

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division.

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

Attention: Mr. S. McCombie

DATE: February 3, 1967

OUR FILE REF.

IN REPLY TO:

FEB 14 1967

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Ottawa Queensway
Extension and C.N.R. Overpass
District #9 (Ottawa)

W.J. 66-F-109 -- W.P. 108-65

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 suffice for your design requirements. Should additional information be required, please feel free to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
S. J. Markiewicz
C. R. Robertson
G. Scott
J. E. Gruspier
B. A. Singh

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

Foundations Files
Gen. Files

TABLE OF CONTENTS

1. INTRODUCTION.
 2. DESCRIPTION OF THE SITE.
 3. FIELD AND LABORATORY INVESTIGATION.
 4. SUBSOIL CONDITIONS:
 - 4.1) General.
 - 4.2) Clayey Silt to Silty Clay (Leda Clay).
 - 4.3) Silt, Sand and Gravel.
 - 4.4) Bedrock.
 5. GROUNDWATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS.
 7. SUMMARY.
 8. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT
For
Proposed Ottawa Queensway
Extension and C.N.R. Overpass
District #9 (Ottawa)
W.J. 66-F-109 -- W.P. 108-65

1. INTRODUCTION:

In a memo, dated November 18, 1966, a foundation investigation was requested by Mr. G. Scott, Regional Bridge Location Engineer - Kingston, at the site of the proposed Ottawa Queensway extension and the C.N.R. overpass.

Accordingly, field and laboratory foundation studies were carried out by this Section, in order to establish the existing soil conditions at the site. Given in this report are the results of the above investigation with the recommendations as to the structure foundation, approach fill stability and predicted settlements.

2. DESCRIPTION OF THE SITE:

The proposed crossing is situated some 2.5 miles west of the westerly city limit of Ottawa along design line 'D'. The vicinity west of the C.N.R. tracks is generally rolling countryside, while towards the east, it is rather flat.

The terrain belongs to the "Ottawa Valley Clay Plains" physiographic region. The overburden around this area is built up by sediments of deep silty clays, underlain by sandstone bedrock. This is the most fertile portion of the Ottawa Valley region; hence, it is occupied by farming communities.

3. FIELD AND LABORATORY INVESTIGATION:

3.1) Four sampled boreholes and five dynamic cone penetration tests were carried out during the recent field exploration. The borings were numbered: 3, 4, 5, 6, and 7. Boreholes #1 and 2 were

cont'd. /2 ...

3. FIELD AND LABORATORY INVESTIGATION: (cont'd.) ...

drilled during the preliminary investigation, along line 'C', the results of which were reported in May 1966, under W.J. 66-F-17. Borehole #1 is incorporated in this report; hole #2, however, falls outside the crossing of the proposed line 'D'.

The field work was performed by means of a conventional diamond drill rig adapted for soil sampling purposes. Samples were recovered by means of 2-inch I.D. Shelby tubes and occasionally by a 2-inch O.D. split-spoon sampler. The split-spoon sampler was advanced by a driving energy of 350 ft.-lb.

The locations and elevations of the borings, together with the stratigraphical sections, are presented on Drawing #66-F-109A.

3.2) A comprehensive laboratory test program was performed on representative samples, including tests for soil classification purposes, strength characteristics and consolidation.

Laboratory test results as well as the standard penetration and field vane tests, are plotted on the borelog sheets, accompanying this report.

4. SUBSOIL CONDITIONS:

4.1) General:

Subsoil at the site was found to consist of a deposit of clayey silt to silty clay, followed by a 1 - 4 ft. thick granular glacial till around el. 232 - 237 ft. which, in turn, was underlain by sound sandstone bedrock. A thickness of five ft. of the bedrock was proved by diamond drilling.

A description of the subsoil follows:

4.2) Clayey Silt to Silty Clay (Leda Clay):

This is the predominant stratum of the area, and was found in every borehole, extending down to el. 232 - 237 ft. The total thickness of the layer varies between 31 and 56 ft. The uppermost

cont'd. /3 ...

4. SUBSOIL CONDITIONS: (Cont'd.) ...

4.2) Clayey Silt to Silty Clay (Leda Clay): (cont'd.) ...

portion of the material has an oxidized brown colour, being slightly desiccated. The layer is somewhat heterogeneous being intercepted by numerous minute seams of silt and fine sand. The presence of these coarser seams causes a decrease of the plasticity of the soils, thereby decreasing the overall toughness and dry strength. There appears to be an even more essential influence of the silt seams, and that is the increase of the sensitivity of the soil.

The sensitivity is defined as the ratio of the natural strength to the remoulded strength at the same water content. The marine sediments of the St. Lawrence Lowland (commonly called Leda clays) are known to exhibit very high values of sensitivity. At this site, values between 7 and 24 were measured by the field vane test. It is to be pointed out that usually high sensitivities are accompanied by low plasticity indices and low liquid limits; this will suggest values even higher than those observed.

Almost all the samples tested displayed liquidity indices higher than 1, and some of these reached values as high as 2.9. This phenomenon is also typical of the sensitive Leda clays.

