#69-F-62

W.P. 113-66-01

H.W.Y. #2

HAMILTON ENTRANCE

BRIDGE #1

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

Attention: Mr. S. McCombie

Our File Ref.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Date: October 10, 1969

In reply to OCT 17 1969

Subject:

FOUNDATION INVESTIGATION REPORT
For
Hamilton Entrance Bridge #1
On Hwy. #2
County of Halton
District No. 4 (Hamilton)
W.J. 69-F-62 - W.P. 113-66-01

Attached, we are forwarding to you, our detailed foundation investigation report on the subsoil conditions existing at the above structure site.

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

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
H. Greenland
W. S. Melinyshyn
T. J. Kovich
B. A. Singh

Foundations Files
Gen. Files

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FOUNDATION INVESTIGATION REPORT
For
Hamilton Entrance Bridge #1
On Hwy. #2
County of Halton
District No. 4 (Hamilton)
W.J. 69-F-62 - W.P. 113-66-01

1. INTRODUCTION:

A foundation investigation was requested by Mr. W. S. Melinyshyn, Regional Bridge Location Engineer, Central Region, for the proposed new entrance Bridge #1 in Burlington. The request was submitted in a memo dated August 5, 1969. The new proposal will replace the existing Bridges #1 and #2 with one long structure over Waterdown Creek, Snake Road and the C.N.R. tracks.

Upon receipt of the memo, a field investigation and subsequently, a laboratory testing program were carried out by this Section.

Presented in this report are the results of the investigations, together with recommendations concerning foundations.

2, DESCRIPTION OF THE SITE:

Existing Bridges #1 and #2 are located some 1500 ft. east of the intersection of Hwys. #2 and #6, along Hwy. #2 in the City of Burlington.

The general terrain is undulating with steep to moderate slopes. At the crossing the elevation of existing Hwy. #2 is around 332 ft., while the floodplain of Waterdown Creek, below the bridge, is at el. 246.0 ft. - a difference of 86 ft. North and South of the bridge, there are several smaller and larger ponds and swampy depressions.

2. DESCRIPTION OF THE SITE: (cont'd.) ...

Geologically the area belongs to the physiographic region known as the Iroquois Plain. Lake Iroquois was the fore-runner of Lake Ontario in the late Pleistocene times. Its old shorelines, including cliffs, bars, beaches and boulder pavements are easily identifiable features. In our particular area, the great gravel bar, which separates Coot's Paradise from Hamilton Harbour, marks the shoreline of the glacial lake.

3. FIELD AND LABORATORY INVESTIGATIONS:

The field investigation consisted of 15 sampled boreholes and adjacent to certain boreholes, some 7 dynamic cone penetration tests. One or two boreholes were placed at each proposed abutment and pier location. The borings were implemented by means of conventional diamond drill rigs adapted for soil sampling purposes. Split-spoon and "undisturbed" Shelby tube samples were taken at regular intervals in every borehole. Split-spoon samplers were advanced by performing standard penetration tests, as described at the end of this report under the heading: "Abbreviations Used in This Report".

Penetration 'N' values are recorded on the accompanying borelog sheets, together with the other field and laboratory test results. Shelby tubes were generally pushed into the soil manually. Within the cohesive strata, field vane tests were carried out in order to determine the undrained shear strength of the material. Bedrock was proved by advancing a BXL type core barrel, using diamond drilling techniques. All samples were recorded in the field logs immediately after recovery.

Upon arrival in the laboratory, the samples were again classified by means of some simple visual tests. Further laboratory tests were performed on representative specimens to obtain the Atterberg limits, grain-size distributions, natural moisture contents, unconsolidated undrained shear strengths and consolidation characteristics of the various deposits. A concise description of the subsoils is given in the following section:

..... 3

4. SUBSOIL CONDITIONS:

4.1) General:

The overburden within the investigated area was found to consist of several heterogeneous and mixed strata of glacial and glacio-fluvial origin. With some simplification, the layers were divided into three main deposits. A reddish-brown clayey silt to silty sand with shale fragments was underlain by grey clayey silts at the east slope of the valley. A similar reddish-brown mixture was found on the west slope as well, but with much larger amounts of gravel and shale. The valley floor is underlain by thick deposits of organic clays and clayey silts. Shale bedrock follows the overburden at various elevations.

A brief description of the layers is given below:

4.2) Clayey Silt to Silty Sand, Red-Brown:

This material forms the uppermost layer along the east slope, the overall thickness of which is roughly 30 - 40 ft. Within the first 15 ft. or so, the soil has some slight plasticity with an average plastic limit moisture content of 17% and a liquid limit of 28%. In this zone the material is identified to be clayey silt with some sand. The consistency of this cohesive portion ranges from stiff to very stiff, corresponding to penetration 'N' values of 8 to 28 blows per ft. In the lower elevations the deposit changes to silty sands to sandy silts, containing some 37 - 85% sand size particles. The relative density of the granular portion is compact to very dense with penetration 'N' values of 25 blows per ft. to above 100 blows per ft. The reddish-brown colour and the small fragments of shale indicated that the soils were derived from the parent shale bedrock. Along the west slope (B.H.'s #7 - #11) this material was found to contain a much larger amount of shale fragments and a fair percent of large size boulders. At these locations, however, the entire stratum is cohesive, exhibiting slight plasticity and hard consistency. Penetration 'N' values are usually in excess of 100 blows per ft.

..... 4

4. <u>SUBSOIL CONDITIONS</u>: (cont'd.) ...

4.3) <u>Clayey Silt, Traces of Sand, Grey</u>:

A fairly homogeneous stratum of clayey silt, containing occasionally a few grains of coarse sand underlies the red silty sands at the east slope. In B.H.'s #3 and #3A, near the middle of the slope, this layer was found at ground elevation, the original red stratum likely being completely eroded. The material has a grey colour and hard to firm consistency. Laboratory unconfined and quick triaxial compression tests yielded shear strength values ranging from 700 PSF to 2200 PSF. The average plrstic limit may be taken to be 17T and the liquid limit 32% with some 20 - 21% natural moisture content. The thickness of the deposit varies between 15 ft. and 35 ft., the average bulk density being 129 PCF.

4.4) <u>Clay to Clayey Silt with Organic Matter</u>:

In B.H.'s #4, #5 and #6, placed near the creek and Snake Rd. at the bottom of the valley, a deep deposit of clays, silty clays and clayey silts was encountered. The material - especially at the higher zones - is heavily contaminated with black organic and vegetable matters and numerous shells. At the lower elevations an increasing number of shale fragments was observed. This material also appears to be a derivation of the parent bedrock, decomposed, transported and redistributed under glacial and post-glacial streams. The shear strength - measured by field and laboratory tests - was found to be around 500 PSF with the upper 20-ft. layer, increasing slightly with depth. The natural moisture contents are very high down to el. 230 ft., and within this zone, the stratum exhibits high plasticity and compressibility on account of the constituent organic matters. The Atterberg limits and moisture contents gradually decrease with depth so that the soil becomes clayey silt of slight plasticity.

..... 5

4. <u>SUBSOIL CONDITIONS</u>: (cont'd.) ...

4.5) <u>Bedrock</u>:

Bedrock was proved at 10 borehole locations by means of diamond drilling with BXL size core barrels. Because of the extremely weathered nature of the upper portion of the bedrock, the identification of the actual bedrock surface was very difficult, if not impossible. Discrepancies as to the specified surface may be encountered when excavating for the footings. The rock was classified to be calcareous shale of the Queenston formation. The colour is generally red, having interbedded grey seams and layers. These shales tend to weather mechanically when uncovered and break down into clays and clayey silts. Several waterbearing seams were observed within the upper portion of the rock along which the material is badly leached out and softened.

5. <u>GROUNDWATER CONDITIONS</u>:

At the east slope the groundwater level was established between el. 282 ft. and 284 ft., just above the grey clayey silt stratum. In B.H. #1 near the east abutment, a slight artesian pressure was noticed around el. 245 ft. within the sand and gravel. The water has risen to one ft. above ground level in the BX casing. During one night, however, the head of water dropped some 20 ft.

The groundwater level in B.H.'s #4, #5 and #6, at the bottom of the valley, was found at approx. 1 - 2 ft. below ground level, corresponding roughly to the water level of the creek. Some seepage was noted in B.H. #7 above the bedrock at el. 248 ft. No free water was detected in holes #8 to #11.

6. <u>DISCUSSION AND RECOMMENDATIONS</u>:

6.1) <u>General</u>:

The proposal calls for multi-span twin structures to replace the existing Bridges #1 and #2 along Hwy. #2 in Burlington. The centre-line of the structure will be just north of the existing bridge, the piers being proposed on individual footings.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.1) General: (cont'd.) ...

