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GEOCRES No. 31F-83

DIST 9 REGION Eastern

W.P. No. 10-67-02

CONT. No. 79-28

W. O. No. _____

STR. SITE No. 29-158

HWY. No. 17N

LOCATION Muskrat River
Bridge, 2.4 mi East of Hwy. 41

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 1

REMARKS: documents to be unfolded
before microfilming

FOUNDATION INVESTIGATION REPORT

for

W.P. 10-67-02

Site No. 29-158

Hwy. 17N District 9

Muskrat River Bridge

2.4 Miles East of Hwy. 41

1. INTRODUCTION

Soil Mechanics Section has been requested to carry out a foundation investigation at the site where proposed Hwy. 17N crosses Muskrat River at 2.4 miles east of Hwy. 41, Lot 21, Concession 1, Township of Stafford, in the County of Renfrew, District 9, Ottawa. The request was contained in a memorandum dated September 10, 1975, from Mr. T.C. Kingsland, Regional Structural Planning Engineer.

Subsequently, a foundation investigation was carried out at the proposed site to determine the subsoil conditions. This report contains the field and laboratory test results, together with recommendations pertaining to the bridge foundations and stability of approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY

The site is located about four miles south of the Town of Pembroke and 2.4 miles east of Hwy. 41.

Physiographically, the site is situated in the Ottawa Valley Clay Plains. Based on available geological information, it is known that the site lies in a section of a preglacial river valley excavated along an old fault line called the Muskrat Fault.

The area is wooded with cleared patches used for pastureland. About 400 feet on the southwest side of the site a rock outcrop, in the form of a knob, is exposed.

3. FIELD AND LABORATORY WORK

The fieldwork consisted of seven sampled boreholes accompanied by a dynamic cone penetration test adjacent to each borehole, three sampled

boreholes, and five dynamic cone penetration tests. The soil investigation was carried out by means of a bombardier mounted C.M.E. hollow stem auger machine adapted for soil sampling. In cohesive soils undisturbed samples were obtained at required depths by pushing 2 inch I.D. Shelby tubes manually or hydraulically. Disturbed samples were obtained by using a 2 inch O.D. split-spoon sampler driven according to the specifications of the Standard Penetration Test. In situ shear strength measurements were carried out, where possible, with a field vane. Bedrock was proven in four boreholes by obtaining BXL rock cores.

The locations and elevations of all borings were surveyed in the field by personnel from the Eastern Region, Engineering Surveys, and are shown on Drawing No. W.P. 106702 A & B, together with the estimated stratigraphical profile. All elevations in the report are referred to a geodetic datum.

Samples were visually examined and identified in the field and subsequently in the laboratory. Laboratory tests were conducted on selected representative samples to determine:

- Atterberg Limits
- Natural Moisture Content
- Bulk Density
- Grain Size Distribution
- Undrained Shear Strength
- Consolidation Characteristics
- Organic Content

Results of the laboratory and field tests, together with locations and elevations of the boreholes, are presented in the Appendix of this report.

4. SUBSOIL CONDITIONS

(4.1) General

The predominant subsoil at the site consists of a deposit of clayey silt to silty clay with silt seams, underlain by gneiss bedrock. At one location a surficial 4 ft. thick sandy silt layer was found to overlie the cohesive deposit. At two other locations on the east side of the river, a 1-2 ft. thick layer of silty sand

and gravel was found between the clayey silt to silty clay stratum and gneiss bedrock. The thickness of the cohesive deposit and its shear strength characteristics show considerable variation over the entire site. Depth to bedrock varies from 9 to 46 ft.

The boundaries between the various soil layers, as determined in the boreholes, are shown on the accompanying borehole log sheets. The stratigraphy shown on Drawing No. 106702 A & B, is inferred from this data.

(4.2) Sandy Silt

This material was found in Borehole 4 on the east side from ground surface to a depth of 4 ft. and consists of sandy silt with traces of clay. The relative density is estimated to be compact. This deposit appears to be local in nature.

(4.3) Clayey Silt to Silty Clay

(4.3.1) General

This is the predominant deposit occurring at the site. The thickness of this deposit varies from 9 to 45 ft. The ground slopes towards the river. On the east side the deposit is relatively thinner near the river and increases in thickness with distance away from the river. However, on the west side this pattern is reversed.

The material varies from clayey silt to silty clay. Occasional silt seams were encountered throughout the stratum. However, in Boreholes 11 and 12 on the east banks which were farthest away from the river, numerous silt seams were found in the upper 22 and 11 ft. respectively. The thickness of the silt seams ranges from $\frac{1}{2}$ inch to 6 inches.

The physical properties of this stratum are plotted on the Record of Borehole Sheets and on the Plasticity Chart (Fig. 1), and are summarized below:

Liquid Limit (W_L)	23 - 52 %
Plastic Limit (W_p)	15 - 23 %
Natural Moisture Content (W)	22 - 60 %
Liquidity Index (L_I)	0.4 - 2.2

These test results indicate that the deposit is mainly inorganic soil of low to intermediate plasticity.

The grain-size distributions are shown on Fig. 2.

(4.3.2) Shear Strength Characteristics

The undrained shear strength of the deposit was determined by means of field vane, unconfined compression and quick triaxial tests. The results show that, in general, the shear strength is lower in the immediate vicinity of the river water and is higher away from the river. Furthermore, in general, the shear strength on the west bank is greater than that on the east bank. The deposit is thicker on the east bank (25 to 45 ft.) than on the west bank (9 to 14 ft.).

East Bank

The lowest shear strength values were recorded in Boreholes No. 1 and 2 which were close to the river water on the east bank. In these boreholes, the shear strength generally ranged between 360 and 480 p.s.f., with some exceptions where it was higher (680 to 1440 p.s.f.), due to the presence of silt seams. The average shear strength in these boreholes was about 400 p.s.f.

In three other boreholes on the east bank, i.e. Boreholes No. 4, 12 and 13, the shear strength generally varied between 500 and 900 p.s.f. Boreholes No. 1, 2 and 13 were all located near the river, but the shear strength in B.H. No. 13 was found to be significantly higher than in B.H's 1 and 2. Boreholes No. 4 and 12 were farther away from the river. Borehole No. 12 showed numerous silt seams in the upper 11 ft. of the stratum.

In Borehole No. 11 on the east bank, which was farthest from the river, the upper 22 ft. showed numerous silt seams. In view of the presence of numerous silt seams, no in situ or laboratory shear strength tests were carried out in this zone. In the lower portion, the field vane

indicated that the shear strength varies between 800 - 1200 p.s.f.

West Bank

On the west bank, thickness of the overburden was between 9 and 14 ft. Only two field vane tests were carried out in one borehole, i.e. Borehole No. 6. These indicated shear strengths of 480 and 640 p.s.f. The following is inferred from a comparison of dynamic cone penetration tests in Boreholes No. 6A, 7 and 7A with the dynamic cone penetration test in Borehole No. 6 where shear strength measurements were taken:

- (i) In Borehole No. 7 the consistency of the deposit is similar to Borehole No. 6, i.e. soft to firm throughout its depth.
- (ii) In Boreholes No. 6A and 7A, which are farther from the river, the upper 5-6 ft. is soft and thereafter it is stiff in consistency.

(4.3.3) Consolidation Characteristics

The consolidation characteristics of the stratum were determined by carrying out two laboratory consolidation tests on samples from Boreholes No. 1 and 4. The results are shown as void ratio (e) versus logarithm of pressure (log p) curves on Figure 3 in the Appendix. The results show that in these boreholes the deposit is somewhat over-consolidated by about 0.5 - 1.0 ton/sq. ft. above the existing overburden pressure in these boreholes, as determined from Casagrande's method of estimating pre-consolidation.

(4.4) Silty Sand and Gravel

This 1-2 ft. thick layer was intersected in Boreholes No. 11 and 12 on the east bank away from the river. This layer immediately overlies gneiss bedrock and consists of compact silty sand and gravel.

(4.5) Bedrock - Metamorphic Gneiss

The bedrock profile was established by coring BXI size rock samples in Boreholes No. 1, 2, 6 and 9. The bedrock is identified as sound gneiss, grey pink in colour and of medium texture. In the remaining eight holes the bedrock was assumed by augering down or driving the cone down to refusal. A complete description of the bedrock details as intersected in each borehole was presented in the geologist's report attached to the Appendix of this report.

The estimated bedrock surface elevation at the site, as determined from the boreholes, range from elevation 409.8 (Borehole No. 10) to elevation 370.3 (Borehole No. 4). The bedrock surface, in general, slopes in an easterly direction.

5. GROUNDWATER

Water level observations were carried out in the open boreholes during the period of subsoil investigation. Results of the investigation show the water level in boreholes No. 1 to 7 is at about elevation 403.5 which corresponded to water level in the Muskrat Creek during the time of investigations. In boreholes No. 8 to 10 the groundwater contact was at the bedrock surface elevations, i.e. 405, 403, 410.5 ft. respectively.

6. DISCUSSION AND RECOMMENDATIONS

(6.1) General

It is proposed to construct a structure to carry new Hwy. 17N, Line 'A', over Muskrat River. The present proposal calls for a 3 span (60' - 100' - 60') structure with a skew of 45°. The grade of the proposed Hwy. 17 in this area will be at elevation 423, which is about 28 ft. above the river bottom. For stability purposes it will be necessary to add another span at the east end of the bridge or extend the east span. A detailed discussion of this aspect is contained later in the report.

The predominant soil type at the site consists of a deposit of clayey silt to silty clay which overlies gneiss bedrock. The consistency of the cohesive deposit varies from soft to hard depending upon the proximity to the river and location, whether east or west of the river.

(6.2) Approach Embankments

The proposed grade is at elevation 423, i.e., about 28 ft. above the river bed. The maximum height of embankment above the prevailing ground level will be about 17 ft. on the east side at Sta. 285 + 58, and about 14 ft. on the west side at Sta. 283 + 38. The Structural Planning Office proposed that the abutments be placed at the above Stations.

East banks of the river have a very gentle slope of 10 horizontal to 1 vertical, and the west banks slope at 5 horizontal to 1 vertical. During the investigation it was found that the undrained shear strength of the cohesive overburden on the east side of the river is, in general, lower than that on the west side.

As mentioned earlier, the following trends regarding the variation in undrained shear strength of the clayey silt to silty clay deposit were observed. In general:

- (a) the undrained shear strength of the deposit is lower (360 - 1440 p.s.f.) on the east side than on the west side (480 - > 2000 p.s.f.)
- (b) the undrained shear strength of the deposit decreases towards the river on both approaches.

At some places on the east side, the average undrained shear strength of the deposit was in the order of 400 p.s.f. Furthermore, thickness of the cohesive stratum on the east side is much greater than on the west side.

(6.2.1) Stability Considerations

The combination of a lower shear strength and thicker deposit on the east side presents stability problems for the approach embankments.

Stability analyses in terms of total stresses were carried out to investigate the stability of the proposed embankments.

(a) East Approach

The stability analyses revealed that if the toe of the east approach embankment with 2:1 forward slopes is located at approximate Sta. 285 + 25 (i.e. abutment at Sta. 285 + 58) as suggested on the E Plan submitted, this approach will be unstable.

The stability will be most critical in a direction perpendicular to the direction of the river as shown on Fig. No. 4, Section 1-1. This is the case because the bridge is skewed at an angle of 45° . In other directions the stability will be less critical. In order to provide a stable section a berm will be necessary along the Section 1-1. The properties and the geometry used for the stability analyses are shown on Fig. No. 4. Since the area along Section 1-1 is a localized one and in order to minimize the length of the additional bridge required for stability purposes, a lower factor of safety of 1.14 was chosen in this direction. This resulted in a geometry so that the toe of the forward slope of the east approach should be located at Sta. 285 + 67 (Fig. 4). It is also concluded that if the east approach is constructed in this manner, no stability problems incorporating 2:1 slopes are anticipated in the transverse direction.

(b) West Approach

The soft to firm consistency of the cohesive overburden will create unstable conditions for the proposed approaches at this location. In order to ensure stability both in the longitudinal and transverse directions, the following measures should be carried out.

- (1) The upper soft cohesive material should be excavated from ground elev. 410 as shown on Fig. 5. Elsewhere the excavation slopes should not be steeper than 1.5:1.
- (2) The material excavated should be backfilled with a granular type of material at least to 1 ft. above the river water level.

