DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

30M12-57

TO: Mr. G.C.E. Burkhardt, (2)
Regional Bridge Planning Eng.,
Central Region,
90 Floral Pkwy., Downsview.

FROM: Foundations Office,
Design Services Branch,
Central Bldg., Downsview.

ATTENTION:

DATE: March 9, 1972.

OUR FILE REF.

IN REPLY TO MAR 24 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
The Mimico Creek Crossing of the
Proposed American Drive, Proposed Hwy. #427
District #6, Toronto. Site #24-311
W.O. 72-11005 -- W.P. 48-71-08
CONT. 76-01

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 design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER.

AGS/ao
Attach.
cc: Messrs. D. W. Farren
B. R. Davis
A. Rutka
G. K. Hunter
H. Greenland
B. J. Giroux
T. J. Kovich
G. A. Wrong
B. A. Singh
McCormick & Rankin & Associates Ltd.
Foundations Files
Documents

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FOUNDATION INVESTIGATION REPORT
For
The Mimico Creek Crossing of the
Proposed American Drive, Proposed Hwy. #427
District #6, Toronto
W.O. 72-11005 -- W.P. 48-71-08

1. INTRODUCTION:

In a memo dated December 29, 1971, Mr. G. C. E. Burkhardt, Regional Bridge Planning Engineer, Central Region, requested foundation investigations at the site of several crossings, proposed to be built in conjunction with the future Hwy. #427. The proposed crossing of American Drive over Mimico Creek, discussed below is one of the requested projects.

The subsequent field investigation was carried out under the supervision of this Office. The boreholes were located and surveyed in the field by personnel of the Engineering Survey Office, Central Region.

Presented in this report are the results of the investigation as well as recommendations pertaining to foundations and approach slope stability.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

At the proposed crossing Mimico Creek is roughly 20-25 ft. wide and 1-2 ft. deep. East of the creek the bank rises steeply from elevation 505 ft. up to elevation 530 ft.; west of the creek, there is a rather wide flood plain lying around elevation 512 ft. The immediate vicinity is covered by woods and brush.

Geologically the area belongs to the physiographic region known as the "Peel Plain". The characteristic surface feature of this plain is a gradual and fairly uniform slope towards Lake Ontario, across which deep valleys have been cut by several creeks and rivers. The underlying geological material of this plain is a till or boulder clay, containing large amounts of Palaeozoic shale and limestone.

3. FIELD AND LABORATORY INVESTIGATIONS:

Some three boreholes and adjacent to B.H.'s #1 and 3, two dynamic cone penetration tests were carried out during the course of the field work. A conventional diamond drill rig, adapted for soil sampling purposes was used to advance the holes, by wash boring techniques. Soil samples were recovered by split spoons and by 2" diam. thin walled Shelby tubes. Split-spoon samples were taken by performing standard penetration tests, while the Shelby tubes were pushed 18" into the undisturbed soils manually. Visual inspections and classifications of the soil samples were carried out immediately upon recovery and again in the laboratory. The natural moisture contents of the samples were measured in the laboratory. In addition some samples were subjected to Atterberg Limit tests and grain size analysis.

Undrained shear strength parameters and bulk densities were determined by performing unconfined compression and quick triaxial tests on undisturbed samples.

Field and laboratory test results are marked on the accompanying borelogs, while the locations and elevations of the borings are plotted on Drawing #72-11005A in the Appendix. A stratigraphical profile projected to the centre line of the proposed creek relocation is also given on the drawing.

4. SUBSOIL CONDITIONS:

4.1) Overburden:

Heterogeneous glacial and glacio fluvial deposits were found to form the 40 - 47 ft. deep overburden, beneath which shale bedrock was encountered. The uppermost 18 - 23 ft. layer

.....3

was identified to be clayey silt with some sand to sandy silt with some clay and traces of gravel. The largest portion of the stratum is cohesive with firm to hard consistency. Penetration "N' values within the surficial 4 - 5 ft. or so were measured to be 6 - 7 blows per ft. beneath which "N" values increased considerably, averaging approx. 50 blows per ft. The layer has slight plasticity, with plastic limits of 12% - 14% and liquid limits of 17% - 24%. The laboratory grain size analyses revealed a great deal of variation; the constituent gravel portion ranging from 6% to 31%, sand from 30% to 51%, silt from 30% to 45%, and clay from 8% to 18%. Around elevation 489 ft. - 494 ft. a 10 - 20 ft. thick layer of silty clay with some sand and traces of gravel was observed. The consistency of this layer varies between firm and very stiff. A few unconfined and quick triaxial compression tests were carried out on samples taken from the silty clays, yielding undrained shear strengths of 1,000 p.s.f. to 1,800 p.s.f. This portion of the overburden has intermediate plasticity, with liquid limits of 36 - 44%. Below the silty clays the consistency of the soils increases again corresponding to "N" values of 90 blows per ft. and over. This zone was classified to be clayey silt with some sand and gravel of very slight plasticity. The natural moisture content within the clayey silts are much lower than the plastic limits confirming the hard consistency.

4.2) Bedrock:

Around elevation 472 ft. - 465 ft. bedrock was established and proved in B.H. #1 by diamond drilling within a depth of 8 ft. The bedrock was identified to be a dark grey shale of the Dundas formation. Some 2 - 3 ft. of the upper zone of the rock was found to be weathered and fractured with soft, decomposed seams. Beneath the weathered zone the drilling yielded 80 - 90% recovery, indicating relatively sound rock.

4.3) Groundwater Conditions:

The groundwater level in the boreholes was established to be roughly at elevation 510 ft. some 2 - 3 ft. below general ground surface. The creek water level at the time of the field work (February 1972) was measured to be around elevation 508 ft. There appears to be a slight hydraulic gradient towards the creek.

5. DISCUSSION AND RECOMMENDATIONS:

5.1) General:

The present proposal calls for a single span rigid frame structure over the realigned Mimico Creek in conjunction with the construction of American Drive. The bridge is proposed to be 110 ft. wide, the span length being 36 ft. The top of pavement of American Drive at the crossing is designed to be roughly at elevation 531 ft. The valley will be crossed with embankments of 18 ft. - 25 ft. heights. The slopes of the approach fills at the upstream and downstream end of the bridge will be supported laterally by retaining walls.

Subsoil at the proposed crossing was found to consist of glacial and glacio fluvial deposits of firm to hard consistency and very dense relative density.

5.2) Foundations:

I 's surmised that the creek bed of the realigned Mimico Creek will be at elevation 505 - 506 ft. similar to the existing one. The uppermost clayey silt and sandy silt stratum appears to have sufficient strength to support both the bridge and the adjacent retaining walls on spread footings. Footings should be placed at 4 ft. below river bed, also ensuring adequate cover against scour. This later should be determined by the Hydrology Section. Assuming that the footings will be placed between elevation 505 ft. and 498 ft., safe loads of 3 t.s.f. may be employed on the footing bases. Should the footings - for structural or hydrological reasons - be lowered below elevation 495 ft., the design load on the base must be decreased to allowable net pressures of 1 t.s.f. Footings may also be supported on steel H or tubular piles driven to refusal on bedrock, between elevation 465 ft. and 472 ft. Safe loads equal to the structural strength of the particular piles used may be designed on piles, supported on sound bedrock.

5.3) General Comments:

5.3.1) Dewatering:

Excavations will reach below the groundwater level, but since the subsoil contains sufficient fines, no major dewatering problems are anticipated. It is believed that seepage water can be eliminated from the excavations by conventional open pumping, without inducing instability of the excavation bottom. Pumping should be carried out from sumps, dug around the perimeter of the oversize excavations.

A lean concrete working slab should be poured at the bottom of the footing excavations immediately upon completion, in order to prevent softening of the ground.

5.3.2) Earth Pressures:

Retaining walls should be designed to resist sliding along the footing base due to the horizontal component of the earth pressures. The resistance against sliding may be calculated by using a coefficient of friction of 0.55. The factor of safety against sliding should not be less than 1.5.

5.3.3) Slope Stability:

No stability problems are foreseen for the approach fills, provided they are constructed with slopes of 2 horizontal to 1 vertical.

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6. <u>MISCELLANEOUS</u>:

The field work, carried out during the period of February 1 to February 9, 1972, was supervised by Mr. J. T. Bangs, Project Foundation Engineer. Equipment used was owned and operated by Johnston Drilling Company Ltd., Toronto.

This report was written by Mr. A. K. Barsvary, Senior Foundation Engineer, and reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

A. K. Barsvary

A. K. Barsvary, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
A. K. BARSVARY
PROVINCE OF ONTARIO

K. G. Selby

K. G. Selby, P. Eng.

AKB/ao
March 3, 1972.

APPENDIX I

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

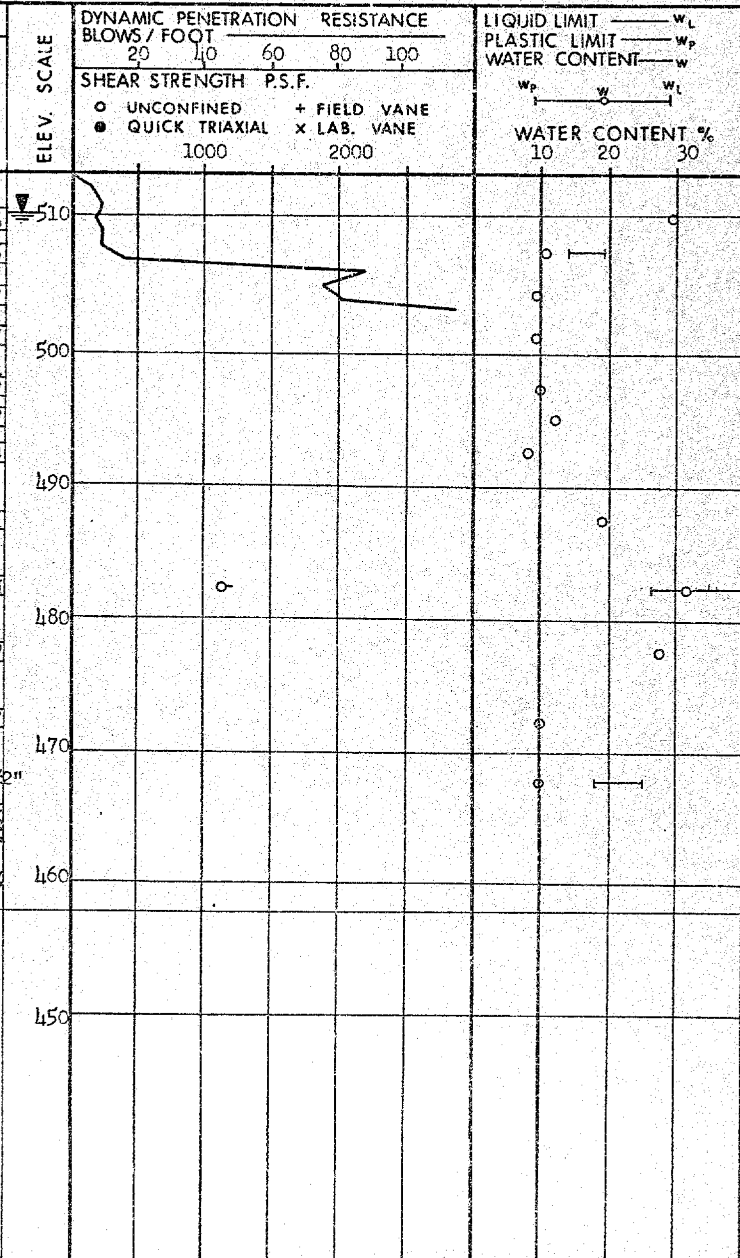
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 72-11005
W.P. 48-71-08
DATUM Geodetic
LOCATION Co-ords. 15,879,547 N; 969,894 E.
BORING DATE Feb. 4-8, 1972
BOREHOLE TYPE Washboring, NX Casing
ORIGINATED BY JTB
COMPILED BY AKB
CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
512.9	Ground Level					
0.0	Sandy silt to silty sand with some clay. Very Dense to Hard. Brown and Grey	1	SS	6		
		2	SS	35		
		3	SS	87		
		4	SS	97		7 35 43 15
		5	SS	45		
		6	SS	89		
489.9		7	SS	104		11 51 30 8
23.0	Silty clay, some sand and gravel. Hard to Stiff	8	SS	44		
		9	TW	PM	117	
474.9		10	SS	9		2 11 51 36
38.0	Clayey silt, some sand, traces of gravel. Hard	11	SS	97		
465.9		~~12~~	~~SS~~	~~100/2"~~		
47.0	weathered Shale Bedrock Grey	13	RC	75%		
		14	RC	80%		
457.9	sound	15	RC	90%		
55.0	End of Borehole					



FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

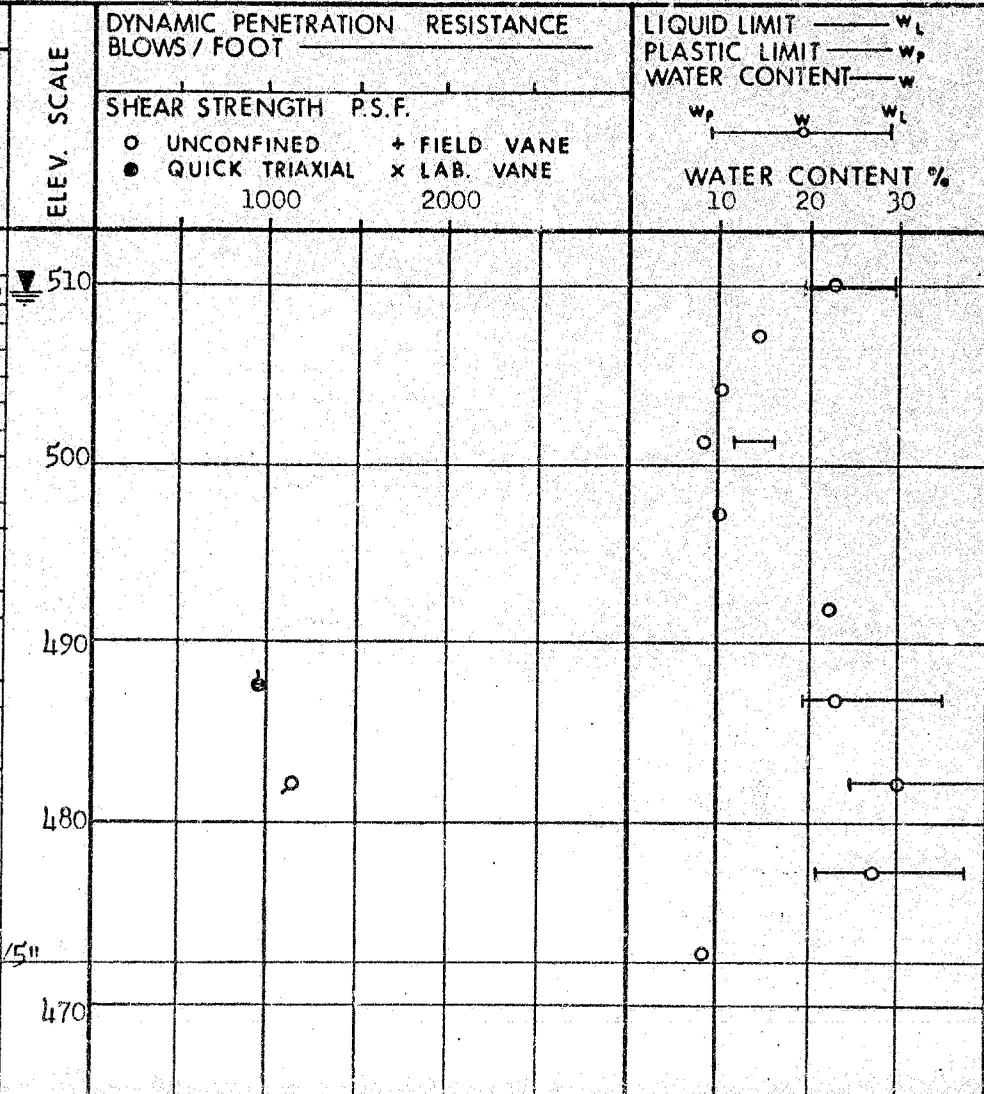
FOUNDATION SECTION

JOB 72-11005
W.P. 48-71-08
DATUM Geodetic

LOCATION Co-ords. 15,879,444 N; 969,935 E.
BORING DATE Feb. 9, 1972
BOREHOLE TYPE Washboring, NX Casing

ORIGINATED BY JTB
COMPILED BY AKB
CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	REMARKS (GR SA SI CL)	BULK DENSITY γ P.C.F.
512.7	Ground Level					
0.0	Clayey silt to sandy silt with some clay & traces of gravel. Firm to Hard Brown to Grey	1	SS	6		
		2	SS	11		
		3	SS	56		
		4	SS	138		
494.7		5	SS	51	6 31 45 18	
18.0	Silty clay with traces of sand and gravel. Stiff to Hard Grey	6	SS	19		
		7	TW	PM		129
		8	TW	PM		129
		9	SS	14		
472.2		10	SS	100/5"		
40.5	End of Borehole Probable Shale Bedrock					



FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 72-11005
W.P. 48-71-08
DATUM Geodetic

LOCATION Co-ords. 15,879,456 N; 969,870 E.
BORING DATE Feb. 1-3, 1972
BOREHOLE TYPE Washboring, BX Casing

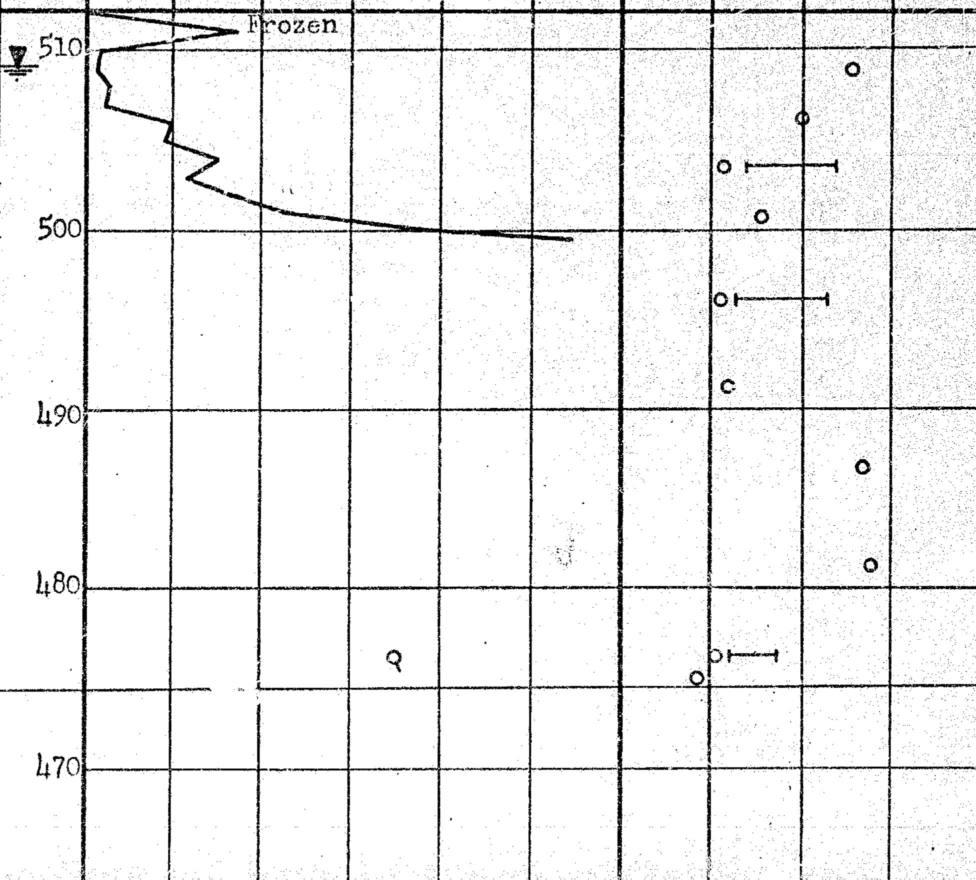
ORIGINATED BY JEB
COMPILED BY AKB
CHECKED BY AK

ELEV. / DEPTH	SOIL PROFILE — DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
512.0	Ground Level						
0.0	Clayey silt to sandy silt with some clay & gravel. Firm to Hard. Brown and Grey		1	SS	7		
			2	SS	17		31 30 30 9
			3	SS	29		
			4	SS	31		
			5	SS	55		
489.0			6	SS	67		
23.0	Silty clay, traces of sand and gravel. Very Stiff		7	SS	27		
478.0			8	SS	15		
34.0	Clayey silt to silt, some sand and gravel. Hard		9	TW	PM	147.5	
474.2			10	SS	185		
37.8	End of Borehole						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 1000 2000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT % 10 20 30

ELEV. SCALE



ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' : - 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 : -

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

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

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS 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
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' and ϕ':) IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
	(c_u and ϕ_u:) IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3{\cdot}1416$
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
σ'	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 (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

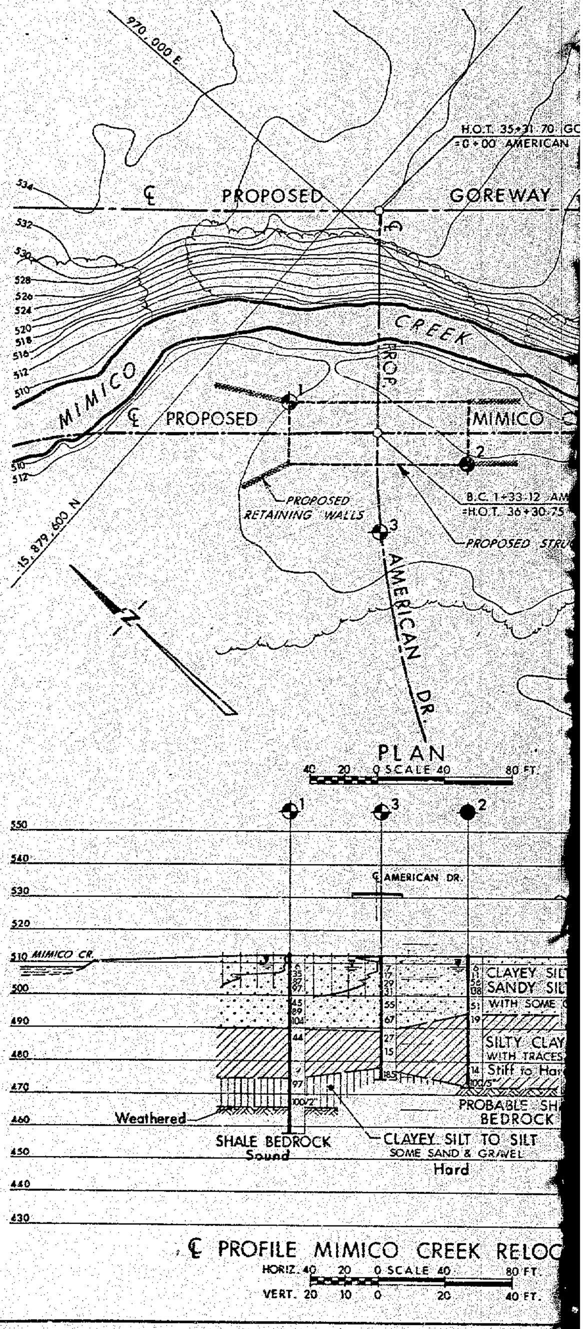
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

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

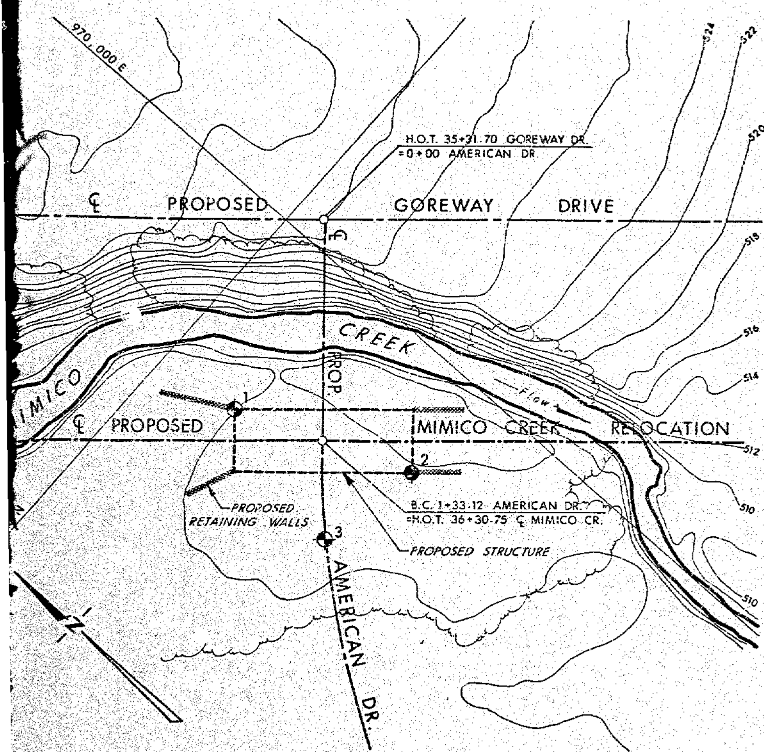
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



970,000 E
H.O.T. 35+31.70 GO
=0+00 AMERICAN
℄ PROPOSED GOREWAY
534
532
530
528
526
524
520
518
516
512
510
CREEK
MIMICO
PROP.
℄ PROPOSED
MIMICO
PROPOSED RETAINING WALLS
B.C. 1+33.12 AM
=H.O.T. 36+30.75
PROPOSED STRU
15,879,600 N
N
AMERICAN DR.
PLAN
40 20 0 SCALE 40 80 FT.
550
540
530
520
510 MIMICO CR.
500
490
480
470
460
450
440
430
℄ AMERICAN DR.
CLAYEY SIL
SANDY SIL
WITH SOME
SILTY CLAY
WITH TRACES
Stiff to Har
PROBABLE SH
BEDROCK
Weathered
SHALE BEDROCK
Sound
CLAYEY SILT TO SILT
SOME SAND & GRAVEL
Hard
℄ PROFILE MIMICO CREEK RELOC
HORIZ. 40 20 0 SCALE 40 80 FT.
VERT. 20 10 0 20 40 FT.

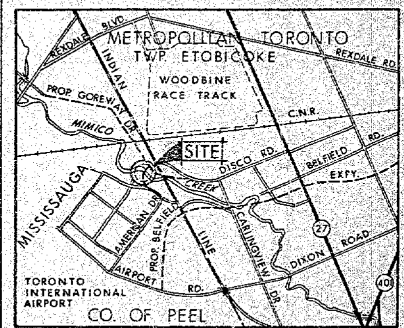
ECORD

DATE



PLAN

40 20 0 SCALE 40 80 FT.



KEY PLAN

SCALE IN MILES

1 0.5 0 1

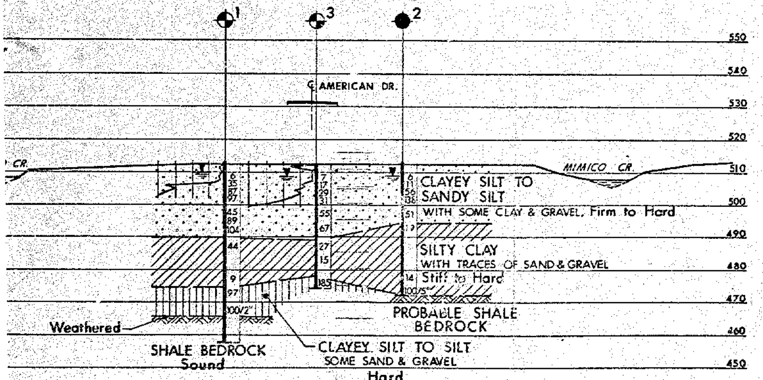
LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. February 1972.

NO.	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	512.9	15,879,547	969,894
2	512.7	15,879,444	969,935
3	512.0	15,879,456	969,870

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



℄ PROFILE MIMICO CREEK RELOCATION

HORIZ. 40 20 0 SCALE 40 80 FT.

VERT. 20 10 0 20 40 FT.

REVISIONS		
DATE	BY	DESCRIPTION

REGISTERED PROFESSIONAL ENGINEER CIVIL A. K. BARSVARY PROVINCE OF ONTARIO

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH —— FOUNDATION OFFICE

MIMICO CREEK RELOCATION

HIGHWAY NO. AMERICAN DRIVE DIST. NO. 6

CO. PEEL

TWP. TOWN OF MISSISSAUGA

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. A. B.	CHECKED	W.P. NO. 48 - 71 - 08	DRAWING NO. 72 - 11005A
DRAWN S. R.	CHECKED	JOB NO. 72 - 11005	
DATE MARCH 21, 1972.		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

Department of Highways Ontario

Copy for the information of A. STERMAC

Mr. G.C.E. Burkhardt
~~Regional Bridge Planning~~ Engineer
DOWNSVIEW, Ontario

Structural Office
West Building
DOWNSVIEW, Ontario

July 19, 1972

American Dr. Crossing at Mimico Cr.
(approx. 150' west of Goreway Dr. Extension)
W.P. 48-71-08 Site 24-311
District 6

Attached herewith are prints of the Preliminary Bridge Plan Drawing D24-311-P1 for the above mentioned structure.

The estimated cost of the proposed structure is $60,000 which includes tender, materials, engineering and sundry construction.

Any comments or revisions you may have should be submitted within three weeks.

CSG/hvh
Encls.

C.S. Grebski
Structural Design Engineer

cc A. McKim
B. Davis
A. Stermac
J. Anderson
R. Fitzgibbon

No comments

K.G. Selby
July 27th 1972

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

Copy for the information of

FOUNDATION OFFICE

~~Mr. G. C. E. Burkhardt,~~
Regional Structural Planning Engineer,
90 Floral Pkwy, DOWNSVIEW.

Structural Office,
West Building, DOWNSVIEW.

September 22, 1972

American Drive Crossing
at Mimico Creek,
Approx. 150' West of
Goreway Dr. Extension,
W.P. #48-71-08, Site #24-311,
Hwy. #- Local, District #6.

72-11-005

Attached herewith are prints of the Preliminary Bridge Plan Drawing D-24-311-P2 for the above-mentioned structure.

The estimated cost of the proposed structure is $60,000.00 which includes tender, materials, engineering and sundry construction.

Any comments or revisions you may have should be submitted within three weeks.

C. S. Grebski,
Structural Design Engineer.

CSG:dp
Attach.

cc. A. McKim,
B. R. Davis,
A. Stermac,
J. Anderson,
R. Fitzgibbon.

25 SEP 72

NO COMMENTS

A. U. B.

KGB.

Department of Highways Ontario
Copy for the information of

A. Stermac,
Principal Foundation Engineer,
Room 107, West Buldg.

Structural Office,
West Bldg., DOWNSVEEW,

January 2nd, 1973.

American Dr. Crossing at Mimico Cr.,
Approx. 150' W. of Goreway Dr. Ext.,
W.P.#48-71-08, Site #24-311,
Hwy. - Local, District #6.

72-11-005

Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure.

Kindly give us your comments at your earliest convenience.

CSG:dp
Attach.

C. S. Grebski,
Structural Design Engineer.

cc. Foundation Office.

No comments

K. G. Selby

Dwg to Structure Office
23 Feb 73

Ministry of Transportation and Communications

Ontario

Cent 76-01

48-71-08

Memorandum

To: Mr. J.E. Callaghan,
Director,
Design & Construction Branch.

From: G. A. Metcalfe

Attention:

Date: 77 07 18

Our File Ref.

In Reply to

Subject:

The Engineering Materials Office and the Structural Office have carried out a comprehensive study in preparing this report.

This office and the Estimating Office provided some input pertaining to construction procedure, vibratory rollers, etc.

The Engineering Materials and Structural Office have initiated some changes and are requesting a Research study be carried out on vibratory roller forces.

Recommend the report be distributed to interested sections in the Ministry.

G.A. Metcalfe

G.A. Metcalfe,
Contract Management Engineer.

GAM/sc

INVESTIGATION REPORT

For
Retaining Wall and Box Culvert Failure
at Mimico Creek and Hwy. 427
Dist. #6, Toronto
Cont. #76-01

INTRODUCTION

In response to a request from the District, the Structural Office and the Soil Mechanics Section investigated the above mentioned failure during the time of construction. The failure of the structural elements was mainly in the area of the eastern portion of the south wall of the box culvert and adjoining retaining wall panels (Retaining Wall 'B'). The distress occurred shortly after the completion of the retaining walls, the box culvert and the associated approach fill in November, 1976.

The retaining wall and the box culvert were being constructed as part of the new Hwy. #427 - Hwy. #409 complex at the western extremity of Metropolitan Toronto, near the Toronto International Airport. Construction of the complex has been carried out under the Ministry's Contract No. 76-01 awarded to C.A. Pitts Construction Company Ltd.

To determine the causes of the distress an investigation has been carried out by the Ministry which consisted of the following:

- A review of the Foundation Investigation and Design Report prepared for the retaining wall, the box culvert and the approach fills.
- A review of the structural design of the retaining wall and the box culvert.
- A review of the sequence and methods of construction.
- A documentation of the patterns of cracks.
- A field investigation to determine the types and the extent of fill material used and their respective geotechnical properties.
- An analysis of the earth pressures acting on the retaining wall and the box culvert as well as the effect of the heavy compaction equipment.

This report was prepared jointly by the Structural Office and the Soil Mechanics Section. It includes all the factual data obtained, the relevant field observations made during the site visits as well as an assessment of the causes of the failure of the various structural elements. In addition, this report presents the immediate remedial measures adopted during the time of construction. Finally the recommendations to prevent future re-occurence of such failure are discussed.

SITE CONDITIONS AT THE TIME OF FAILURE

The construction site is located in the immediate vicinity of Mimico Creek and Indian Line Road, which is partially in the City of Mississauga and partially in Metropolitan Toronto.

At this location, twin three-span (70'-100'-70') structures were being constructed to carry the northbound lanes and southbound lanes of Hwy. 427 over Disco Road. Under and parallel to Disco Road a double cell concrete box culvert (21' span x 18' high x 705' long) was constructed to facilitate the Mimico Creek realignment. To retain the approach fill, end walls up to 240 ft. long and up to 23.5 ft. high, at the east end of the culvert were also constructed. The fill retained by Retaining Wall 'B' during the time of construction has a maximum height of 35 ft. in excess of the height of the wall and was built with 2:1 slopes extending almost to the crest of the wall in the distressed area. It appears that the berms in the transverse direction of the southeast approach fills were not fully developed to the required details in this portion. However, elsewhere the berms were constructed as per contract drawings.

The substructure of the twin overpass structures at Hwy. 427 and Disco Road were completed at the time of failure. The overpass footings were supported on end bearing piles driven into the competent glacial till deposit; whereas, the retaining wall foundations were supported on spread footings and the box culvert on mat foundations, both being founded within very stiff to hard glacial till. The foundations of the above mentioned structural elements were constructed as per the recommendations contained in the Foundation Reports. (Twin Overpass Structures - W.P. 387-65. Twin Box Concrete Culvert and Related Retaining Walls - W.P. 280-65)

The distressed area and location of the retaining walls and box culvert together with other pertinent details are shown on Drawing No. 7601-A of Appendix II.

DESCRIPTION OF THE DISTRESS

The first signs of distress became evident on November 22, 1976. This was shortly after the completion of the retaining wall backfilling operation and after the south approach fill of Hwy. 427 was completed. Panels 5 to 8 of Retaining Wall 'B' and for a distance of 100 feet westerly of the south wall of the box culvert showed extensive cracks and excessive deformation.

The cracks in the distressed portion of Retaining Wall 'B' and the box culvert have been recorded and also photographed by the Soil Mechanics Section for documentation. The typical pattern of cracking of the distressed areas are shown in Drawing No. 7601-B of Appendix I and may be briefly described as follows:

a) Culvert Southwall: On the front face of the wall, wide horizontal cracks up to 3/16" appeared at the top for a distance of about 30 ft westerly from the downstream end and numerous horizontal cracks 1/32" to 1/8" wide occurred 5 to 10 ft. above the floor for a distance of about 100 feet (2 sections) from the downstream end. In addition, spasmodic hairline cracks were also found along the wall for about 425 ft. Wide cracks and serious spalling of the concrete 5 to 6 ft. below the top of the box culvert deck were visible at the backface for a distance of about 30 ft. westerly from the downstream end.

b) Retaining Wall 'B': Panels 5 to 8 deformed and cracks were evident on both front and back faces of the wall. On the front face, delamination of concrete could be spotted at the level immediately above the channel lining floor. In the lower portion of the backface, reinforcement was exposed due to spalling of concrete and bent as a result of shifting forward of the top portion of the wall.

   In the upper portion of Panel 8 adjacent to the culvert, extensive diagonal cracks were noted and they appeared to have progressed from the culvert through the retaining wall - culvert wall interface. With regard to these observations reference should be made to the photographs and sketches contained in Appendix I.

REVIEW OF THE BACKGROUND INFORMATION

A summary of the results of our review of the background information is given below:

Review of Foundation Investigation and Design Reports

Foundation investigations for the design of Mimico Creek culvert, the related retaining walls and the southeast approach fills for Hwy. 427 were carried out by the Soil Mechanics Section during the periods January 19, 1972 to February 11, 1972 and April 4, 1972 to April 13, 1972. Factual data together with our recommendations were submitted in Foundation Reports for W.P. 280-65-00 issued on April 27, 1972 and for W.P. 387-65 issued on June 11, 1973.

The investigations revealed that the predominant deposit at the site is a very stiff to hard glacial till 30 to 60 ft. thick, which is composed of a heterogeneous mixture of clayey silt, sand and gravel. Occasional random water bearing sand and gravel layers were also encountered in this deposit. At certain locations, the upper 3 to 4 ft. of the glacial till has been softened somewhat by weathering and the material in this zone has a firm to stiff consistency. The glacial till deposit is underlain by shale bedrock of the Dundas-Meadford formation.

Observations indicated at the time of the investigation, the groundwater level was found to lie between elev. 500 to 521, which corresponds to depths ranging from 2-23 ft. below existing ground surface. Except in the high ground the water level was found to correspond closely to the water level in the creek which was at about elevation 502. This indicates a gentle hydraulic gradient towards Mimico Creek.

The recommendations contained in the various foundation reports for the design and construction of the box culvert, the retaining walls, and the associated earth works are as follows:

Box culvert: It was recommended that the twin cell box culvert be founded within the competent glacial till deposit with an allowable bearing pressure of up to 2½ tsf provided any water bearing pockets or softened zones are subexcavated and replaced with mass concrete. Further the report stated that it may be advantageous to protect the subsoil at the footing foundation level by covering it with a lean concrete working slab immediately after completion of the excavation. For a continuous mat foundation of the size contemplated, imposing the aforementioned pressure it was estimated that the settlement would not exceed ½ inch and would be of a recompression nature.

<u>Retaining walls</u>: It was recommended that the retaining walls be supported on spread footings within the till below the frost penetration zone, using an allowable bearing pressure of up to 3½ tsf if proper foundation treatments such as prevention of ground softening and subexcavation of softened zones as outlined previously were observed. Settlements were estimated to be less than ½ inch.

<u>Other considerations</u>: The following is an excerpt from the foundation report concerning the recommendations pertaining to the earth pressure computations for the retaining wall.

"All the walls will be inherently stable with respect to a deep-seated rotational type of failure within the foundation subsoil.

"In computing the sliding resistance between the base of the rough concrete footing and i) the cohesive glacial till an adhesion value of 3,000 psf should be employed, and ii) granular fill a coefficient of friction of 0.65 should be employed.

"If the structures are designed as rigid frames, then a coefficient of earth pressure at rest (Ko) of 0.5 should be assumed for the granular fill material placed behind the wall when designing the sections. However, if some movement of the top of the wall is permitted, then a coefficient of active earth pressure (Ka) of 0.33 can be used.

"In order to relieve the buildup of excess hydrostatic pressure behind the retaining walls, suitable measures should be provided. Backfill behind the retaining walls should be carried out in accordance with current D.T.C. practices, specifically Standard #S.D. 4-58."

Approach fills: Stability of the approach fills were analysed by Janbu's method, using the following soil parameters:

	Earth Fill	Parent Glacial Till
Bulk Unit Weight (γ)	140 pcf	140 pcf
Apparent Angle of Internal Friction (ϕ')	27°	27°
Apparent Cohesion(c')	150 pcf	500 pcf

In the analysis, the full effects of the induced porewater pressure in the fill were also considered.

To ensure the safety and satisfactory performance of the fills, the following recommendations were made:

- In the Mimico Creek Valley floor between Sta. 629+60 and 695+00, the upper 3 to 4 ft. of the softened glacial till should be subexcavated as it is in a softened condition. (Refer to Contract Drawing Sheet No. 72-2)
- Wherever the fill heights are in excess of 20 ft., topsoil within the plan limits of the embankment should be stripped.
- In areas where fills are to be placed on existing slopes of the Mimico Creek Valley, topsoil should be stripped and the new fills should be benched into the banks in accordance with current MTC practices. (Refer to M.T.C. Standard DD-414)
- Earth fills should be placed and compacted at a moisture content within 1% of the optimum moisture content, as determined by Standard Proctor Compaction test.
- Fills up to 52 ft. should be constructed with a side slope not steeper than 2:1. The proposed grade for the transverse slope, which incorporates a mid-height berm of varying width, was shown in Contract Drawing Sheet No. 52 and also the pertinent cross-sections was shown on Sheet 72-2.
- It was assumed in the Foundation Report that the following construction procedure would be followed.