Subsoils at the site were found to consist of clayey silt, silty sand and organic clay to clayey silt, underlain by shale bedrock. General remarks about the foundations are summarized below, after which specific recommendations are given pertaining to individual abutments and piers.

6.2) Foundations:

(a) Regardless of the suggested footing elevations, a four-ft. cover should be provided above the bottom of spread footings or pile caps for frost protection.

(b) Weathered and sound bedrock elevations are given for estimating purposes only, since by drilling alone, it was not possible to establish these elevations with absolute certainty.

A careful visual examination of the rock will be necessary when the footing excavations are in progress, in order to determine the exact elevations of the weathered and sound rock surface. Some modifications of the suggested depth of footings are anticipated.

(c) No major instability was noticed along the existing valley slopes. It is recommended that the overall slopes be not disturbed nor steepened. If new slopes are constructed, they should be built with 2 horizontal to 1 vertical.

(d) Piers are numbered from east to west on Drawing #69-F-62A for easier reference.

(e) The full structural strength of the particular pile section used may be employed in the locations where steel H-piles are recommended to be driven to practical refusal on bedrock.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Foundations: (cont'd.) ...

(f) A somewhat reduced bearing pressure was used in those locations where spread footings are supported on bedrock below steep slopes.

It was felt that, due to the sloping terrain, this additional safety factor was justified.

(g) Recommendations for the individual footings are as follows:

East Abutment, Piers #1, 2, 3, 4 and 5 -

Subsoils at these locations do not appear to have sufficient strength to support the structure on spread footings at shallow depths, consequently it is suggested that the footings be constructed on steel H-piles, driven to refusal on bedrock. The estimated elevations of the bedrock surface are tabulated below:

Locations	Elevation of Bedrock (Ft.)
East Abutment	243 - 244
Pier #1	249 - 250
Pier #2	252 - 253
Pier #3	193 - 195
Pier #4	175 - 180
Pier #5	185 - 186

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Foundations: (cont'd.) ...

Pier #6 -

The overburden was proved to be quite shallow at these footing locations, so that spread footings are recommended, the base of footings being supported on sound rock.

The sound rock surface is estimated to be around el. 247 ft. at the location of the north structure footing and around el. 270 ft. at the south footing. Design loads of 7.5 TSF may be used on the footings, provided they are placed on sound shale bedrock.

Pier #7 -

No borehole could be placed at the location of the proposed south structure footing on account of the existing sidewalk and the adjacent underground telephone and electric cables. Based on the findings of boreholes #8 and #8B, the sound bedrock surface is estimated to be around el. 294 ft. Depending upon structural requirements, either spread footings or short steel H-piles are suggested, supported on sound rock. 7.5 TSF safe loads are assumed on the spread footings.

Piers #8 and 9 -

The sound bedrock surface is anticipated to be around el. 300 ft. at the location of Pier #8 and around el. 299 ft. at Pier #9.

At both locations spread footings may be employed, the footing bases being placed on sound rock. Up to 10 TSF safe loads may be used on such footings.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Foundations: (cont'd.) ...

West Abutment -

The west abutment may be supported on spread footings placed on weathered bedrock around el. 309 - 310 ft., with an allowable bearing capacity of 5 TSF. The bearing pressure may be augmented to 10 TSF by lowering the footing to approx. el. 299 ft. into the sound bedrock. Alternatively, short steel H-piles driven to refusal on bedrock, can be utilized if more economical. Refusal may be reached around el. 299 - 305 ft.

7. MISCELLANEOUS:

The field work, carried out during the period September 8 - 25, was supervised by Messrs. A. K. Barsvary, Senior Foundation Engineer and G. Allen, Project Foundation Engineer.

Equipment used was owned and operated by Master Soil Investigations Ltd., and Canadian Longyear Ltd.

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

October 1969

APPENDIX I

FORM 08-MT-[illegible]26 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

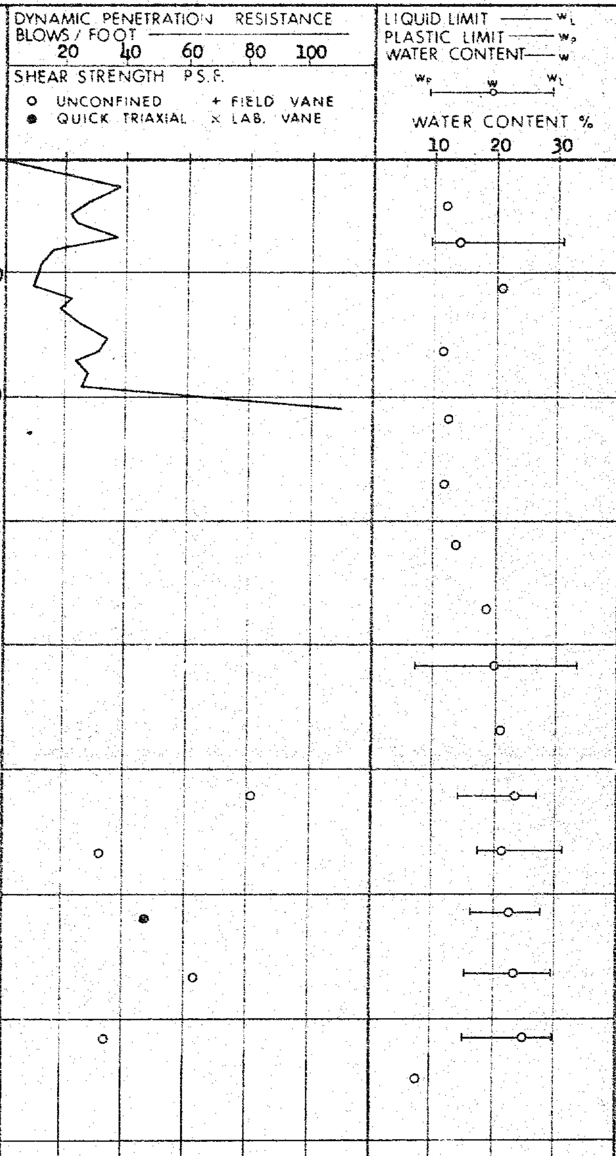
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,174 N; 898,150 E. ORIGINATED BY AKB
W.P. 133-66-01 BORING DATE Sept. 8 - 11, 969 COMPILED BY AKB
DATUM Geodetic BOREHOLE TYPE Washboring, NX & BX Casings CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
318.9	Ground Level						
0.0	Clayey silt with some sand	1	SS	28			
		2	SS	14			
	Stiff to very stiff	3	SS	8	310		
	Red-Brown						
303.9 / 15.0	Silty sand, traces of clay	4	SS	16			0 6 34 6
		5	SS	33	300		
	Compact to very dense	6	SS	64			
	Brown	7	SS	59	290		
		8	SS	110			0 56 39 5
279.4					▼ 280		
39.5	Clayey silt with traces of coarse sand & fine gravel	9	SS	41			
		10	SS	28	270		
		11	TW	PM		129	
	Hard to firm	12	TW	21		131	
					260		
	Grey	13	TW	PM		130	
		14	TW	PM		129	
					250		
		15	TW	PM		128	
245.9 / 243.8	Silt, sand & grav. (Till)	~~16~~	~~SS~~	~~100/1"~~			43 19 36 2
75.1	Shale Bedrock	17	RC	75%	240		
238.8 / 80.1	End of Borehole						



20
15 -◇- 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1A

FOUNDATION SECTION

JOB 69-F-62	LOCATION Co-ords. 730,143 N; 898,140 E.	ORIGINATED BY GA
W.P. 143-66-01	BORING DATE Sept. 8 - 10, 1969	COMPILED BY AKB
DATUM Geodetic	BOREHOLE TYPE Washboring, NX Casing	CHECKED BY [signature]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
329.0	Ground Level					
0.0	Clayey silt with some sand. Stiff Red-Brown	1	SS	15		
318.0		2	SS	15		
11.0	Silty sand to sandy silt with some clay & traces of gravel	3	SS	28		11 34 38 17
		4	SS	25		0 37 59 4
		5	SS	98		5 80 (15)
	Compact to very dense	6	SS	96		
		7	SS	118		
	Brown	8	SS	70/6"		1 77 (22)
		9	SS	110		
		10	SS	113		
280.0						
49.0	Clayey silt, traces of sand	11	SS	62		
		12	SS	42		
	Hard to Firm	13	SS	26		
	Grey	14	TW	PM	129	
243.8		15	SS	50/4"		
85.2	End of Borehole					