If the above details are adopted no stability problems are anticipated for the proposed west approach fills constructed with 2:1 slopes.

The above treatment may be eliminated if the west abutment is designed as a closed-end abutment founded on bedrock.

The approach fills should be protected against the scour action of the river; rip-rapping to a point above the anticipated high water level.

(6.2.2) Settlement Considerations

The underlying compressible cohesive sub-soil will undergo settlements due to consolidation over a period of time under the load imposed by the approach embankments. Settlements were calculated using consolidation curves obtained from laboratory testing. (Fig. 3)

The estimated settlements are as follows:

East Approach	4 - 6 inches.
West Approach	1 - 2 inches.

The total amount of this predicted settlement should take place within a period of 12 to 15 months. It is estimated that 50% of the ultimate settlements will occur within the first four to six months of loading. Since the predicted settlements will occur relatively quickly, it would be advantageous to place the fills prior to construction of the structure, in order to minimize post construction maintenance. If scheduling permits, a period of at least six months should be provided for this purpose. In any event final paving should be delayed as long as possible.

(6.3) Foundations

(6.3.1) General

The clayey silt to silty clay deposit, in general, has a soft to firm consistency, except at the west abutment location. It is not suitable for spread footing type foundations because of its relatively low shear strength and compressive nature. Therefore, it is recommended that the entire structure be founded on bedrock.

All footings and pile caps should have a minimum of 5 ft. of frost cover to their underneath.

(6.3.2) East Abutment

As mentioned earlier it will be necessary to provide an additional span at the east end of the bridge or extend the east span because of the stability considerations. The toe of the forward slopes should be located at Sta. 285 + 67 at centreline of the Hwy. as shown on Fig. 4. The east abutment may be supported on end-bearing steel-piles driven to bedrock. Maximum allowable load for the particular section chosen may be used for design purposes. No major dewatering problems are anticipated for pouring pile caps. The infiltration of water into the excavation can be handled by pumping.

(6.3.3) East Pier

The present proposal calls for the east pier to be located at Sta. 284 + 98. This pier may be supported on end-bearing steel piles driven to bedrock. Piles should be designed for the maximum allowable load for the particular section chosen. If the pier is constructed at Sta. 284 + 98, the excavation for the pier cap will extend into the river and it will be necessary to drive sheeting to keep water out of the excavation.

Alternatively, the pier may be supported on caissons as outlined in sub-section 6.3.6.

It is possible that a pier may be provided at Sta. 285 + 58, where originally an abutment was proposed, in order to accommodate an additional span at the east end of the bridge. In this case, the above recommendations will apply for this additional pier also.

(6.3.4) West Pier (Sta. 283 + 98)

The west pier is located inside the river water. Here the bedrock is about 10 ft. below ground level, and 12 - 14 ft. below the river water level. It is recommended that the west pier be supported on spread footing type foundations placed on bedrock. An allowable bearing capacity of 20 tons/sq. ft. may be used

for design purposes. A dewatering scheme will be necessary to pour concrete in the dry. Alternatively, tremie concrete may be used.

The nearest boreholes were away from the proposed footing location. It will be necessary to carry out additional field work to establish bedrock elevation precisely, once the footing location is finalized.

If the additional investigation reveals that bedrock at the pier location is at significantly lower elevation than in the nearest boreholes, then the pier may be supported on short steel piles. A dewatering scheme will be required in order to pour concrete for the pile caps which will be under the river water. The dewatering may be achieved by means of a cofferdam.

As another alternative, the pier may be supported on caissons as outlined in sub-section 6.3.6.

(6.3.5) West Abutment (Sta. 283 + 38)

At the west abutment location the depth to bedrock is only 9 to 13 ft. Therefore, it is recommended that it should be supported on spread footing type foundations placed on bedrock. An allowable bearing capacity of 20 tons/sq. ft. may be used for design. Infiltration of water in the excavation can be handled by a pumping method.

If the abutment is designed as a closed-end abutment, then treatment for the west approach as contained in sub-section 6.2.1 (b) and Fig. 5 may be eliminated.

(6.3.6) Caisson Foundations

The entire structure, except the west abutment, may be supported on caissons. These caissons could be continued to the underside of bridge deck to act as columns. This would eliminate any dewatering required for pouring pile caps below water level. The caissons should be socketed at least 1 ft. inside bedrock. An allowable load carrying capacity of 50 tons per sq. ft. may be assumed for design purposes, e.g.

a 30 in. diameter caisson may be designed to carry 250 tons and a 36 in. diameter caisson, 350 tons.

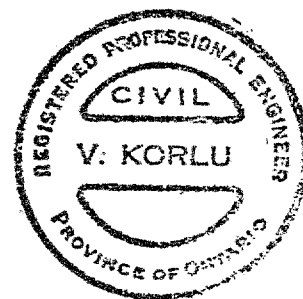
7. MISCELLANEOUS

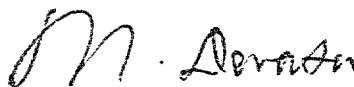
The field work for this project was carried out during September 29 to October 3, 1975, under the supervision of Mr. V. Korlu, Project Engineer, who also prepared this report, with the help of Mr. A. Prakash, Senior Engineer.

This report was reviewed by Mr. M. Devata, Supervising Engineer.

The drilling equipment was supplied and operated by Atcost Drilling Company of Toronto.


V. KORLU
Project Engineer




M. DEVATA
Supervising Engineer

January, 1976

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

WP 10-67-02

LOCATION Co-ords. 634,154 N; 850,548 E.

ORIGINATED BY VK

DIST 9 HWY 17N

BORING DATE September 30, 1975

COMPILED BY VR

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger, BXL Core, and Cone Test

CHECKED BY

[illegible]

20
15 ϕ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

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

RECORD OF BOREHOLE NO 2

WP 10-67-02 LOCATION Co-ords. 634,245 N; 850,515 E. ORIGINATED BY VK
DIST 9 HWY 17N BORING DATE September 29, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger, BXL Core, Cone Test CHECKED BY

SOIL PROFILE			SAMPLES			GRIND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT				LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L	
404.0	Ground Level														
0.0	Clayey silt to silty clay, occasional silt seams. Soft		1	TW	PM	400									
			2	TW	PM	390									
			3	TW	PM										
			4	SS	2										
			5	SS	2										
375.5															
28.5	Bedrock Gneiss		6	RC	100										
370.5															
33.5	End of Borehole					370									

RECORD OF BOREHOLE No 3

WP 10-67-02 LOCATION Co-ord. 634,196 N: 850,511 E. ORIGINATED BY VK
DIST 9 HWY 17N BORING DATE September 30, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Test CHECKED BY SB

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

15 ϕ 5 % STRAIN AT FAILURE

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

RECORD OF BOREHOLE NO 4

WP 10-67-02 LOCATION Co-ords. 634,134 N; 850,588 E. ORIGINATED BY VK
DIST 9 HWY 17N BORING DATE Oct. 1, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger and Cone Test 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 γ P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	VALUES		20	40	60	80	100	w_p	w	w_L		
410.3	Ground Level															
0.0	Sandy silt, trace of clay.		1	SS	17	410										
406.3	Loose to Compact		2	SS	6											0 0 88 12
4.0	Clayey silt to silty clay, occasional silt seams.		3	TW	PM										115	
			4	TW	PM											
	Soft to Firm		5	TW	PM	390									111	0 0 88 12 0 0 65 35
			6	TW	PM											
			7	TW	PM	380									120	0 2 73 25
370.3																
40.0	End of Borehole Probable Bedrock					370										Refusal to cone Hammer bouncing

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

RECORD OF BOREHOLE NO 5

WP 10-67-02 LOCATION Co-ords. 634,202 N; 850,551 E. ORIGINATED BY VK
DIST 9 HWY 17 N BORING DATE October 1, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Test 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 γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W_P	W	W_L		
405.2	Ground Level															
0.0	Probable clayey silt to silty clay, occasional silt seams Soft to Firm															
370.4																
34.8	End of Cone Test Probable Bedrock Refusal, hammer bouncing															

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

RECORD OF BOREHOLE NO 5

WP 10-67-02 LOCATION Co-ords. 634,294 N; 850,428 E. ORIGINATED BY VK
 DIST 9 HWY 17N BORING DATE October 2, 1975 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger, BXL Core & Cone Test CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	w_p	w	w_L		
403.5	Ground Level					ELEV										
0.0	Clayey silt to silty clay, occasional silt seams.		1	TW	PM	400									103	0 0 56 44
			2	TW	PM											
	Soft to Firm		3	SS	3											
389.8						390										
13.7	Bedrock Gneiss			RC												
385.2			4	BXL	100%											Refusal to cone hammer bouncing
18.3	End of Borehole					380										

20
15 ϕ 5 % STRAIN AT FAILURE
10

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

RECORD OF BOREHOLE NO 6A

WP 10-67-02 LOCATION Co-ords. 634,309 N; 850,410 E. ORIGINATED BY VK
DIST 9 HWY 17 N BORING DATE October 2, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Test CHECKED BY *10*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT —WL PLASTIC LIMIT —WP WATER CONTENT —W			UNIT WEIGHT Y	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	WP	W	WL		
405.0	Ground Level															
0.0	Probable clayey silt to silty clay, occasional silt seams															
393.5	Soft to Stiff															
11.5	End of Cone Test Refusal, hammer bouncing Probable Bedrock															

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

RECORD OF BOREHOLE NO 7

WP 10-67-02 LOCATION Co-ords. 634,337 N; 850,431 E. ORIGINATED BY VK
DIST 9 HWY 17N BORING DATE October 2, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Dynamic 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		UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		20	40	60	80	100	w_p	w		
404.0	Ground Level													
0.0	Probable clayey silt to silty clay, occasional silt seams				400									
	Soft to Firm													
390.5														
13.5	End of Cone Test Refusal, hammer bouncing Probable Bedrock				390									

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

RECORD OF BOREHOLE NO 7A

WP 10-67-02 LOCATION Co-ords. 634,351 N; 850,416 E. ORIGINATED BY VK
DIST 9 HWY 17N BORING DATE October 2, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Test 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 γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w		
405.5	Ground Level														
0.0	Probable clayey silt to silty clay, occasional silt seams					400									
396.8	Soft to Stiff														
8.7	End of Cone Test Refusal, hammer bouncing Probable Bedrock					390									

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

RECORD OF BOREHOLE NO 8

WP 10-67-02 LOCATION Co-ords. 634,329 N: 850,392 E. ORIGINATED BY VK
DIST 9 HWY 17N BORING DATE October 3, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger & 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			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	W_P	W	W_L		
414.0	Ground Level															
0.0	Clayey silt to silty clay, occasional silt seams		1	SS	14	410										
404.8	Stiff to Very Stiff		2	SS	21	400										0 0 73 27
9.2	End of Borehole Probable Bedrock															Refusal to cone hammer bouncing

SOIL PROFILE		SAMPLES		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 w_p — w — w_L WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER TYPE 'N' VALUES					
413.0	Ground Level							
0.0	Clayey silt to silty clay with sand and some gravel.		1 SS 12	410				
	Stiff to Hard		2 SS 69	400		8		18 40 38 4 9 73 (18) Refusal to cone hammer bouncing
400.5								
12.5	Bedrock Gneiss		3 RC EXL 100					
395.5								
17.5	End of Borehole			390				

15 $\frac{30}{0.5}$ % STRAIN AT FAILURE

RECORD OF BOREHOLE NO 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 γ	REMARKS
ELV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER		TYPE	'N' VALUES		
418.9	Ground Level							
0.0	Clayey silt to silty clay, occasional silt seams		1	SS	17			
409.8	Stiff to Very Stiff		2	SS	11			0 0 79 21 Refusal to hammer bouncing
9.1	End of Borehole Probable Bedrock							
					400			

15 ²⁰ - 5 % STRAIN AT FAILURE

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

RECORD OF BOREHOLE NO 11

WP 10-67-02 LOCATION Co-ords. 634,089 N: 850,627 E. ORIGINATED BY RWB
DIST 9 HWY 17N BORING DATE December 5, 1975 COMPILED BY RWB
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT				LIQUID LIMIT —WL PLASTIC LIMIT —WP WATER CONTENT —W			UNIT WEIGHT Y	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	100	WP	W	WL		
413.2	Ground Level														
0.0															
	Clayey silt to silty clay	Stiff Firm	1	SS	18	410									
			2	SS	7										
			3	TW	PH										
			4	TW	PH	400									
			5	TW	PH										
	numerous silt seams		6	TW	PH										
	occasional silt seams		7	TW	PH	390									
			8	TW	PH										
			9	TW	PH	380									
			10	TW	PH										
368.2						370									
366.4	Silty sand & gravel		11	SS	60/										
46.8	End of Borehole Probable Bedrock					360									