Three different kinds of shear strength tests were carried out; namely, field vane, unconsolidated quick triaxial and unconfined compression tests.

Values obtained by the field vane and the triaxial compression tests were found to be more comparable; the unconfined compression tests, however, yielded considerably lower values. By studying the failure planes and failure mechanism, it was noted that the majority of samples tested by unconfined compression, failed in tension by vertical splitting. A tensile failure would probably not occur if there were any lateral effective stress on the specimen; consequently, it may not be serious in practice. For the stability analyses of the approach fills, average values of

cont'd. /4 ...

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

4.2) Clayey Silt to Silty Clay (Leda Clay): (cont'd.) ...

shear strength were used. A value of $S = 800$ p.s.f. was assumed down to el. 250 ft., and $S = 1,000$ p.s.f. below that elevation - (Fig. #1).

The results of the consolidation tests indicate that the layer is overconsolidated more heavily at the upper portion. The ratio of overconsolidation was computed to be 3.45 at a depth of 16 ft., gradually decreasing to a value of 1.65 at around 46 ft.

4.3) Silt, Sand and Gravel:

A mixture of silt, sand and gravel (glacial till) stratum underlies the Leda clay at el. 232 - 237 ft. The thickness of the glacial deposit varies from one to four ft., the relative density being generally compact. Since the layer is very thin, it has not too much practical significance, except that, after the proposed embankment loading, it will act as a drainage during the consolidation.

4.4) Bedrock:

The upper surface of the sound sandstone bedrock was observed between el. 230 and 234 ft., and was proved for a thickness of 3 - 5 ft. by drilling it with an AXT core barrel with diamond shoe.

5. GROUNDWATER CONDITIONS:

Water level was assumed to follow the general topography of the site, being found at approx. el. 281 ft. in borehole #1 at the west side of the railway track, and at around el. 265 ft. in hole #4 near the creek.

6. DISCUSSION AND RECOMMENDATIONS:

The proposal calls for the construction of two overhead structures to carry the east and west bound lanes of the Ottawa Queensway separately over the C.N.R. tracks. From the geometry of the crossing, it may be seen that approach fills at the east side

cont'd. /5 ...

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

of the tracks will be in the order of 40 ft.

The predominant soil at the site was found to be a 31 - 56 ft. thick layer of sensitive marine clayey silt to silty clay, having firm to very stiff consistency. The stratum does not possess adequate strength for an economical design of spread footing at a shallow depth. Piled foundations are recommended, therefore, utilizing steel H-piles driven to bedrock. The maximum load for the particular H-section may be assumed for design purposes.

Piles should be designed to resist all lateral forces induced by the embankment on the structure. It is pointed out, that no frictional resistance may be assumed between the bottom of the pile cap and the soil beneath.

Stability analyses in terms of total stresses, for the proposed approach embankments, were carried out by the use of an electronic computer. Average shear strength of the Leda clay was used for the computations. Based on the information obtained by the Regional Road Design Section, Kingston, it was further assumed that the fills would be constructed using rock of a bulk density of $\gamma = 110$ p.c.f. Compacted earth fill must be used, however, in the vicinity of the proposed piles without any boulders or rock. The stability analyses indicated, that rock fills up to a height of 32 ft. will be stable, provided that they are built with final slopes of 2 horizontal to 1 vertical. Fills, however, in excess of 32 ft., will require counterbalancing berms to maintain stability. The required berm length relative to the height of fill may be obtained from the attached diagram (Fig. #2). The height of the berms should be equal to one-half the height of the fill.

Calculations of the consolidation settlements beneath the proposed fills were carried out, based on laboratory consolidation tests.

Two typical cross sections were considered for the settlement calculations, as shown on Fig. #3. The first section,

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

corresponding to the west side of the crossing, was assumed to have a maximum height of 30 ft. The depth of the compressible layer was taken to be 50 ft. No berm is necessary for this section.

The east approach fill was assumed to be 45 ft. high with 50-ft. long berms. At this side, however, the thickness of the compressible layer is only 30 - 32 ft.

The results of the three typical consolidation tests are plotted as: void ratio, e , versus logarithm of pressure, p , on Fig. #4, indicating the existing effective overburden, and the estimated preconsolidation pressures.

Settlements were calculated beneath the middle and beneath the toe of the embankments. The calculated settlements were corrected for the silt content or, rather, silt seams within the Leda clay. For the correction, it was estimated that approx. 14% of the layer was silt, and it was further assumed that the settlement of the silt will be approx. 25% of that of the clay.

The total corrected consolidation settlement beneath the centre of both embankments, was computed to be:

Settlement, approx. = 32 - 33 inch.

Beneath the toe of the west approach fill, the consolidation settlement will be about 2 in.; below the toe of the east fill, it is estimated to be only 0.5 in.

No attempt was made to compute the elastic part of the settlement, since it will take place during and immediately after the construction of the fills.

Due to the anticipated large settlements, it is recommended that a flexible type of pavement be considered.

cont'd. /7 ...

7. SUMMARY:

The foundation investigation for the proposed Ottawa Queensway and C.N.R. overpass is reported.