ELEV. SCALE: 320, 310, 300, 290, 280, 270, 260, 250

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

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

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

20
15 — 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

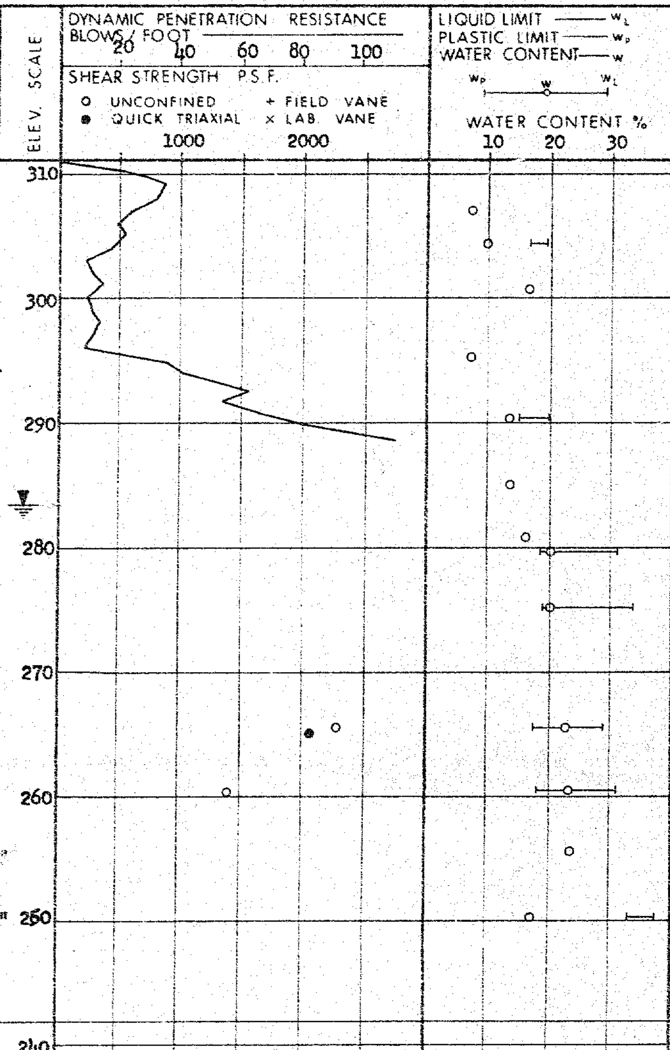
FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,179 N; 898,073 E. ORIGINATED BY GA

W.P. 143-66-01 BORING DATE Sept. 11-15, 1969 COMPILED BY AKB

DATUM Geodetic BOREHOLE TYPE Washboring, NX & BX Casing CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
311.0	Ground Level					
0.0	Clayey silt to silt, some sand	1	SS	21		
		2	SS	25		
	Very stiff to stiff	3	SS	14		
295.5	Red-Brown					
15.5	Silty sand to sandy silt with some clay	4	SS	38		0 85 (15)
		5	SS	55		
	Very dense	6	SS	56		
279.3		7	SS	58		
31.7	Clayey silt	8	SS	35		
	Hard to stiff	9	SS	20		
		10	TW	PM	130 129	
	Grey	11	TW	PM	127	
		12	SS	17		
249.8		13	SS	100/5"		
61.2	Shale Bedrock	14	RC	75%		
		15	RC	72%		
241.8		16	RC	75%		
69.2	End of Borehole					



20
15 -◊- 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,225 N; 897,935 E. ORIGINATED BY GA
W.P. 113-66-01 BORING DATE Sept. 16, 1969 COMPILED BY AKB
DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 1000 2000	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
281.8	Ground Level									
0.0						280				
	Silty clay to clayey		1	SS	49					
	Silt.		2	SS	49					
	Hard to stiff		3	SS	53	270				
	Grey & Brown		4	SS	38					
			5	SS	12	260				
			6	TW	PM					
252.8	Bedrock		7	SS	100/3"					
29.3	End of Borehole									

20
15 ⌖ 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3A

FOUNDATION SECTION

JOB 69-F-62
W.P. 113-66-01
DATUM Geodetic

LOCATION Co-ords. 730,208 N; 897,930 E.
BORING DATE Sept. 12-15, 1969
BOREHOLE TYPE Washboring, NX Casing

ORIGINATED BY GA
COMPILED BY AKB
CHECKED BY

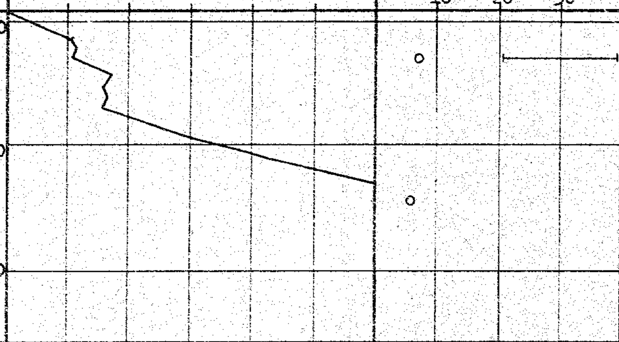
SOIL PROFILE: ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
270.7	Ground Level					
0.0	Silty clay to clayey silt traces of sand Hard Grey		1	SS	61	270
			2	SS	65	
			3	SS	58	260
255.2	Clayey silt with shale fragments		4	SS	50/2"	
252.7 / 18.0	Shale Bedrock		5	RC	50%	250
244.1			6	RC	90%	
26.6	End of Borehole					

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

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

BULK DENSITY γ P.C.F.

REMARKS GR. SA. SI. CL.



20
15 — 5 % STRAIN AT FAILURE
10

FORM OS-MT-128 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

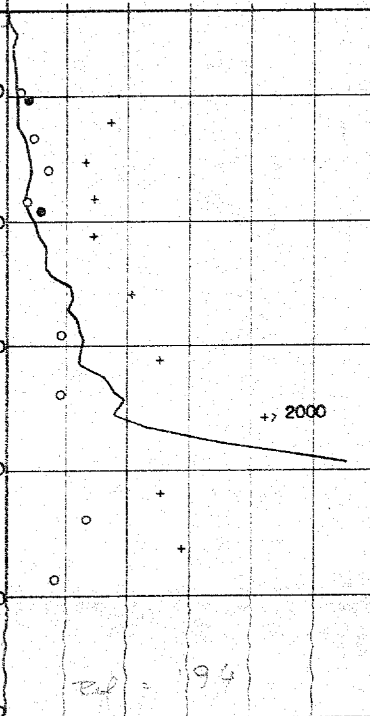
FOUNDATION SECTION

JOB 69-F-62 | LOCATION Co-ords. 730,270 N; 897,818 E. | ORIGINATED BY GA

W.P. 113-66-01 | BORING DATE Sept. 16-18, 1969 | COMPILED BY AKB

DATUM Geodetic | BOREHOLE TYPE Washboring, NX Casing | CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	W_P	W_L	W	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
246.8	Ground Level										
0.0	Clay to clayey silt with organic matter & shells, Fragments of shale at the lower zone		1	SS	1		40.6	60.2	50.8		
			2	TW	PM	240	50.0	91.3	83.7	91.5, 93.0	
			3	TW	PM		37.9	56.8	71.5	91.5	
			4	TW	PM				80.5	95.0	
			5	TW	PM	230	50.0	79.5		109.0, 114.5	
			6	SS	2						
	Soft to Stiff		7	TW	PM	220	28.1	52.0		114	
			8	TW	PM					107.0	
	Red-Brown		9	SS	4	210					
			10	TW	PM					130	
			11	TW	25/3"					138.0	
			12	SS	57	200					
194.8			13	SS	102						
52.0	Shale Bedrock		14	RC	100%						
			15	RC	33%						
189.			16	RC	100%	190					
57.8	End of Borehole										



