

DOCUMENT NO. _____ DATE OF RECEPTION _____

GEOCRES No. 316-55

DIST 8 REGION Eastern

W.P. No. 827-73-02

CONT. No. 77-76

W. O. No. _____

STR. SITE No. 15-120

HWY. No. 43

LOCATION Potter's Bridge (Lindale Creek)

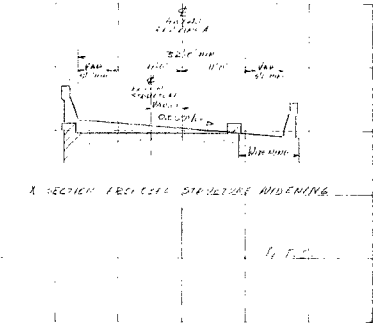
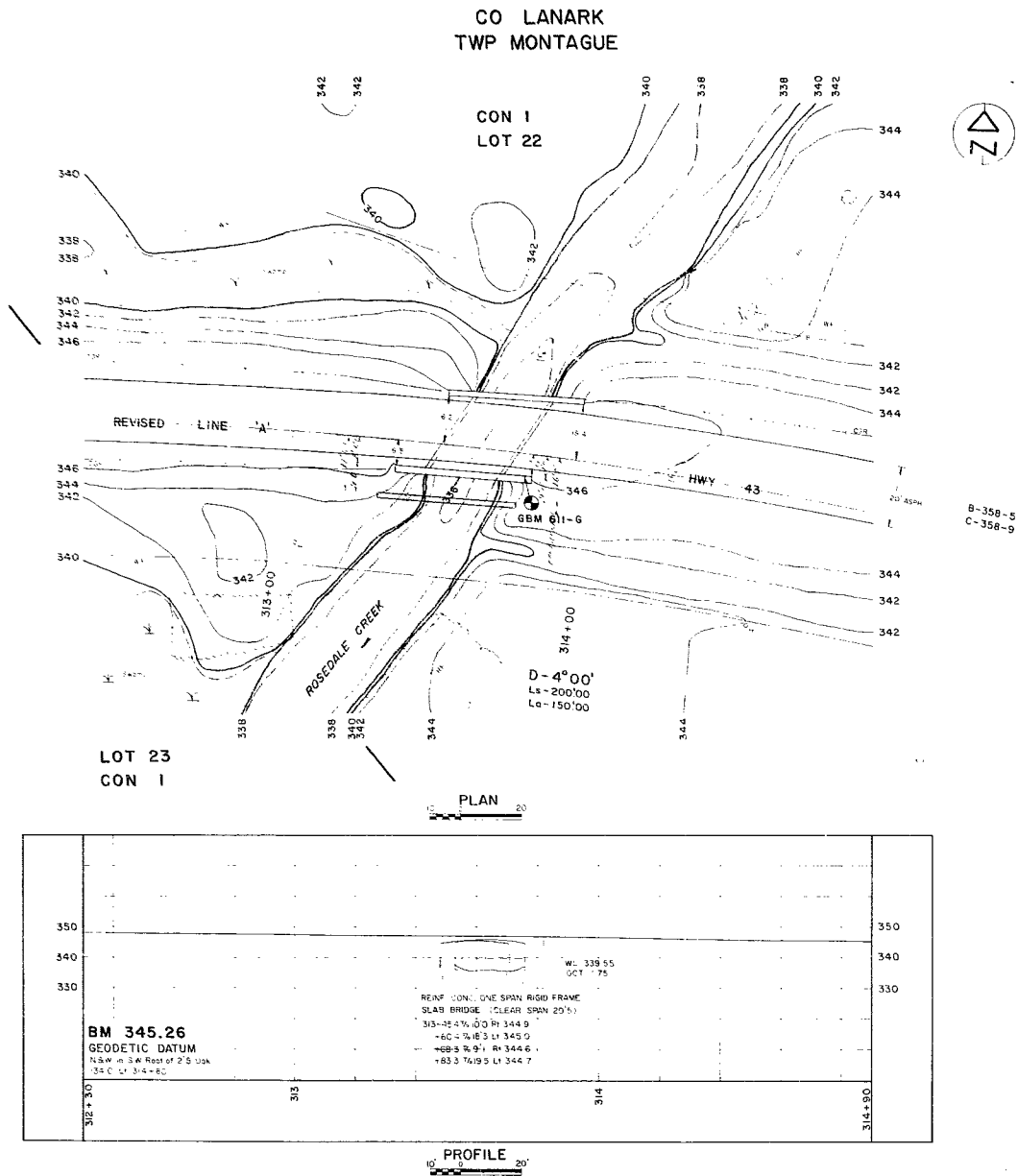
OVERSEER'S SIGNATURE TO BE INDICATED IN THE SPACE 4

REMARKS: _____

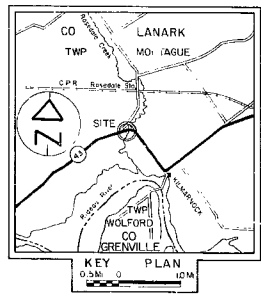
14.55.42

E-2528-1

38-1



B-358-5
C-358-9



GBM 611-6 EL 347.199
Concrete Bridge over Creek, 1.0 mile W.W. of top
on 1-WY 43. Tablet set in top of S. End Wall, 2.0 feet
from E. End, 0.3 feet from inner edge
6' Rn 313+78 QUAD 44075 LINE 215

NEW LINE
W P 827-73-01
STR WP 827-73-02

DATE	REVISIONS & ADDITIONS	BY	CHK'D
MAR 1976	REVISED LINE 'A' ADDED	H.A. GILES	G.S. BLAIR
MAR 21/75	SKETCH SHOWING REVISED PROPOSAL FOR WIDENING		

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO
ENGINEERING SERVICES BRANCH — ENGINEERING PLANS OFFICE

BRIDGE SITE

PROPOSED CROSSING

AT

ROSEDALE CREEK

AND

KING'S HIGHWAY 43 LINE 'A'

LOT 22
TWP. MONTAGUE

CON 1
CO LANARK

SCALE AS SHOWN	DISTRICT KINGSTON	REGION EASTERN
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W.O. - 827-73-01	Date of Plan NOV/75	Survey OCT/75	SITE - 15-120
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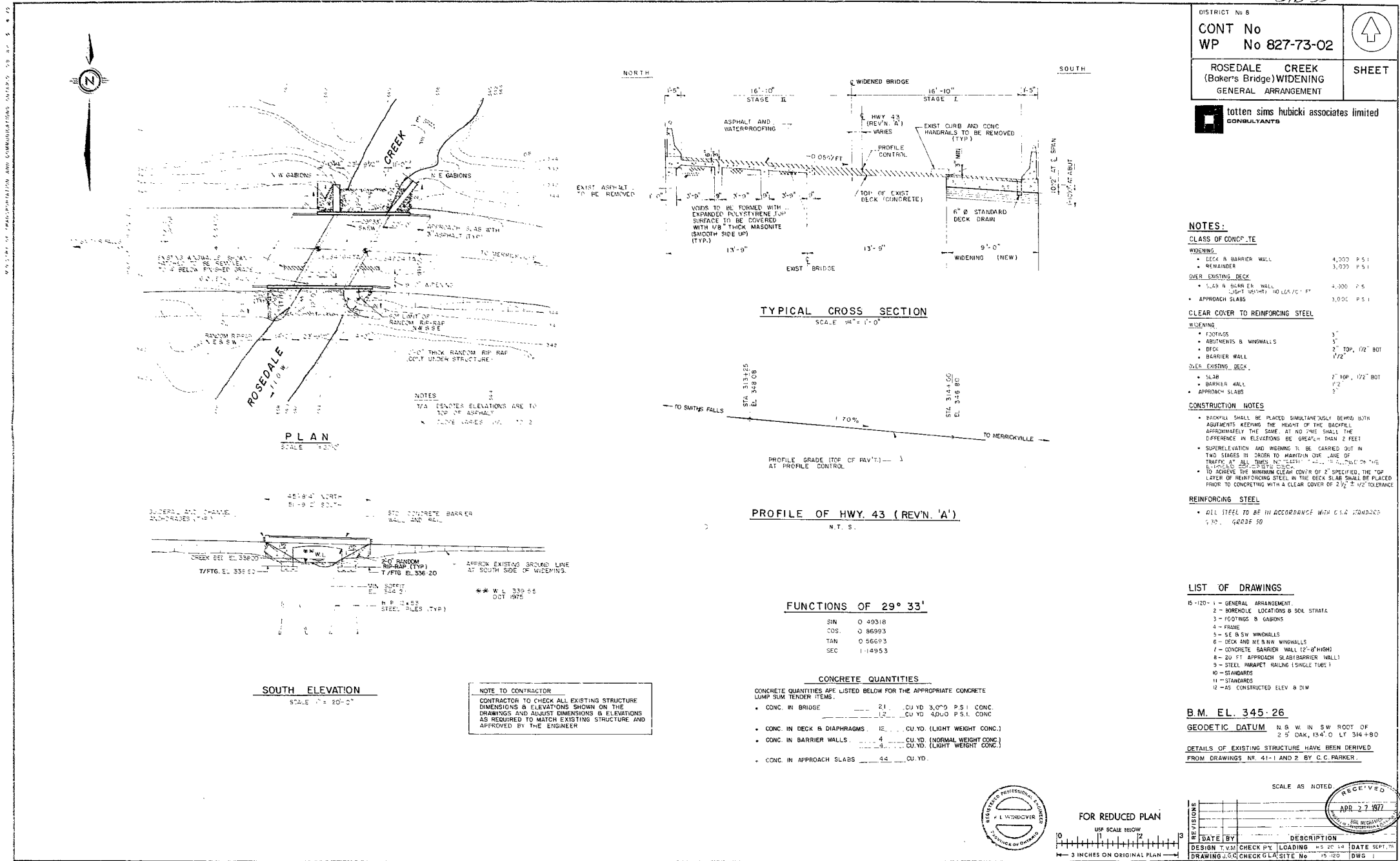
SURVEY BY Chief of Party - B. MADDEN Supervisor - A. GILES	DRAWN BY Draftsman - S. MOORE Supervisor - S. CAMILLERI
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CHECKED BY Draftsman - H. KIRKLAND Supervisor - S. CAMILLERI	PLAN E-5268-1
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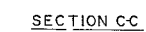
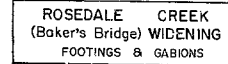
Soils Noted as per 11/75

