

FOUNDATION  
INVESTIGATION & DESIGN  
REPORT

SOIL MECHANICS SECTION

ENGINEERING SERVICES BRANCH  
GEOTECHNICAL OFFICE



Ministry of  
Transportation and  
Communications

31 F - 92

GEOCRES No.

FOUNDATION INVESTIGATION & DESIGN REPORT

W.P. 199-62-00

DIST. 9

HWY. 17N

STR. SITE 29-200

CPR Overhead - Mileage 39.49  
Chalk River Subdivision  
Hwy. 17N, Arnprior Bypass

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## INTRODUCTION

The proposed Arnprior Bypass would require a structure at the crossing of relocated CPR tracks and Hwy. 17N. The relocation of the CPR tracks was necessitated by the Ontario Hydro Dam Project.

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

## 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 ft. in depth. The bottom 10 to 15 ft. 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 midheight berm of about 16 ft. followed by a lower slope of 3.5:1. In addition a berm of a minimum distance of 5 ft. 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.

## FIELD AND LABORATORY INVESTIGATION

The field investigation consisted of six boreholes, three of which were accompanied by a dynamic cone penetration test. In addition two independent dynamic cone penetration tests and three bedrock soundings were also carried out. Bedrock elevations where exposed in the vicinity of the footing locations were also established. Boring was achieved by wash-boring techniques using a conventional diamond drill

rig adapted for soil sampling purposes. Disturbed samples were obtained using a 2 inch O.D. split-spoon sampler driven according to the specifications for the Standard Penetration Test. Undisturbed samples were obtained using 2 inch I.D. Shelby tubes pushed manually into the soil. In addition field vane tests were carried out where possible. NX size rock core samples were taken at two of the borings and BX size at four other boring locations to prove bedrock. Probable bedrock surface was established at three locations by manually driving a  $\frac{1}{2}$  inch pipe to refusal. The soil, bedrock and groundwater conditions encountered in the borings are presented on the Record of Borehole Sheets and on Dwg. 1996200-A.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Laboratory tests were then carried out on representative samples to determine the following engineering properties of the overburden:

- Natural Moisture Contents
- Atterberg Limits
- Bulk Unit Weights
- Undrained Shear Strength Measurements
- Shear Strength Measurements in terms of Effective Stress
- (C.I.U. Tests with Pore Pressure Measurements)

The results of laboratory testing were plotted on the individual Record of Borehole Sheets and on the plasticity chart (Fig. 1), all of which are appended to this report.

### SUBSURFACE CONDITIONS

#### General

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

As previously mentioned the bedrock is exposed in the cut section a distance of up to 30 ft. 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 Dwg. 1996200-A 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 ft. in thickness and is sensitive, brittle, and fissured. According to the geology of the area, this cohesive material was deposited by the post glacial Champlain Sea.

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

<u>Index Properties</u>		<u>Range</u>	<u>Average</u>
Natural Moisture Content	W (%)	28 - 44	38
Liquid Limit	W <sub>L</sub> (%)	25 - 42	34
Plastic Limit	W <sub>p</sub> (%)	14 - 22	18
Liquidity Index	I <sub>L</sub>	1.0 - 1.8	1.2
Bulk Unit Weight	γ (pcf)	111 - 117	113

#### Undrained Shear Strength (C<sub>u</sub>) psf

In situ vane tests	1200 - >2,240
Unconfined Compression Tests	1070 - 1,795
Quick Triaxial Tests	2,335 - 2,380

The Atterberg Limit Test results, given in the Table, are also summarized on the Plasticity Chart, Fig. 1. The testing indicates that the cohesive stratum is inorganic and of 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 which are presented in the Table, it is estimated that the consistency of the stratum varies from stiff to very stiff. However, at the western portion of the cut near the exposed surface of the bedrock, the dynamic cone penetration test gave values ranging from 1 to 8 blows/ft. and from this it is estimated that the cohesive stratum in this area varies from firm to stiff.

Referring to the undrained shear strength values between the in-situ and the laboratory results, it can be inferred that there is reasonable agreement between the field vanes and the quick triaxials. The unconfined compression tests gave lower shear strength values, and this is believed to be attributed to the fissured nature of the cohesive deposit.

The engineering properties of the cohesive deposit in terms of effective stresses were also determined. This was performed by carrying out isotropically consolidated undrained triaxial compression tests (CIU tests) in which the excess pore water pressure buildup is measured and eventual dissipation, due to the applied load, was monitored throughout. The results of the tests are plotted on Fig. 3 which indicate that shear strength parameters ( $C'$  and  $\phi'$ ) in terms of effective stresses are as follows:

$$C' = 355 \text{ psf}$$

$$\phi' = 25.8^\circ$$

As discussed by Skempton\* the shear stress parameters obtained by conventional tests do not necessarily bear any relation to the values which may be operative in the clay at the time of failure. In our slope stability analyses we used  $C'$  of 250 psf which is about the suggested value by Crawford\* ( $2/3$  of  $C'$  obtained from the laboratory tests) since the rate of strain used in the laboratory is much faster than that which occurs in the field. The  $\phi'$  value of  $22^\circ$  was used in the analyses since this is believed to be the probable value of the clay in this area.

\* See references in the Appendix.

### Silty Sand and Gravel

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

### Bedrock

Bedrock was proven by obtaining 10 ft. to 13 ft. of NX size rock core samples at two of the boring locations, and also by obtaining 5 ft. to 11 ft. of BX size rock core samples at four other boring locations. The CPR tracks are located in a rock cut 10 to 15 ft. deep. Bedrock is exposed along the top of the rock cut for a distance of up to 30 ft. from the face of the cut. The surface of the bedrock varies between elevations 303 and 321, which indicates that the depth to bedrock varies from the existing ground surface to 26 ft. below ground surface.

The bedrock can be identified as hard to medium hard crystalline limestone. It is generally sound, as evidenced by the quality and the percentage recovery of the rock cores. In one of the borings however, the upper six feet of the rock was weathered. Rock Quality Designation (RQD) values are shown on the Record of Borehole Sheets. The Rock Quality Designation (RQD) is defined as the sum of the lengths of recovered core pieces which are 4 inches in length or longer divided by the total length of the rock core drilled. It is a method of judging the quality of rock core based on the relative amount of fracturing and alteration. RQD values in the sound bedrock ranged from 50% to 100% indicating that the rock quality is fair to excellent. Detailed descriptions of the rock cores as described by Mr. B. Glassford, Geologist for MTC are presented on the Diamond Drill Record Sheets included in the Appendix. The exposed bedrock in the vicinity of the CPR tracks contains vertical, as well as inclined jointing system and the majority of them are facing towards the river. The geological mapping of the CPR rock cut which is in the vicinity of the proposed structure location is shown on Fig. 2 which is included in the Appendix. This is a portion of the geological mapping prepared by Acres Consulting

Services Ltd. in connection with the Ontario Hydro Dam project.

#### GROUNDWATER CONDITIONS

Groundwater level observations were carried out during the course of the recent field investigation by recording the water level in the open boreholes. The observations show all the boreholes to be dry. The piezometric surface, as measured in the bedrock, by personnel from Acres Consulting Services, before the start of excavation, was within 10 ft. of the original ground surface. It has now been lowered below the track elevation by means of a permanent subsurface drainage system (pump wells) in order to carry out excavation for the newly constructed CPR tracks and to ensure the long term stability of the cut slopes.

#### DISCUSSIONS AND RECOMMENDATIONS

The proposed Arnprior Bypass would require a structure at the crossing of relocated CPR tracks and Hwy. 17N. The relocation of the CPR tracks was necessitated by the Ontario Hydro Dam Project.

Two alternate proposals have been considered; one being a three span structure (span lengths 100' - 135' - 100'), the other being a single span structure approximately 100' long.

The subsoil generally consists of a thin mantle of topsoil (8 to 12 in.) overlying a deposit of up to 26 feet of firm to very stiff clayey silt to silty clay which is somewhat fissured, brittle and sensitive. Some seams or thin layers (up to 3 inches) of silt and fine sand are also present within this cohesive stratum. This cohesive overburden is underlain by limestone bedrock, except in certain locations where the bedrock is covered with a thin granular layer of about 0.5 feet to 2.5 feet.

Recommendations pertaining to the structure foundations are presented below for both the schemes.



### Three Span Structure

#### Abutments (West - Sta. 800 + 70, East - Sta. 803 + 95)

The abutments may be supported on spread footings placed within the silty clay to clayey silt deposit with an allowable bearing pressure of 2 tsf. Since the cohesive deposit is sensitive to disturbance we recommend that the base of excavation be covered immediately with 3 in. of lean concrete.

The induced stresses beneath the footings will not exceed the pre-consolidation pressure of the soil. Therefore, settlement will be of a recompression nature and will take place during or immediately following the construction period. No major settlement problems are anticipated. A minimum of 5 feet of earth cover should be provided above the base of the footing for frost protection purposes.

The bedrock surface for the west abutment is approximately at elev. 309 which is some 24 to 25.5 feet below existing ground surface, while for the east abutment, the bedrock surface is about elev. 318 to 321 which is 15 to 18 feet below ground level. The abutments could also be supported on end bearing piles driven to bedrock. The piles can be designed using maximum allowable capacity of the pile section chosen.

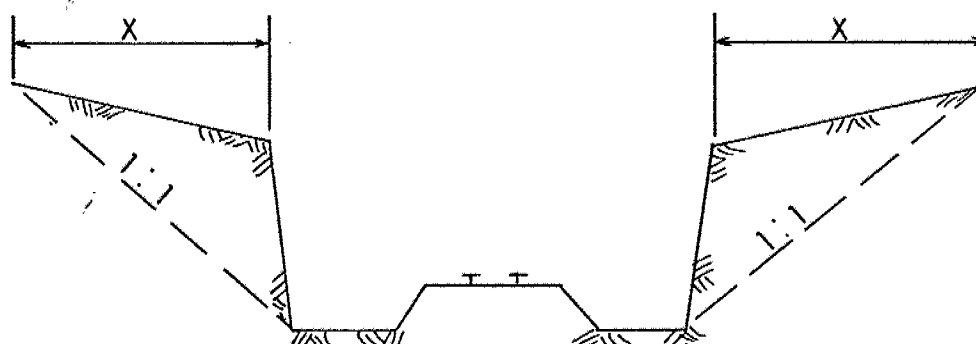
The east abutment could also be founded on spread footings placed within the bedrock with an allowable load of up to 20 tsf.

#### Piers (West - Sta. 810 + 70, East - 802 + 95)

The bedrock surface for the west pier is approximately at elev. 305 which is some 11 to 14 feet below existing ground surface, while for the east pier the bedrock is exposed. Since a minimum of 5 ft. earth cover has to be provided for frost protection requirements, it would be advantageous to carry the excavation deeper for the west pier and thus be able to place the footings on bedrock. The east pier could be founded on spread footings placed on bedrock. Both the pier footings could be designed with an allowable load of up to 20 tsf.

### Single Span Structure

The footings for the west and east abutments were originally located at Sta. 802 + 05 and Sta. 802 + 70 respectively. The bedrock in this area contains vertical, as well as inclined jointing system and the majority of them are sloping towards the river. In order to improve the rock condition at some critical locations, reinforcement with rock anchors will be essential. Alternatively, the footing should be located at a distance  $X$  as per sketch below or 10 feet, whichever is greater.



At the time of the site visit undertaken by Mr. T. Kingsland, Mr. M. Devata and Mr. H. Shah, it was tentatively agreed to locate the west and east abutments at Sta. 801 + 93 and 802 + 82 respectively. Accordingly, the investigation was confined to these locations.

Bedrock at the west abutment location ranges from elev. 306 to 312 and this corresponds to depths of 4 to 7 feet below existing ground. At the east abutment location, bedrock is exposed. If the above mentioned footing locations are adopted the abutments could be founded on spread footings placed within the bedrock and with an allowable load of up to 20 tsf.

### Embankments

#### Three Span Structure

Fill heights of up to 8 feet will be required on the east approach to

attain the proposed profile grade. However, this amounts to only 3 feet of fill above the original ground before the excavation for the relocated CPR tracks started. Cuts of about 3 feet would be required to attain the profile grade for the west approach. Standard 2:1 slopes would be stable. No settlement problems are anticipated.