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

20
15 5 % STRAIN AT FAILURE
10

OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

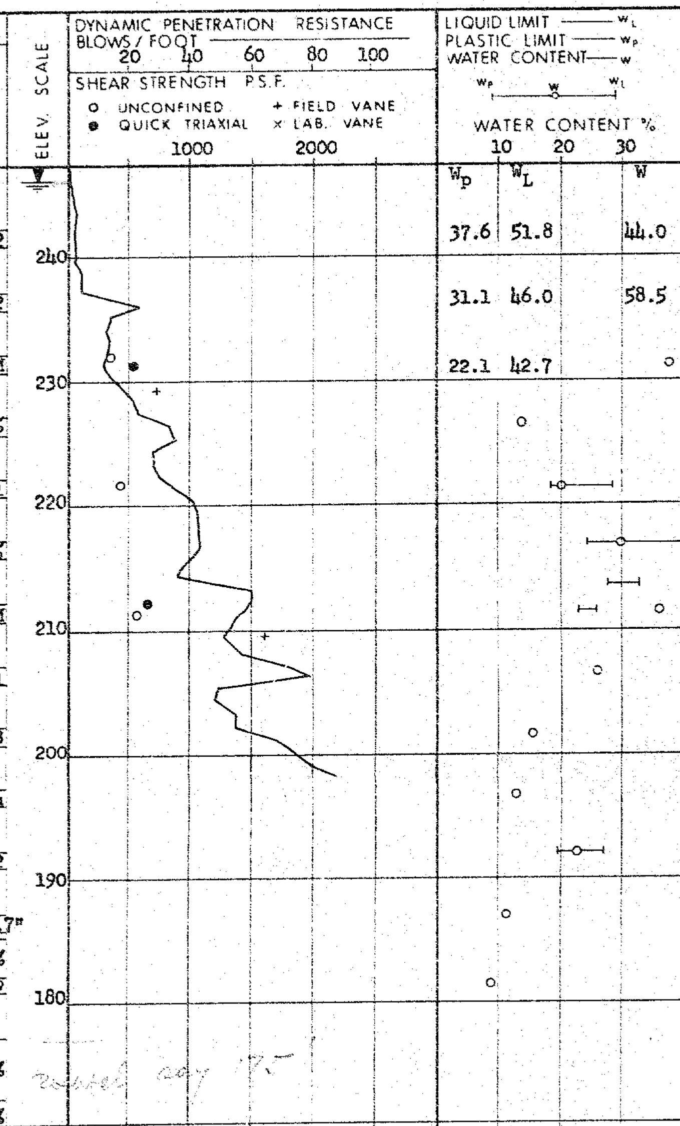
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 69-F-62	LOCATION Co-ords. 730,306 N; 897,685 E.	ORIGINATED BY AKB
W.P. 113-66-01	BORING DATE Sept. 16-22, 1969	COMPILED BY AKB
DATUM Geodetic	BOREHOLE TYPE Washboring BX & NX Casings	CHECKED BY

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	W_p	W_L	W	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
247.3	Ground Level								
0.0	Silty clay to clayey	1	SS	2	37.6	51.8	44.0		
	Silt with	2	SS	2	31.1	46.0	58.5		
	organic matter & shells	3	TW	PM	22.1	42.7		119.0 113.5	
		4	SS	15					
	becoming clayey silt	5	TW	11				120.0	
	with increasing amount of shale	6	SS	15					
		7	TW	PM				112.0 115.0	
		8	SS	11					
	Soft to hard	9	SS	23					18 31 (51)
		10	SS	104					
	Red-Brown	11	SS	12					
		12	SS	100/7"					
		13	RC	14%					
		14	SS	79					43 26 (31)
	Probable Bedrock	15	RC	13%					
170.3		16	RC	25%					
77.0	End of Borehole								



20
15 -o- 5 % STRAIN AT FAILURE
10

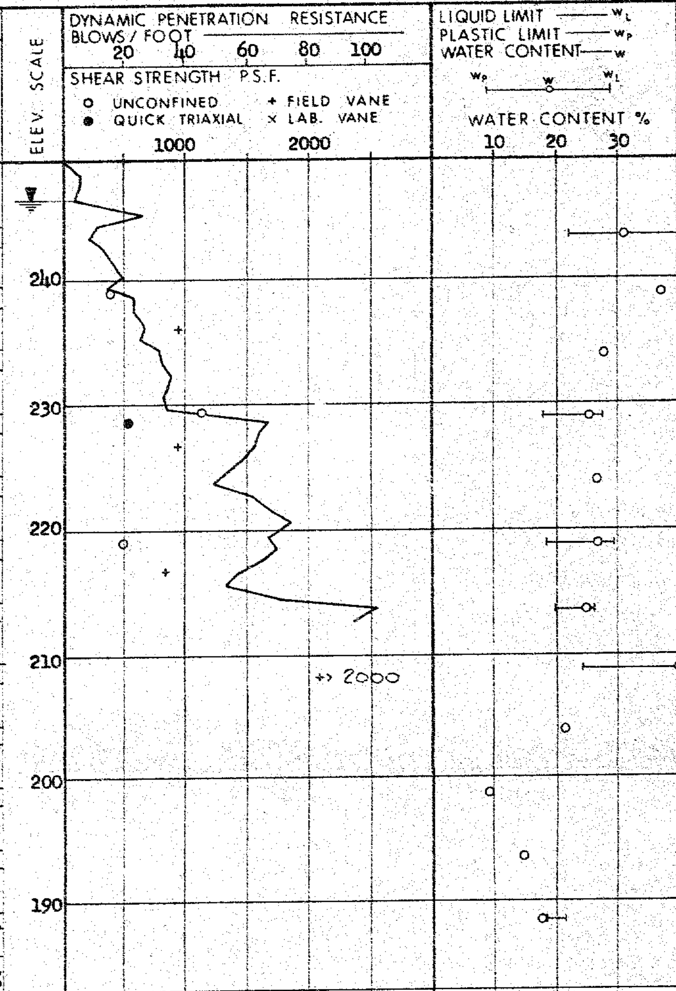
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 69-F-62	LOCATION Co-ords. 730,367 N; 897,561 E.	ORIGINATED BY AKB
W.P. 113-66-01	BORING DATE Sept. 16-19, 1969	COMPILED BY AKB
DATUM Geodetic	BOREHOLE TYPE Washboring, BX & NX Casing	CHECKED BY [signature]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
249.5	Ground Level					
0.0						
	Silty clay to clayey	1	SS	3		
	silt with organic	2	TW	PM	115.0	
	matter & shells	3	SS	3		
	Shale fragments at	4	TW	PM	128.0 123.0	
	the lower zone	5	SS	10		
		6	TW	PM		
	Soft to Hard	7	SS	8		8 15 62 15
		8	TW	PM		
	Red-Brown	9	SS	19		
		10	SS	30		
		11	SS	19		
		12	SS	31		
186.0 63.5	Shale Bedrock	13	RC	97%		
183.0 66.5	End of Borehole					



20 15 5 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,425 N. 897,447 E. ORIGINATED BY AKB
W.P. 113-66-01 BORING DATE Sept. 15-16, 1969 COMPILED BY AKB
DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing CHECKED BY

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
259.0	Ground Level									
0.0	Clayey silt with gravel & boulders.		1	SS	30					
	Hard		2	SS	57					
						250				
247.0	Red		3	SS	115/6"					
12.0	Shale Bedrock		4	RC	59%					
			5	RC	80%					
240.5										
18.5	End of Borehole									

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7A

FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,399 N; 897,429 E. ORIGINATED BY AKB
W.P. 113-66-01 BORING DATE Sept. 15-16, 1969 COMPILED BY AKB
DATUM Geodetic BOREHOLE TYPE Washboring, BX & NX Casing CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_p, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
275.6	Ground Level									
0.0	Clayey silt with gravel (Glacial Till)									
272.6										
3.0	Weathered Shale Bedrock		1	SS	100/4"	270				27 11 49 13
			2	RC	82%					
	Sound									
			3	RC	64%					
259.1						260				
16.5	End of Borehole									

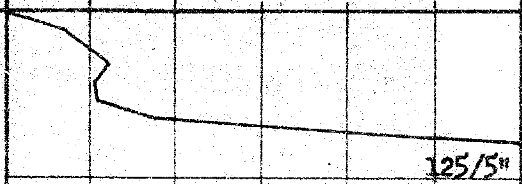
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

JOB	69-F-62	LOCATION	Co-ords. 730,478 N; 897,309 E.	ORIGINATED BY	GA
W.P.	113-66-01	BORING DATE	Sept. 18, 19, 22, 1969	COMPILED BY	GA
DATUM	Geodetic	BOREHOLE TYPE	Washboring NX & BX Casing	CHECKED BY	[signature]

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT (20, 40, 60, 80, 100); SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w — WATER CONTENT % (10, 20, 30)	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
309.4	Ground Level								
0.0	Clayey silt with sand & gravel	1	SS	21					11 39 39 11
	Large fragments of shale	2	SS	50/3"					
		3	RC	40%		125/5"			
					300				
	Very stiff to hard	4	SS	22					
294.4		5	SS	70/1½"					
		6	RC						
15.0		7	RC	22%					
		8	RC	63%					
	Shale Bedrock	9	RC	71%	290				
287.4									
22.0	End of Borehole								



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

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8 B

FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,452 N; 897,303 E. ORIGINATED BY GA

W.P. 113-66-01 BORING DATE Sept. 26, 29, 1969 COMPILED BY GA

DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing CHECKED BY

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
324.7	Ground Level					
0.0	Clayey silt with sand & gravel, Shale fragments	1	SS	12	320	
		2	SS	14		7 17 46 30
		3	SS	14		
	A Stiff to Hard	4	SS	29	310	
	Red-Brown	5	SS	68		
		6	SS	33	300	
295.4		7	SS	100/4"		
29.3	End of Borehole					

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE

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

WATER CONTENT %: 10 20 30

BULK DENSITY γ P.C.F.