Mimico Creek would be diverted through the box culvert - open channel sections as soon as they have been completed. Following this operation the obsolete portion of the existing channel would be backfilled. Prior to placing any fill in this area all organic or unacceptable material present would be completely subexcavated.

Review of the Structural Design

The cross-section dimensions and length of the culvert were designed to meet the road geometric and hydrology requirements. The structural design was in accordance with the Structural Office's standard practice in culvert design. In this project, the culvert was designed for the following service loads.

a) Granular Backfill with Bulk Density of 120 pcf
b) Lateral Earth Pressure equivalent to Fluid Pressure of
   36 psf (with no surcharge)
   45 psf (with 2:1 slope surcharge)
c) H20-S16 Truck (no impact)

The design and analysis were carried out with the aid of a computer program for culvert design (Program No. 140).

The retaining wall was designed for a lateral earth pressure of 45 psf which includes a 2:1 slope surcharge. The proportioning of the dimensions and the detailing of the reinforcement were prepared with the aid of the Structural Office's standard design table for retaining walls and a computer program was used for checking. The recommended allowable bearing capacity was used for the design of the culvert foundations and also retaining wall foundation.

Sequence of Construction

The sequence of construction for the culvert, in the distressed area from Sta. 21+18 to Sta. 25+41 and for Retaining Wall 'B', as recorded in the Ministry Construction Inspectors' Diaries, are summarized in the Appendix III of this report. In general, construction for the culvert commenced from its centre and proceeded towards the east, thence to the erection of Retaining Wall 'B'. After the completion of Retaining Wall 'B' the western portion of the culvert was constructed.

The culvert construction began with excavation at Sta. 25+50 slightly beyond the distressed area on August 3, 1976. The eastern portion of the culvert was constructed in 9 sections about 50 ft. long each. The construction of each section proceeded according to the following typical schedule:

Day 1 Excavate for the Culvert
Day 2 Pour Concrete Floor
Day 4 Strip Floor Forms
Day 10 Pour Concrete Wall
Day 11 Strip Wall Forms
Day 31 Pour Concrete Deck
Day 33 Strip Deck Forms
Day 35 Backfill behind Concrete Walls
Day 37 Backfill on Deck

On October 25, 1976, prior to completion of the construction of the box culvert, construction began for Retaining Wall 'B'. This retaining wall, 240 ft. long, consists of 8 panels tapering from a wall height of 23.5 ft. down to 4 ft. The construction of each 30 ft. long panel proceeded according to the following typical schedule:

Day 1 Excavate for Retaining Wall
Day 2 Pour Concrete Footing
Day 3 Strip Footing Forms
Day 4 Pour Concrete Wall
Day 5 Strip Wall Forms
Day 6 Backfill behind Wall

Construction Equipment

Two types of heavy compaction machinery were used; one being a Bomag BW 210 drum drive vibratory roller which delivers a total force static plus dynamic of 52 kips over an 84 inch wide drum, the other type being a Hyster C 450 A capable of delivering a static force of 60 kips over an 80 inch wide drum. The pertinent technical literature related to the respective equipment is contained in Appendix IV. To take advantage of the high compactive energy of these equipments, the fill was compacted in lifts up to 4 ft. thick. It was also ascertained that these compactors had been used in very close proximity of the retaining wall, as well as the box culvert.

POST FAILURE INVESTIGATION

An investigation was initiated immediately after failure had occurred in order to assess whether all pertinent recommendations contained in the Foundation Reports were carried out during construction. A field investigation was carried out to determine the engineering properties of the fill material, as well as backfill material for embankment constructions and the associated culvert and retaining wall structure. The results of the field work may enable us to re-assess the stability of the earth fills in the retained areas and also to evaluate the most probable earth pressure exerted on the retaining structures. This will give us an insight as to the causes of the failure of the culvert, as well as the retaining wall.

Immediately after the failure the investigation was carried out on November 30, 1976 and also from December 10 to December 18, 1976. The subsurface investigation consisted of a total of 21 sampled boreholes. All borings were advanced by a continuous flight auger machine equipped with a 3¼" I.D. hollow stem auger, except B.H. #501 which was advanced by washboring techniques. Locations and elevations of all the boreholes carried out during the design stages, as well as subsequent to the failure, are shown in Drawing No. 7601-A. The post failure subsurface investigation revealed the following:

It was found that the approach embankments were constructed with a material composed of a cohesive glacial deposit consisting of clayey silt with some sand and a trace of gravel. It was also established that the backfill for the retaining structures is composed of two granular types of material, one being a fine crushed stone and the other a silty sand with gravel. The approach fill and the backfill were placed directly on the parent glacial till. However, at certain locations near the original Mimico Creek where subexcavation of organic or softened material was specified, isolated inclusions of organic material were observed at the contact between the parent glacial till and the fill material.

The detailed descriptions of various soil types encountered in the post failure investigation, together with their geotechnical properties, are as follows.

Embankment Fill

The embankment fill is composed of clayey silt of a glacial origin containing some sand and a trace of gravel. Typical grain size distribution curves for the embankment fill material are shown in Fig. 1 in an envelope form. This fill material has a typical liquid limit of about 17% to 28% and a typical plastic limit of about 10% to 16%. These results, when plotted on a plasticity chart,(Fig. 2)fall in the CL zone, indicating that the fill is inorganic and has a low plasticity. The in-situ undrained shear strengths as measured by field vane tests, were found to be in excess of 2,000 psf. The effective shear strength parameters were determined by carrying out consolidated undrained triaxial tests with pore pressure measurements (C.I.U. tests). These tests gave the following results (Fig. 3).

	Range
Effective Cohesion Intercept (C')	450 - 600 psf
Effective Angle of Shearing Resistance (Ø')	26 - 27.5 degrees

The field moisture contents for this fill material range from about 9% to 14%, which indicates that the fill material was compacted within 1 to 2% of the optimum moisture contents (10-12%). The in-situ density of this material was found to be ranging from 130 pcf. to 143 pcf. with an average value of 135 pcf. The Standard Penetration Test 'N' values range from 13 to 20 blows per foot. These values indicate that the fill material was well compacted over the entire area.

In certain locations (Refer B.H. 505A, 506, 507, 509, 530) immediately below the embankment material, isolated inclusions of organic material up to 5 feet thick were encountered at the contact with the parent subsoil. This indicates that in the vicinity of the old creek area subexcavation specified in the contract drawings has not been fully carried out.

Granular Backfill Behind the Retaining Wall and the Box Culvert

The granular backfill on this project is comprised of two types of materials used randomly and the geotechnical properties of these materials are described below.

Crushed stone (source: Indusmin From Acton)

Typical grain size distribution of the crushed stones are shown in Fig. 4. It is to be noted that this material contains 13% to 28% particles finer than silt size (i.e. Sieve #200). The maximum bulk density ranges from 147 pcf to 150 pcf with corresponding optimum moisture contents ranging from 7.3% to 7.9%. The falling head permeability test carried out in the laboratory indicated that the permeability is in the order of 5×10^{-5} cm/sec. Consolidated undrained triaxial with pore pressure measurements tests (C.I.U. tests) were carried out on Proctor moulded samples. The results on Fig. 5 show that the material has an effective angle of shearing resistance of 45°.

Silty sand with gravel (source: From Brampton and Milton)

The second type of backfill is of glacial origin and may be described as silty sand with gravel containing numerous cobbles and trace of clay. Typical grain size distributions of this material are shown in Fig. 6 & 7 in an envelope form. It is noted that this material also contains about 17% to 25% particles passing sieve #200. Standard and Modified Proctor compaction tests have been performed on this material. The results are summarized below:

	W_{opt} (%)	Max. Bulk Density (pcf)
Standard Proctor	6.1 to 6.3	149 to 151
Modified Proctor	4.8 to 5.1	151 to 153

This material at Proctor density was so impervious that a back pressure of 70 psi had to be applied in order to induce a flow of water through it. The permeability of the saturated specimen as measured by this method was found to be in the order of 4×10^{-7} cm/sec. During the permeability test it was observed that this material has a swelling ratio of 1.5% upon soaking. Consolidated undrained triaxial tests with pore pressure measurements were carried out on Proctor moulded samples of this material and are shown on Fig. 8. The tests gave the following shear strength parameters in terms of effective stresses:

Effective Angle of Shearing Resistance	$\phi' = 37°$
Effective Cohesion Intercept	$c' = 720$ psf

The Standard Penetration Test 'N' values were found to be in excess of 100 blows per foot which indicates that this backfill material has been subjected to a very high degree of compaction.

Since there were no records of the in-situ field densities of the backfill it was not possible to compare the in-situ field densities with the Proctor Densities as determined in the laboratory.

Groundwater Condition

Groundwater elevations at the time of post-failure investigation were found to be at or above the original ground elevation in the vicinity of Mimico Creek. Elsewhere, even in the area of the high embankment, there was no water between the natural ground and the fill material.

At the time of failure, the granular backfill was excavated and found to be relatively dry.

DISCUSSION OF FAILURE

In order to investigate the reasons for the failure, the following probable causes have been studied:

1. Stability of the embankment
2. Strength of the Concrete
3. Earth Pressure
    a) Effects of Sloping Surcharge on Retaining Walls
    b) Effect of Sloping Surcharge on Box Culvert
    c) Effects due to Compaction Equipment
    d) Effects due to Compaction

1. Stability of the Embankment

The stability of the constructed embankment in relation to the retaining wall 'B' and adjacent box culvert was investigated. The conclusion of the analyses is that the earth fills are inherently stable with regard to rotational or sliding types of failure, inspite of the presence of localized organic layers which were not fully subexcavated in the area of the old creek channel.

The parent subsoil is competent. The field observations, together with the results of the post failure investigation, confirmed that there is no possibility for a bearing capacity failure of the foundation.

2. Strength of the Concrete

In order to assess whether the failure may be due to low strength of the concrete the compressive strength of the concrete used for the retaining wall and box culvert was examined. The design requirements call for a minimum strength of 4000 psi for the box culvert and 3000 psi for the retaining wall. The compressive strengths of the concrete test cylinders obtained from the Regional Quality Assurance Office are as follows:

Ages (in Days)	Range (psi)	Average (psi)
28	5000 to 7500	6000
4	--	2800 (one test)
2	2700 to 3300	3000 (Considered to be)

The above results show that the concrete had attained sufficient strength. Consequently, the failure is not attributed to lack of compressive strength of the concrete.

3. Earth Pressure

The earth pressure on a retaining structure is affected by a number of factors, among which the deformation characteristics of the wall, the types of backfill material and the method of placing the backfill. An evaluation of the most probable earth pressure acting on the distressed retaining structures has been back calculated using the results of field and laboratory data.

3-a) Effect of sloping surcharge on retaining wall: A cantilever retaining wall is usually designed to withstand an active earth pressure because of its flexibility to rotate above the base. The amount of deformation at the crest of the wall required for the retained soil mass to mobilize its shearing resistance, thereby reducing the earth pressure to an active one, is very small (generally about 0.002 H for granular soil with no sloping surcharge). Deflection of such magnitude can be easily attained by a cantilever retaining wall. The lateral force acting on a retaining wall due to active earth pressure quite often is evaluated from the limit equilibrium of the retained soil mass. In such an analysis, a slip surface is assumed to have developed in the soil mass, causing the soil immediately behind the wall to slide down along the slip surface, thereby exerting a force on the retaining wall. In the analysis of the state of limit equilibrium, it is also assumed that Mohr-Coulomb resistance is mobilized along the slip surface. A variational method given in the Appendix V is used to locate the most critical slip surface which would induce the largest active force on the wall. Once the location of the slip surface has been determined, the lateral force, together with its point of application, can be obtained by the method of slides. The results carried out for a 25 foot wall retaining a soil mass with the following parameters

γ = 135 pcf
c = 150 psf
ϕ = 27°

and a 2:1 sloping surcharge are summarized in Fig. 9, 10, 11 . In the computation, a cohesion intercept (c) of 150 psf was used in order to account for the residual or the post-peak condition and the effect of the rate of strain on the shear strength parameters. It was found that the lateral force has a magnitude of 18000 lb. acting at a point about 13.7 feet above the base of the wall. This lateral force under the above mentioned condition can also be stated as an equivalent fluid pressure in the order of 60 psf which is 33% greater than the pressure used for design. Further, the point of application of this lateral force was also found to be much higher than the 1/3 height of the wall originally assumed in the design of the retaining wall (refer to Fig. 11).

3-b) <u>Effect of sloping surcharge on box culvert</u>: The box culvert is a relatively rigid structure. Its deformation is usually not sufficient for the retained soil mass to mobilize its shearing resistance. Under the circumstances where little or no lateral deformation is allowed, the horizontal earth pressure acting on the wall is usually approximated by the lateral stress within the soil mass under an at-rest condition.

This lateral stress σ_h at any depth z below ground surface can be evaluated from:

$$\sigma_h = Ko\ (\gamma z + \Delta\sigma_v)$$

where $\Delta\sigma_v(z) = \int I_v(z,x)\ q(x)\ dx$

is the vertical stress increment induced by the surcharge q(x), which in this case is the 2:1 approach fill. In the above equation, $I_v(z,x)$ is the influence function for the vertical stress.

Using $K_o = 0.5$, $\gamma = 140$ pcf and evaluating I_v numerically from Boussinesq's theory, it is found that, for practical purposes, the lateral stress in this case can be estimated from

$\sigma_h = 70\ z + 375$ (psf) for $z \geq 7$ ft. (thickness of earth cover for the box culvert)

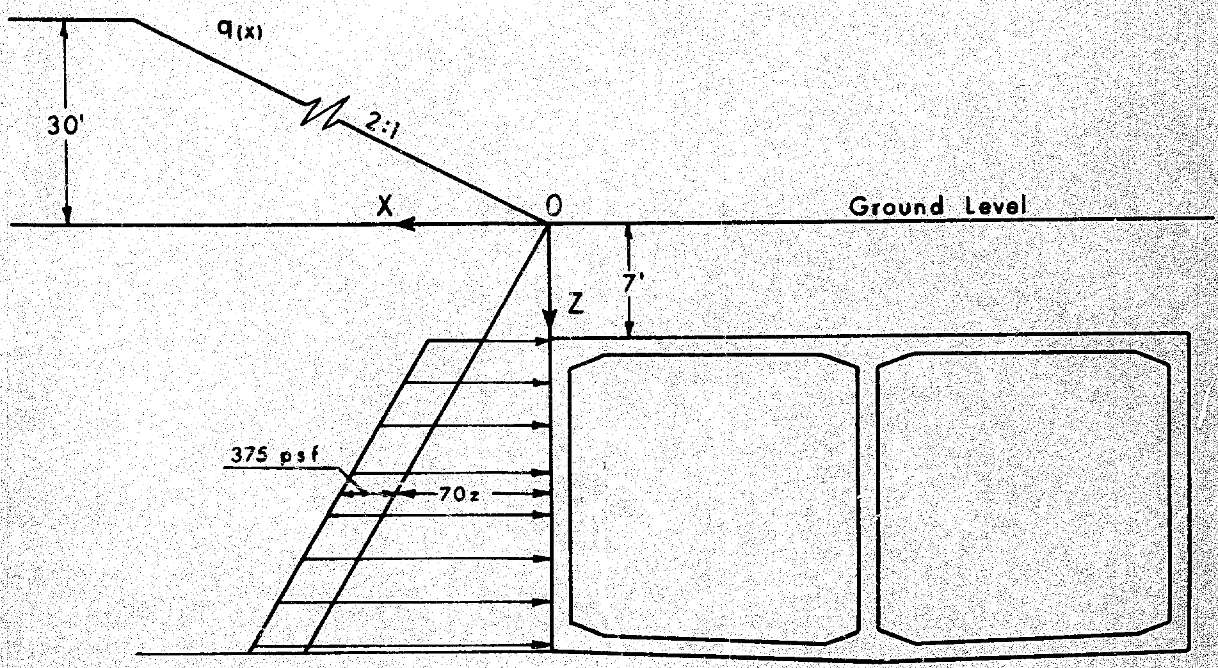
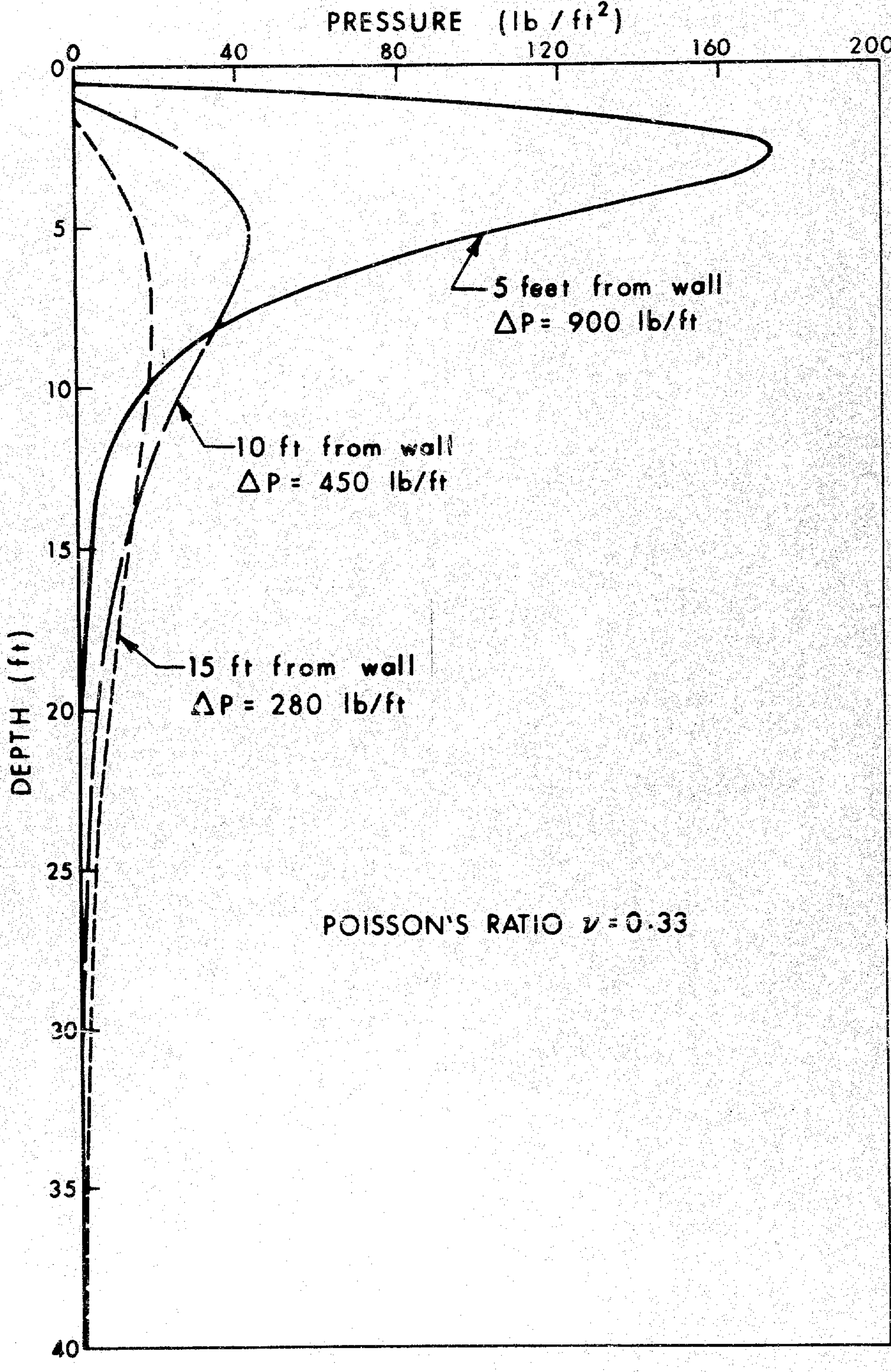


Fig. (A) Lateral Force on Box Culvert

The lateral force exerted on the twin box culvert, taking into account the sloping surcharge of 2:1, together with the deformation characteristics of this structure, were found to be much higher than the 45 psf equivalent fluid pressure used in the design.

3-c) Effect due to compaction equipment: As mentioned previously, heavy equipment was used to compact the granular backfill and the associated high sloping earth embankment behind the retaining structures.

The earth pressures induced by the compacting equipment at various distances away from the wall was computed by Boussinesq's theory, using a Poisson's ratio of 1/3 for the compacted fill. The additional lateral earth pressure distribution due to the heavy equipment is given in Fig. B. The results indicate that the additional lateral earth pressure is highly concentrated near the top of the wall.



Earth Pressure on Retaining Wall induced by Concentrated Load of 20 Tons.

3-d) Effect due to compaction: Compaction will generate high horizontal stresses. These stresses may be retained in the soil mass as locked-in residual stresses and may be accumulated to a very high value. These residual stresses are affected by a large number of variables among which include roller size and weight, roller frequency and forward speed, lift height, number of roller coverages, gradation of the fill and its moisture content. Results of research on residual stresses caused by compaction published by Sowers et al 1957, D'Appolonia et al 1969, Gould 1970, Casagrande 1973, Aggour & Brown 1974, Ingold 1977 may be summarized as follows:

- Overcompaction or the use of vibratory equipment will result in a residual stress considerably greater than the at-rest pressure. If the backfill is well graded and contains a high percentage of fines, even the use of light vibratory machinery will cause the lateral pressure greater than the Ko value.

- Compaction is a form of "over consolidation". Therefore, the resultant Ko values are much higher than those of a "normally consolidated" soil. It is quite often that Ko of a compacted fill attains a value greater than one. (Fig. C)

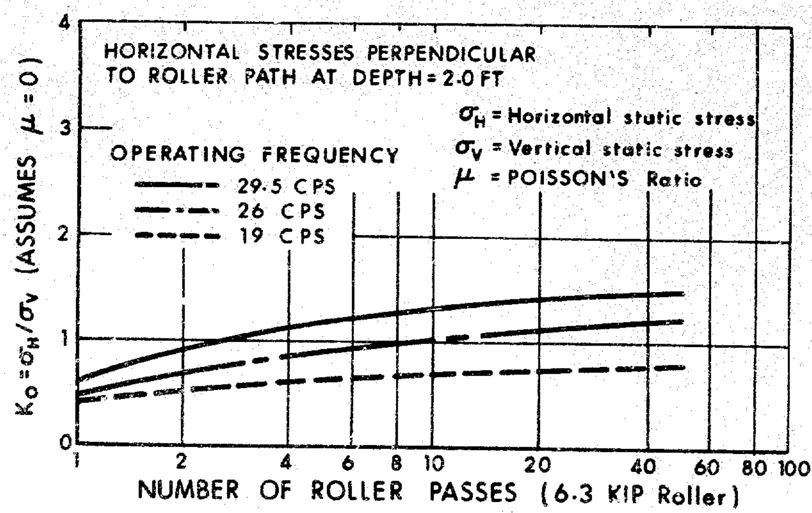


Fig. (C) Variation of K_o with respect to the number of roller passes and operating frequencies
(From D'Appolonia et al 1969)

- The residual stresses generally assume a distribution with depth similar to that as shown in Fig. (D), i.e. heavily concentrated near the top of the wall.

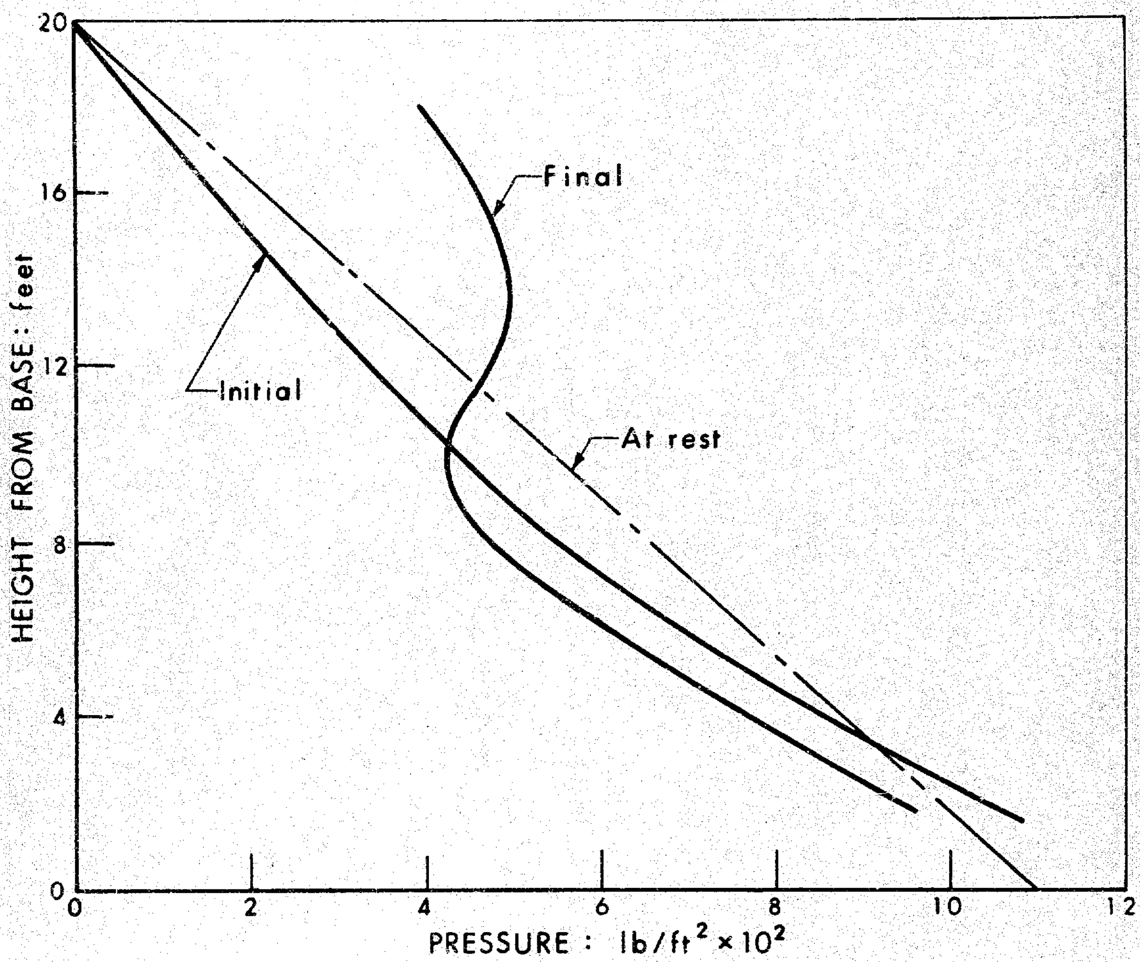


Fig. (D) Distribution of Residual Stresses (From Aggour & Brown, 1974)

- While the residual stresses do not appear to change appreciably with time in coarse granular material following the compaction, for fill containing a high percentage of fines, the lateral stresses may increase with time due to the gradual release of the locked-in energy.

- The residual stresses are anisotropic. The maximum value occurs on a plane perpendicular to the direction of the roller advance while the minimum occurs on the plane parallel to the roller path (Fig. E).

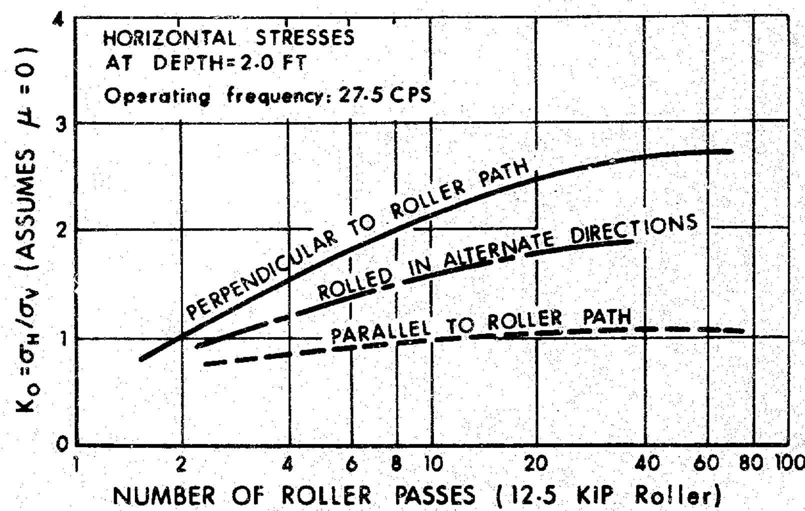


Fig. (E) Variation of K_0 with respect to the number of roller passes and Anisotrophy of K_0 (From D'Appolonia et al 1969)

Reasons For Failure

The aforementioned discussions show that the primary cause of the failure of the retaining structures was not due to instability of the high embankment in this area or due to the quality of the concrete. The various analyses, together with the patterns of crack formation, indicate that the distress was due to high lateral earth pressures exerted on the retaining structures (Retaining Wall B and the Twin Box Culvert South Wall). The high lateral pressures are mainly due to a combination of the following factors:

1. Surcharge due to high sloping backslope
2. Overcompaction due to heavy vibratory compaction equipment.
3. Improper backfill material.

As mentioned before, Retaining Wall B and the Twin Box Culvert were designed for an equivalent surcharge of 2 feet which will exert a fluid pressure of 45 psf. The results of our earth pressure computation which takes into account the full effect of the sloping surcharge, indicate that the lateral forces, together with the lever arm of the point of application, are much greater than the values assumed in design. The combination of these could produce a maximum theoretical flexural stress as much as 100% larger than the design values. However, this aspect alone should not result in the failure of the retaining structure. The ultimate capacity of the wall calculated for the sections where the cracks occurred (i.e. immediately above the floor lining and immediately above the cut-offs of the F reinforcing bars) still has a factor of safety greater than one with respect to the bending moment caused by the lateral force induced by the retained soil mass (Appendix VI).

The heavy vibratory compacting equipment used in this project resulted in over-compaction of the granular backfill and the related earth fill. This method of construction contributed to large horizontal stresses in the soil mass. These residual (locked-in) stresses induced additional lateral earth pressures on the retaining structures. As mentioned previously, the walls would be subjected to high flexural stresses due to sloping surcharge. These additional lateral earth pressures induced by heavy vibratory compacting equipment, together with the sloping surcharge, resulted in the failure of the retaining structures.

In the distressed area, well-graded granular material containing a high portion of fines was used as backfill. This material is very easy to compact and attains a very high density when compacted. Further, because of the high degree of interlocking of particles, the compactive energy transmitted to this material will be locked in and accumulated. Consequently, very high horizontal residual stresses are resulted in the soil mass.

CORRECTIVE MEASURES INCORPORATED DURING CONSTRUCTION

Since the cause of failure was established as excessive earth pressure, the immediate step was to release the earth pressure acting against the culvert and the retaining wall.

To this purpose, the following measures were taken:

1. Removal of the approach fill immediately behind the culvert and retaining wall was recommended. This was achieved by reducing the width of the approach fill of the overpass, as the required width was designed as a provision for future widening of Hwy. 427 roadway in the ultimate scheme. As a result, the toe of this slope was moved 15 feet to 55 feet away from the back of the culvert and the retaining wall.

2. A trench was excavated behind the culvert to a depth of about 3 feet from the bottom of the culvert. This space was backfilled with sand and gravel which was loosely placed to relieve the residual stresses induced by compaction. The pertinent details are shown in Drawing 7601-A.

In addition, the damaged retaining wall and the culvert wall were repaired. The repair works involved the following:

1. The 105 feet of damaged retaining wall was removed to 2 feet above the footing and replaced with new concrete and reinforcing steel.

2. The end 30 feet of the east wall of the culvert was partially removed and replaced with new concrete and reinforcing steel.

3. A relieving slab 14 inches thick was built against the outside wall of the culvert to strengthen the culvert wall. (Sta. 21 + 50 to Sta. 23 + 20).

To find out the actual earth pressure acting against the culvert and retaining wall, the Research and Development Branch carried out measurements and installed strain gauges in the new retaining wall, part of the culvert and the relieving slab. The instrumentation layout and the related data is given in Appendix VII of this report. The readings obtained up to May, 1977 indicate that subsequent to the repair works, only small forces were acting on the retaining structures. Further monitoring will be carried out by R & D in order to establish definite conclusions with regard to pressures exerted against the structure.

RECOMMENDED PREVENTIVE MEASURES

The failure of the retaining wall and box culvert has revealed certain inadequacies in the present Ministry Design Standards and Procedures pertaining to retaining structures. To prevent recurrence of similar failures the following measures are suggested for design and construction:

1. The lateral earth pressure is known to be affected by a number of factors, among which are the deformation characteristics of the retaining wall, types of backfill material, method of placing the backfill, surcharge conditions, etc. If the wall is of a rigid type, the earth pressure at rest (k_o) should be used. In addition, the full effect of any sloping surcharge should be taken into account by using any well known method of earth pressure computation

2. The process of compaction, under certain conditions of lift thickness, roller weight, vibration frequency and amplitude, and other variables, generates incremental earth pressures that often exceed the earth pressure at rest. To prevent forces on retaining structures from attaining excessive levels due to such incremental earth pressures, the following interim criterion is

suggested. Vibratory equipment exceeding 12.5 kips static weight should be kept outside of a 1½ vertical to 1 horizontal line extending upward from the base of the retaining wall stem. Further research will be required to confirm this criterion.

3. The use of granular materials that contain a high percent of fines, (e.g. Granular D) should be discouraged* since they tend to over compact easily and do not drain as freely as desired.

4. On future similar projects the backfill should be placed prior to construction of any restraint,such as a channel lining at the base of the wall to allow necessary movement of the wall in order to develop the active earth pressure case. In general, any hindrance to the yielding of a retaining structure under backfilling operations should be reduced or minimized as much as possible.

MISCELLANEOUS

This report was prepared by Mr. B. Ly, Senior Engineer, Soil Mechanics Section, with the assistance of Mr. M. MacLean, Project Engineer. The review of the structural design was done by Mr. W. Lin, Regional Structural Design Engineer, Structural Office. This report is a joint effort of the Soil Mechanics Section and the Structural Office and was reviewed by Messrs. M. Devata and C.S. Grebski.

Walton Lin

W. Lin, P. Eng.
Regional Structural Design Engineer
Structural Office

M. Devata

M. Devata, P. Eng.
Supervising Engineer
Soil Mechanics Section

June, 1977

* A joint study between Central Region and the Soil Mechanics Section is in progress to determine appropriate grain size and compaction criteria for retaining wall backfills.

REFERENCES

Aggour, M.S. and Brown, C.B. (1974). The prediction of earth pressure on retaining walls due to compaction. Geotechnique Vol. 24, No. 4, pp. 489-502

D'Appolonia, D.J. et al (1969). Sand compaction with vibratory rollers. Proc. of ASCE J.F. SM & FE, Vol. 95 SM1, pp. 263-284

Casagrande, L. (1973). Comments on convertional design of retaining structures. ASCE, J. of SM & FE, Vol. 99 SM2, pp. 181-198

Gould, J. (1970). Lateral pressures on rigid permanent structures. ASCE (SM & FE) Special Conf. on Lateral Stresses & Earth Retaining Structure, p. 219-269

Ingold, T.S. (March, 1977). Performance of a retaining wall with deep foundation, Ground Engineer, p. 24-25

Sowers, G.F. et al (1957). The residual lateral pressures produced by compacting soils. Proc. 4th International Conf. SM & FE. Vol. 2. London, pp. 243-247

REFERENCES

Aggour, M.S. and Brown, C.B. (1974). The prediction of earth pressure on retaining walls due to compaction. Geotechnique Vol. 24, No. 4, pp. 489-502

D'Appolonia, D.J. et al (1969). Sand compaction with vibratory rollers. Proc. of ASCE J.F. SM & FE, Vol. 95 SM1, pp. 263-284

Casagrande, L. (1973). Comments on convertional design of retaining structures. ASCE, j. of SM & FE, Vol. 99 SM2, pp. 181-198

Gould, J. (1970). Lateral pressures on rigid permanent structures. ASCE (SM & FE) Special Conf. on Lateral Stresses & Earth Retaining Structure, p. 219-269

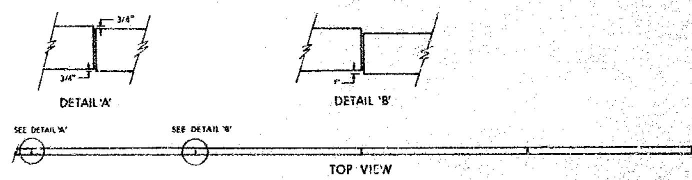
Ingold, T.S. (March, 1977). Performance of a retaining wall with deep foundation, Ground Engineer, p. 24-25

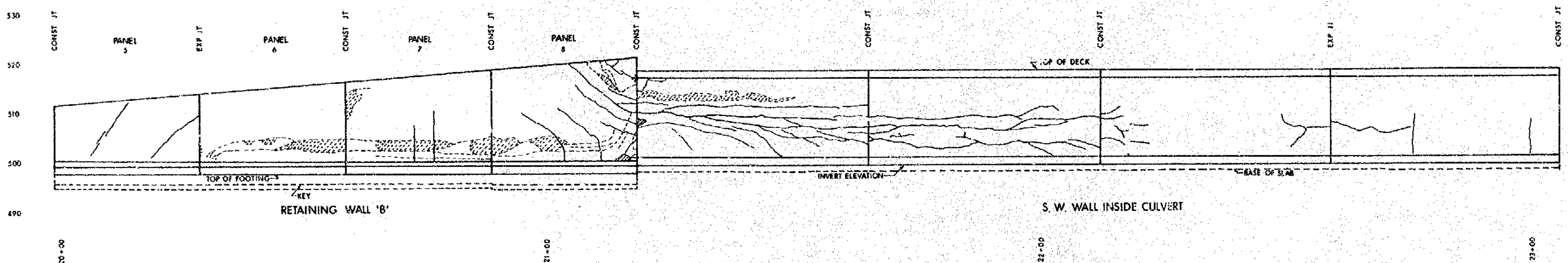
Sowers, G.F. et al (1957). The residual lateral pressures produced by compacting soils. Proc. 4th International Conf. SM & FE. Vol. 2. London, pp. 243-247

APPENDICES

APPENDIX I

CONT No 76-01 WP No 280-65	SHEET
MIMICO CR CULVERT & RETAINING WALL (HWY 427 AT DISCO ROAD)	





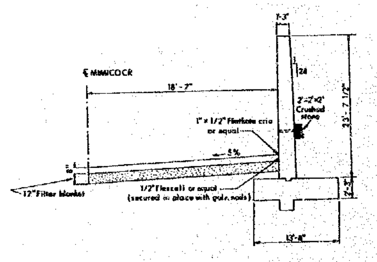
NOTES:

1. All crack mapping done Dec 9-24, 1976.
2. Mapping on back face of culvert as inferred from photographs.

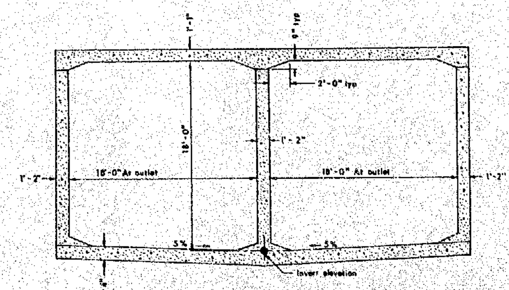
1"=10'-0"

LEGEND

- ——— Cracks on front face (stream side)
- - - - Cracks on back face
- Spalled areas



SHOWING RET WALL & CONC LINING AT PANEL 8 STATION 21+18



TYPICAL CULVERT CROSS SECTION

HWY No 427 AT DISCO RD			DIST 6
SUBM'D MM	CHECKED	DATE May 19/77	SITE 37-558
DRAWN JN	CHECKED	APPROVED	DWG 7601-B

PLATE 1: GENERAL VIEW OF THE SITE (LOOKING SOUTH)

PLATE 2: BACKFACE OF THE CULVERT AND THE PRESSURE RELIEF TRENCH

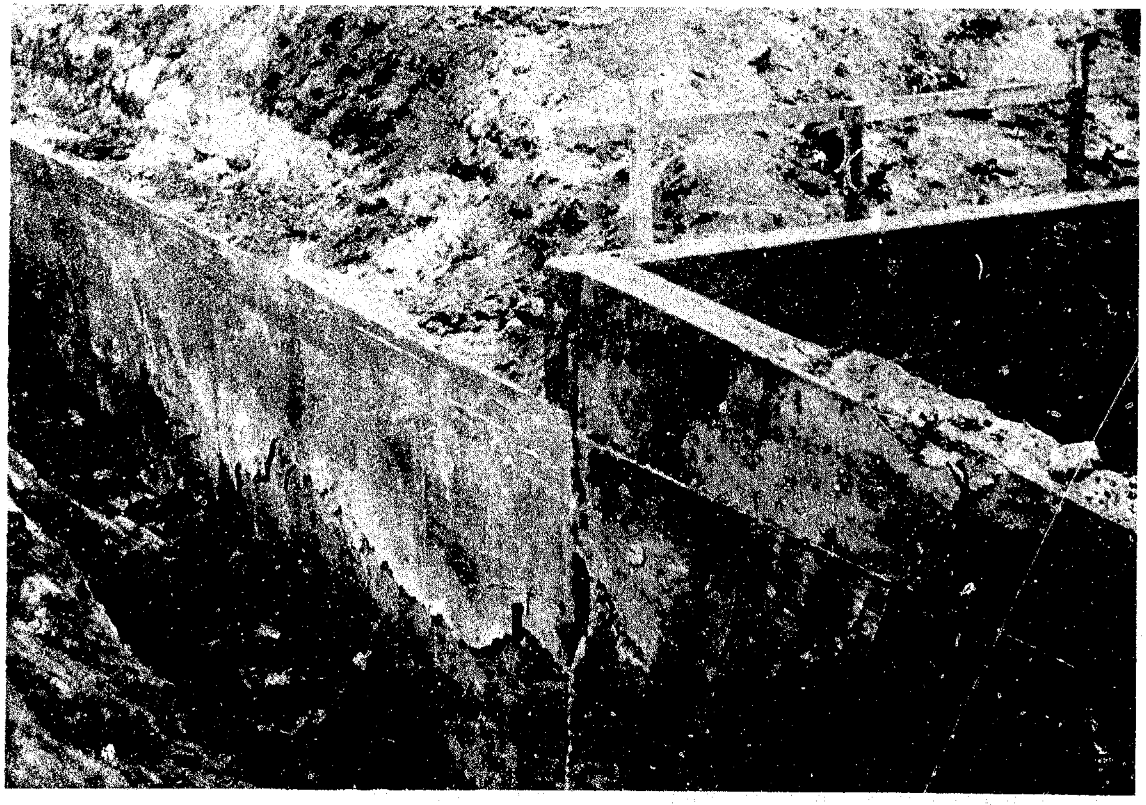


PLATE 3: BACKFACE OF THE CULVERT AND THE RETAINING WALL

PLATE 4: AT TOP OF CULVERT WALL

PLATE 5: FRONT FACE OF RETAINING WALL PANNEL '8'