### Single Span Structure

Fills required to attain the profile grade would in essence amount to backfilling that portion which was excavated to relocate the CPR tracks. Retaining walls would be required to retain the embankment fills in the forward direction. Analyses, in terms of total stresses have been carried out in the transverse direction to determine the stability of the fills immediately after construction. In this method of analysis, stability is governed by undrained shear strength properties of the foundation and fill material. The following data and values were used in carrying out the stability analyses.

### Fill Material

<u>Granular Type</u>		<u>Cohesive Type</u>
Bulk Density	$\gamma = 130$ pcf	$\gamma = 130$ pcf
Apparent Angle of Shearing Resistance	$\phi = 35^\circ$	$\phi = 0^\circ$
Apparent Cohesion	$C_u = 0$	$C_u = 750$ psf

### Foundation Subsoil

<u>Soil Type</u>	<u>Elev (Ft.)</u>	<u><math>\gamma</math> (PCF)</u>	<u><math>\gamma'</math> (PCF)</u>	<u><math>\phi_u^\circ</math></u>	<u><math>C_u</math> (PSF)</u>
Clayey Silt to Silty Clay	319-305	130	67.6	0	800
Bedrock	305	-	-	-	-

Water level at existing ground surface.

The stability computations carried out indicate that the proposed embankment height will be stable, if constructed with 2.5:1 slopes.

The long term stability of the fills was also studied in terms of effective shear strength parameters. The following values were used for computational purposes:

Fill Material (Granular or Cohesive Type - same as used in Total Stress Analysis)

<u>Foundation Subsoil</u>					
<u>Soil Type</u>	<u>Elev. (Ft.)</u>	<u><math>\gamma</math> (PCF)</u>	<u><math>\gamma'</math> (PCF)</u>	<u><math>\phi'</math></u>	<u><math>C'</math> (PSF)</u>
Clayey silt to Silty Clay	319 - 305	130	67.6	22	250
Bedrock	305	-	-	-	-

Water level at existing ground surface -(no excess pore water pressure since water table lowered for CPR excavations).

The stability analyses in terms of effective stresses indicate that the long term stability of the proposed embankment height will have an adequate factor of safety if constructed with 2.5:1 slopes.

#### MISCELLANEOUS

The field work for this investigation was carried out during the period of June 28, 1976 to July 7, 1976 under the supervision of Mr. S. Maloney, Student Technician.

The equipment used for subsoil sampling was owned and operated by Johnston Drilling Company. This report was written by Mr. H. Shah and was reviewed by Mr. M. Devata, Supervising Engineer.

H. Shah

H. Shah, P. Eng.  
Project Engineer

M. Devata

M. Devata, P. Eng.  
Supervising Engineer



October/76

## APPENDIX



## REFERENCES

1. Crawford, C.B. 1963 "Cohesion in an Undisturbed Sensitive Clay".  
Geotechnique, Vol. 13, pp 132-146.
2. Skempton, A.W. "Long Term Stability of Clay Slopes". Geotechnique,  
I.C.E., London, Vol. 14, No.2, 1964. pp 75-102.

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

## RECORD OF BOREHOLE NO 1

WP 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 10

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS  % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
323.2	Ground Level															
0.0	Crystalline Limestone		1	RC NX	Rec. 67%	320										RQD=80%
	Bedrock		2	RC NX	Rec. 100%											RQD=63%
	Sound		3	RC NX	Rec. 100%											RQD=87%
			4	RC NX	Rec. 100%											RQD=55%
310.1																
13.1	End of Borehole															

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ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 2

WP 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

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$ PCF	REMARKS  % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100					$w_p$ $w$ $w_L$				
							SHEAR STRENGTH PSF									
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					WATER CONTENT % 20 40 60				
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		
			3	TW	PM											
			4	SS	6											
			5	TW	PM											
321.8	Stiff to Very Stiff		6	SS	6											
321.3	Silty sand, some grav. Compact															
14.5	Crystalline Limestone Bedrock		7	RC BX	Rec. 95%	320									RQD=63%	
	Sound		8	RC BX	Rec. 100%										RQD=100%	
312.2																
23.6	End of Borehole Borehole Dry															



# RECORD OF BOREHOLE NO 3

W.P. 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

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			UNIT WEIGHT $\gamma$ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$W_P$	$W$	$W_L$		
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											
			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															

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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

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			UNIT WEIGHT $\gamma$ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$W_P$	$W$	$W_L$		
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.		2	TW	PM										111	
			3	TW	PM											
			4	SS	7											
			5	TW	PM											
	Stiff to Very Stiff		6	TW	PM										111	
311.8			7	SS	4											
21.8	Silty sand & gravel		8	SS	20											
309.8	Compact															
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																
34.4	End of Borehole Borehole Dry															

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ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

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 rSM  
DATUM Geodetic BOREHOLE TYPE Washboring, BX Rock Coring, Cone Test - CHECKED BY *LB*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			UNIT WEIGHT $\gamma$ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$W_P$	$W$	$W_L$		
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		3	TW	PM										117	
			4	TW	PM											
308.9			5	SS	7	310										
25.5	Crystalline Limestone		6	RC BX	82% Rec.											RQD=50%
	Bedrock															
	Sound		7	RC BX	91% Rec.	300										RQD=55%
297.6																
36.8	End of Borehole															
	Borehole Dry															

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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

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

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

RECORD OF BOREHOLE NO 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 10

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$ $w_p \quad w \quad w_L$ WATER CONTENT %	UNIT WEIGHT $\gamma$	REMARKS  % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100			
317.2	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								
303.0														
14.2	End of Cone Probable Bedrock													

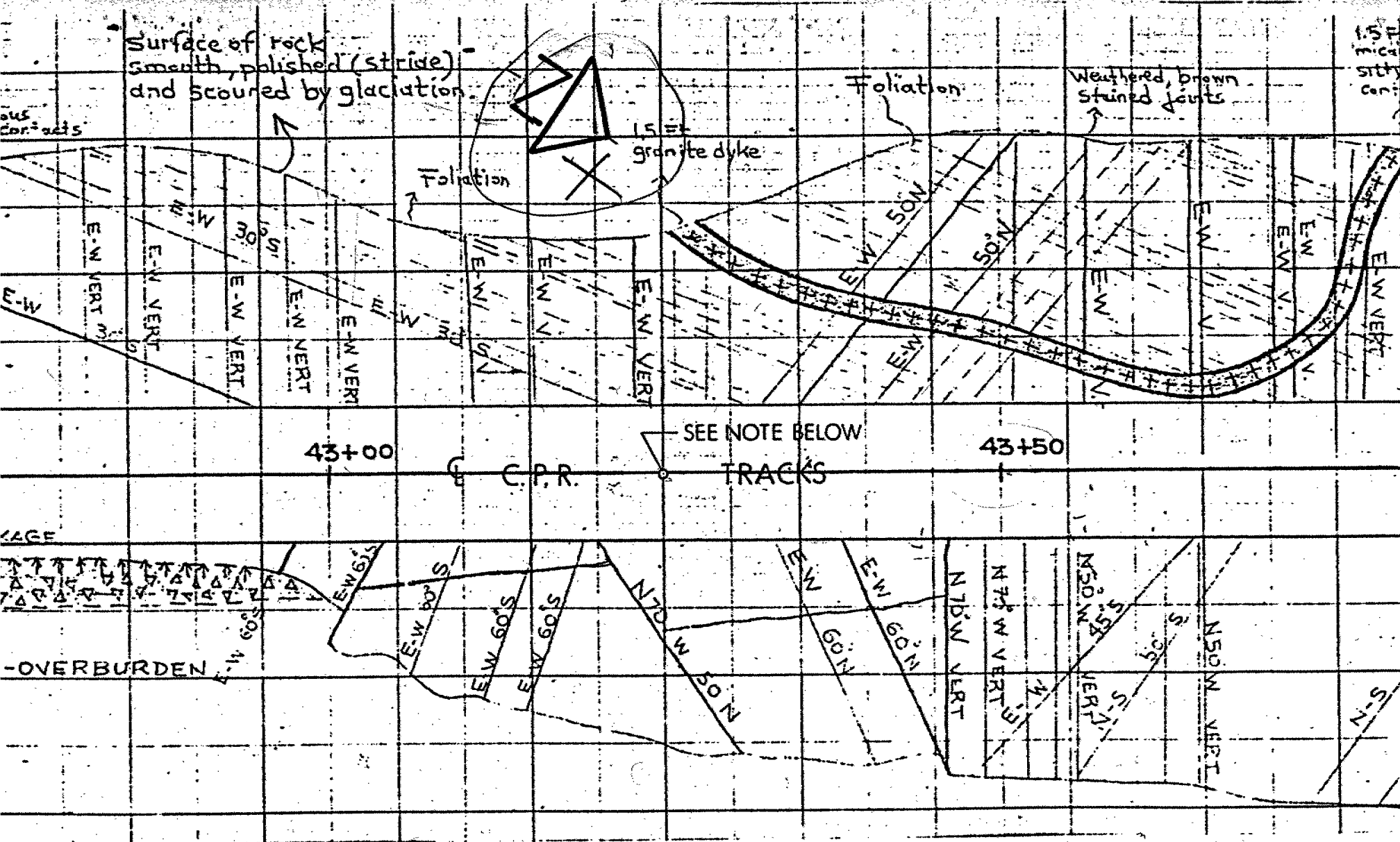
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ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

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 *SM*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			UNIT WEIGHT $\gamma$	REMARKS  % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$ — $w$ — $w_L$ WATER CONTENT %				
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	End of Cone Probable Bedrock															
11.0																





NOTE: PROPOSED HWY. 17'N' CORRESPONDS TO STA. 43+25± ON THIS FIGURE *I think this station is wrong. It should be Sta. 43+85 for the old line & 43+80 for the new line.*

SCALE: 1"=10'

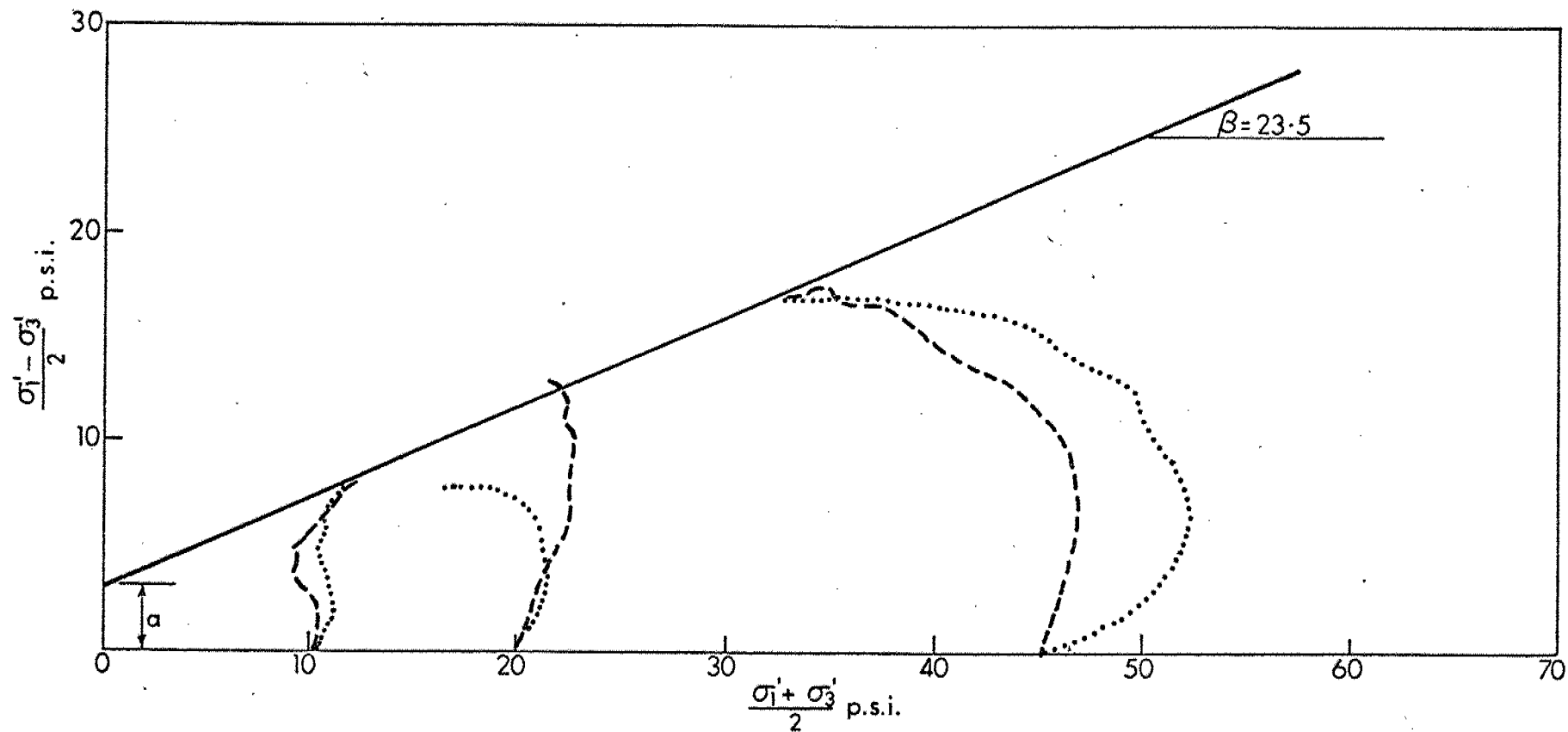
THIS GEOLOGICAL MAPPING IS A PORTION OF  
DWG. NAF-9 PREPARED BY ACRES CONSULTING  
SERVICES FOR ONT. HYDRO

Bm (Feb. 15, '77)

FIG. 2

W.P. 199-62-00





LEGEND		
SYMBOL	B. H.	S. A.
---	4	6
.....	5	3

SUMMARY OF C.I.U. TESTS  
SILTY CLAY TO CLAYEY SILT

FIG. 3

W.P. 199-62-00



HOLE NO. 1 SHEET NO.           