E-2528-1

E-2528-1



CONT No
WP No.827-73-02



NOTES

- SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTINGS.
- PILES TO BE DRIVEN IN ACCORDANCE WITH STD. SS 3-II USING DESIGN LOAD OF 70 TON/PILE.

LIST OF PILES			
LOCATION	NO	LENGTH	TYPE
EAST ABUT	4	36'-0"	HP 12x53
WEST ABUT	4	36'-0"	HP 12x53



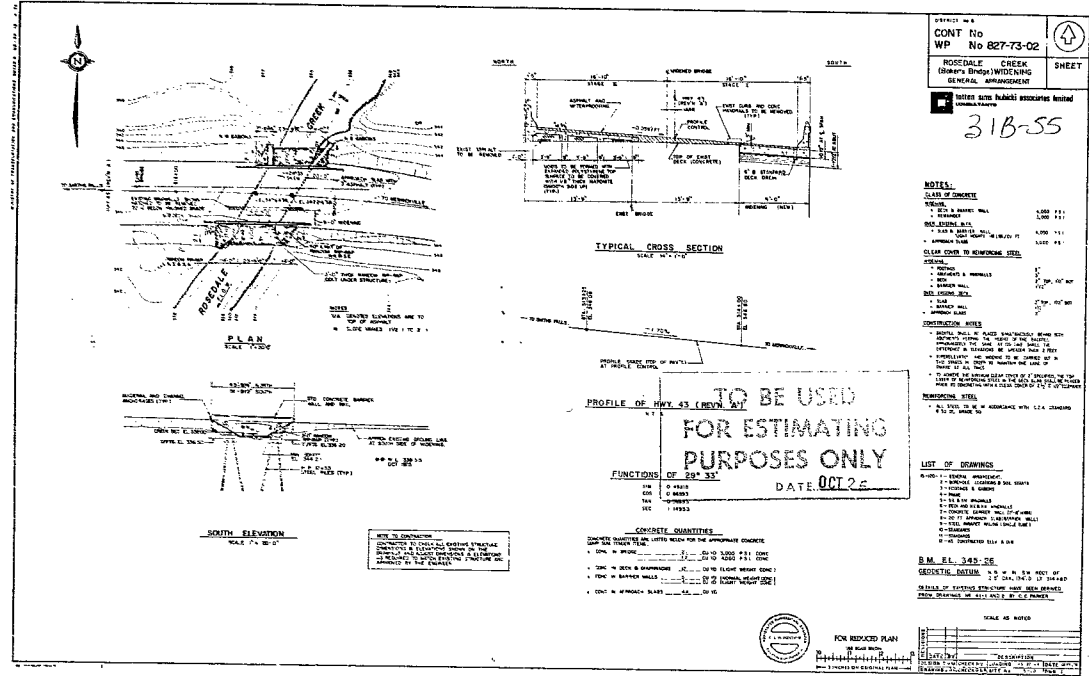
USE SCALE BELOW

0 1 2 3

3 INCHES ON ORIGINAL PLAN

RECEIVED
APR 27 1977
SOIL MECHANICS
DIVISION OF TRANSPORTATION & CONSTRUCTION

[illegible]



~~SECRET~~

REPORTS MISCLASSIFIED

BEWARE: Documents to be included

CLASSIFICATION TO BE INCLUDED WITH THIS REPORT

4

(Kosovo Crisis)

LOCATION BAKER BRIDGE

HWY. NO. 43

STB. SITE NO. 12-180

M. O. NO.

CONT. NO. 11-10

M. B. NO. 891-13-08

DIST. 8 REGION Eastern

GEOGRAPHIC NO. 318-22

01-70 2641 103

DOCUMENT MISCLASSIFICATION IDENTIFICATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: T.C. Kingsland (2)
Regional Structural Planning Eng.
Eastern Region Kingston

FROM: Soil Mechanics Section
Geotechnical Office
West Bldg.

ATTENTION:

DATE: April 21, 1976

OUR FILE REF.

IN REPLY TO

MAY 05 1976

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

W.P. 827-73-02 Site No. 15-120
Hwy. 43, District 8, Kingston
Widening of Bakers Bridge (Rosedale Creek)
5.7 Miles West of Rideau Canal Bridge
at Merrickville

Attached we are forwarding to you our detailed Foundation Investigation Report on the subsoil conditions existing at the above mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your requirements. Should additional information be required, please do not hesitate to contact our Office.



M. Devata
Supervising Engineer

MD/bp

CC: R.S. Pillar
C.S. Grebski
B.J. Giroux
G.A. Wrong
S. Radbone
E.R. Saint
V.A. Snell

R. Hore
J. Anderson)
R. Forest) Memo only
G. Sloan)

Files

FOUNDATION INVESTIGATION REPORT

For

W.P. 827-73-02 Site No. 15-120
Hwy. 43, District 8, Kingston
Widening of Bakers Bridge (Rosedale Creek)
5.7 Miles West of Rideau Canal Bridge
at Merrickville

1. INTRODUCTION

The Soil Mechanics Section was requested to investigate subsurface conditions for the proposed widening of the existing 24 ft. wide bridge at the above mentioned location. The original proposal was to widen the existing bridge on either side by some 8 ft. Taking this into account, an investigation was carried out. Subsequently, we have been advised, both by Structural Planning and Structural Design Offices, that a widening of approximately 9 ft. will be carried out on the south side only.

This report contains the results of the investigation and our recommendations pertaining to the design of foundations for the proposed structure widening, as well as the related approach embankments.

2. SITE AND GEOLOGY

The site is located 4 miles east of Smith Falls on Hwy. 43, Lot 22, Conc.1, Township of Montague, County of Lanark.

At the crossing, the meandering Rosedale Creek is approximately 24 ft. wide, and the water and ice depth together was found to be approximately 3.5 ft. Prior to the construction of the existing structure at this location, the old stream bed was situated some 280 ft. west of the present crossing. In order to facilitate the construction of the existing bridge in a relatively dry condition, a stream realignment was carried out to its present location.

The existing structure is a reinforced concrete single span rigid frame slab bridge with a clear span of 20.5 ft. and was built in 1947. Details of the

foundations for the existing structure are not known. However, from our communications with the local farmers, it is believed that the bridge foundations are supported on spread footings. The proposal for the existing structure called for the west and east abutments to be founded on spread footings with base elevations of about 333.7 and 334.0 respectively. However, the founding levels for the footings as constructed are not documented. The existing structure appears to be in a satisfactory condition.

It is understood from our discussions with the local MTC Patrolmen, that heaving of the pavement in the immediate area where the approaches meet the bridge was a problem, and thus, remedial measures were taken which consisted of installing weep holes in the abutment walls to relieve the buildup of excess hydrostatic pressure. This measure appears to have alleviated the problem.

The relief of the area in the vicinity of the site is generally flat. The surrounding area is utilized mainly for agricultural purposes.

Geologically, the area is part of the Smiths Falls Limestone Plain which is the largest and most unbroken tract of shallow soil over limestone in Southern Ontario. The exposed rock strata belong to the Beekmantown formation and include grey limestone, magnesium limestone, blue-grey dolomite and some calcareous sandstone.

3. FIELD AND LABORATORY INVESTIGATIONS

The field investigation consisted of three boreholes which were advanced by means of a skid mounted diamond drill. Disturbed samples were obtained using a 2 inch O.D. split-spoon sampler driven according to the specifications for the Standard Penetration Test. Some of the samples of the cohesive stratum were obtained in 2 inch I.D. Shelby tubes. In situ vane tests were also carried out within this zone to determine the undrained shear strengths. Bedrock was proven at two boring locations by obtaining BXL size rock core samples. The soil, bedrock, and groundwater conditions encountered in the borings are presented on the Record of Borehole Sheets and on Dwg. #8277302-A.

The results of laboratory testing are plotted on Record of Borehole Sheets, and on Figs. 1 to 4.

