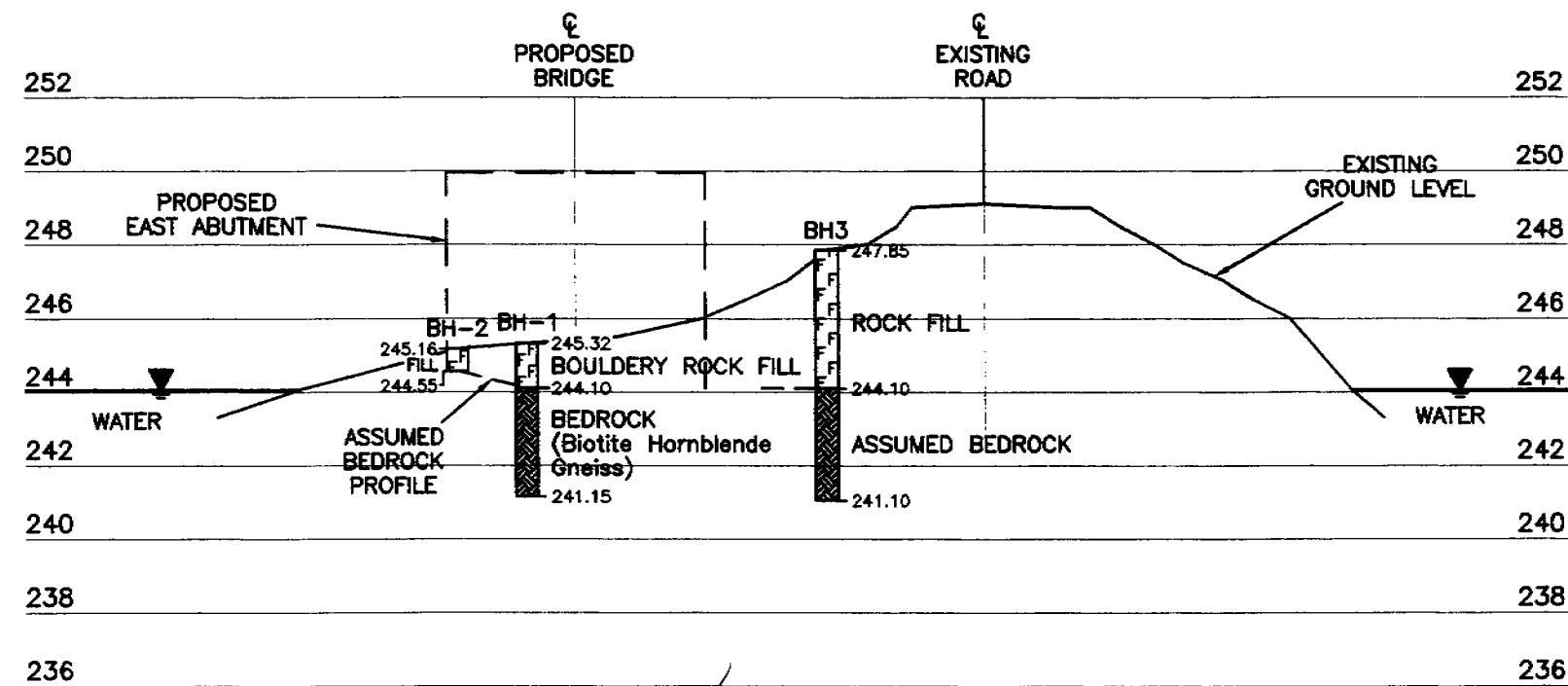
PROJ. No. S07455G

DATE: DECEMBER 1997

DWG. No. 1C



SECTION A-A', WEST ABUTMENT
SCALE 1:200



SECTION B-B', EAST ABUTMENT
SCALE 1:200

Trow

GEOTECHNICAL EVALUATION

SECTIONS AT ABUTMENTS
MAPLE ISLAND BRIDGE REPLACEMENT
M.T.O. BRIDGE SITE No. 44-57

PROJ. No. S07455G

DATE: DECEMBER 1997

DWG. No. 1D

NOTES ON SAMPLE DESCRIPTIONS

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel		Cobbles		
				Fine	Medium	Coarse	Fine	Coarse			
I.S.S.M.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
Sieve Sizes											
<div><div>0.001</div><div>0.002</div><div>0.003</div><div>0.004</div><div>0.006</div><div>0.008</div><div>0.01</div><div>0.02</div><div>0.03</div><div>0.04</div><div>0.06</div><div>0.075</div><div>0.08</div><div>0.1</div><div>0.2</div><div>0.3</div><div>0.4</div><div>0.6</div><div>0.8</div><div>1.0</div><div>2.0</div><div>3.0</div><div>4.0</div><div>6.0</div><div>8.0</div><div>10</div><div>20</div><div>30</div><div>40</div><div>60</div><div>80</div></div>											
Particle Size (mm)											

2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No:

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

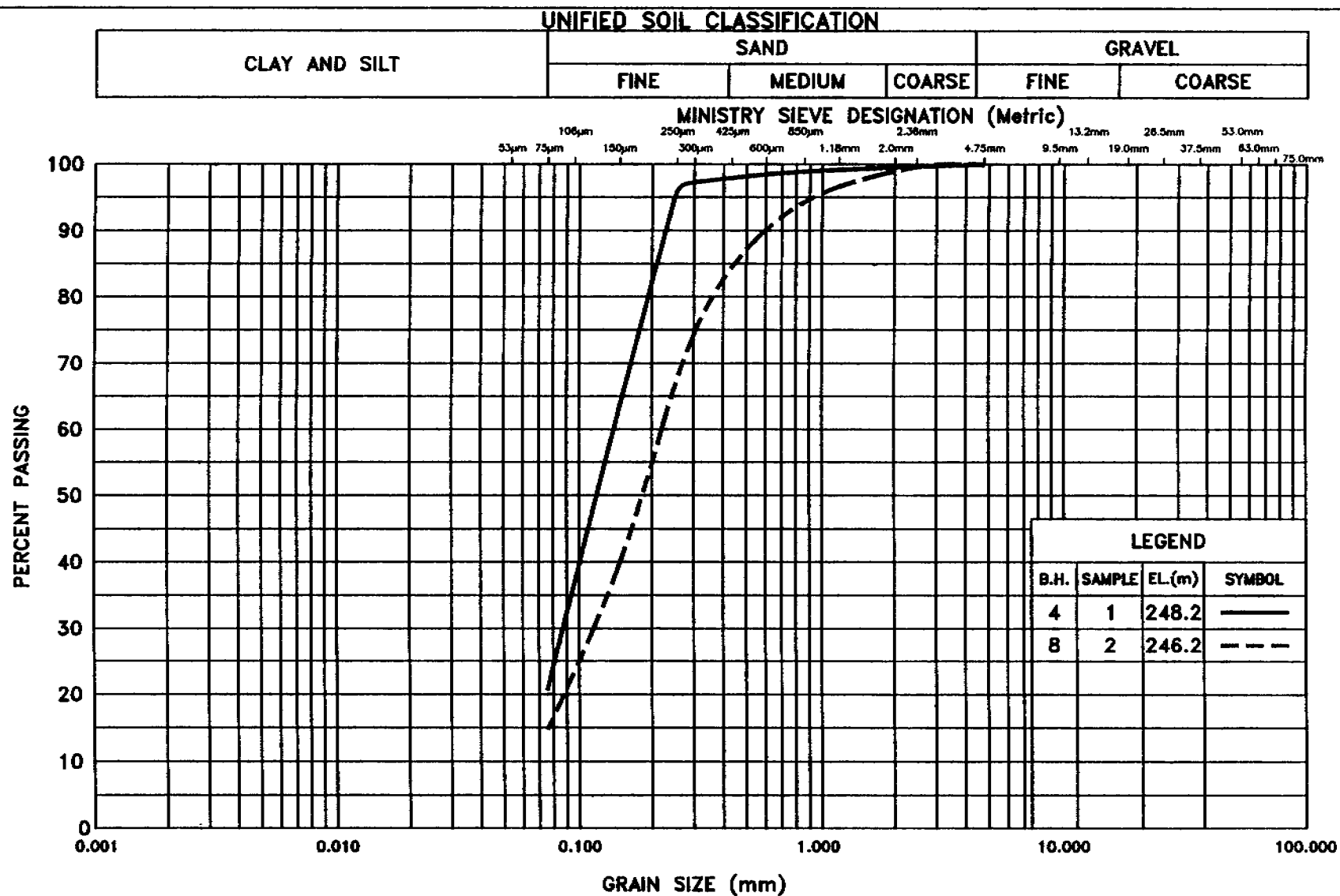
Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

Ministry of
Transportation

METRIC

GRAIN SIZE DISTRIBUTION

BH-4, SAMPLE 1 : SILTY SAND

BH-8, SAMPLE 2 : SAND some silt

DRAWING No. 3

W.P. 217-89-00

RECORD OF BOREHOLE BH-1

1 OF 1

METRIC

W.P. 9750-7813-5270

LOCATION Station 10+019, offset 1 m left of centreline.

ORIGINATED BY S.M.

DIST 52 HWY

BOREHOLE TYPE Wash Boring / Rock Core

COMPILED BY M.D.

DATUM Geodetic

DATE November 22, 1997

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST						
						20	40	60	80					
245.32	WATER SURFACE	F												
0.00	TOPSOIL, ~50 mm over FILL, sand & gravel ~250 mm thick then BOULDERY ROCK FILL, some sand & gravel matrix, grey, (dense)	F												
244.10		F												
1.22	BEDROCK, Biotite Hornblende Gneiss, Unweathered to Slightly Weathered													
			1	BQ										Rec 100% RQD 98%
			2	BQ										Rec 100% RQD 90%
			3	BQ										Rec 100% RQD 100%
241.15	END OF BOREHOLE													
4.17	Note: Standard split spoon attempted at 0.75 m depth in bouldery fill. Zero penetration (bouncing refusal) after 60 blows. Unable to retrieve sample from bouldery fill.													



RECORD OF BOREHOLE BH-2

1 OF 1

METRIC

W.P. 9750-7813-5270

LOCATION Station 10+021, offset 2 m left of centreline.

ORIGINATED BY S.M.

DIST 52 HWY

BOREHOLE TYPE Hollow Stem Augers /

COMPILED BY M.D.

DATUM Geodetic

DATE November 22, 1997

CHECKED BY I.G.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE			CONE PENETRATION TEST								
						20	40	60	80						
245.16	WATER SURFACE														
0.00	FILL, sand, gravel and cobbles, brown. (dense)	TI				245									
244.55															
0.61	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



RECORD OF BOREHOLE BH-3

1 OF 1

METRIC

W.P. 9750-7813-5270 LOCATION Station 10+033, offset ~1 m right of centreline. ORIGINATED BY S.M.
 DIST 52 HWY BOREHOLE TYPE Standard Augers / COMPILED BY M.D.
 DATUM Geodetic DATE November 22, 1997 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. / DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST									
						20	40	60	80	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)			
										UNCONFINED QUICK TRIAXIAL				FIELD VANE LAB VANE			
										20 40 60 80				10 20 30 40			
246.54	WATER SURFACE																
0.00	TOPSOIL ~150 mm over SAND, GRAVEL & COBBLES, brown, moist, (dense)																
245.88	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
0.66	Notes: 1) Drill moved ~1 m east of BH-3 & met auger refusal at ~0.7 m depth. 2) Bedrock outcrop ~10 m north of borehole.																



RECORD OF BOREHOLE BH-4

1 OF 1

METRIC

W.P. 9750-7813-5270

LOCATION Station 10+044, offset 1 m right of centreline.

ORIGINATED BY S.M.

DIST 52 HWY

BOREHOLE TYPE Standard Augers /

COMPILED BY M.D.

DATUM Geodetic

DATE November 24, 1997

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE			20	40	60	80	wp	w	wl		
248.84	WATER SURFACE														
0.00	TOPSOIL, 75 mm over SILTY SAND, some layering, brown, moist, occasional gravel sizes in upper 1 m depth. (compact to dense)		1	SS	18										0 79 21 0
			2	SS	33										
	Gravel inclusions at base.														
246.48															
2.36	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



1 OF 1

METRIC

W.P. 9750-7813-5270

LOCATION Station 9+983, offset ~4 m left of centreline.