Subsoil was found to consist of a 31 - 56 ft. thick sensitive marine (Ieda) clay deposit, underlain by a thin layer of silt, sand and gravel (glacial till) which, in turn, is followed by sound sandstone bedrock.

Piled foundations are recommended, adopting steel H-piles driven to bedrock (approx. el. 230 - 234 ft.). Allowable loads will depend upon the pile section chosen.

Fills in excess of 32 ft. in height, will not be stable. Berms should be constructed for fills in excess of 32 ft. height, and the details are given on Fig. #2.

The computed average consolidation settlements beneath the centre of the approach fills may be taken to be approx. 32 - 33 in. 0.5 - 2 in. settlements are estimated to occur beneath the toe of the fills. Flexible pavements should be used on the approach fills.

8. MISCELLANEOUS:

The field work, performed during the period December 13 - 16, 1966, was supervised by Mr. L. Palmer, Project Foundation Engineer. Equipment used was owned and operated by Johnston Drilling Company, Ottawa. This report was prepared by Mr. A. K. Barsvary, Senior Foundation Engineer, and reviewed by Mr. M. Devata, Supervising Foundation Engineer.

February 1967

APPENDIX 1.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-109

LOCATION Sta. 141+62.5; 90' Lt. of E

ORIGINATED BY L.P.

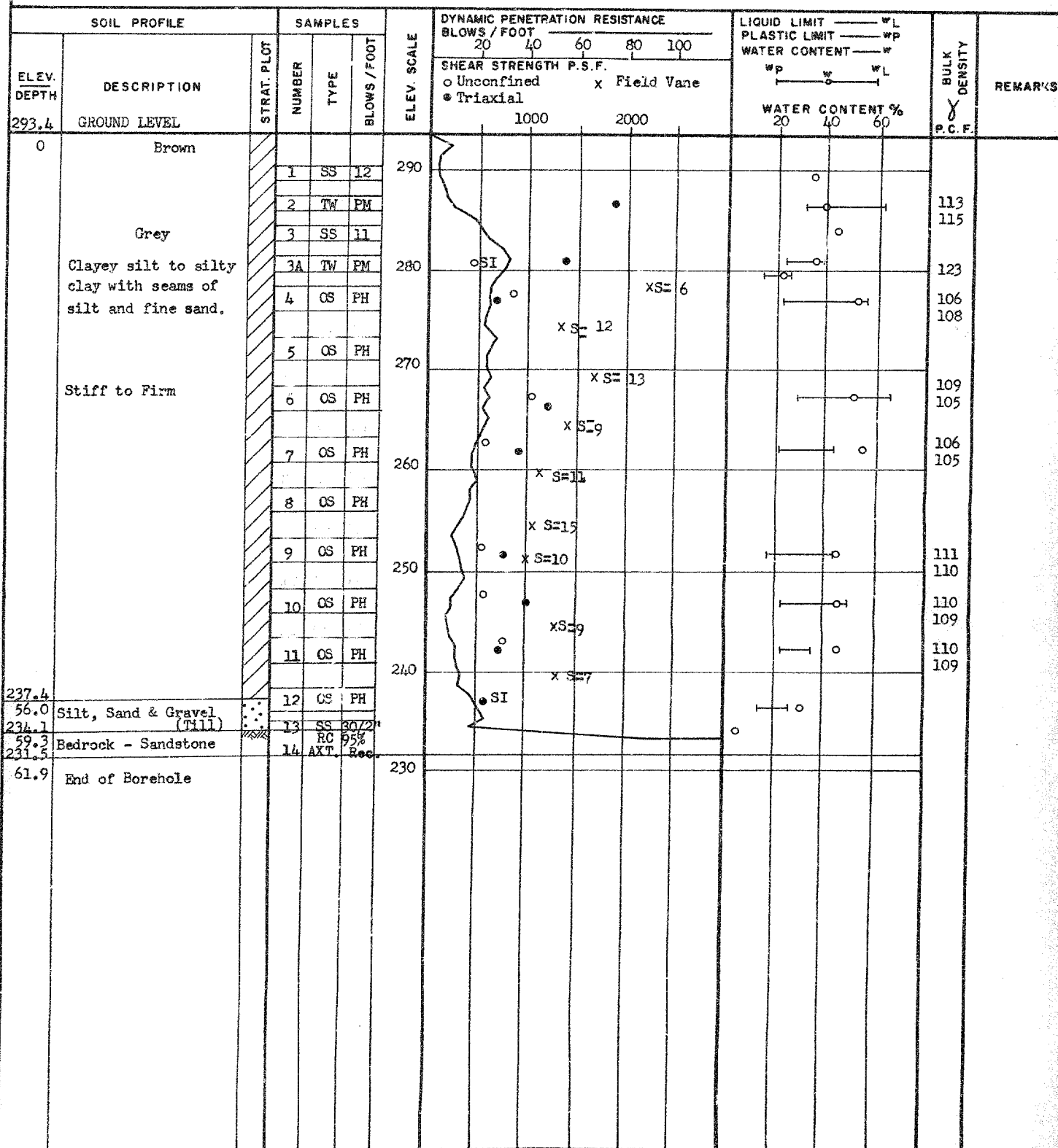
W.P. 108-65

BORING DATE December 13, 14, 1966

COMPILED BY A.K.B.