FORM OE-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 9

FOUNDATION SECTION

JOB 69-F-62
LOCATION Co-ords. 730,522 N; 897,167 E.
ORIGINATED BY GA

W.P. 113-63-01
BORING DATE Sept. 24, 25, 1969
COMPILED BY GA

DATUM Geodetic
BOREHOLE TYPE Washboring, NX Casing
CHECKED BY [signature]

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % (10, 20, 30)	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
307.9	Ground Level									
0.0	Black cinder fill									
304.0	with some gravel									
302.0	Clayey silt with shale		1	SS1	52/7"			o		
5.9	weathered		2	SS2	bounc.	300	150	o		
	Shale									
293.5	Sound Bedrock		3	RC	80%					
14.4	End of Borehole									

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

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 10

FOUNDATION SECTION

JOB 69-F-62
W.P. 113-66-01
DATUM Geodetic
LOCATION Co-ords 730,538 N; 897,103 E.
BORING DATE Sept. 26 & 29, 1969
BOREHOLE TYPE Washboring NX & BX Casing
ORIGINATED BY GA
COMPILED BY GA
CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT
308.4	Ground Level			
0.0	Clayey silt with shale fragments	1	SS	100
304.0 / 4.4	Weathered Shale	2	SS	101/1½"
		3	SS	bounce 300
	Sound Bedrock	4	RC	90%
294.6 / 13.8	End of Borehole			

Column headings: SOIL PROFILE (ELEV. DEPTH, DESCRIPTION, STRAT. PLOT); SAMPLES (NUMBER, TYPE, BLOWS / FOOT); ELEV. SCALE; DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE; LIQUID LIMIT — w_L, PLASTIC LIMIT — w_P, WATER CONTENT — w; WATER CONTENT % 10 20 30; BULK DENSITY γ P.C.F.; REMARKS GR. SA. SI. CL.

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

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 11

FOUNDATION SECTION

JOB 69-F-62 LOCATION Co-ords. 730,540 N; 897,050 E. ORIGINATED BY GA

W.P. 113-66-01 BORING DATE Sept. 19, 22, 23, 24, 25, 1969 COMPILED BY GA

DATUM Geodetic BOREHOLE TYPE Washboring, NX & BX Casing CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
324.6	Ground Level				
0.0	Clayey silt, some sand and large fragments of shale	1	SS	18	320
		2	SS	9	
		3	SS	bouncing	
309.6		4	RC	40%	310
15.0	Shale Bedrock (Weathered / Sound)	5	SS	119/6"	
		6	SS	50/1½"	
		7	SS	bouncing	300
294.6		8	RC	90%	
30.0	End of Borehole				290

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB. VANE

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

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS GR. SA. SI. CL.

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

TYPE OF SAMPLE

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

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

π = 3·1416
e BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$ NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$ LOGARITHM OF a TO BASE 10
t TIME
g ACCELERATION DUE TO GRAVITY
V VOLUME
W WEIGHT
M MOMENT
F FACTOR OF SAFETY

STRESS AND STRAIN

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

EARTH PRESSURE

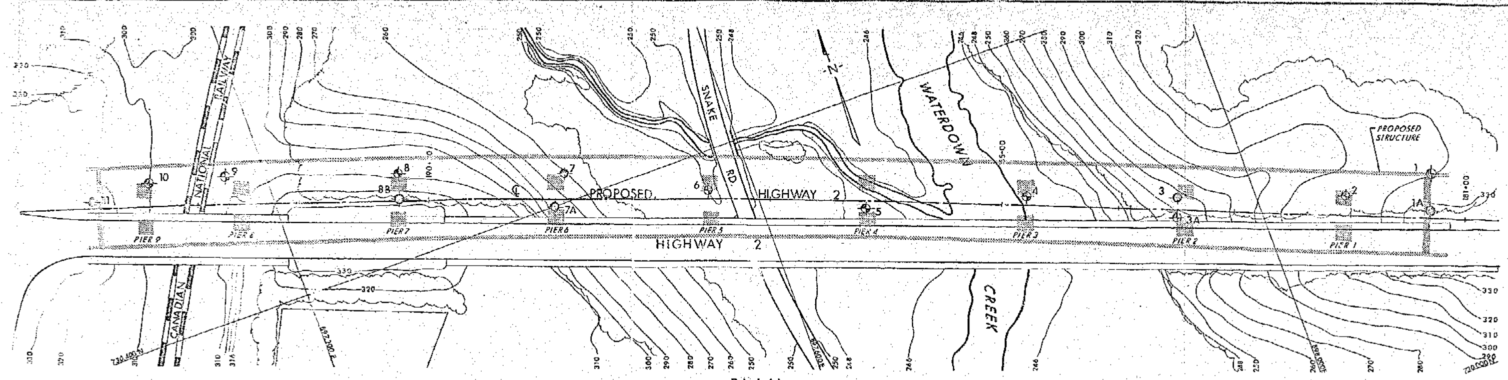
d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ ANGLE OF WALL FRICTION
K DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

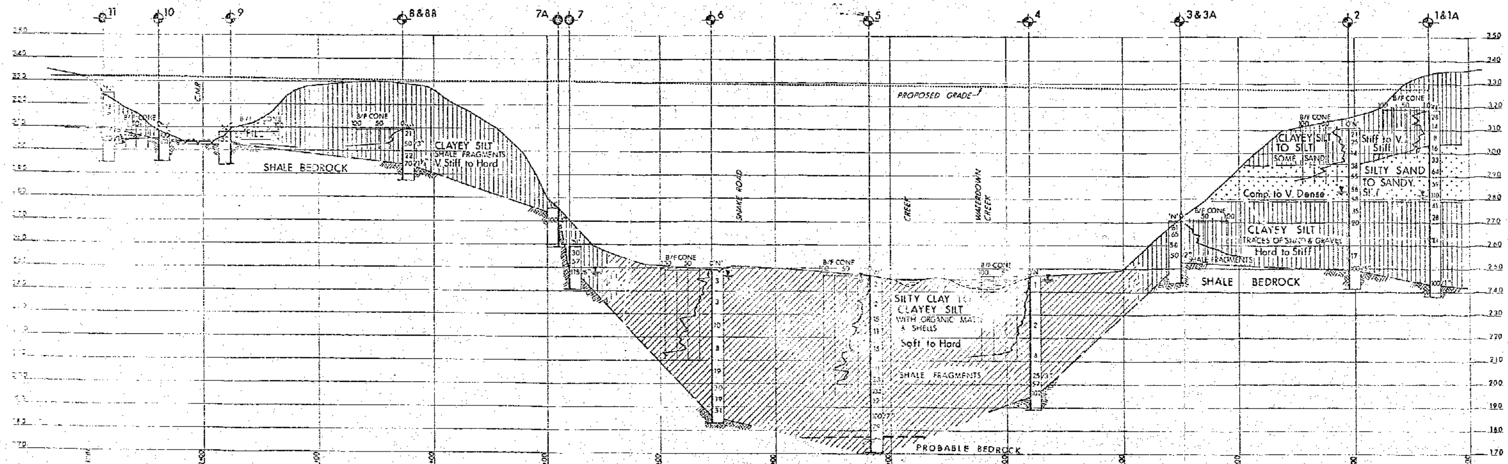
SLOPES

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



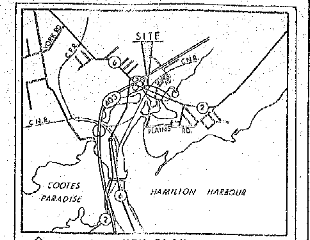
PLAN

SCALE 40 20 0 40 80 FT.



PROFILE

SCALE VERT. 20 10 0 20 40 FT.
HORIZ. 40 20 0 40 80



KEY PLAN

SCALE IN MILES

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. SEPT. 1969

NO	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	318·9	730,174	898,150
1A	329·0	730,143	898,140
2	311·0	730,179	898,073
3	281·6	730,225	897,935
3A	270·7	730,205	897,930
4	246·8	730,270	897,816
5	247·3	730,306	897,665
6	249·5	730,367	897,561
7	259·0	730,425	897,447
7A	275·6	730,399	897,429
8	309·4	730,478	897,309
8B	324·7	730,452	897,303
9	307·9	730,522	897,167
10	306·4	730,538	897,103
11	324·6	730,540	897,050

- NOTE -

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

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING OFFICE – FOUNDATION SECTION

WATERDOWN CREEK & C.N.R.