PLATE 6: DELAMINATION OF CONCRETE IN THE FRONT FACE OF THE RETAINING WALL JUST ABOVE THE CHANNEL LINING

PLATE 7: CRACKS IN THE FRONT FACE IMMEDIATELY ABOVE THE CHANNEL FLOOR LINING

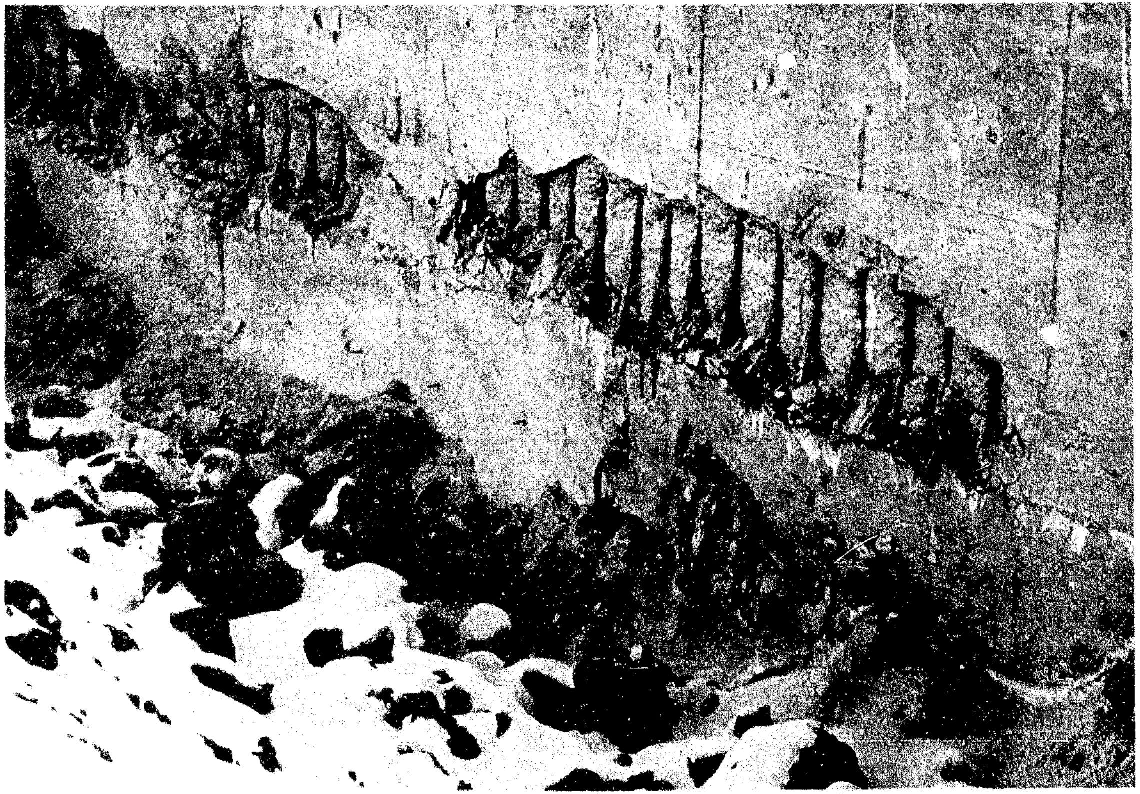


PLATE 8: SPALLING OF CONCRETE AND DEFORMED 'G' BARS IN THE BACKFACE OF RETAINING WALL (PANNEL 8)

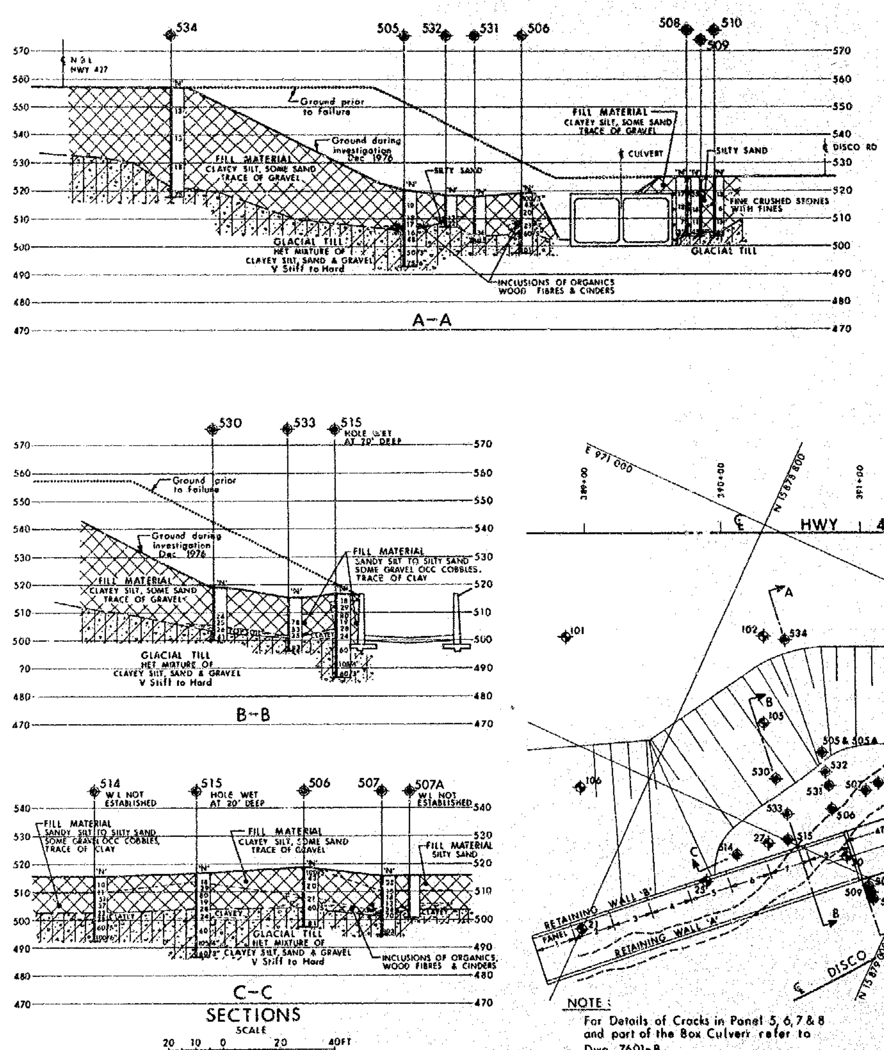
PLATE 9: SPALLING OF CONCRETE IN THE BACKFACE OF RETAINING WALL IMMEDIATELY ABOVE THE CUT-OFFS OF THE 'F' BARS

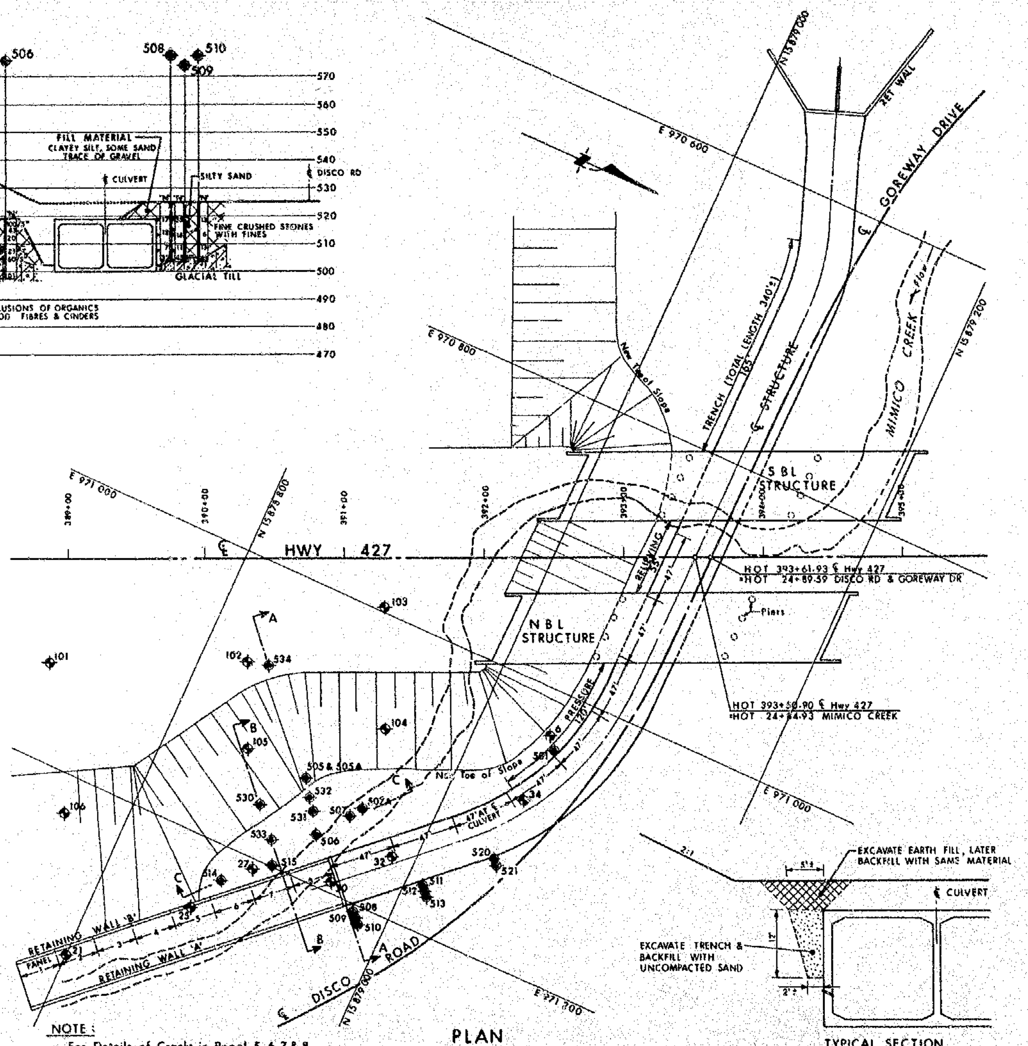
APPENDIX II

CONT No 76-01
WP No 280-65

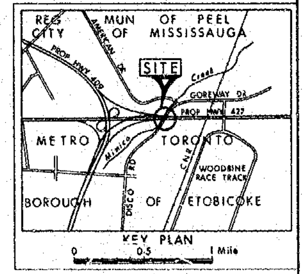
MIMICO CR CULVERT & RETAINING WALLS
(HWY 427 AT DISCO ROAD)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET





NOTE:
For Details of Cracks in Panel 5, 6, 7 & 8 and part of the Box Culvert refer to Dwg 7601-B



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- 'N' Blows/ft (Std Pen Test 350 ft lbs energy)
- CONE Blows/ft (60° Cone, 350 ft lbs energy)
- WL at time of investigation Dec 1976. B H # 506, 508, 509, 510 & 530 to 534 DRY on Completion
- Bore Holes put down for Post Failure Investigation

No	ELEVATION	CO-ORDINATES NORTH	EAST
501	525.5	15 879 058	971 040
505	520.9	15 878 905	971 130
506	519.1	15 878 928	971 166
507	515.8	15 878 945	971 143
507A	515.8	15 878 951	971 143
508	524.9	15 878 975	971 204
509	524.9	15 878 978	971 208
510	524.9	15 878 982	971 212
511	525.9	15 879 012	971 167
512	525.9	15 879 015	971 171
513	525.9	13 879 018	971 175
514	515.6	15 878 880	971 224
515	516.9	15 878 909	971 199
520	526.3	15 879 051	971 129
521	526.5	15 879 054	971 133
530	519.0	15 878 882	971 163
531	518.0	15 878 920	971 151
532	518.5	15 878 913	971 144
533	515.0	15 878 901	971 183
534	557.0	15 879 047	971 069

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

CONSTRUCTION PROCEDURE FOR PRESSURE RELIEVING TRENCH

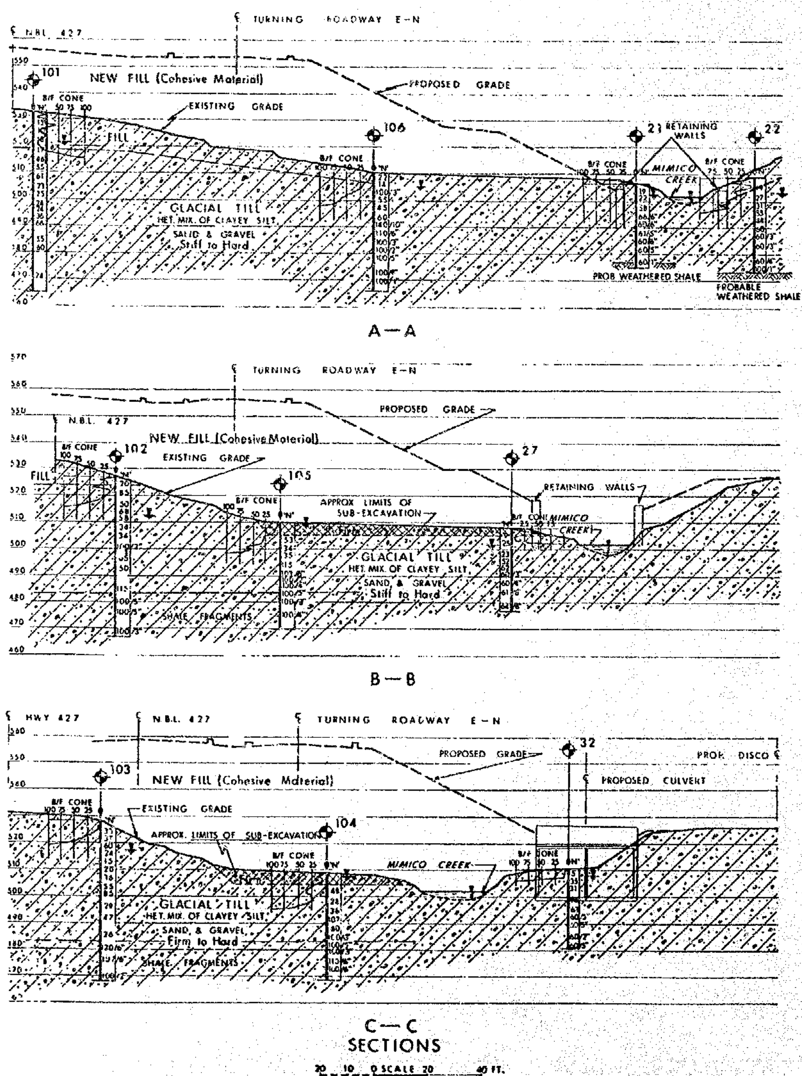
1. Excavate Earth to top of Culvert as shown.
2. Excavate Trench, as shown behind South side of Box for length shown in plan
3. Backfill Trench with Sand. (no compaction)
4. Replace Earth Fill using light compaction only.

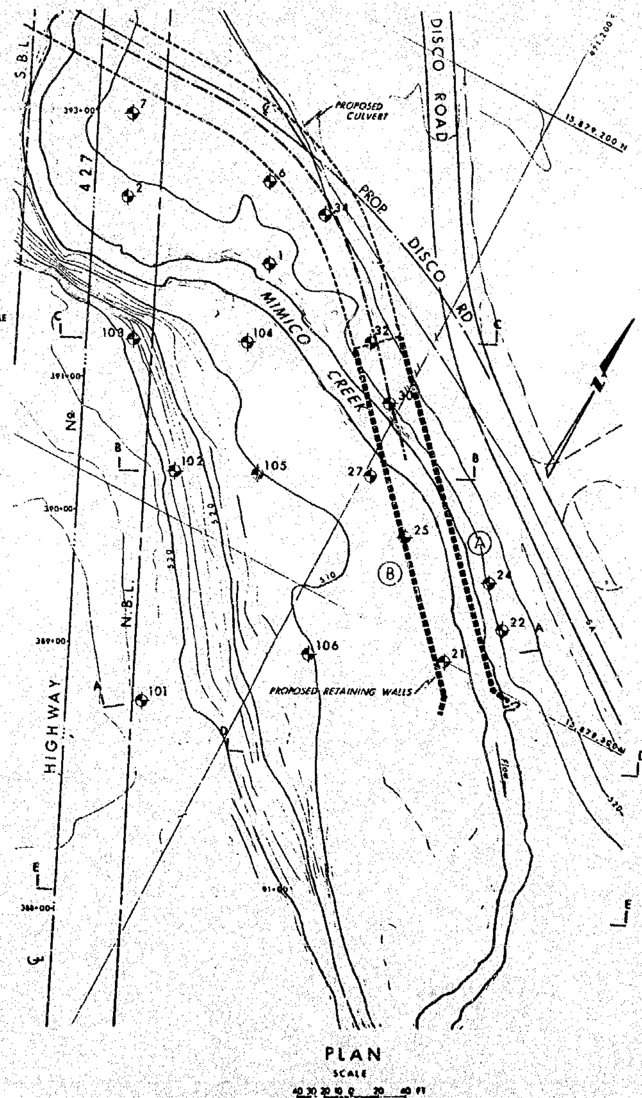
For safety reason the construction of the relieving trench shall be carried out in approx 25ft sections

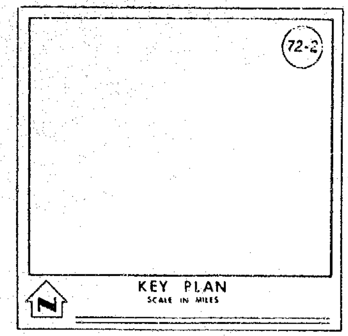
REVISIONS DATE	BY	DESCRIPTION

HWY No 427 AT DISCO RD			DIST 6
SUBM'D	CHECKED	DATE March 25/77	SITE 37-558
DRAWN	CHECKED	APPROVED	DWG 7601-A

REF No STRUCTURAL DWG 37-558-1, Dec 1976







LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation APRIL, 1972.

NO	ELEVATION	CO-ORDINATES NORTH	EAST
21	507.6	15,878,800	971,319
22	508.?	15,878,834	971,348
27	509.1	15,878,697	971,207
32	510.5	15,878,984	971,158
101	531.4	15,878,702	971,133
102	528.0	15,878,832	971,073
103	528.5	15,878,905	970,997
104	508.6	15,878,942	971,076
105	510.5	15,878,858	971,130
106	510.5	15,878,757	971,227

NOTE: The complete foundation file for this project may be examined at the Geotechnical Office, Downsview. Information in this file is specifically excluded in accordance with the conditions of subsection 102-2 of Form 100 as amended by the special provision "Contractor's Investigations" included with the contract documents

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO
ENGINEERING SERVICES BRANCH — GEOTECHNICAL OFFICE

DISCO ROAD & GOREWAY DRIVE

HIGHWAY NO 427 DIST. NO 6
MUN. OF METROPOLITAN TORONTO & PEEL
BOROUGH OF ETOBICOKE & CITY OF MISSISSAUGA

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD HS	CHECKED	W.P. NO. 387 - 65	M&T DRAWING NO 72-11002 B
DRAWN T.T	CHECKED	JOB NO 72 - 11002	
DATE MAY 7, 1972		SITE NO.	
APPROVED		CONT. NO 76 - 01	

APPENDIX III

Construction of Culvert Sta. 25 + 41 to Sta. 21 + 18

	Date	Sta. 25 + 41 to Sta. 24 + 94	Sta. 24 + 94 to Sta. 24 + 47	Sta. 24 + 47 to Sta. 24 + 00	Sta. 24 + 00 to Sta. 23 + 53	Sta. 23 + 53 to Sta. 23 + 06	Sta. 23 + 06 to Sta. 22 + 59	Sta. 22 + 59 to Sta. 22 + 12	Sta. 22 + 12 to Sta. 21 + 65	Sta. 21 + 65 to Sta. 21 + 18
August	1									
	2									
	3	Excavate for Culvert								
	4									
	5									
	6									
	7									
	8									
	9		Excavate for Culvert							
	10			Excavate for Culvert						
	11		Erect Floor Forms. Place Floor R/F Steel	Excavate for Culvert						
	12		Place Floor R/F Steel. Pour Concrete Floor	Erect Floor Forms. Place Floor R/F Steel						
	13			Pour Concrete Floor						
	14									
	15									
	16		Place Wall R/F Steel	Pour Concrete Floor	Excavate for Culvert					
	17		Erect Wall Forms	Place Walls R/F Steel. Erect Wall Forms	Excavate for Culvert					
	18		Erect Wall Forms & Place R/F Steel		Erect Floor Forms. Place Floor R/F Steel. Pour Concrete	Excavate for Culvert				
	19	Excavate for Culvert	Pour Concrete Walls		Place Wall R/F Steel	Erect Floor Forms. Place Floor R/F Steel	Excavate for Culvert			
	20	Erect Floor Forms	Strip Wall Forms		Place Wall R/F Steel	Pour Concrete Floor				
	21									
	22									
	23	Place Floor R/F Steel.Pour Concrete Floor	Strip Wall Forms	Erect Wall Forms		Strip Floor Forms	Excavate for Culvert			
	24	Strip Floor Forms		Pour Concrete Walls	Place Wall R/F Steel		Erect Floor Forms. Place Floor R/F Steel	Excavate for Culvert		
	25			Strip Wall Forms	Erect Wall Forms & Place R/F Steel		Pour Concrete Floor	Erect Floor Forms. Place Floor R/F Steel		
	26				Pour North and Middle Concrete Walls	Place Wall R/F Steel	Strip Floor Forms	Erect Floor Forms. Place Floor R/F Steel		
	27		Erect Deck Forms		Pour South Concrete Wall	Place Wall R/F Steel	Strip Floor Forms	Pour Concrete Floor		
	28									
	29									
	30									
	31									
September	1		Erect Deck Forms							
	2		Erect Deck Forms							
	3				Strip Wall Forms	Erect Wall Forms. Pour Concrete Walls	Place Wall R/F Steel			
	4									
	5									
	6									
	7		Erect Deck Forms			Strip Wall Forms	Erect Wall Forms & Place Wall R/F Steel	Place Wall R/F Steel		
	8		Place Deck R/F Steel			Strip Wall Forms	Pour Concrete Walls	Strip Floor Forms		
	9		Pour Concrete Deck				Strip Wall Forms	Place Wall R/F Steel		
	10	Place Wall R/F Steel					Strip Wall Forms	Pour Concrete Walls		
	11									
	12									
	13	Place Wall R/F Steel.Erect Wall Forms	Strip Deck Forms	Erect Deck Forms & Place Deck R/F Steel			Strip Wall Forms	Pour Concrete Walls	Excavate for Culvert	
	14	Pour Concrete Walls		Pour Concrete Deck				Strip Wall Forms	Excavate for Culvert	
	15	Pour Concrete Walls						Strip Wall Forms	Erect Floor Forms. Place Floor R/F Steel	Excavate for Culvert
	16	Strip Wall Forms.Erect Deck Forms		Strip Deck Forms					Pour Concrete Floor	Erect Floor Forms. Place Floor R/F Steel.- Pour Concrete Floor
	17									
	18									
	19									
	20	Place Deck R/F Steel.Pour Concrete Deck							Strip Floor Form, Place Wall R/F Steel	Pour Concrete Floor
	21	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall				Strip Floor Forms
	22	Strip Deck Forms. Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall			Erect Wall Forms	Place Wall R/F Steel
	23	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall			Pour Concrete North Wall	
	24									
	25									
	26									
	27	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall. Erect Deck Forms	Backfill Behind Wall			Strip North Wall Forms	Erect North Wall Forms. Pour Concrete North Wall
	28				Erect Deck Forms					
	29				Place Deck R/F Steel. Pour Concrete Deck Backfill Behind Wall				Erect Middle & South Wall Forms	Erect Middle & South Wall Forms
	30					Backfill Behind Wall	Backfill Behind Wall		Pour Concrete Middle & South Walls	
October	1								Strip Middle & South Wall Forms	Pour Concrete Middle & South Walls
	2									
	3									
	4			Backfill on Deck	Backfill on Deck, Strip Deck Forms	Erect Deck Forms				Strip Wall Forms
	5					Erect Deck Forms				
	6					Place Deck R/F Steel. Pour Concrete Deck				
	7						Erect Deck Forms			
	8									
	9									
	10									
	11									
	12									
	13									
	14						Erect Deck Forms			
	15	Backfill on Deck	Backfill on Deck			Backfill on Deck	Place Deck R/F Steel & Pour Concrete Deck			
	16									
	17									
	18					Backfill Behind Wall	Backfill Behind Wall	Erect Deck Forms		
	19					Backfill Behind Wall & on Deck	Backfill Behind Wall & on Deck	Place R/F Steel. Pour Concrete.Deck Backfill Behind [illegible]	Backfill Behind Wall	
	20					Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	
	21							Strip Deck Forms	Erect Deck Forms	
	22								Place Deck R/F Steel. Pour Concrete Deck	
	23									
	24									
	25						Backfill on Deck	Backfill on Deck	Strip Deck Forms	Erect Deck Forms
	26						Backfill on Deck			Place Deck R/F Steel. Pour Concrete Deck
	27						Backfill on Deck	Backfill on Deck		
	28					Backfill on Deck	Backfill on Deck	Backfill on Deck		
	29									
	30									
	31									
November	1									Strip Deck Forms
	2					Backfill Behind Wall & on Deck	Backfill Behind Wall & on Deck	Backfill Behind Wall & on Deck		
	3									
	4									
	5									
	6									
	7									
	8									
	9									
	10									
	11									Backfill Behind Wall Deck
	12									
	13									
	14									
	15									
	16									
	17									
	18									
	19									
	20									
	21									
	22									Noticed Hairline Crack

Construction of Retaining Wall B

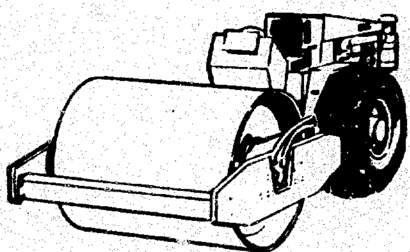
Month	Day	Panel 1	Panel 2	Panel 3	Panel 4	Panel 5	Panel 6	Panel 7	Panel 8
October	15								
	16								
	17								
	18								
	19								
	20								
	21								
	22								
	23								
	24								
	25	Excavate for Wall	Excavate for Wall	Excavate for Wall	Excavate for Wall	Excavate for Wall	Excavate for Wall	Excavate for Wall	Excavate for Wall
	26								
	27							Excavate for Footings	Excavate for Footings
	28							Form Ftg. Place R/F. Pour Concrete Ftg.	Form Ftg. Place R/F. Pour Concrete Ftg.
	29							Strip Ftg. Forms	Strip Ftg. Forms
	30								
	31								
November	1								
	2								
	3				Form Ftg. Place R/F. Pour Concrete Ftg.	Form Ftg. Place R/F. Pour Concrete Ftg.	Form Ftg. Place R/F. Pour Conrete Ftg.		
	4				Strip Ftg. Forms	Strip Ftg. Forms	Strip Ftg. Forms		
	5				Place R/F. Erect Forms. Pour Concrete Wall		Place R/F. Erect Form. Pour Concrete Wall		
	6				Strip Wall Forms		Strip Wall Forms		
	7								
	8					Place R/F. Erect Forms. Pour Concrete Wall			
	9							Place R/F. Erect Forms	Place R/F. Erect Forms. Pour Concrete Wall
	10		Excavate for Ftg.	Excavate for Ftg.		Strip Wall Forms			
	11	Excavate for Ftg. Place R/F, Form, Pour Concrete Ftg.	Place R/F. Form & Pour Concrete Ftg.	Place R/F, Form & Pour Concrete Ftg.				Erect Forms. Pour Concrete Wall	Strip Wall Forms
	12	Place R/F	Place R/F. Form & Pour Concrete Wall	Place R/F				Strip Wall Forms	
	13								
	14								
	15	Erect Wall Forms & Pour Concrete Wall	Strip Wall Forms	Erect Wall Forms & Pour Concrete Wall					
	16	Strip Wall Forms		Strip Wall Forms					
	17	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall
	18	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall
	19	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall	Backfill Behind Wall
	20								
	21								
	22	Noticed Hairline Crack							
	23								
	24								
	25								
	26								
	27								
	28								
	29								
	30								

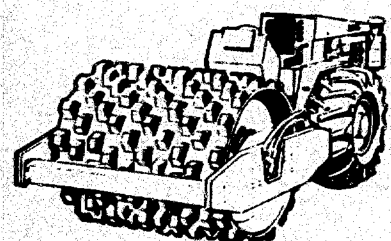
Nov. 16 & 17 Pour Concrete Liner For Panels 1-8

APPENDIX IV

BOMAG BW 210D/ 210PD

DRUM DRIVE VIBRATORY ROLLERS





Deep Lift Compaction in a Variety of Soils (Granular to Cohesive)
- Variable frequency
- Hydrostatic drum drive
- High impact pad foot drum design

Highly Maneuverable
- Center-articulation
- 30° oscillation of drum
- Flush side for work close to obstacles
- Superior traction with ribbed tires
- No-spin axle

Operator-Oriented
- Manual and automatic control of vibratory mechanism
- Exceptional visibility
- Clean, comfortable operator station

Minimal Maintenance
- Oil-lubricated vibratory bearings
- Drum cleaners adjustable and replaceable
- Maintenance points located for easy access
- Vandalism instrument shield

BW 210D/BW 210PD SPECIFICATIONS

DIMENSIONS

Weight (mass) BW 210D	18,500 lbs (8392 kg)
Weight (mass)* BW 210PD	23,500 lbs (10660 kg)
Width (shipping & operating)	8'8" (2642 mm)
Overall length	18'3" (5563 mm)
Height	7'3" (2210 mm)
Drum diameter	59" (1499 mm)
Drum width	84" (2134 mm)
Turning radius (inside)	16'10" (5131 mm)
Wheelbase	109" (2769 mm)
Curb clearance	15" (381 mm)

*Includes Tire Ballast

ENGINE OPTIONS

GM 4-53	110 hp (SAE) @ 2000 rpm Operating rpm 2000
Deutz F6L912	110 hp (SAE) @ 2800 rpm Operating rpm 2000
Air cleaner	Heavy duty dry type
Electrical system	12 volt
Fuel tank capacity	44 gal (167 l)

POWER TRAIN

Drum and wheels hydrostatic; No-spin differential

SPEEDS

1st . . . 3.1 mph (5.0 km/h)	2nd . . . 4.8 mph (7.7 km/h)	3rd . . . 6.8 mph (11.0 km/h)

BRAKES

Service	Hydrostatic thru drive system
Emergency and parking	Disc brake on drive shaft

STEERING SYSTEM

Full power, hydrostatic

Tires BW 210D	23.1 x 26-8 ply "All Weather"
Tires BW 210PD	23.1 x 26-8 ply "Heavy Lugged"

VIBRATORY COMPACTION DRUM BW 210D

Hydrostatically driven--automatic and manual control

Dynamic force	40,000 lbs (18 144 kg)
Total applied force	51,350 lbs (23 292 kg)
Frequency	1450 to 1850 vpm (24 to 31 Hz)
Applied force per lineal inch of drum width	611 lbs (277 kg)
Amplitude (double)	0.104" (2.6 mm)

VIBRATORY COMPACTION DRUM BW 210PD

Hydrostatically driven—automatic and manual control

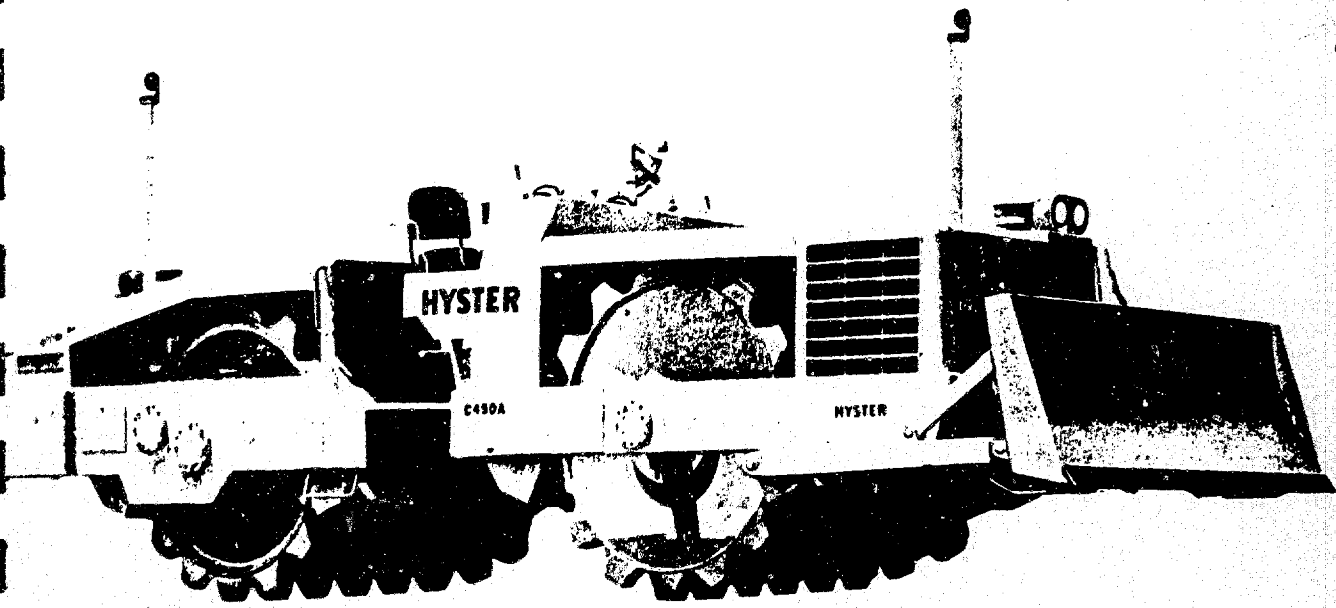
Dynamic force	40,000 lbs (18 144 kg)
Total applied force	54,500 lbs (24 721 kg)
Frequency	1450 to 1850 vpm (24 to 31 Hz)
Ground pressure	1266 psi (89.1 kg/sq cm)
Pad penetration depth	4¼" (108 mm)
Amplitude (double)	0.080" (2.0 mm)

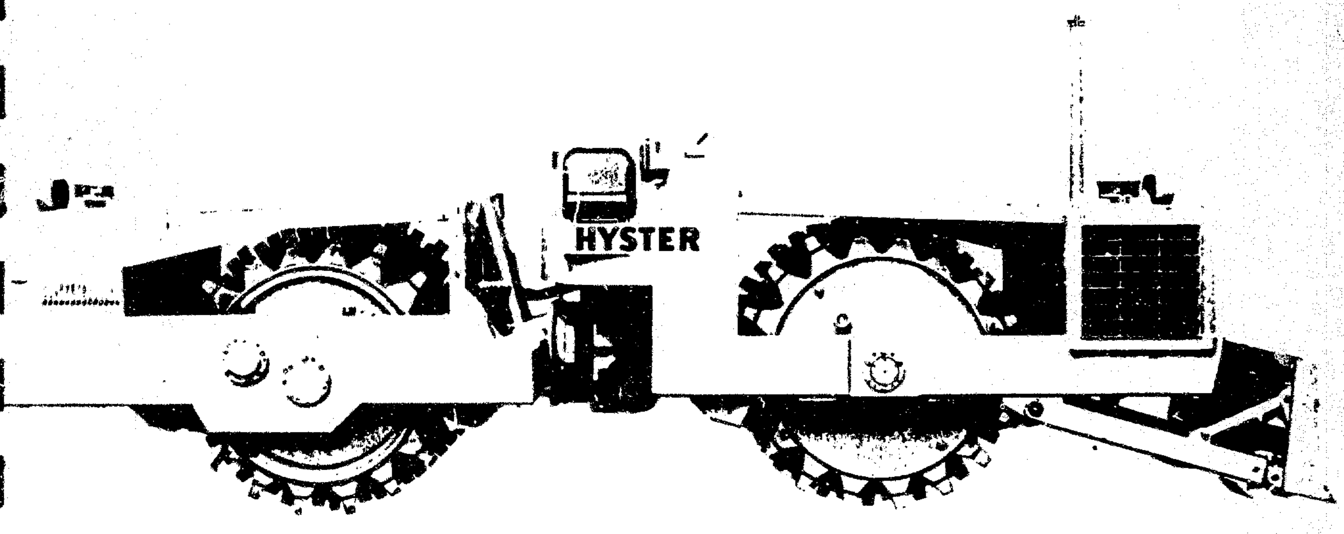
STANDARD AND OPTIONAL EQUIPMENT

See BW 210, page 21

NB - Smooth roller was used to compact the granular backfill

- Sheepsfoot roller was used to compact the embankment earthfill





HYSTER COMPANY
CONSTRUCTION EQUIPMENT OPERATIONS
Box 289 • Kewanee, Illinois 61443

Hyster products are subject to change without notice.

Manufactured in: Portland, Oregon • Peoria, Illinois • Kewanee, Illinois • Australia
Belgium • Brazil • France • Republic of South Africa • Scotland • The Netherlands

Form 2254C

5M 8/73 Litho in U.S.A.

APPENDIX V

DETERMINATION OF THE CRITICAL SLIP SURFACE
AND THE ACTIVE EARTH PRESSURE
BY MEANS OF A VARIATIONAL METHOD

There are a number of theories to compute the lateral force acting on a retaining wall due to active earth pressure. Common to all the theories is the requirement that the soil mass be in a state of limit equilibrium and that the strength of the soil be expressed in terms of the Coulomb-Mohr failure criterion. However, they differ in the assumption about the shape of the slip surface and in the application of statics.

Coulomb's theory assumes that sliding occurs along a planar slip surface. The relations used by the U.S. Dept. of Navy are based on logarithmic spiral slip surfaces. The method developed by Spencer assumes slip along a circular arc and makes use of the method of slices. Since in these methods the slip surfaces are restricted a-priori to a certain class of functions, the active force so computed may not be the most critical one. Recognizing that the determination of the active force is an extremum problem, in which the force is computed for a number of trial slip surfaces until a maximum value is obtained for the lateral force, a variational technique is used in this report to locate the most critical slip surface among all admissable functions, without restricting to a particular shape of slip surface. In this method, a functional in terms of arbitrary slip surfaces is derived for the lateral force by the method of slices and a differential equation, together with the boundary conditions, is obtained by setting the variation of the functional with respect to the slip surface equal to zero. The critical slip surface is then obtained as the solution to the differential equation which satisfies all the boundary conditions. Once the critical slip surface has been located, the lateral force, together with its point of application, can be determined from the method of slices. Derivation of the method is given below.

1. Assumptions

The following assumptions, in addition to those in the analysis of the state of limit equilibrium, are made in the derivation of the method:

- The soil mass is homogeneous
- The retaining wall has a vertical back face
- The shear stresses between the wall and the soil are neglected
- The inter-slice shear forces are neglected
- No porewater pressure.

2. Derivation of the Differential Equation and the Boundary Conditions

Let us consider the equilibrium of an arbitrary slice as shown in Figure 9.