DIP

90°

PROPERTY	W. P199-62-00
LOCATION	Arnprior By-pass and C. P. R. X-ing Station 802+80 18' Lt. of G.
LATITUDE	
DEPARTURE	
BEARING	

TOTAL FOOTAGE 13.1'

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

[illegible]

DATE OF EXAMINATION Aug. 13/76

B. K. Glassford



Ministry of  
Transportation and  
Communications

# DIAMOND DRILL RECORD

DIP

90°

HOLE NO. \_\_\_\_\_ SHEET NO. \_\_\_\_\_

PROPERTY \_\_\_\_\_ W. P. 199-62-00  
LOCATION \_\_\_\_\_ Arnprior By-pass Hwy. 17  
\_\_\_\_\_ C. P. R. Crossing  
\_\_\_\_\_ Station 803+00  
LATITUDE \_\_\_\_\_  
DEPARTURE \_\_\_\_\_  
BEARING \_\_\_\_\_

\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_  
TOTAL FOOTAGE \_\_\_\_\_

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

FOOTAGE		FORMATION	SAMPLE NUMBER			REMARKS
FROM	TO					
		Hole #2				Joints
14.5'	23.6'	Crystalline limestone, white grey colour, coarse texture, bedding appears to be horizontal, hard to medium hard				50° at 15.5' open, core is moderately fractured throughout
		Hole #3				Joints
18.3'	23.4'	Crystalline limestone, white grey colour, coarse texture, bedding appears to be horizontal, hard to medium hard				55° at 20.0' open, 60° at 23.0' open core is moderately fractured throughout
		Hole #4				Joints
23.9'	34.4'	Limestone boulder 23.9'-24.3', crystalline limestone, white grey colour, coarse texture, hard to medium hard, bedding appears to be horizontal, mica and sulphide mineralization at 25.8'-31.0'				0° at 31.0' open, 80° at 32.0' open broken, ground core missing, core 24'-30' core is badly fractured throughout this section
		Hole #5				Joints
25.8'	36.9'	Crystalline limestone, white grey colour, coarse texture, bedding appears horizontal, hard to medium hard, mica and sulphide mineralization at 25.8'-26.3', rock appears leached 31.0'-36.0'				90° at 25.8' open 80° at 26.8' open 45° at 28.0' open 90° at 34.7' open 90° at 35.8' open Weathered zones at 25.8'-26.3' 31.0'-36.0'
		All horizontal fracturing appears to have been caused by the drilling procedures				

DATE OF EXAMINATION \_\_\_\_\_ Aug. 18/76

\_\_\_\_\_  
B. K. Glassford



HOLE NO. 6 SHEET NO. \_\_\_\_\_

90°

TOTAL FOOTAGE	24.0'

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

[illegible]

B. K. Glassford

## ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

### PENETRATION RESISTANCE

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

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

### DESCRIPTION OF SOIL

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

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

TERMS TO BE USED IN DESCRIBING SOILS:-

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

### TYPE OF SAMPLE

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

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

### SOIL TESTS

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

## ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

### SOIL PROPERTIES

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

### GENERAL

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

### STRESS AND STRAIN

$u$	PORE PRESSURE
$\sigma$	NORMAL STRESS
$\sigma'$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
$E$	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
$G$	MODULUS OF SHEAR DEFORMATION
$K$	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

### EARTH PRESSURE

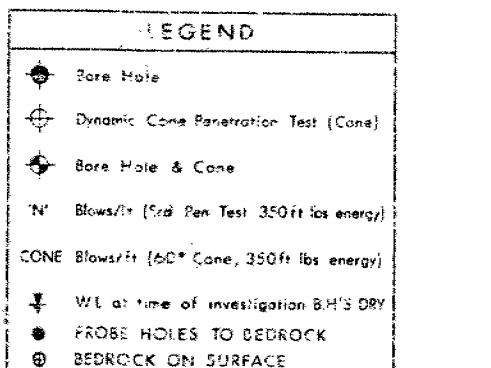
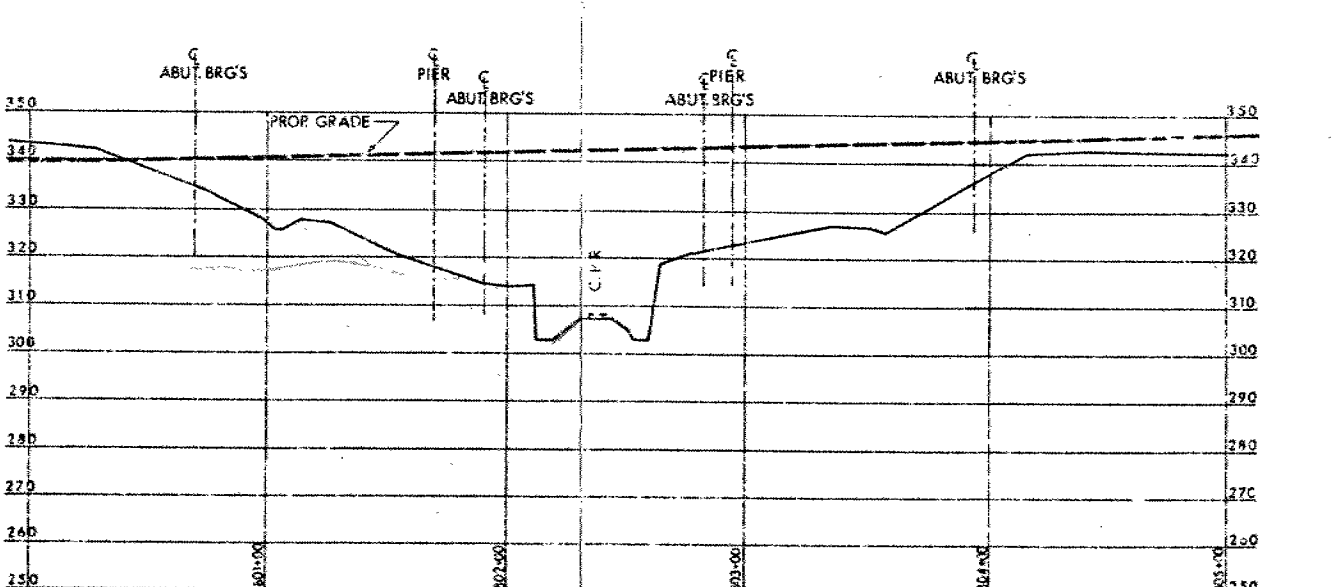
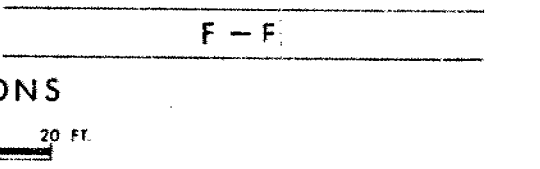
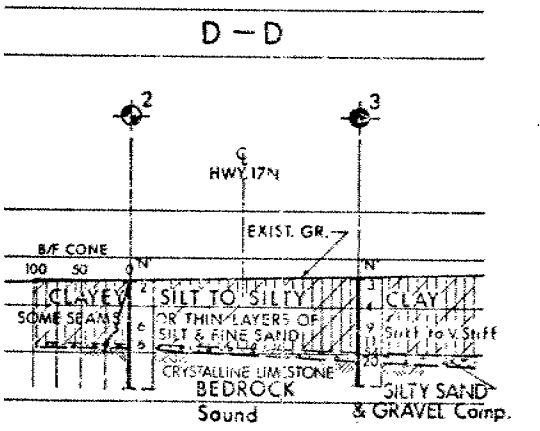
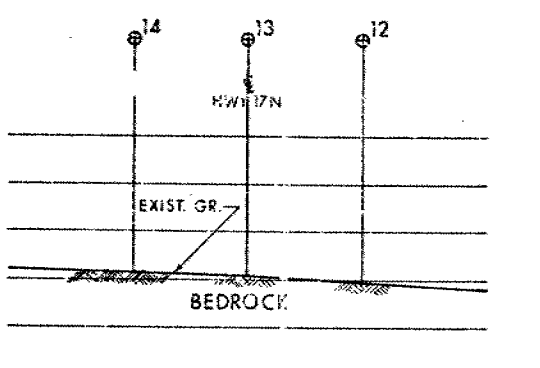
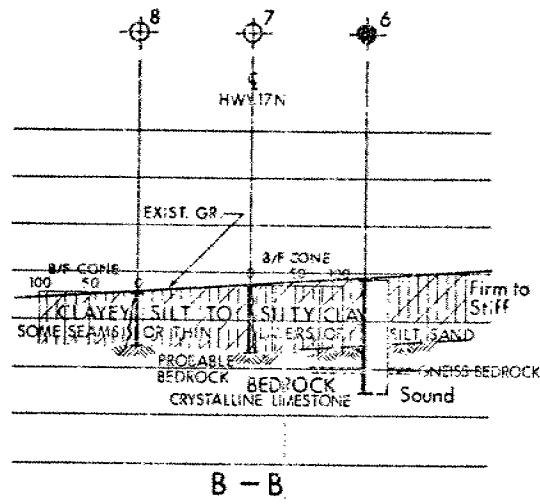
$d$	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
$K$	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

### FOUNDATIONS

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

### SLOPES

$H$	VERTICAL HEIGHT OF SLOPE
$D$	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL



**-NOTE-**

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

HWY No 174 DIST 9  
SUBMITTER'S PHONE NO DATE 14 Oct 79 SITE 29-200  
DRAWN BY J. J. J. APPROVED DWG 1996200-A



NOTES:

- CLASS OF CONCRETE:**
- GIRDERS — 6000 P.S.I.
  - DECK & BARRIER WALLS — 4000 P.S.I.
  - REMAINDER — 3000 P.S.I.
- REINFORCING STEEL:**
- GRADE 400
  - REINF. BARS WITH THE DESIGNATION 'C' AT THE END OF BAR MARKS SHALL BE COATED BARS.
  - CLEAR COVER TO REINF. STEEL:
  - FOOTINGS, ABUTMENTS & RET. WALLS — 3"
  - DECK TOP & APPROACH SLABS — 2"
  - DECK BOTTOM & DIAPHRAGMS — 1 1/2"
  - UNLESS NOTED OTHERWISE

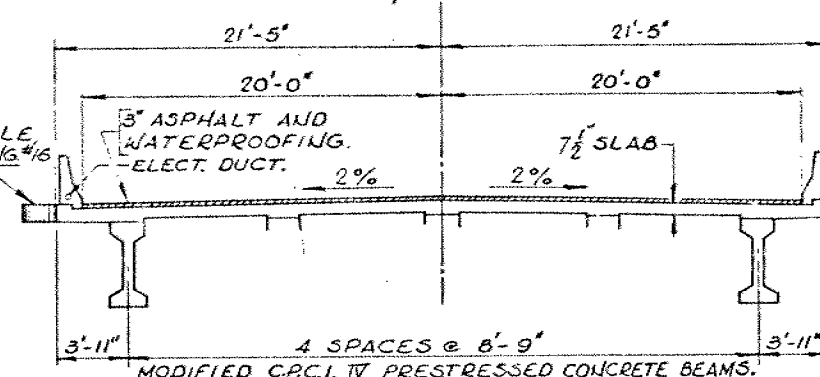
CONSTRUCTION NOTES:

- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF 1/8".
- NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.
- TO ACHIEVE THE MIN. CLEAR COVER OF 2" SPECIFIED AT TOP OF DECK, THE TOP LAYER OF REINFORCEMENT SHALL BE PLACED, PRIOR TO CONCRETING, WITH A CLEAR COVER OF 2 1/2" ± 1/2" TOLERANCE.