KING'S HIGHWAY NO. 2 RE-LOCATED DIST. NO. 4
CO. HALTON
TWP. WENTWORTH LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D A.B.	CHECKED	W.P. NO. 113-66-01	W.B.T. DRAWING NO. 69-F-62 A
DRAWN S.O.	CHECKED	JOB NO. 69-F-62	
DATE 9 OCT. 1969		SITE NO	BRIDGE DRAWING NO.

PILE DATA

LOCATION	BATTER	NO.	LENGTH	TYPE
E. ABUT.	1:4	14	75	HP 12 × 74
	1:10	4	75	HP 12 × 74
PIER A	1:6	38	35 40	HP 12 × 74
	0	16	35 40	HP 12 × 74
PIER C	1:6	44	65	HP 12 × 74
	0	18	65	HP 12 × 74
PIER D	1:6	44	65	HP 12 × 74
	0	18	65	HP 12 × 74
PIER E		3	64	3'0" DIA 3/8" THICK
CAISSONS		1	58	"
		3	54	"
		2	46	"
		1	41	"
		2	38	"
		1	31	"
		2	28	"
		2	18	"
		1	9	"
W. ABUT	1:3	12	25	HP 12 × 74
	1:10	4	25	HP 12 × 74

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. A. G. Stermac,
Principal Foundation Engineer,
Room 107,
Lab. Building.

ATTENTION: Mr. K. Selby

OUR FILE REF.

FROM: G. C. E. Burkhardt,
Bridge Planning Section,
Central Building.

DATE: June 1, 1971.

IN REPLY TO

SUBJECT: W.P. 113-66-03, Site 10-125,
Hamilton Entrance Bridge #3,
Highway 2 & 6, District 4.

Herewith are two prints of drawing D-6965-P1 showing the span arrangement for the proposed structure.

Please arrange for additional borings at the location of the piers so that we may proceed with the design.

W. M. Killin

W. M. Killin,
for:
G. C. E. Burkhardt
REG. BRIDGE PLANNING ENGINEER.

WMK:lc
Encl.

JOB GIVEN TO WATROLO
JUL 7/71
AGS

Department of Highways Ontario
Copy for the information of

Foundation Office.

~~Mr. A. Stermac,~~
Principal Foundation Engineer,
Foundation Office,
Central Building.

C. S. Grebski,
Structural Office.

November 4, 1971.

Hamilton Entrance Bridge #3,
W.P. #113-66-03, Site #10-125,
Highway #2 & #6, District #4.

Trow /71

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

Kindly give us your comments at your earliest convenience.

C. S. Grebski,
Structural Design Engineer.

CSG/mh
ENCL*
cc: Foundation Office.

9 Nov 71

pile requirement for Pier #3 appears to be some 1000 - 1200 ft less than our estimate.
Pile lengths for Pier #2 however might be somewhat over estimated (appr. 300 ft longer than our estimate)

[illegible]

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

C. S. Grebski
Structural Design Engineer
Design Services Branch
West Bldg.

ATTENTION:

FROM: Foundation Office
Design Services Branch
Central Bldg. Room 107

DATE: December 13, 1971

OUR FILE REF.

IN REPLY TO

SUBJECT: W. P. 113-66-01 Hamilton Entrance Bridge #1
W.P. 113-66-03 Hamilton Entrance Bridge #3

Due to the sloping nature of the bedrock surface at the location of Pier E, Bridge #1, and the South Pier of Bridge #3, we recommend that these footings be supported on cast insitu concrete caissons installed with permanent steel liners and socketed into sound bedrock. For 36 inch dia. concrete caissons with 5 ft. sockets at Bridge #1 and 8 ft. sockets Bridge #3 a design capacity of 500 tons per caisson should be achieved.

We recommend that the work be written in the contract as three separate items and for your information and use we have prepared the necessary special provision which we believe will reduce the possibility of unreasonable claims by a contractor. These are as follows:

1. <u>SUPPLY ALL EQUIPMENT NECESSARY TO INSTALL CAISSONS:</u>

Under this item the Contractor shall supply all equipment necessary to install 36 inch diameter concrete caissons, caisson reinforcing steel and permanent steel liners as shown on the Drawings. Rock core samples recovered during the foundation investigation are available for inspection at the D.T.C. Laboratory in Downsview. Contractors are advised to inspect these samples prior to bidding in order to satisfy themselves as to the type and quality of the rock.

..........2

2. <u>INSTALL CAISSONS IN OVERBURDEN:</u>

Under this item the Contractor shall provide all materials and shall carry out all work necesssary to install 36 inch diameter concrete caissons including 36 inch i.d. permanent liners and reinforcing steel within the overburden as shown on the drawings. Suitable holes to ensure a snug fit of the permanent liners shall be drilled to the bedrock surface. As the drilling proceeds the steel liners shall be installed in the holes and advanced so as to prevent cave in of the walls of the holes. When the holes have been drilled and lined to the bedrock they shall be unwatered and bases visually insepcted by lowering a man into the hole to confirm the presence of bedrock. The holes shall then be advanced within the bedrock to a depth of five (or eight) feet below the surface of the sound bedrock as determined by the Engineer. The steel liners shall then be advanced to an elevation one foot below the sound bedrock surface as determined by the Engineer. The Contractor shall then unwater the holes and shall remove all soil or broken rock from the bases of the holes and shall provide facilities for inspection by the Engineer. After the holes have been inspected and found to be satisfactory by the Engineer, the reinforcing steel shall be placed and the concrete poured. Concrete shall be placed in the caissons in the dry and in sufficiently small quantities so as to prevent the formation of voids and shall be compacted by vibrating continuously from the bottom upwards. The ℄ of each caisson installed shall not deviate from that shown on the drawings by more than 2% of the distance between the point considered and the top of the caisson.

For purposes of these special provisions the bedrock surface is defined as the actual surface of the bedrock, weathered or unweathered as it exists, immediately below the overburden, and the sound bedrock is defined as that portion of the bedrock which is unweathered and is considered to be structural sound.

All materials supplied by the Contractor must be approved by the Engineer.

Payment for this item shall be unit price per lin.ft. and the payment quantity for each caisson shall be the distance in lin. ft. along the caisson ℄ between the caisson top as shown on the Drawings and the bedrock surface encountered within the caisson.

....3

3. INSTALL CAISSONS IN BEDROCK:

Under this item the Contractor shall provide all materials and shall carry out all work necessary to install 36 inch diameter concrete caissons including 36 inch i.d. permanent steel liners and reinforcing steel, within the bedrock, as shown on the Drawings. Procedures to be followed are as described in the S.P. for Item No. . Payment for this item shall be unit price per lin. ft. and the payment quantity for each caisson shall be equal to the distance in lin. ft. along the caisson ℄ between the bedrock surface and the base of the caisson.

For estimating purposes we would advise you that we consider the average depth of weathered rock at Bridge #1 to be 3 feet, and at Bridge #3 to be 5 feet. We suggest that you ammend the foregoing special provisions so as to be in accordance with normal Department procedures.

We would be pleased to discuss the foregoing with you at any time.

K. G. Selby

KGS:mt

K. G. Selby
Supervising Foundation Engineer

cc: W. Lin
B. Richardson
M. Stoyanoff

Mr. L. Francis,
Structural Office,
West Building.

Structural Services Section,
West Building.

January 10, 1972.

Hamilton Entrance Bridge #3,
Hwys. 2 & 6, Dist. 4,
Site 10-127, W.P. 113-66-03.

We note that on the bridge drawings it is specified for Pier 3 that

"The Contractor shall demonstrate to the satisfaction of the Engineer that each anchor can resist an uplift of 50 tons."

We wish to advise that this statement is very broad in nature and the Contractor may have some trouble determining what is required. This could result in a claim for extra work.

In addition there is insufficient information on the drawings for the scheme shown to enable the Contractor to bid properly.

We would suggest that a firm scheme be detailed on the drawings permitting an equivalent alternative if allowed.

We presume that a pull out test is intended although not specified. If so, then a tender item should be set up at a unit price per test with special provisions describing what is to be done.

It is our understanding that Mr. K. Selby of the Foundation Section has been involved with this proposal, and could provide you with the necessary specifications for a pull out test.

Any pertinent information to be included in the contract should be forwarded as soon as possible.

M. Stoyanoff
Structural Contract Engineer

MS/js

c.c. C. Grebski
K. Selby

Department of Highways Ontario
Copy for the information of
K. Selby

M. Stoyanoff,
Structural Services Section,
West Building.

Structural Office,
West Bldg., Downsview.

January 13, 1972.

Hamilton Entrance Bridge #3,
Hwy's. 2 and 6, District #4,
Site 10-127, W.P. 113-66-03.

Re: Your memo dated January 10,1972.