4. SUBSURFACE CONDITIONS

4.1 General

The natural deposits at this location consist of a stratum of 13 to 17 ft. of firm to stiff silty clay followed by 16 to 24 ft. of compact to very dense gravelly sand to sandy gravel. The overburden is underlain by sandstone bedrock.

In locations where the approaches were constructed, the natural deposit is overlain by cohesive roadway embankment material consisting of clayey silt with some sand and gravel, and is up to about 4.5 ft. in height.

Boundaries between different deposits are shown on the Record of Borehole Sheets which are contained in the Appendix to this report. The locations and elevations of the borings are shown on Dwg. # 8277302-A together with the estimated profile and section. A description of the soil types encountered in the borings is as follows:

4.2 Fill Material

Roadway embankments in the immediate vicinity of the approaches which were constructed in conjunction with the present structure were found to be up to about 4.5 ft. in height. The material consists of clayey silt with some sand and gravel. The fill material was found to be moderately compacted to a stiff consistency.

4.3 Silty Clay

This natural deposit was observed immediately below the fill material or the creek bed. Within this stratum, random pockets or seams of silt to silty sand were present. In addition, the upper 5 to 7 ft. of the deposit within the creek bed showed the presence of organics, and the organic content by weight ranged from 0.4 to 0.6%. The total thickness of this

cohesive deposit was found to vary from 13.5 to 16.5 ft.

The engineering properties of the cohesive material, as determined by the field and laboratory testing, are plotted on the Record of Borehole Sheets and summarized in tabular form below:

<u>INDEX PROPERTIES</u>		<u>RANGE</u>	<u>AVERAGE</u>
Natural Moisture Content	W (%)	32 - 39	35
Liquid Limit	W_L (%)	35 - 48	42
Plastic Limit	W_p (%)	15 - 21	18
Bulk Density	(pcf)	116 - 119	118

<u>COMPRESSIBILITY CHARACTERISTICS</u>		<u>B.H. 1 Sample 3</u>	<u>B.H. 3 Sample 5</u>
Initial Void Ratio	(e_o)	1.02	1.07
Compression Index	(C_c)	0.27	0.32
Degree of Preconsolidation	$(P_c - P'_o)$ tsf	3.35	2.51

UNDRAINED SHEAR STRENGTH (C_u) psf

In situ vane tests	1,200 - 2,240
Unconfined Compression Tests	865 - 915
Sensitivity (by in situ vanes)	6 - 15

The Atterberg limit test results, given in the table, are also summarized on the Plasticity Chart, Fig. 1. The testing indicates that the cohesive stratum is inorganic and of intermediate plasticity.

Standard Penetration testing within this deposit gave 'N' values ranging from 1 to 22 blows/ft. Based on this together with the undrained shear strength values, it is estimated that the consistency of the deposit varies from firm to stiff. The sensitivity, defined as the ratio of the undrained shear strength of the soil in an undisturbed state to that of the soil in a remoulded condition, as determined by in situ vane tests ranges from 6 to 15 which indicates that the cohesive stratum is generally sensitive.

The consolidation characteristics of the cohesive deposit were determined by carrying out two laboratory oedometer tests, the results of which are shown as void ratio vs. pressure plots on Fig. 4. The results indicate that deposit is preconsolidated by about 2.5 to 3.3 t.s.f. in excess of the existing overburden pressure.

4.4 Gravelly Sand to Sandy Gravel, Some Silt

This deposit was observed immediately below the cohesive stratum and above the bedrock. The thickness of this stratum varies from 16 to 24 ft. Standard Penetration testing performed in this deposit gave 'N' values ranging from 8 to 345 blows/ft. Based on these results, it is estimated that the relative density of this material in general is estimated to be compact for the upper 4 to 7 ft., and beneath this it becomes dense to very dense with depth. Grain size distributions carried out on samples within this deposit are plotted on Fig. 3.

4.5 Bedrock - Sandstone

The overburden is underlain by sandstone bedrock and it was proven in two boreholes by obtaining from 6.0 to 9.2 ft. of BXL size rock core samples. The surface of the bedrock varies from elevations 297.5 to 308.1, which indicates that the depth to bedrock ranges from 34 to 42 ft. below ground surface.

The bedrock can be identified as sandstone. It is fractured with tight joints, and fine to medium grained. The logging of the rock cores as described by Mrs. Z. Koniuszy, Geologist for MTC, are presented on the Diamond Drill Record Sheet included in the Appendix.

5. GROUNDWATER CONDITIONS

The groundwater level is at or slightly higher than the creek water level which was found to be at el. 341.3 at the time of the investigation. Artesian conditions were encountered in two of the boreholes, and the source of the artesian water is primarily confined within the upper portion of the

bedrock. The flow of the artesian water observed from the 3 inch I.D. casing, when extended to the bedrock surface, was estimated to be half a gallon per minute near the ground surface. The casing was then extended above ground level in order to determine the stabilized head and was found to be 1 to 2 ft. above the ice level to elevations 342.3 - 343.3.

6. DISCUSSION AND RECOMMENDATIONS

6.1 General

It is proposed to widen the existing Hwy. 43 structure which crosses over Rosedale Creek (Bakers Bridge). The initial proposal was to widen the structure some 8 ft. on either side. The recent proposal calls for widening by some 9 ft. on the south side of the 24 ft. wide existing bridge. A grade revision at the structure crossing has not been proposed.

The subsoil consists of 13 to 17 ft. of firm to stiff silty clay underlain by a 16 to 24 ft. thick deposit of compact to very dense gravelly sand to sandy gravel. The overburden is underlain by sandstone bedrock.

Our recommendations pertaining to the design of the foundations for the proposed structure widening, as well as the related approach embankments are presented below.

6.2 Structure Foundations

As discussed previously, the precise details of the existing foundations and their founding levels are not known. The existing foundations might have settled due to the induced loadings; the extension or the widening if founded on spread footings may settle in a similar manner due to the imposed loads. This may result in differential settlements between the existing footing and the widening. The upper 13 to 17 ft. of the subsoil is silty clay of firm to stiff consistency. The strength and compressibility characteristics of this cohesive deposit are such that the widening may not be able to be supported on a spread footing type of foundation.

In order to minimize the differential movements between the existing and the widening portion of the structure, our recommendation is to support the

new portion of the structure on end-bearing steel H-piles designed for the maximum allowable pile capacity. It is expected that the piles for the west abutment extension will reach bedrock surface. However, in the case of east abutment extension, the piles may develop the maximum allowable load within the lower portion of very dense gravelly sand to sandy gravel stratum between elevations 300 and 305; and here the pile driving should be controlled by the Hiley Formula. The structure extension should be articulated in such a manner that any minor movements between the existing and the new one will not endanger the integrity of the existing structural elements.

Backfill for the widened portion of the abutments should be free draining granular material as per current MTC standards. Provision for drainage from this material should be made to ensure that no excess hydrostatic or ice pressure builds up behind the abutment walls.

In order to construct the structure in a relatively dry condition, a temporary dewatering scheme will be necessary. This could be achieved by constructing a coffer dam or a temporary earth dyke consisting of relatively impervious type material. Excavation for the pile caps which would follow will take place within the relatively impervious silty clay stratum. Any minor flow within the excavation could be controlled by conventional means such as pumping from sumps.

6.3 Approach Embankment

The new centerline of Hwy. 43 (Line 'A') will be shifted approximately 5 ft. south of the existing one, without any change in the grade. At the structure crossing the roadway grade is at elev. 347 and creek bed is at about elev. 336. No stability problems are anticipated for 2 horizontal to 1 vertical slopes. The portion of the new embankment should be 'keyed' into the existing embankment in accordance with current MTC practices.

The subsoil, beneath the new portions of the approach embankments, will settle due to the imposed loading. The differential settlement between the existing embankment where most of the settlement has already taken place and the widened portion of the embankment will be in the order of one inch.

7. MISCELLANEOUS

The field work for this investigation was carried out during the period of January 21, 1976 to February 3, 1976, under the supervision of Mr. S. Kirkwood, Student Technician.

The equipment used for subsoil sampling was owned and operated by Johnston Drilling Co.

This report was written by Mr. H. Shah and was reviewed by Mr. M. Devata, Supervising Engineer.