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

RECORD OF BOREHOLE NO 12

WP 10-67-02 LOCATION Co-ords. 634,094 N: 850,583 E. ORIGINATED BY RWB
DIST 9 HWY 17N BORING DATE December 6, 1975 COMPILED BY RWB
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY SO

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
410.1	Ground Level						SHEAR STRENGTH					WATER CONTENT %				
							○ UNCONFINED + FIELD VANE • QUICK TRIAXIAL x LAB VANE 400 800 1200 1600 2000									
0.7	Clayey silt to silty clay					410										
	numerous silt seams		1	VS	-	400										
	occasional silt seams		2	VS	-	390										
	Firm		3	VS	-	380										
374.2	Silty sand & gravel		4	SS	14	370										
372.6	Compact															
37.5	End of Borehole Probable Bedrock															WL not established

OFFICE REPORT ON SOIL EXPLORATION

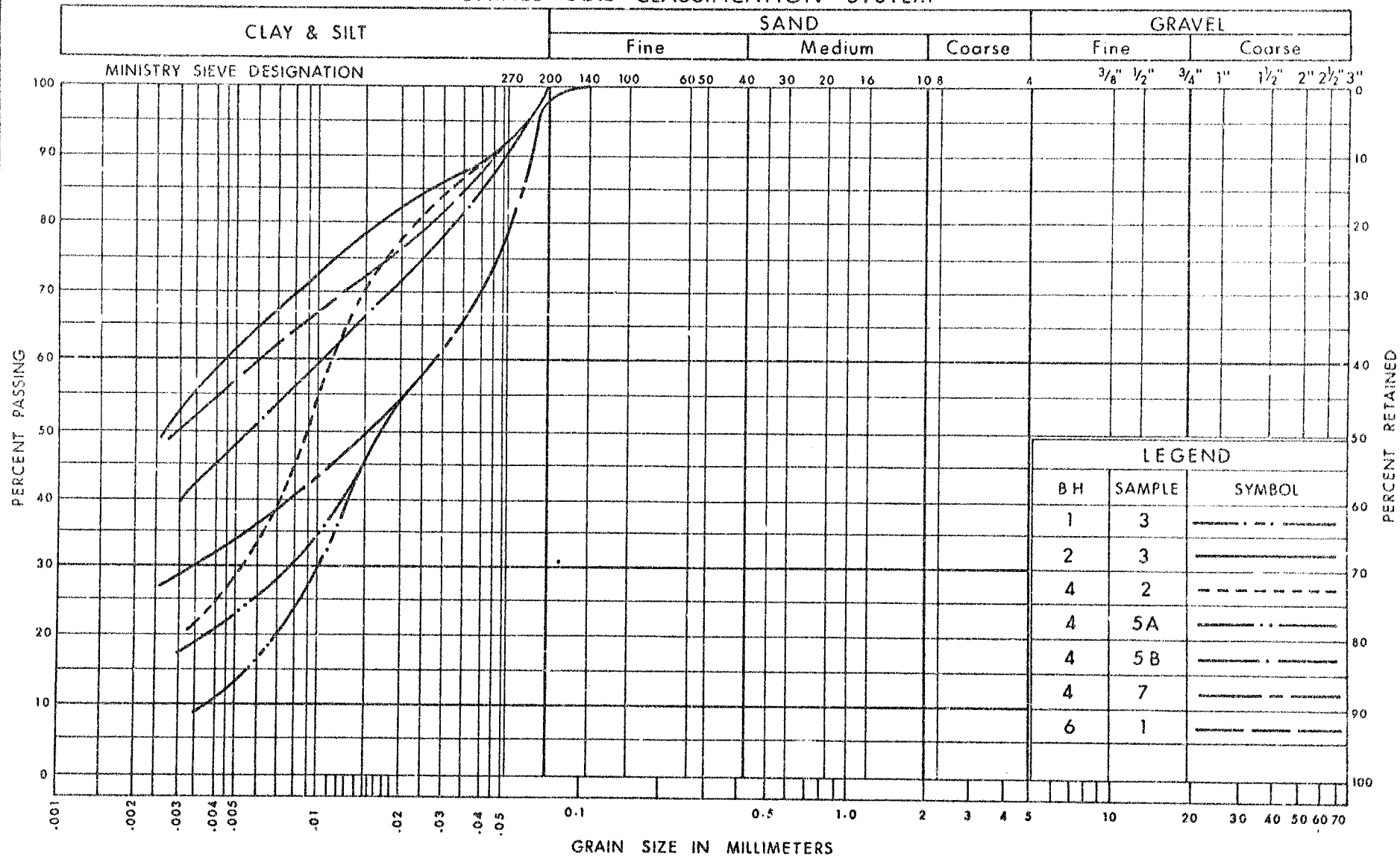
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 13

WP 10-67-02 LOCATION Co-ords. 634,121 N: 850,548 E. ORIGINATED BY RWB
DIST 9 HWY 17N BORING DATE December 5, 1975 COMPILED BY RWB
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger 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 γ P.C.T.	REMARKS % SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W_P	W	W_L		
405.7	Ground Level															
0.0	Clayey silt to silty clay, occasional silt seams		1	SS	3	490							o			
			2	TW	PH											
			3	VS	-											
			4	TW	PH											
			5	VS	-	390							o		118	
			6	TW	PH										102.5	
	Firm															
			7	TW	PH	380										
375.7																
30.0	End of Borehole Probable Bedrock					370										WL not established

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY

FIG No 2

W P 10-67-02

VOID RATIO - PRESSURE CURVES

W.P. NO. 10-67-02

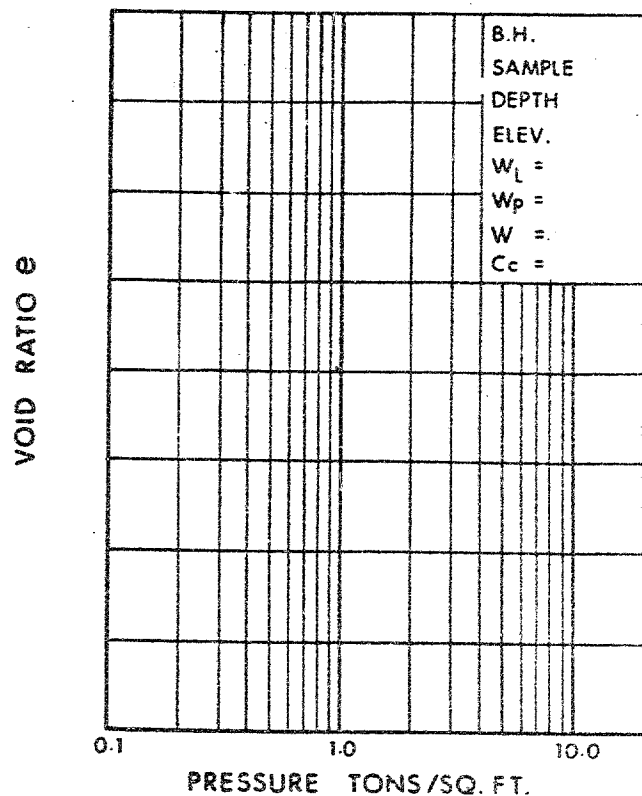
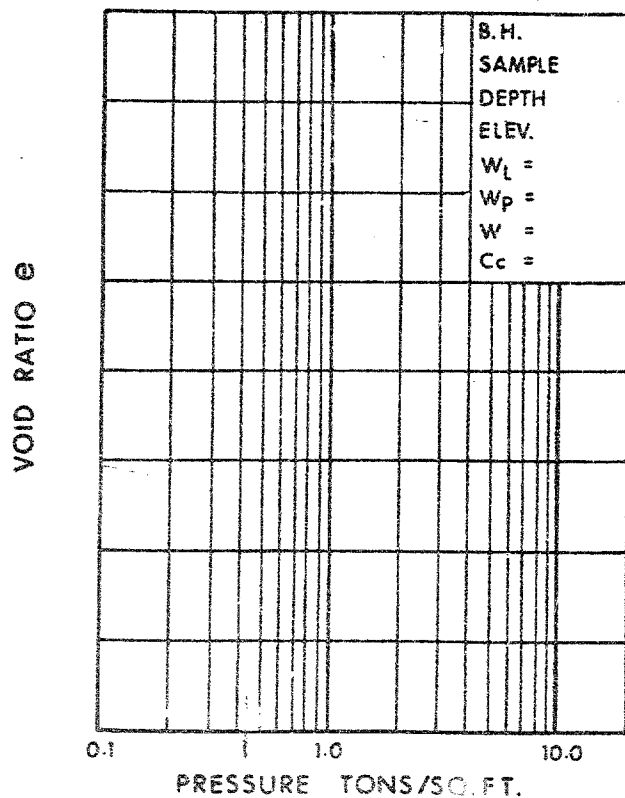
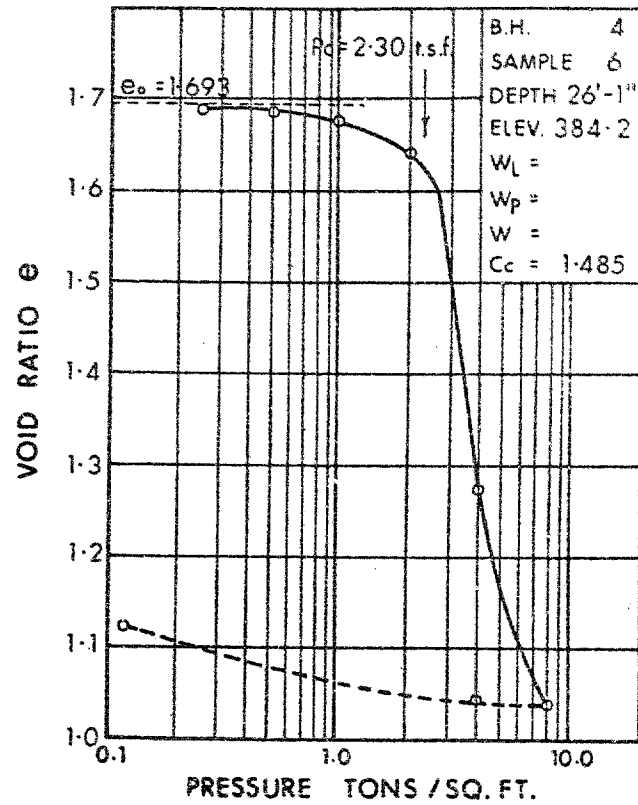
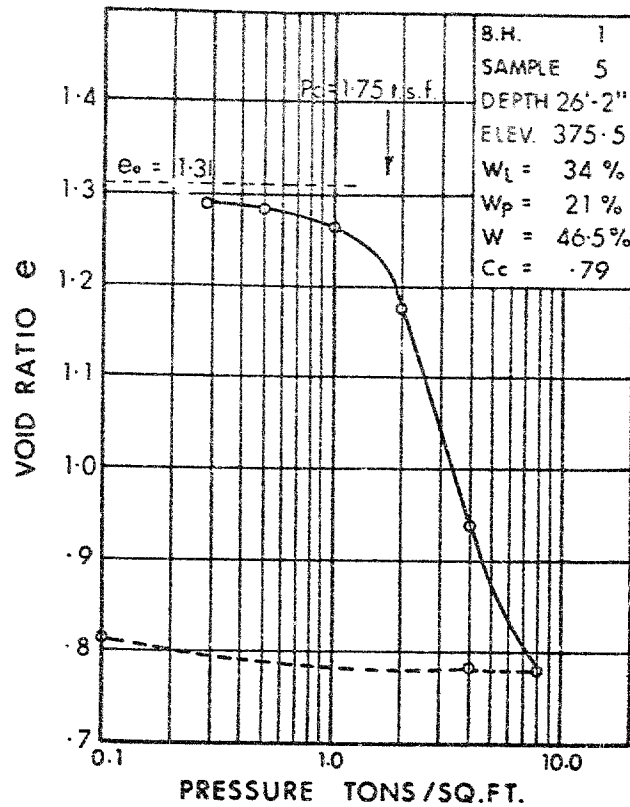
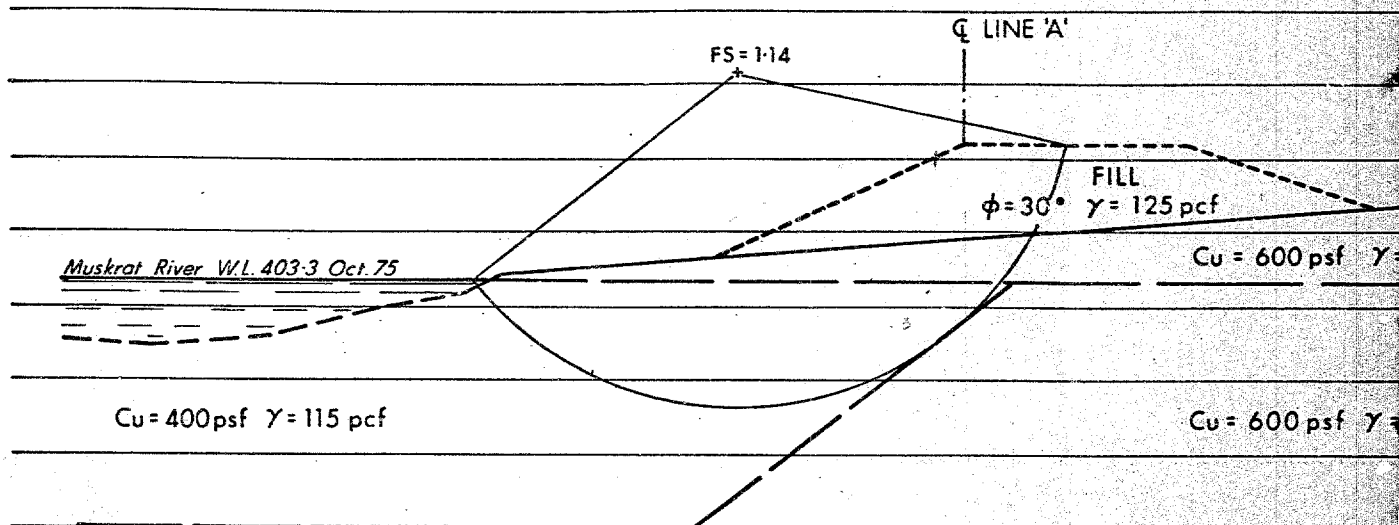
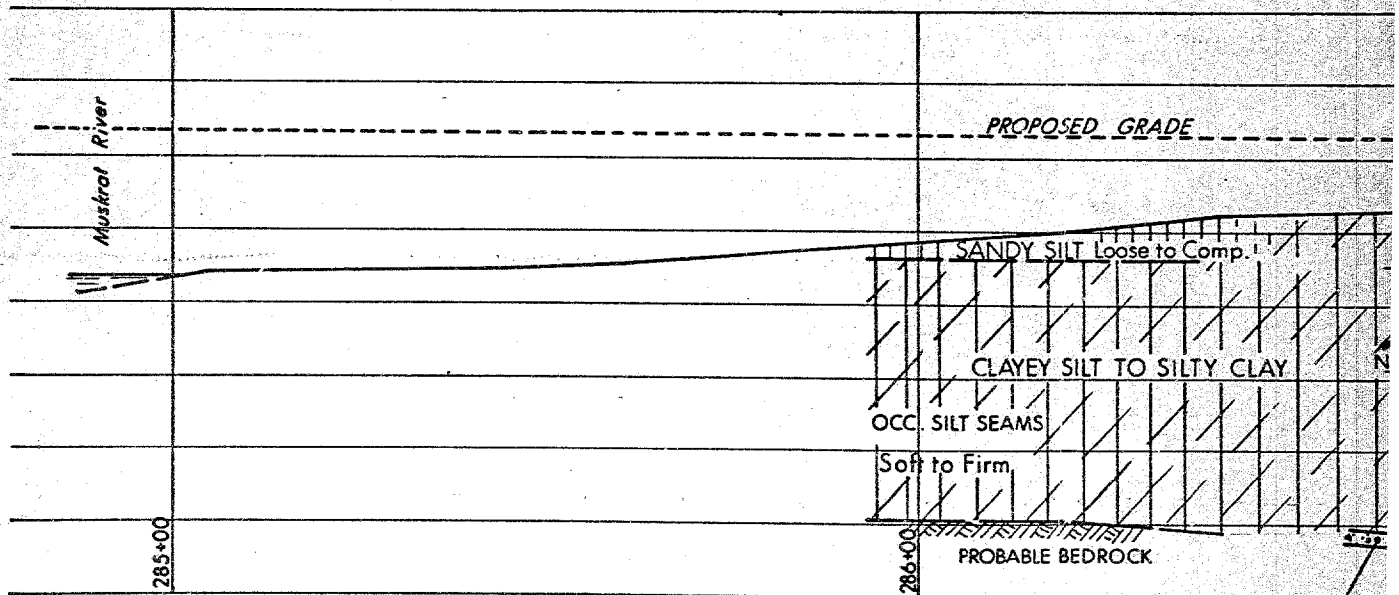