From $\sum Fx = 0$ and $\sum Fy = 0$ we obtain

$$\Delta E = S \cos\alpha - N \sin\alpha$$

and $$W = S \sin\alpha + N \cos\alpha$$

since $$W = \gamma(H + \beta x - y)\Delta x$$

$$S = C \sec\alpha \Delta x + N \tan\phi$$

$$\sin\alpha = \left(\frac{1}{1+(y')^2}\right)^{\frac{1}{2}} y'$$

and $$\cos\alpha = \frac{1}{\sqrt{1+(y')^2}}$$

by substitution and solving for ΔE, we get

$$\Delta E = -\Theta(x,y,y')\Delta x \qquad \text{------------ (1)}$$

where $$\Theta = \frac{\gamma(H + \beta x - y)(y' - \tan\phi) - c\left[1 + (y')^2\right]}{1 + y'\tan\phi}$$

Integration of (1) from x=0 to x=b and making use of the conditions

$E(b) = 0$

$E(0) = P$

results in

$$P(b,y,y) = \int_0^b \Theta\, dx \qquad \text{------------ (2)}$$

which expresses a functional relationship between the lateral force and the slip surface. For P to attain a maximum value, it is necessary that P be stationary with respect to y. Setting the variation of δP equal to zero, i.e. $\delta P = o$, we obtain

$$\left(\Theta\delta x + \frac{d\Theta}{dy'}\delta y\right)\Bigg|_{x=b} + \int_0^b \left(\frac{d\Theta}{dy} - \frac{d}{dx}\frac{d\Theta}{dy'}\right)\delta y\, dx = 0$$

$$\text{-------------(3)}$$

From (3), we obtain the Euler differential equation

$$\frac{d}{dx}\Psi = \Omega \qquad \text{-------------(4)}$$

where $$\Psi = \frac{\gamma(H + \beta x - y)\sec^2\phi + C\left\{\left[1 - (y')^2\right]\tan\phi - 2y'\right\}}{(1 + y'\tan\phi)^2} \qquad \text{-------------(5)}$$

$$\Omega = \frac{(\tan\phi - y')}{1 + y'\tan\phi} \qquad \text{-------------(6)}$$

and the natural boundary conditions

$$\Theta\,\delta x + \frac{d\Theta}{dy'}\,\delta y = 0 \qquad \text{at } x = b \qquad \text{-----------(7)}$$

The natural boundary condition given by (7) can be further simplified by making use of the geometrical boundary conditions

$$y\,(0) = 0 \qquad \text{-----------(8)}$$

$$y\,(b) = H + \beta b - \frac{2c}{\gamma}\tan\left(\frac{\Pi}{4} + \frac{\phi}{2}\right) \qquad \text{-----------(9)}$$

and the transversity condition

$$\delta y\,(b) = \left[\beta - y'(b)\right]\,\delta b$$

to yield another boundary condition

$$\Theta + \Psi\,(\beta - y') = 0 \quad \text{at } x = b \qquad \text{-----------(10)}$$

which specifies the value of y' at x = b

The differential equation given in (4), together with the boundary conditions (8), (9) and (10), determine the location and the shape of the critical slip surface.

3. Method of Solution

Equation (4) is a non-linear differential equation.Close form solution is not generally available except for some simple cases such as when β= o or when either C = 0 or ϕ = 0. In general, equation (4) has to be solved by numerical method. Although it can be solved by the method of perturbation (or the method of small parameter) by considering tan ϕ as a small parameter, a relaxation technique given below is favoured because of its ease to be computerized. The method is as follows:

- Assume an arbitrary value for b, and set $\Delta x = \frac{b}{n}$
- Evaluate y_n = y (b) from (9) and y'_n = y' (b) from (10)
- With y_n and y'_n known, compute

  Ψn from (5)
- Calculate $y_{n-1} = y_n - y'_n \Delta x$

  and $\Psi n\text{-}1 = \Psi n - \Omega n \Delta x$

  which in turn yields y'_{n-1}
- Repeat the above procedures until

  $y_o = y_{(o)}$

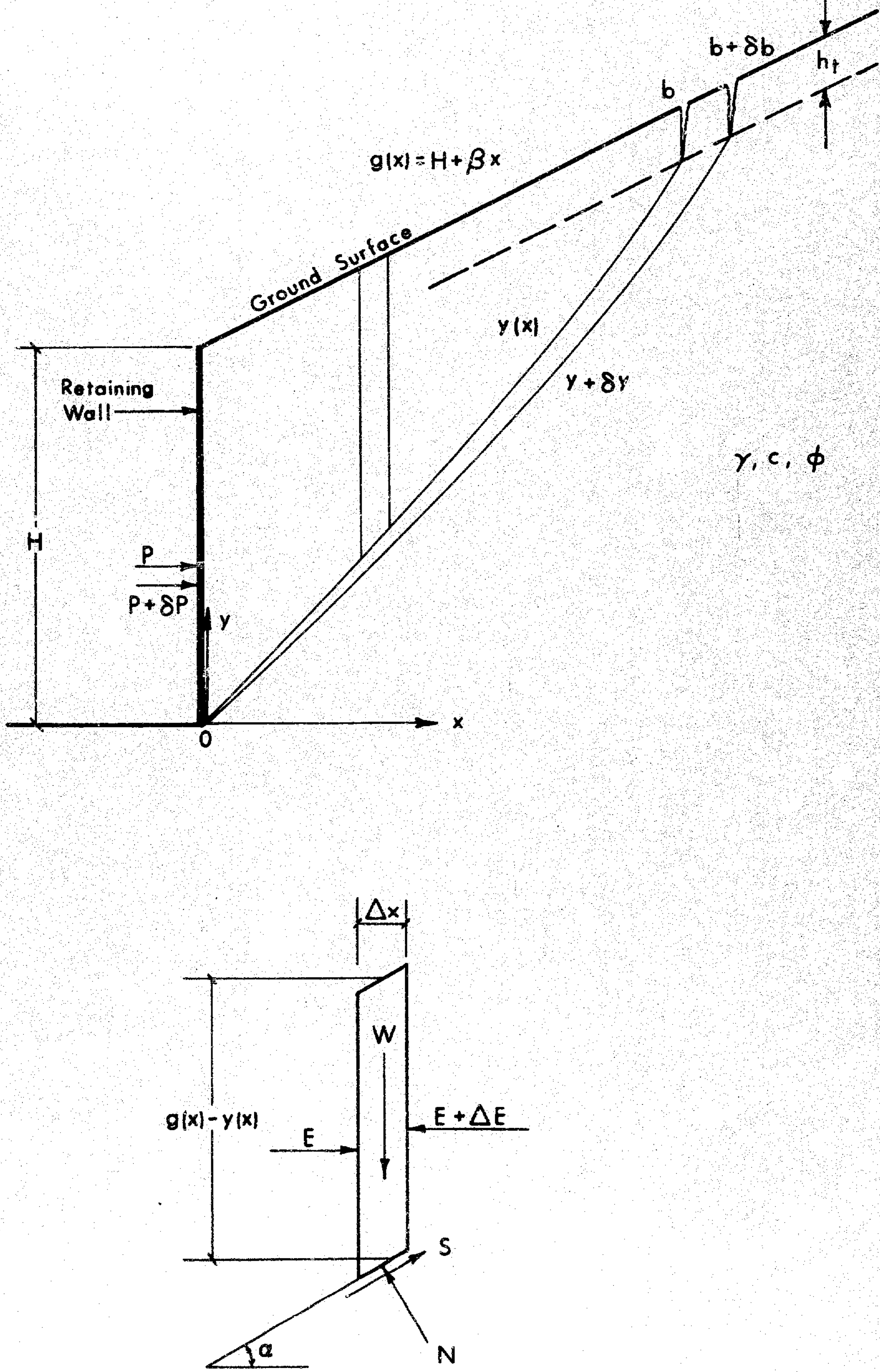
  is computed
- If y_o = 0, then the solution is obtained, otherwise, a new value for b has to be

assumed and repeat the computation until $y_o = 0$ is achieved.

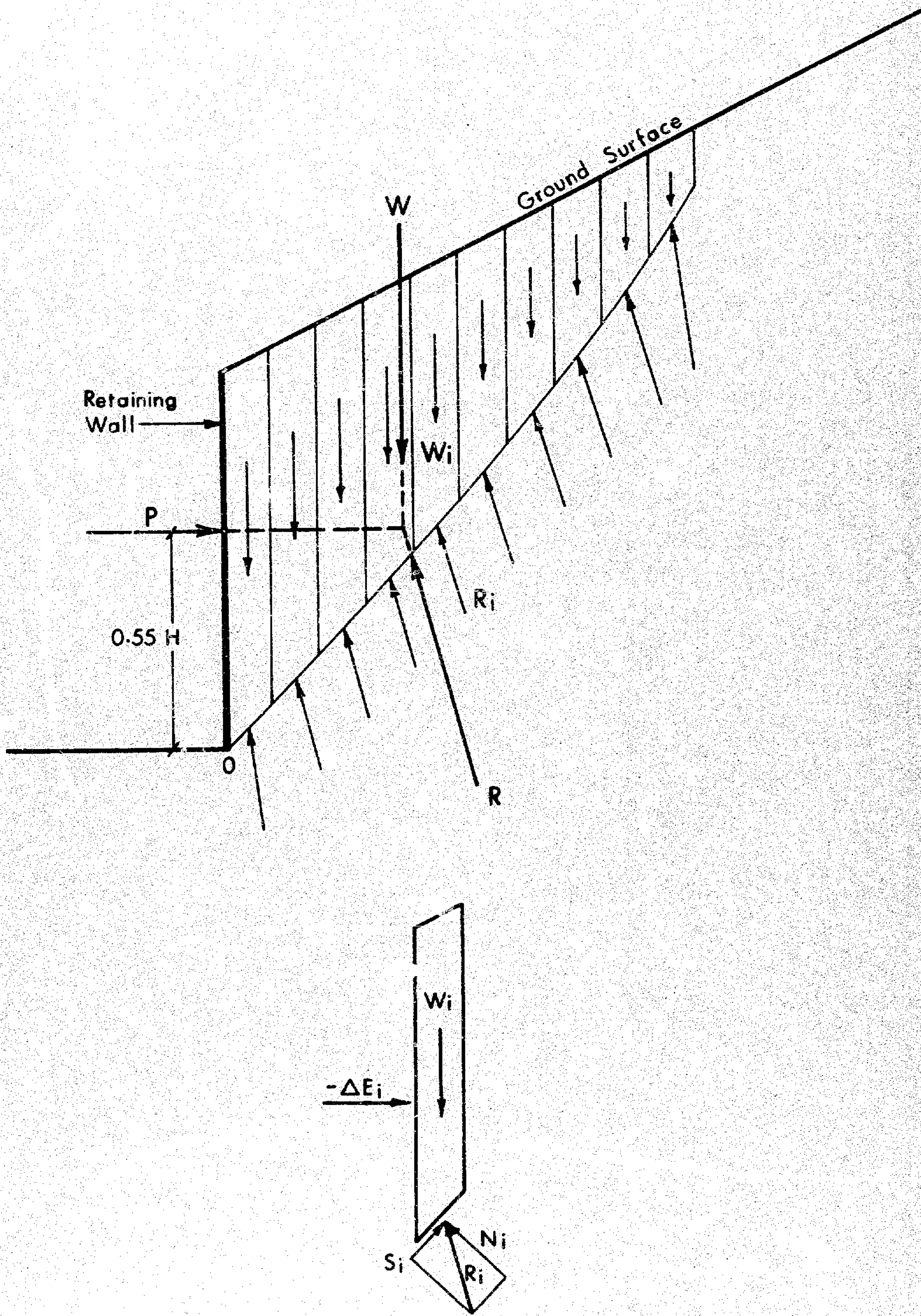
- Once y has been determined, P can be obtained from (2) and E_k from (1)
- Consequently, $E_{k-1} = E_k + \Delta E_k$
- With E_k, W_k and R_k of each slice known, the point of application of P can be determined by the method of string polygon as shown in Fig. (10).

Solutions obtained by this method for the cases when a) $\beta = 0$, b) $c = 0$, c) $\phi = 0$ are identical to the classical results. Results of our solution for the Mimico Creek Retaining Wall B are summarized in Fig. (11).

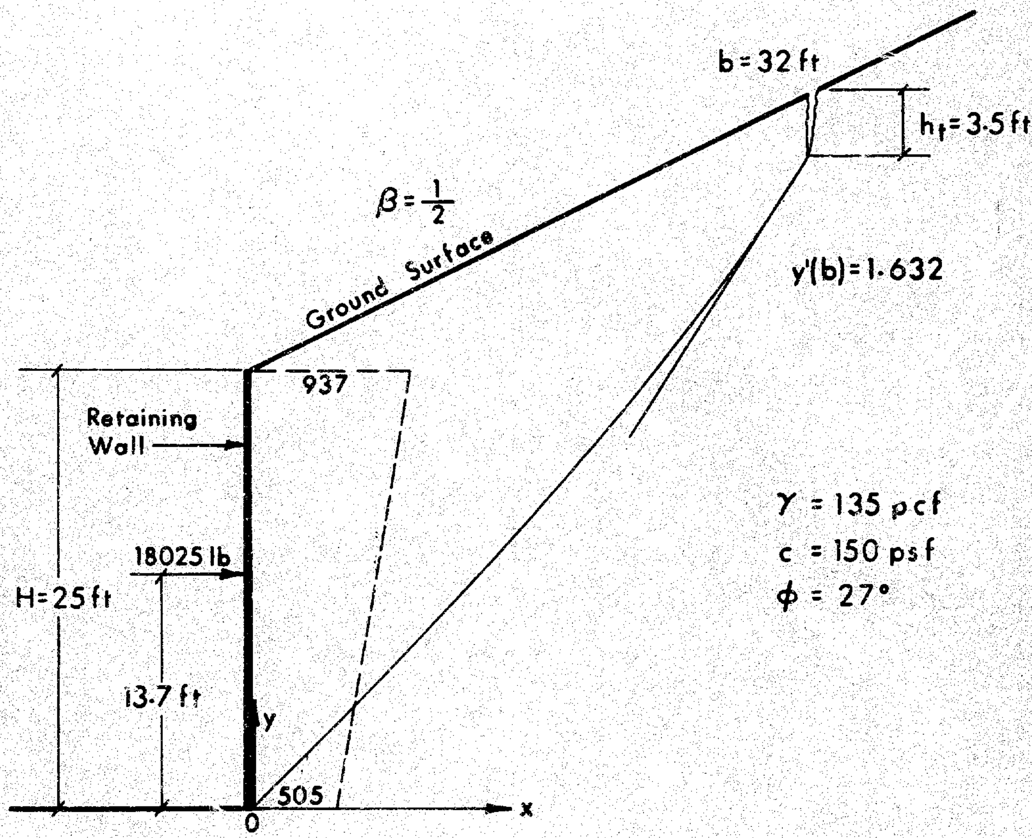
B. Ly
Soil Mechanics Section



Method of Slices and Variation in Slip Surface



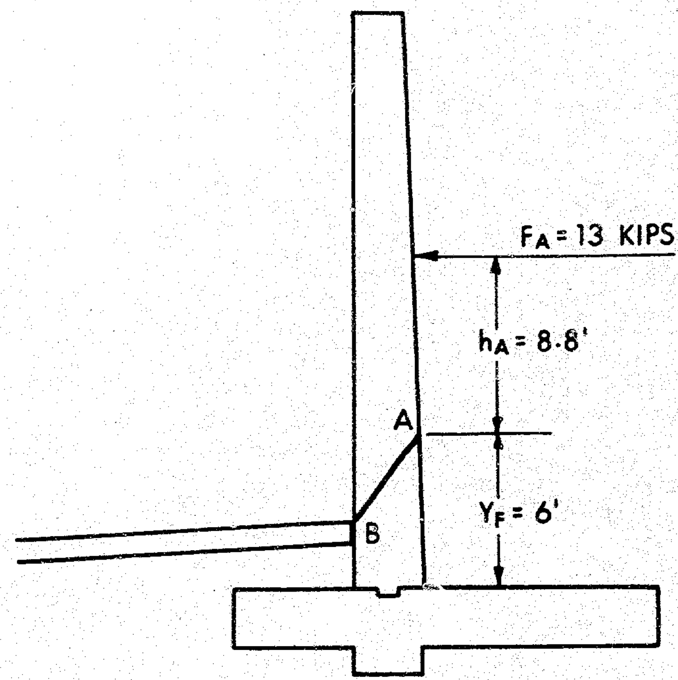
Determination of the Point of Application of Earth Pressure



The Critical Slip Surface and the Lateral Earth Pressure

APPENDIX VI

APPENDIX VI
ULTIMATE CAPACITY OF THE RETAINING WALL



<u>Calculation For Mu at point A.</u>

$$t = (1.25 + \frac{16.25}{24}) \times 12 = 23.125 \text{ in.}$$

$$As = \frac{1.27}{10} \times 12 = 1.524 \text{ in}^2/_{ft.} \qquad (\#10 @ 10")$$

$$bd = 12 \times (23.125-3-0.638) = 12 \times 19.49$$
$$= 233.88$$

$$p = \frac{As}{bd} = 0.00652$$

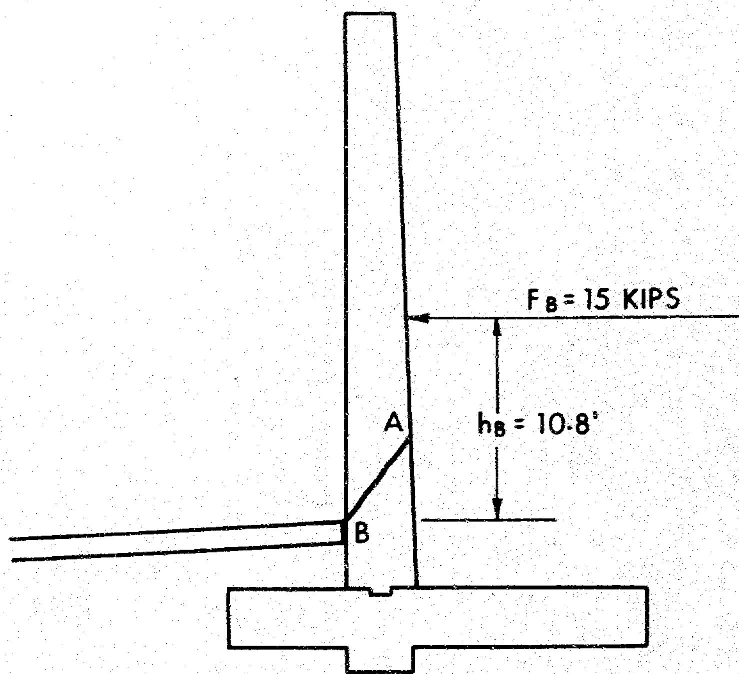
$$q = p\frac{fy}{f'c} = 0.00652 \times \frac{60}{4} = 0.0978$$

$$Mu = \emptyset \left[bd^2 f'c \; q \; (1-0.59q)\right]$$
$$= 0.9 \times \left[12 \times 19.49^2 \times 4 \times 0.0978 \times (1-0.59 \times 0.0978)\right] \times \frac{1}{12}$$
$$= 126 \text{ kip-ft.}/_{ft.}$$

$$Mu = \emptyset \; As \; fy \; (d - Asfy/f'cb \times 1.7)$$
$$= 0.9 \times \left[1.524 \times 60 \times (19.49 - \frac{1.524 \times 60}{1.7 \times 4 \times 12})\right] \times \frac{1}{12}$$
$$= 126 \text{ kip-ft.}/_{ft.}$$

$$M_A = 13 \times 8.8 = 114 \text{ kip-ft.}/_{ft.}$$

$$F.S. = 1.1$$



Calculation For Mu at point B.

$$t = (1.25 + \frac{20}{24}) \times 12 = 25 \text{ in.}$$

$$As = 3.048 \text{ in}^2/_{\text{ft.}} \qquad (\#10 \ @ \ 5")$$

$$bd = 12 \times (25-3-0.635) = 12 \times 21.365$$
$$= 256.38 \text{ in.}^2$$

$$p = \frac{3.048}{256.38} = 0.0119$$

$$q = 0.0119 \times \frac{60}{4} = 0.178$$

$$Mu = 0.9 \times [12 \times 21.365^2 \times 4 \times 0.178 \times (1-0.59 \times 0.178)] \times \frac{1}{12}$$
$$= 262 \text{ kip-ft.}/_{\text{ft.}}$$

$$Mu = 0.9 \times [3.048 \times 60 \times (21.365 - \frac{3.048 \times 60}{1.7 \times 4 \times 12})] \times \frac{1}{12}$$
$$= 262 \text{ kip-ft.}/_{\text{ft.}}$$

$$M_B = 15 \times 10.8 = 162 \text{ kip-ft.}/_{\text{ft.}}$$

F. S. = 1.6

(B. Ly, Soil Mechanics Section)

APPENDIX VII

Ministry of Transportation and Communications

Ontario

Memorandum

To: Dr. R.A. Dorton
Manager,
Structural Office

From: Engineering Research & Developmen
Branch

Attention:

Date: May 20, 1977

Our File Ref.

In Reply to

Subject: Re: Mimico Creek Structure - Hwy. 427 - Contract No. 7601

After the failure of this structure, a total of 23 strain gauge sites were installed. These sites indicated in Figures 1 to 3 were selected to monitor reinforcing steel strains in portions of the retaining wall, relieving slab, culvert wall and test slab.

Strain gauges, compensated for temperature, were bonded to the reinforcing steel bars to measure the axial strains induced in the steel. By monitoring these strains, an indication of the earth pressures acting on the structure can be obtained.

Monitoring of strain data began shortly after the concrete was cast. Figures 4 to 7 show the various strain levels in the reinforcing. These are plotted against time with the ambient air temperature recorded on the same date as the readings. This strain data is approximately 3 to 6 percent higher than the nominal strain in the reinforcing steel due to a small loss in the cross-sectional area of the reinforcing steel at the gauge site.

The strain readings vary from +61 to -108 in/in x 10 -6 for all locations, between February and May 16. This represents an axial force in the reinforcing steel of 1425 to -2475 lbs. or steel stresses of 1800 to -3100 psi. These strains can be caused by earth pressures or temperature change. The temperature change can cause strain if the structure is not free to move as the temperature changes.

It appears that up-to-now only small forces have been acting on the structure. Further monitoring is necessary to establish definite conclusions with regard to pressures exerted against the structure.

It is suggested that monthly readings be continued for at least 6 months to observe subsequent strain levels.

M. Holowka

M. Holowka
Research Officer

encls.

cc: P. Csagoly
C.S. Grebski

MH/pm

RECEIVED
MAY 24 1977
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

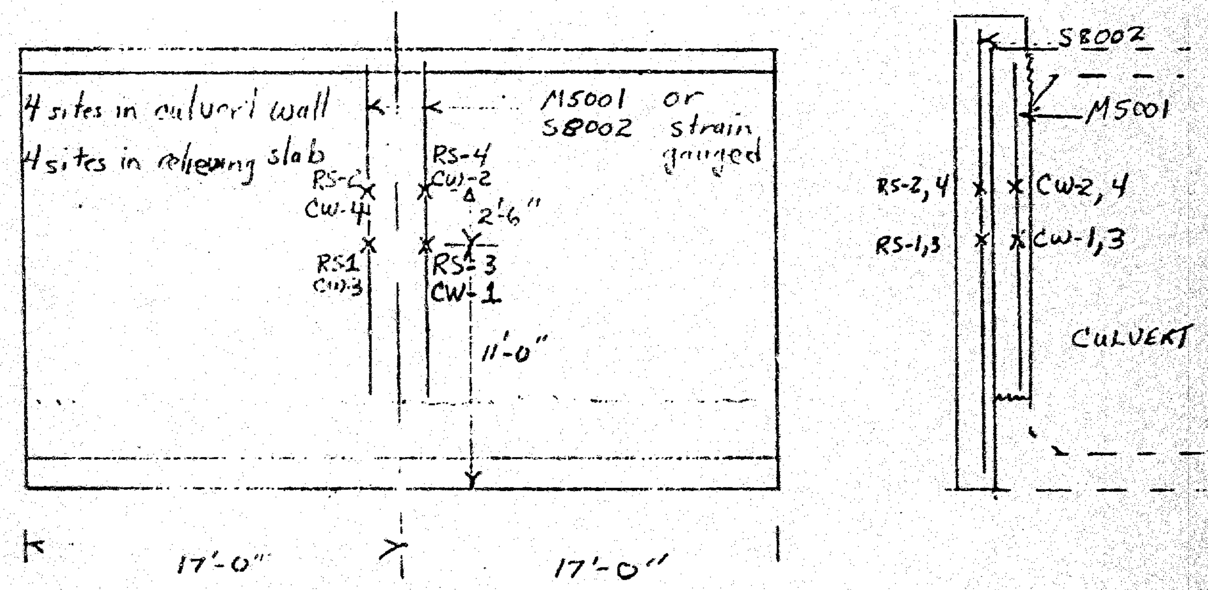


Figure 1: LOCATION OF STRAIN GAUGES IN RELIEVING SLAB AND CULVERT WALL

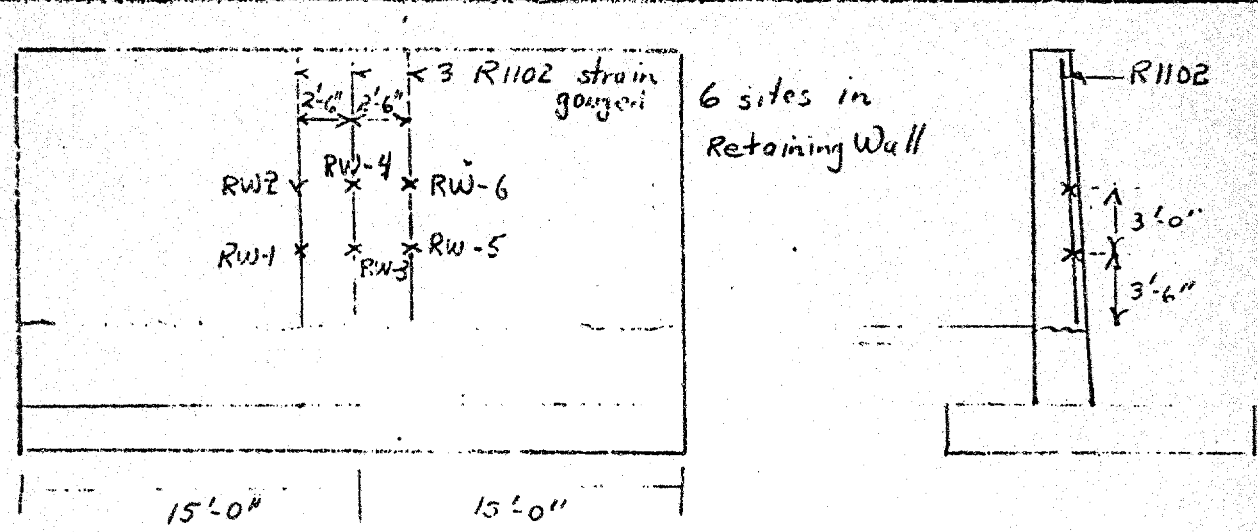


Figure 2: LOCATION OF STRAIN GAUGES IN RETAINING WALL PANEL 8.

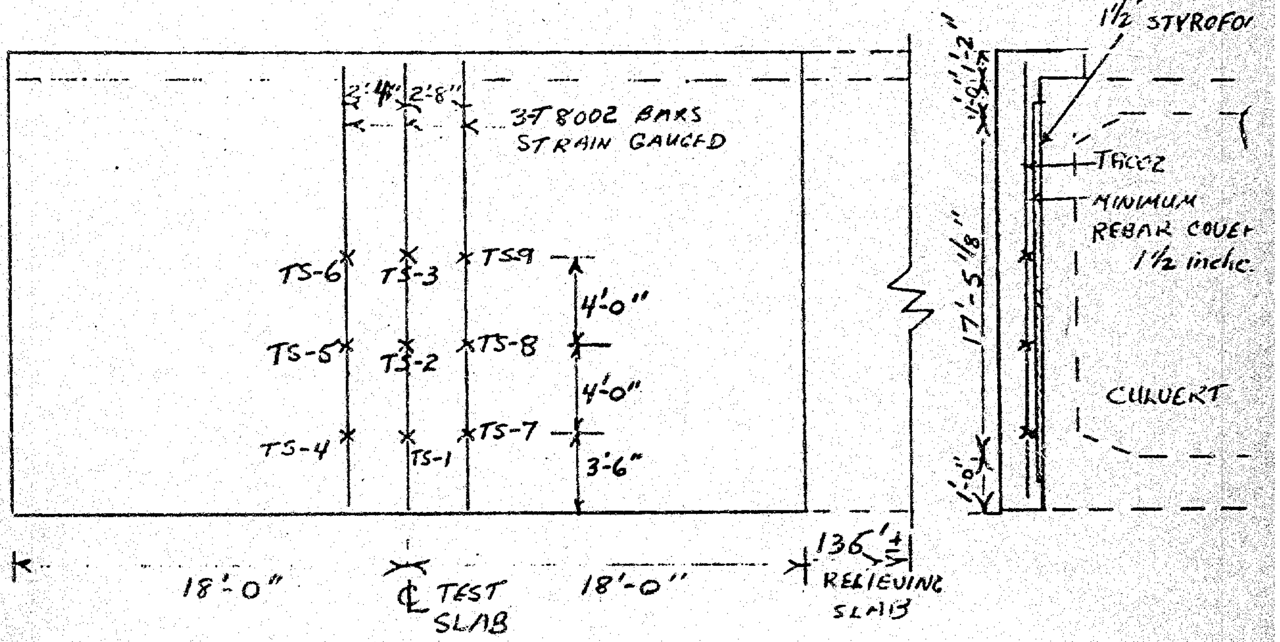


FIGURE 3: LOCATION OF STRAIN GAUGES IN TEST SLAB.