We enclose drawing D-6965-9 incorporating suggestions outlined in your above-mentioned letter. Special Provisions covering this item should be included in the Tender Documents.

L. Francis,
Bridge Project Engineer.

LF:sr
Encl.

c.c. C.S. Grebski
B.S. Richardson
K. Selby

K. Selby

Box 5020, Burlington
September 13, 1972

MINUTES OF SITE MEETING NO. 3
CONTRACT 72-119
HELD SEPTEMBER 12, 1972

Attending:-

Ministry of Transportation and Communications
D. A. Waller
K. Saarits
R. Jasper
D. Hamlin
N. Ilton
K. Selby

Stead & Evans Limited
R. White
J. Noy

Franki Canada Limited
H. J. Vander Noot
R. Millard
N. Stobodian

Bermingham Construction Limited
E. Bermingham
R. McArthur

William Trow Associates Limited
F. Anderson

BUSINESS ARISING FROM PREVIOUS MINUTES

1. Telegraph lines at Bridge #1, Pier "H" cannot be relocated before September 21st according to the latest information available from Mr. Isner of C. N. Telegraph.

2. The position of the new abutments on Bridge #3 is still under examination and bore holes will be placed next to the existing abutment in order to determine exact elevations. The contractor is asked to provide equipment for this work and in order to proceed with our final determination it is essential that broken concrete slabs at the south abutment be removed as soon as possible and a ramp constructed to the site in order that we can bring in augering equipment.

continued/2

3. The contractor is proceeding with the drilling of rock anchors in Pier 1, Bridge #3 and expects to have the first set of anchors ready for testing on September 14th.

Please note correction:- Change Minutes of Site Meeting No. 1 to read No. 2.

NEW BUSINESS

1. Mr. White questioned the fact that since a considerable amount of the excavation from structure foundation will not be used as structure backfill but will instead be used in construction of fills and since under Form 9 provision is made for payment of compaction and under Form 200 (ordinary fill construction) payment for compaction is included in the excavation price, the contractor wishes to receive payment for compacting material which was taken from structure foundations.

   It is our present opinion that the contractor's assumption is correct and our Project Supervisor is therefore directed to record the compaction times involved and include this for payment as provided under Form 527.

2. The top of rock elevation on Footing "J", Bridge 1 has now been determined and R. Jasper can point this out to J. Noy.

3. The contractor intends to proceed with the retaining wall footings with Section 2 being poured on September 12th and 3 and 4 later this week. The forming of Section 2 wall should start on September 14th.

4. The contractor intends to drive piles on Footing "C" and "D" and then "A" and pour the Footings "J" and "A".

5. Construction of caissons - Pier "E", Bridge #1. A lengthy discussion of the method of construction and the intention of our design was carried out between the contractor and Ministry personnel with no firm decision having been reached. The major point brought forth by the contractor was that he felt it would be much simpler for him to pour the bottom of the caissons as tremie concrete rather than try de-watering these. According to the Ministry's interpretation of the plans and specifications, however, tremie concrete is not permissible and since our interpretation of the term "sound bedrock", which is the determining factor of the depth of penetration of the caisson liner, does not agree with the contractor's interpretation of the same term.

continued/3

A further meeting is to be carried out on September 13th at 9:00 A.M. at the construction site. This meeting will be attended by representatives from Steed Evans Limited, Bermingham Construction Limited, Franki Canada Limited, William Trow Associates Limited and M.T.C. Bridge Office.

K. Saarits
Construction Supervisor

KS:cdk

c.c. All Attending

ONTARIO

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

MINISTER: HONOURABLE GORDON CARTON, Q.C. DEPUTY MINISTER: A.T.C. McNAB

Box 5020, Burlington
September 14, 1972

MINUTES OF SITE MEETING NO. 4
CONTRACT 72-119
HELD SEPTEMBER 13, 1972

Attending:-

Ministry of Transportation and Communications

P. McWatt	K. Saarits
K. Selby	R. Jasper
W. Lyn	D. Hamlin
F. Gormek	N. Ilton

William Trow Associates Limited
C. Thompson
P. Anderson

Franki Canada Limited
A. Millard
N. Stobodian

Steed & Evans Limited
R. White

Caissons, Bridge #1, Pier "E"

As a result of difficulties encountered by Franki Canada Limited in constructing the caissons this meeting was called in order to discuss these problems and seek possible solutions.

The contractor originally augered the caissons #1, 3 and 5 approximately 3 feet into bedrock and then proceeded to drill or chip the rock out in advance of the casing. This method of construction, however, would not permit the casing to be driven deep enough and it was felt that the major lack of the casing penetration was caused by the size of the hole beneath the casing.

On caisson #5 the contractor has changed to a larger rock bit (34") and is now able to drive the casing down approximately another 5 feet. According to the construction drawings D6964-5 the casing is supposed to penetrate approximately 1 foot into sound bedrock. According to the inspection of the caissons, however, it is found that this penetration has not been obtained and therefore a considerable amount of water is entering into the caisson.

continued/2

It is the contractor's opinion that even if the casing was driven further it would still not completely seal off the water from entering into the caisson and he therefore feels that he could not construct the caissons in a manner which would permit the pouring of concrete in the dry as required by the specifications. Mr. A. Millard is therefore proposing that tremie concrete would be used instead and he feels that he should have no difficulty in obtaining the 5,000 P.S.I. for the concrete in caissons. Mr. White stated that he would not be looking for additional payment for this work but he felt that he could not accept any responsibility for this change other than accepting responsibility for the quality of construction.

In view of this statement by the contractor the M.T.C. requested that the contractor submit his proposal in writing in order that the various points can be considered. The Ministry feels that our plans and specifications are quite clear in outlining what we wish to obtain and if the contractor feels that he cannot build according to our plans then it is up to him to submit a proposal to the M.T.C. for approval.

Bridge #3

R. White advised that his tressel construction operations are encountering old timber piles at Bridge #3, Pier #3. These piles appear to be at the bottom of the lagoon in clusters and he is afraid when he commences his pile driving operations for the footings, interference from the timber piles could occur and create serious problems. Since the problems which might occur cannot be predetermined at this time it is the Ministry's opinion that nothing can be done until the contractor commences with his pile driving operations at which time each individual problem will have to be investigated on it's own merits.

Mr. White requested that all mail directed to Contract 72-119 should be sent to Steed & Evans Limited, 1300 Plains Road West, Burlington, Ontario.

K. Saarits
K. Saarits
Construction Supervisor

KS:cdk

c.c. All Attending

Mr. C.H. Robertson,
District Engineer,
Hamilton.

Mr. P. McWatt.

Mr. D.A. Waller.

September 19, 1972.

W.P. 113-66-01 & 03,
Sites 10-127 and 10-125,
Contract 72-119,
Hamilton Entrance Bridges Nos. 1 & 3,
Highway No. 2 & 6, District 4.

This is in answer to the District memo of 15th September, and in particular the Contractor's submission requesting that he be allowed to use tremie concrete in the caissons rather than concrete to be placed in the dry as called for in the specifications. Our answer is as follows:-

(1) We do not agree that the caissons cannot be placed in the dry but we are sympathetic to the problem presented by the inflow of water.

(2) Under the circumstances we are prepared to go along with the Contractor's suggestion that tremie concrete be used. Where the water inflow is sufficiently low to permit placing of concrete in the dry the Contractor should adhere to the specifications. The Consulting Inspector to determine this condition.

(3) The tremie concrete mix supplied by the Contractor seems satisfactory except that we do not agree to the use of any air entrainment.

(4) The Contractor should note there is no change in design and that the caisson still must carry a design load of 500 tons. This means that the rock must be competent to take the load and that the concrete must test out at 5000 p.s.i. in compression.

(5) It is not expected that the testing of the tremie concrete will require any additional time over the testing of concrete placed in the dry. It is anticipated that cores will be taken from the caissons, exact details of the testing will come from our Materials and Testing Section.

continued..../2

(6) Concrete not coming up to the necessary strength requirements will possibly need further strength tests and additional caissons may be required. The Contractor would be considered to be responsible in this situation.

(7) We recommend that W. [illegible] Associates have their contract, for inspection of the rock, extended to include supervision of the tremie concrete operation.

(8) We note that the Contractor will not be making any claims in the suggested change from placing concrete in the dry to tremie concrete.