FIG. 3

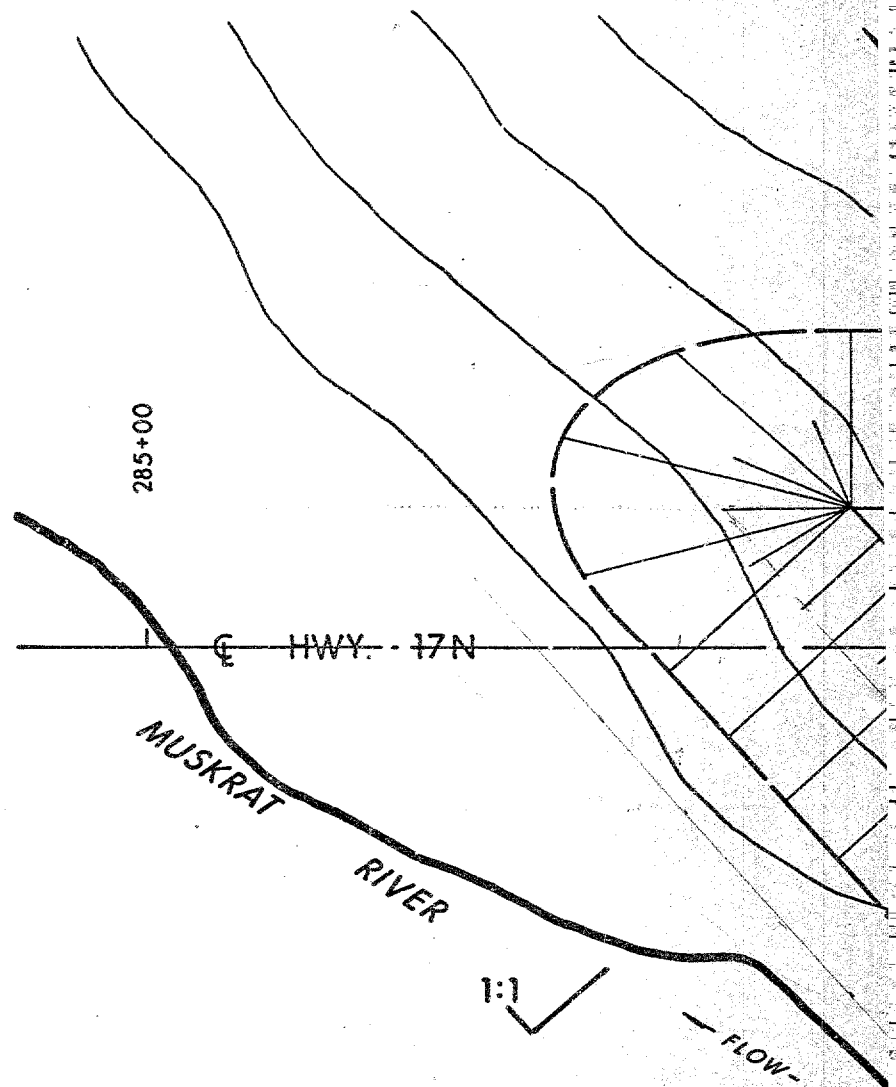
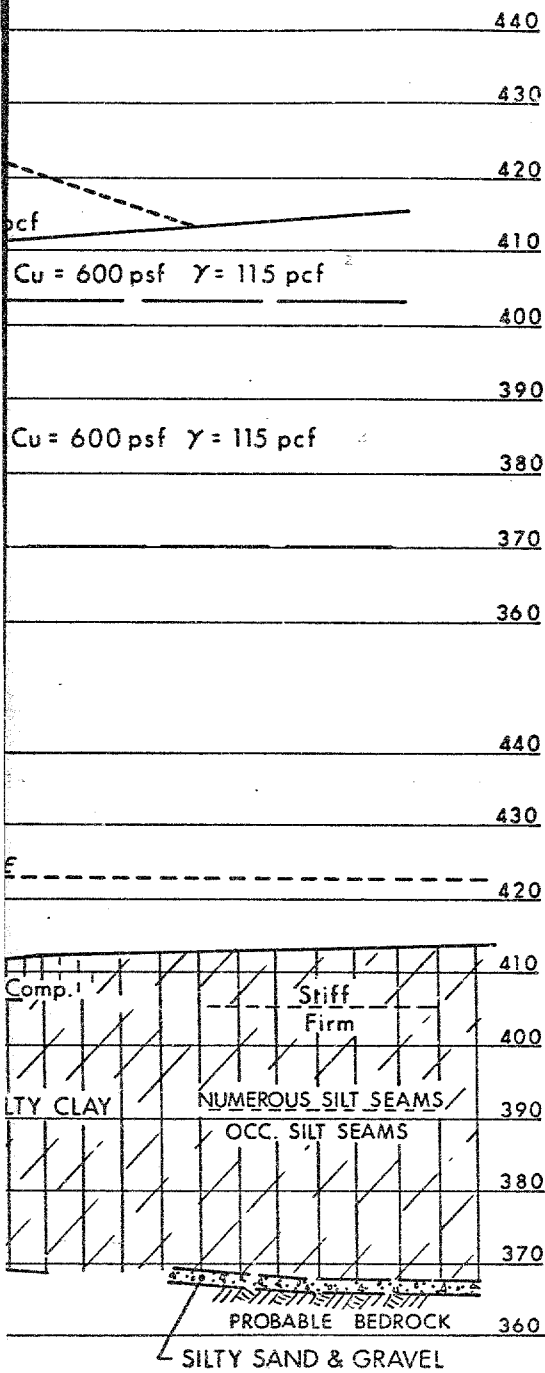


SECTION 1:1




Q PROFILE - LINE 'A'