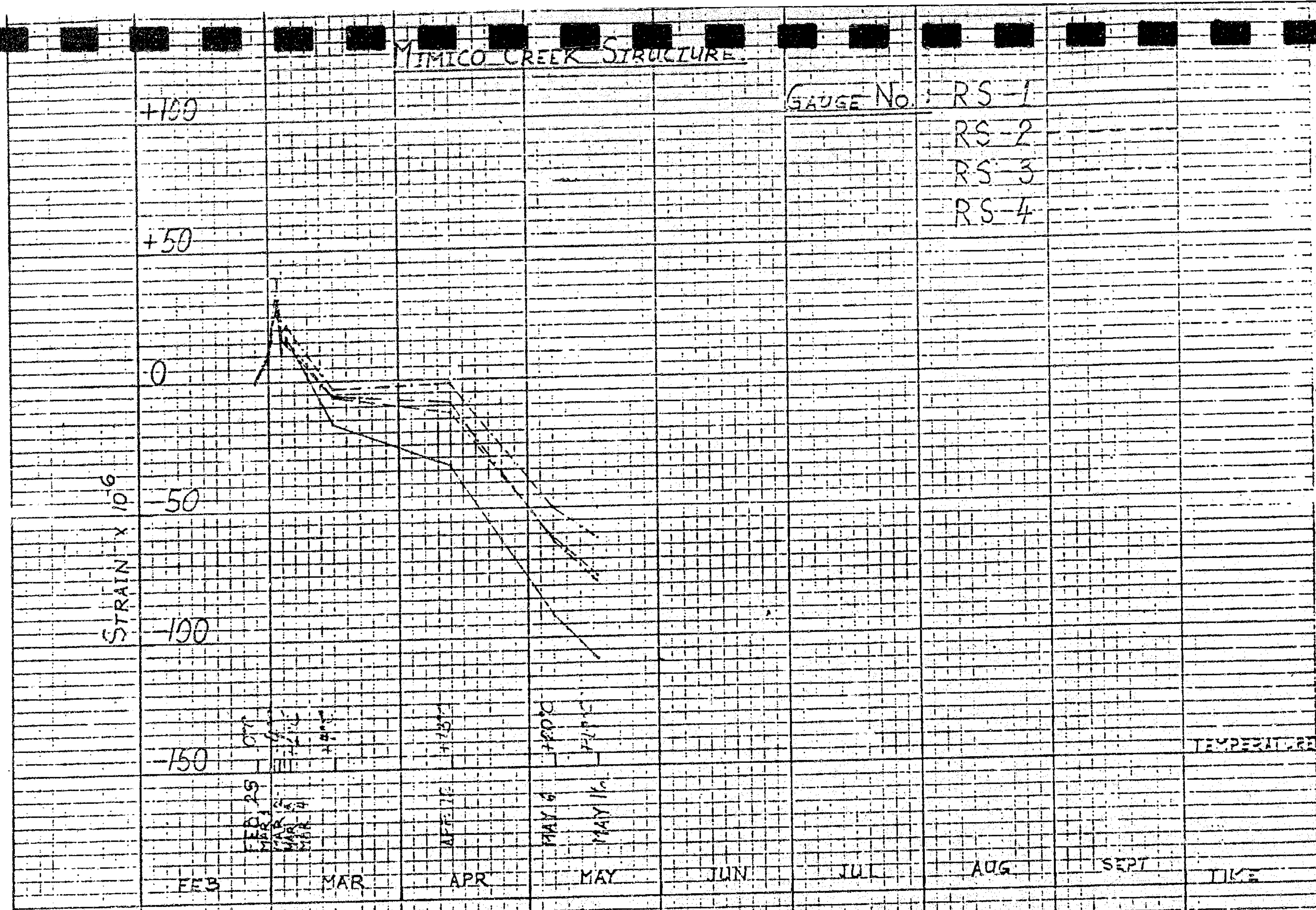


FIGURE 4: STRAIN DATA - FOR THE RELIEVING SLAB

MIMICO CREEK STRUCTURE

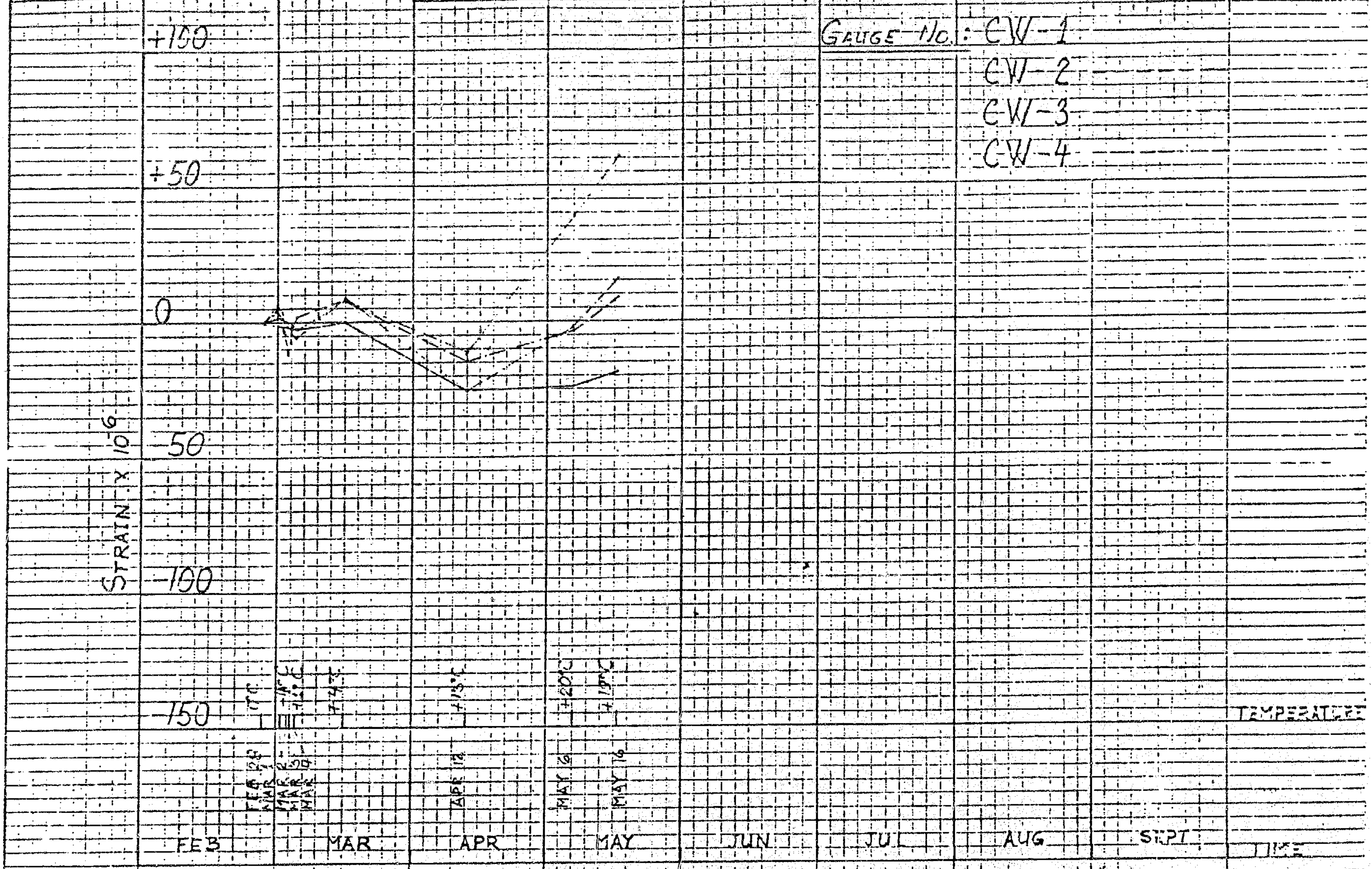


FIGURE 5 : STRAIN DATA FOR THE CULVERT WALL

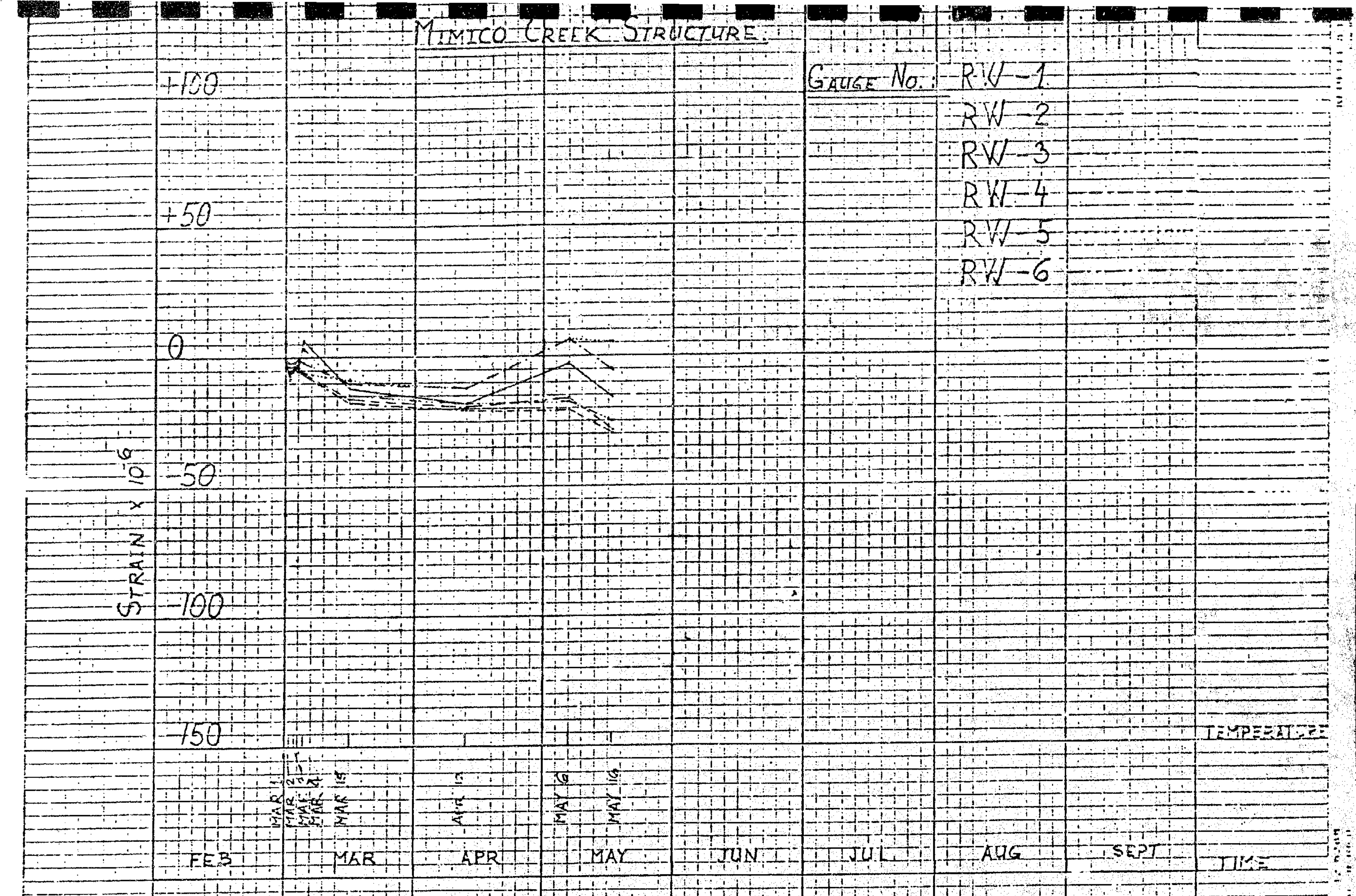


FIGURE 6: STRAIN DATA FOR RETAINING WALL

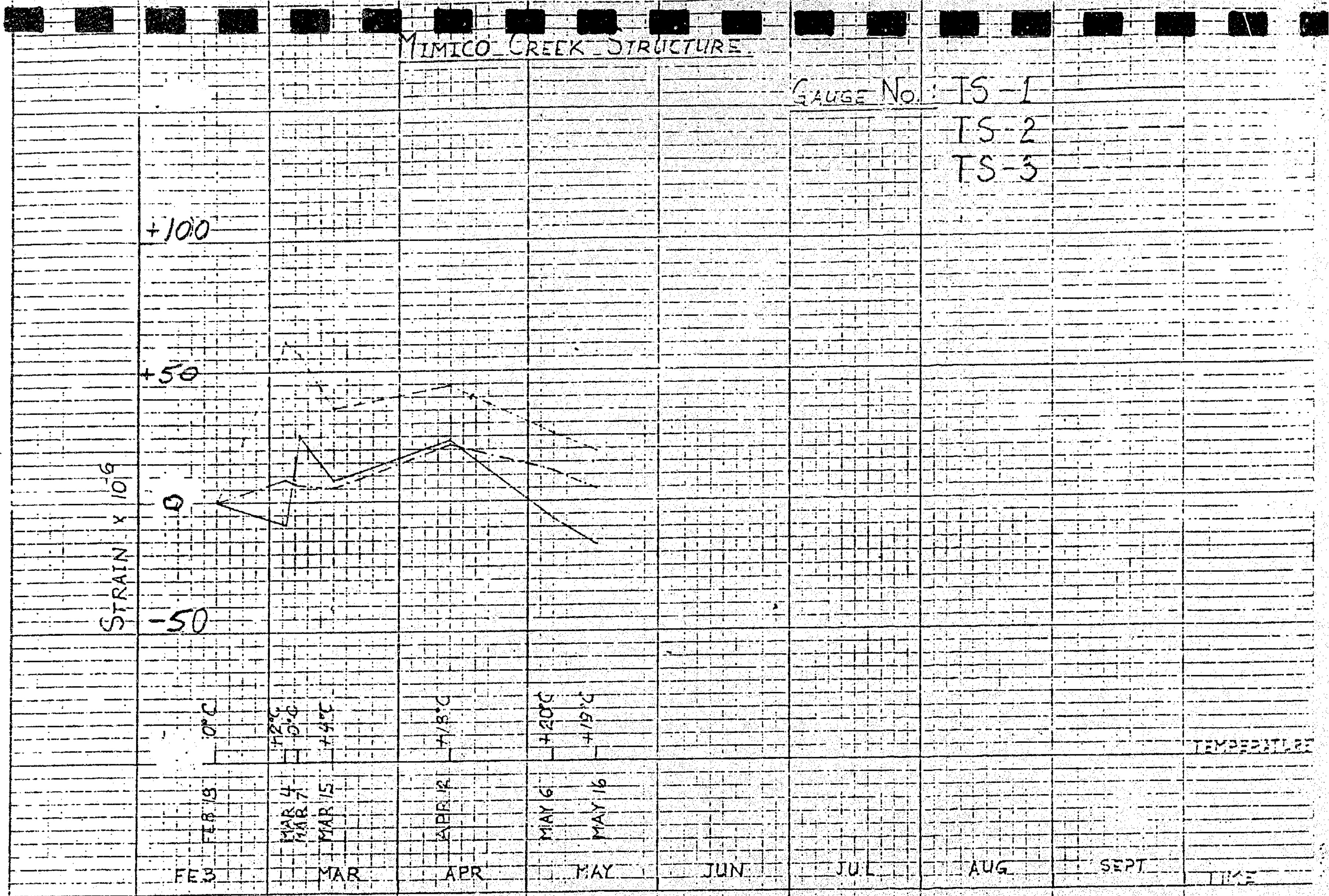


FIGURE 9a: STRAIN DATA FOR TEST SLAB

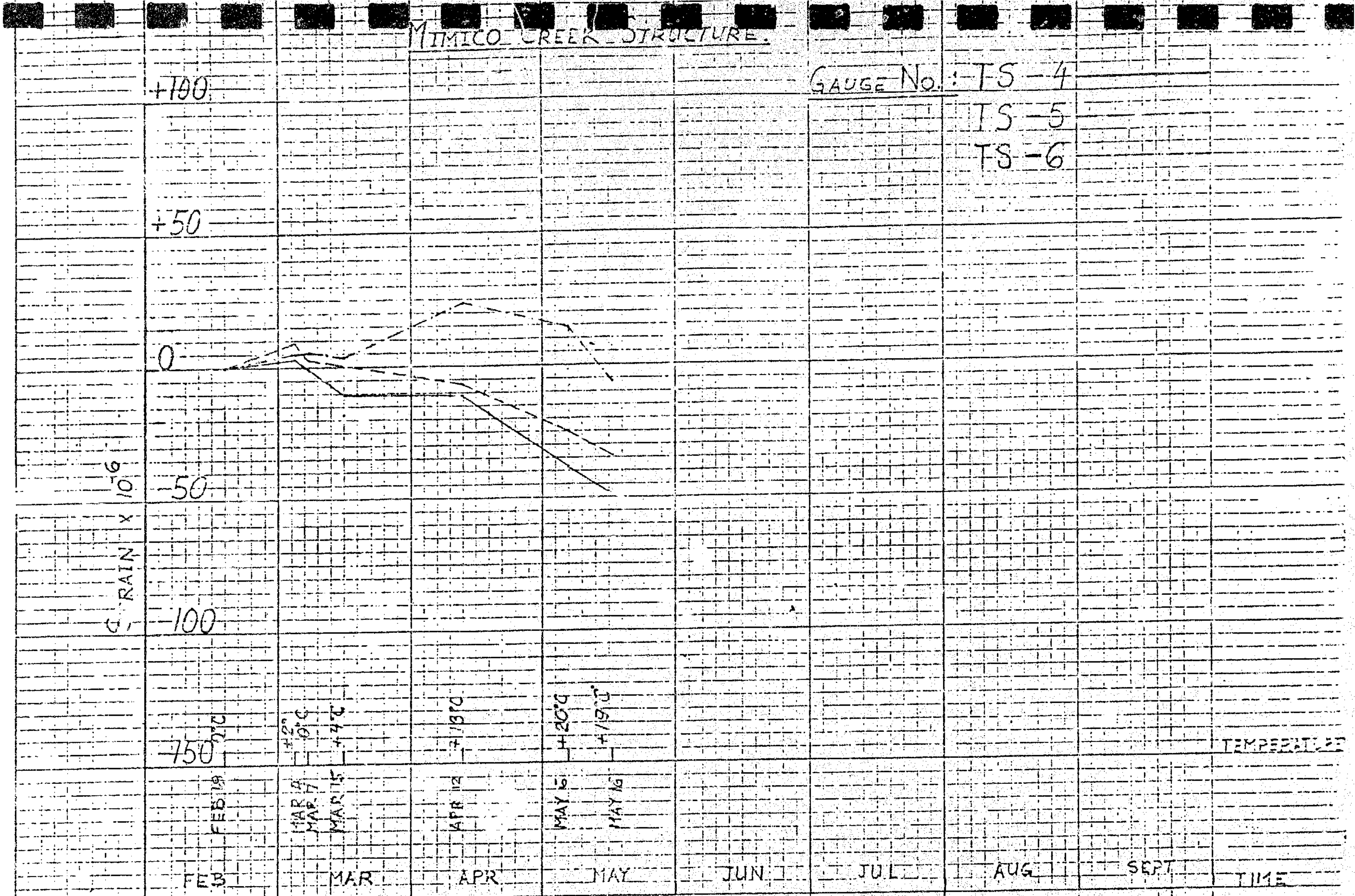


FIGURE 7b: STRAIN DATA FOR TEST SLAB

THE ALGOMA STEEL CORPORATION, LIMITED

SAULT STE. MARIE, ONTARIO

QUALITY - SERVICE

STEEL FOR STRENGTH

...ER	C.T. ABBR. NAME	ENTRY DATE	ITEMS	ORDER WEIGHT	TAPE REF	MILL ORDER NO
	MT AND C	APR 18/77			NO	19031

...HEREIN CALLED BUYER)
...T... OF TRANSP AND COMMUNICATIONS
...W...SON AVE
...SVIEW ONTARIO , M3M 1J8

SHIP TO
MTC BOWEN RD PATROL YARD-NW QUADRANT OF
BOWEN ROAD AND QEW INTERCHANGE APPROX
14 MILES SOUTH OF NIAGARA FALLS ONTARIO

	PROV. ST.	TERMS	CUST. CODE	OBS	SALES C/C	ADD ACK
...T	EXTRA	NET 30 DAYS	5936-A0 25	82	23456	1 ACK

VENDOR CODE NO

...SPORT
...D
...AD

FREIGHT: COLLECT
F.O.B.: OUR PLANT

...PECIFICATION
...CTURAL SHAPES - CARBON - CSA G40.21-44W

...S/M 2 INV 1 ACK 1 PROBILL TO MTC C/O M STOYANOFF STRUCTURAL
...F...CE DOWNSVIEW 1 TR MTC BRIDGE ENG OFFICE MTC DOWNSVIEW

X	B/L	S/M	INV.	T/R
CT				
ST			2	
OT		X	X	X

...ILL SM/TR
...U...LY CONTRACT S76-2251
...RK LOAD PACK

P -

ADD'L SHIP DATA: PLEASE NOTIFY TOR OFFICE ASAP BEFORE LOADING

	CARRIER CODE	SHIPPED DATE	SHIPPER'S MILL & NO
...ICEWAY	0671	MAY 10/77	05 8837-1-1

IB/L NO.	IB/L WGT.	P.P.D.	COL.	INVOICE DATE	MONTH	INVOICE NUMBER
73247			30	AS ABOVE	5	1902

...M	OUR ITEM	HEAT NO	NUMBER PCS.	DESCRIPTION	LENGTH FT. IN	WEIGHT	UNIT PRICE	AMOUNT
3	3		31	12 x 74	33-0	75702	12.80	968986
		PLUS PROVINCIAL SALES TAX @ 7%						678.29

TOTAL PIECES	TOTAL WEIGHT	TOTAL AMOUNT
		10368.15

9
9
9

OUR ITEM	BASE PRICE	SIZE	LENGTH		GRADE	CHEMISTRY	SILICON	PICKLE	TESTING	M.P.L.	QUANTITY		UNIT PRICE
	11.75	80	25										12.80

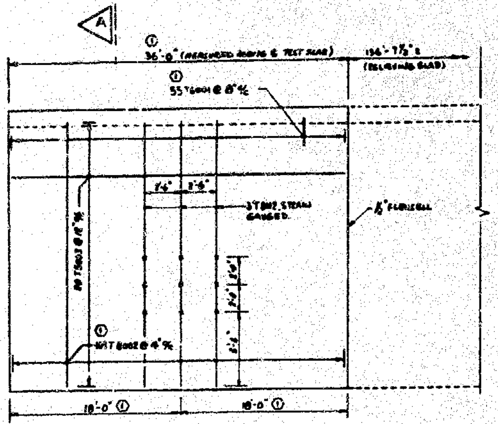
(EXTRAS)

HRNWARE

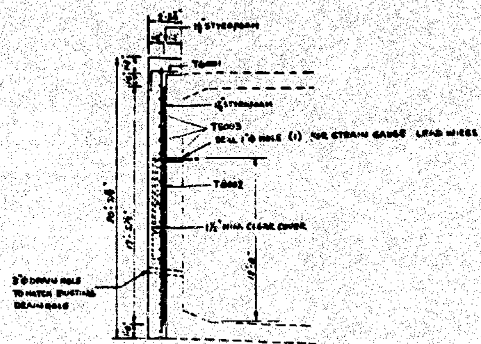
CONT No 76-01
WP No 53-65

MIMICO CREEK STR. HWY 427
36' TEST BURYING SLAB AND STRAIN GAUGING DETAILS

SHEET



ELEVATION OF TEST SLAB ¼" = 1'-0"



NOTE: BACKFILL TO TEST SLAB SECTION (36') ONLY TO BE GRANULAR "A" AND COMPACTED.

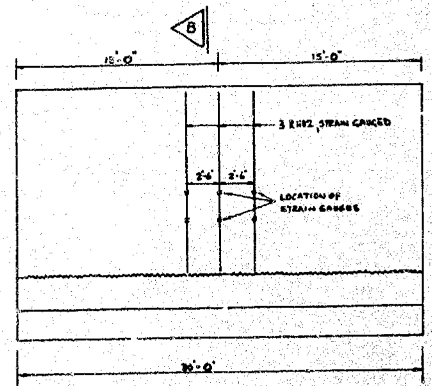
SCALE : ¼" = 1'-0"

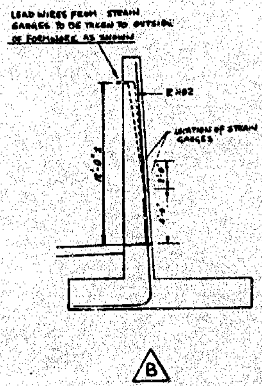
NOTES:

1. THE FOLLOWING LISTED REBARS WILL BE STRAIN GAUGED AND DELIVERED TO THE SITE BY THE MINISTRY. APPROX 7 DAYS WILL BE REQUIRED BY THE MINISTRY TO INSTALL THE STRAIN GAUGES.

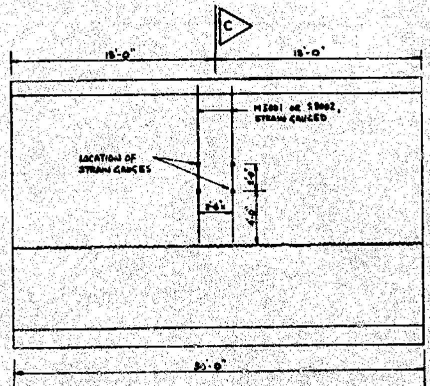
REBAR MARK	NO.
R 802	3
M 5001	2
S 8002	2
T 8002	5

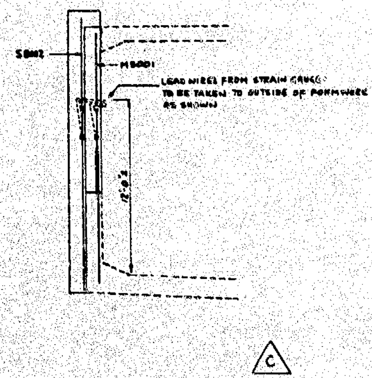
2. THE ABOVE REBARS SHALL BE ACCURATELY PLACED AS SHOWN ON THIS DWG.
3. THE CONTRACTOR SHALL NOTIFY THE ENGINEER 2 DAYS BEFORE COMMENCEMENT OF THE PLACING OF THE ABOVE REBARS SO THAT MINISTRY TECHNICIANS CAN SUPERVISE ITS PLACING.
4. NO TORCH CUTTING OR WELDING OF ANY GAUGED REBARS, OR OF ANY OTHER REBAR WITHIN 3 FEET OF ANY STRAIN GAUGE, WILL BE PERMITTED.





ELEVATION OF R/W PANEL 8





ELEVATION CULVERT WALL

FOR REDUCED PLAN

REVISIONS		
DATE	BY	DESCRIPTION
DESIGN	CHECK	LOADING
DRAWING	CHECK	SITE No

APPENDIX VIII

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 501

Contract 76-01

DIST 6 HWY 427

DATUM Geodetic

LOCATION Co-ords N 15 879 058; E 971 040

BORING DATE Nov. 30, 1976

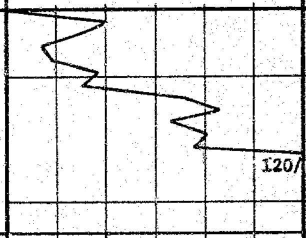
BOREHOLE TYPE Wash-Boring & Dynamic Penetration Test

ORIGINATED BY BL

COMPILED BY BL

CHECKED BY

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH	WATER CONTENT % 10 20 30 40	UNIT WEIGHT γ P.C.F.	REMARKS % GR SA SI CL
525.5	Ground Level								
0.0	Fill: Silty Sand with Clay and gravel	1	SS	14					
		2	SS	47	520				
	Fine crushed stone with fines, white sand	3	SS	72					20 57 (23)
		4	SS	80/9"			o		
	with numerous gravel & cobbles. Some silt & trace of clay	5	SS	76		120/6"	o		28 50 (22)
507.7	Reddish Brown	6	SS	100/6"	510		o	157	34 48 (18)
		7	SS	100/3"					
17.8	End of Borehole								
	Note: Hole stayed dry & Open on completion								



LIQUID LIMIT —— W_L
PLASTIC LIMIT —— W_P
WATER CONTENT —— W

SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

15 ◇ 5 (20 / 10) % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 505

Contract 76-01 | LOCATION Co-ords. N. 15 878 905; E 971 130 | ORIGINATED BY BL

DIST 6 HWY 427 | BORING DATE December 14, 1976 | COMPILED BY BL

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	'N' VALUES	GROUND WATER – ELEV	UNIT WEIGHT γ P.C.F.	REMARKS % GR SA SI CL
520.9	Ground Level						
0.0	Fill: Clayey silt traces of gravel and sand				520		
		1	SS	19			
		2	SS	13	510		
507.4		3	SS	17		Org. Cont.	
505.9	Topsoil	3b	AS	-		4.2%	5 45 32 18
15.0	very clayey	4	SS	16			
	Glacial Till: Het. Mixture of clayey silt, sand & gravel	5	SS	48			
					500		18 24 38 20
		6	SS	50/3"			
492.9	Very stiff to hard	7	SS	75/6"			
28.0	End of Borehole						

BOREHOLE No 505A

LOCATION Same as Borehole 505

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV	UNIT WEIGHT	REMARKS
520.9	Ground Level						
0.0	Fill: Clayey silt traces of gravel & sand				520		
					510	137.5	4 29 40 27
		1	TW	PH			Org. Co t
		2	TW	PH		130.0	5.1%
504.4	Charcoal & Organic inclusions	3	TW	PH		143.0	4.5%
		4	TW	PH			
16.5	End of Borehole						

Note: Hole was wet at Elv. 506±

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 500 1000 2000

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W

WATER CONTENT % 10 20 30 40

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 506

Contract 76-01 | LOCATION Co-ords N 15 878 928; E 971 166 | ORIGINATED BY BL

DIST 6 HWY 427 | BORING DATE Dec. 15, 1976 | COMPILED BY BL

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY [signature]

ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W / WATER CONTENT % 10 20 30 40	UNIT WEIGHT γ	REMARKS % GR SA SI CL
519.1	Ground Level									
0.0	Fill: Clayey silt		1	SS	100/5"					
	some sand & gravel		2	SS	43					
									Org. Cont.	
			3	SS	20				1.7%	
						510				
	inclusions of organics, wood fibres & cinders		4	SS	21				1%	4 43 34 19
			5	TW	PH				1.8%	8 40 33 19
504.1			6	TW	PH				3.2%	
			7	SS	60/5"					
15.0	Glacial Till									
	Very stiff to hard					500				
497.6			8	SS	81					40 30 (30)
21.5	End of Borehole									
	Note: Hole dry on completion									

OFFICE REPORT ON SOIL EXPLORATION

20
15 ⊕ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 507

Contract 76-01	LOCATION Co-ords N 15 878 945; E 971 143		ORIGINATED BY BL
DIST 6	HWY 427	BORING DATE December 14, 1976	COMPILED BY BL
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger		CHECKED BY

SOIL PROFILE · SAMPLES · GROUND WATER · DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) · SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 500 1000 1500 2000 2500) · LIQUID LIMIT — W_L, PLASTIC LIMIT — W_P, WATER CONTENT — W; WATER CONTENT % (10 20 30 40) · UNIT WEIGHT γ · REMARKS % GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	REMARKS (% GR SA SI CL)
515.8	Ground Level					
0.0	Fill: Brown Silty Sand	1	SS	33		11 73 (16)
	Clayey silt	2	SS	15	510	2 23 45 30
		3	SS	14		
	Some Sand	4	SS	13		
		5	SS	34		
501.3	with ~~organics & decayed vegetation~~	6	SS	70		Org. Cont 2.8%
14.5	Glacial Till: very stiff to hard				500	
494.3		7	SS	103		
21.5	End of Borehole					

BOREHOLE No 507A

Co-ords N 15 878 951; E 97[illegible] 135

ELEV / DEPTH	DESCRIPTION	GROUND WATER ELEV
515.8	Ground Level	
0.0	Fill: Brown Silty Sand	
	Clayey Silt	510
	some sand	
503.3	Organics & wood fibres	
12.5	Clayey	
500.8	Glacial Till	
15.0	End of Borehole	

Note: water level not established

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 508

Contract 76-01	LOCATION Co-ords N 15 878 975; E 971 204	ORIGINATED BY BL
DIST 6 HWY 427	BORING DATE Dec. 16, 1976	COMPILED BY BL
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger	CHECKED BY

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
524.9	Ground Level									
0.0	Fill Clayey silt some sand & gravel					520				
			1	SS	17					15 57 (28)
	Fine crushed stones, white, with fines		2	SS	12					8 67 (25)
						510				
			3	SS	7					
503.4	Glacial Till @ 503.5		4	SS	32					
21.5	End of Borehole Note: Hole was dry on completion									

15 ◊ 5 (20, 10) % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 509

Contract 76-01 LOCATION Co-ords N 15 878 978; E 971 208 ORIGINATED BY BL

DIST 6 HWY 427 BORING DATE Dec. 16, 1976 COMPILED BY BL

DATUM Geodetic BOREHOLE TYPE Hollow Stem CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES	ELEV				
524.9	Ground Level									
0.0	Fill: Clayey silt some sand									
			1	SS	35	520				
	Brown Silty Sand		2	SS	14					
	Fine crushed stones , white		3	SS	11	510				
505.4	Traces of Organics		4	AS	-					
503.4	Glacial Till		5	SS	45/4"					
21.5	End of Borehole									

Note: Hole was dry on completion.

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 510

Contract 76-01   LOCATION Co-ords N 15 878 982; E 971 212   ORIGINATED BY BL

DIST 6 HWY 427   BORING DATE Dec. 15, 1976   COMPILED BY BL

DATUM Geodetic   BOREHOLE TYPE Hollow Stem   CHECKED BY

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
524.9	Ground Level									
0.0	Fill: Clayey silt some sand & gravel					520				
			1	SS	13					
	fine crushed stones, white, with fines		2	SS	6					
						510				
			3	SS	13					
504.9										
503.4	Glacial Till		4	SS	85					
21.5	End of Borehole									
	Note: Hole was dry on completion									

20 / 15 – 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 511

Contract 76 -01 | LOCATION Co-ords. N 15 879 012; E 971 167 | ORIGINATED BY EL

DIST 6 HWY 427 | BORING DATE December 15, 1976 | COMPILED BY MM

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY [signature]

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100; SHEAR STRENGTH: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

LIQUID LIMIT w_L; PLASTIC LIMIT w_P; WATER CONTENT w; WATER CONTENT %

UNIT WEIGHT γ; REMARKS %: GR SA SI CL

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV
525.9	Ground Level				
0.0	Fill: Clayey silt some sand				
	fine crushed stones, white	1	SS	20	520
		2	SS	12	
	brown silty sand some cobbles & gravel	3	SS	11	510
		4	SS	8	
500.4 / 25.5	Glacial Till: very stiff to hard	5	SS	38	500
494.4 / 31.5	End of Borehole. Note: Hole was wet at 30ft. deep.	6	SS	50/4"	

BOREHOLE No 512

Co-ords. N 15 879 015; E 971 171

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV
525.9	Ground Level				
0.0	Fill Clayey silt				
	fine crushed stones, white				520
	Brown Sand with gravel				
	Silty sand/Sandy Silt numerous cobbles & gravel traces of clay				510
503.9 / 22.0	Glacial Till: Hard	1	SS	88	500
499.4 / 26.5	End of Borehole. Note: hole was dry on completion				

BOREHOLE No 513

Co-ords. N 15 879 018; E 971 175

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV
525.9	Ground Level				
0.0	Fill: Clayey silt				
	Sandy silt brown				520
	fine crushed stones white				
	Silty Sand/Sandy silt brown with cobbles & gravel				510
504.9 / 21.0	Traces of Clay. Glacial Till				
500.9 / 25.0	End of Borehole. Note: water level not established				

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 514

Contract 76-01 — LOCATION Co-ords. N 15 878 880; E 971 224 — ORIGINATED BY BL

DIST 6 HWY 427 — BORING DATE December 10, 1976 — COMPILED BY MM

DATUM Geodetic — BOREHOLE TYPE Hollow Stem Auger — CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	WATER CONTENT %	REMARKS % GR SA SI CL
515.6	Ground Level						
0.0	Fill: Clayey silt some sand & gravel	1	SS	10			
		2	SS	11	510		
	Silty sand to sandy silt	3	SS	51		o	12 63 (25)
502.6	with gravel & cobbles	4	SS	37		o	13 70 (17)
13.0	Very clayey Glacial Till: Het Mix. of clayey silt, sand & Gravel	5	SS	23			
		6	SS	28	500	w_p–o–w_L	
		7	SS	60/5"			
492.6	Very stiff to hard	8	SS	100/6"		o	
23.0	End of Borehole						
	Note: water level not established						

Legend: LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W. DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100. SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE. WATER CONTENT % 10 20 30 40. UNIT WEIGHT γ. STRAT. PLOT.

20 / 15 – 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 515

Contract 76-01 | LOCATION Co-ords. N 15 878 909; E 971 199 | ORIGINATED BY BL

DIST 6 HWY 427 | BORING DATE December 10, 1976 | COMPILED BY MM

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY [signature]

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	REMARKS % GR SA SI CL
516.9	Ground Level					
0.0	Fill: Clayey silt					
	Sandy silt to Silty sand	1	SS	18		
	some gravel & occasional cobbles	2	SS	29	510	
		3	SS	80		
		4	SS	19		10 73 (17)
503.9	traces of clay					
13.0	clayey	5	SS	28		
	Glacial Till: Het. Mix. of clayey Silt, Sand & Gravel	6	SS	24	500	
		7	SS	60		
	Very Stiff	8	SS	105/4"	490	22 43 (35)
486.6	to Hard	9	SS	60/3"		
30.3	End of Borehole					
	NOTE: Hole was wet at 20ft. deep					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT % 10 20 30 40

UNIT WEIGHT γ

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 520

Contract 76-01

LOCATION Co-ords N 15 879 051; E 971 129

ORIGINATED BY BL

DIST 6 HWY 427

BORING DATE Dec. 16, 1976

COMPILED BY BL

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; WATER CONTENT % 10 20 30 40	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
526.5	Ground Level									
0.0	Fill:									
	Silty Sand to Sandy Silt, Reddish Brown some gravel & cobbles. traces of clay		1	SS	18	520				
			2	SS	22					
	Also some fine, white crushed stones		3	SS	27	510		○		30 53 (17)
			4	SS	11					
501.5	Glacial Till at 502.5									
25.0	End of Borehole									
	Note: Water Level not Established									

BOREHOLE No 521

Co-ords. N 15 879 054; E 971 133

ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				REMARKS
526.5	Ground Level									
0.0	Fill: Silty sand with gravel									
	clayey silt		1	SS	16	520				
	fine crushed stones white		2	SS	25					15 66 (19)
515.1										
11.5	End of Borehole									
	NOTE: Water Level not established									

20
15 ⊕ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 530

Contract 76-01

DIST 6 HWY 427

DATUM Geodetic

LOCATION Co-ords N 15 878 882; E 971 163

BORING DATE Dec. 18, 1976

BOREHOLE TYPE Hollow Stem Auger

ORIGINATED BY BL

COMPILED BY BL

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W; WATER CONTENT % 10 20 30 40	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
519.0	Ground Level									
0.0	Fill: Clayey Silt some sand & traces of gravel									
						510				
			1	SS	24				Org.	Cont.
			2	SS	25				4.2%	
503.0 16.0	Some Topsoil & root hair		3	SS	26					
500.0	Glacial Till		4	SS	41					
19.0	End of Borehole									
	NOTE: Hole stayed dry & open on completion.									

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 533

Contract 76-01

DIST 6 HWY 427 | LOCATION Co-ords N 15 878 901; E 971 183 | ORIGINATED BY BL

DATUM Geodetic | BORING DATE Dec. 18, 1976 | COMPILED BY BL

BOREHOLE TYPE Hollow Stem Auger | CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % 10 20 30 40	UNIT WEIGHT γ	REMARKS % GR SA SI CL
515.0	Ground Level									
0.0	Fill: Clayey silt some sand					510				
	Silty Sand, some Cobbles & gravel, traces of clay. Reddish Brown		1	SS	78					12 66 (22)
			2	SS	33					
501.5 / 13.5	-Topsoil-		3	SS	35	500		o		Org. Cont. 2.3%
	Glacial Till:									
496.0	Hard		4	SS	83					
19.0	End of Borehole									

Note:

Hole was dry & open on completion.

20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 531

Contract 76-01	LOCATION Co-ords N 15 878 920; E 971 151	ORIGINATED BY BL
DIST 6 HWY 427	BORING DATE Dec. 18, 1976	COMPILED BY BL
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger	CHECKED BY [illegible]

ELEV DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
518.0	Ground Level									
0.0	Fill: Clayey silt, some sand & gravel					510				
503.5			1	SS	36					3 45 35 17
501.5	Glacial Till		2	SS	61					
16.5	End of Borehole Note: Hole stayed dry & open on completion									

BOREHOLE No 532

Co-ords. N 15 878 913; E 971 146

ELEV DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER ELEV	REMARKS % GR SA SI CL
518.5	Ground Level						
0.0	Fill: Clayey Silt						
508.5	Silty Sand		1	SS	13	510	8 70 (22)
507.0	Glacial Till		2	SS	22		
11.5	End of Borehole Note: Hole was dry and open on completion.						

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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 534

Contract 76-01 | LOCATION Co-ords N 15 878 847; E 971 069 | ORIGINATED BY BL

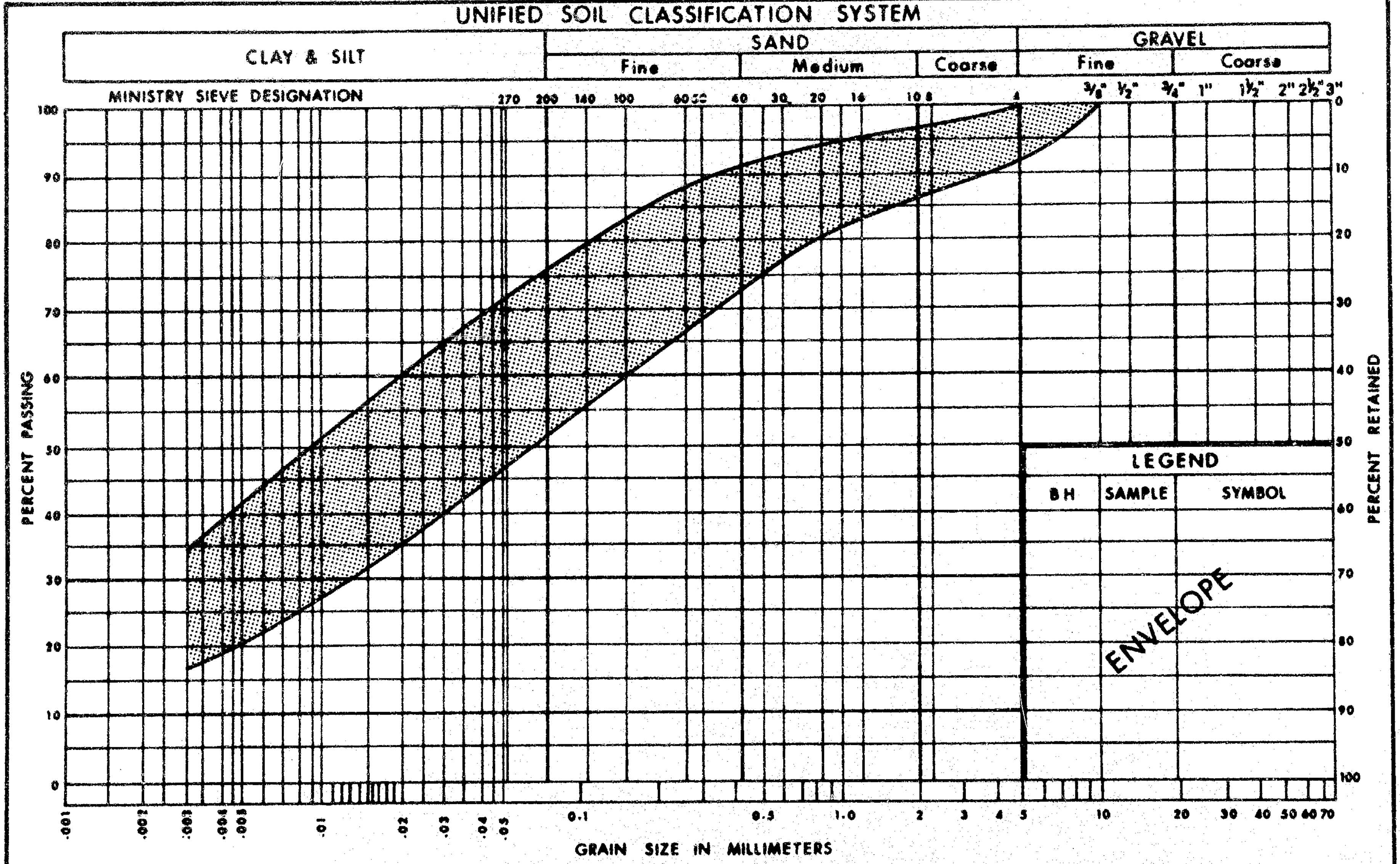
DIST 6 HWY 427 | BORING DATE Dec. 18, 1976 | COMPILED BY BLA

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY [illegible]

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH	WATER CONTENT % (10 20 30)	UNIT WEIGHT γ	REMARKS % GR SA SI CL
557.0	Ground Level									
0.0	Fill: Clayey Silt some sand traces of gravel Brown to brownish grey									
			1	SS	13	550				
			2	SS	13	540		⊢o⊣		0 27 47 26
			3	SS	18	530				
521.5 / 35.5	Glacial Till: Hard					520				
518.0			4	SS	75			o		10 30 40 20
39.0	End of Borehole. Note: Hole was dry & & open on completion									

Shear strength legend: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE. Liquid limit W_L, plastic limit W_P, water content W. Dynamic cone scale: 20 40 60 80 100.

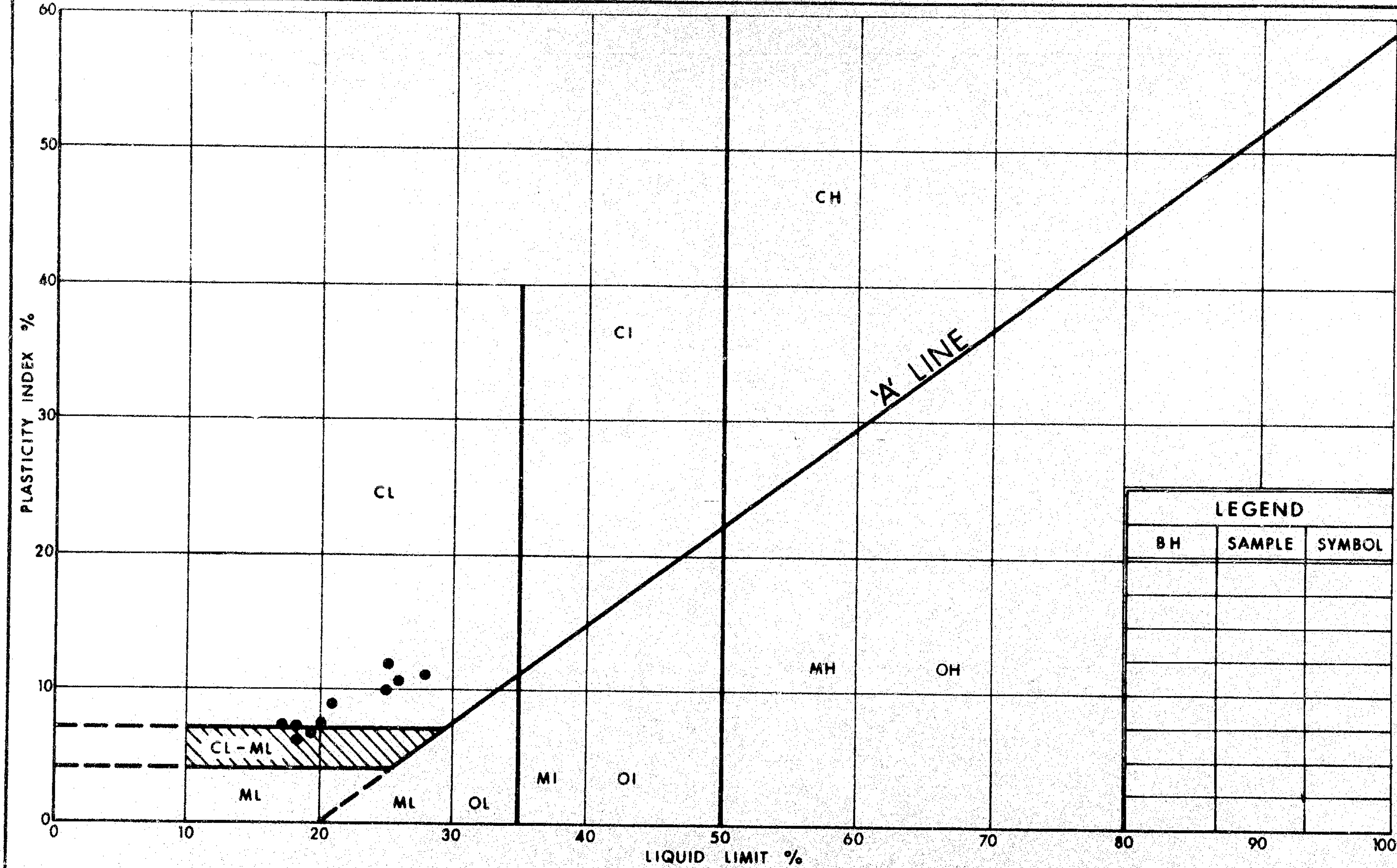
20 / 15 5 / 10 % STRAIN AT FAILURE



Ministry of Transportation and Communications
Ontario

GRAIN SIZE DISTRIBUTION
FILL MATERIAL
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL

FIG No 1
CONT 76-01

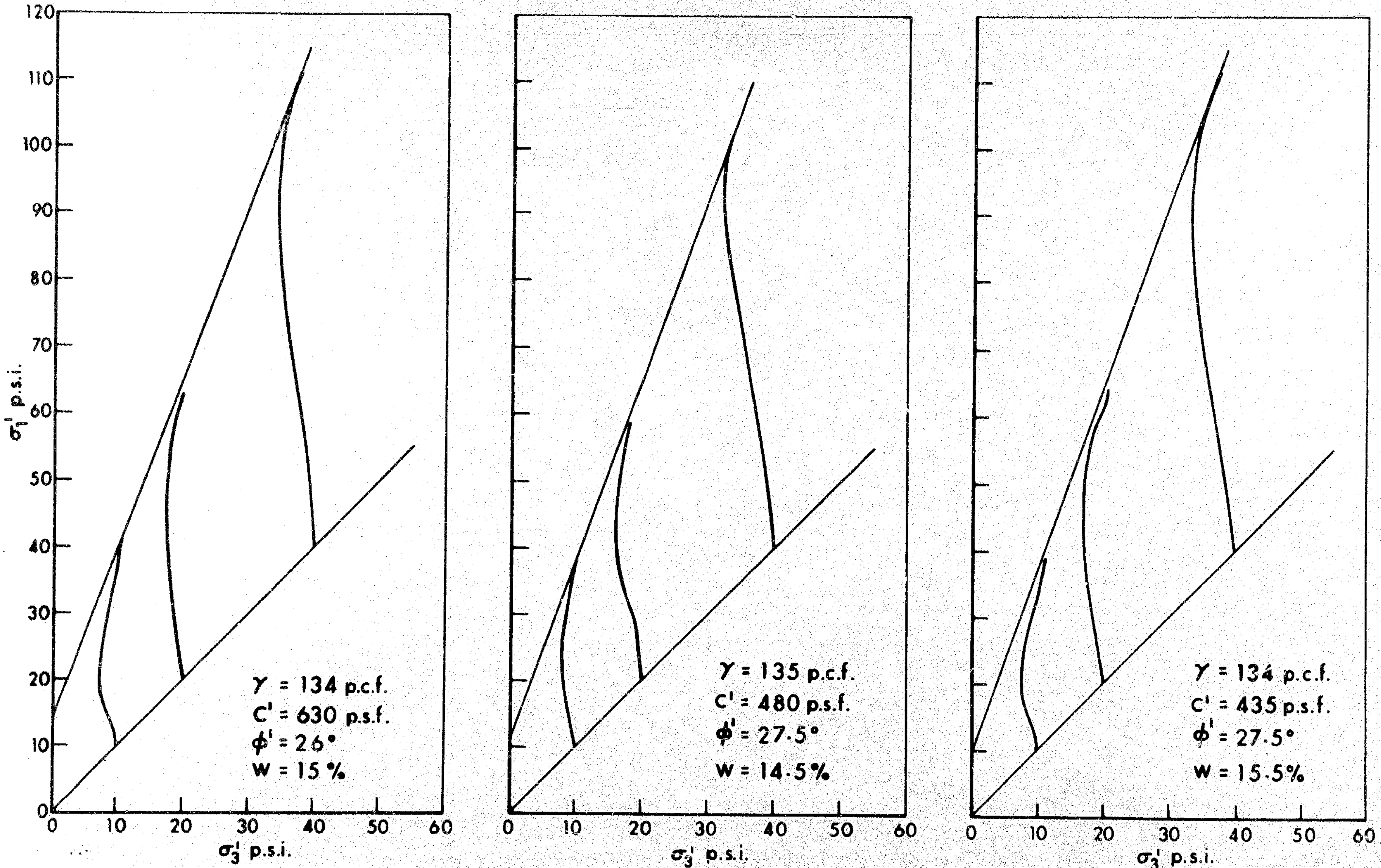


Ministry of Transportation and Communications
Ontario

PLASTICITY CHART
FILL MATERIAL
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL

FIG No 2

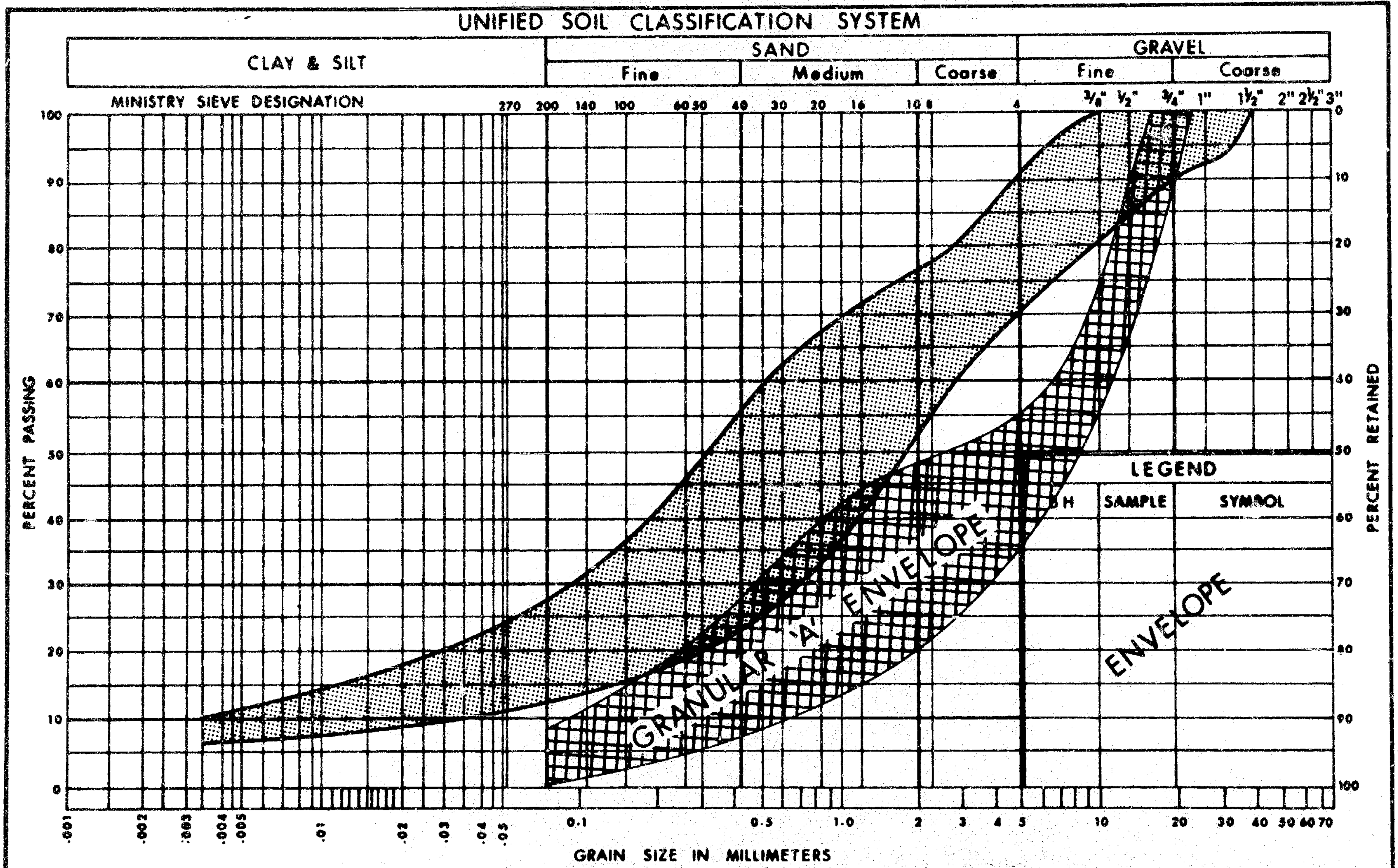
CONT 76-01



Results of Staged Triaxial Tests on Recompacted Clayey Silt Fill

CONT. 76-01

Fig. 3

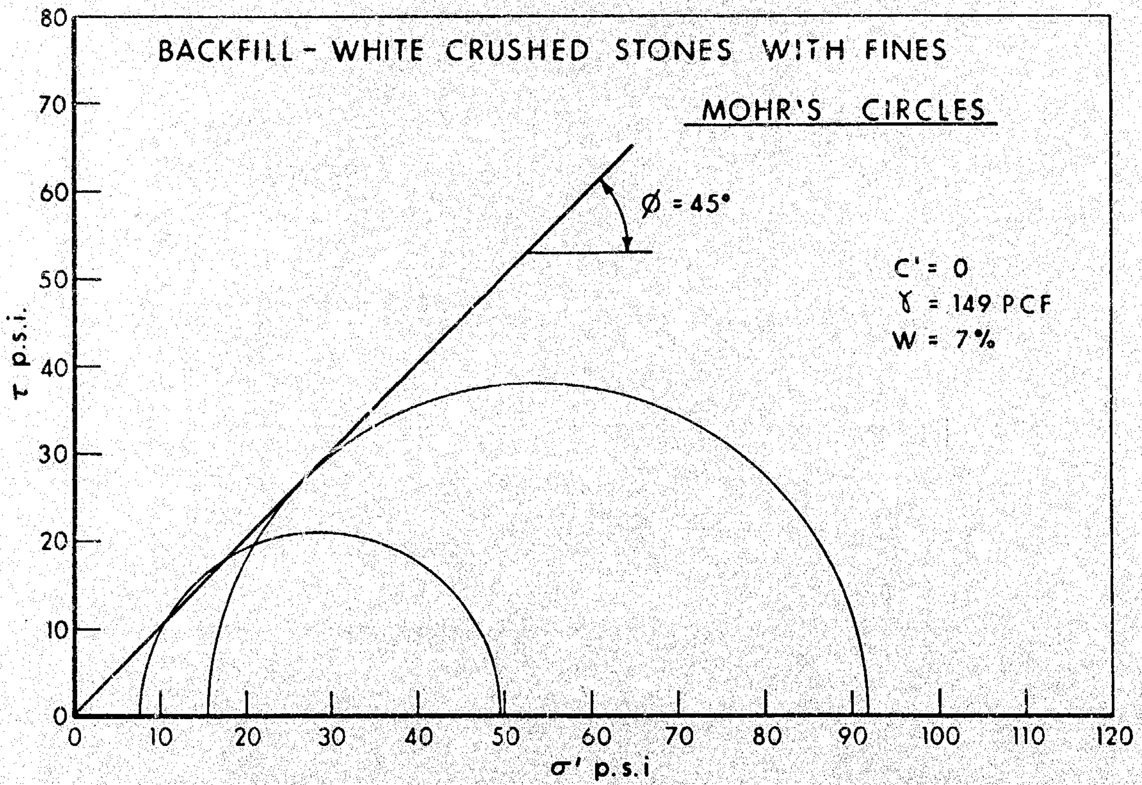


Ministry of Transportation and Communications
Ontario

GRAIN SIZE DISTRIBUTION
GRANULAR BACKFILL MATERIAL
FINE CRUSHED STONES, WHITE

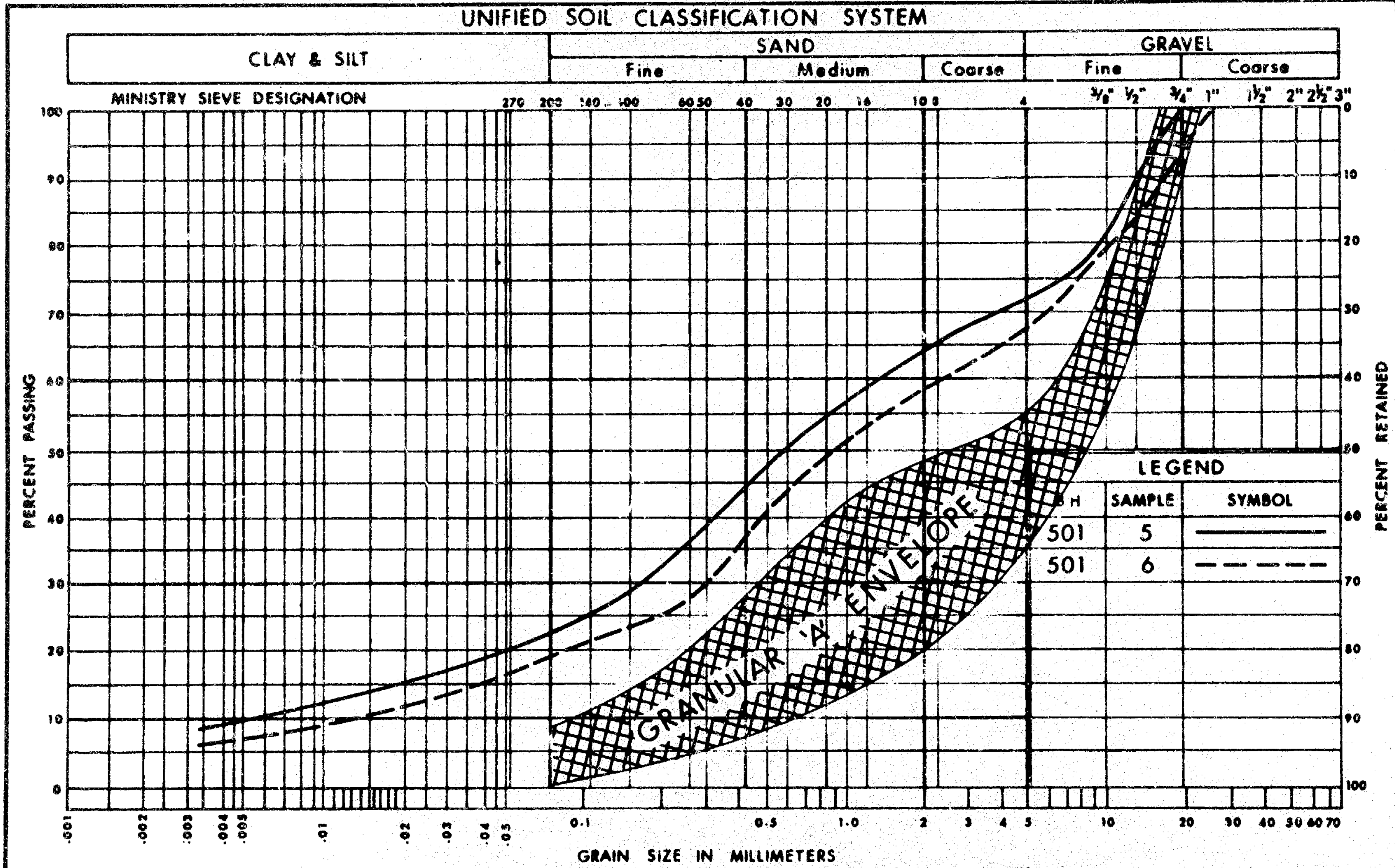
FIG No 4

CONT 76-01



CONT 76-01

Fig 5

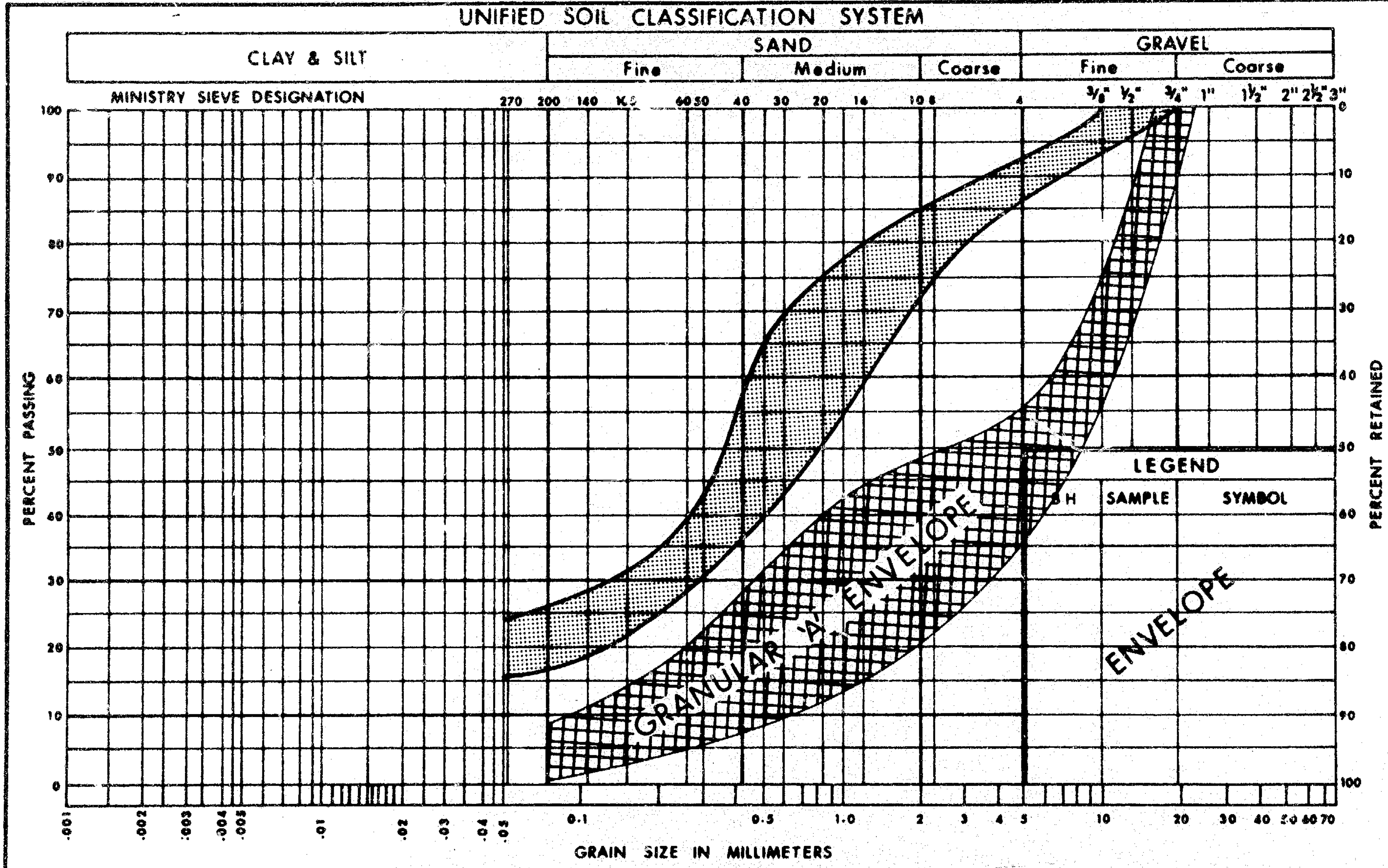


Ministry of Transportation and Communications
Ontario

GRAIN SIZE DISTRIBUTION
GRANULAR BACKFILL MATERIAL
SAND WITH GRAVEL, SOME SILT, TRACE OF CLAY

FIG No 6

CONT 76-01

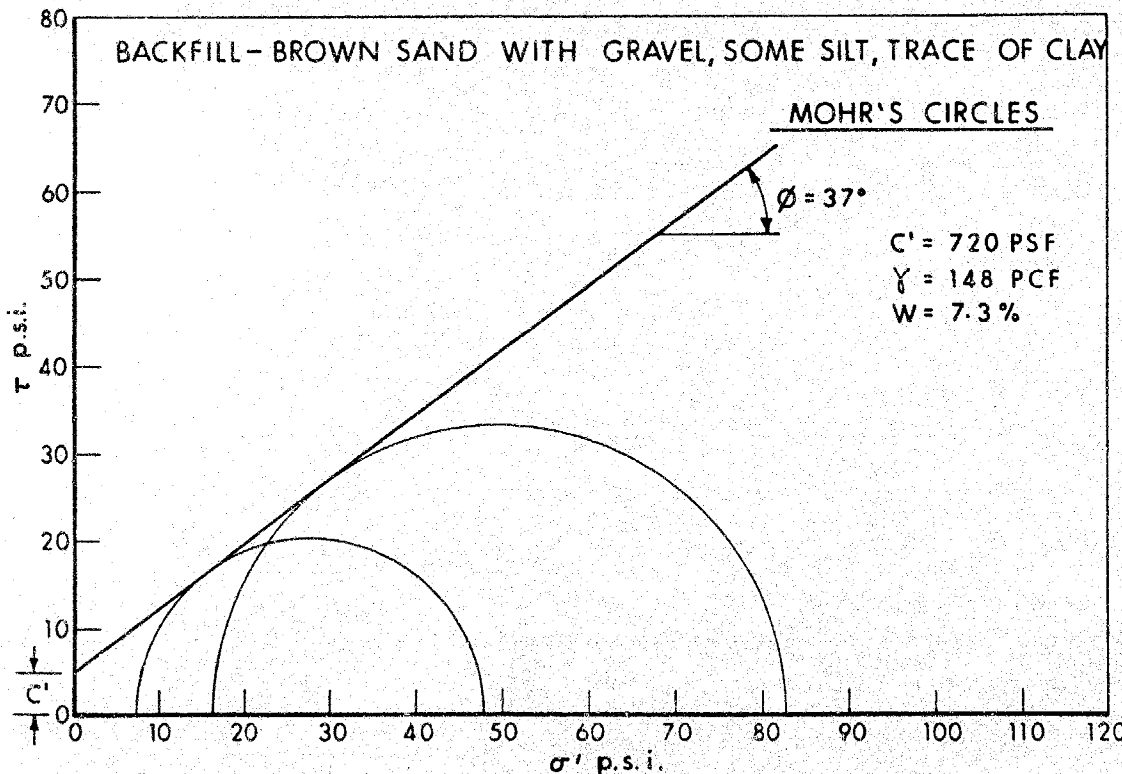


Ministry of Transportation and Communications
Ontario

GRAIN SIZE DISTRIBUTION
GRANULAR BACKFILL MATERIAL
BROWN SILTY SAND, SOME GRAVEL

FIG No 7

CONT 76-01



CONT 76-01 Fig 8

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

To: J.B. Wilkes Executive Director East Bldg.	From: Engineering Materials Office & Structural Office West Bldg.
Attention:	Date: 77 06 29
Our File Ref.	In reply to

Subject:

Report on Failure of
Retaining Wall and Twin Box Culvert
Mimico Creek & Hwy 427
Cont. 76-01 Dist. #6

Attached please find a copy of the report dealing with the failure of Retaining Walls and Twin Box Culvert which occurred in November of last year on Contract 76-01. This report has been prepared jointly by the Structural Office and the Soil Mechanics Section. You will note that the main cause of the failure has been attributed to the use of unusally heavy vibratory equipment for compacting the backfill material behind the structures and to the use of inferior backfill material containing a high percentage of fines. (for purposes of the wall)

We await your instructions regarding further distribution of this report.

A.G. Stermac
Manager,
Engineering Materials Office

R. Dorton
Manager,
Structural Office

AGS/RD/bp
Attach.

cc: J.C. Callaghan
F. Allen

Look into rate of placement of fill in approach – how high – how high when?

only two reasons given here
delete ref to "inadequate backfill"
... to Regional Directors under JBW signature

77 06 03

5201-4

All Regional Directors J.F. Callaghan

77 06 01

Backfill to Retaining Walls
and Large Box Culverts

On a recent contract we experienced the failure of a rigid twin box concrete culvert wall and a cantilever type retaining wall. Investigations indicate the possible existence of excessively high lateral earth pressures acting on structures of this type, particularly when the backfill has a surcharge loading condition. A report on the failure is presently being prepared jointly by the Structural Office and the Soil Mechanics Section.

Preliminary analyses indicate that compaction with heavy vibratory rollers was also a key factor in the generation and transmission of high lateral earth pressures through a dense-graded granular backfill material. For this reason compaction of backfill materials behind retaining walls is being controlled on forthcoming contracts by a Special Provision limiting the use of heavy vibratory equipment to a distance not closer than 25 ft. to the wall or box culvert.

The intent of this memorandum is to advise you of the events that have occurred and the reason for the Special Provision for retaining wall projects.

J.F. Callaghan,
Director,
Design & Construction Branch.

JFC/CEB/sc

c.c. J.B. Wilkes
R. Dorton
A.G. Stermac

capacity of the wall calculated for the sections where the cracks occurred (i.e. immediately above the floor lining and immediately above the cut-offs of the F reinforcing bars) still has a factor of safety greater than one with respect to th bending moment caused by the lateral force induced by the retained soil mass (Appendix VI).

The heavy vibratory compacting equipment used in this project resulted in over-compaction of the granular backfill and the related earth fill. This method of construction contributed to large horizontal stresses in the soil mass. These residual (locked-in) stresses induced additional lateral earth pressures on the retaining structures. As mentioned previously, the walls would be subjected to high flexural stresses due to sloping surcharge. These additional lateral earth pressures induced by heavy vibratory compacting equipment, together with the sloping surcharge, resulted in the failure of the retaining structures.

In the distressed area, well graded granular material containing a high portion of fines was used as backfill. This material is very easy to compact and attains a very high density when compacted. Further, because of the high degree of inter-locking of particles, the compactive energy transmitted to this material will be locked in and accumulated. Consequently, very high horizontal residual stresses are resulted in the soil mass.

CORRECTIVE MEASURES INCORPORATED DURING CONSTRUCTION

Since the cause of failure was established as excessive earth pressure, the immediate step was to release the earth pressure acting against the culvert and the retaining wall.

To this purpose, the following measures were taken:

1. Removal of the approach fill immediately behind the culvert and retaining wall was recommended. This was achieved by reducing the width of the approach fill of the overpass, as the required width was designed as a provision for future widening of Hwy. 427 roadway in the ultimate scheme. As a result, the toe of this slope was moved 15 feet to 55 feet away from the back of the culver and the retaining wall.

2. A trench was excavated behind the culvert to a depth of about 3 feet from the bottom of the culvert. This space was backfilled with sand and gravel which was loosely placed to relieve the residual stresses induced by compaction. The pertinent details are shown in Drawing 7601-A.

In addition, the damaged retaining wall and the culvert wall were repaired. The repair works involved the following:

1. The 105 feet of damaged retaining wall was removed to 2 feet above the footing and replaced with new concrete and reinforcing steel.

2. The end 30 feet of the east wall of the culvert was partially removed and replaced with new concrete and reinforcing steel.

3. A relieving slab 14 inches thick was built against the outside wall of the culvert to strengthen the culvert wall.

To find out the actual earth pressure acting against the culvert and retaining wall, the Research and Development Branch carried out measurements and installed strain gauges in the new retaining wall, part of the culvert and the relieving slab. The instrumentation layout and the related data is given in Appendix VII of this report. ?

RECOMMENDATIONS FOR FUTURE PROJECTS

The failure of the retaining wall and the box culvert has revealed that a number of inadequacies exist in the present Ministry's design standards for retaining structures and the construction of the backfill. To prevent failures of similar nature from re-occurrence in the future, the following ~~considerations~~ should be ~~adopted.~~ considered.

1. The lateral earth pressure is known to be affected by a number of factors, among which are the deformation characteristics of the retaining wall types of backfill material, method of placing the backfill, surcharge conditions, etc. If the wall is a rigid type, the earth pressure at-rest (Ko) should be considered. In addition, the full effect of the sloping surcharge on the retaining structures should be taken into account.

2. If the backfill behind the wall is heavily compacted or is subject to vibrations, the earth pressure will increase above the value at-rest (Ko). Therefore, it is recommended that no heavy vibratory equipment should be allowed to operate in the close proximity of the retaining structures. Such equipment should be kept away from the wall at least 15 feet or 2/3 of the height of the wall, whichever is the greater.

3. The backfill should be compacted, but compaction to over 100% of the Proctor how density should be avoided. It is further recommended that the granular backfill should be compacted to a minimum of 90% and a maximum of 95% of the

maximum dry density as determined by the current M.T.C. procedures and in lifts of 6 to 9 inches.

4. It has been stated in textbooks and manuals that free draining type of granular material should be used for backfill. The importance of the use of proper backfill material cannot be overemphasized. Backfill should be simil to M.T.C.'s Granular 'A'. Material containing a high portion of fines such the one used in this project should not be used. Further, the current M.T.C. backfill standards for retaining walls and box culverts should be revised accordingly.

5. In recent years the construction industry is very eager to use heavy vibrato equipment for compaction purposes. In this regard, to develop a balance between the construction innovation and design, further field observations to monitor the performance of heavy equipment close to the structure will be required.

6. Joint material used between the channel lining and the retaining wall should reviewed. Also, it may be advisable that backfill should be placed before th channel lining is poured. This will allow the wall to yield or rotate under pressure.

MISCELLANEOUS

The soil mechanics portion of this report was prepared by Mr. B. Ly, Senior Engineer, assisted by Mr. M. MacLean, Project Engineer, and reviewed by Mr. M. Devata, Supervising Engineer. The structural portion was prepared by Mr. W. Lin, Regional Structural Design Engineer and reviewed by Mr. C. Grebski, Structural Design Engineer.

W. Lin, P. Eng.
Regional Structural Design Engineer
Structural Office

M. Devata, P. Eng.
Supervising Engineer
Soil Mechanics Section

June, 1977

FOUNDATION INVESTIGATION REPORT
For
Madawaska River Bridge
Arnprior Bypass
W.P. 198-62-00, Site 29-191
Hwy. 17N, District 9, Ottawa

INTRODUCTION

This report contains the results of our investigations of the subsurface conditions existing at this site. The fieldwork was done in various stages and the results of 18 borings pertinent to the proposed structure and the related approaches are presented. Borings on land were advanced by means of an auger machine or a diamond drill rig. Borings located in the river were done by means of a diamond drill rig mounted on a raft. The borings ranged in depth between 11 feet and 42 feet below ground surface or the prevailing river water level. Bedrock was proven by obtaining NX size rock core samples. In addition, a field permeability test using double packers was also performed in one of the boreholes on either side of the river.

SITE AND GEOLOGY

The site is located near the Town of Arnprior on Lot 1, Conc. B, Township of McNab, County of Renfrew and is situated some 550 feet downstream north of the existing Ontario Hydro Dam. At this location, Madawaska River is about 300 feet wide and is deeply incised into the surrounding tablelands. The river water level, which is controlled by the Hydro Dam, generally varies between elevations 254 feet and 258 feet. According to available information, the maximum depth of water in the river in the vicinity of the site is in the order of 45 feet.

The river banks in this locale vary from moderately steep rockface on the east to steeply shelving rockface on the west. In the vicinity of the site bedrock is generally exposed, except at certain locations where it exists at a shallow depth. The area east of the river is sparsely vegetated, whereas the area west of the river is heavily covered with spruces, pines and cedars. According to available information, the approaches to the structure have been constructed to their full height under MTC Contract 78-42.

Geologically, the area is located between the Precambrian Upland to the north and west and the Ottawa Lowland to the south and east. A depression geologically known as the 'Ottawa-Bonnechere' graben, is located in the County of Renfrew. Within it are prominent east-west trending scarps (fault zones). One fault zone is located just upstream of the main dam area. Bedrock here mainly consists of crystalline limestone and has been subjected to faulting, weathering and erosion. The overburden mainly consists of sensitive marine clays deposited by the Champlain Sea when it inundated the Ottawa-St. Lawrence Lowland.

SUBSURFACE CONDITIONS

General

In the area under investigation, bedrock is exposed or exists at a shallow depth. The overburden, wherever it exists, mainly consists of a thin layer of clayey silt. On the east side of the river, the thickness of the overburden appears to be increasing towards the Hydro Dam. At certain locations on the east bank rockfill up to 8 feet thick was placed on the original ground after construction of the Hydro Dam. The locations and elevations of the borings are shown on Drawing No. 29-191-2, together with the estimated stratigraphical sections. It should be noted that the approach fills were already constructed under a separate contract. For details about the type of fill material and the geometry of the approaches, references should be made to Contract 78-42. A description of the subsoil and bedrock conditions encountered during the time of investigation is as follows.

Clayey Silt

This cohesive deposit has a variable thickness, being generally less than 2.5 feet thick on the west side of the river to about 12 feet thick at one location on the east side of the river. According to the geology of the area, the cohesive subsoil was deposited by the post-glacial Champlain Sea. The clayey silt is desiccated, brittle and fissured. The Standard Penetration Test 'N' values recorded in the cohesive deposit ranged from 9 blows/foot to 18 blows/foot and it is inferred from this that the consistency of the clayey silt is stiff to very stiff.

Bedrock

Bedrock, in general, is exposed on both sides of the river banks and also within the river bed. Elsewhere in some locations, bedrock is covered with a thin layer of clayey silt.

Bedrock is a crystalline limestone of Precambrian Age. This metamorphic rock is light grey in color, with a coarse texture and generally hard. The beddings generally incline towards the river at 45° to 55°, but sometimes are contorted or largely destroyed during metamorphism. Jointing is well developed in the bedrock. Two principal joint sets exist throughout the site, one being parallel to the beddings and the other perpendicular to the beddings. The joints are continuous, smooth and moderately to widely spaced. In addition to the jointings, there are occasional discontinuous steep fractures. Some of these fractures are open and weathered.

In the upper 15 feet of the bedrock the Rock Quality Designation (RQD) varies randomly from 25% to 100%, indicating a poor to excellent rock quality. At depths below 15 feet from bedrock surface, the rock quality can be classified as excellent, as evidenced by the consistently high RQD values of 90% to 100%.

Detailed descriptions of the rock cores are given in the Record of Borehole Sheets and the Diamond Drill Record Sheets included in the Appendix.

Tightness of the joints were examined by means of borehole permeability tests using double packers. The results of the tests are reported in the Record of Borehole Sheets. It can be inferred from these results that the joints in the bedrock on the west bank below a depth of 10 feet are generally tight, whereas those in the east bank are moderately open for at least to a depth of 35 feet below the rock surface.

Groundwater Conditions

The results of groundwater observations are reported in the Record of Borehole Sheets. The groundwater level at the west bank appears to be influenced by the continuity of the joints and fractures in the bedrock and varies randomly between elevation 260 and elevation

268, considerably higher than the river water level. At the east bank where jointings in the bedrock are moderately open, the groundwater level descended gradually towards the river from elevation 269 in the high ground to elevation 255 near the river. The river water level at the time of investigation was found to be at elevation 255.

B. Ly

B. Ly, P. Eng.
Senior Engineer

M. Devata

M. Devata, P. Eng.
Supervising Engineer

APPENDIX

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

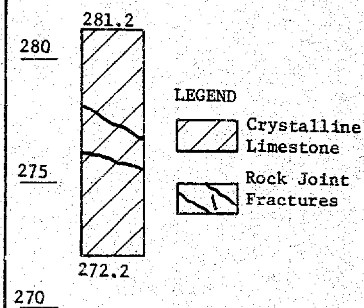
HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 5

WP 198-62-00	LOCATION Co-ords. N 16,505,627; E 1,038,463	ORIGINATED BY BL
DIST 9 HWY 17N Line'C' EBL	BORING DATE March 31, 1977	COMPILED BY BL
DATUM Geodetic	BOREHOLE TYPE NX Rock Coring	CHECKED BY [illegible]

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —w_L; PLASTIC LIMIT —w_P; WATER CONTENT —w; w_P — w — w_L; WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
283.8	Ground Surface									
0.0 / 281.2	Topsoil & clayey silt									
2.6	Crystalline Limestone Bedrock. Coarse texture, weathered joints at 6.4 ft. and 7.8 ft.		1	NX RC	Rec 100 %	280			100	
272.2			2	NX RC	Rec 100 %				100	
11.6	End of Borehole Note: GWL not established									

ROCK CORE DESCRIPTION



20 15 ◇ 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

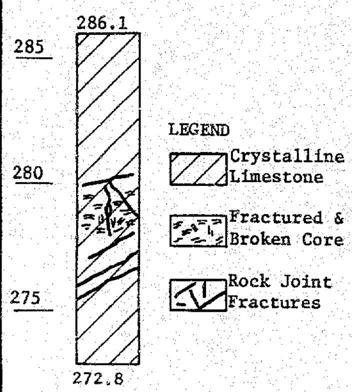
HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 6

W P 198-62-00	LOCATION Co-ords. N 16,505,580; E 1,038,472	ORIGINATED BY BL
DIST 9 HWY 17N Line 'C' EBL	BORING DATE March 31,1977	COMPILED BY BL
DATUM Geodetic	BOREHOLE TYPE NX Rock Coring	CHECKED BY

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W / WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
286.1	Ground Surface									
0.0	Crystalline Limestone Bedrock. Coarse texture, several joint fractures from 6 ft. to 10 ft.		1	NX RC	Rec 96%				95	
			2	NX RC	Rec 90%	280			75	
272.8			3	NX RC	Rec 100 %				95	
13.3	End of Borehole									
	Note: GWL not established									

ROCK CORE DESCRIPTION



20
15 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 7

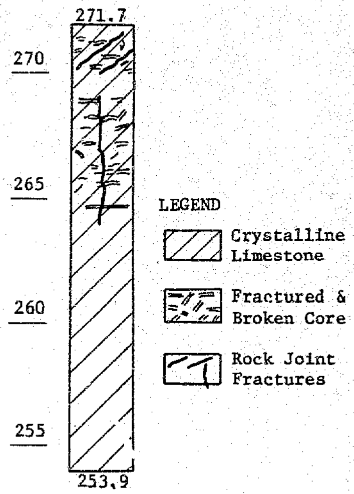
WP 198-62-00 LOCATION Co-ords. N 16,505,598; E 1,038,678 ORIGINATED BY BL

DIST 9 HWY 17N Line 'C' EBL BORING DATE March 29, 1977 COMPILED BY BL

DATUM Geodetic BOREHOLE TYPE NX Rock Coring CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; WATER CONTENT %		REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV			RQD %	% GR SA SI CL
273.4	Ground Surface									
271.7 0.0	Topsoil & clayey silt									
1.7	Crystalline Limestone Bedrock. Coarse texture. Weathered bedding plane fractures at 1.7'-2.2'		1	NX RC	Rec 75%	270			50	
			2	NX RC	Rec 95%				65	
	A vertical fracture at 4.5' to 9.5'		3	NX RC	Rec 100 %	260			95	
253.9			4	NX RC	Rec 100 %				95	
19.5	End of Borehole									

ROCK CORE DESCRIPTION



20 15 5 % STRAIN AT FAILURE

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

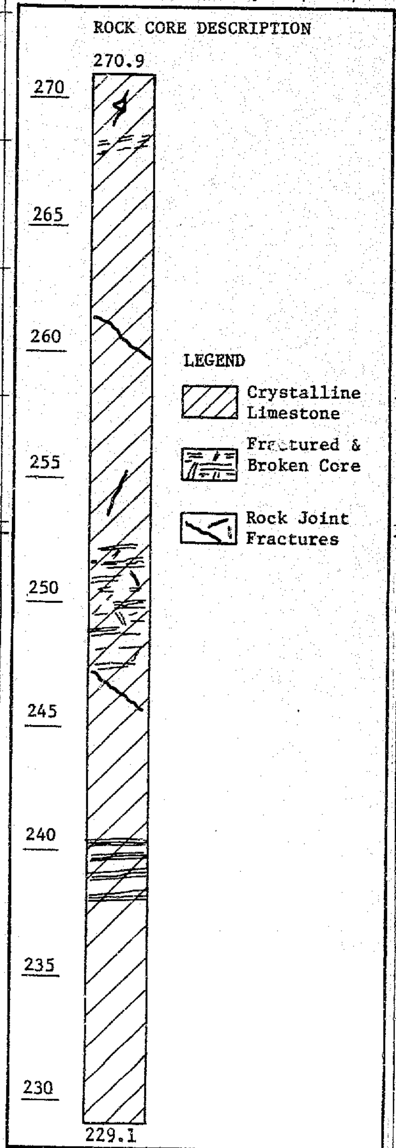
RECORD OF BOREHOLE № 7A

W P 198-62-00 LOCATION Co-ords. N 16,505,600; E 1,038,686 ORIGINATED BY BL

DIST 9 HWY 17N Line 'C' EBL BORING DATE March 30, 1977 COMPILED BY BL

DATUM Geodetic BOREHOLE TYPE NX Coring & Packer Test CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT %	RQD %	REMARKS Packer Test Results Pressure p (p.s.i.)	Flow q (gal/min)
272.0	Ground Surface										
270.9	Clayey silt with boulder		1	NX	50%				–		
1.1	Crystalline Limestone Bedrock. Coarse texture, hard.		2	NX	100 %	270			50	*	*
			3	NX RC	Rec 100 %				90		
	Joint fractures at 2 ft., 11 ft., 18 ft. and 26 ft.		4	NX RC	Rec 95%	260			95	34	60
			5	NX RC	Rec 100 %				100	56	0
			6	NX RC	Rec 100 %	250			100	48	4
			7	NX RC	Rec 100 %				100	108	0
			8	NX RC	Rec 98%	240			95	98	0
			9	NX RC	Rec 100 %				100	83	0
229.1			10	NX RC	Rec 100 %	230			100	68	0
42.9	End of Borehole										
	Note: GWL not established									* Upper Packer could not seal	



20
15 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

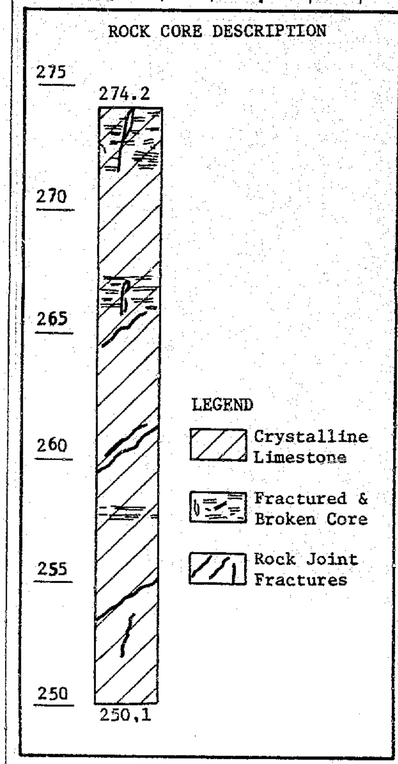
RECORD OF BOREHOLE Nº 8A

W P 198-62-00 LOCATION Co-ords. N 16,505,571; E 1,038,683 ORIGINATED BY BL

DIST 9 HWY 17N Line 'C' ~~EBL~~ BORING DATE March 30, 1977 COMPILED BY BL

DATUM Geodetic BOREHOLE TYPE NX Rock Coring CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W — WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
275.5	Ground Surface									
274.2	Topsoil & clayey silt									
1.3	Crystalline Limestone Bedrock. Coarse texture. Weathered joint fractures		1	NX RC	Rec 75%				25	
			2	NX RC	Rec 90%	270			80	
	parallel to bedding at 10 ft., 15 ft., and 21 ft.		3	NX RC	Rec 73%	260			55	
	Steep fractures at 2.5 ft. and		4	NX RC	Rec 100%				95	
250.1	23 ft.		5	NX RC	Rec 99%				95	
25.4	End of Borehole									



OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 9

W P 198-62-00 LOCATION Co-ords. N 16,505,586; E 1,038,698 ORIGINATED BY BL

DIST 9 HWY 17N Line 'C' EBL BORING DATE March 29 & 30, 1977 COMPILED BY BL

DATUM Geodetic BOREHOLE TYPE NX Rock Coring CHECKED BY [signature]

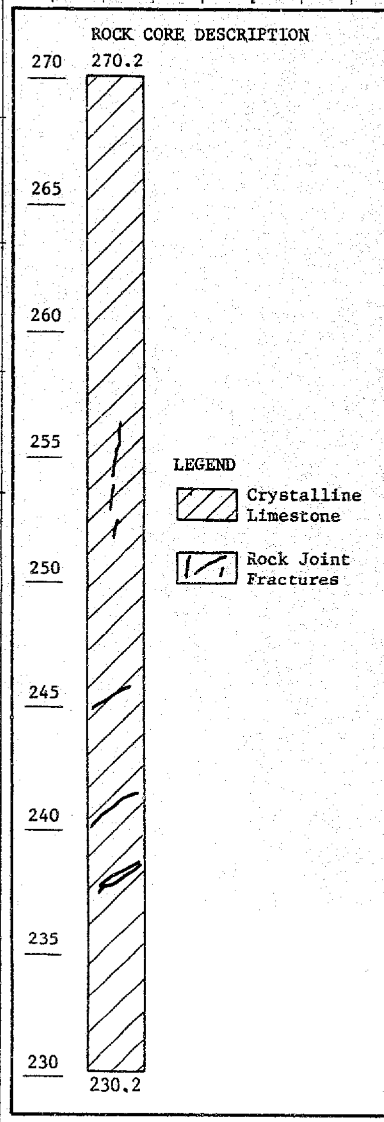
SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	RQD %	REMARKS % GR SA SI CL
272.3	Ground Surface						
270.2 / 0.0	Topsoil & clayey silt				270		
2.1	Crystalline Limestone Bedrock	1	NX RC	Rec 100%		100	
	Coarse texture.	2	NX RC	Rec 100%	260	100	
	Steep, weathered open fractures at 16' - 19.3'	3	NX RC	Rec 100%	▽	100	
		4	NX RC	Rec 100%	250	60	
	Bedding plane fractures at 27 ft., 32 ft. and 35 ft.	5	NX RC	Rec 100%		95	
		6	NX RC	Rec 100%	240	95	
		7	NX RC	Rec 100%		95	
230.2		8	NX RC	Rec 100%		95	
42.1	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W

WATER CONTENT %



20
15 ○ 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 10

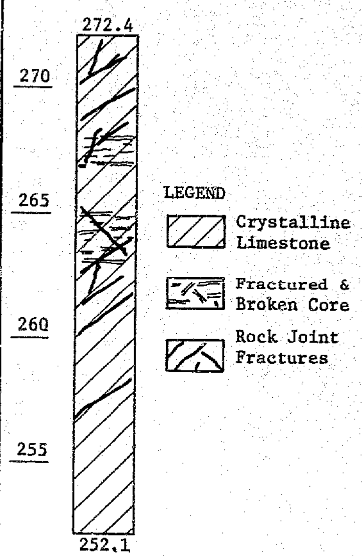
WP 198-62-00 — LOCATION Co-ords. N 16,505,565; E 1,038,706 — ORIGINATED BY BL

DIST 9 — HWY 17N Line'C' EBL — BORING DATE March 29, 1977 — COMPILED BY BL

DATUM Geodetic — BOREHOLE TYPE NX Rock Coring — CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L / PLASTIC LIMIT — W_P / WATER CONTENT — W / W_P W W_L / WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
273.7	Ground Surface									
272.4	Topsoil & Clayey silt									
1.3	Crystalline Limestone. Bedrock		1	NX RC	Rec 99%	270 ▼			70	
	Coarse texture		2	NX RC	Rec 100%				60	
	Several Bedding plane fractures from 3 ft. to 5.7 ft.		3	NX RC	Rec 100%	260			100	
252.1	& from 11.7 ft. to 13.3 ft.		4	NX RC	Rec 100%				100	
21.6	End of Borehole									

ROCK CORE DESCRIPTION



20
15 ⌖ 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

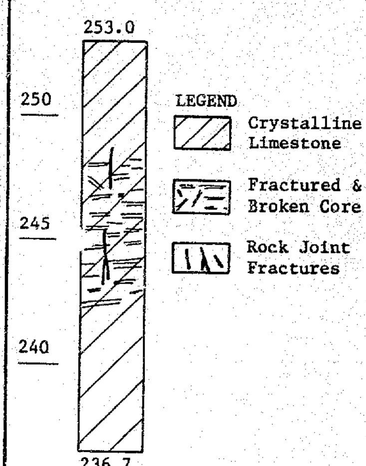
HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 11A

W P 198-62-00	LOCATION Co-ords. N 16,505,577; E 1,039,106	ORIGINATED BY MM
DIST 9 HWY 17N Line'C' EBL	BORING DATE April 6, 1977	COMPILED BY BL
DATUM Geodetic	BOREHOLE TYPE NX Rock Coring	CHECKED BY [signature]

SOIL PROFILE			SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —w_L PLASTIC LIMIT —w_P WATER CONTENT —w / w_P w w_L / WATER CONTENT %		
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV			RQD %	REMARKS % GR SA SI CL
261.5	Ground Level									
0.0	Rockfill		1	NX	-	260				
253.0						▽				
8.5	Crystalline Limestone Bedrock		2	NX RC	Rec 90%				50	
	Coarse texture Vertical joint fractures at 13.5 ft - 14.5 ft.		3	NX RC	Rec 100 %	250			50	
			4	NX RC	Rec 95%				75	
	16.7 ft. - 18.0 ft.		5	NX RC	Rec 100 %	240			100	
236.7										
24.8	End of Borehole									

ROCK CORE DESCRIPTION



OFFICE REPORT ON SOIL EXPLORATION

20
15 5 % STRAIN AT FAILURE

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 12A

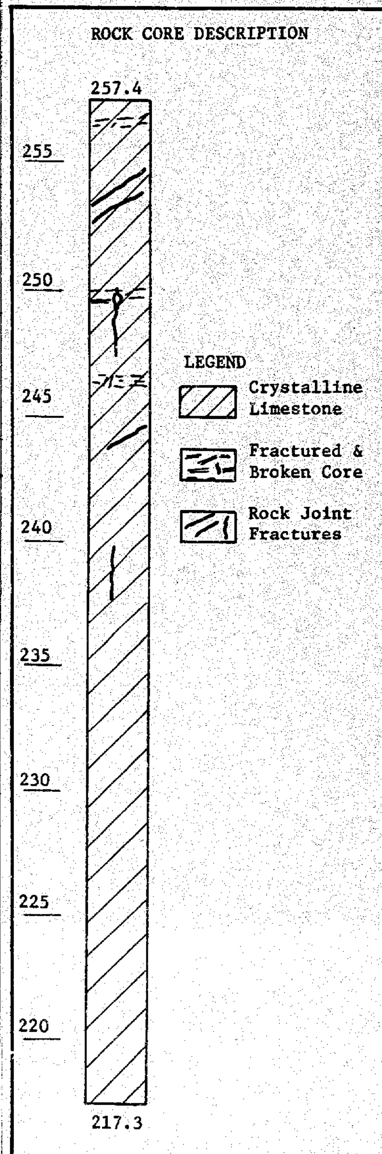
W.P. 198-62-00　LOCATION Co-ords. N 16,505,531; E 1,039,115　ORIGINATED BY MM

DIST 9　HWY 17N Line'C' ~~EBL~~　BORING DATE April 4, 1977　COMPILED BY BL

DATUM Geodetic　BOREHOLE TYPE NX Rock Coring　CHECKED BY Cf.