LIST OF DRAWINGS:

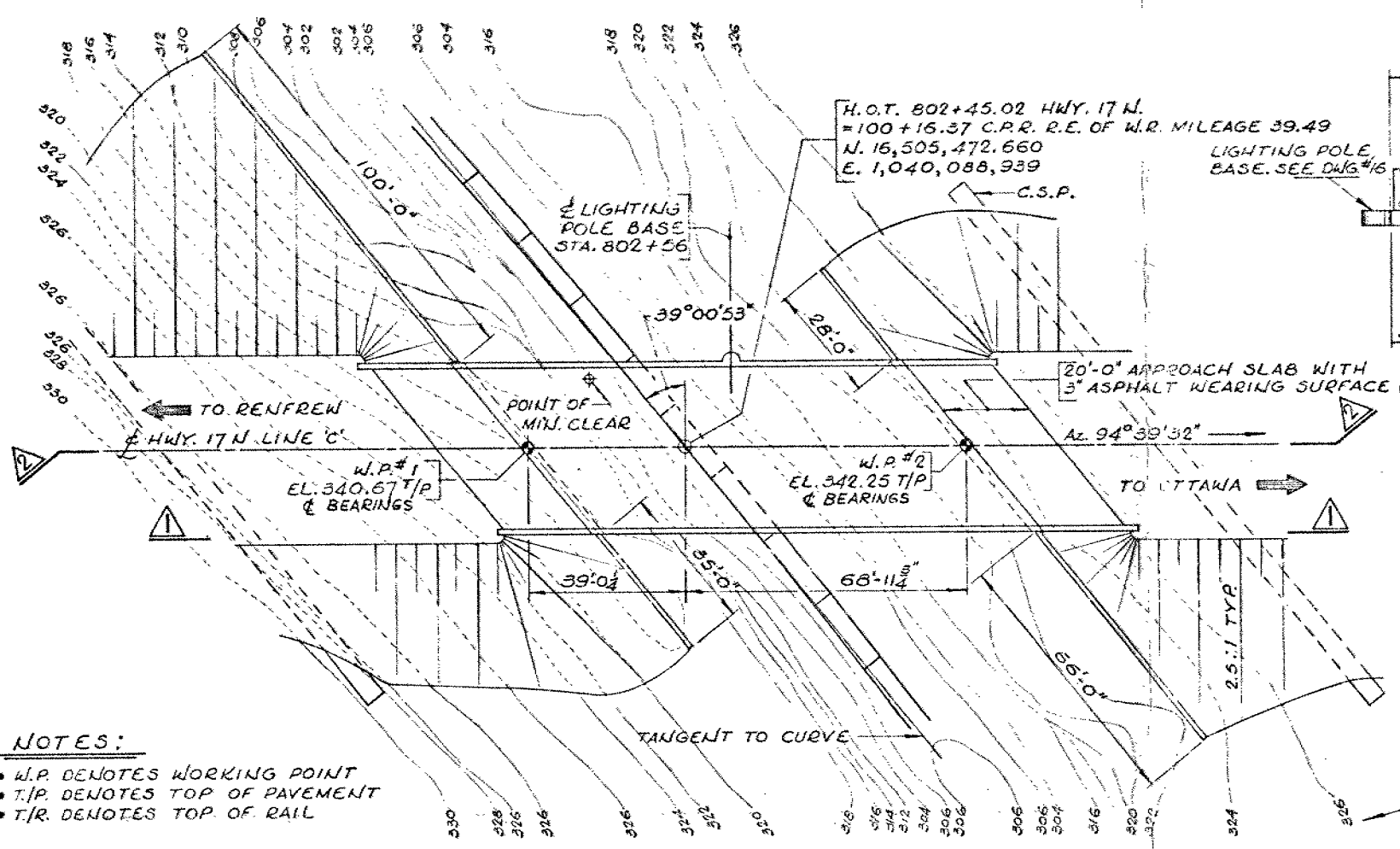
- GENERAL PLAN
- BOREHOLE LOCATION & SOIL STRATA
- FOUNDATION LAYOUT
- ABUTS. FOOTING REINF.
- WEST ABUTMENT
- EAST ABUTMENT
- RETAINING WALLS DETAILS I
- RETAINING WALLS DETAILS II
- PRESTRESSED GIRDERS & BEARINGS
- DECK
- BARRIER WALL
- STEEL RAILING (SINGLE TUBE)
- 20 FT. APPROACH SLAB
- STANDARD DETAILS I
- STANDARD DETAILS II
- STANDARD DETAILS III
- AS CONSTRUCTED ELEV. & DIM.

HWY. 17N LINE 'C'



TYP. DECK SECTION

SCALE: 3/16" = 1'-0"



PLAN

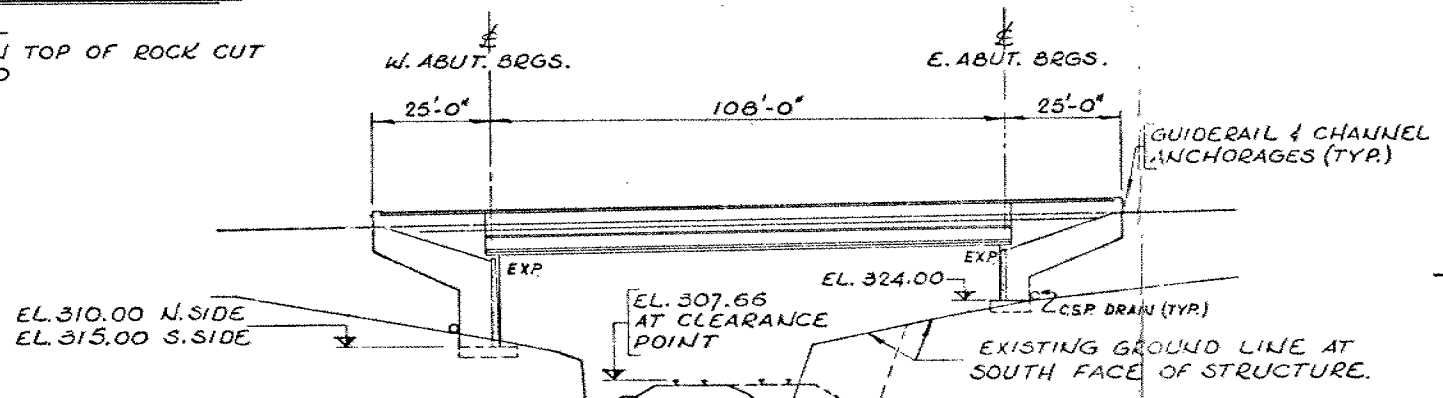
SCALE: 1" = 20'-0"

NOTES:

- W.P. DENOTES WORKING POINT
- T.P. DENOTES TOP OF PAVEMENT
- T.R. DENOTES TOP OF RAIL

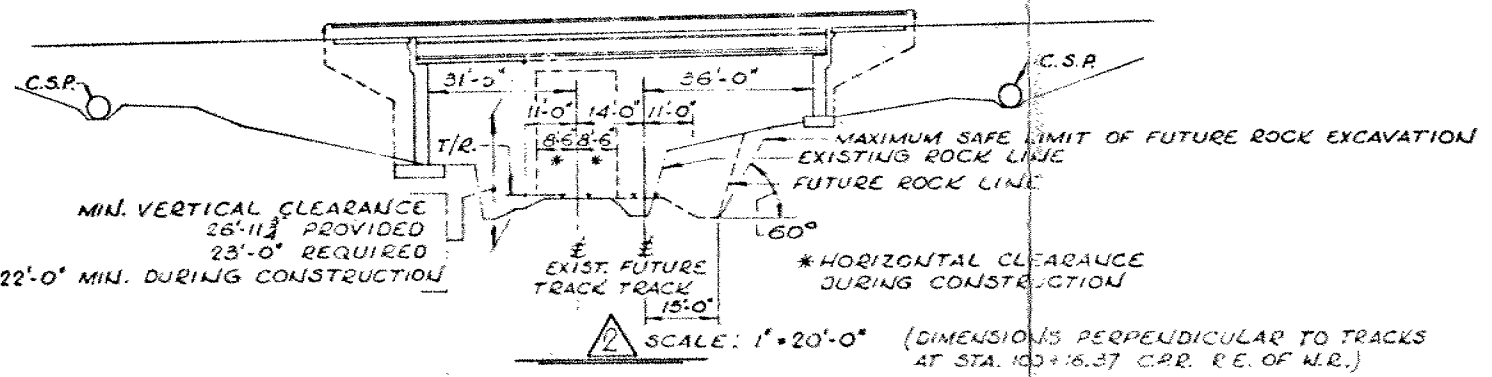
REFERENCE BENCH MARK

B.M. 320.66  
CUT CROSS ON TOP OF ROCK CUT  
212' RT. 804+40



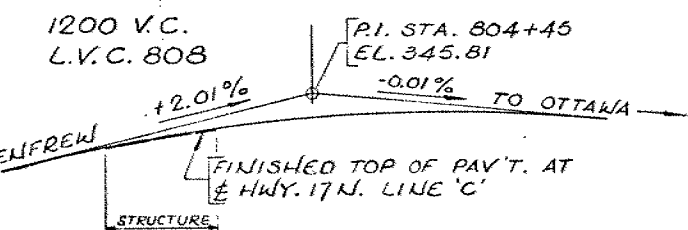
ELEVATION

SCALE: 1" = 20'-0"



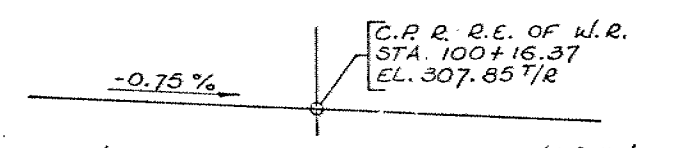
SCALE: 1" = 20'-0"

(DIMENSIONS PERPENDICULAR TO TRACKS AT STA. 100+16.37 C.P.R. R.E. OF W.R.)



PROFILE OF HWY. 17N LINE 'C'

N.T.S.



PROFILE OF C.P.R.

N.T.S.

CONCRETE QUANTITIES:

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONC. LUMP SUM TENDER ITEMS.

- CONCRETE IN ABUTMENTS, WING WALLS AND RETAINING WALLS — 495 CY.
- CONCRETE IN DECK AND DIAPHRAGMS — 145 CY.
- CONCRETE IN BARRIER WALLS — 25 CY.
- CONCRETE IN APPROACH SLABS — 49 CY.



FOR REDUCED PLAN



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	6-2	CHECK	LOADINGS 20-44 DATE 10-78
DRAWING	5-5	CHECK	SITE No. 20-200 DWG. 1





USE SCALE BELOW

0 1 2

3 INCHES ON ORIGINAL PLAN

[illegible]



Ministry of  
Transportation and  
Communications

## Memorandum

To: C. E. Pritchard  
Head, Planning & Design  
Eastern Region, Kingston

From: Geotechnical Section  
Eastern Region, Kingston

Attention: D. E. Swain

Date: 78 12 20

Our File Ref.

In Reply to

Subject:

W.P. 198-62-00 and 199-62-00  
Madawaska River Bridge and C.P.R.  
Overhead, Arnprior Diversion  
District 9, Ottawa

This memo will confirm our discussion earlier today.