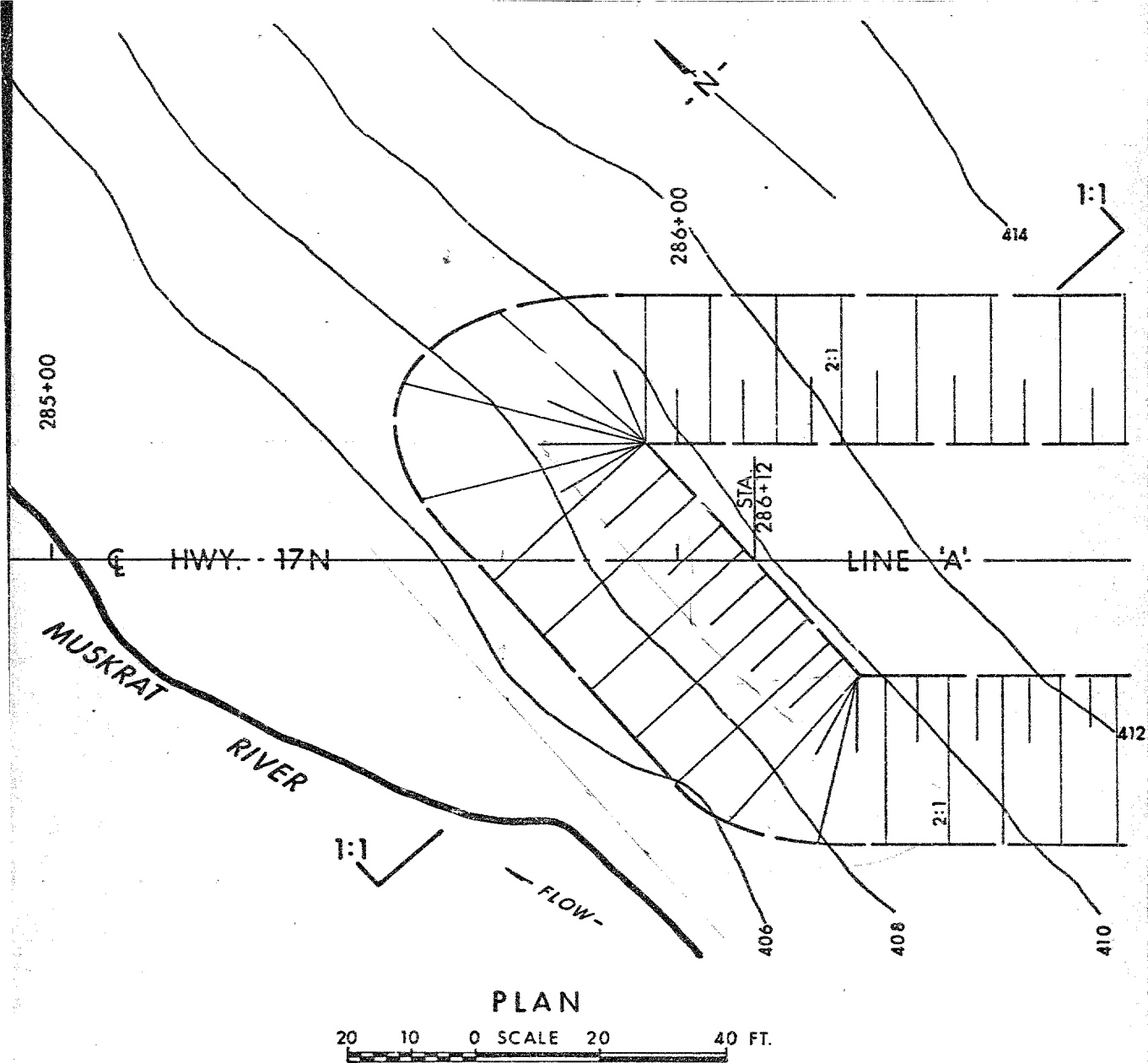




PLAN
20 10 0 SCALE 20

PROBABLE BEDROCK
SILTY SAND & GRAVEL

 Ministry of Transportation and Communications ENGINEERING SERVICES BRANCH	STABIL
	DATE 8 JAN. 1976 WP 10-6



Ministry of
Transportation and
Communications

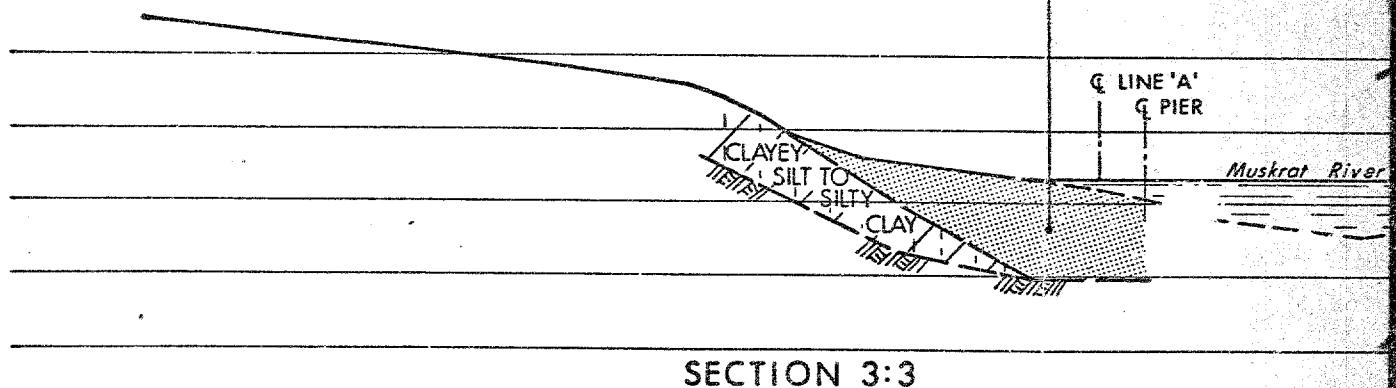
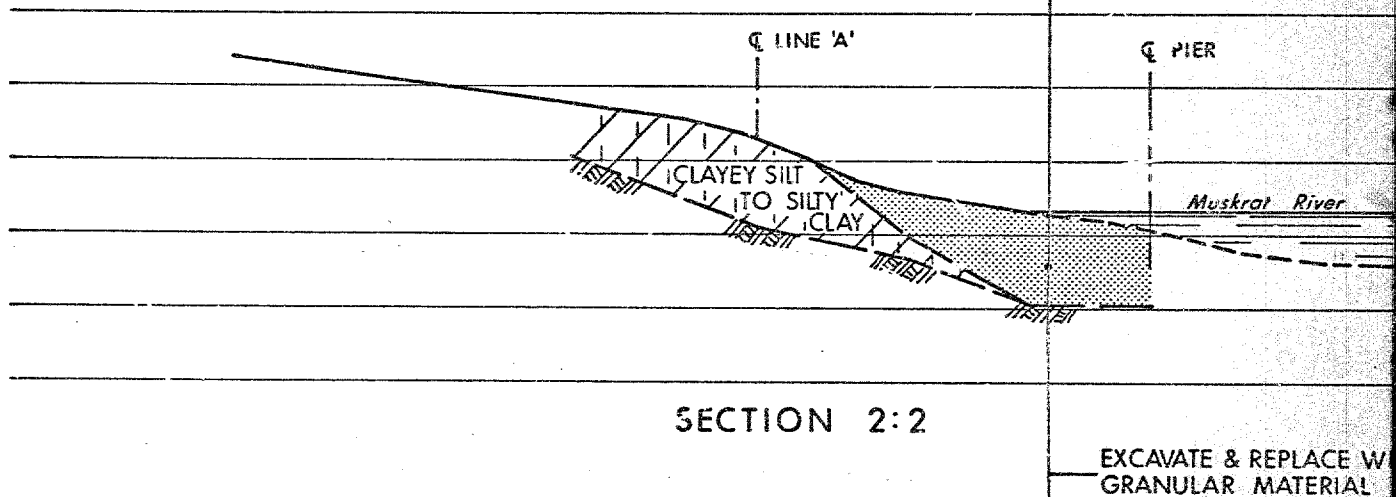
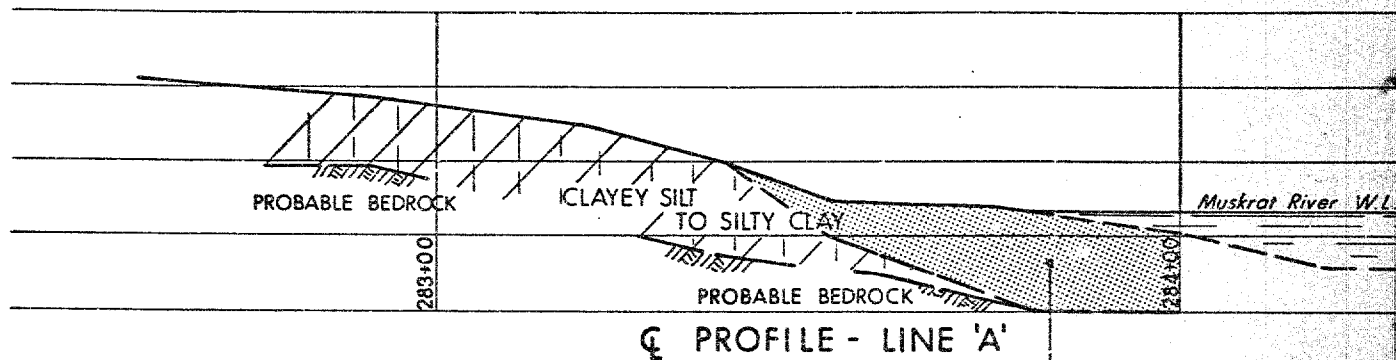
ENGINEERING SERVICES BRANCH

STABILITY FOR EAST APPROACH

DATE 8 JAN. 1976

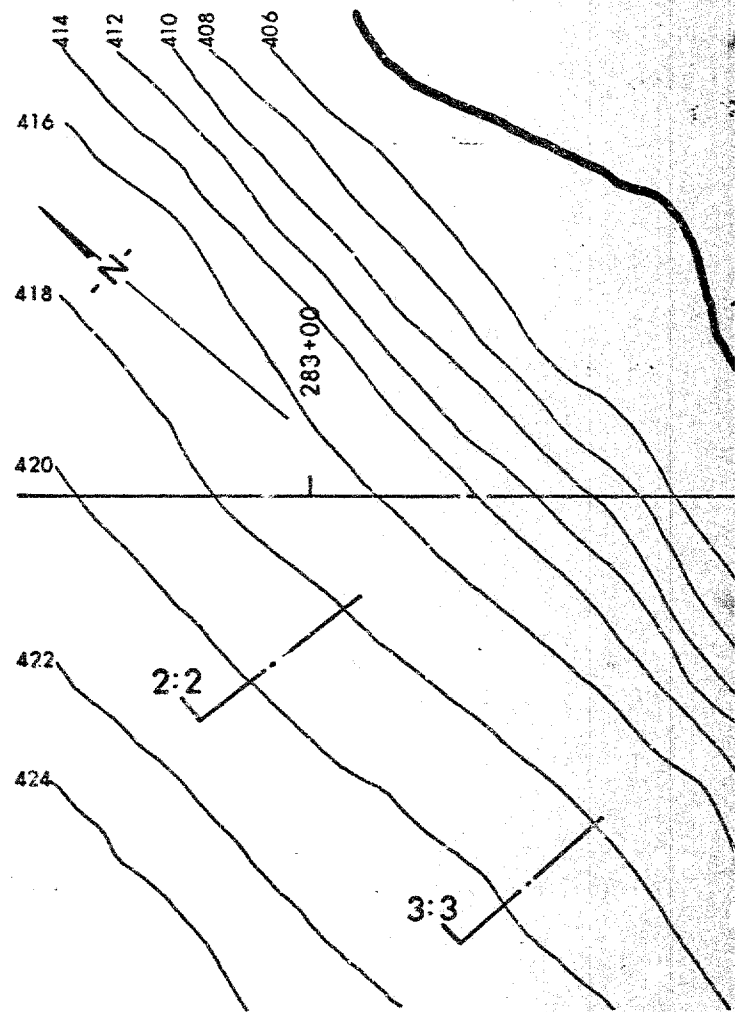
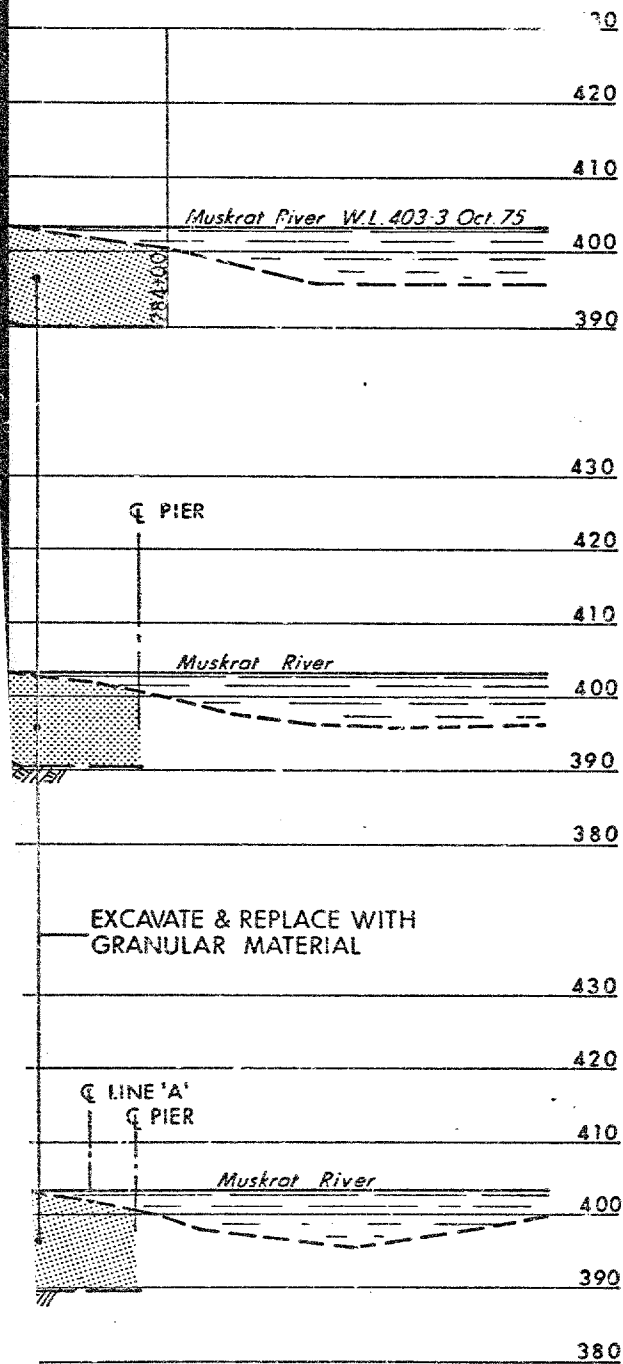
WP 10-67-02