ORIGINATED BY S.M.

DIST 52 HWY

BOREHOLE TYPE Wash Boring / Rock Coring

COMPILED BY M.D.

DATUM Geodetic

DATE November 20, 1997

CHECKED BY J.G.

SOIL PROFILE						SAMPLES
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPED	BLOWS/0.3m	GROUND WATER CONDITIONS	
244.05 0.00	WATER SURFACE					
243.14 0.91	BOULDERY ROCK FILL, some sand & gravel matrix, grey. (dense)	F 				
241.51 2.54	BEDROCK, Biotite Hornblende Gneiss, unweathered to slightly weathered.	Hatched pattern	1 BQ			
238.70 5.35	MARBLE, unweathered.	Dotted pattern	2 BQ			
238.28 5.77	END OF BOREHOLE		3 BQ			
	Note: Standard split spoon attempted at 1.2 & ~2.2 m depths in bouldery fill. Zero penetration (bouncing refusal) after 60 blows. Unable to retrieve samples of bouldery fill.		4 BQ			



RECORD OF BOREHOLE BH-6

1 OF 1

METRIC

W.P. 9750-7813-5270

LOCATION Station 9+981, offset 4 m left of centreline.

ORIGINATED BY S.M.

DIST 52 HWY

BOREHOLE TYPE Hollow Stem Augers /

COMPILED BY M.D.

DATUM Geodetic

DATE November 22, 1997

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT				NATURAL MOISTURE CONTENT				LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl	10	20	30	40	kn/m ³	GR	SA	SI		
244.05 0.00	WATER SURFACE																							
243.14 0.91	WATER																							
242.81 1.24	BOULDERY ROCK FILL (dense)																							
	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BOULDER																							



RECORD OF BOREHOLE BH-7

1 OF 1

METRIC

W.P. 9750-7813-5270

LOCATION Station 9 + 966, offset 2 m left of centreline.

ORIGINATED BY S.M.

DIST 52 HWY

BOREHOLE TYPE Hand Power Auger /

COMPILED BY M.D.

DATUM Geodetic

DATE November 24, 1997

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST						
						20	40	60	80					
246.56	WATER SURFACE													
0.00	TOPSOIL, 150 mm over SAND, some silt with gravel & occasional cobble sizes, brown, moist. (compact)													
			1	AS										
244.96														
1.60	END OF BOREHOLE DUE TO REFUSAL TO HAND POWER AUGER ON BEDROCK OR BOULDER													




RECORD OF BOREHOLE BH-8

1 OF 1

METRIC

W.P. 9750-7813-5270 LOCATION Station 9+954, offset 1 m left of centreline.
 DIST 52 HWY BOREHOLE TYPE Power Hand Auger /
 DATUM Geodetic DATE November 24, 1997

ORIGINATED BY S.M.
 COMPILED BY M.D.
 CHECKED BY I.G.

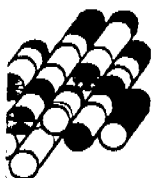
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) 				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE			20	40	60	80	wp	w	wl		
247.65	WATER SURFACE													
0.00	TOPSOIL, ~100 mm over SAND, some silt, odd gravel sizes with occasional cobbles, brown, moist. (compact)													
		1	SS	10										
		2	SS	13										
245.47	END OF BOREHOLE DUE TO REFUSAL TO HAND POWER AUGER ON BEDROCK OR BOULDER													
2.18														



SO7455G		TABLE 1 ROCK CORE DESCRIPTION				
BH #	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
1	1	1.22 to 2.74	100	98	1.22 to 4.17	Biotite-Hornblende Gneiss , greyish black to dark grey, some brownish metasedimentary inclusions, medium to coarse grained, strong, unweathered, fractures widely spaced, dipping at 50° from vertical, planar, smooth
	2	2.74 to 3.57	100	90		
	3	3.57 to 4.17	100	100		
2	1	2.54 to 3.66	100	75	2.54 to 5.35	Biotite-Hornblende Gneiss (Garnetiferous) , greyish black to very light grey, medium to coarse grained, strong, unweathered to slightly weathered, fractures moderate to very close spaced, dipping to near vertical, planar, smooth
	2	3.66 to 4.11	100	78		
	3	4.11 to 4.81	100	91		
	4	4.81 to 5.77	100	70		
					5.35 to 5.77	Marble , pinkish white to white, medium to very coarse grained, strong, unweathered, no fractures

*CR - Core Recovery

**RQD - Rock Quality Designation



Terraprobe

PROJECT No: 96 - 1063

CLIENT: Sutcliffe Limited

LOCATION: See Plan Figure

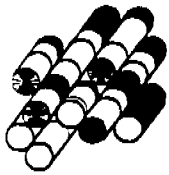
LOG OF BOREHOLE 1 Fig. 2

BORING DATE: January 21, 1997

ELEVATION DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE IN METRES	SOIL PROFILE		SAMPLES			SPT Value	STATIC CONE PLOT	WATER CONTENT (%)	INSTALLATION INFORMATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	"N" VALUE	20 40 60 80		
							SHEAR STRENGTH kPa		
0	GROUND SURFACE		248.70						
	ROCK FILL		0.0						
1									
2									
3									
4									
5									
6									
7									
8	Assumed Bedrock Contact		241.3						
			7.4						
8	END OF BOREHOLE		240.7						
			8.0						
9	NOTES:								
	1) Borehole advanced by air track equipment on January 21, 1997.								