In order to properly key the approach fills to the existing earth cut slopes at the C.P.R. overhead and prevent the possible formation of a slippage plane, both of the existing earth slopes should be benched.

It is felt by the Soil Mechanics Section that this benching is not necessary for the west approach to the Madawaska River Bridge.

W. L. Ball  
Soils Engineer

WLB:sh

c.c. R. W. Franks  
M. Devata  
G. A. Wrong  
G. Luyt



Mr. K.G. Bassi  
Head, Eastern Section  
Structural Office  
2nd Floor, West Building

Soil Mechanics Section  
Engineering Materials Office  
Room 315, Central Building

78 07 31

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

---

We have reviewed the final bridge drawings and find the design of the foundations and approaches to be satisfactory from a soil mechanics' point of view. However, we would like to bring it to your attention that no fills shall be placed on the existing slopes prior to the completion of the retaining walls.

B. Ly  
Senior Engineer

BL/gs

cc: Files ✓

Mr. K. Bassi  
Head, Eastern Section  
Structural Office  
West Building, Downsview

Soil Mechanics Section  
Engineering Materials Office  
Room 315, Central Building

78 06 16

Re: C.P.R. Overhead  
Arnprior Bypass  
W.P. 199-62-00, Site 29-200  
District 9, Ottawa

---

We have reviewed the preliminary design drawings. Our comments are as follows.

The drawings indicate that the various footings will be stepped so that the abutment footings can be keyed at least 6 inches into sound bedrock and the retaining wall footings can be poured against sound bedrock. Due to the irregular nature of the bedrock surface, mass concrete will be required in certain locations in order to achieve the design footing formation levels.

*B. Ly*  
B. Ly  
Senior Engineer

For: M. Devata  
Supervising Engineer

BL/MD/gs

cc: Files ✓

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

Soil Mechanics Section  
Engineering Materials Office  
West Bldg.

77 08 26

CPR Overhead  
Arnprior Bypass, Hwy. 17N  
W.P. 199-62-00 Site 29-200  
Dist. #9

---

We have reviewed the revised preliminary bridge plan drawing 29-200-P3. Our comments are as follows:

Bedrock surface at the west abutment location slopes in a northerly direction from elevation 312 to elevation 304. The west abutment footing, which is to be founded on bedrock, may have to be stepped accordingly, and it may be necessary to place mass concrete on bedrock in order to achieve the footing formation level as shown in the above mentioned preliminary drawings.

B. Ly  
Senior Engineer

BL/gs

cc: Files ✓

Mr. K.C. Bassi  
Structural Engineer  
Structural Office  
West Building, Downsview

Soil Mechanics Section  
Engineering Materials Office  
West Building, Downsview

77 08 11

Re: ✓ W.P. 199-62-00, Site 29-191, CPR Overhead  
W.P. 198-62-00, Site 29-200, Madawaska River Bridge  
Annprior Bypass, Hwy. 17N  
District 9, Ottawa

---

We have reviewed the design drawing of the first stage grading of these projects. Our comments are as follows:

1. Fill should not be placed on the existing slopes adjacent to the CPR tracks until the retaining walls have been constructed to their full heights. If surplus excavated material is available, this can be placed at the culvert site near Sta. 777+75. However, if this material is intended to be used subsequently as a roadway embankment fill, it should be of an acceptable quality.
2. The forward slope of the west approach to the Madawaska River Bridge should be maintained at 2.5:1.

*B. Ly*

B. Ly  
Senior Engineer

For: M. Devata  
Supervising Engineer

BL/gs

cc: T.C. Kingsland  
J. Childs  
G. Wrong  
E.V. Saint  
D.E. Moorhouse  
S. Radbone  
Files ✓

77 07 29

MEMO TO FILE

SUBJECT: C.P.R. Overhead - Arnprior Bypass,  
W.P. 199-62-00, Site 29-200,  
Highway #17N, District #9, Ottawa.

---


A meeting was held on 77 07 25 in the Structural Office to discuss the location of the east abutment footing in relation to the toe of the future rock cut to accommodate an extra track. Present at the meeting were:

K. Bassi - Structural Office  
M. Devata - Soil Mechanics Section  
B. Ly - Soil Mechanics Section

After some discussion it was agreed that:

1. The front edge of the east abutment footing should be located beyond the theoretical 1:1 line from the toe of the future rock cut or 5 ft. from the top of the rock cut, whichever is greater.
2. To make provision for a future track located at 14 ft. from centre line of existing track and with a clearance of 11 ft. from the centre line of the future track to the toe of the rock cut will result in an increase in the span length from 90 ft. to 100 ft.

KGB/cf

  
K. G. Bassi,  
Head, Eastern Section.

c.c./M. Devata  
T. Kingsland



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

Mr. K. Bassi

Soil Mechanics Section  
Engineering Materials Office  
West Building, Downsview

77 04 29

Re: C.P.R. Overhead - Mileage 39.49  
Chalk River Subdivision, W.P. 199-62-00, Site 29-200  
Hwy. 17N, Arnprior Bypass

A foundation investigation was carried out at the site of the above mentioned project during the period of June 28, 1976 to July 7, 1976. During the preliminary structural design stage, the alignment was altered slightly and four retaining walls, up to 100 feet long, were incorporated in the overhead structure. Further probing was necessary to establish subsurface conditions for the retaining walls. Subsequently, probings were carried out on April 6, 1977 and the results of these probings are outlined below.

#### Subsurface Conditions

As established in the foundation investigation during June 28, 1976 to July 7, 1976 and as inferred in the more recent probings, the subsoil may be described as follows. The subsoil at the site generally consists of a thin mantle of topsoil (8 to 12 inches) overlying a deposit (up to 26 feet thick) of stiff to very stiff clayey silt to silty clay. Underlying the cohesive stratum is the limestone bedrock. Furthermore, the bedrock is exposed in the railway cut section for a distance of up to 30 feet back from the face of the cut near the tracks.

The probings carried out recently consisted of obtaining the bedrock elevation at outcrops and where the rock was covered with overburden 1 5/8" diameter steel rods were hammered into the overburden. The elevation of refusal of the rods was further inferred to be the elevation of rock.

Rock elevations as determined by the above assumptions are as follows.

Prob. H. No.	Station	Offset	Ground Elev.	Rock Elev.
51	801 + 25	100' lt.		307.4
52	801 + 40	82' lt.		303.7
53	801 + 53	67' lt.	306.4	300.0
54	801 + 56	62' lt.	307.1	301.4
55	801 + 66	51' lt.	307.2	302.4
56	801 + 90	20' lt.	312.0	306.4
57	802 + 25	40' rt.		317.9

cont'd.....



	<u>Station</u>	<u>Offset</u>	<u>Ground Elev.</u>	<u>Rock Elev.</u>
58	802 + 80	40' lt.		323.1
59	803 + 30	40' rt.		321.5
60	803 + 50	60' rt.		323.1

#### Retaining Wall Foundations

The proposed retaining walls can be supported on spread footings. If the retaining walls are founded on the stiff to very stiff silty clay to clayey silt stratum then a minimum cover of 5 feet should be provided to the underside of the footings for frost protection purposes. Since bedrock at this location outcrops at most locations and elsewhere is underlain by up to 6 feet of overburden, it will be advantageous to found all the retaining walls on spread footings placed within the bedrock and using an allowable design load of 20 t.s.f.

No dewatering problems are anticipated since the footings are located above the prevailing groundwater table. Any surface run-off into the footing excavations can be removed by pumping from sumps.

For estimating the earth pressure on the retaining wall a coefficient of active earth pressure of  $K_a=0.33$  may be used if some movement at the top of the wall is permitted, whereas if no movement at the top of the wall is anticipated, a coefficient of earth pressure at rest  $K_o=0.5$  may be used for design purposes. Furthermore, the calculation for earth pressure should take into account the full effect of the sloping 2:1 surcharge earth fill above the retaining wall.

To estimate horizontal resistance to sliding between rough concrete and the bedrock surface, a coefficient of friction of 0.8 may be used. If additional horizontal resistance to sliding is required, the footing should be keyed into the bedrock or alternatively, the footing may be dowelled to bedrock.

Furthermore, to prevent the build up of hydrostatic pressures behind the retaining wall, free draining Granular 'A' type material should be used for backfill behind the wall placed to the dimensions as shown in current M.T.C. standards.

In addition, no heavy vibratory compaction equipment or other heavy machinery should be allowed within a distance of 15 feet or  $2/3$  the height of the retaining wall. Compaction should be carried out by means of lightweight mechanical compaction equipment to a minimum 95% of the maximum dry density and to a maximum of 100% of the maximum dry density.

cont'd.....

The other recommendations in the foundation report issued on October 28, 1976, are still applicable.

*M Maclean*

M. MacLean  
Project Engineer

For: M. Devata  
Supervising Engineer

MM/gs

cc: Files ✓  
Record Services

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

Mr. K. Bassi

Soil Mechanics Section  
Engineering Materials Office  
West Building, Downsview

February 17, 1977

Re: C.P.R. Overhead  
Arnprior Bypass, Hwy. 17N  
W.P. 199-62-00, Site 29-200  
District 9, Ottawa

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This memorandum contains the results of our review of the Preliminary Bridge Plan Drawing 29-200-P1 for this project.

Since the submission of our foundation report (W.P. 199-62-00), the alignment of the proposed Madawaska River crossing has been modified. As a result of this, the alignment of the proposed CPR overhead has also been shifted. The new alignment is some 16 feet to the south of the old alignment at this particular location.

Furthermore, the proposed overhead has been finalized as a single span structure with closed type abutments founded on bedrock. For this structure scheme, retaining walls along the railway tracks are required to retain the approach fill so that the fill will not spill onto the railway tracks. To design and construct these retaining walls, subsurface data at the wall locations will be required.

We have reviewed the geological mapping prepared by H.G. Acres Ltd. for Ontario Hydro at this particular location. According to Acres' mapping, part of the wall on the west side of the railway tracks will be located in an area where the bedrock is badly weathered and is covered with 4 to 5 feet of overburden. In addition, seepage was also found to exist in that area. In view of this, we intend to carry out an investigation to ascertain the subsurface conditions at the retaining wall areas. The investigation will be carried out in April, 1977, and the results will be submitted to you in May, 1977, together with the pertinent recommendations for you to design these retaining walls.

B. Ly  
B. Ly  
Senior Engineer

For: M. Devata  
Supervising Engineer

BL/gs

cc: T.C. Kingsland  
Files  
Record Services

Mr. T.C. Kingsland  
Regional Structural Planning Engineer  
Structural Planning Office  
Eastern Region, Kingston

Soil Mechanics Section  
Geotechnical Office  
West Building, Downsview

October 7, 1976

W.P. 199-62-00, Site 29-200  
C.P.R. Overhead - Mileage 39.49  
Chalk River Subdivision  
Hwy. 17N, Arnprior Bypass  
District #9, Ottawa

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The proposed Arnprior Bypass would require a structure at the crossing of relocated CPR tracks and Hwy. 17N. The relocation of the CPR tracks was necessitated by the Ontario Hydro Dam Project.

Two alternate proposals have been considered; one being a three span structure (span lengths 100' - 135' - 100'), the other being a single span structure approximately 100' long.

The Soil Mechanics Section carried out the subsurface investigation at the proposed crossing, and the recommendations necessary for design purposes are presented in this memorandum. This memorandum confirms our verbal recommendations on this project provided previously to the Structural Office. The complete Foundation Investigation Report would follow in the near future.

The subsoil generally consists of a thin mantle of topsoil (8 to 12 in.) overlying a deposit (up to 26 feet) of firm to very stiff clayey silt to silty clay which is somewhat fissured, brittle, and sensitive. Some seams or thin layers (up to 3 inches) of silt and fine sand are also present within this cohesive stratum. This cohesive overburden is underlain by limestone bedrock, except in certain locations where the bedrock is covered with a thin granular layer of about 0.5 feet to 2.5 feet.

Recommendations pertaining to the structure foundations are presented below for both the schemes.

#### Three Span Structure

Abutments (West - Sta. 800+70, East - Sta. 803+95)

The abutments may be supported on spread footings placed within the silty clay to clayey silt deposit with an allowable bearing pressure of 2 t.s.f. The induced stresses beneath the footings will not exceed the preconsolidation pressure of the soil.

cont'd.....