H. Shah

H. Shah, P. Eng.
Project Engineer



M. Devata

For M. Devata, P. Eng.
Supervising Engineer

MD/bp
April, 1976

APPENDIX

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

WP 827-73-02 LOCATION Sta. 313 + 64 16.5' Rt. Line 'A' ORIGINATED BY SE
 DIST 9 HWY 43 Line 'A' BORING DATE January 21 to 27, 1976 COMPILED BY NT
 DATUM Geodetic BOREHOLE TYPE NX Casing, BXL Core CHECKED BY RS.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ PCF	REMARKS Artesian Head % CR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
341.3	Ice Level															
0.0																
337.5	Creek Bottom															
3.8	Silty clay, random pockets or seams of silt to silty sand, sensitive, grey		1	SS	1	12"										0.45% Org.
			2	TW	PM											0.57% Org.
			3	TW	PM											0 0 37 63
	Firm to stiff trace of org. in the upper 7 ft.)		4	SS	6											$e_o = 1.02$
			5	TW	PM											$c_c = 0.27$
			6	SS	7											
321.5	some sand & gravel		7	SS	5											
19.8	Gravelly sand to Compact sand to Dense to sandy Very Dense gravel, some silt.		8	SS	14											
			9	SS	46											19 38 18 5
			10	SS	100	3"										
			11	SS	135	6"										
297.5	Boulder		12	SS	177											
43.8	Bedrock Sandstone (Fractured)		13	BXL RC	Rec 74%											
			14	BXL RC	Rec 91%											
288.3			15	BXL RC	Rec 94%											Encountered
53.0	End of Borehole															

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 2

WP 827-73-02 LOCATION Sta. 313 + 64 27.0' Lt. Line 'A' ORIGINATED BY SK
 DIST 9 HWY 43 Line 'A' BORING DATE January 28/29, 1976 COMPILED BY NT
 DATUM Geodetic BOREHOLE TYPE NX Casing, BXL Core CHECKED BY R.S.

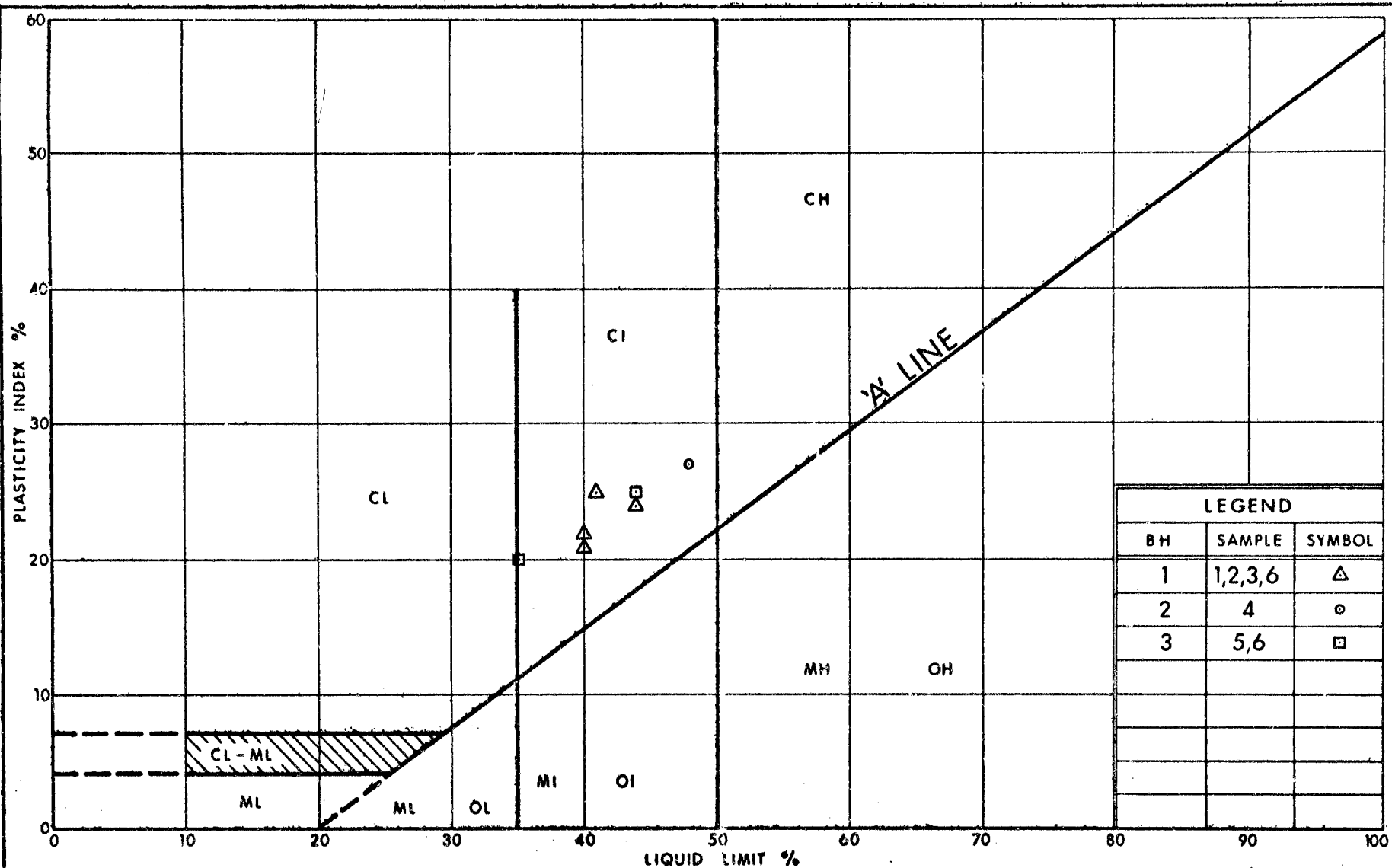
SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ PCF	REMARKS Artesian Head
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
341.3	Ice Level															
0.0	Creek Bottom					340										
2.0	Silty clay, random pockets or seams of silt to silty sand, sensitive, grey		1	SS	22											
			2	TW	PM											
			3	TW	PM											
	Firm to Stiff (some gravel & trace of org. in upper 5 ft.)		4	TW	PM	330										
325.8			5	SS	8											
15.5	Gravelly sand to sandy gravel, some silt		6	SS	10	320										
	Compact		7	SS	156											
	Dense to Very Dense		8	SS	225	310										
307.3																
34.0	Bedrock Sandstone (Fractured)		9	BXL RC	Rec 61%											
301.3																
40.0	End of Borehole					300										

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

WP 827-73-02 LOCATION Sta. 313 + 30 18.0' Rt. Line 'A' ORIGINATED BY SK
 DIST 9 HWY 43 Line 'A' BORING DATE January 30 to February 3, 1976 COMPILED BY NT
 DATUM Geodetic BOREHOLE TYPE NX Casing CHECKED BY R.S.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			UNIT WEIGHT γ PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	W_P	W	W_L		
345.1	Ground Level															
0.0	Fill		1	SS	11											
340.6	Clayey silt, some sand & gravel. Stiff		2	SS	6											Water Level inferred
4.5	Silty clay, random pockets or seams of silt to silty sand, sensitive, grey		3	SS	10											
			4	TW	PM											
			5	TW	PM											
	Firm to Stiff		6	TW	PM											
			7	TW	PM											
324.1			8	TW	PM											
21.0	Gravelly sand to sand & gravel, some silt.		9	SS	32											
			10	SS	13											
	Compact to Very Dense		11	SS	118											
308.1			12	SS	345											
37.0	End of Borehole Probable Bedrock															



Ontario
ENGINEERING SERVICES BRANCH

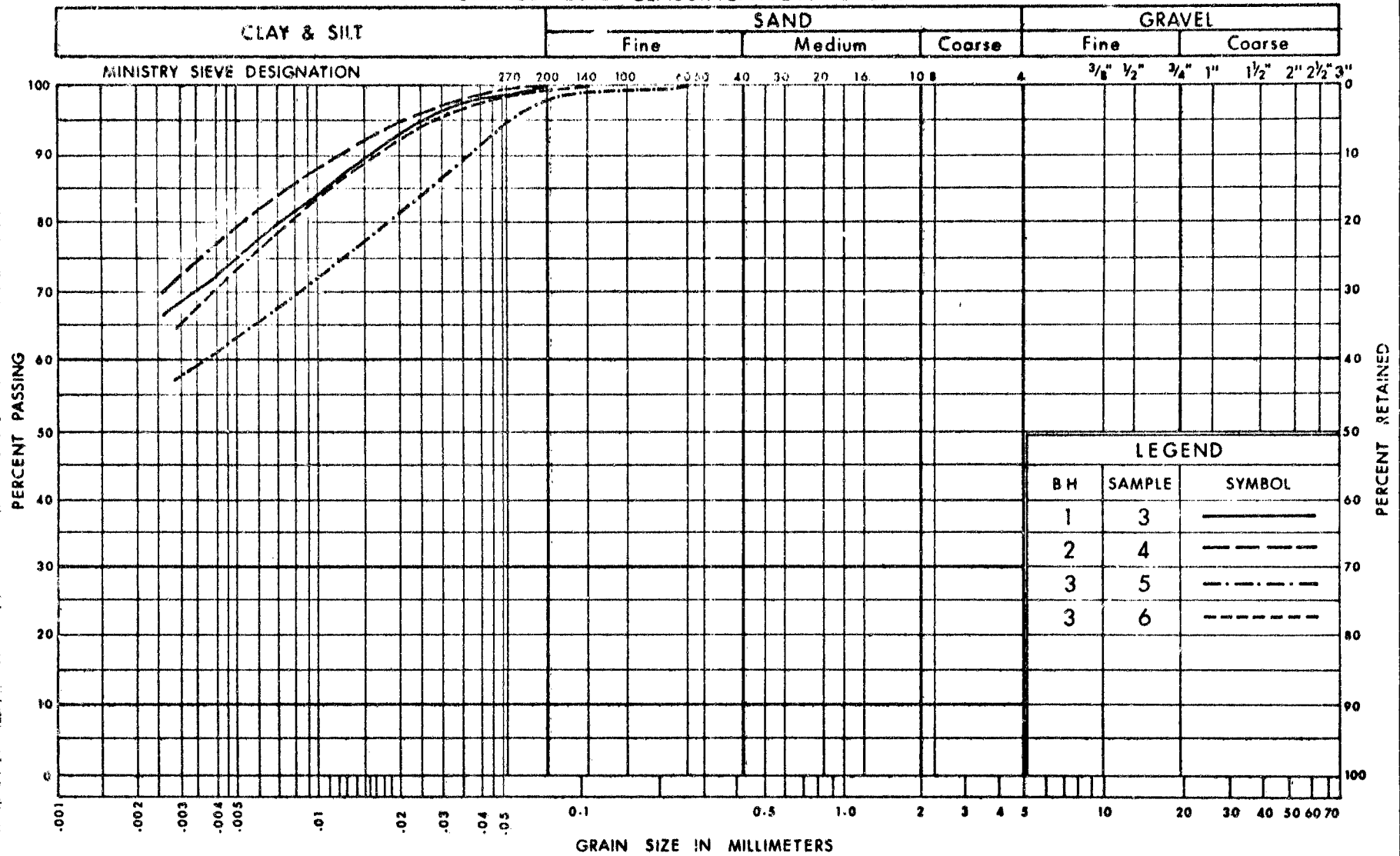
Ministry of
Transportation and
Communications