DATUM G.S.C.

BOREHOLE TYPE Washboring, NX Casing

CHECKED BY *SR*

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 66-F-109 LOCATION Sta. 139 + 28. 84' Lt. of C.

ORIGINATED BY L.P.

W.P. 108-65 BORING DATE December 15, 1966

COMPILED BY A.K.B.

DATUM G.S.C. BOREHOLE TYPE Washboring, NX Casing

CHECKED BY AK

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT ——— w _L PLASTIC LIMIT ——— w _p WATER CONTENT ——— w			BULK DENSITY Y P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS /FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. o Unconfined x Field Vane e Triaxial			WATER CONTENT % w _p w w _L					
265.9	GROUND LEVEL						1000	2000		20	40	60			
0	Brown		1	TW	PM		260						111		
	Grey		2	TW	PM									108	
	Clayey Silt to Silty Clay with seams of Silt and Fine Sand.		3	TW	PM									105	
			4	OS	PH									106	
			5	OS	PH									105	
	Firm to Stiff		6	OS	PH									106	
			7	OS	PH									109	
			8	OS	PH									101	
234.1			9	OS	PH									111	
31.8	Silt, Sand & Gravel			10	RC		80%							116	
229.9	Compact (Till)			AXT	Rec							109			
36.0	Bedrock - Sandstone														
224.7															
41.2	End of Borehole														

FOUNDATION SECTION

ORIGINATED BY L.P.

COMPILED BY A.K.B.

CHECKED BY AK

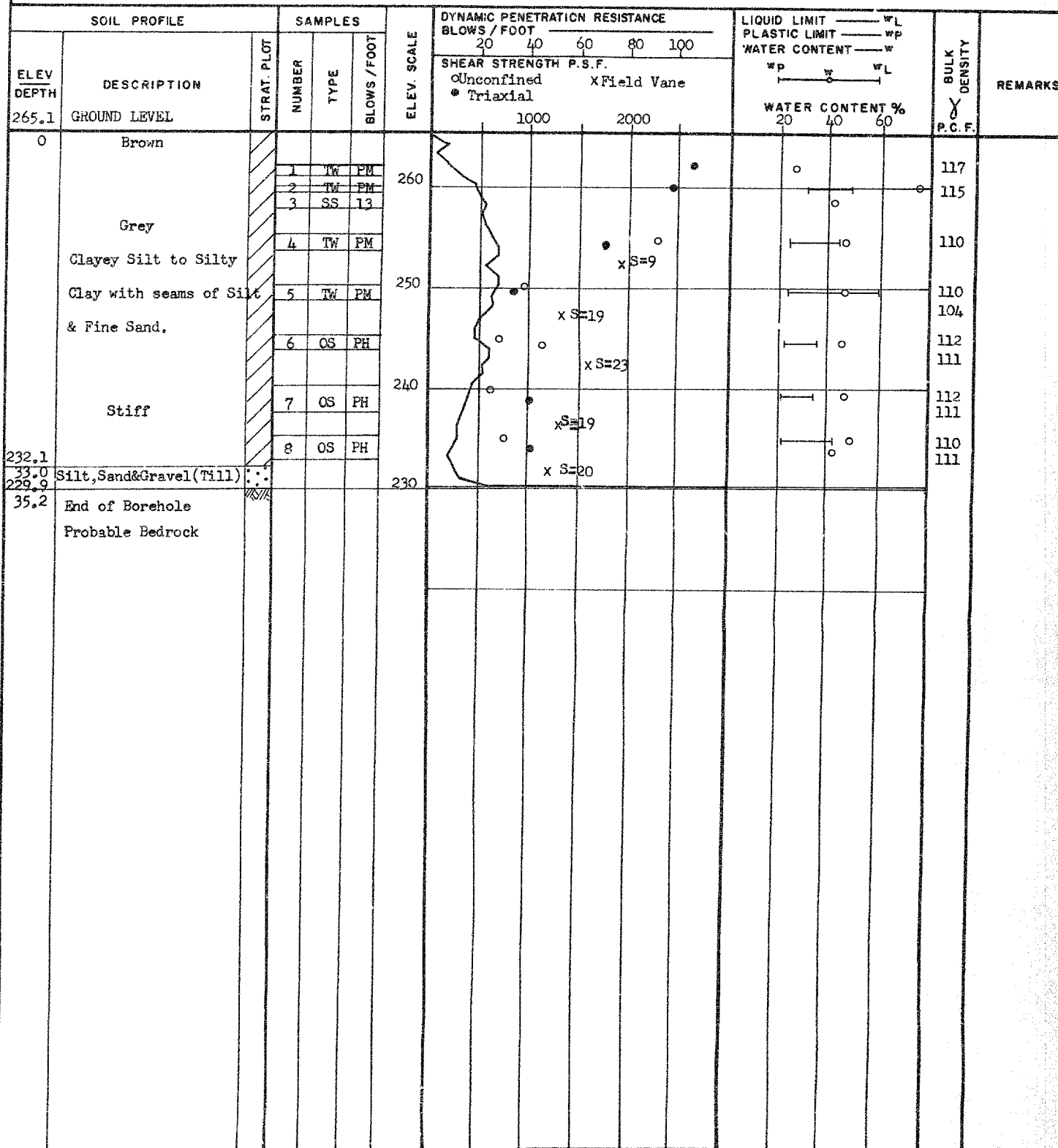
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 66-F-109 LOCATION Sta. 138 + 00; 68' RT. of 0 ORIGINATED BY L.P.
W.P. 108-65 BORING DATE December 15, 1966 COMPILED BY A.K.B.
DATUM G.S.C. BOREHOLE TYPE Washboring, NX Casing CHECKED BY LR