Therefore, settlement will be of a recompression nature and will take place during or immediately following the construction period. No major settlement problems are anticipated. A minimum of 5 feet of earth cover should be provided above the base of the footing for frost protection purposes.

The bedrock surface for the west abutment is approximately at elev. 309 which is some 24 to 25.5 feet below existing ground surface, while for the east abutment, the bedrock surface is about elev. 318-321 which is 15 to 18 feet below ground level. The abutments could also be supported on end bearing piles driven to bedrock. The piles can be designed using the maximum allowable capacity of the pile section chosen.

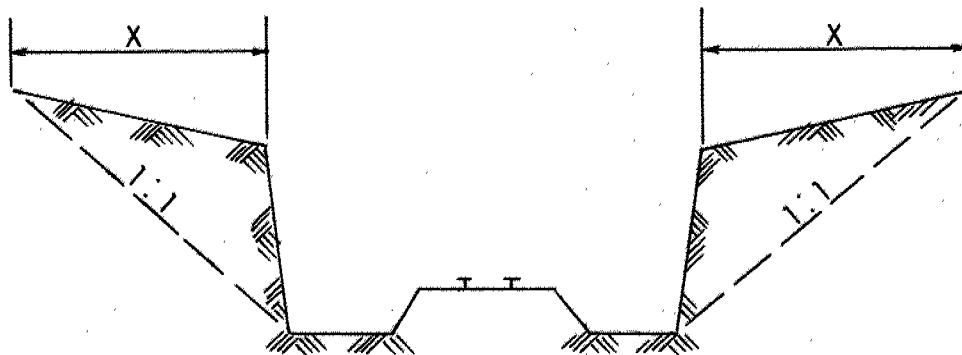
The east abutment could also be founded on spread footings placed within the bedrock with an allowable load of up to 20 t.s.f.

#### Piers (West - Sta. 810+70, East - 802+95)

The bedrock surface for the west pier is approximately at elev. 305 which is some 11 to 14 feet below existing ground surface, while for the east pier the bedrock is exposed. Since a minimum of 5 ft. earth cover has to be provided for frost protection requirements, it would be advantageous to carry the excavation deeper for the west abutment and thus be able to place the footings on bedrock. The east pier could be founded on spread footings placed on bedrock. Both the pier footings could be designed with an allowable load of up to 20 t.s.f.

#### Single Span Structure

The footings for the west and east abutments were originally located at Sta. 802+05 and Sta. 802+70 respectively. The bedrock in this area contains vertical, as well as inclined jointing system and the majority of them are sloping towards the river. In order to improve the rock condition at some critical locations, reinforcement with rock anchors will be essential. Alternatively, the footing should be located the distance X as per sketch below or 10 feet, whichever is greater.



cont'd/.....

At the time of the site visit undertaken by Mr. T. Kingsland, Mr. M. Devata and Mr. H. Shah, it was tentatively agreed to locate the west and east abutments at Sta. 801+93 and 802+82 respectively. Accordingly, the investigation was confined to these locations.

Bedrock at the west abutment location ranges from elev. 306 to 312 and this corresponds to depths of 4 to 7 feet below existing ground. At the east abutment location, bedrock is exposed. If the above recommendations are followed the abutments could be founded on spread footings placed within the bedrock and with an allowable load of up to 20 tsf.

#### Embankments

##### Three Span Structure

Fill heights of up to 8 feet will be required on the east approach to attain the proposed profile grade. However, this amounts to only 3 feet of fill above the original ground before the excavation for the relocated CPR tracks started. Cuts of about 3 feet would be required to attain the profile grade for the west approach. Standard 2:1 slopes would be stable. No settlement problems are anticipated.

##### Single Span Structure

Fills required to attain the profile grade would in essence amount to backfilling that portion which was excavated to relocate the CPR tracks. Analyses both in terms of total stresses and effective stresses, were carried out to determine the stability of the fills immediately after construction and long term stability, respectively. These analyses indicated that the proposed embankment height would be satisfactory provided 2.5:1 slopes are utilized. Retaining walls would be required to retain the embankment fills in the forward direction, due to the close proximity of the CPR tracks. These retaining walls can be supported in a similar manner as those discussed for the single span structure abutments.

Further fieldwork may be necessary depending upon the finalized footing locations.

H. Shah

H. Shah  
Project Engineer

For: M. Devata <sup>Super</sup>  
Supervising Engineer

MD/HS/gs

cc: R. Forrest  
C.S. Grebski - Attn: K. Bassi  
S.C.J. Radbone

Files  
Record Services

GEOCRES No. 31F-92DIST 9 REGION EasternW.P. No. 199-62-00CONT. No. 79-17

W. O. No. \_\_\_\_\_

STR. SITE No. 29-200HWY. No. 17NLOCATION CPR OverheadOVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 4REMARKS: documents to be unfolded  
before microfilming

## 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 10 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.

<u>Index Properties</u>		<u>Range</u>	<u>Average</u>
Natural Moisture Content	W (%)	28- 44	38
Liquid Limit	W <sub>L</sub> (%)	25- 42	34
Plastic Limit	W <sub>p</sub> (%)	14- 22	18
Liquidity Index	I <sub>L</sub>	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	$\phi' = 25\frac{1}{2}^\circ$

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	$\phi' = 22^\circ$

### 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

M. Devata, P. Eng.  
Supervising Engineer



## RECORD OF BOREHOLE NO 1

WP 199-62-00

LOCATION Co-ords. 16,505,500 N; 1,040,128 E.

ORIGINATED BY RS

DIST 9 HWY 17N

BORING DATE June 28, 1976

COMPILED BY SM

DATUM Geodetic

BOREHOLE TYPE NX ROCK CORING

CHECKED BY

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

# RECORD OF BOREHOLE NO 2

WP 199-62-00 LOCATION Co-ords. 16,505,490 N: 1,040,226 E.  
 DIST 9 HWY 17N BORING DATE June 28, 1976  
 DATUM Geodetic BOREHOLE TYPE Washboring, BX Rock Coring, Cone Test

ORIGINATED BY SM  
 COMPILED BY SM  
 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_P$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$w_P$	$w$	$w_L$		
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	
			3	TW	PM											
			4	SS	6											
			5	TW	PM											
321.8	Stiff to Very Stiff		6	SS	6											
321.3	Silty sand, some grav. 15% Compact															
14.5			7	RC EX	Rec. 95%											RQD=63%
	Crystalline Limestone Bedrock															
			8	RC BX	Rec. 100%											RQD=100%
	Sound															
312.2																
23.6	End of Borehole Borehole Dry															

## RECORD OF BOREHOLE NO 3

WP 199-62-00 LOCATION Co-ords. 16,505,450 N; 1,040,254 E.  
 DIST 9 HWY 17N BORING DATE June 29 & 30, 1976  
 DATUM Geodetic BOREHOLE TYPE Washboring and BX Rock Coring

ORIGINATED BY SM  
 COMPILED BY SM  
 CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$ PCF	REMARKS  % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
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															

## 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

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_P$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
333.6	Ground Level															
0.0	Topsail		1	SS	5											
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured.		2	TW	PM										111	
			3	TW	PM											
			4	SS	7											
			5	TW	PM											
	Stiff to Very Stiff		6	TW	PM										111	
311.8			7	SS	4											
21.8	Silty sand & gravel		8	SS	20											
309.8	Compact															
23.8	Boulder															
24.3			9	RC BX	40% Rec.											RQD=0%
	Crystalline Limestone Bedrock															
	weathered sound		10	RC BX	100% Rec.											RQD=80%
299.2																
34.4	End of Borehole															
	Borehole Dry															

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 SB

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			UNIT WEIGHT $\gamma$ PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
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	FM										112	
			2	TW	FM										112	
	Stiff to Very Stiff		3	TW	FM										117	
			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															

OFFICE REPORT ON SOIL EXPLORATION



## RECORD OF BOREHOLE NO 6

WP 199-62-00

LOCATION Co-ords. 16,505,470 N; 1,040,029 E.

ORIGINATED BY SM

DIS: 9 HWY 17N

BORING DATE July 6, 1976

COMPILED BY SM

DATUM Geodetic

BOREHOLE TYPE Washboring and NX Rock Coring

CHECKED BY *SM*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT				LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$	
318.8	Ground Level														
0.0	Topsoil														
	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured.														
	Firm to Stiff					310									
305.2															
13.6	Crystalline Limestone Bedrock Sound		1	RC	96% NX Rec.										RQD=87%
	Gneiss Bedrock		2	RC	97% NX Rec.	300									RQD=80%
294.8															
24.0	End of Borehole														
	Borehole Dry														

## RECORD OF BOREHOLE NO. 7

WP 199-62-00 LOCATION Co-ords. 16,505,490 N; 1,040,016 E.  
 DIST 9 HWY 17N BORING DATE July 7, 1976  
 DATUM Geodetic BOREHOLE TYPE Cone Test

ORIGINATED BY SM  
 COMPILED BY SM  
 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$		UNIT WEIGHT $\gamma$	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		75	25	60	40	100	$w_p$ — $w$ — $w_L$			
							SHEAR STRENGTH					WATER CONTENT %			
															% GR SA SI CL
317.2	Ground Level														
0.0	Clayey silt to silty clay, some seams or thin layers of silt & fine sand - sensitive, brittle, fissured.														
	Firm to Stiff														
303.0															
14.2	End of Cone Probable Bedrock														

## RECORD OF BOREHOLE NO 8

W.P. 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

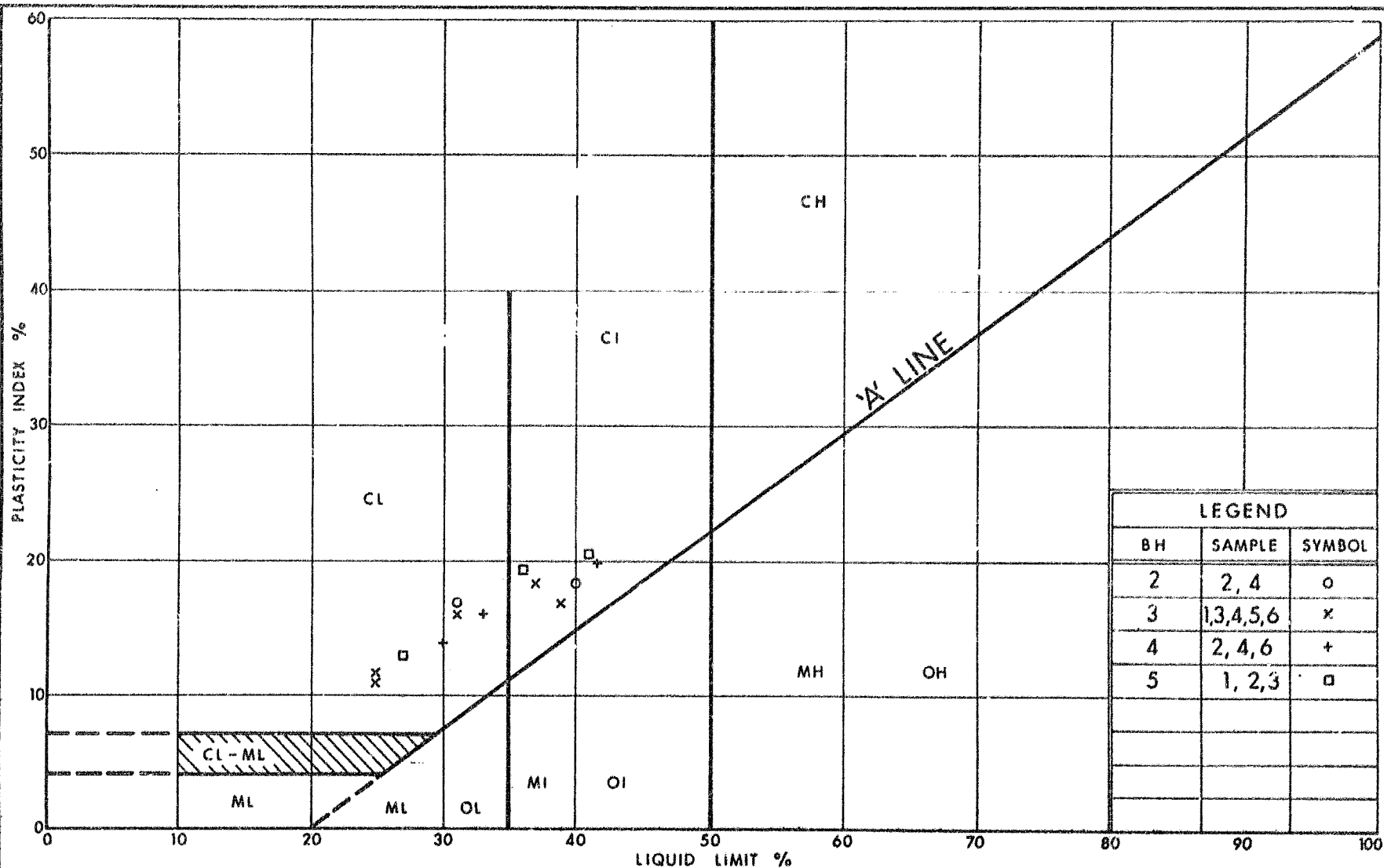
DATUM Geodetic

BOREHOLE TYPE Cone Test

 CHECKED BY *SM*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
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  $\phi$  5 % STRAIN AT FAILURE  
10