Terraprobe

PROJECT No: 96 - 1063

CLIENT: Sutcliffe Limited

LOCATION: See Plan Figure 1

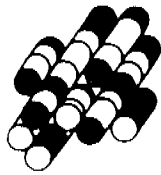
LOG OF BOREHOLE 3 Fig. 4

BORING DATE: January 21, 1997

ELEVATION DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD DEPTH SCALE IN METRES	SOIL PROFILE		SAMPLES			SPT Value STATIC CONE PLOT		WATER CONTENT (%)	INSTALLATION INFORMATION			
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	N" VALUE	SHEAR STRENGTH kPa					
							SHEAR STRENGTH kPa					
							SHEAR STRENGTH kPa					
0	GROUND SURFACE ROCK FILL		247.85 0.0									
1												
2												
3												
4	Assumed Bedrock Contact		244.1 3.8									
5												
6												
7	END OF BOREHOLE		241.1 6.8									
8	NOTES: 1) Borehole advanced by air track equipment on January 21, 1997.											
9												



Terraprobe

*Consulting Geotechnical Engineers & Hydrogeologists
Construction & Materials Inspection & Testing*

**PRELIMINARY GEOTECHNICAL INVESTIGATION
MAPLE ISLAND BRIDGE, MTO SITE # 44-057
MACKENZIE TOWNSHIP
DISTRICT OF PARRY SOUND , ONTARIO**

Prepared For: Sutcliffe Limited
9 Wellington Street, P.O. Box 1208
New Liskeard, Ontario, P0J 1P0

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FILE NO. 96-1063

Date: MARCH 3, 1997

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March 3, 1997

File No: 96-1063

Sutcliffe Limited
9 Wellington St.
P.O. Box 1208
New Liskeard, Ontario P0J 1P0

Attention: Mr. El Amin

RE: Preliminary Geotechnical Investigation
Maple Island Bridge MTO Site #44-057
MacKenzie Township, District of Parry Sound, Ontario

Dear Sir:

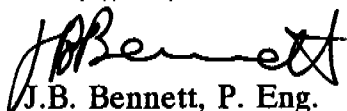
Terraprobe Limited were retained and authorized by Mr. El Amin, of Sutcliffe Limited to undertake a subsurface investigation for a proposed Maple Island Bridge replacement which is located on the local Maple Island Road, where it crosses the Magnetewan River, approximately 2 km North of Hwy 520 in the Township of MacKenzie, District of Parry Sound, Ontario.

The purpose of this preliminary investigation was to carry out a geotechnical investigation, in accordance with the guidelines of the Ministry of Transportation and the Ontario Bridge Design Code. The purpose of the investigation was to determine the subsurface conditions at the site, and to provide preliminary geotechnical engineering recommendations for the design and construction to replace the existing bridge.

The report is provided in Two parts; Part 'A' Foundation Investigation
Part 'B' Foundation Design

We trust the following report is satisfactory. If there are any questions please do not hesitate to call this office.

Yours truly,
TERRAPROBE Ltd.



J.B. Bennett, P. Eng.
Manager-Sudbury, Associate

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PART 'A'
FOUNDATION INVESTIGATION REPORT

Preliminary Geotechnical Investigation
North Maple Island Bridge
MTO Site #44-057
Mackenzie Township District of
Parry Sound, Ontario

A.1 SITE AND PROJECT DESCRIPTION

The existing bridge was apparently constructed in the 1930's and is located approximately 2 km north of Hwy 520 on the local road that crosses the Magnetewan River between the communities of Whitestone and Dunchurch in the Township of MacKenzie, in the District of Parry Sound, Ontario. The river is approximately 23m wide at the bridge crossing. The slopes of the creek vary dramatically, from less than 3:1 to as steep as 1:1. There is exposed bedrock at the bridge location in the river bed. At the time of the fieldwork the depth of water was approximately 2.1m. The elevation of the water level of the river at the time of the fieldwork was 244.94m as recorded by Sutcliffe Limited. The creek bed was near elevation 242.8m at the mid point of the river.

The existing bridge is a single span structure. The width of the bridge allows a single lane of traffic. The sides of the bridge have steel trusses and safety rails. The approaches to the bridge extend into the river coarse and appear to be made up of rock fill.

We understand that there are several options that are being presented to allow the staged construction of the new bridge. One of the options allows for a temporary re-alignment 10 m to the north, with the installation of a temporary Bailey bridge.

A. 2 FIELD WORK

The field investigation for the project was conducted on January 21, 1997. Four(4) exploratory boreholes (boreholes BH1, BH2, BH3 and BH4) were attempted to be advanced at the locations of the existing bridge abutments and at an offset to the north. Four additional boreholes (boreholes PHA, PHB, PHC and PHD) were to be advanced to shallow depths to determine the pavement profile in the approach areas.

The presence of rock fill was anticipated at two boreholes to the north of the existing alignment. Therefore we anticipated drilling these holes with air track equipment, that would penetrate the rock fill and the bedrock. The contact between rock fill and bedrock would be determined empirically by noting the changes in drilling energy.

Sand fill was expected at the two bore holes at the existing abutments. However, initial attempts at the BH2 location with conventional soil drilling equipment refused on assumed rock fill near the ground surface. The air track equipment was then set up at BH2 and used to penetrate the rock fill and stop at or near the contact of bedrock so that diamond coring equipment could be used to retrieve a core of the bedrock. However, the air track equipment was not able to remove the cuttings. The field crew then dropped casing into the hole but it was not possible to advance the casing by flushing the hole or using bi-cone equipment. The casing was removed and it was determined that the efforts had worn the casing shoe. Borehole BH4 location was

advanced to the bedrock contact using the air track equipment, with the intention to move onto BH4 with the diamond drill equipment the morning of January 22, 1997.

However, a severe ice storm made conditions hazardous the next morning and the equipment was demobilized.

The soil samples from boreholes PHA, PHB, PHC, and PHD were retrieved using the auger equipment. All samples obtained in the investigation were sealed into bags, and transported to our laboratory for detailed inspection and testing. All of the borehole samples were examined (tactile) in detail by the project engineer, and classified according to visual and index properties.

The field work was supervised throughout by a member of our technical staff, who directed the drilling and sampling operations, and transported the samples to our laboratory.

The locations and elevations of the borings were determined by Terraprobe in the field. The elevations of the borings were determined relative to a local temporary benchmark established by Sutcliffe personnel and located on top of an iron bolt in the bridge deck, at the south east corner. The bench mark is understood to have a geodetic elevation of 249.064 m.

A.3 SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered at the site are summarized below, and are also presented on the accompanying Borehole Logs, shown as Figures 2 to 5 and the shallow pavement boreholes are presented in the Ministry format in Appendix 'A'.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary at other locations, particularly with respect to depth and condition of the rock fill and bedrock surface. The boundaries between the various strata as shown on the logs and sections are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

A3.1 Deep Bore Holes

In summary, the deep borings BH2 and BH4 were located on the roadway behind the abutments. Borings BH1 and BH3 were located on the rock fill embankments to the north of the existing alignment, near the location of proposed widening of abutments and proposed detour structure.