SOIL PROFILE / SAMPLES / GROUND WATER / DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE / LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W / W_P W W_L WATER CONTENT %

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	RQD %	Packer Test Results: Pressure p (p.s.i.)	Packer Test Results: Flow q (gal/min)
257.4	Ground Level								
0.0	Crystalline Limestone		1	NX	100 %		85		
	Bedrock. Coarse texture Hard		2	NX RC	Rec 100 %	250	91	*	*
	Vertical joint at 8' - 10.8'		3	NX RC	Rec 100 %		46	*	*
	Weathered vertical joint at 17 ft. - 18.5 ft.		4	NX RC	Rec 100 %	240	100	22	4.0
	Bedding plane		5	NX	100 %		40		
	fractures at 3.5 ft, 4.5 ft., and 13.5 ft		6	NX RC	Rec 100 %		100	32	4.5
			7	NX RC	Rec 100 %	230	100	52	2.4
			8	NX RC	Rec 100 %		94	52	2.4
217.3			9	NX RC	Rec 100 %	220	100	82	1
40.1	End of Borehole								
	Note: GWL not established							* Packers could not seal	



20
15 ○ 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

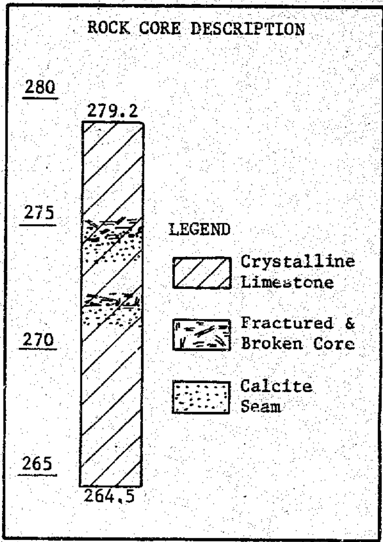
RECORD OF BOREHOLE Nº 15

WP 198-62-00 | LOCATION Co-ords. N 16,505,565; E 1,039,324 | ORIGINATED BY MM

DIST 9 HWY 17N Line'C' EBL | BORING DATE April 4, 1977 | COMPILED BY BL

DATUM Geodetic | BOREHOLE TYPE NX Rock Coring | CHECKED BY [signature]

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W / WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
279.2	Ground Level									
0.0	Crystalline Limestone Bedrock Hard Soft weathered calcite seams at 4.2' - 5.6' and 7.3' - 8.1'		1	NX RC	Rec 100 %				75	
			2	NX RC	Rec 95%	270			50	
264.5			3	NX RC	Rec 100 %				100	
14.7	End of Borehole									



20 15 10 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 16

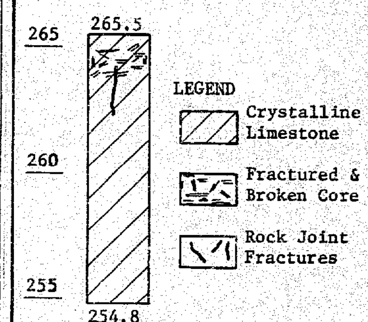
WP 198-62-00 LOCATION Co-ords. N 16,505,505; E 1,039,319 ORIGINATED BY MM

DIST 9 HWY 17N Line'C' EBL BORING DATE April 4, 1977 COMPILED BY B.

DATUM Geodetic BOREHOLE TYPE 3¼" Hollow Stem Auger & NX Rock Coring CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w / WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
273.1	Ground Level									
0.0 / 265.5	Some boulder & cobbles over clayey silt, trace to some sand, Brown, Stiff					270				
7.6	Crystalline Limestone Bedrock Coarse texture Steep joint fracture at 17.5 ft.		1	NX RC	Rec 100 %	260			100	
254.8			2	NX RC	Rec 100 %				70	
18.3	End of Borehole									
	Note: GWL not established									

ROCK CORE DESCRIPTION



20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF PROBEHOLE Nº 17 & 18

W P 198-62-00 LOCATION See Below ORIGINATED BY MM

DIST 9 HWY 17N Line 'C' EBL BORING DATE April 4, 1977 COMPILED BY BL

DATUM Geodetic BOREHOLE TYPE 3¼" Hollow Stem Auger CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
							PROBEHOLE #17	Co-ords. N 16,505,520; E 1,039,320		
273.5	Ground Level									
272.0	Fill: sand with gravel									
1.5	End of Probehole Auger refusal on probable bedrock					270				
							PROBEHOLE #18	Co-ords N 16,505,550; E 1,039,322		
276.1	Ground Level									
274.6	Fill: sand with gravel									
1.5	End of Probehole Auger refusal on probable bedrock					270				

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 51

W.P. 198-62-00 LOCATION Co-ords. N 16,505,584; E 1,038,954 ORIGINATED BY HS

DIST 9 HWY 17N Line'C' ~~EBL~~ BORING DATE June 22, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE NX Rock Core CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	RQD %	REMARKS
254.4	Water Level						
0.0	Water				250		
245.5	River Bottom						
8.9	Crystalline Limestone Bedrock Sound	1	NX RC	Rec 100 %		81	
		2	NX RC	Rec 100 %	240	67	
		3	NX RC	Rec 100%		93	
225.3		4	NX RC	Rec 100 %	230	100	
29.1	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %
REMARKS: % GR SA SI CL

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 52

WP 198-62-00 | LOCATION Co-ords. N 16,505,563; E 1,038,962 | ORIGINATED BY HS

DIST 9 HWY 17N Line'C' EBL | BORING DATE June 23, 1976 | COMPILED BY SM

DATUM Geodetic | BOREHOLE TYPE NX Rock Core | CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —w_L PLASTIC LIMIT —w_P WATER CONTENT —w; WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
254.4	Water Level									
0.0	Water					250				
246.1	River Bottom									
8.3	Crystalline Limestone Bedrock		1	RC	84%				58	
			2	NX RC	Rec 98%	240			88	
	Sound		3	NX RC	Rec 99%				88	
229.6			4	NX RC	Rec 100 %	230			100	
24.8	End of borehole									

20
15 ◇ 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 53

WP 198-62-00 | LOCATION Co-ords. N 16,505,605; E 1,038,941 | ORIGINATED BY HS

DIST 9 HWY 17N Line 'C' EBL | BORING DATE June 23 & 24, 1976 | COMPILED BY SM

DATUM Geodetic | BOREHOLE TYPE NX Rock Core | CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT %	RQD %	REMARKS % GR SA SI CL
254.7	Water Level									
0.0	Water					250				
						240				
233.3	River Bottom									
21.4	Crystalline Limestone Bedrock Sound		1	NX RC	Rec 100 %	230			78	
			2	NX RC	Rec 94%				89	
			3	NX RC	Rec 97%	220			69	
213.2			4	NX RC	Rec 100 %				100	
41.5	End of Borehole									

20 15 ✧ 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 54

WP 198-62-00 LOCATION Co-ords. N 16,505,468; E 1,039,250 ORIGINATED BY SM

DIST 9 HWY 17N Line 'C' EBL BORING DATE July 13, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE Washboring, BX Rock Core & Cone Test CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; WATER CONTENT %	RQD %	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV	400 800 1200 1600 2000			% GR SA SI CL
269.3	Ground Level									
0.0 267.3	Road fill, gravel, sand rock boulders		1	CS						
2.0	Silty clay		2	SS	18					
			3	TW	PM					
	with some fine		4	SS	15					
	sand seams		5	TW	PM	260				
	Very stiff		6	SS	9					
255.2	Bouldery material		7	SS	50/2"		+ 2240			
14.1	Crystalline Limestone Bedrock		8	BX RC	Rec 100 %				98	
250.6	Sound						Bouncing			
18.7	End of Borehole									
	Note: Water level not established									

OFFICE REPORT ON SOIL EXPLORATION

20
15 ⟡ 5 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 106

WP 198-62-00 LOCATION Co-ords. N 16,505,577; E 1,039,272 ORIGINATED BY VK

DIST 9 HWY 17N Line 'C' EBL BORING DATE December 11, 1974 COMPILED BY SO

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger & BXL Rock Core CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —w_L PLASTIC LIMIT —w_P WATER CONTENT —w / WATER CONTENT % 20 40 60		REMARKS
ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				% GR SA SI CL
272.2	Ground Level									
	Topsoil									
0.0	Clayey silt, trace to some sand Brown Stiff		1	SS	10	270		o		0 6 76 18
268.2 / 4.0	Crystalline Limestone Bedrock Sound Grey									
263.0 / 9.2	End of Borehole									

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 5 SHEET NO.

PROPERTY: W. P. 198-62-00
LOCATION: Madawaska River Bridge
Arnprior By-Pass Hwy. 17
LATITUDE:
DEPARTURE:
BEARING:

DIP: 90°
NX Core
TOTAL FOOTAGE: 11'7"

ELEV. COLLAR: 283.8
DATUM: Geodetic
DATE STARTED:
DATE COMPLETED:
DRILLED BY:
LOGGED BY:

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
2'7"	11'7"	Crystalline limestone, coarse texture, light grey colour, hard, bedding generally at 55° inclination and sometimes contorted.				joint fracture 30° at 6'5" open and weathered
						joint fracture at 7'10" open and weathered.
		RQD 2'7" - 11'7" 100%				Horizontal fracture breakage appears fresh.

DATE OF EXAMINATION: April 28, 1977

D.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 6 SHEET NO.

PROPERTY W.P. 198-62-00
LOCATION Madawaska River Bridge
Arnprior By-Pass Hwy. 17
LATITUDE
DEPARTURE
BEARING

DIP
90°
NX Core
TOTAL FOOTAGE 13'4"

ELEV. COLLAR 286.1
DATUM Geodetic
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
0'0"	13'4"	Crystalline limestone, coarse texture, light grey colour, hard, bedding inclination at 50° sometimes contorted.				Broken core, 6'8" - 7'6" weathered, possibly a vertical or inclinet joint fracture area
						Joint fracture 50° along bedding plane, open and weathered.
		RQD 0 - 5' 95%				
		RQD 5' - 8'4" 75%				Joint fracture 30° and at 90° to bedding planes at 5'11".
		RQD 8'4" - 13'4" 95%				
						Joint fracture 50° and parallel to bedding at 8'6" weathered, also at 9'5" and 10'1"
						Horizontal fracture breakage appears fresh

DATE OF EXAMINATION April 28, 1977

B.K. Glassford

08-MY-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 7 SHEET NO.

PROPERTY	W. P. 198-62-00	DIP	90°	ELEV. COLLAR	273.4
LOCATION	Madawaska River Bridge Arnprior By-Pass Hwy. 17		NX Core	DATUM	Geodetic
				DATE STARTED	
				DATE COMPLETED	
LATITUDE				DRILLED BY	
DEPARTURE				LOGGED BY	
BEARING		TOTAL FOOTAGE	19'5"		

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
1'8"	19'5"	Crystalline limestone, coarse texture, light grey colour, hard, bedding inclination 40°, sometimes contorted.				Broken core 1'8" - 2'8" " " 4'6" - 9'6" partly Bedding plane fractures 1'8" - 2'8" weathered. Vertical joint fracture at 4'6" - 9'6" parallel to ℄ to Hwy. open and weathered. RQD - 1'8" - 4'6" 50% RQD - 4'6" - 9'6" 50% RQD-9'6" -19'5" 95% Horizontal fracture breakage appears fresh.

DATE OF EXAMINATION April 28, 1977

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 7A SHEET NO.

DIP 90°

PROPERTY W. P. 198-62-00
LOCATION Madawaska River Bridge
Arnprior By-Pass Hwy. 17
LATITUDE
DEPARTURE
BEARING

TOTAL FOOTAGE 42'11"

ELEV. COLLAR 272'. 0
DATUM Geodetic
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
4"	1'2"	NX Casing Core				Broken core 3'4" - 4'0"
						" " 19'9" - 26'6"
1'2"	42'11"	Crystalline limestone, coarse texture, hard,				and at 32' - 39'8" broken
		light grey colour, bedding inclination 40°, sometime				horizontally into 3" sections.
		contorted.				
						Joint fracture 80° at 2'0"
						Joint fractures at 11'2" and
		RQD 1'2" - 42'11" 90 . - 95%				26'2" at 90° to bedding planes.
						Joint fracture 80° at 18'0".
						Stripe of bedding is parallel to ℄.
						Horizontal fracture breakage
						appears fresh.

DATE OF EXAMINATION April 28, 1977 B. K. Glassford

08-MT-113

Ministry of Transportation and Communications

Ontario

DIAMOND DRILL RECORD

HOLE NO. 8A SHEET NO.

DIP

90°

NX Core

TOTAL FOOTAGE 25'4"

PROPERTY W. P. 198-62-00

LOCATION Madawaska River Bridge
Arnprior By-Pass Hwy. 17

LATITUDE

DEPARTURE

BEARING

ELEV. COLLAR 276.00

DATUM Geodetic

DATE STARTED

DATE COMPLETED

DRILLED BY

LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
1'4"	25'4"	Crystalline limestone, coarse texture, light grey colour, hard, bedding inclination 45°, sometimes contorted.				Broken core 1'4" - 4'4" " " 7'5" - 9'7" " " 18'
						Joint fracture 80° 1'4" -4'4" open and weathered.
		RQD 1'4" - 4'4" 0%				Joint fracture 90° and inclined at 7'5" - 9'7" open and weathered with cavities.
		RQD 1'4" - 18'0" 50%				
		RQD 18'0" - 25'4" 95%				Fractures parallel to bedding planes at 15'10", 15'4" and 10'6".
						Joint fracture parallel to bedding inclined at 85° open and weathered with cavities.
						Two fracture joints at 90° to each other and each at 90° to bedding planes.
						Bedding plane fracture 45° at 21'7"
						Joint fracture 80° at 23'2"
						Horizontal fracture breakage appears fresh.

DATE OF EXAMINATION April 28, 1977

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications

Ontario

DIAMOND DRILL RECORD

HOLE NO. 9 SHEET NO.

PROPERTY W. P. 198-62-00

LOCATION Madawaska River Bridge

Arnprior By-Pass Hwy. 17

LATITUDE

DEPARTURE

BEARING

DIP 90°

NX Core

TOTAL FOOTAGE 42'1"

ELEV. COLLAR 272.3

DATUM Geodetic

DATE STARTED

DATE COMPLETED

DRILLED BY

LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
2'1"	42'1"	Crystalline limestone, coarse texture, light grey colour, hard, bedding inclination 45°, sometimes contorted.				Joint fracture 85° at 16' -19'3" parallel to ℄, open and weathered.
						Joint fracture, 80° at 20'5"
						" " 50° at 38'3"
						" " 45° at 40'0"
						Bedding plane fractures at 27'6"
						32'0"
						35'8"
						Broken Core at 38'3"
						" " " 40'0"
						Horizontal fractures appear fresh
						RQD - 2'1" - 16' 100%
						" 16' - 19' 0%
						" 19' -42' 95%

DATE OF EXAMINATION April 28, 1977

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 10 SHEET NO.

DIP 90°

PROPERTY W.P. 198-62-00
LOCATION Madawaska River Bridge
Arnprior By-Pass Hwy. 17
LATITUDE
DEPARTURE
BEARING

NX Core

TOTAL FOOTAGE 21'6"

ELEV. COLLAR 272.7
DATUM Geodetic
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
1'4"	21'6"	Crystalline limestone, coarse texture, light grey colour, hard, bedding inclination 45°, sometimes contorted.				Broken core at 5'8" - 6'11" " " at 8'9" - 10'8"
						Bedding plane fractures at 3'3", 4'7", 5'8", 11'9", 12'2", 13'4", 17'0"
		RQD 1'4" - 6'5" 70% RQD 6'5" - 11'5" 60% RQD 11'5" - 21'6" 100%				
						Horizontal fracture breakage appears fresh. Inclined fracture 85° at 2' -3'3" open and weathered at 2'9".
						Inclined fracture 85° at 5'6" -6'8" oriented 125° to ℄, open and weathered.
						Vertical fracture 11'3" - 11'9" open and weathered.
						Fractures open and weathered 8'10" 10'8" at 90° to bedding.

DATE OF EXAMINATION April 28, 1977

B.K. Glassford

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 11A SHEET NO.

PROPERTY W.P. 198-62-00
LOCATION Madawaska River Bridge
Arnprior By-Pass Hwy. 17
LATITUDE
DEPARTURE
BEARING

DIP 90^0
NX Core
TOTAL FOOTAGE 24'9"

ELEV. COLLAR 261.50'
DATUM Geodetic
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
4'6"	24'9"	Crystalling limestone, coarse texture, light grey colour, hard, bedding 40^0 inclination and contorted in some areas.				Broken and ground core at vertical joint fracture at 5'0"
						Broken core 4'10" - 5'3"
						Broken core 13'6" - 19'8"
5'3"	7'6"	(no core log submitted)				
						Vertical joint fracture indicated at 13'6" - 14'6"
7'6"	8'6"	foreign particles of rock of igneous and metamorphic origin				16'8" - 18'0"
						RQD 4'6" - 5'3" 50%
						RQD 7'6" - 8'6" 0%
						RQD 8'6" - 10'6" 50%
						RQD 10'6" - 19'8" 60%
						RQD 19'8" - 24'9" 100%
						Horizontal fracture breakage appears fresh.

DATE OF EXAMINATION April 28, 1977 B.K. Glassford

08-MT-113

Ministry of Transportation and Communications

Ontario

DIAMOND DRILL RECORD

HOLE NO. 12A SHEET NO. ______

PROPERTY: W.P. 198-62-00

LOCATION: Madawaska River Bridge
Arnprior By-Pass Hwy. 17

LATITUDE: ______

DEPARTURE: ______

BEARING: ______

DIP: 90°

NX Core

TOTAL FOOTAGE: 40'2"

ELEV. COLLAR: 257.40'

DATUM: Geodetic

DATE STARTED: ______

DATE COMPLETED: ______

DRILLED BY: ______

LOGGED BY: ______

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
8'6"	40'2"	Crystalline limestone, coarse texture, light grey colour,				Broken core 8" - 12"
		hard, bedding 45° inclination and contorted in some areas.				7'6" - 7'10"
						11'2" - 11'5"
						Joint fractures 90° 8' - 10'0"
						weathered 90° 17" - 18'6"
						Fractures 45° at 3'8"
						" " " 4'5"
						" " " 13'6"
						probably bedding plane joints
						Horizontal fracture, breakage,
						appears fresh.
						RQD 8" - 13'0" 70%

DATE OF EXAMINATION: April 28, 1977

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 15 SHEET NO.

DIP 90°

PROPERTY W. P. 198-62-00
LOCATION Madawaska River Bridge
Arnprior By-Pass Hwy. 17

NX Core

LATITUDE
DEPARTURE
BEARING

TOTAL FOOTAGE 14'8"

ELEV. COLLAR 279.21'
DATUM Geodetic
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
1'6"	14'8"	Crystalline limestone, coarse texture, light grey colour, bedding 45% inclination and contorted in some areas.				Broken core 4'5" - 5'0" " " 7'5" - 8'0" Calcite seam 4'3" - 5'7" " " 7'4" - 8'2" Calcite seams are soft, weathered and stained. Horizontal fracture breakage appears fresh. RQD 1'6" - 6'7" 75% RQD 6'7" - 9'8" 50% RQD 9'8" - 14'8" 100%

DATE OF EXAMINATION April 27, 1977

B.K. Glassford

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 16 SHEET NO.

PROPERTY W. P. 198-62-00
LOCATION Madawaska River Bridge, Arnprior By-Pass Hwy. 17
LATITUDE
DEPARTURE
BEARING

DIP 90°

TOTAL FOOTAGE 18'4"

ELEV. COLLAR 273.11
DATUM Geodetic
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
7'7"	18'4"	Crystalline limestone, coarse texture, light grey colour, hard, bedding 45% inclination and contorted in areas.				Broken core 16'10" - 17'10"
						Joint fracture 80° at 17.6" - 17'10"
						Micaeous content in limestone at 15'6" - 17'4".
						Horizontal fracture breakage appears fresh.
						RQD 0' - 13'3" 100%
						RQD 13'3" - 18'4" 70%

DATE OF EXAMINATION April 28, 1977 B.K. Glassford

08-MT-113

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Ministry of Transportation and Communications
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DIAMOND DRILL RECORD

HOLE NO. 51 SHEET NO.

DIP: 90°

PROPERTY: W. P. 198-62-00
LOCATION: Madawaska River and Hwy. 17N

LATITUDE:
DEPARTURE:
BEARING:

TOTAL FOOTAGE: 29.15'

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

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
8.9'	29.15'	Crystalline limestone, light grey, medium to coarse textured, hard. Liniation 40-45°, slight weathering and staining on some of the joints. Joints appear to be open				Widely spaced 45° joints at 9.0', 9.7', 10.7', 11.2' which are open.
						Vertical joint 13.5' to 14.8' open
						Vertical joint 19'
						Core moderately to badly horizontally fractured throughout
		All horizontal fracturing appears to have been caused by the drilling procedures				

DATE OF EXAMINATION: Aug. 13/76

B. K. Glassford

08-MT-113

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Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 52 SHEET NO. ______

DIP: 90°

TOTAL FOOTAGE 24.8'

PROPERTY W.P. 198-62-00
LOCATION Madawaska River and Hwy. 17N
LATITUDE
DEPARTURE
BEARING

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

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
8.55'	24.8'	Crystalline limestone, white to light grey, medium to coarse textured, hard, liniation 40° - 60°, slight weathering on some of the joints which are open				Joints 40° at 13.5' 40° at 14.8' open
						8.55' - 8.8' core ground 8.8' - 24.8' core badly to moderately fractured
		All horizontal fracturing appears to have been caused by the drilling procedures				

DATE OF EXAMINATION Aug. 13/76

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 53 SHEET NO.

DIP: 90°

PROPERTY W. P. 198-62-00
LOCATION Madawaska River and Hwy. 17N
LATITUDE
DEPARTURE
BEARING

TOTAL FOOTAGE 41.55'

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

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
21.95'	41.55'	Crystalline limestone, light grey colour, medium to coarse texture, hard, liniation varies 45° to 75° slight weathering on some of the joints				21.95' - 27.9' core is fractured moderately to badly
						Joints 45° at 24.0' 45° at 24.5' 45° at 25.6' 45° at 28.0' 45° at 34.5' 45° at 36.0'
		All horizontal fracturing appears to have been caused by the drilling procedures				27.9' - 36.0' core is badly fractured
						36.0' - 37.5' broken and ground core
						37.5' - 41.55' core is moderately fractured

DATE OF EXAMINATION Aug. 13/76

B.K. Glassford

06-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 54 SHEET NO.

DIP

PROPERTY W.P. 198-62-00
LOCATION Arnprior Bypass, Hwy. 17N
LATITUDE
DEPARTURE
BEARING

90°

TOTAL FOOTAGE

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

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
		Hole #54				
13.6'	14.15'	Limestone boulder				
						Joints
14.15'	18.6'	Crystalline limestone, white grey colour, medium to				45° at 15.0' open
		coarse texture, hard.				70° at 15.4' open
						moderately fractured throughout

DATE OF EXAMINATION August 13, 1976

B.K. Glassford

08-MT-113

FOUNDATION INVESTIGATION REPORT

For

CPR Overhead
W.P. 199-62-00, Site 29-200
Hwy. 17N, District 9, Ottawa

INTRODUCTION

This report contains the results of our investigation of the sub-surface conditions existing at this site. The fieldwork was carried out in two stages. The early investigation was done during the period of June 28, 1976 to July 7, 1976, consisting of six sampled boreholes, three of which were accompanied by a dynamic cone penetration test. In addition, two separate dynamic cone penetration tests were also carried out. The subsequent investigation was done on April 6, 1977 consisting of eighteen probe holes. The boreholes were advanced by means of a skid mounted diamond drill rig to depths ranging from 13 feet to 36 feet below ground surface. Bedrock was proven by obtaining NX and BX size rock core samples.

SITE AND GEOLOGY

The site is located near the Town of Arnprior on Lot 1, Concession B, Township of McNab, County of Renfrew.

The relocated CPR tracks are situated in a cut section some 40 feet in depth. The bottom 10 to 15 feet is within the bedrock with almost vertical faces. In the overburden the earth cut slopes of the upper portions are as flat as 4:1 with a mid-height berm of about 16 feet followed by a lower slope of 3.5:1. In addition, a berm of a minimum width of 5 feet is maintained between the toe of the earth cut and the top of the rock face.

Geologically the area is located between the Precambrian Upland to the north and west and the Ottawa Lowland to the south and east. Bedrock consists of limestone with interbeds of sandstone and shale and has been subjected to faulting, weathering and erosion. When the Champlain Sea inundated the Ottawa-St. Lawrence Lowland, it left deposits of marine clay.

SUBSURFACE CONDITIONS

General

Underneath a thin mantle of topsoil (8 to 12 inches) is a deposit up to 26 feet thick of stiff to very stiff clayey silt to silty clay. Underlying the cohesive stratum is limestone bedrock. In certain locations a compact to dense deposit up to about 2.5 feet of silty sand and gravel is sandwiched between the cohesive stratum and bedrock.

As previously mentioned, the bedrock is exposed in the cut section and in some cases extending to a distance of 30 feet back from the face of the cut near the tracks.

Boundaries between different deposits are shown on the Record of Borehole Sheets which are appended to this report. The locations and elevations of the borings are shown on Drawing 29-200-2, together with the estimated stratigraphical sections. A description of the soil types encountered in the borings is as follows.

Clayey Silt to Silty Clay

Beneath a thin mantle of topsoil is the predominant deposit of clayey silt to silty clay with some seams or thin layers up to 3 inches or pockets of silt and fine sand. This deposit is up to 26 feet in thickness and is sensitive, brittle and fissured. According to the geology of the area, this cohesive material was deposited by the postglacial Champlain Sea.

Typical engineering properties of the cohesive subsoil are plotted on the Record of Borehole Sheets and summarized in tabular form below.

Index Properties		Range	Average
Natural Moisture Content	W (%)	28- 44	38
Liquid Limit	W_L (%)	25- 42	34
Plastic Limit	W_p (%)	14- 22	18
Liquidity Index	I_L	1.0-1.8	1.2
Bulk Unit Weight	γ (pcf)	111-117	113

Undrained Shear Strength (c_u) psf

In Situ Vane Tests	1200 - > 2240
Unconfined Compression Tests	1070 - 1795
Quick Triaxial Tests	2335 - 2380

Effective Stress Shear Strength Parameters

Apparent Cohesion	c'=350 psf
Apparent Angle of Internal Friction	ø'=25½°

The Atterberg Limit Test results are also summarized on the Plasticity Chart, Figure 1. The testing indicates that the cohesive stratum is inorganic and of low to intermediate plasticity.

The natural moisture content is above the liquid limit as indicated by the liquidity indices which are greater than unity. This is usually typical of the sensitive marine clay.

From the undrained shear strength values it is estimated that the consistency of the cohesive stratum varies from stiff to very stiff. However, at the western portion of the cut adjacent to the exposed surface of the bedrock, the dynamic cone penetration tests gave 'N' values ranging from 1 to 8 blows/foot and from this it is estimated that the consistency of the cohesive overburden in this area varies from firm to stiff.

The results of isotropically consolidated undrained triaxial compression tests which were performed to determine the effective stress shear strength parameters, are plotted on Figure 2.

For design purposes the following values are to be used:

Bulk Density	= 115 pcf
Submerged Density	= 53 pcf
Undrained Shear Strength	c_u = 800 psf
Apparent Cohesion in Terms of Effective Stresses	c' = 250 psf
Apparent Angle of Internal Friction in Terms of Effective Stresses	ø' = 22°

Silty Sand and Gravel

In certain locations a thin granular layer up to 2.5 feet in thickness consisting of silty sand and gravel underlies the cohesive deposit. Standard penetration testing gave 'N' values of 20 blows/ foot which indicates that the relative density of this stratum is generally compact.

Bedrock

The CPR tracks are located in a rock cut 10 to 15 feet deep. Bedrock is exposed along the top of the rock cut for a distance of up to 30 feet from the face of the cut. The surface of the bedrock varies between elevations 303 and 323 which indicates that the depth to bedrock varies from the existing ground surface to 26 feet below ground surface.

The bedrock can be identified as hard to medium hard crystalline limestone. It is generally sound, except in one location near the north corner of the western portion of the site (B.H. 4), where the upper six feet of the rock is weathered. The exposed bedrock in the vicinity of the CPR tracks contains vertical, as well as inclined jointing systems and the majority of them are facing towards the river.

R.Q.D. values of the overall rock core samples ranged from 50% to 100% indicating a fair to excellent rock quality. However, in one location near the north corner of the western portion of the site the R.Q.D. recorded in the upper 6 feet of the bedrock is very low being close to 0%.

A detailed description of the rock cores is given in the Diamond Drill Records which are included in the Appendix of this report.

Groundwater Conditions

During the course of our field investigation all boreholes were found to be generally dry. According to available information, groundwater in this area has been lowered below the track elevation by means of a permanent subsurface drainage system consisting of pumped wells.

B. Ly

B. Ly, P. Eng.
Senior Engineer

M. Devata, P. Eng.
Supervising Engineer

APPENDIX

RECORD OF BOREHOLE Nº 1

W.P. 199-62-00 LOCATION Co-ords. 16,505,500 N; 1,040,128 E. ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE June 28, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE NX ROCK CORING CHECKED BY

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
323.2	Ground Level									
0.0	Crystalline Limestone Bedrock Sound		1	RC NX	Rec. 67%					RQD=80%
			2	RC NX	Rec. 100%	320				RQD=63%
			3	RC NX	Rec. 100%					RQD=87%
			4	RC NX	Rec. 100%					RQD=55%
310.1 / 13.1	End of Borehole									

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE № 2

W P 199-62-00 LOCATION Co-ords. 16,505,490 N; 1,040,226 E. ORIGINATED BY SM
DIST 9 HWY 17N BORING DATE June 28, 1976 COMPILED BY SM
DATUM Geodetic BOREHOLE TYPE Washboring, BX Rock Coring, Cone Test CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	UNIT WEIGHT γ PCF	REMARKS
335.8	Ground Level						
0.0	Topsoil	1	SS	2			
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle fissured.	2	TW	PM		113	
					330		
		3	TW	PM			
		4	SS	6			
		5	TW	PM			
321.8	Stiff to Very Stiff	6	SS	6			
321.3 / 14.5	Silty sand, some grav. Compact						
	Crystalline Limestone Bedrock	7	RC BX	Rec. 95%	320		RQD=63%
312.2	Sound	8	RC BX	Rec. 100%			RQD=100%
23.6	End of Borehole Borehole Dry						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH PSF: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 400 800 1200 1600 2000
LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %: 20 40 60
REMARKS % GR SA SI CL

20
15 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE Nº 3

WP 199-62-00 LOCATION Co-ords. 16,505,450 N; 1,040,254 E. ORIGINATED BY SM

DIST 9 HWY 17N BORING DATE June 29 & 30, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE Washboring and BX Rock Coring CHECKED BY

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH PSF: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE; 400 800 1200 1600 2000

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %: 20 40 60

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
336.3	Ground Level							
	Topsoil		1	SS	3			
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured.		2	TW	PM		116	
			3	SS	4	330		
			4	TW	PM			
			5	SS	9			
	Stiff to Very Stiff		6	SS	11			
320.3	some gravel		7	SS	54			
16.0	Silty sand & gravel					320		
318.0	Compact		8	SS	20			
18.3	Crystalline Limestone Bedrock		9	RC BX	Rec. 98%			RQD=84%
312.9	Sound							
23.4	End of Borehole Borehole Dry							

20
15 ◇ 5 % STRAIN AT FAILURE
10

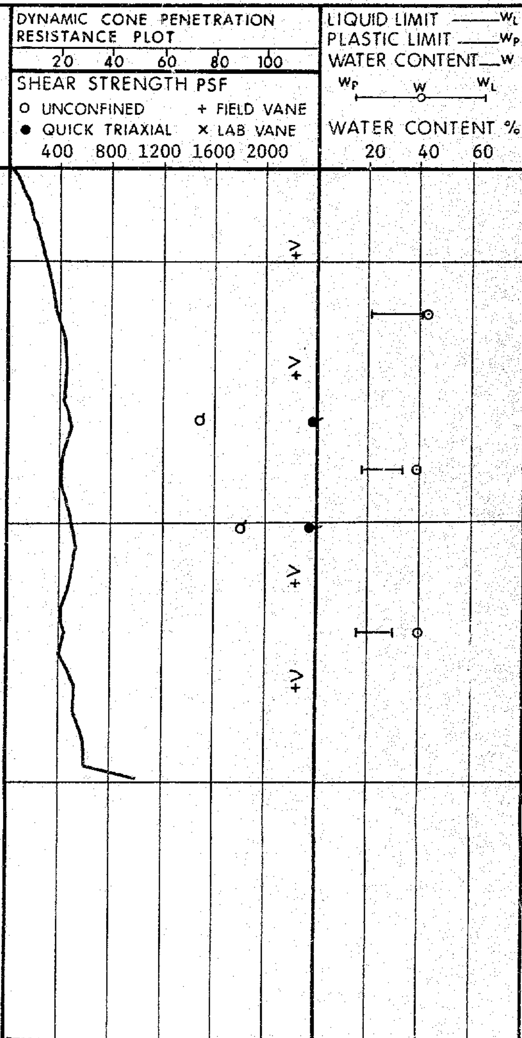
RECORD OF BOREHOLE Nº 4

WP 199-62-00 LOCATION Co-ords. 16,505,517 N; 1,039,902 E. ORIGINATED BY SM

DIST 9 HWY 17N BORING DATE July 1, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE Washboring, BX Rock Coring, Cone Test CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
333.6	Ground Level						
0.0	Topsoil	1	SS	5			
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured.				330		
		2	TW	PM		111	
		3	TW	PM			
		4	SS	7			
		5	TW	PM	320		
	Stiff to Very Stiff	6	TW	PM		111	
311.8		7	SS	4			
21.8	Silty sand & gravel						
309.8	Compact	8	SS	20	310		
23.8	Boulder						
24.3	Crystalline Limestone Bedrock	9	RC BX	40% Rec.			RQD=0%
	weathered / sound						
		10	RC BX	100% Rec.			RQD=80%
299.2					300		
34.4	End of Borehole						
	Borehole Dry						



20 / 15 ◊ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

OFFICE REPORT ON SOIL EXPLORATION

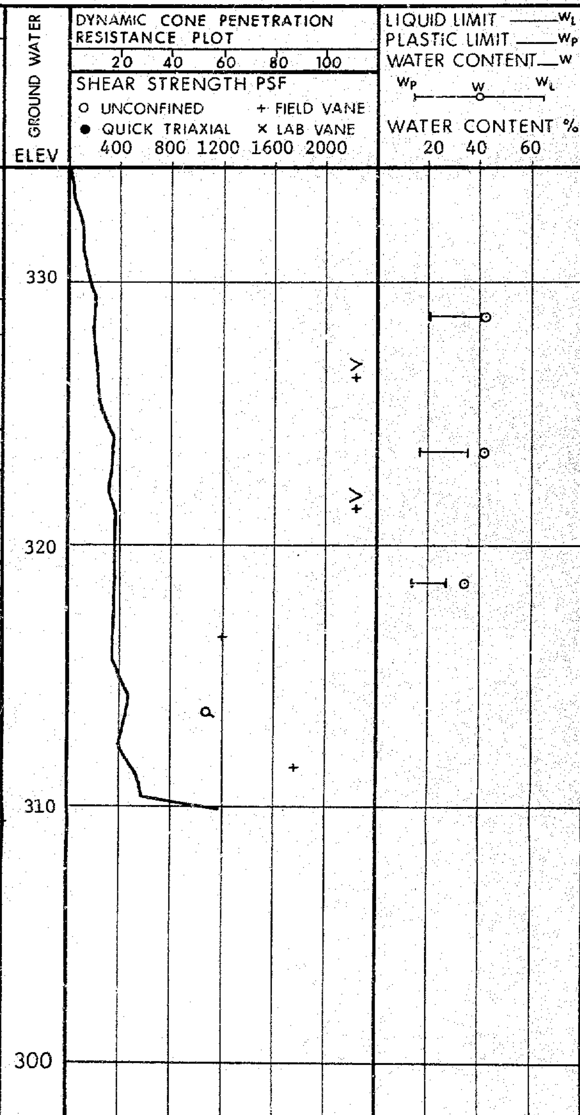
RECORD OF BOREHOLE Nº 5

WP 199-62-00 LOCATION Co-ords. 16,505,478 N; 1,039,929 E. ORIGINATED BY SM

DIST 9 HWY 17N BORING DATE July 5, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE Washboring, BX Rock Coring, Cone Test CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ PCF	REMARKS
334.4	Ground Level					
0.0	Topsoil					
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand – sensitive, brittle, fissured.	1	TW	PM	112	
		2	TW	PM	112	
	Stiff to Very Stiff				117	
		3	TW	PM		
		4	TW	PM		
308.9		5	SS	7		
25.5	Crystalline Limestone Bedrock	6	RC BX	82% Rec.		RQD=50%
	Sound	7	RC BX	91% Rec.		RQD=55%
297.6						
36.8	End of Borehole Borehole Dry					



