

MEMORANDUM

To: Mr. E. R. Davis,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie

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

Date: November 15, 1966

Our File Ref.

In Reply To: **NOV 18 1966**

Subject:

FOUNDATION INVESTIGATION REPORT
At the Proposed Crossing of
Queen Elizabeth Way and Tufford Rd.,
County of Lincoln
District #4 (Hamilton)
W.J. 66-F-83 -- W.P. 220-63

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

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please feel free to contact our Office.

AGS/MdeF
Attach.

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

Foundations Office
Gen. Files ✓

Afternoon
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

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FOUNDATION INVESTIGATION REPORT
At the Proposed Crossing of
Queen Elizabeth Way and Tufford Rd.,
County of Lincoln
District #4 (Hamilton)
W.J. 66-F-83 -- W.P. 220-63

1. INTRODUCTION:

In a memo, dated July 12, 1966, the Regional Bridge Location Engineer requested a foundation investigation, to be carried out at the proposed site of Queen Elizabeth Way and Tufford Road.

Accordingly, a field and laboratory investigation was undertaken by the Foundation Section, the results of which are presented in this report, together with recommendations pertaining to the foundations.

2. DESCRIPTION OF THE SITE:

The site is located in the County of Lincoln, some 6 miles east of Grimsby. The general area is known as the Niagara Fruit Belt, which is part of the Iroquois Plain physiographic region. The plain is cut by a number of smaller streams; all of them are drowned in their lower courses, producing lagoons or marshes cut off from Lake Ontario by a barrier beach.

From Grimsby east, the lake plain contains areas of sandy soils which have enabled the area to become an outstanding fruit-growing region. As the beds of sand are never very deep and often overlie clay at two to three feet, drainage is usually a problem.

cont'd. /2 ...

3. FIELD AND LABORATORY INVESTIGATION:

3.1) Seven boreholes and six dynamic cone penetration tests were carried out during the recent field work. Borehole #15, drilled during the preliminary soil study, is also incorporated in this report. A conventional diamond drill, adapted for soil sampling was used for the investigation. Samples were recovered by means of a split-spoon sampler, which was driven by utilizing a driving energy of 350 ft.-lbs. The number of hammer blows for a penetration of one ft. were recorded as Standard Penetration 'N' values. Rock samples were obtained by means of an AXT diamond core barrel.

Locations and elevations of the borings, together with the soil profile projected to the centre-line of the bridge, are shown on Drawing #66-F-83A, attached to this report.

3.2) In the laboratory, soil samples were visually examined and identified. Laboratory tests of liquid and plastic limits, also natural moisture content determinations, were performed together with a few grain-size analyses.

Field and laboratory test results are plotted on the borelog sheets under Appendix I.

4. GROUNDWATER CONDITIONS:

Groundwater level was established in each borehole, between ground elevation and a depth of 10 ft. below ground, corresponding to elevations 268 - 279.

5. SUBSOIL CONDITIONS:

5.1) General:

Boreholes placed at or near the proposed footing locations revealed an approximately 12-to 24-ft. deep overburden followed by shale bedrock. The overburden was identified to be of glacial origin, and might be separated to cohesive and granular soils. A brief discussion of the various strata is given as follows:

5. SUBSOIL CONDITIONS: (cont'd.) ...

5.2) Clayey Silt with Sand and Gravel:

This material was found in each boring, except in borehole #3. The depth of the layer varies between six and twelve ft., terminating around el. 265 - 272. The consistency of the material was found to be hard, corresponding to penetration 'N' values ranging from 49 blows/ft. to much in excess of 100 blows/ft.

5.3) Silty Sand to Sandy Silt with Gravel:

Underlying the clayey silt in boreholes #4, 5, 6, and 7, and right below ground level in boreholes #3, the silty sand to sandy silt (granular nature of glacial till) was encountered. In boreholes #1 and 2, this layer was not observed. The stratum exhibits very dense relative density, indicated by the extremely high standard penetration resistance.

5.4) Bedrock:

Between el. 254 and 266 ft., shale bedrock follows the glacial till overburden. The upper portion of the bedrock was observed to be weathered. In certain locations the effect of weathering could be noticed as deep as 15 - 20 ft. below rock surface. A 25-ft. thickness of the shale bedrock was proved by diamond drilling.

6. DISCUSSION AND RECOMMENDATIONS:

The proposal at this site calls for a 6-span bridge with approach fills of about 20 - 21 ft. in height. It is understood that the grade of the future Q.E.W. will roughly be the same as the existing one.