The borings BH2 and BH4 penetrated road base granular overlying the rock fill which was encountered down to the assumed bedrock contact. Borings BH1 and BH3 found rock fill down to the bedrock contact. The contact elevations are summarized on Table 1.

TABLE 1
Assumed Bedrock Contacts

Borehole Location	Elevation of Ground Surface (m)	Depth to Assumed Rock Fill/Bedrock Contact (m)	Elevation of Contact (m) [Drill to]
1	248.70	7.37	241.33 [240.73]
2	250.08	5.77	244.31 [244.2]
3	247.85	3.76	244.09 [241.1]
4	250.08	5.87	244.21 [244.1]

The river bed was surveyed to have an elevation of approximately 242.8 m.

Interviews with two local residents indicated the following:

- 1) Residents in the area reported that in the summer, the river level drops to expose relatively flat bedrock in the river bed.
- 2) Also, an older resident (Mr. George Butler) recalls the actual construction of the bridge. He stated that the bedrock was at a shallow depth in the river with a local drop in the north west corner.

Due to the presence of deep rock fill, that exists at the site, the drilling conditions are extremely difficult and must be taken into account if further geotechnical investigation is to proceed.

A.3.2 Shallow Bore Holes

The shallow borehole results are found in Appendix 'A'. The field findings indicated that there was approximately 150 mm of a Granular 'A' type material on the road surface.

The road is a gravel road surface. Borehole PHB encountered refusal at 750 mm on possible rock fill. The remaining holes encountered sandy soils to penetration depths of 1.5 to 1.8m. A clayey seam was found in PHA at 1.2 m.

A.3.3 Groundwater

The soils below the roadway approaches appear to consist of rock fill and silt sands. The water level is expected to follow the river level seasonal fluctuation with a slightly higher level in the silty sand material.

PART 'B'
FOUNDATION DESIGN REPORT

Preliminary Geotechnical Investigation
North Maple Island
MacKenzie Township
District of Parry Sound, Ontario

The following discussions and recommendations are based on the factual data obtained from the investigation, and are presented for guidance of the design engineer only. The comments pertain to a specific project and location of the proposed Maple Island Bridge replacement. If significant parts of the project change, they should be reviewed by Terraprobe to determine the affect of the changes on the recommendations.

Contractors bidding on or conducting work associated with this project should review the factual data presented in the preceding sections of the report, to assess their effect on proposed construction methods and scheduling.

B.1 FOUNDATIONS

B.1.1 Proposed New Bridge

We have been informed by the design team that it is intended that the new structure consist of a single span structure of approximately 30m in length. Based upon the proposed design, the topography of the site, and the soil conditions encountered, the following comments are provided.

The preliminary information regarding bedrock levels that were found in the bore holes, suggests that the proposed structures may bear their foundations directly on bedrock with the exception of BH1 area where the bedrock appears to be 3 m lower.

The bedrock may be considered a non-yielding stratum (therefore no serviceability limit state required). The ultimate limit state, without load tests, for the factored bearing resistance may be taken as 10 Mpa for the bedrock.

The footings bearing on bedrock should not require protection from frost or erosion. The coefficient of friction of the foundation materials may be taken as 30 degrees (unfactored), for assessment of sliding resistance of the abutment footings where there is contact between the concrete and bedrock.

Due to the sloping bedrock conditions near the north west area, the foundation options become slightly more complex. The foundation choices include excavating the entire abutment foundation width down to a constant level and constructing the spread footing at this level or alternatively filling with a concrete fill. A third option would be to support the end of the abutment on short piles or caissons or have a combination of spread footing design on the south portion and pile/caisson support on the north side.

In order to bear entirely on a relatively flat bearing surface on bedrock, an excavation to below elevation 241m would be required or an alternative is to place a mass concrete pad to fill up to the higher rock level. Cofferdams would be required. The second alternative would be to support the south portion of the structure on

bedrock near elevation 244m and support the north half on driven piles or caissons founded down to the bedrock. Pile or caisson foundations would require penetration into the bedrock and the use of pile caps and grade beams which must have an underside below potential frost penetration unless clean rock fill is used, in which case frost is not an issue. Frost protection depths of soil cover should be at least 2.4m for this site.

Piles or caissons must also be protected from ice forces. The MTO structural manual however does not recommend pile lengths to be less than 5m. This option may require further discussion with MTO personnel. If piles remain an option, special pile shoes must be used to aid the setting of pile toe on the sloping bedrock. Alternatively, predrilling and casing of the soil overburden to allow a pilot hole to be drilled into the bedrock is also a potential solution. Removal of the existing rock fill material would also be required or predrill the rock fill and install casing to minimize potential for damage to the piles. The pile layout and capacities would be dictated by the steel pile cross-sections and yield strengths since the bedrock provides an unyielding stratum. The Ontario Highway Bridge Design Code(OHBDC) states that the stresses should not exceed 100MPa unless testing is carried out. For example, an HP310X79 steel pile would have a factored ultimate stress of 100MPa resulting in a load carrying capacity of approximately 75 Tonnes(75 imperial tons).

Drilled caissons foundations are an additional option that may provide an economical design for the abutments at the deeper bedrock. Again penetration through the rock fill with traditional augers would be extremely difficult. The base of the caisson should be set on flat bedrock or the bedrock must be benched or socketed. The lateral resistance of caissons set into bedrock would be based on shear resistance at the base of the caisson by installing shear dowels into the bedrock.

B.2 EARTH PRESSURES - ABUTMENT, WING AND RETAINING WALLS

Select granular fill such as OPSS Granular 'A' or Granular 'B', should be used as backfill behind the abutment, wing and retaining walls. The select granular fill should be placed in a wedge shaped zone extending from 1 m behind the rear toe of the walls and up at a 45 degree angle. The granular fill should be placed in thin lifts and compacted to a minimum of 95 per cent of Standard Proctor Maximum Dry Density (SPMDD). Heavy compaction equipment should not be used behind the wall within a lateral distance equal to the current height of fill above the wall footing, in order to minimize deflection or possible damage of the wall.