MATERIALS & TESTING DIVISION

JOB 66-F-109

108-65

W.P. _____
 DATUM _____ G.S.C. _____

DATUM G.S.C.

RECORD OF BOREHOLE NO. 7A

LOCATION Sta. 139 + 50; 64' Rt.

EDUCATION _____
BORING DATE December 16, 1966

BOREHOLE TYPE Dynamic Cone Penetration

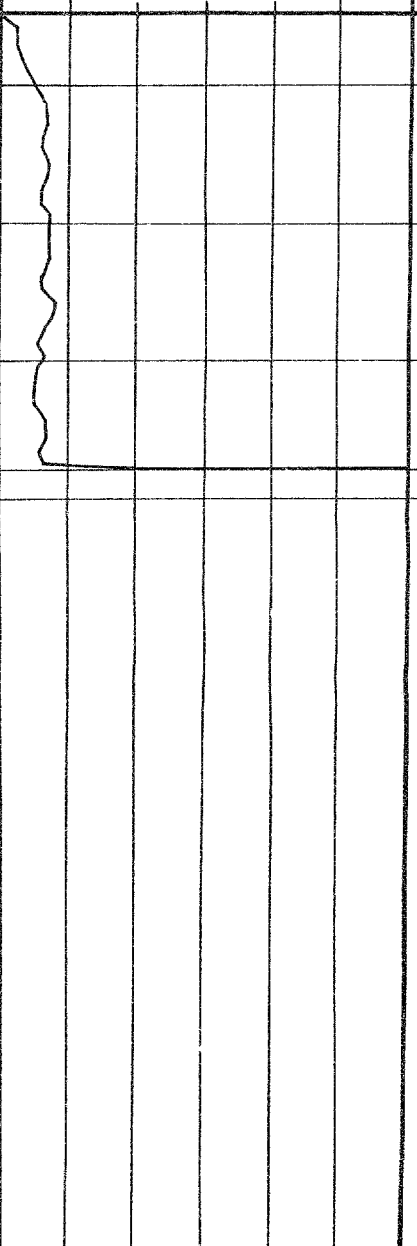
BOREHOLE TYPE Dynamic Cone Penetration

FOUNDATION SECTION

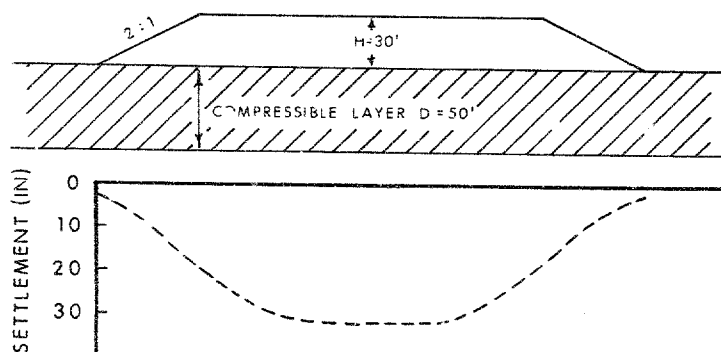
ORIGINATED BY L.P.

COMPILED BY A.K.B.

CHECKED BY AK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— w _L	PLASTIC LIMIT ——— w _p	WATER CONTENT ——— w	BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.	w _p	w	w _L		
265.2 0	GROUND LEVEL											
232.1 33.1	Dynamic Cone Penetration Only End of Cone Probable Bedrock					260 250 240 230						