The hard and very dense glacial till overburden appears to have sufficient strength to support the proposed structure on a spread footing type of foundation. It is recommended that the footings be placed at el. 273, some 5 ft. below the existing grade

6. DISCUSSION AND RECOMMENDATIONS:

of Q.E.W. At this elevation a safe load of 3.5 t.s.f. may be used for design purposes.

The footings of the abutments may be placed within the approach fills and be supported on steel H-piles, driven to practical refusal. For estimating purposes, it is assumed that refusal will be reached around el. 254 - 260 within the weathered shale bedrock. The maximum allowable load for the particular H-section used may be employed on the piles, driven according to the above recommendations.

Care should be taken to ensure that no bouldery fill is placed at the locations through which piles have to be driven.

As an alternative, the abutments can be supported on spread footings founded on granular fill using a safe bearing pressure of 2 t.s.f. The fill material below the tops of the footings should consist of well compacted G.B.C. Class 'A' material and should extend for a horizontal distance of at least 10 ft. from the footing edges in the plane of the footing tops. This portion of the fill should be built with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for a distance of about 50 ft. behind the abutments before re-excavating for the abutment footings. In this case, it is recommended that the approach fills be constructed as early as possible prior to the construction of the structure footings.

No major dewatering problems are anticipated for the footing excavations, due to the relatively low hydrostatic head above the excavation bottoms, and due to the low permeability of the clayey silt.

The approach embankments will be stable, provided that they are built with standard slopes of 2 horizontal to 1 vertical.

cont'd. /5 ...

7. SUMMARY:

The foundation report of the proposed crossing of Q.E.W. and Tufford Rd. is presented.

The soil stratigraphy at the site consists of hard and very dense glacial till deposits followed by shale bedrock. Spread footings are recommended for the proposed structure, to be placed at el. 273 ft., some 5 ft. below existing grade. 3.5 t.s.f. allowable pressure may be imposed on the footing bases.

The abutments may be supported on steel H-piles; pile caps to be constructed within the approach fills. Piles should be driven to practical refusal, which is expected to be reached between el. 254 and 260, within the weathered shale. The maximum allowable load for the section used may be assumed on the H-piles for design purposes. As an alternative, the abutments can be supported on spread footings founded on granular fill using a safe bearing pressure of 2 t.s.f.

No dewatering problems for the excavations, and no stability problems for the approach fills are anticipated.

8. MISCELLANEOUS:

The field investigation, carried out during the period September 16 - 22, 1966, was supervised by Mr. V. Korlu, Project Foundation Engineer. Equipment used was owned and operated by Johnston Drilling Company Limited.

This report was written by Mr. A. K. Barsvary, Senior Foundation Engineer. The project was under the general supervision of Mr. M. Devata, Supervising Foundation Engineer.

November 1966

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-E-83 LOCATION QEW-Tufford Rd. Sta. 31+66 14' Rt. ORIGINATED BY Vk
W.P. 220-63 BORING DATE September 16, 1966 COMPILED BY VK JM
DATUM Geodetic BOREHOLE TYPE Drive-Drill BX casing and wash CHECKED BY HR

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WP	W	WL		
277.7	Ground level															
	Clayey silt to silt with fine sand and occasional gravel -grey or brown -hard or very dense		1	SS	140											
			2	SS	159/9"	270										
			3	SS	100 for 5"											
265.7			4	SS	130											
12.0			5	SS	139											
	Shale (weathered)		6	SS	100 for 5"	260										
			7	SS	100 for 4"											
			8	SS	100 for 5"											
				AXT	50%	250										
				AXT	60%											
244.7																
33.0	(sound)			AXT	100%	240										
239.9																
37.8	End of Borehole															

Elev. 269.2
W.L.

Sa6% Si80%
Cl14%

Sa32% Si56%
CL 12%

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-83 LOCATION Q.E.W.-Tufford Rd. Sta. 30/80 14' Rt.
W.F. 220-63 BORING DATE September 19, 1966
DATUM Geodetic BOREHOLE TYPE Drive & Drill BX casing and wash

ORIGINATED BY VK
COMPILED BY VK, JM
CHECKED BY SP

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	wp	w	WL		
277.4	Ground Level															
	Sandy silt, trace of clay and gravel		1	SS	65											
			2	SS	100 for 5"	270										
			3	SS	100 for 4"											
			4	SS	100 for 4"											
			5	SS	100 for 5"	260										
259.9 17.5	Shale (weathered and fractured)		6	SS	100 for 5"											
				AXT	50%											
				AXT	33%	250										
				AXT	60%											
244.4 33.0	(sound)			AXT	100%											
240.4 37.0	End of Borehole					240										