Provided the above backfill criteria are satisfied, a coefficient of active earth pressure $K_a = 0.3$ may be used in estimating maximum lateral earth pressures. It should be noted that the mobilization of the active earth pressure behind the wall will require an outward deflection of up to 0.5 per cent of the wall height. The effect of this deflection should be allowed for in the design of the wall and any adjacent or connected structures.

A drainage system should be provided behind the wall to prevent the build-up of hydrostatic forces. The drainage system should incorporate a properly designed filter to protect against clogging of any drainage pipes. The outlet of the drainage system should be protected against freezing to ensure proper functioning of the system in the winter months.

Provided the above backfill criteria are satisfied, the following soil properties and parameters may be used in calculation of lateral earth pressures, in accordance with the

OHBDC. It is noted that these are based on the assumption that the surface of backfill behind the wall is close to horizontal.

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Effective Angle of Internal Friction (Phi),degrees, unfactored	35	30
Unit Weight (Gamma), kN/m ³	22.8	21.2
Active Earth Pressure Coefficient, K _a (SLS)	0.27	0.33
(ULS)	0.36	0.41
At rest Earth Pressure Coefficient, K _o (SLS)	0.43	0.50
(ULS)	0.53	0.58

We would be pleased to provide more specific details for other design criteria, such as calculation of lateral loadings on the wall due to surcharge loadings, pending more specific details of the wall design and magnitude of surcharge loading.

B.3 EXCAVATIONS

It is anticipated that excavations for proposed foundations and underground services will extend to depths of about 2.5 to 5.5 m below existing grades. Based on the findings in the boreholes, it is anticipated that granular, rock fill and sand soils will be encountered with bedrock underlying much of the site.

Support of the sand soils with an engineer designed shoring system is anticipated due to the high water table.

The method of support may consist of sheet pile walls, soldier pile and lagging systems or other types of shoring systems. The walls should be designed by a qualified contractor and professional with experience with such systems.

B.4 SLOPE PROFILES - EMBANKMENTS

The native soils consist of rock fill and compact sandy soil. The existing slopes down to the creek range from moderate to steep.

The stability of slopes would have adequate factors of safety if maintained at an inclination of 3:1 (H:V) or flatter. Due to the road profile and levels of the creeks, wing walls and extended retaining walls will be required to confine and support the soils immediately behind the abutments. The embankment slopes may be increased to 2:1 but adequate vegetation coverage must be assured to prevent surface erosion or the use of rock fill considered. Surface erosion may occur prior to the vegetation taking root and maturing and measures should be provided to prevent this short term condition.

B.5 PAVEMENT DESIGN

The pavement subgrade is expected to consist of native materials or clean fill compacted to a minimum of 95 percent SPMDD. The following minimum pavement thickness may be used for design of the roadway and the temporary detour:

Granular Surface Coarse	150 mm OPSS Granular 'A'
Granular Subbase	530 mm OPSS Granular 'B'

For the bridge deck, the following asphalt thicknesses should be applied above the waterproofing:

Asphalt 40 mm HL 4
..... 40 mm HL 8

Organic (topsoil) or soft subsoils should be removed from beneath the proposed roadway areas prior to fill placement.

All common earth fill materials placed beneath the roadway should be compacted to a minimum of 95 percent of the Standard Proctor Maximum Dry Density in lifts not exceeding 150 mm. The granular subbase and base fill materials should be compacted to a minimum of 100 percent SPMDD in lifts of 150 mm or less. Asphalt materials should be rolled and compacted to a minimum of 97 percent Marshall Bulk Density (MBD).

Immediately prior to placement of the pavement granular courses, the subgrade should be proof rolled with a heavy rubber tired vehicle (such as a grader) and any loose, soft, or unstable areas should be sub-excavated and backfilled with compacted earth materials.

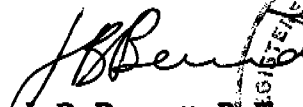
The above pavement design thicknesses are considered adequate for this local road. However, if pavement construction occurs in wet inclement weather it may be necessary to provide additional subgrade support for heavy construction traffic by increasing the thickness of the granular subbase or base. Further, main traffic access areas for construction equipment may experience unstable subgrade conditions. These areas may be stabilized utilizing additional thicknesses of granular materials.

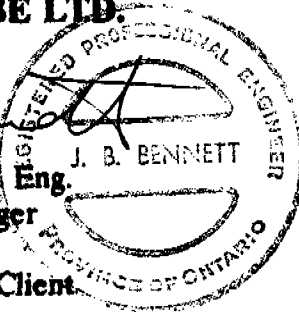
It is recommended that inspection and testing be carried out during construction to confirm material quality, thickness and to ensure adequate compaction.


B.5 CLOSURE

We trust the preceding report meets with your requirements. Should there be any questions please do not hesitate to contact Terraprobe's office.

TERRAPROBE LTD.


J. B. Bennett, P. Eng.
Associate-Manager
Distribution: 1. Client

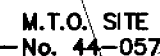



Kirk Johnson, P. Eng.
Associate






FIGURES





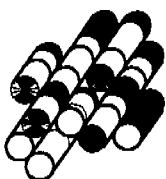
KEY PLAN
N.T.S.



-  Rock Fill material
 Silty Sand material
 Bedrock

REVISION DATE: 10/02/97

[illegible]



Terraprobe

PROJECT No: 96 - 1063

CLIENT: Sutcliffe Limited

LOCATION: See Plan Figure

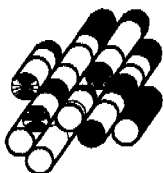
LOG OF BOREHOLE 1 Fig. 2

BORING DATE: January 21, 1997

ELEVATION DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD	DEPTH SCALE IN METRES	SOIL PROFILE		SAMPLES			SPT Value		WATER CONTENT (%)	INSTALLATION INFORMATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	"N" VALUE	STATIC CONE PLOT			
								20			40
	0	GROUND SURFACE		248.70							
		ROCK FILL		0.0							
	1										
	2										
	3										
	4										
	5										
	6										
	7										
		Assumed Bedrock Contact		241.3							
				7.4							
	8	END OF BOREHOLE		240.7							
				8.0							
	9	NOTES: 1) Borehole advanced by air track equipment on January 21, 1997.									