20
15 –5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

RECORD OF BOREHOLE Nº. 6

WP 199-62-00 | LOCATION Co-ords. 16,505,470 N: 1,040,029 E. | ORIGINATED BY SM

DIST 9 HWY 17N | BORING DATE July 6, 1976 | COMPILED BY SM

DATUM Geodetic | BOREHOLE TYPE Washboring and NX Rock Coring | CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
318.8	Ground Level									
0.0	Topsoil									
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured.									
						310				
	Firm to Stiff									
305.2										
13.6	Crystalline Limestone Bedrock Sound		1	RC NX	96% Rec.					RQD=87%
						300				
	Gneiss Bedrock		2	RC NX	97% Rec.					RQD=80%
294.8										
24.0	End of Borehole									
	Borehole Dry									

20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE № 7

WP 199-62-00 LOCATION Co-ords. 16,505,490 N; 1,040,016 E. ORIGINATED BY SM

DIST 9 HWY 17N BORING DATE July 7, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE Cone Test CHECKED BY [illegible]

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; W_P — W — W_L; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
317.2	Ground Level									
0.0	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - senstive, brittle, fissured. Firm to Stiff					310				
303.0 14.2	End of Cone Probable Bedrock									

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE Nº 8

WP 199-62-00 LOCATION Co-ords. 16,505,508 N; 1,040,003 E. ORIGINATED BY SM

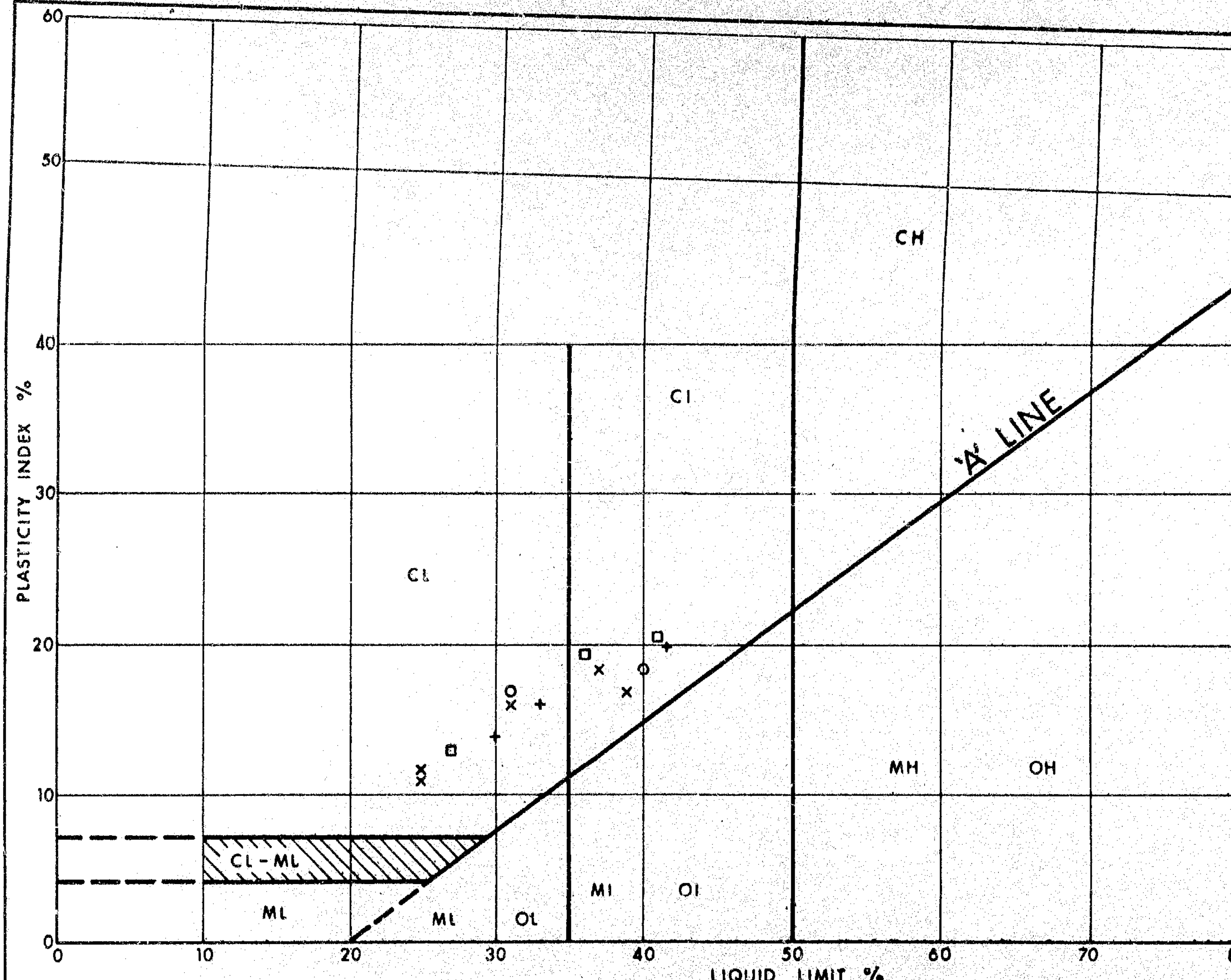
DIST 9 HWY 17N BORING DATE July 7, 1976 COMPILED BY SM

DATUM Geodetic BOREHOLE TYPE Cone Test CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES					
315.5	Ground Level									
0.0	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured. Firm to Stiff					310				
304.5 / 11.0	End of Cone Probable Bedrock									

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

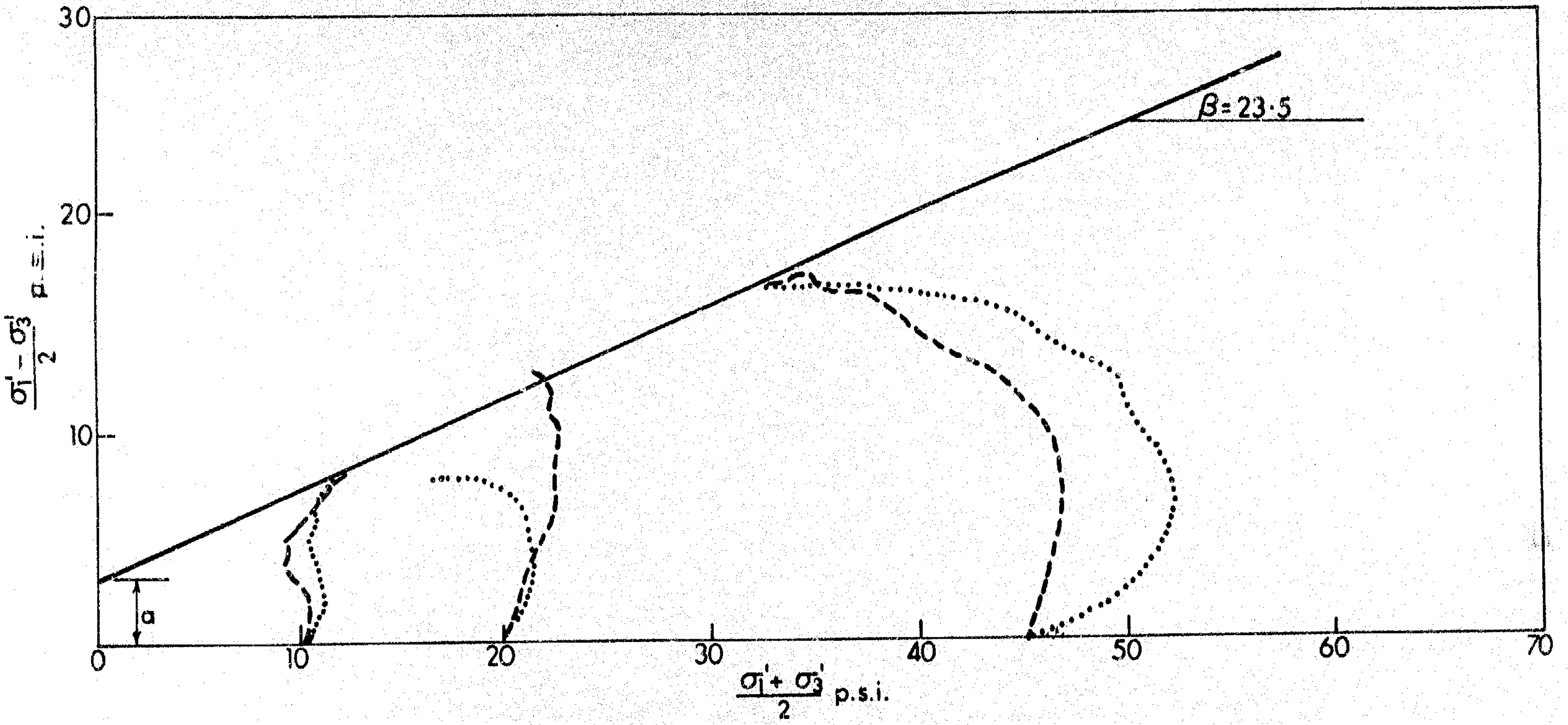


LEGEND		
BH	SAMPLE	SYMBOL
2	2, 4	o
3	1,3,4,5,6	×
4	2, 4, 6	+
5	1, 2,3	□

Ministry of Transportation and Communications
Ontario

PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY

FIG No 1
W P 199-62-00



LEGEND		
SYMBOL	B. H.	SA.
— — — —	4	6
··········	5	3

SUMMARY OF C.I.U. TESTS

SILTY CLAY TO CLAYEY SILT

FIG. 2

W.P. 199-62-00

Ministry of Transportation and Communications

Ontario

DIAMOND DRILL RECORD

HOLE NO. 1 SHEET NO.

PROPERTY W. P199-62-00

LOCATION Arnprior By-pass and C. P. R. X-ing
Station 802+80 18' Lt. of C_L

LATITUDE

DEPARTURE

BEARING

DIP

90°

TOTAL FOOTAGE 13.1'

ELEV. COLLAR

DATUM

DATE STARTED

DATE COMPLETED

DRILLED BY

LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
						Joints
0.0'	13.1'	Crystalline limestone, light grey, medium to coarse textured, hard. Slight weathering on some of the joints and mica mineralization present.				75°, 2 joints parallel at 2.9' and 3.3' 60° joint at 5.5' to 5.J' open 85° joint at 8.2' to 10.9' open
						Core moderately fractured
		All horizontal fracturing appears to have been caused by the drilling procedures				

DATE OF EXAMINATION Aug. 13/76

B. K. Glassford

OB-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. ______ SHEET NO. ______

DIP

PROPERTY	W. P. 199-62-00	90°		ELEV. COLLAR	
LOCATION	Arnprior By-pass Hwy. 17			DATUM	
	C. P. R. Crossing			DATE STARTED	
	Station 803+00			DATE COMPLETED	
LATITUDE				DRILLED BY	
DEPARTURE				LOGGED BY	
BEARING		TOTAL FOOTAGE			

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
		Hole #2				Joints
14.5'	23.6'	Crystalline limestone, white grey colour, coarse texture,				50° at 15.5' open, core is
		bedding appears to be horizontal, hard to medium hard				moderately fractured throughout
		Hole #3				Joints
18.3'	23.4'	Crystalline limestone, white grey colour, coarse texture,				55° at . .0' open, 60° at 23.o' open
		bedding appears to be horizontal, hard to medium hard				core is moderately fractured throughout
		Hole #4				Joints
23.9'	34.4'	Limestone boulder 23.9'-24.3', crystalline limestone, white				0° at 31.0' open, 80° at 32.0' open
		grey colour, coarse texture, hard to medium hard, bedding				broken, ground core missing,
		appears to be horizontal, mica and sulphide mineralization				core 24'-30' core is badly frac-
		at 25.8'-31.0'				tured throughout this section
		Hole #5				Joints
25.8'	36.9'	Crystalline limestone, white grey colour, coarse texture,				90° at 25.8' open
		bedding appears horizontal, hard to medium hard, mica and				80° at 26.8' open
		sulphide mineralization at 25.8'-26.3', rock appears leached				45° at 28.0' open
		31.0'-36.0'				90° at 34.7' open
						90° at 35.8' open
		All horizontal fracturing appears to have been caused by the				Weathered zones at 25.8'-26.3'
		drilling procedures				31.0'-36.0'

DATE OF EXAMINATION Aug. 18/76

B. K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. 6 SHEET NO.

DIP: 90°

PROPERTY: W. P. 199-62-00
LOCATION: Arnprior By-pass and C. P. R. X-ing
Station 801+85 18' Rt. ℄

LATITUDE:
DEPARTURE:
BEARING:

TOTAL FOOTAGE: 24.0'

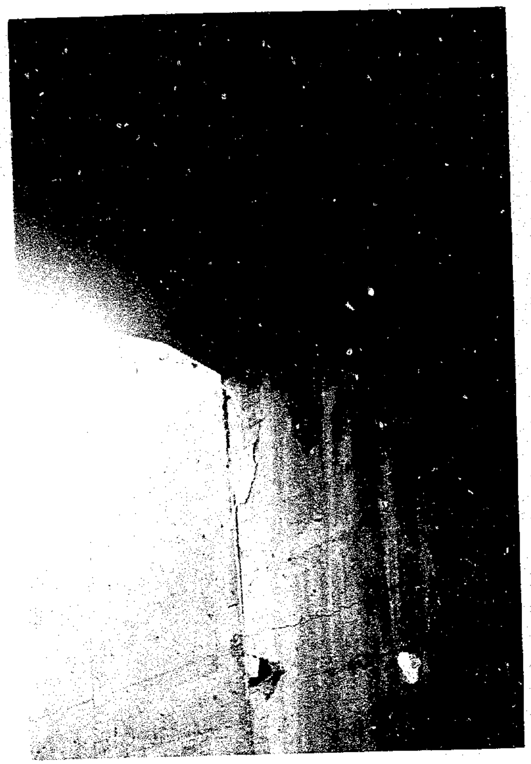
ELEV. COLLAR:
DATUM:
DATE STARTED:
DATE COMPLETED:
DRILLED BY:
LOGGED BY:

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
13.6'	18.6'	Crystalline limestone, light grey to grey, medium to coarse textured, hard				Joints
						45° - 14.85'
						30° - 16'
18.6'	20.25'	Gneiss, dark grey colour, fine textured, hard, mica rich				10° - 16.6'
						10° - 17.1'
20.25'	24.0'	Crystalline limestone, light grey to grey, medium to coarse textured, medium hard, all core mineralized (mica)				30° - 17.65'
						20° - 19.9'
						45° - 20.5'
						45° - 21.2'
		All horizontal fracturing appears to have been caused by the drilling procedures				17.2' - 17.5' weathered zone

DATE OF EXAMINATION: Aug. 13/76

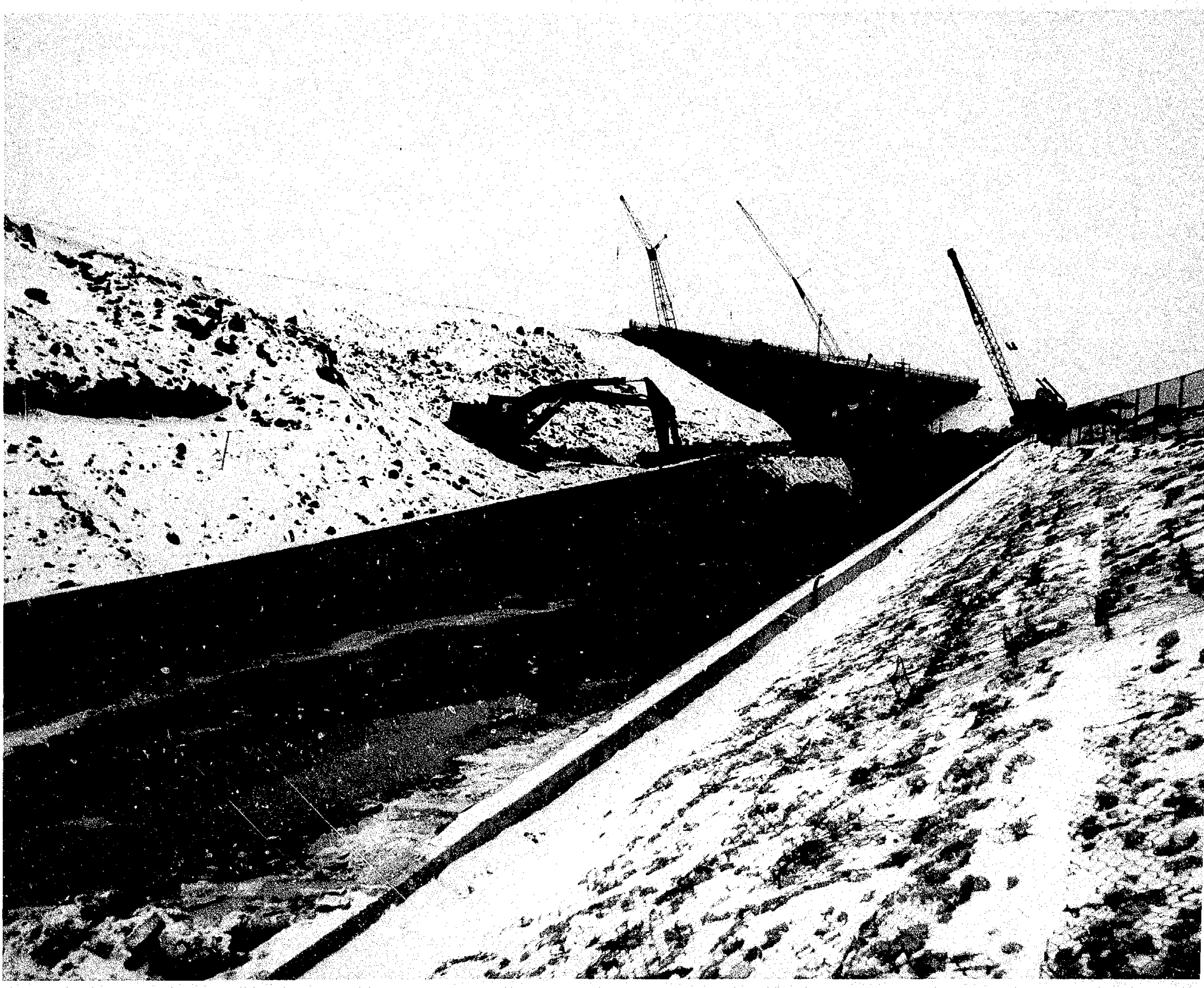
B. K. Glassford

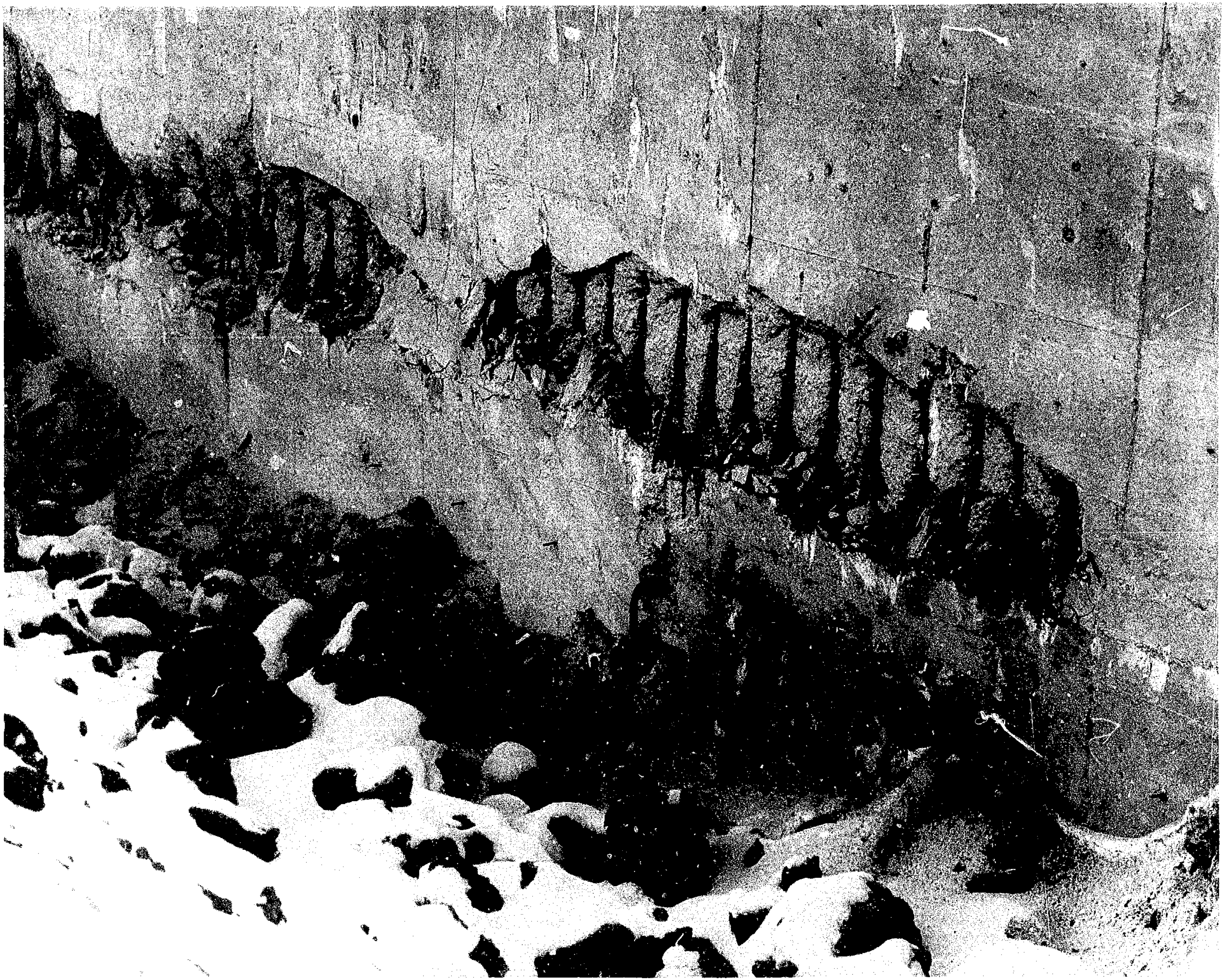
08-MT-113

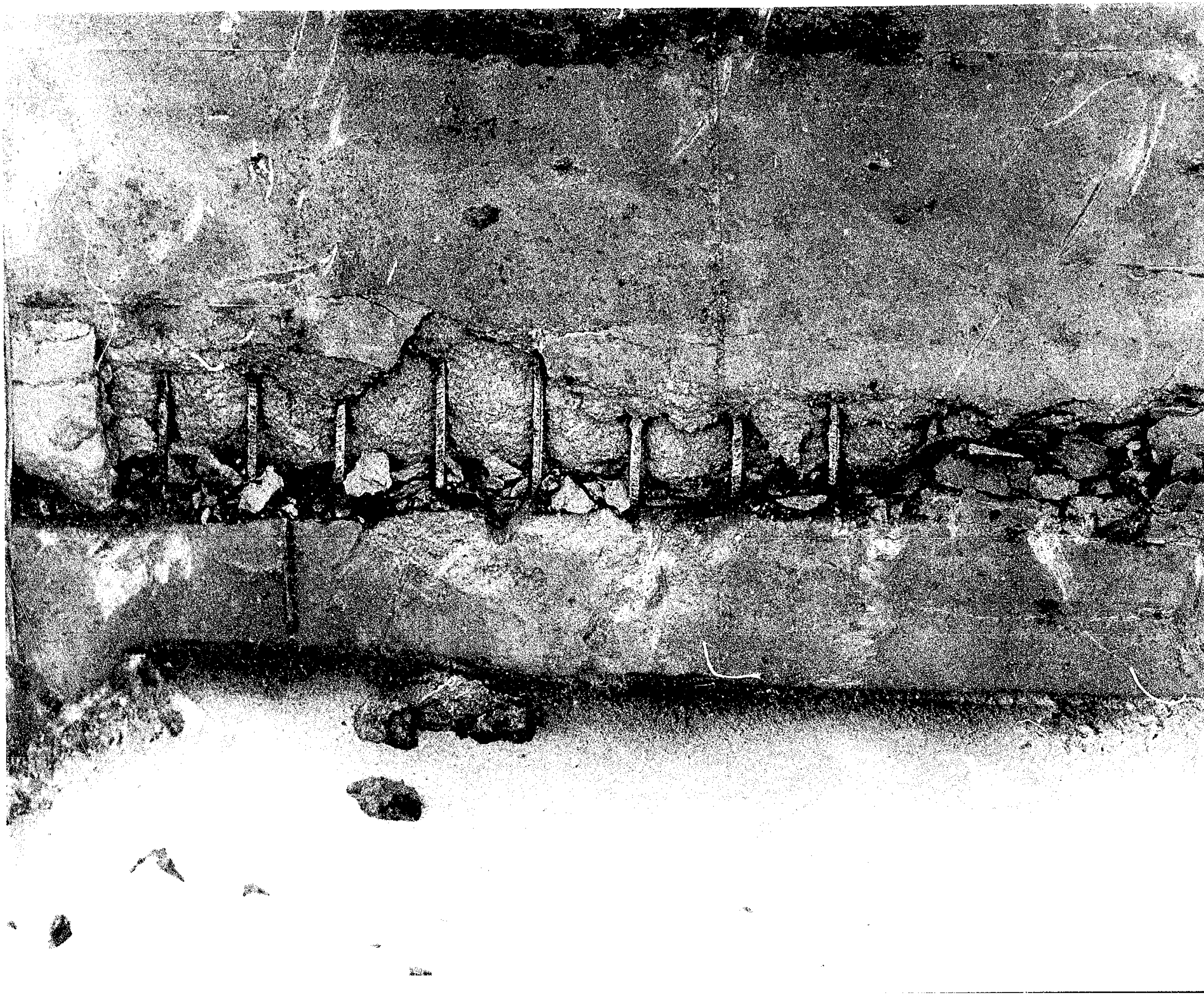


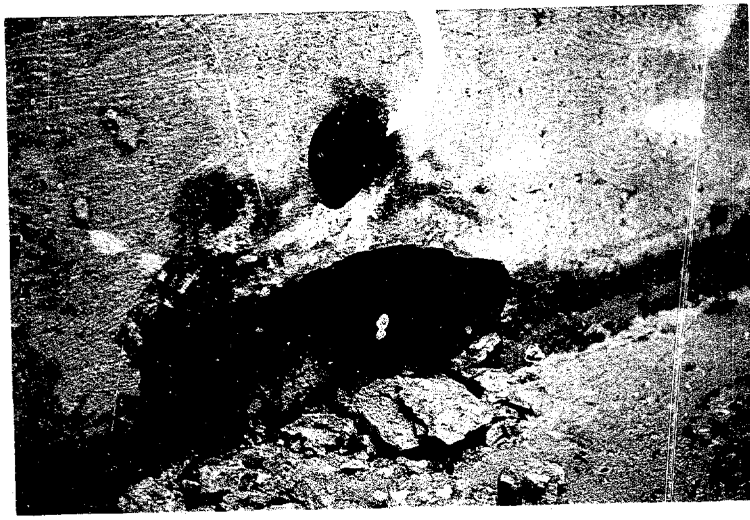
AT TOP OF CULVERT WALL

AT BOTTOM OF
RETAINING WALL

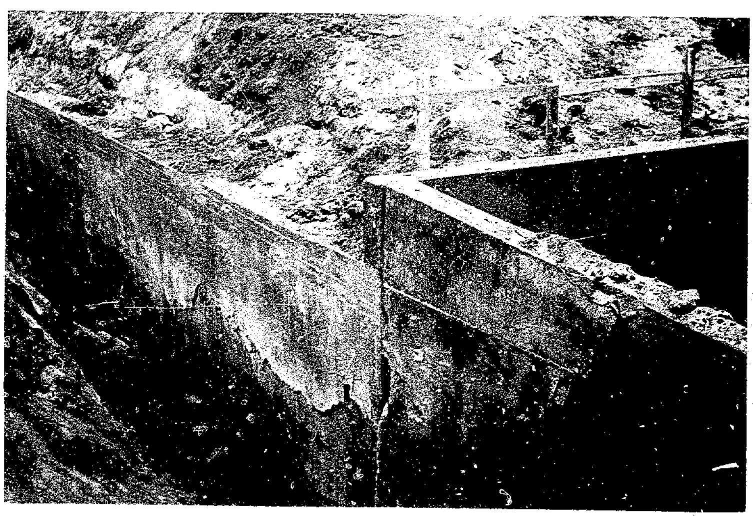








DELAMINATION OF CONCRETE AT BOTTOM OF RETAINING WALL, NEXT TO CHANNEL LINING



BACK FACE OF CULVERT AND RETAINING WALL

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M 12-57

DIST. 6 REGION CENTRAL

W.P. No. 48-71-08

CONT. No. 76-01

W. O. No. 72-11005

STR. SITE No. 24-311

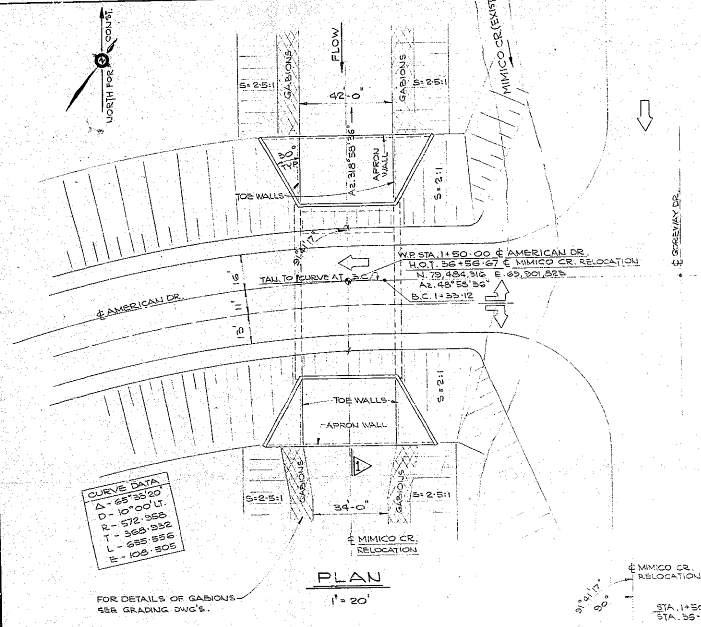
HWY. No. 427

LOCATION American Drive : Hwy 427

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT 2

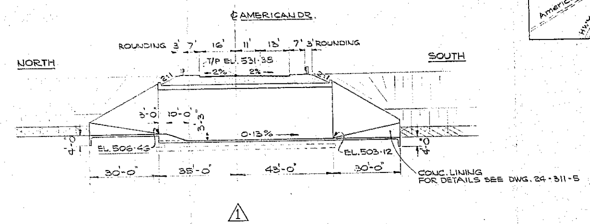
REMARKS: 2 documents to be unfolded before microfilming

G.I.-30 SEPT. 1976



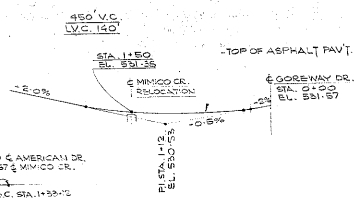
PLAN

1" = 20'



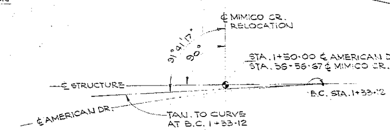
1" = 20'

NOTE: W.P. DENOTES WORKING POINT
T/P DENOTES TOP OF PAVEMENT



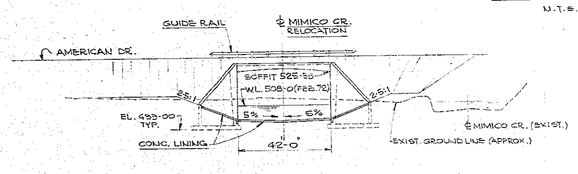
PROFILE OF AMERICAN DR.

N.T.S.



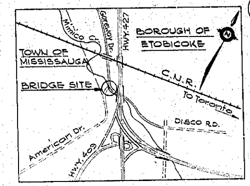
CONSTRUCTION ALIGNMENT

N.T.S.



ELEVATION

1" = 20'



KEY PLAN

2·50" = 1 MI

GENERAL NOTES

CLASS OF CONCRETE 4000 P.S.I.
CLEAR COVER FOR REINF. STEEL: (UNLESS OTHERWISE NOTED)
FOOTINGS, ABUTMENTS & RET. WALLS 3"
DECK (TOP & BOTT.) & CURBS 2"

BACKFILL: FILL TO BE PLACED AND COMPACTED SIMULTANEOUSLY FROM BOTH SIDES.

REFERENCE BENCH MARK
G.B.M. EL. 527·83
TABLET SET IN CONC. BASE OF LIGHT STANDARD AT ENTRANCE TO BOROUGH OF ETOBICOKE YARD 387·0' RT. OF STA. 390+28 PROP. HWY. 427

30m12-57
GEOCRES No.

LIST OF DRAWINGS

24-311-1 GENERAL LAYOUT
-2 BORE HOLE LOCATIONS & SOIL STRATA
-3 FOOTINGS & RET. WALLS
-4 FRAME DETAILS & REINFORCEMENT
-5 CONCRETE LINING, APRON & TOE WALL
-6 STANDARD DETAILS

PRINT RECORD

No.	FOR	DATE

REVISIONS

DATE	BY	DESCRIPTION

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
2 INCHES ON ORIGINAL PLAN

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

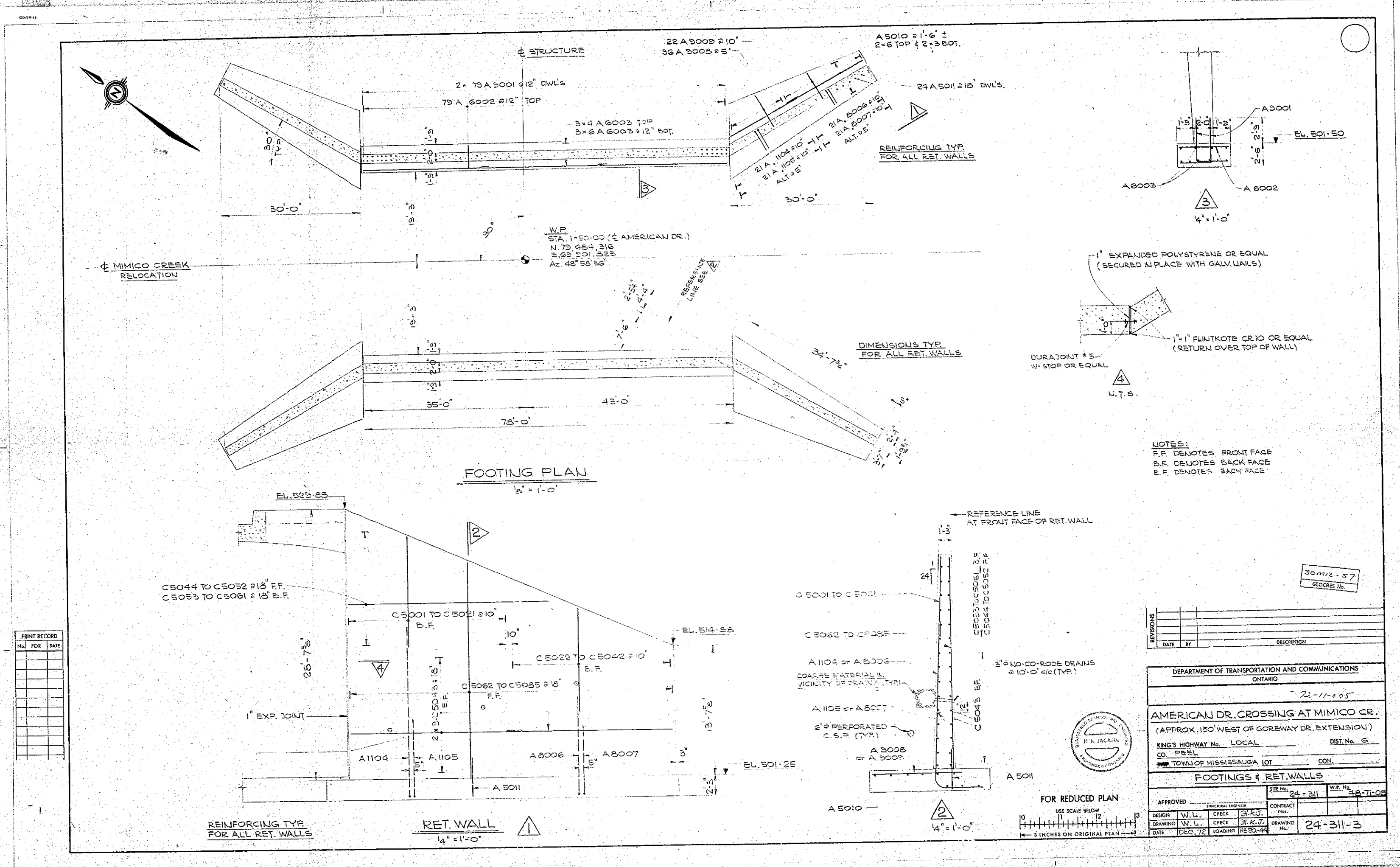
72-11-005

AMERICAN DR. CROSSING AT MIMICO CR.
(APPROX. 150' WEST OF GOREWAY DR. EXTENSION)

KING'S HIGHWAY No. LOCAL DIST. No. 6
CO. PEEL
~~TWP.~~ TOWN OF MISSISSAUGA LOT CON.

GENERAL LAYOUT

APPROVED	STRUCTURAL ENGINEER			SITE No. 24-311	W.P. No. 48-71-08
				CONTRACT Nos.	
DESIGN	W.L.	CHECK	H.K.J.	DRAWING No.	24-311-1
DRAWING	W.L.	CHECK	H.K.J.		
DATE	DEC. 72	LOADING	HS20-44		



℄ STRUCTURE
22 A 3009 @ 10"
36 A 3008 @ 5"
A 5010 = 1'-6" ±
2×6 TOP & 2×3 BOT.
2× 79 A 3001 @ 12" DWL'S
79 A 6002 @ 12" TOP
3×4 A 6003 TOP
3×6 A 6003 @ 12" BOT.
24 A 5011 @ 18" DWL'S.
REINFORCING TYP. FOR ALL RET. WALLS
30'-0"
W.P.
STA. 1+50.00 (℄ AMERICAN DR.)
N. 79, 484, 316
E. 63, 201, 523
Az. 48° 58' 36"
℄ MIMICO CREEK RELOCATION
REFERENCE LINE SEE 2
DIMENSIONS TYP. FOR ALL RET. WALLS
35'-0"
43'-0"
78'-0"
FOOTING PLAN
1/8" = 1'-0"
A 3001
EL. 501.50
A 6003
A 6002
3
1/4" = 1'-0"
1" EXPANDED POLYSTYRENE OR EQUAL (SECURED IN PLACE WITH GALV. NAILS)
1"×1" FLINTKOTE CR10 OR EQUAL (RETURN OVER TOP OF WALL)
DURAJOINT # 5 W-STOP OR EQUAL
4
N.T.S.
NOTES:
F.F. DENOTES FRONT FACE
B.F. DENOTES BACK FACE
E.F. DENOTES EACH FACE
EL. 523.65
C 5044 TO C 5052 @ 18" F.F.
C 5053 TO C 5061 @ 18" B.F.
C 5001 TO C 5021 @ 10" B.F.
C 5022 TO C 5042 @ 10" B.F.
C 5062 TO C 5085 @ 18" F.F.
EL. 514.65
1" EXP. JOINT
A 1104
A 1105
A 8006
A 8007
A 5011
EL. 501.25
REINFORCING TYP. FOR ALL RET. WALLS
RET. WALL
1/4" = 1'-0"
REFERENCE LINE AT FRONT FACE OF RET. WALL
C 5001 TO C 5021
C 5062 TO C 5085
A 1104 or A 8006
COARSE MATERIAL IN VICINITY OF DRAINS (TYP.)
A 1105 or A 8007
6" Ø PERFORATED C.S.P. (TYP.)
A 8008 or A 8009
3" Ø NO-CO-RODE DRAINS @ 10'-0" c/c (TYP.)
A 5010
A 5011
2
1/4" = 1'-0"
FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN
30 M12 - 57
GEOCRES No.
REVISIONS
DATE
BY
DESCRIPTION
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO
72-11-005
AMERICAN DR. CROSSING AT MIMICO CR.
(APPROX. 150' WEST OF GOREWAY DR. EXTENSION)
KING'S HIGHWAY No. LOCAL
DIST. No. 6
CO. PEEL
TOWN OF MISSISSAUGA
LOT
CON.
FOOTINGS & RET. WALLS
SITE No. 24-311
W.P. No. 48-71-08
APPROVED
STRUCTURAL ENGINEER
CONTRACT Nos.
DESIGN W.L.
CHECK H.K.J.
DRAWING W.L.
CHECK H.K.J.
DATE DEC. 72
LOADING HS20-44
DRAWING No. 24-311-3