FIG No 4



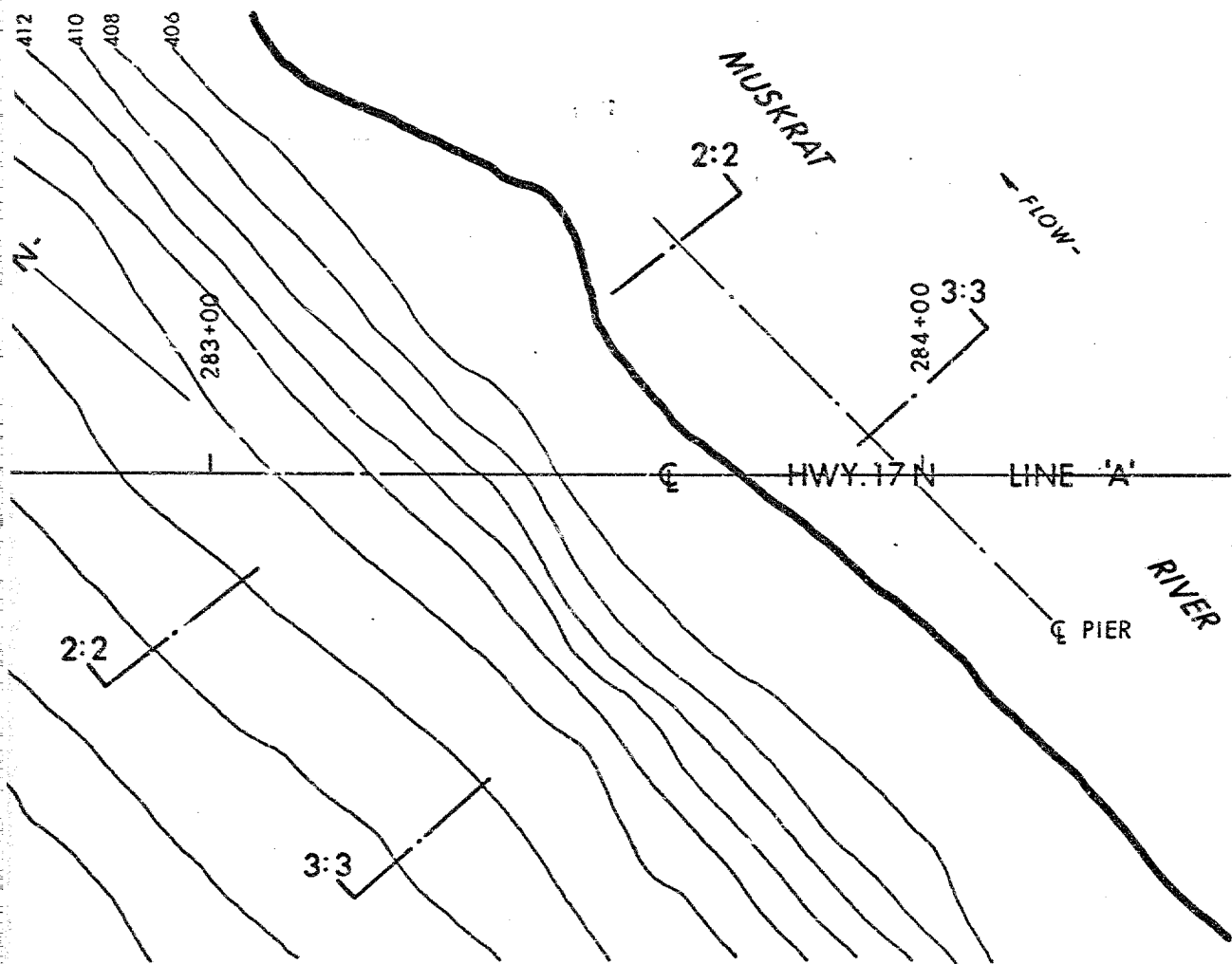
20 10 0 SCALE 20 40 FT.

A horizontal scale bar with markings at 20, 10, 0, 20, and 40 feet. The bar is divided into segments corresponding to these measurements.



20 10 0 SCALE

	Ministry of Transportation and Communications
	ENGINEERING SERVICES BRANCH
	DATE 8 JAN. 1976



PLAN

20 10 0 SCALE 20 40 FT.



Ministry of
Transportation and
Communications

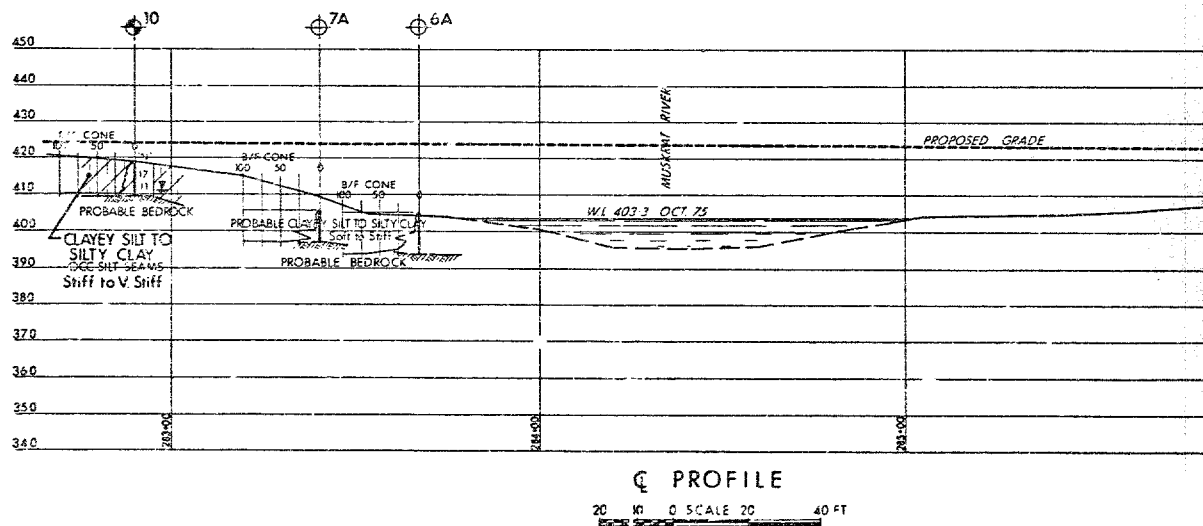
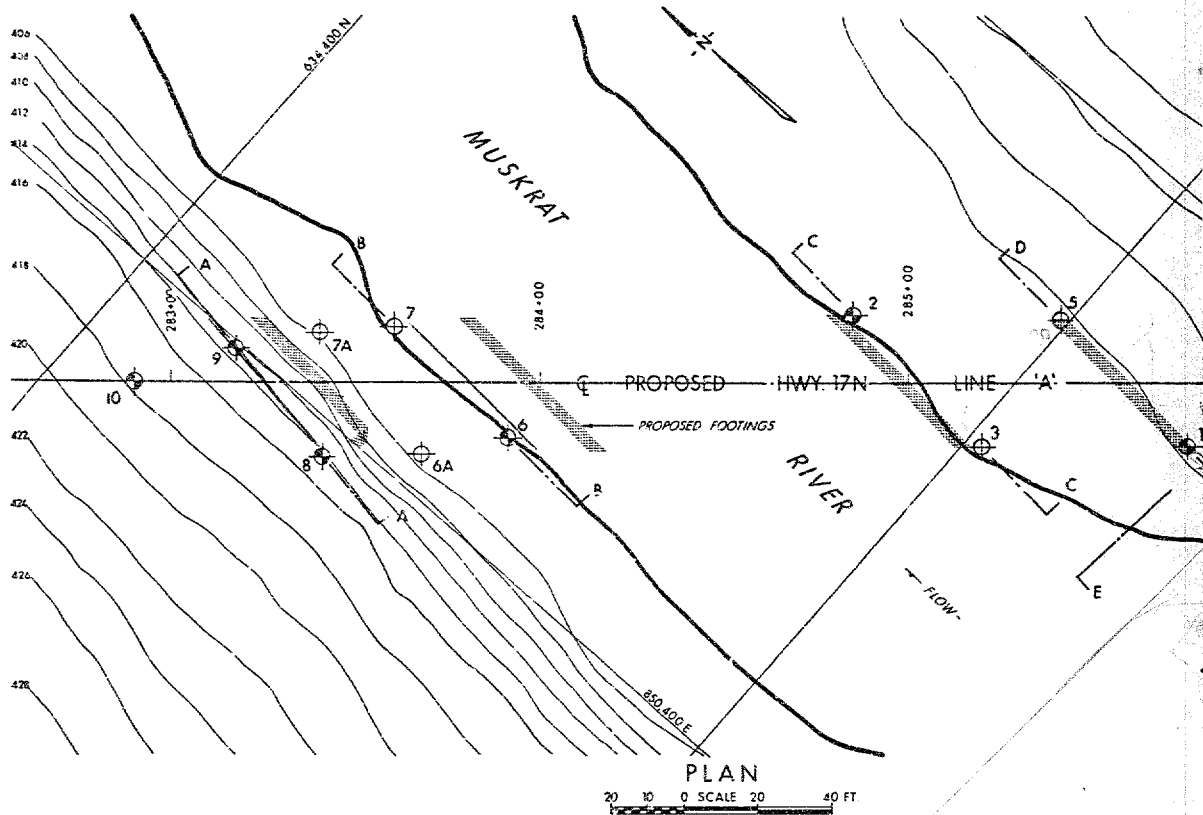
ENGINEERING SERVICES BRANCH

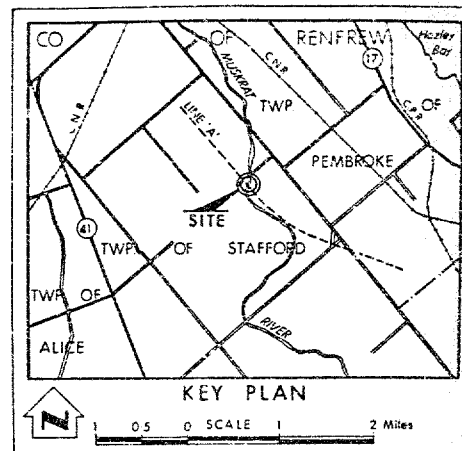
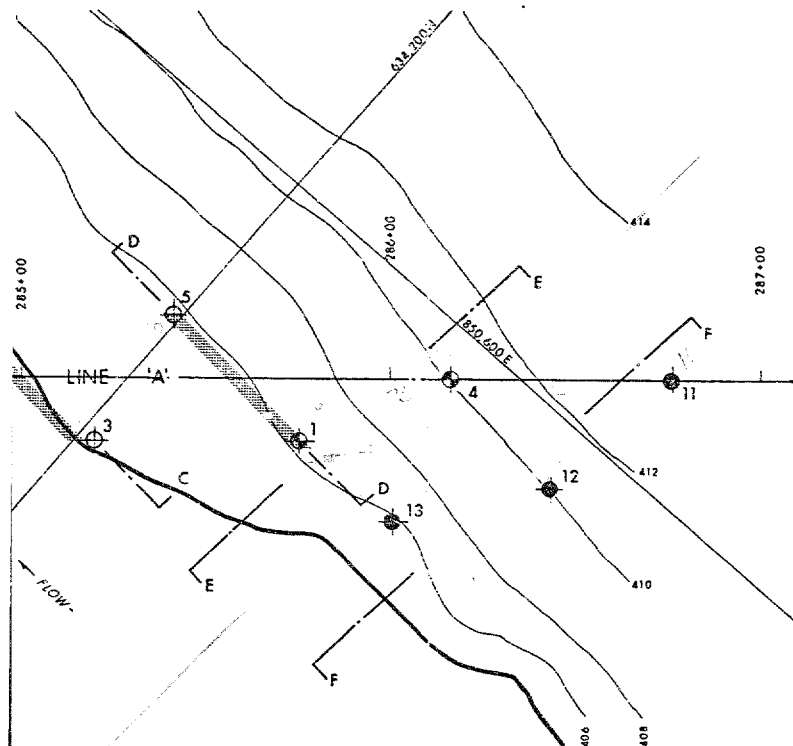
TREATMENT FOR WEST APPROACH

DATE 8 JAN. 1976

WP 10 - 67 - 02

FIG No 5





LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Resistance Test
B/CONE - Blows/Ft. Cone Test (350 ft. lbs. energy/blow)
- ⊕ Bore Hole & Cone Test
- ⊕ Water Levels established at time
of field investigation.
Sept. Oct. & Dec. 1975

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	405.7	634,154	850,548
2	404.0	634,245	850,515
3	404.9	634,196	850,511
4	410.3	634,134	850,588
5	405.2	634,202	850,551
6	403.5	634,294	850,428
6A	405.0	634,309	850,410
7	404.0	634,337	850,431
7A	405.5	634,351	850,416
8	414.0	634,329	850,392
9	413.0	634,365	850,396
10	418.9	634,360	850,373
11	413.2	634,089	850,627
12	410.1	634,094	850,583
13	405.7	634,121	850,548

NOTE -

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

NOTE FOR CONTRACT DOCUMENT

The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the OTTAWA District Office.