PLASTICITY CHART
SILTY CLAY
SENSITIVE

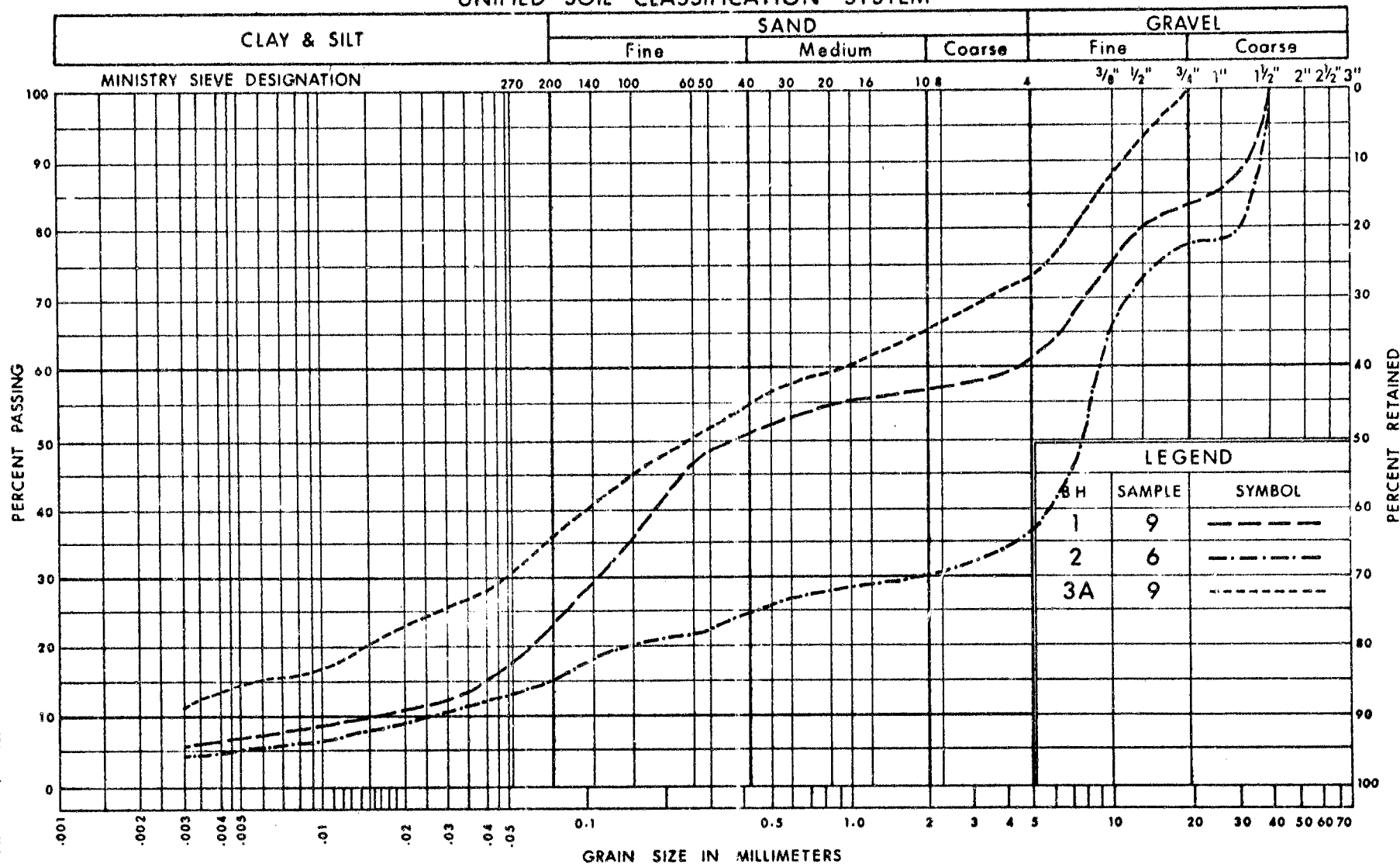
FIG No 1

W P 827-73-02

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

Ontario

ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
GRAVELLY SAND TO SANDY GRAVEL
SOME SILT, TRACE OF CLAY

FIG No 3

W P 827-73-02

VOID RATIO - PRESSURE CURVES

W.P. NO. 827-73-02

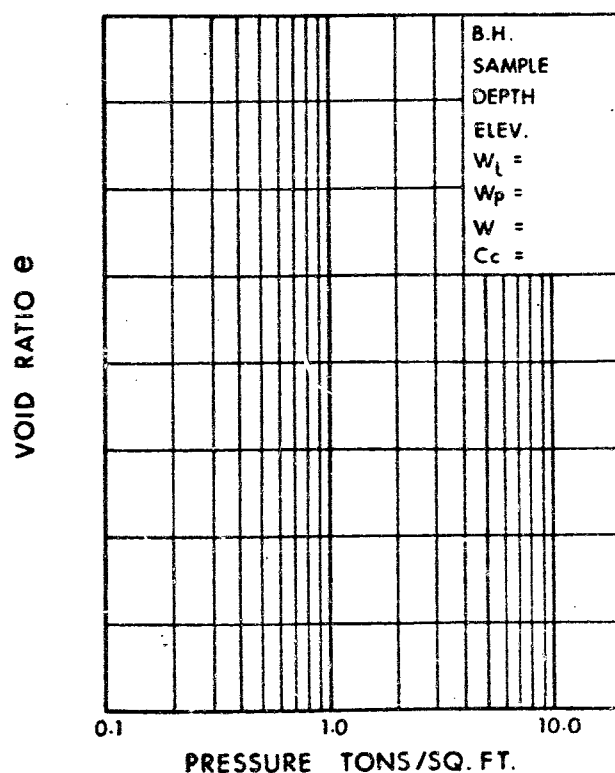
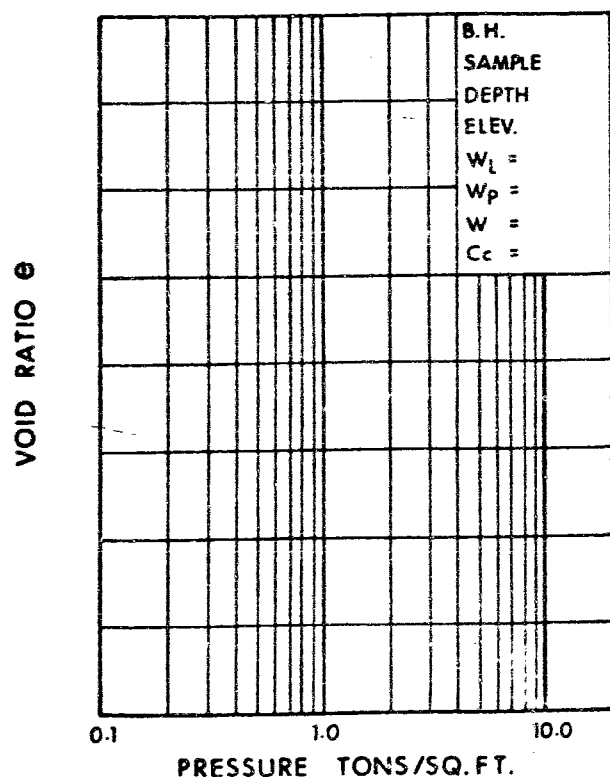
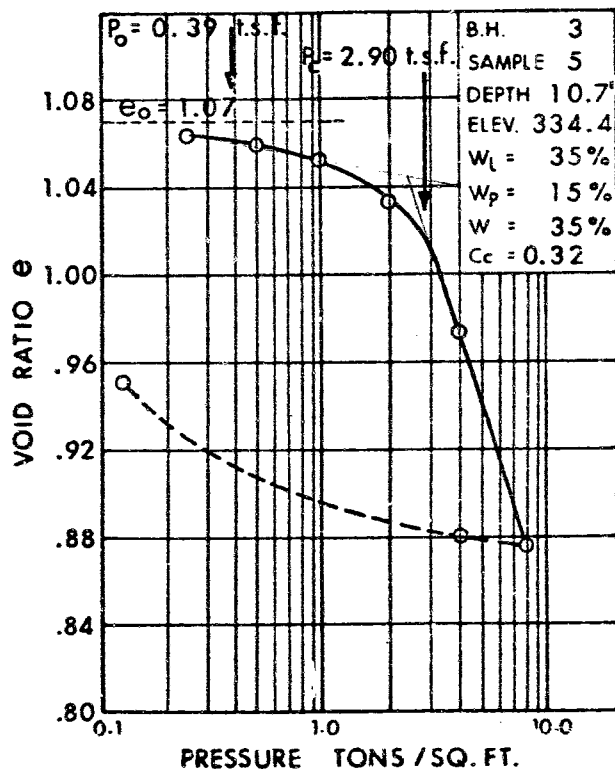
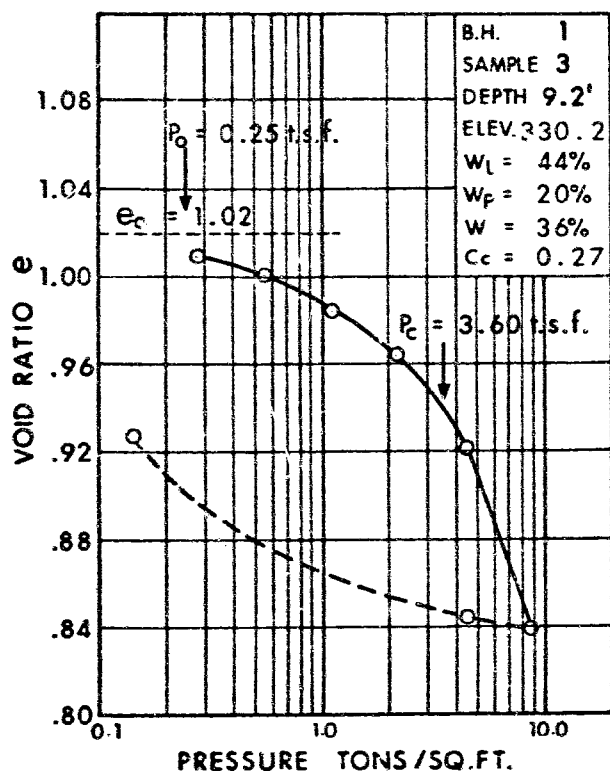