SETTLEMENT BENEATH APPROACH FILL, WEST SIDE



SETTLEMENT BENEATH APPROACH FILL, EAST SIDE

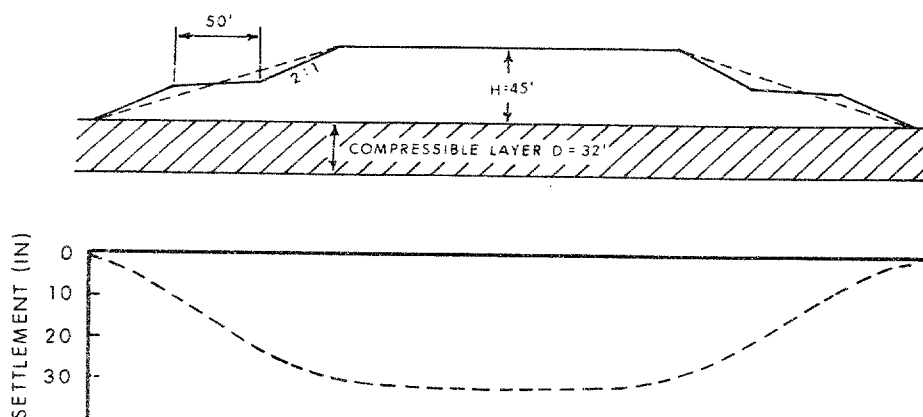
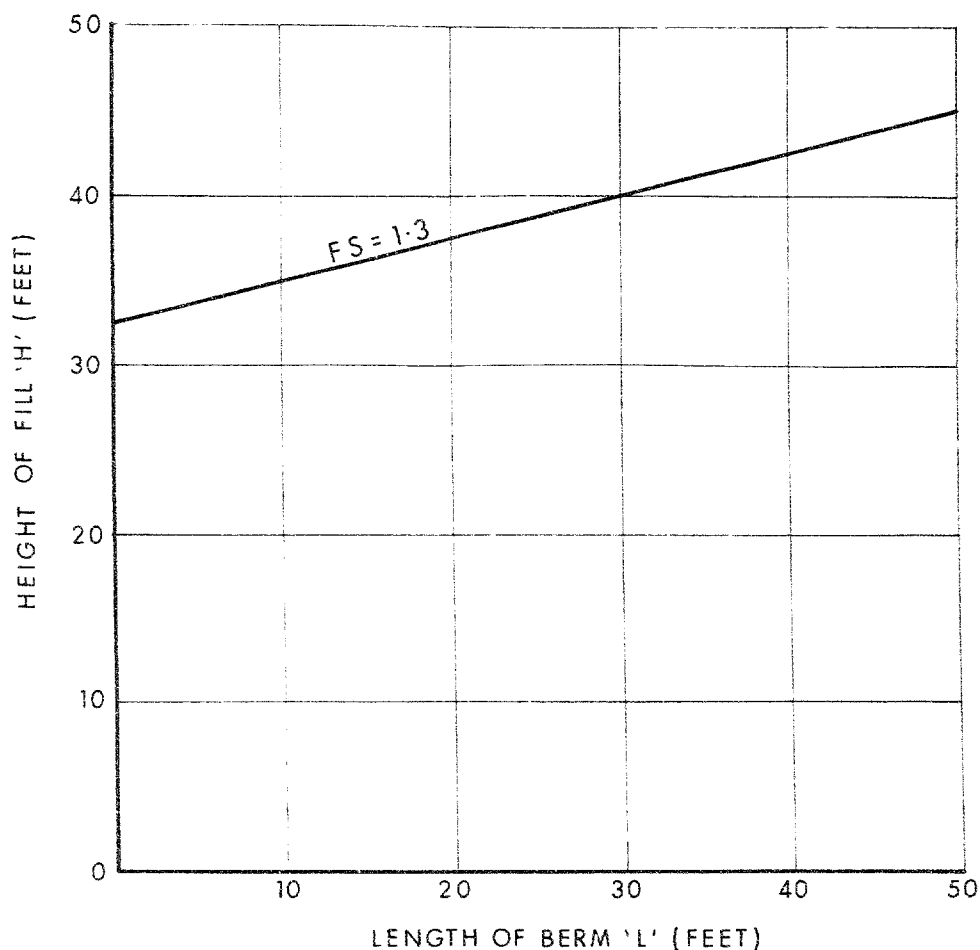
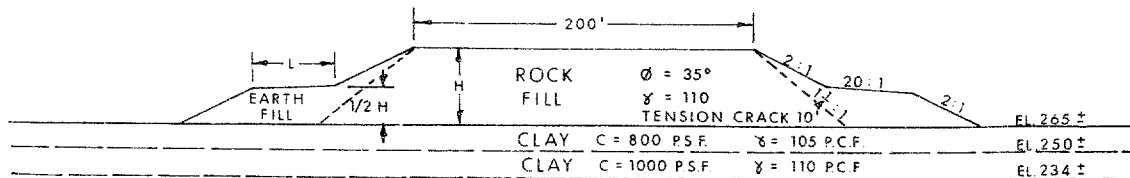


Fig. 3



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

ONTARIO

THE HEIGHT OF FILL VS BERM LENGTH OTTAWA QUEENSWAY & C.N.R.