I trust that this includes all the points discussed at the meeting in Downsview on Friday 15th September, with the following people present:

C. [illegible]	Structure Design
H. Lip	" "
K. Selby	Foundations
A.E. [illegible]	Construction - Structures
P. McWatt	" "
F. Wilson	Senior Materials Engr., Concrete
J. Ryell	Research
E. Szarits	District 4 Construction
R. Jasper	" "

P. McW.

PM/jc

P. McWatt,
Bridge Construction Engineer.

c.c. K. Szarits
D. Davis
J. Ryell
F. Wilson
K. Selby

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

A&S

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

Copy for the information of

A. STERMAC

Box 5020, Burlington
September 26, 1972

RECEIVED
FOUNDATIONS OFFICE
OCT 2 1972
Ministry of Transportation & Communications

Wm. Trow Associates
1870 Barton Street
Hamilton 31, Ontario

Attention: C. D. Thompson, P. Eng.

Re: Caisson Inspection
Hamilton Entrance Bridges Nos. 1 and 3
Contract 72-119

Dear Sir:

This is further to your confirming letter of September 5, 1972 in reply to our telephone conversation.

We would like to confirm instructions given to your field staff to expand your inspection services to cover the complete caisson installation including any necessary testing, with the exception that the Ministry will take concrete cylinders, arrange for necessary coring and establish strengths of same.

Yours truly,

D. A. Waller, P. Eng.
District Construction Engineer

DAW:lc

c.c. A. Stermac

W.P. 113-66-01, 03, 04
68-F-78
Trow/71

Design Services Branch,
1201 Wilson Ave.,
Downsview 464, Ontario.

October 5, 1972.

Telephone: 248-3282.

H. Q. Golder & Associates Ltd.,
3151 Wharton Way,
Mississauga, Ontario.

Gentlemen:

This is to confirm our telephone conversation of October 4, 1972, regarding the hiring of the services of Mr. Roy Van Ryswyk of your organization.

Mr. Van Ryswyk will review the rock conditions as exposed at the site of Pier F of Bridge No. 1 - Hamilton Entrance, Hwy. 2, Contract No. 72-119, and will submit his findings, conclusions and recommendations in a letter form to this Office.

It is agreed that you will invoice this Ministry for the services of Mr. Van Ryswyk based on rates suggested by the Association of Professional Engineers of Ontario.

Yours truly,

AGS.

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ao

cc: Foundations Files ✓
Documents

Golder Associates

CONSULTING GEOTECHNICAL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
J. L. SEYCHUK
C. O. BRAWNER
D. L. TOWNSEND

F. J. HEFFERNAN
B. E. W. DOWSE
J. B. DAVIS

October 6, 1972

Ministry of Transportation
and Communications,
Design Services Branch,
1201 Wilson Avenue,
Downsview 464, Ontario.

Attention: Mr. K. G. Selby, P.Eng.

RE: HAMILTON ENTRANCE BRIDGE #1
NORTH HALF PIER F FOOTING
W. O. 69 - 11062
CONTRACT 72 - 119

Dear Sirs:

This letter reports the results of a visit made to the above site on October 4, 1972 by our Mr. R. Van Ryswyk accompanied by your Messrs. K. Selby and R. Jasper. The purpose of the visit was to inspect a crack observed in the bedrock during excavation of the north half of the footing excavation for Pier F.

The footing excavation is located adjacent to and immediately north of an existing bridge pier. At this location, the ground surface slopes down at a slope of about 2 horizontal to 1 vertical from the existing bridge to a creek valley which parallels the existing road. At the footing, the ground surface is underlain by some 10 to 15 ft. of overburden followed by bedrock; the surface of which approximately parallels the ground surface.

At the time of the site visit, the footing excavation had been completed to a depth of 5 to 15 ft. below the bedrock surface. The excavation bottom was

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stepped to conform generally to the bedrock surface profile. Because the bedrock (alternating bands of red and green horizontally bedded shale) could only be excavated on slabs equal in thickness to the shale beds, the excavation extended a few feet below the design founding level.

It is understood that on October 3, 1972, following excavation for the footing, a large open crack was observed in the bedrock. This crack extended down one face of the excavation and crossed the base of the excavation at an angle of about 65 deg. to the pier centreline. As this crack had not been observed during excavation (this does not necessarily mean it was not present at that time). There was some concern about the overall stability of the founding rock.

Observations made during the course of the site visit indicate that the crack presently under consideration is one of a set of joints predominating within the shale. The joints are spaced at about 10 in. to 2 ft. intervals, have a strike of 135^o azimuth and dip at 80^o to 85^o towards the northeast. The edges of the steps on the excavation bottom (perpendicular to the pier centerline) are oriented at 110^o azimuth, making an angle of 25^o with the crack which crosses the middle step. The original hill side runs at approximately 120^o azimuth and slopes northeast towards the tributary-ravine floor. The crack can be followed up the west wall of the excavation for a height of about 5 ft. above the middle step with an opening of as much as 1/2 in. The joint is then discontinuous for about 4 ft., but at a point 9 ft. above the middle step, the joint is again open approximately 1/4 in. The joint terminates about 12 ft. above the middle step.

The crack has an oxidized or rust coating similar to that of the other joints. However, no recent gouge filling is evident, and no other indication of shearing (slickensides or bedding displacement) are present.

Across the excavation floor, the crack narrows from 1/2 in. open at the west wall to completely closed at the east wall. A second joint 10 ft. to the northeast of the main crack shows slight opening only across the floor of the middle step.

The shale was tested with dilute hydrochloric acid and fizzed vigorously. This indicates a high content of calcium carbonate cement and thus the shale could be broken down gradually under the action of flowing groundwater.

Sections plotted from borings put down in the area of the footing indicated that the bedrock surface slopes at between 20° and 30° to the horizontal. However, on the basis of available boring information, the geologic history of the area and from the existing topography it is not possible to project these sections more than about 10 ft. north of the excavation. It is unlikely however that the bedrock surface would slope away steeper than about 45° to the horizontal.

Following discovery of the crack on October 3, nails were installed in the excavation floor on both sides of the crack. At the time of the site visit (October 4) no movement across the crack had been detected by measuring between these nails.

Based on the results of the site visit, it is our opinion that the crack observed in the footing excavation is not indicative of a potential major instability of the foundation rock. However, as a precaution, we suggest that a boring be put down immediately adjacent to and some 50 ft. northeast of the footing excavation to confirm that the bedrock surface does not dip down excessively steeply.

Assuming the bedrock surface does not dip steeper than about 45°, we recommend that the following construction procedure be adopted.

1) Fill the crack by gravity grouting (no pressure) where the cracks are exposed on the floor and wall of the excavation.

2) Take a final reading on the monitoring nails immediately prior to covering the nails with concrete to confirm that no displacement has occurred.

3) Place backfill concrete as soon as possible to the proposed founding level.

4) The slope to north and east of the excavation should be maintained at present angle or at a flatter angle (i.e. place waste material at the toe of the slope rather than on the upper part of the slope).

5) Place brass benchmark pins at the upper corners of the concrete footing immediately after pouring and take regular elevation readings.

We trust this letter will be adequate for your immediate purposes. If you have any questions regarding this letter or if we can be of any further service to you on this project, please call us.

Yours truly,

H. Q. GOLDER & ASSOCIATES LTD.

R. J. van Rijswijk

for J. B. Davis, P.Eng.

RvR:JBD:jb
72122

FRANKI

CANADA LIMITED

CANADIAN CONSTRUCTION ASSOCIATION

TELEX NO.
02-21159
CABLEGRAMS
"FRANKITOR"
TELEPHONE:
751-4200

105 NANTUCKET BLVD.

SCARBOROUGH, ONT.

Our Reference:
C. 4170

November 2, 1972.

Mr. R. White,
Stead & Evans,
1300 Plains Rd. W.,
Burlington, Ontario.

Re: M.T.C. Contract #72-119
Hwy. #2 Relocation - Hamilton
Grouting of Cored Caisson
Bridge #1

Dear Sir:

This will confirm verbal agreements on the site with Mr. C. Thompson of Wm. Trow Associates Limited and Mr. R. Jasper of the M.T.C. regarding the procedures and equipment to be used to grout the cored caisson in Bridge #1.

As the concrete core and the video camera indicates that the porous concrete at the joint exists on only one side, we will first probe the core hole to determine on which side to drill a vent hole. We will then drill a vent hole approximately 1-1/2" in diameter at the outer edge of the caisson down into the zone to be grouted.

The actual grouting will be done using our Colcrete Collaidal Grout Pump Model #DD8. A grout hose will be inserted to the bottom of the cored hole and grout pumped in till all the water has been displaced, both in the cored hole and the porous zone. The drilled vent hole will allow trapped water to escape, and will also confirm that the grout has penetrated throughout the porous zone.

We will also grout the cored hole of the second caisson that was cored on Bridge #1.

Yours very truly,

FRANKI CANADA LIMITED,

V. Slobodian.

VS:AL
C.C.--Bermingham Construction Ltd.
Mr. A. G. Stermac, Principal Foundation Engineer, M.T.C.