FIG. 4



MOLE NO. _____ SHEET NO. _____

90°

PROPERTY LOCATION W.P. 827-73-02

LATITUDE

DEPARTURE

BEARING

TOTAL FOOTAGE _____

ELEV. COLLAR _____
 DATUM _____
 DATE STARTED _____
 DATE COMPLETED _____
 DRILLED BY _____
 LOGGED BY _____

[illegible]

DATE OF EXAMINATION February 13, 1978

Z. Koniuszy

FF-A-24(a) (Rev Jan 73)

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N' STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL. THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>c LB./SQ.FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
w_s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
C_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{C_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

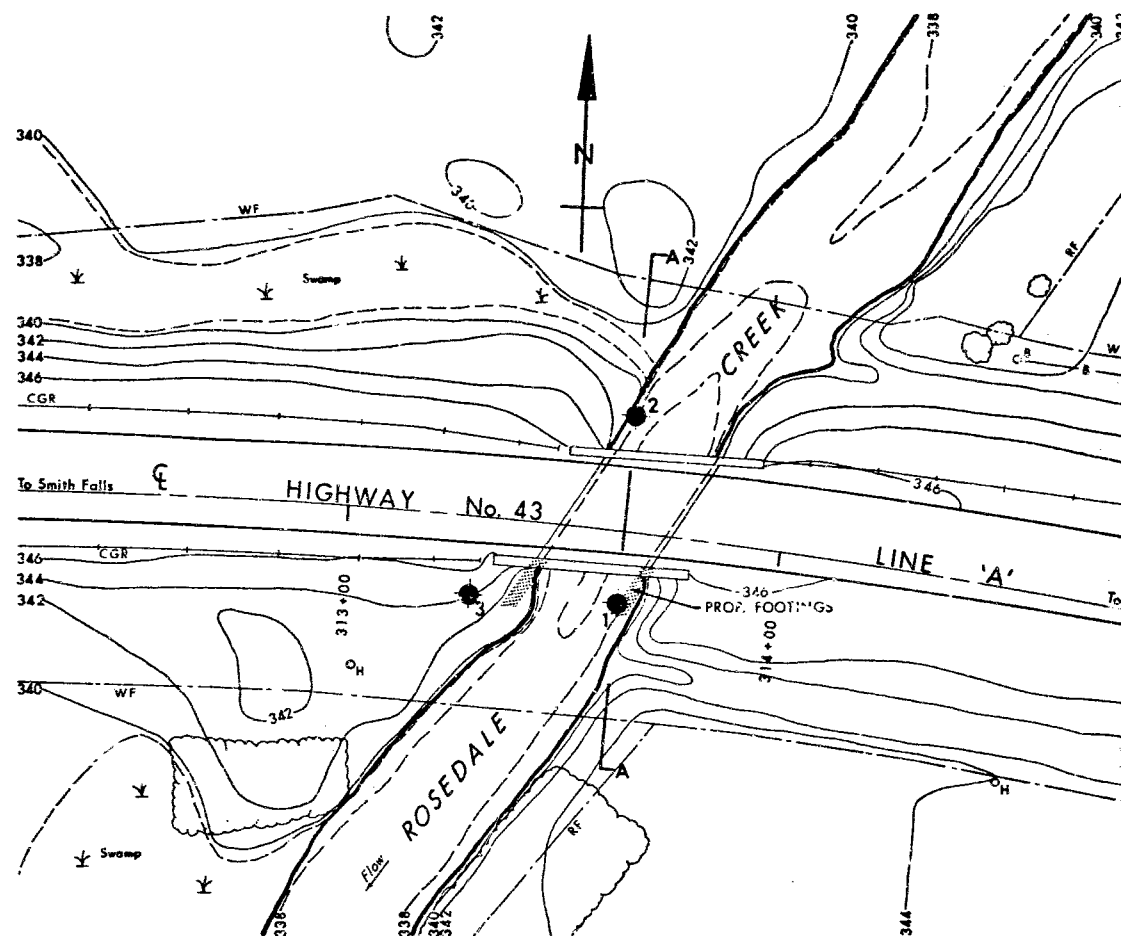
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

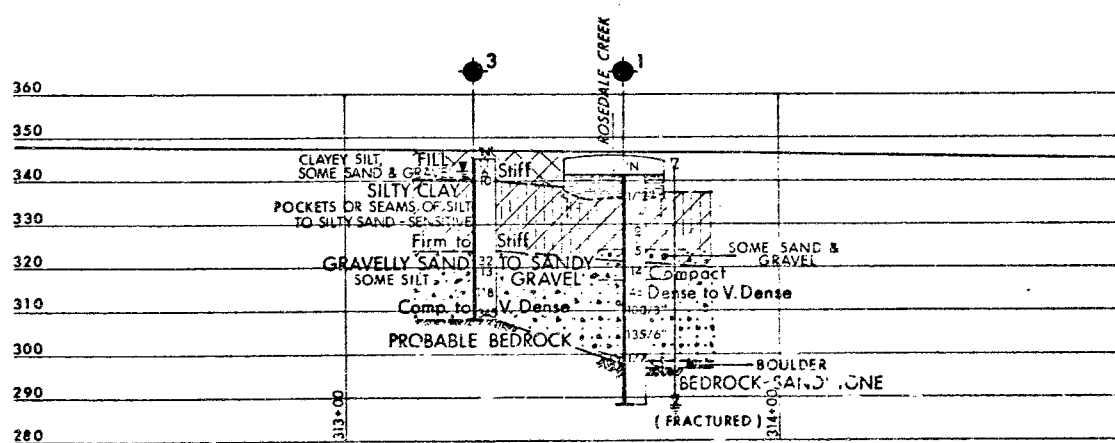
SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