REVISION	DATE	BY	DESCRIPTION

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

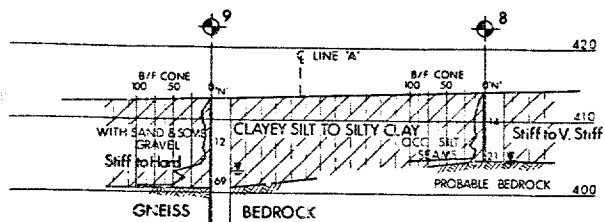
MUSKRAT RIVER

HIGHWAY NO. 17N LINE 'A' DIST. NO. 9
CO. RENFREW
TWP. STAFFORD LOT 21 CON 1

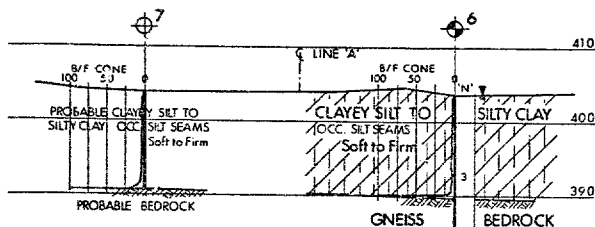
BORE HOLE LOCATIONS & SOIL STRATA

SUBMD V.R.	CHECKED	W.P. NO. 10-67-02	DRAWING NO.
DRAWN SOH	CHECKED	W.D. NO.	106702-A
DATE 29 Dec 1975	SITE NO. 29-158	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		

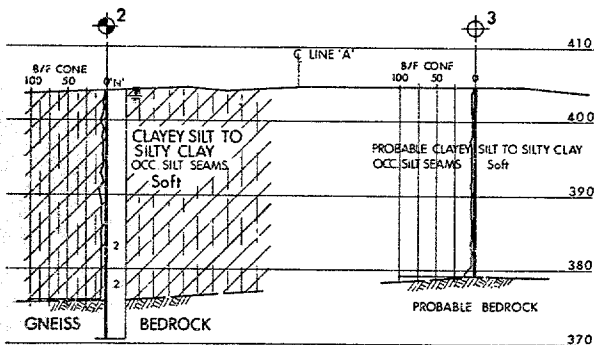




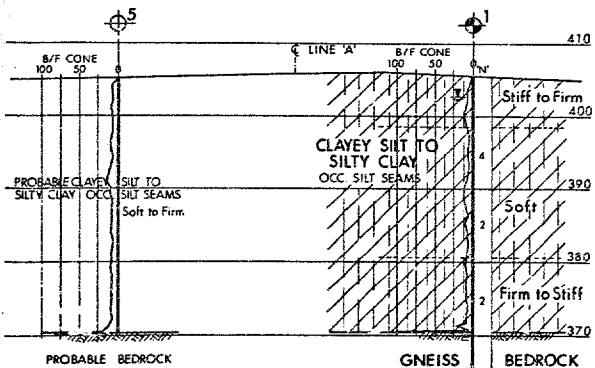
A - A



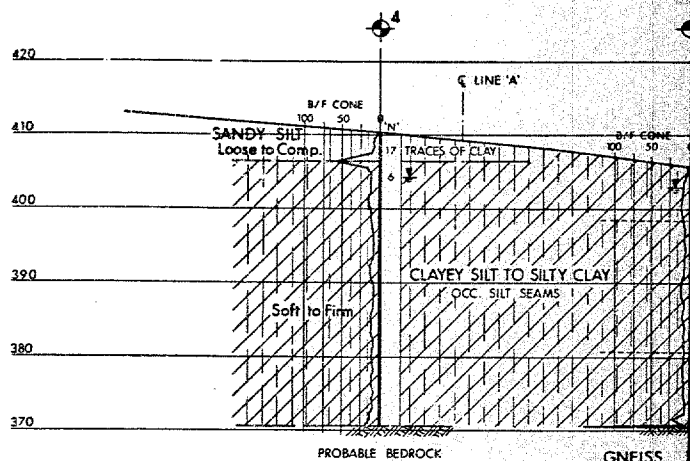
B - B



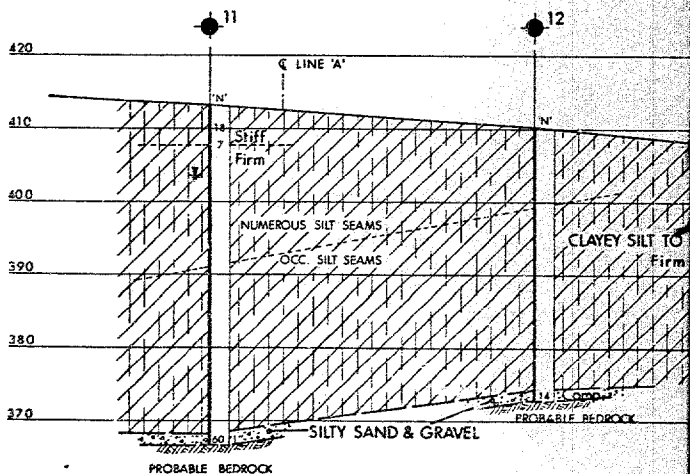
C - C



D - D



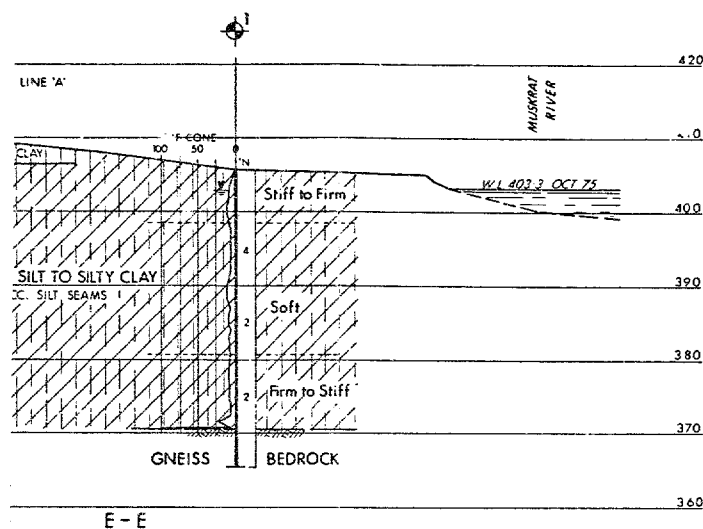
E - E



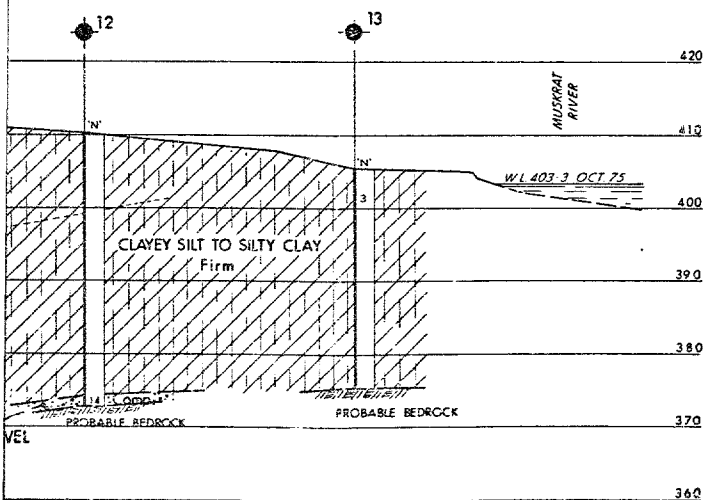
F - F

SECTIONS

10 5 0 SCALE 10 20 F



NOTE
W.L. NOT ESTABLISHED IN B.H. 12 & 13



SECTIONS

5 0 SCALE 10 20 FT

SEE DRAWING NO. 106702-A



KEY PLAN

LEGEND

- Bore Hole
- Dynamic Cone Penetration resistance Test
B/C CONE - Blows/Ft. Cone Test (250 ft. lbs. energy/blow)
- Bore Hole & Cone Test
- Water Levels established at time of field investigation.
Sept. Oct. & Dec. 1975

NO.	ELEVATION		

NOTE

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

REVISION	DATE	BY	DESCRIPTION

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

MUSKRAT RIVER

HIGHWAY NO. 17N LINE 'A' DIST NO. 9
CO. RENFREW
TWP. STAFFORD LOT 21 CON. 1

BORE HOLE LOCATIONS & SOIL STRATA

SUBMIT V.K. CHECKED BY W.P. NO. 10-67-02 DRAWING NO.
DRAWN SOCH. CHECKED BY W.O. NO. 106702-B
DATE 29 DEC 1975 SITE NO. 29-158 BRIDGE DRAWING NO.
APPROVED: J. J. L. CONT. NO.



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

Soil Mechanics Section
Engineering Materials Office
West Building, Downsview

January 27th, 1977

Mr. K. Bassi

Muskat River Bridge
W.P. 10-67-02, Site #29 - 158
Highway #17N, District 9, Ottawa

We have reviewed the final bridge design drawings (29 - 158 - 1 and 3) of this project and submit the following comments:

According to the drawings, the footings of the west abutment and the west pier are founded on sloping bedrock (about 3H to 1V in the longitudinal direction of the structure). Under such circumstances, the stability of the footings against sliding and overturning is very critical. To calculate the sliding resistance, a coefficient of friction of 0.85 between concrete and bedrock can be assumed. In the calculation, the passive earth pressure in front of the footings should be neglected. A factor of safety of at least 1.5 against sliding and at least 2.0 against overturning should be used. If the required resistance cannot be obtained from the frictional forces alone, considerations should be given to the use of dowels or anchors. Alternatively, the footings can be placed on relatively flat bedrock surface, together with the use of keys if necessary.

B. Ly

B. Ly
Senior Engineer

for M. Devata
Supervising Engineer

DL/km

c.c. T. C. Kingsland
Files /
Record Services

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

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

May 10, 1976

Muskrat River Bridge
Hwy. 17N, District #9, Ottawa
W.P. 10-67-02, Site 29-158

We have carried out a stability analysis for the east approach with the final grade lowered to elevation 420 as per your memorandum of April 23, 1976.

The analysis indicates that the forward slope as shown on Fig. No. 4 of the above report can be shifted towards the river parallel to itself along the centreline of the highway. The centreline of the highway should intersect the top of the new slope at Station 285+93 and the toe of the slope at Station 285+52 approximately. The new geometry will have a factor of safety of 1.09 along the most critical section, i.e., perpendicular to the river. In view of the low factor of safety it is recommended that additional rip rap be placed beyond the toe of the slope. Furthermore, the river should be partially filled where the slope comes very close to the river, as discussed with you on May 4, 1976.