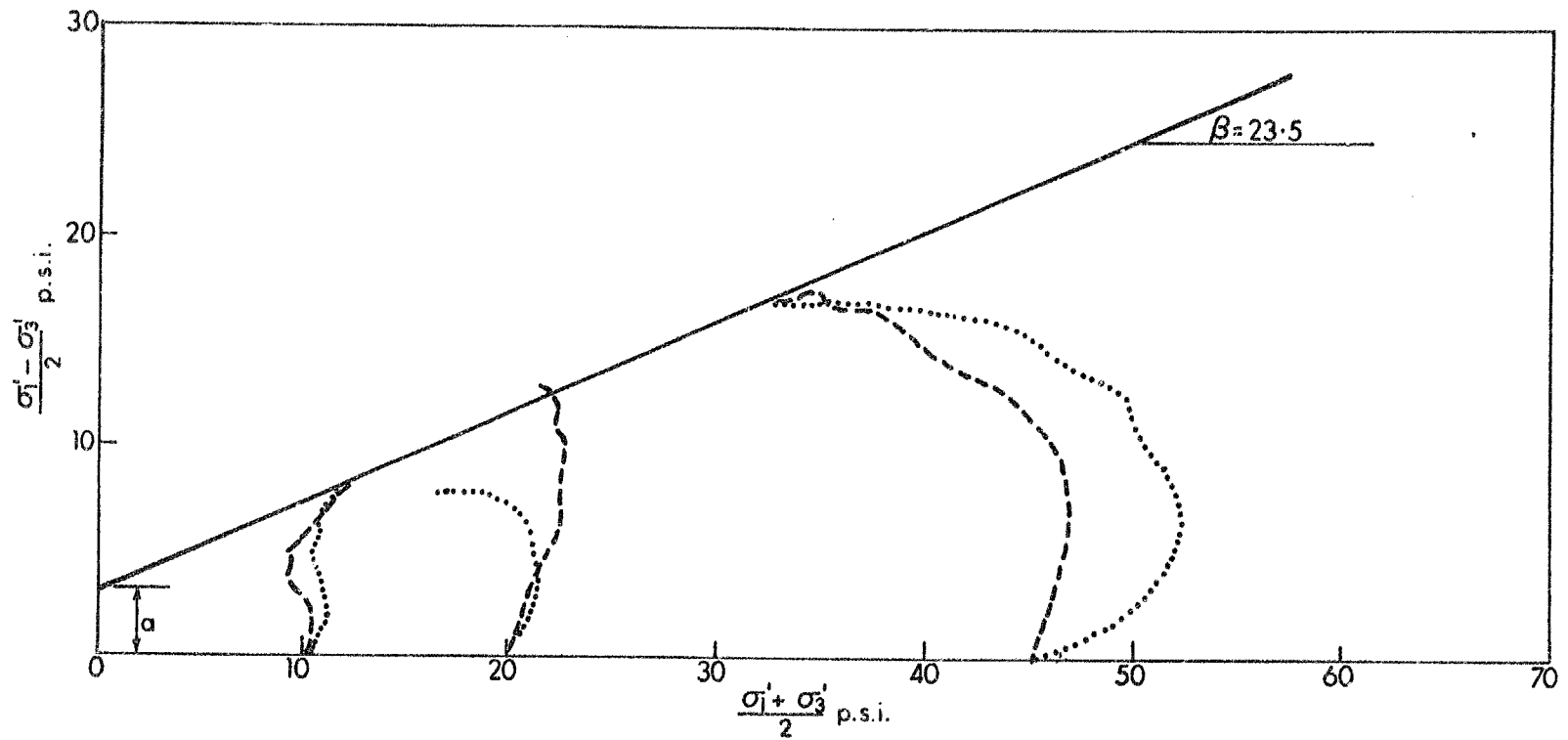


Ministry of  
Transportation and  
Communications

# PLASTICITY CHART CLAYEY SILT TO SILTY CLAY

FIG No 1

W P 199-62-00



LEGEND		
SYMBOL	B. H.	S. A.
---	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

# DIAMOND DRILL RECORD

HOLE NO. \_\_\_\_\_ SHEET NO. \_\_\_\_\_

DIP

PROPERTY \_\_\_\_\_ W. P. 199-62-00  
LOCATION \_\_\_\_\_ Arnprior By-pass Hwy. 17  
\_\_\_\_\_ C. P. R. Crossing  
\_\_\_\_\_ Station 803+00  
LATITUDE \_\_\_\_\_  
DEPARTURE \_\_\_\_\_  
BEARING \_\_\_\_\_

90°  
\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_  
TOTAL FOOTAGE \_\_\_\_\_

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

FOOTAGE		FORMATION	SAMPLE NUMBER			REMARKS
FROM	TO					
		Hole #2				Joints
14.5'	23.6'	Crystalline limestone, white grey colour, coarse texture, bedding appears to be horizontal, hard to medium hard				90° at 15.5' open, core is moderately fractured throughout
		Hole #3				Joints
18.3'	23.4'	Crystalline limestone, white grey colour, coarse texture, bedding appears to be horizontal, hard to medium hard				55° at 20.0' open, 60° at 23.0' open, core is moderately fractured throughout
		Hole #4				Joints
23.9'	34.4'	Limestone boulder 23.9'-24.3', crystalline limestone, white grey colour, coarse texture, hard to medium hard, bedding appears to be horizontal, mica and sulphide mineralization at 25.8'-31.0'				0° at 31.0' open, 80° at 32.0' open, broken ground core missing, core 24'-30' core is badly fractured throughout this section
		Hole #5				Joints
25.8'	36.9'	Crystalline limestone, white grey colour, coarse texture, bedding appears horizontal, hard to medium hard, mica and sulphide mineralization at 25.8'-26.3', rock appears leached 31.0'-36.0'				90° at 25.8' open 80° at 26.8' open 45° at 28.0' open 90° at 34.7' open 90° at 35.8' open
		All horizontal fracturing appears to have been caused by the drilling procedures				Weathered zones at 25.8'-26.3' 31.0'-36.0'

DATE OF EXAMINATION \_\_\_\_\_ Aug. 18/76

B. K. Glassford



WOLE NO. 6 SHEET NO.         

PROPERTY W. F. 199-62-00  
LOCATION Arnprior By-pass and C. P. R. X-ing  
Station 801+85 18' Rt. C  
LATITUDE \_\_\_\_\_  
DEPARTURE \_\_\_\_\_  
BEARING \_\_\_\_\_

90°

TOTAL FOOTAGE 24.0'

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

[illegible]

DATE OF EXAMINATION Aug. 13/76

B. K. Glassford

DB-MT-113



EXPERIMENTAL \_\_\_\_\_ DATE \_\_\_\_\_

GEOCRES No. 31F - 92

DIST 9 REGION EASTERN

W.P. No. 199 - 62 - 00

CONT. No. 79 - 17

W. O. No. \_\_\_\_\_

STR. SITE No. 29 - 200

HWY No. 17N

LOCATION CPR OVERHEAD

\_\_\_\_\_

\_\_\_\_\_

DETAILS OF THE RESEARCHER'S REPORT 4

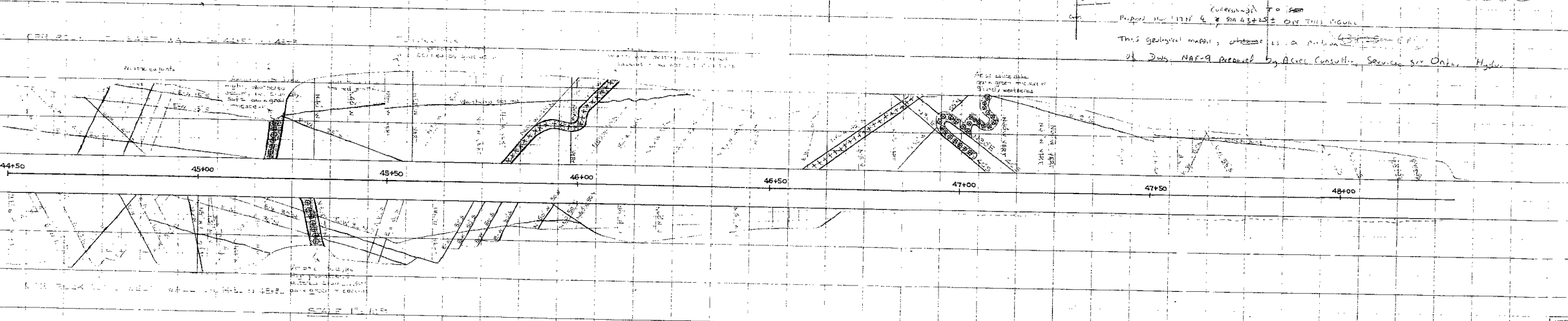
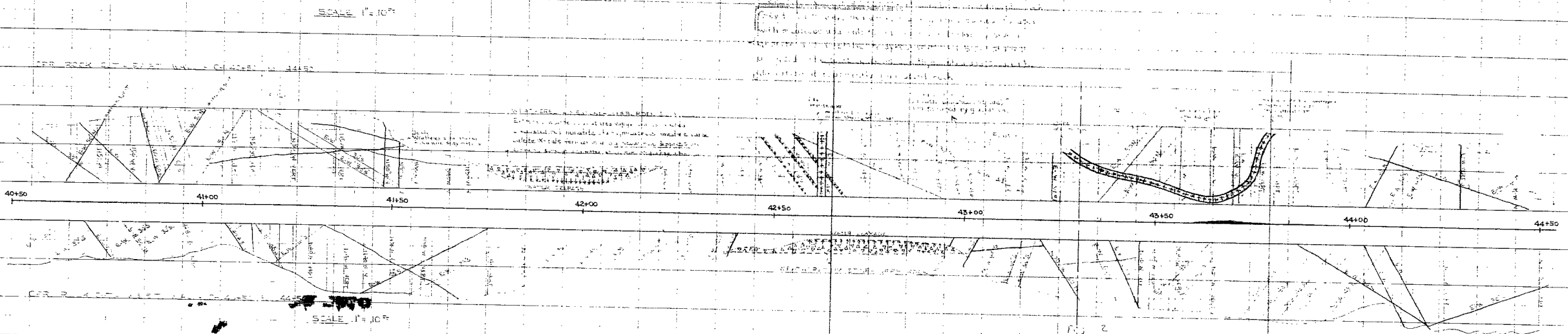
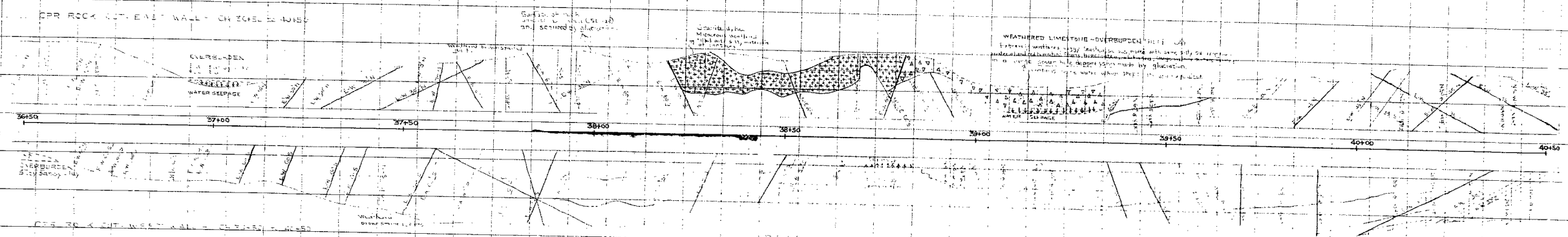
REMARKS \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