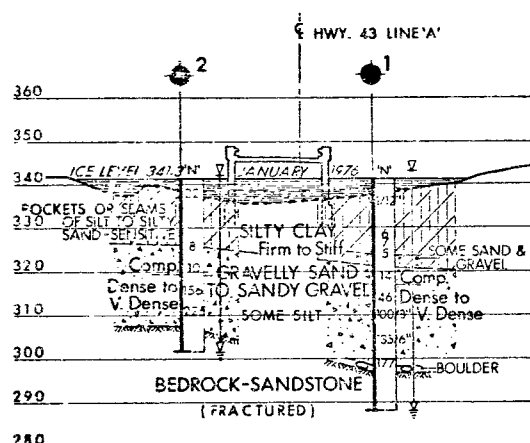
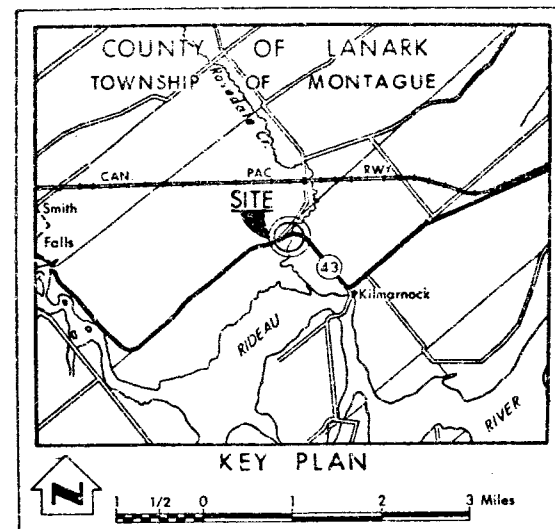
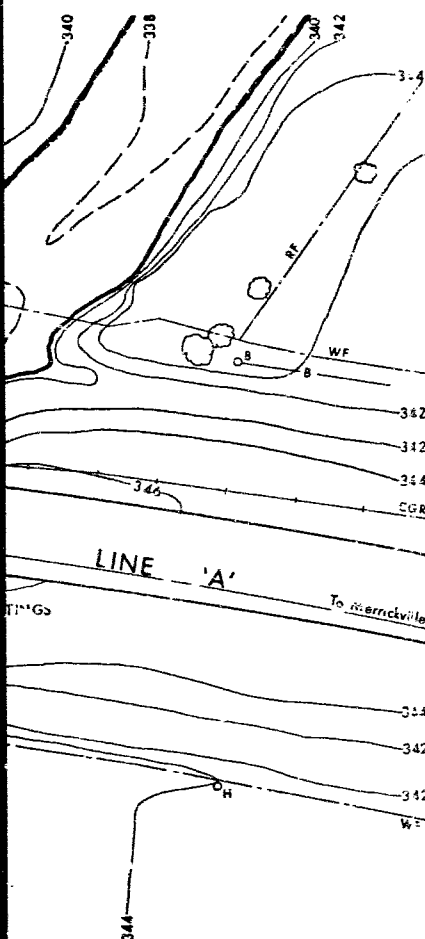
PLAN

20 10 0 SCALE 20 40 FT.



PROFILE - LINE 'A'

20 10 0 SCALE 20 40 FT.



SECTION A-A
20 10 0 SCALE 20 40 FT.

LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Resistance Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation JAN. & FEB. 1976		
	Head		
	Encountered		
	ARTESIAN WATER		
NO.	ELEVATION	STATION	OFFSET
1	341.3	313+64	16.5' RT.
2	341.3	313+64	27.0' LT.
3	345.1	313+30	18.0' RT.

— NOTE —

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

	340
	350
	360
	370
	380
	390
	400
	410
	420
	430
	440
	450
	460
	470
	480
	490
	500
	510
	520
	530
	540
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	790
	800
	810
	820
	830
	840
	850
	860
	870
	880
	890
	900
	910
	920
	930
	940
	950
	960
	970
	980
	990
	1000

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
ENGINEERING SERVICES BRANCH—GEOTECHNICAL OFFICE—SOIL MECHANICS SECTION

BAKERS BRIDGE
(ROSEDALE CREEK)

HIGHWAY NO. 43 - LINE 'A' DIST NO. 8
CO. LANARK
TWP. MONTAGUE LOT 22 CON. 1

BORE HOLE LOCATIONS & SOIL STRATA

SUBMITTAL CHECKED	DATE 7 APRIL, 1976	W. NO. 827-73-02	LEARNING NO. 8277302-A
DRAWN N.T. CHECKED		W. NO.	
APPROVED		DATE NO. 15-120	BRIDGE DRAWING NO.
		CONT NO.	

Mr. C.S. Grebski
Structural Design Engineer
Structural Design Office
West Building, Downsview

Soil Mechanics Section
Engineering Materials Office
West Building, Downsview

November 30, 1976

Rosedale Creek
Bakers Bridge Widening
W.P. 827-73-02, Site 15-120
Hwy. 43, District 8, Kingston

We have reviewed the final bridge drawings (No. 15-120 Sheet 1 and Sheet 3) of this project. Our comments are as follows:

1. The additional loading of 0.6 ksf imposed on the existing footings due to an increased deck elevation is considered acceptable.
2. Under this new loading, the existing structure may settle an additional 0.5 inches. In view of this, construction joints should be provided to accommodate the differential settlements.
3. The gabions on the north bank are found to be stable.
4. Pile lengths of the west abutment could be reduced to 32 feet.

B. Ly

B. Ly
Senior Engineer

For: M. Devata
Supervising Engineer

MD/BL/gs

cc: Files *J*
Record Services



totten sims hubicki associates limited

G. L. TOTTEN B.Sc., P. Eng.
R. E. SIMS B.A.Sc., P. Eng.
J. M. HUBICKI B.A.Sc., P. Eng.
R. L. WINDOVER M.Sc., P. Eng.
P. C. EBERLEE B.A.Sc., P. Eng.

1500 HOPKINS STREET, L1N 2C3
WHITBY, ONTARIO, (416) 668-9363

Mr. C. S. Grebski, P. Eng.
Structural Design Engineer
Ministry of Transportation and Communications
1201 Wilson Avenue
Downsview, Ontario

September 1st, 1976

Attention: Mr. K. G. Bassi, P. Eng.,
Regional Structural Design Engineer

Re: Rosedale Creek (Baker's Bridge) Widening,
W.P. 827-73-02, Site 15-120, Highway 43,
District 8, Kingston

Dear Sir:

As requested, we are pleased to provide the following information relative to the above noted bridge widening:

1. The total additional dead load which will be applied on the existing bridge is approximately 45 kips.
2. The proposed gabion wall cross-section (maximum height 9'), is indicated on the attached sketch.

We trust that the above and enclosure will permit:

- a) A further evaluation of the magnitude of differential settlement between the existing structure which is supported on spread footings and the widening which will be supported on piles, and
- b) A review of the stability of the north embankment slopes.

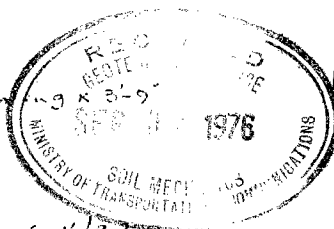
In the event that you require additional information in the above regards, please advise.

Yours very truly,

G. L. Aleong, P. Eng.

GLA/an

Encl:



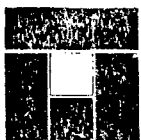
22.5 K/ABT

0.625 K/54

1.42

2043012

0.625 K/54



CONSULTANTS

cobourg whitby kingston toronto muskoka

totten sims hubicki associates limited

ROSGDALE CREEK (BAKERS BR.) WIDENING

WP # 827-73-02

job number

42-3056

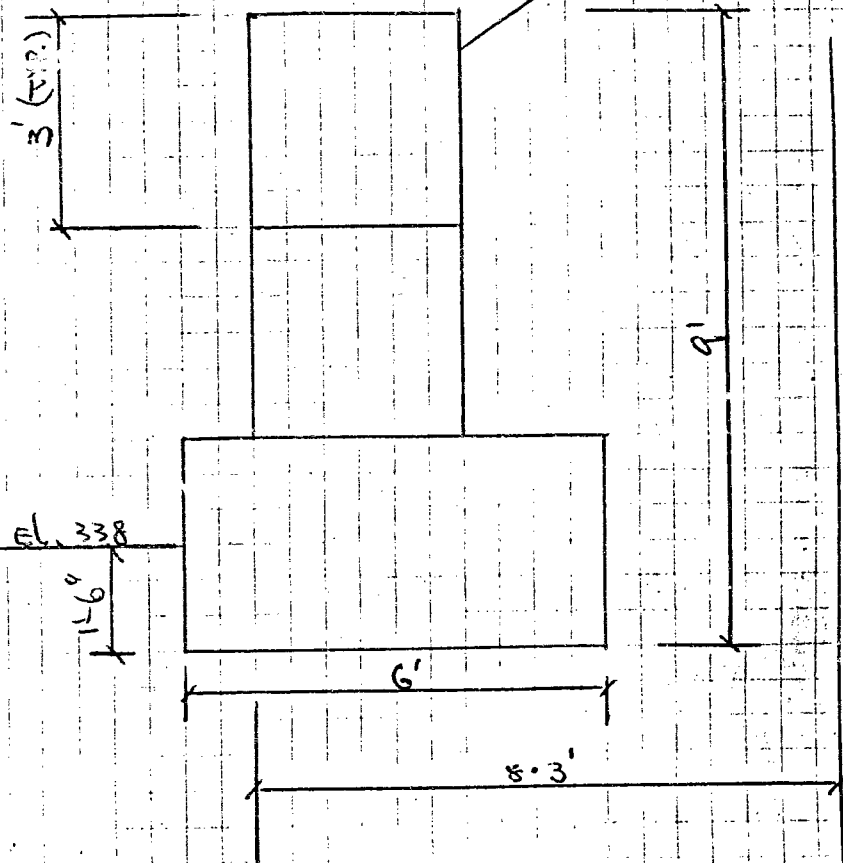
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SEPT 1

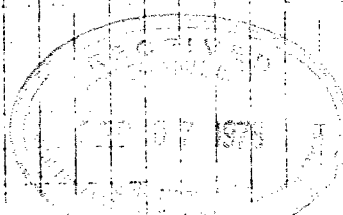
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checked by

STREAMBED EL. 338



TYPICAL GABION WALL SECTION



Mr. C.S. Grebski
Structural Design Engineer
Structural Design Office
West Building, Downsview

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

August 20, 1976

Rosedale Creek
(Baker's Bridge) Widening
W.P. 827-73-02, Site 15-120
District 8, Kingston

We present the following comments regarding the Preliminary Bridge Plan Drawing 15-120-P1 for the above mentioned structure.