Elev. 270.4
W.L.
Gr 5% Sa 28%
Si 61% Cl 6%

FOUNDATION SECTION

JOB 66-F-83 LOCATION Q.E.W.-Tufford Rd. Sta. 30+00 35'lt. ORIGINATED BY VK
W.P. 220-63 BORING DATE September 22, 1966 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Drive & Drill BX casing and wash CHECKED BY AK




SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20 40 60 80 100	PLASTIC LIMIT ——— WP		
							SHEAR STRENGTH P.S.F.	WATER CONTENT ——— W		
								WP ——— W ——— WL		
								WATER CONTENT % 10 20 30		
276.8	Groundlevel									
272.3	Clayey silt with sand and gravel-hard	/ / / /	1	SS	33					Elev. 270.8
4.5	Silt sand to sandy silt with gravel very dense	.	2	SS	100	for 2"				▼ W.L.
		.	3	SS	100	for 4"				Grl 3% Sa 29%
		.	4	SS	100	for 5"				Si 52% Cl 6%
		.	5	SS	100	for 5"	260			
256.8		.	6	SS	76					
20.0	Shale (weathered)	// // //	7	SS	100	for 5"				
251.8		.	8	SS	100	for 1½"				
25.0	(sound)	.		AXT	80%	250				
		.		AXT	100%					
243.0		.		AXT	100%					
33.8	End of Borehole									
						240				

FOUNDATION SECTION

ORIGINATED BY VK

COMPILED BY VK, JM

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT 20 40 60 80 100					WATER CONTENT % 10 20 30			
277.7	Ground Level														
266± 23.4	Clayey silt with sand & gravel Red-Brown hard		1	SS	119										
			2	SS	163	11"									
			3	SS	100	for 5"									
			4	SS	100	for 5"									
			5	SS	100	for 3"									
			6	SS	100	for 3"									
254.7 23.0	Shale (weathered)		7	SS	100	for 6"									
247.9 29.8	(sound)			AXT	40%										
				AXT	100%										
				AXT	100%										
242.4 35.3	End of Borehole														

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 66-E-83

LOCATION C.E.W.-Tufford Rd. Sta. 29+62 16' It.

ORIGINATED BY V.K.

W. P. 220-63




BORING DATE September 21, 1966

COMPILED BY VK2 JM

DATUM Geodetic

BOREHOLE TYPE Drive & Drill BX casing and wash

CHECKED BY AK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT ——— WL		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	PLASTIC LIMIT ——— WP	WATER CONTENT ——— W		
							20 40 60 80 100		WP ——— WL	WATER CONTENT %		
276.8	Ground Level								10 20 30	P.C.F.		
0.0	Clayey silt with sand and gravel-hard		1	SS	49							Gr4% Sa29% Si55% Cl12% ▼ W.L. Elev. 266.8
			2	SS	144/11"	270						
			3	SS	100 for 4"							
266.3			4	SS	100 for 3"							
10.5	Silty sand with occasional gravel		5	SS	100 for 5"	260						
			6	SS	100 for 5"							
			7	SS	100 for 5"							
253.8			8	SS	100 for 4"	250						
23.0	Shale (weathered)											
248.8												
28.0	(sound)			AXT	70%							
				AXT	100%	240						
239.0												
37.8	End of Borehole					230						

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 66-F-83

LOCATION Q.E.W.-Tufford Rd. Sta. 28+21 20' Rt.

ORIGINATED BY VK

W. P. 220-63





BORING DATE September 22, 1966

COMPILED BY VR

DATUM Geodetic

BOREHOLE TYPE Drive & Drill BX casing and wash

CHECKED BY 

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE	LIQUID LIMIT ——— WL	PLASTIC LIMIT ——— WP	BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	WATER CONTENT ——— W			
							20 40 60 80 100		WP W WL		
							SHEAR STRENGTH P.S.F.	WATER CONTENT % 10 20 30			
277.1	Ground Level										Elev. 274.1
	Clayey silt with sand and gravel -hard or very dense		1	SS	89	270					W.L.
			2	SS	130						
266.6			3	SS	100	for 5"					
10.5	Silty sand to sandy silt with gravel		4	SS	100	for 6"					Gr-1% Sa31%
			5	SS	100	for 4"					Si53% CL5%
258.6						260					
18.5	Clayey silt with sand and gravel		6	SS	100	for 4"					
256.1											
21.0	Shale (weathered)		7	SS	100	for 5"					
			8	SS	100	for 6"					
						250					
248.6				AXT	55%						
28.5	(sound)			AXT	60%						
				AXT	100%	240					
236.7											
40.4	End of Borehole					230					

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 :-

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

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

MEMORANDUM

To: Mr. A. G. Sternac,
Principal Foundation Engineer,
Room 107, Lab. Building.