Terraprobe

PROJECT No: 96 - 1063

CLIENT: Sutcliffe Limited

LOCATION: See Plan Figure 1

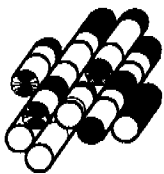
LOG OF BOREHOLE 2 Fig. 3

BORING DATE: January 21, 1997

ELEVATION DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD	DEPTH SCALE IN METRES	SOIL PROFILE		SAMPLES			SPT Value		WATER CONTENT (%)	INSTALLATION INFORMATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	"N" VALUE	STATIC CONE PLOT		
								X		
	0	GROUND SURFACE		250.08						
		ROCK FILL		0.0						
	1									
	2									
	3									
	4									
	5									
	6	Assumed Bedrock Contact at 244.3		244.2						
		END OF BOREHOLE		5.9						
		NOTES:								
		1) Borehole advanced by air track equipment on January 21, 1997.								
	7									
	8									
	9									



Terraprobe

PROJECT No: 96 - 1063

CLIENT: Sutcliffe Limited

LOCATION: See Plan Figure 1

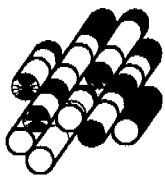
LOG OF BOREHOLE 3 Fig. 4

BORING DATE: January 21, 1997

ELEVATION DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD DEPTH SCALE IN METRES	SOIL PROFILE		SAMPLES			SPT Value STATIC CONE PLOT -				WATER CONTENT (%)	INSTALLATION INFORMATION	
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	"N" VALUE	SHEAR STRENGTH kPa					
							20 40 60 80					
0	GROUND SURFACE ROCK FILL		247.85 0.0									
1												
2												
3												
4	Assumed Bedrock Contact		244.1 3.8									
5												
6												
7	END OF BOREHOLE		241.1 6.8									
8	NOTES: 1) Borehole advanced by air track equipment on January 21, 1997.											
9												



Terraprobe

PROJECT No: 96 - 1063

CLIENT: Sutcliffe Limited

LOCATION: See Plan Figure 1

LOG OF BOREHOLE 4 Fig. 5

BORING DATE: January 21, 1997

ELEVATION DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING METHOD	DEPTH SCALE IN METRES	SOIL PROFILE		SAMPLES			SPT Value				WATER CONTENT (%)	INSTALLATION INFORMATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	"N" VALUE	STATIC CONE PLOT					
								20	40	60			80
								SHEAR STRENGTH kPa					
		GROUND SURFACE		250.08									
		ROCK FILL		0.0									
	1												
	2												
	3												
	4												
	5												
	6	Assumed Bedrock Contact at 244.2		244.1									
		END OF BOREHOLE		6.0									
	7	NOTES: 1) Borehole advanced by air track equipment on January 21, 1997.											
	8												
	9												

NORTH MAPLE ISLAND ROAD
TOWNSHIP OF MACKENZIE

STA 9 + 965 1.9m Lt of C/L PH A

0 - 760 Sa & Gr, Br, Moist to Wet, Med to Co
760 - 1.20 Sa, Br, Moist, F
 NOT Accep GRAN "B" TYPE I
 69.4% PASSING 1.18mm
 Accep SSM
1.20- 1.80 Si(y) Sa, Br, Wet

STA 9 + 980 1.5m Lt of C/L PH B

0 - 760 Sa & Gr, Br, Moist to Wet, Med to Co
 - 760 NFP RF

STA 10 + 022 1.3m Lt of C/L PH C

0 - 175 Sa & Gr, Br, Moist to Wet, Med to Co
175 - 1.80 Sa, Br, Moist, F

STA 10 + 037 1.8m Lt of C/L PH D

0 - 175 Sa & Gr, Br, Moist to Wet, Med to Co
 NOT Accep GRAN A
 97.7% PASSING 13.2mm
 NOT Accep GRAN B TYPE I
 11.9% PASSING 75µm
 Accep SSM
175 - 850 Sa, Br, Moist, F
850 - 1.80 Sa & Sa W Tr of Cl, Br, Wet

**APPENDIX 'A'
MTO Memorandum & Response**



**NORTH MAPLE ISLAND BRIDGE
PRELIMINARY DESIGN REPORT
REPORT PRESENTATION MEETING
MINUTES**

1

The meeting was held March 13, 1997 in the Duchesnay Board Room, North Bay, commencing at 10:15 a.m.

Attending were:

Gerry Chaput	- Assistant District Engineer, District 52, Huntsville
Bruce Westerberg	- Senior Municipal Supervisor, District 52, Huntsville
Ibrahim ElAmin	- H. Sutcliffe Limited
Dalton Farrow	- H. Sutcliffe Limited.

The following items were discussed:

1. *Hydrology Report* - has been forwarded for review by the Ministry but a reply has not yet been received.
2. *Foundation Report* - A reply has been received from Ken Ahmad, Pavements and Foundation Section and forwarded to Terraprobe for response.

His comments were reviewed as follows:

- Sutcliffe was approved.*
1. Alignment not clear - this has been addressed in the Preliminary Design Report. A ? preferred alternative has been selected.
 2. Piles through rock fill - this is one option - Terraprobe to respond. ←
 3. Bedrock elevations - Gerry asked if additional boreholes will be required. I. ElAmin replied that there is sufficient information for design but not for rock quantities. Gerry requested a statement be put in the report recommending additional boreholes at the detail design stage. ←
 4. Lateral support for piles - Ibrahim advised there are several methods. These will be decided during detail design.
 5. Pile capacity units (kN) - to be revised. ←
 6. Rock fill support - the abutment foundation will be supported on rock or concrete dowelled to rock.

**NORTH MAPLE ISLAND BRIDGE
PRELIMINARY DESIGN REPORT
REPORT PRESENTATION MEETING
MINUTES**

2

- .7 Rock fill slopes - agreed with 1.25:1 slopes.

Additional geotechnical items discussed at the meeting:

- .8 Pg. 10 of Report - metric and imperial loads should be different.
- .9 Pg. 14 - can both lifts be HL4? This will reduce the number of design mixes, costs, etc.
- .10 Pg. 13 - 3:1 slopes - seems flat especially for rock fill.
- Is 530 mm of Granular "B" needed? If so, clarify (this is a low volume road). What type of Granular "B"? Is this a factor in recommended depth?