1. The existing structure is believed to be supported on spread footings. The widening will be supported on end-bearing steel H piles. Additional loading will be imposed on the existing structure and thus differential settlements may take place between the existing structure and the widening. More comments pertaining to the magnitude of differential settlements would be made when the loading details are available.
2. Comments pertaining to the stability of the slopes on the north side which will incorporate the gabion walls will be made when the detailed geometry is available.
3. In order to avoid the creek water from entering the excavations, a cofferdam or a temporary earth dyke consisting of relatively impervious type of material may be required.
4. The pile tip elevations shown for the east abutment are approximate; however, the pile driving should be controlled by the Hiley Formula during construction to attain the design load.

H. Shah
Project Engineer

For: M. Devata
Supervising Engineer

MD/HS/gs

cc: Mr. T.C. Kingsland
Files
Record Services



M. Devata

Memorandum

To: Mr. T. C. Kingsland,
Reg. Structural Planning Engineer,
Eastern Region, Kingston.

From: Structural Office,
West Building, Downsview.

Attention: Date: January 30, 1976.

Our File Ref. In Reply to

Subject: Bakers Bridge (Rosedale Creek),
W. P. 827-73-02, Site 15-120,
Highway 43, District 8, Kingston.

As requested in your memo of December 9, 1975 we have investigated the feasibility of widening and super-elevating this structure. Assuming that the structure is in good condition, our findings are as follows:


1. The structure cannot be super-elevated (0.059 ft.) by providing asphalt padding since the reinforcement in the frame will be stressed to 30.35 Ksi under AASHTO Live Load.
2. Using polystyrene voids and lightweight concrete as for Snake River Bridge (W.P. 167-65-03) to provide the required superelevation, the reinforcement will be stressed to 24.10 Ksi under AASHTO Live Load.
3. With the polystyrene voids and lightweight concrete, the existing structure would meet POBBL load factor requirements, but in order to meet the servicibility requirements the yield strength of the reinforcement has to be 50 Ksi (Hard Grade).
4. Again with the polystyrene voids and lightweight concrete, the existing structure would meet the AASHTO load factor requirements only if the reinforcement has a yield stress of 50 Ksi.
5. The plans of the existing bridge call for the reinforcement to be either intermediate grade (40 Ksi) or Hard Grade (50 Ksi). Since this was originally a municipal structure we have no test records to indicate what grade of reinforcement was actually used in this structure. We would therefore suggest that one or two samples of longitudinal reinforcement be removed from the top of curb and tested to establish their yield stress. If the reinforcement in the structure is found to be Hard Grade, it would be feasible to superelevate the deck in the same way as for Snake River Bridge (W.P.167-65-03).

....2



6. We note that the present plans call for widening on both sides of the structure. The amount of widening would appear to be about 5'-6" on the inside and 7'-3" on the outside. With these small widenings it will not be possible to use cantilevered wingwalls. The wingwalls will have to be constructed as independent retaining walls. If possible it would be structurally preferable to provide a large widening on the inside only and adjust the alignment to suit. This would also eliminate the need for new wingwalls on the outside.

KGB/cf



K. G. Bassi,
Regional Structural Design Engineer.

c.c. P. D. Billings
S. C. J. Radbone
M. Devata /

Soil Mechanics Section
Geotechnical Office
West Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Tel: (416) 248-3282

January 22, 1976

F.E. Johnston Drilling Co. Ltd.
P.O. Box 4134
Station 'E'
Hawthorne Road
Ottawa, Ontario
K1S 5A7

Dear Sirs:

This letter confirms our request by telephone of January 16, 1976, for the supply of a Diamond Drill Skid Mounted (Item No. 1.1 (A)), together with all necessary equipment, as per your Tender for Supply Contract S-75-1922 at Hwy. 43 and Rosedale Creek, 4 miles east of Smiths Falls, on January 20, 1976.

Mobilization will be from Ottawa, Ontario.

Our W.P. number is 827-73-02.

Yours truly,

M. Devata
Supervising Engineer

cc: W.W. Fry
(Attn: V. Di Marco)
Files /
Record Services

DRILL ITEM NO. 1.
UNIT REQUIRED 1.1 (A)

START DATE Jan. 20, 1976 ESTIMATED DRILLING FOOTAGE 80
SITE 1 MI. E. OF SMITH FALLS ESTIMATED FEET PER HOUR 2.3
 Hwy. 43 & Russian Creek. ESTIMATED TOTAL HOURS 35
W.P. 827-73-02
W.O. _____

RAFT REQUIRED YES ☐ NO ☒

CONTRACTOR	EQUIPMENT DESCRIPTION AND RATES										MOBILIZATION RATES					MOBILIZATION POINTS	MILES ONE WAY	MOB. COST	DRILLING COST	OTHER COST	TOTAL COST
	1.1 (A) DIAM. DRILL SKID	1.1 (B) DIAM. DRILL TRUCK	1.1 (C) DIAM. DRILL M. V.	1.2 (A) DIAM. DRILL SKID	1.2 (B) DIAM. DRILL TRUCK	1.2 (C) DIAM. DRILL M. V.				3 RAFT	4 RAFT	2 (A) SKID	2 (B) TRUCK	2 (C) M. V.							
ATCOST	3	29.00	29.00	34.00	29.00	29.00	34.00			50.00	NO CHARGE	1.00	1.00	1.00		CONCORD, BELLEVILLE, LONDON & NORTHBAY.	111	277 ⁵⁰	1015.00	200	1437 ⁵⁰
AN. LONGYEAR		33.00	34.00	38.00	35.00	36.00	40.00			50.00	1.25	1.25	1.25	1.65		REXDALE Within 30 miles	22				
	5	35.50	36.50	40.50	37.50	38.50	42.50			50.00	1.25	1.25	1.25	1.65		TORONTO, NORTHBAY, LONDON, SUDBURY.	215	537 ⁵⁰	1242.50	200	1980 ⁵⁰
DODDS	6	35.00	35.00	50.00	35.00	35.00	50.00			50.00	1.25	1.25	1.25	1.50		TORONTO, THUNDER BAY	215	537 ⁵⁰	1225.00	200	1994 ⁵⁰
		Plus \$1.00/mile motel to jobsite or \$22/hour travelling time in client's vehicle																			
DOMINION SOIL	7	37.50	--	--	37.50	--	--			50.00	1.50	1.50	--	--		TORONTO, KITCHENER, LONDON, WINDSOR, THUNDER BAY, SARNIA, NORTHBAY, OTTAWA	215	645 ⁵⁰	1312.50	200	2157 ⁵⁰
		Plus \$1.50/mile motel to jobsite return daily Thunder Bay, North Bay, Ottawa only																			
HAWTHORNE		--	--	--	--	--	--			--	--	--	--	--		OTTAWA					
JOHNSTON		26.00	26.00	30.00	28.00	28.00	30.00			50.00	1.00	1.00	1.00	1.25		OTTAWA, TORONTO Within 30 miles					
	1	28.50	28.50	32.50	30.50	30.50	32.50			50.00	1.00	1.00	1.00	1.25		OTTAWA, TORONTO Outside 30 miles	43	86 ⁵⁰	997.50	200	1283 ⁵⁰
MASTER	2	29.00	29.00	34.00	29.00	29.00	34.00			50.00	1.00	1.00	1.00	1.50		OTTAWA, TORONTO, NORTHBAY, LONDON.	43	86 ⁵⁰	1015.00	200	1301 ⁵⁰
M.K.	4	27.50	--	--	30.00	--	--			50.00	1.50	1.25	--	--		TORONTO, LONDON, BURFORD.	215	537 ⁵⁰	962.50	200	1700 ⁵⁰
SUBSOIL EXPL.		--	--	--	--	--	--			--	--	--	--	--		PETERBOROUGH, TORONTO					
SITE INV. SERV.		--	--	--	--	--	--			--	--	--	--	--		PETERBOROUGH, ORILLIA, PORTHOPE, LONDON, BELLEVILLE, OSHAWA, BRAMPTON.					

ASSIGNED TO JOHNSTON
GIVE REASON IF OTHER THAN LOWEST COST CONTRACTOR ABLE TO SUPPLY EQUIPMENT ON
REQUIRED DATE JAN 20
DATE Jan. 16, 1975 SIGNATURE OF SUPERVISING ENGINEER M. J. Devine

REMARKS ** \$ 200 EST. COST OF DIAMOND BITS USAGE.