From: Bridge Division,
Downsview, Ontario.

Date: July 12th, 1966.

Our File Ref.

In Reply To:

Subject: W.P. #220-63, Site #18-17,
Tufford Road Underpass,
W.P. #221-63, Site #18-198,
Ontario Street Interchange,
Q.E.W., District #4.

66-F-83

66-F-82

Herewith is one print each of Bridge Site Plans E-4746-1 and E-4747-1 for the above structures. The probable location of the footings have been marked in red. Also enclosed are the preliminary structure site reports.

Please arrange for a foundation investigation of sufficient scope to enable us to proceed with the design.

JFW/cew

Encl.

cc R. Forrest
A. Crowley

W. S. Melnyshyn

W. S. Melnyshyn,
Regional Bridge Location Engineer.

COMPLETION DATE

OCT 26/66

BETWEEN GLENNY
& ST. CATHERINE'S

also

Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division,
Admin. Bldg.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

April 24, 1967

Tufford Road Underpass --
8.7 Miles West of St. Catharines West Limits,
W.P. 220-63, W.J. 66-P-83,
Q.E.W., District #4 (Hamilton).

We have reviewed the Preliminary Bridge Plan Drawing D-6109-P1 for the above mentioned structure, and submit the following comments:

The Preliminary Bridge Drawing indicates a piled foundation for the abutments. Since the subsoil mainly consists of hard or very dense glacial till deposits, consideration should be given to supporting the abutments on spread footings founded on granular fill, with a safe bearing pressure of 2 t.s.f. The fill below the tops of the footings should consist of well compacted G.B.C. Class "A" material, and should extend for a horizontal distance of at least 10 ft. from the footing edges in the plane of the footing tops. This portion of the fill should be built with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for a distance of about 50 ft. behind the abutments before re-excavating for the abutment footings.

If the aforementioned procedure is followed, no pile driving equipment will be required for this project, since the entire structure will be supported on spread footings.

MD/MdeF

cc: Messrs. S. McCombie
W. S. Melinyshyn

Foundations Files
Gen. Files

M. Devata
M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

Department of Highways Ontario

Copy for the information of

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

Mr. W. Melingshyn,
Regional Bridge Location Engineer,
Central Region,
Administration Building

Bridge Division,
Downsview, Ontario

April 19, 1967

Tufford Rd. Underpass
8.7 Miles West of St. Catharines West Limits
W.P. 220-63, Site No. 18-17
Q.E.W., District No. 4

Attached herewith are prints of the Preliminary Bridge Plan
Drawing D-6109-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$150,000.
This cost includes tender, materials, engineering and sundry
construction.

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

CSG:rd

C.S. Grebski,
Bridge Design Engineer

Attach.

c.c. S. McCombie
A. Stermac
R. Forrest
E. Cross

66-7-83

66-7-83

Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division,
Admin. Bldg.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

April 24, 1967

Tufford Road Underpass --
8.7 Miles West of St. Catharines West Limits,
W.P. 220-63, W.J. 66-F-83,
Q.E.W., District #4 (Hamilton).

We have reviewed the Preliminary Bridge Plan Drawing D-6109-P1 for the above mentioned structure, and submit the following comments:

The Preliminary Bridge Drawing indicates a piled foundation for the abutments. Since the subsoil mainly consists of hard or very dense glacial till deposits, consideration should be given to supporting the abutments on spread footings founded on granular fill, with a safe bearing pressure of 2 t.s.f. The fill below the tops of the footings should consist of well compacted G.B.C. Class "A" material, and should extend for a horizontal distance of at least 10 ft. from the footing edges in the plane of the footing tops. This portion of the fill should be built with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for a distance of about 50 ft. behind the abutments before re-excavating for the abutment footings.

If the aforementioned procedure is followed, no pile driving equipment will be required for this project, since the entire structure will be supported on spread footings.

MD/MaeF

cc: Messrs. S. McCombie
W. S. Melnyshyn

Foundations Files
Gen. Files

M. Devata
M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Sternac,
PRINCIPAL FOUNDATION ENGR.

CS