All of the above items will require a response from the Geotechnical subconsultant and a supplement to the report issued.

3. *Preliminary Design Report*

- .1 Add names, stamps, etc.
- .2 Insert a summary at the beginning with a general description, what the selected alternative is, public input (2 lane versus 1 lane).
- .3 Add a section for proposed geometrics in the body of the report.
- .4 Section 2.4 - change "no" to "minimal" commercial traffic.
- .5 Add Section 2.5 - Existing Guide Rail and End Treatment.
- .6 Add Section 2.6 - Entrances - address re: property negotiations.
- .7 Section 3.1 - Local Roads Board should read "McKenzie, East Burpee and Burton".
- .8 Section 3.2 - correct typo (starts with period).
- .9 Section 3.4 - comment re: affected properties and add cross reference to Appendix.

- .10 Section 3.5 - correct typo (5th). Refer to notice in Appendix.
Last paragraph on Pg. 4 - add that single lane alternative was discussed at the Public Meeting.
- .11 Pg. 5, General Consensus Section - H & V alignment - this should read that the alignment improvements should not "change" the speed limit.

All references such as "is to be" or "must be" - change to "should be".

Add that concerns of property owners will be addressed during property negotiations.
- .12 Section 4.1 - replace "centered on" with "based on".

Part 'ii' add reference to "new bridge on existing alignment".

Add paragraph at beginning of this section re: "culvert alternative was considered" and why rejected.

Paragraph 2 - discuss why south side alignment was ruled out.
- .13 Delete 4.3.1 and 4.3.2 headers and add brief description to alternative headers, perhaps add a sketch to illustrate each alternative.

Keep this section descriptive only and move "Advantages/Disadvantages" to an analysis section.
- .14 Add analysis section - refer to viability of all five alternatives - show factors considered in arriving at preferred options (cost, environment, property, utilities).
- .15 Section 4.4 - put this under analysis section as property impacts. Reduce the details (put in Appendix).
- .16 Add a section in the analysis discussing the impact on environment, and wetlands.
- .17 Section 4.5 - reduce details for cost breakdowns (move details to Appendix).

**NORTH MAPLE ISLAND BRIDGE
PRELIMINARY DESIGN REPORT
REPORT PRESENTATION MEETING
MINUTES**

4

- .18 Section 5.0 - add reasons for selection (based on cost, environmental defects, utilities, etc.
- .19 Section 5.5 - see above re: Granular "B" depth in Geotechnical Report.
- .20 Section 5.6 - this should be discussed in the analysis and alternative selection section as well.

A discussion of the public concerns recorded on Pg. 5 ensued and the following will be added to the analysis and recommendations:

H & V alignment - this will not be changed significantly, but the sight distance will be improved, increasing safety.

Public Access - a statement that "the Ministry does not have jurisdiction over adjacent land use. The selected alternative does not promote or include access that would facilitate camping at the NE side of the bridge".

Dry hydrant - the selected alternative will allow for this. The need, size, location, etc., will be addressed during detail design.

Parking - the approach road will be designed for safety. Provision for parking will not be increased or decreased. This issue can be addressed with the appropriate signing.

Two Lane Design - this is addressed in the report. It is feasible, but not warranted. The Ministry does not object, but the additional costs would be borne by the local taxpayers.

Property Owners - their concerns will be addressed during property negotiations by MTO Property Section.

4. *Drawings*

Barrier Walls - Ibrahim to discuss with Per Furst to see if the standard conforms to MTO requirements.

Dry Fire Hydrant - delete from drawings.

5. *Design Criteria* - delete "preliminary" from title and include single lane version only.


Gerry to provide digital file with improved Trillium logo.

**NORTH MAPLE ISLAND BRIDGE
PRELIMINARY DESIGN REPORT
REPORT PRESENTATION MEETING
MINUTES**

5

6. *Billing* - all billing to be to Bruce by the end of March. The final report delivery may be delayed slightly if a response is not received regarding the Hydrology Report.

Adjourned at 12:30 p.m.


Dalton P. Farrow.

c.c. - Attendees;
- M.T.O., - Per Furst.

fn: mto96082reportpresen mtg3-13



memorandum

To: Gerry Chaput, P. Eng.
Assistant District Engineer
Huntsville District

From: Pavements and Foundation Section
Room 315, Central Building
Downsview, Ontario

Re: North Maple Island Foundation Investigation
Maple Island Bridge, Site # 44-057
Mackenzie Township
District of Parry Sound, Ontario

1997 03 11

This is in response to your memo of March 7, 1997 that we received on March 10, 1997. Since you needed our comments for your meeting on March 13, 1997, we briefly reviewed the Foundation portion of the Preliminary Geotechnical Investigation Report of the above project produced by Terraprobe Ltd. Our comments are as follows:

It is not clear from the report if the new structure will be at the same alignment as the existing structure.

The pile and caisson options for the new structure (page 10) do not appear to be feasible due to the following reasons.

How the piles will be advanced through the rockfill?

The bedrock elevations are approximate. How would we know if the piles are sitting on the bedrock or the rockfill?

How the piles will be laterally supported?

The pile capacity should be in kN (load) not in MPa (pressure). Also, the capacity seems to be low for piles on bedrock.

It is not known what type of foundation is supporting the existing structure? Can the new bridge be supported on the rockfill?

The side slope 3H:1V (Page 13) seems to be conservative. For rockfill up to 6m high the side slopes can be maintained at 1.5H:1V.

The borehole plan is very small and not suitable for contract package. Normally the full size plans are in 1:200 scale and the half size in 1:400.

I hope our comments are helpful. If you have any other questions please call.

K. Ahmad, P. Eng.
Foundation Engineer

cc:

P. Furt
T. Kim



memorandum

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Assistant District Engineer
Huntsville District

From: Pavements and Foundation Section
Room 315, Central Building
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The borehole plan is very small and not suitable for contract package. Normally the full size plans are in 1:200 scale and the half size in 1:400.

I hope our comments are helpful. If you have any other questions please call.

A handwritten signature in cursive script, appearing to read 'K. Ahmad', with a horizontal line underneath.

K. Ahmad, P. Eng.
Foundation Engineer

cc:

P. Furst
T. Kim