FOR USE BY \_\_\_\_\_



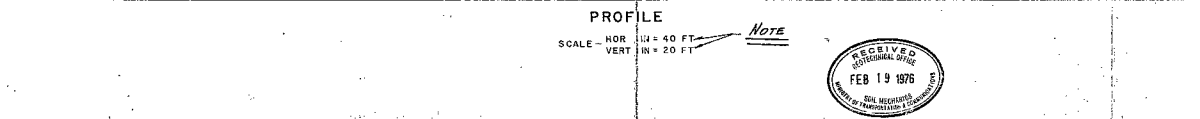
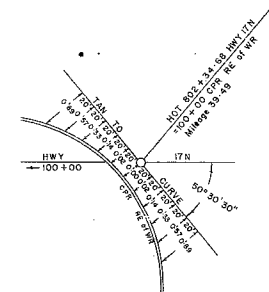
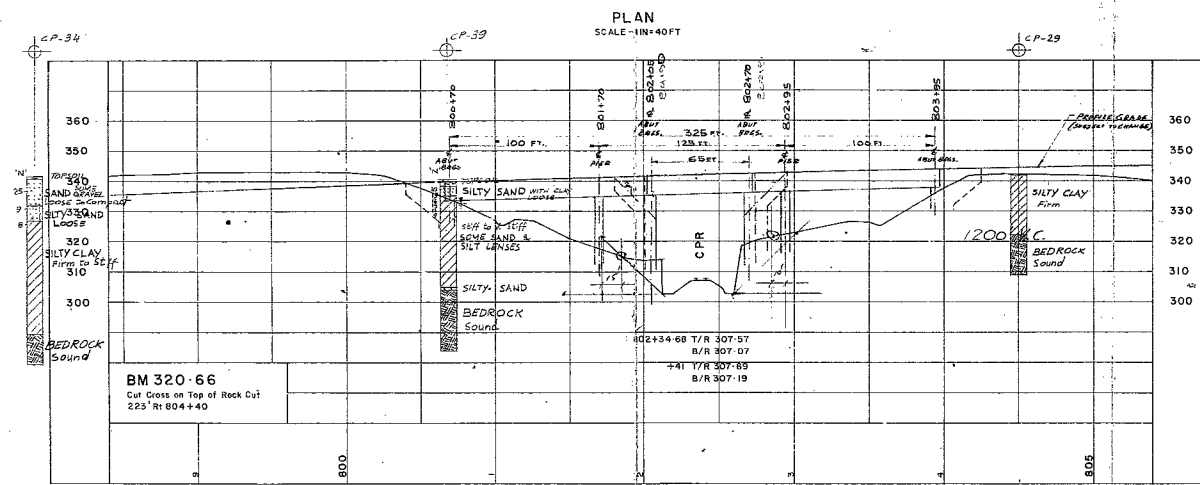
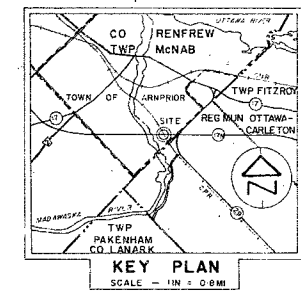
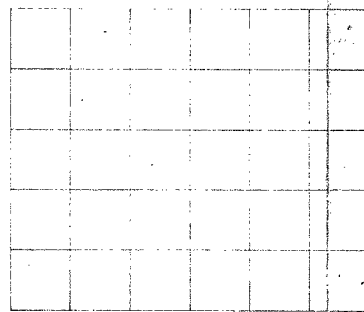
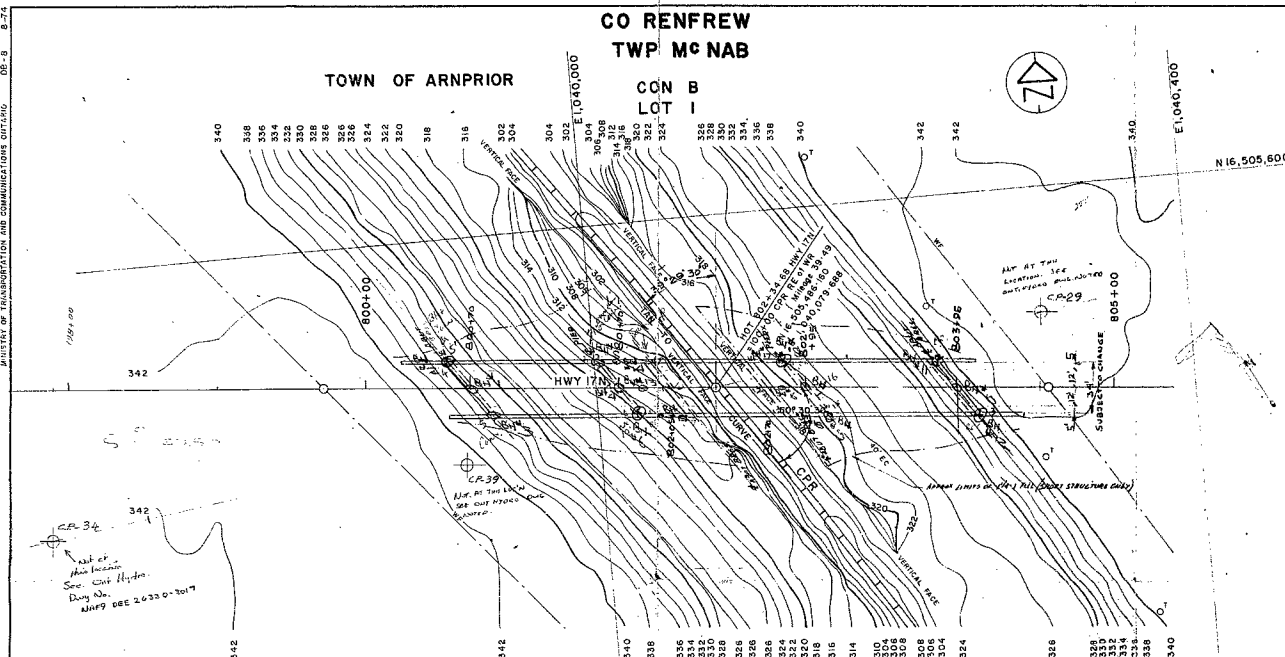
CONTRACT NO. 317-92  
 CRR ROCK CUT EAST WALL  
 GEOTECHNICAL MAPPING  
 BETWEEN CHANGES 31+00-48+00  
 317-92

Conte  
R. 11.5 m  
G. 11  
H. 11  
A. 11.5 m

Debarre 256-1474 Almonte

317-92

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO



RECEIVED  
FEB 19 1976

WP 197-62  
STR WP 199-62

DATE	REVISIONS & ADDITIONS	BY	CHK'D
FEB 17/76	SKETCH SHOWING PROPOSED LOCATION OF STRUCTURE AS SUBMITTED FOR FOUNDATION INVESTIGATIONS.		

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO  
ENGINEERING SERVICES BRANCH - SURVEYS & PLANS SECTION

BRIDGE SITE

PROPOSED CROSSING

AT

CANADIAN PACIFIC RAILWAY

MILEAGE 39-49 CHALK RIVER SUBDIVISION

AND THE

KING'S HIGHWAY 17N

LOT 1 TWP McNAB TOWN OF ARNPRIOR CO RENFREW

SCALE AS SHOWN DISTRICT OTTAWA REGION EASTERN

Survey - DEC 1975 Date of Plan - JAN 1976 SITE - 29-200

SURVEY BY  
Chief of Party - D BARR  
Supervisor - A GILES

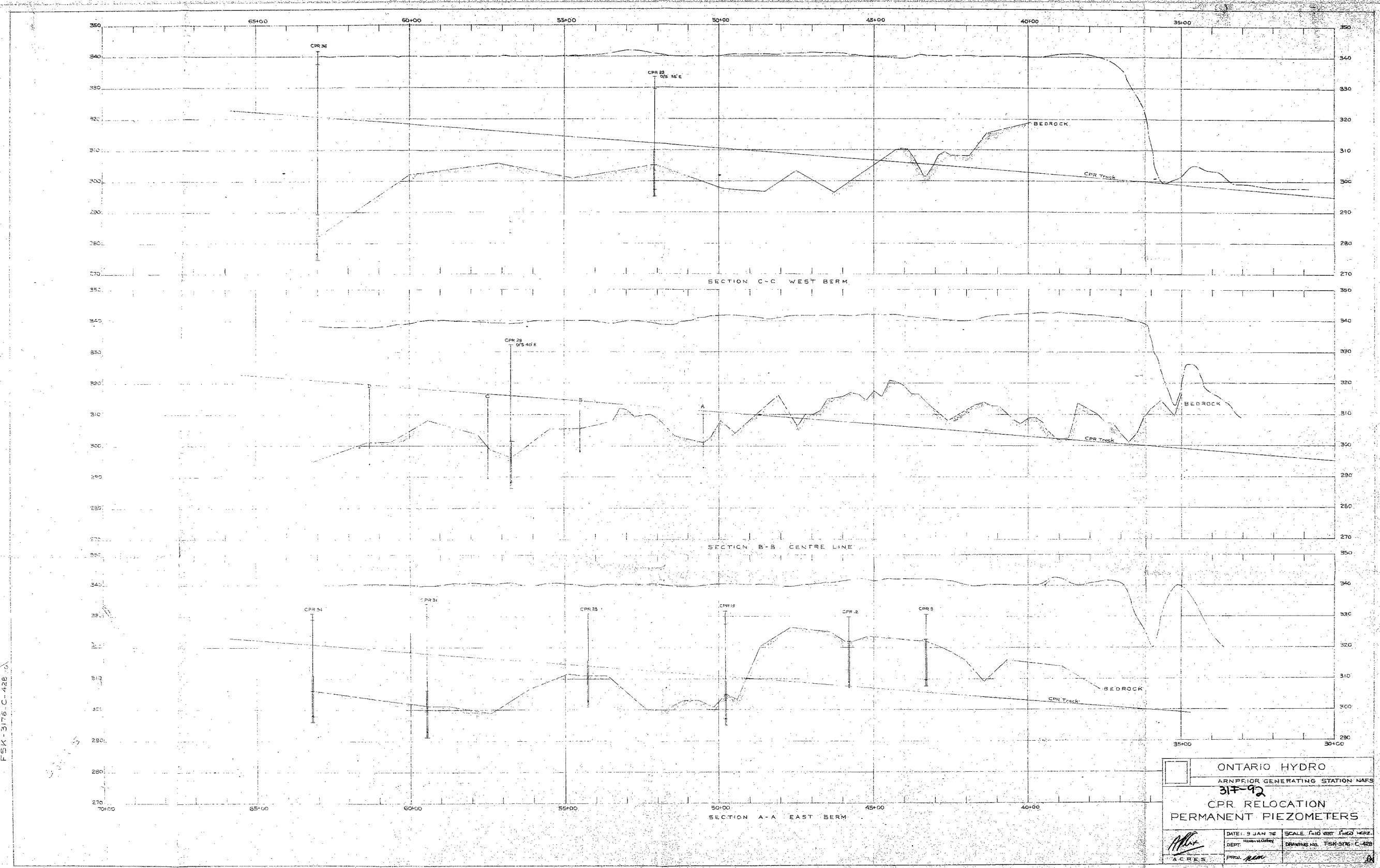
DRAWN BY  
Draftsman - J HAMER  
Supervisor - G LALANDE

CHECKED BY  
Draftsman - R RUBINO  
Supervisor - G LALANDE

PLAN E-5269-1

FEB 1976 SOILS MECHANICS SECTION

FSK-3176-C-428



ONTARIO HYDRO	
ARNHEIM GENERATING STATION NAES	
317-92	
CPR RELOCATION	
PERMANENT PIEZOMETERS	
DATE: 9 JAN 76	SCALE: HORIZ. 1"=100' VERT. 1"=10'
DEPT: HIGHWAY	DRAWING NO. FSK-3176-C-428
ACRES	PROJ. NAME