A. Prakash

A. Prakash
Senior Engineer

For: M. Devata
Supervising Engineer

cc: K. Bassi
S.J. Radbone
E.R. Saint
Files
Record Services



Memorandum

To: Mr. M. Devata,
Supervising Engineer,
Soil Mechanics Section,
Downsview, Ontario.

From: Structural Planning Office,
Kingston, Ontario.

Attention:

Date: 23 April 1976

Our File Ref.

In Reply to

Subject: W.P. 10-67-02, Site 29-158
Muskkrat River Bridge - Pembroke Bypass
Highway 17N, District 9 - Ottawa

I refer to your meeting with Mr. T. C. Kingsland on 6 April 1976 in your office regarding the proposed grade of Highway 17N in the vicinity of the above structure.

Enclosed herewith please find one copy of Site Plan E-5263-1 on which I have plotted the latest grade received from Regional Planning and Design. Also enclosed is a sheet containing the proposed vertical curve information. It is regretted that this is not yet the approved grade for this section of Highway 17N but at this time appears to be acceptable to most Regional offices.

I would be pleased if you could review your recommendations contained in your Foundation Investigation Report for this structure in view of the reduced fill height on the east bank.

Our program date for issuing this structure for design to the Structural Design office is 5 May 1976, and pier and abutment locations for this structure are dependent upon your recommendations.

For: A. Van Dalen
T. C. Kingsland
Regional Structural Planning Engineer

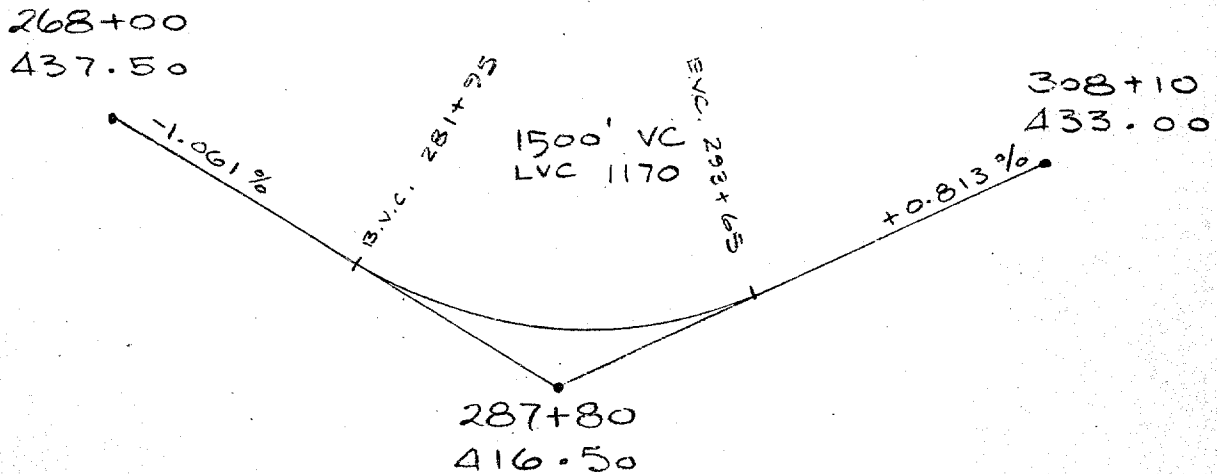
AV/TCK/hl
encls.

c.c. S. C. J. Radbone - Att. D. Thomas
R. Forrest
C. S. Grebski - Att. K. Bassi (+ vertical curve info.)

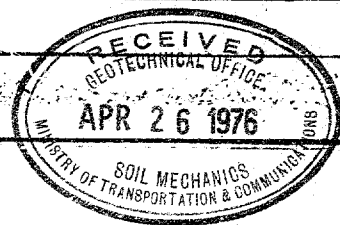


W.P. 10-67-0.1

MUSKOGEE RIVER



STATION	ELEVATION
0282+00.00	422.652
0282+25.00	422.394
0282+50.00	422.145
0282+75.00	421.907
0283+00.00	421.679
0283+25.00	421.461
0283+50.00	421.253
0283+75.00	421.055
0284+00.00	420.867
0284+25.00	420.688
0284+50.00	420.520
0284+75.00	420.362
0285+00.00	420.214
0285+25.00	420.076
0285+50.00	419.948
0285+75.00	419.829
0286+00.00	419.721
0286+25.00	419.623
0286+50.00	419.535
0286+75.00	419.457
0287+00.00	419.389





Memorandum

To: FILE

From: Structural Planning Office,
Kingston, Ontario.

Attention:

Date: 9 February 1976

Our File Ref.

In Reply to

Subject:

W.P. 10-67-02, Site 29-158
Muskrat River Bridge
Highway 17N, District 9-Ottawa

The results of the final foundation investigation and the recommendations contained in the Foundation Investigation Report for the above structure were discussed in a meeting in the Soil Mechanics Office with Mr. M. Devata and Mr. A. Prakash during my visit to Downsview on 6 February 1976.

The recommendations in the report included lengthening the total span to 265 ft. from 220 ft. as proposed, due to the poor subsoil conditions discovered during the final investigation. It was established that, by lowering the grade on the east bank of the river, a considerable reduction in structure length could be achieved.

If, for example, the grade near Sta. 286+00 could be lowered by 2 ft., the immediate saving in structure length would be 4 ft. Because of the decreased fill height, the toe of the 2:1 slope can be brought forward a considerable distance. Due to the heavy skew of the crossing this distance depends on the critical stability of a cross section through the fill perpendicular to the river.

Saving in structure length will depend on new stability analysis to be carried out by the Soil Mechanics Section based on the lowered grade.

Indications at this time are that we may possibly reduce the total structure length to 235 ft., or a saving of some \$45,000 in structural costs. A further saving would presumably be achieved because of the reduced height of fill of the highway over a length of some 1200 ft. on the east approach to the structure.

A total structure length of 265 ft. based on the present high grade would also result in undesirable span divisions, possibly necessitating construction of a pier in mid channel. By lowering the grade on the east side and reducing the overall span, a more pleasing span-ratio can be achieved.

Lowering the grade appears to be quite possible from a hydrological point of view on basis of required clearances as presently established.

A. Van Dalen
A. Van Dalen

For: T. C. Kingsland
Regional Structural Planning Engineer

c.c. M. Devata
S. C. J. Radbone - Att. D. Thomas
C. S. Grebski - Att. K. Bassi



P.S. A.P. → Please note the contents and let me if you have any comments and files



Memorandum

To: Mr. M. Devata,
Supervising Engineer,
Soils Mechanics Section,
Downsview, Ontario.

From: Structural Planning Office,
Kingston, Ontario.

Attention:

Date: 18 December 1975

Our File Ref.

In Reply to

Subject: W.P. 10-67-02, Site 29-158
Muskrat River Bridge
Highway 17N, District 9-Ottawa

Further to our recent telephone discussion, please find attached copy of a memorandum dated 16 December 1975 from Mr. D. B. Thomas, Regional Planning & Design office, concerning the alignment at the proposed Muskrat River crossing. This would indicate that slight realignment is possible if improved foundation soils can be found fairly close to the existing alignment.

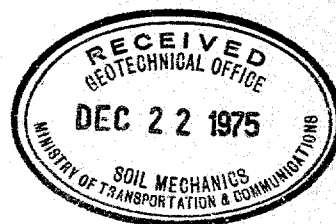
I understand that your previous calculation of an additional 40 ft. longitudinal berm required for the east approach fill for the structure may now be modified in the light of further information to hand. If it is evident from your new calculations that the berm length can be drastically reduced, there would appear to be no reason to change the alignment. If, however, only a small reduction in berm length is envisaged, then we shall be glad if you will arrange to put down some further boreholes within the limits mentioned in Mr. Thomas' memorandum.

I note from our telephone discussion that you will be phoning me very shortly with the results of your new calculations.

T. C. Kingsland
Regional Structural Planning Engineer

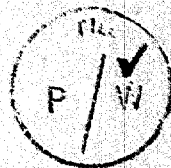
TCK/hl
att.

c.c. P. D. Billings
S. C. J. Radbone - Att. D. B. Thomas
C. S. Grebski - Att. K. Bassi





Memorandum



To: T. G. Kingsland
Reg. Bridge Location Engineer
Kingston, Ontario

From: Planning & Design Office
Kingston, Ontario

Attention:

Date: December 16, 1975

Our File Ref.

In Reply to

STRUCTURE SITE NO. 29-158

Subject:

W.P. 1-67-00, HIGHWAY 17N
PEMBROKE, BYPASS

Further to your memo of December 9th, 1975, please
be advised that some minor alignment shift can be tolerated
in the vicinity of the Muskrat River.

We feel that it is worth while to get some additional
foundations information to the north of the existing Eastbound
lane alignment. The investigation should be confined to a
distance of no more than 200 ft. from the currently proposed
bridge site.

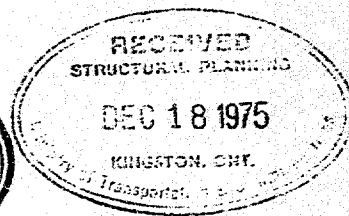
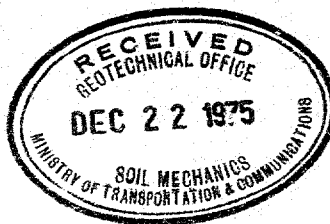
D. B. Thomas

D. B. Thomas
Project Manager

DBT/ijl

Copies made for: (TCK: 18/12/75)

✓ M. Devata
C. S. Grebski - Att. K. Bassi





Memorandum

To: Mr. S. C. J. Radbone, Manager,
Planning & Design Office,
Kingston, Ontario.

From: Structural Planning Office,
Kingston, Ontario.

Attention: Messrs. G. Boggis/
D. Thomas

Date: 9 December 1975

Our File Ref.

In Reply to

Subject:

W.P. 10-67-02, Site 29-158
Muskrat River Bridge
Highway 17N, District 9-Ottawa

Soil Mechanics Section, Downsview, recently informed me that there are borings on the east bank of the Muskrat River at the proposed crossing that reveal a deposit of low strength marine clay approximately 35 ft. in depth. According to Mr. Devata of Soil Mechanics Section, the main effect of this marine clay deposit, assuming existing preliminary grades at the crossing to apply, will be to produce a requirement for a mid-height longitudinal berm of 40 ft. length at the east approach. This requirement will increase the structure length by a similar amount, increasing the structure cost by an estimated \$110,000. Possible ways of reducing the structure length are:

- a) Extra boreholes could be taken on each side of the proposed line and the alignment changed if more favourable foundation conditions were found.
- b) The grade could be dropped by about $2\frac{1}{2}$ ft. (the maximum permissible from a hydrological viewpoint). This would reduce the fill height and therefore the berm length and extra structure length required.
- c) Further encroachment into the river could be considered. This measure would be considered in any case should hydrological considerations permit.

On 8 December 1975, D. Thomas, M. Batten, A. Van Dalen and T. C. Kingsland met in Regional Structural Planning office to discuss the implications of the marine clay pocket. The following points were decided:

- 1) D. Thomas stated that the horizontal alignment cannot be changed at this stage. If this is so, there would appear to be no point in taking extra boreholes to investigate conditions of either side of the proposed alignment.
- 2) From Planning & Design point of view, the preliminary grade over the structure can be lowered by up to 2'-6" but this can occur only at the east side due to large cuts becoming necessary on the west side if the grade is lowered there.



Cont'd...

- 3) From Materials & Testing point of view, the grade can be lowered the required amount on the east side without severe problems, at a preliminary estimated extra cost of \$10,000.
- 4) Since the extra structure length saved by lowering the grade $2\frac{1}{2}$ ft. is about 10 ft. and the saving in structure cost approximate!y \$27,000, lowering the grade is a reasonable proposition and should therefore be carried out.

It is stressed that an alignment change would produce the greatest structural saving of up to the full amount of \$110,000 provided suitable foundation conditions exist nearby. Such a move should therefore be seriously considered before finally discarding this alternative.

The preliminary foundation investigations carried out earlier were for different alignments to the one finally decided on by Planning & Design. The clay pocket causing the above problem did not therefore reveal itself at that stage.



T. C. Kingsland
Regional Structural Planning Engineer

TCK/hl

c.c.

P. D. Billings

B. R. Davis

E. R. Saint - Att. M. Batten

✓ M. Devata

A. Van Dalen

C. S. Grebski - Att. K. Bassi