W.P. NO. 108-65

DATE 27 JAN. 1967

APPROVED *[Signature]*

FIGURE 2

SHEAR STRENGTH PROFILE

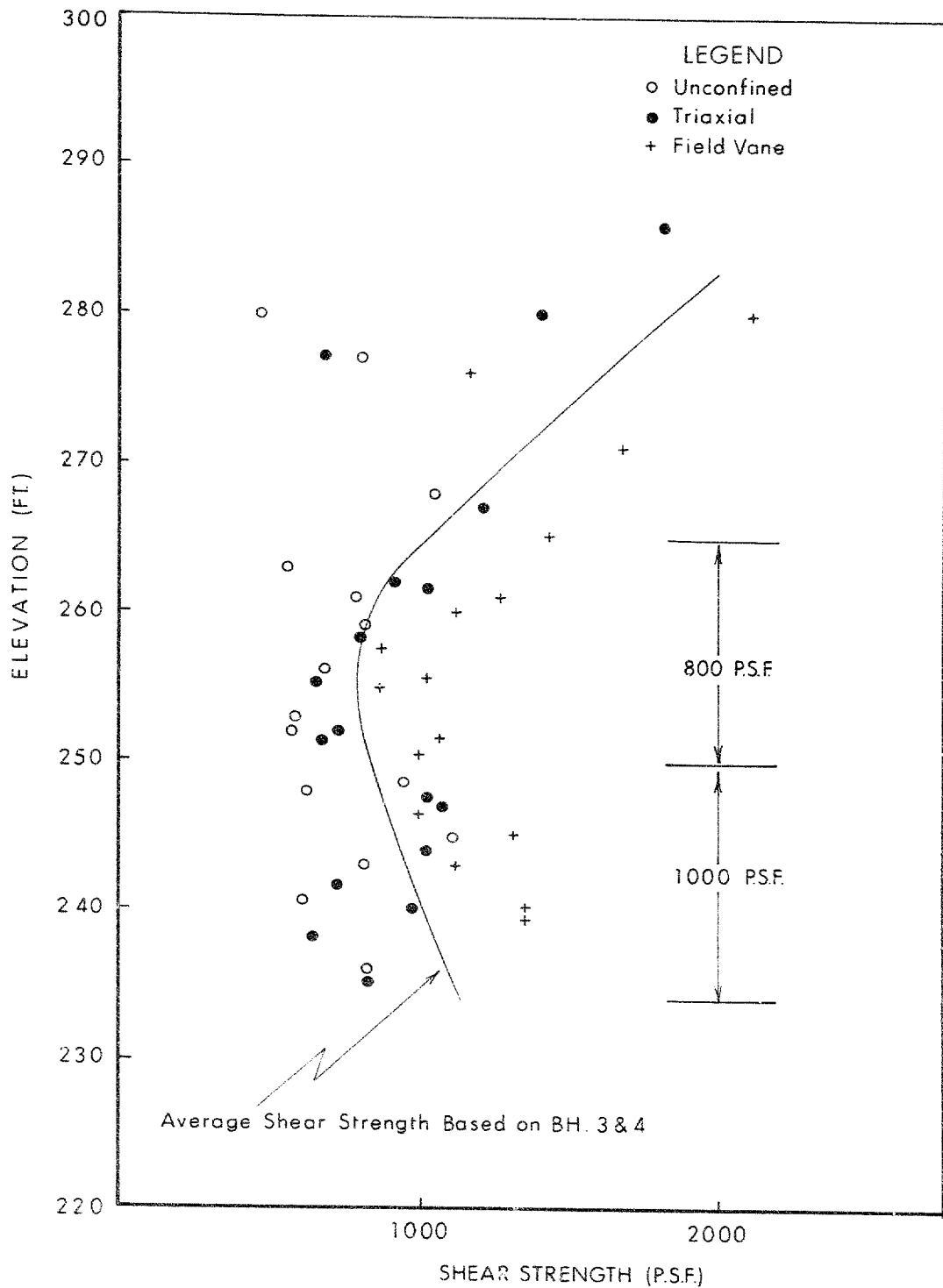


Fig.1

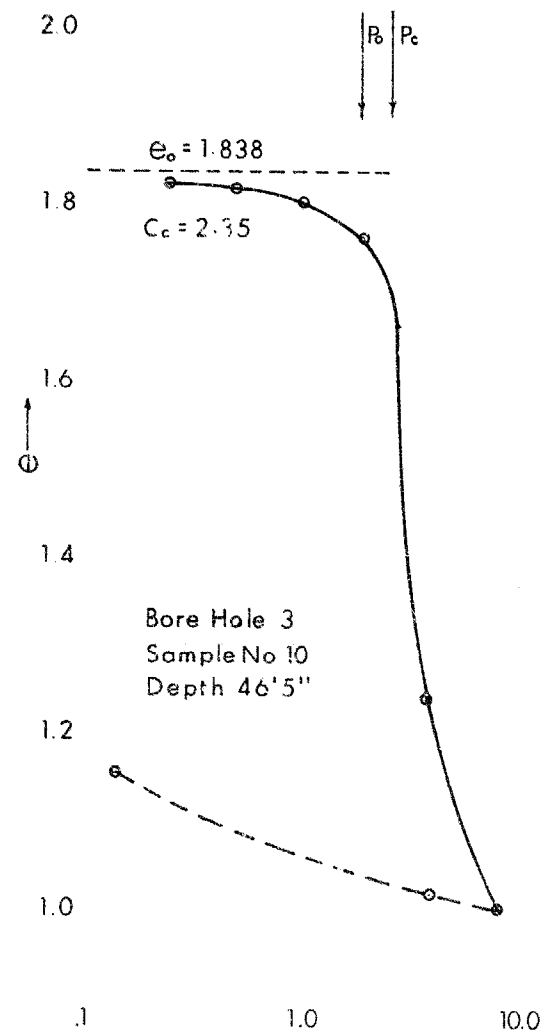
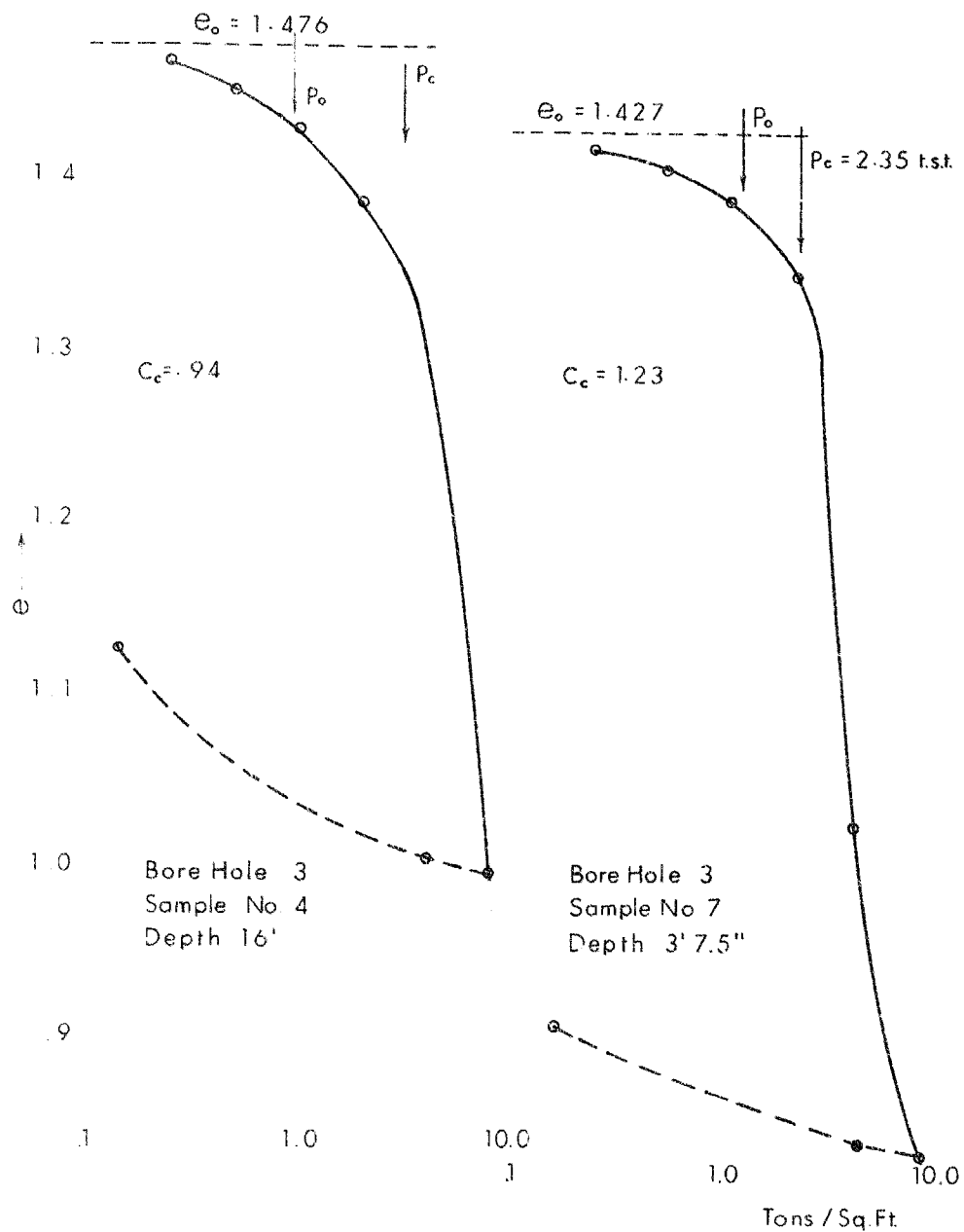


Fig. 4

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	DESTERBERG 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

Q _u	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q _{cu}	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q _d	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
T_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
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 (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

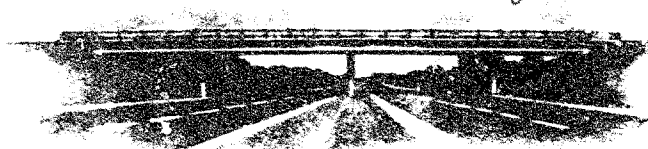
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_a	MODULUS OF SUBGRADE REACTION

SLOPES

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



401 & Keele St.
Downsview, Ontario



DEPARTMENT OF HIGHWAYS
Materials and Testing Division

November 30, 1966

Johnston Drilling Co. Ltd.
378 Bering Street
Toronto, Ontario

Attention: Mr. F. Blackburn

Dear Sirs:

This is to confirm our request of November 30, 1966 for the supply of one Diamond Drill together with all necessary equipment, as specified under the terms of our Contract Agreement, at Bells Corner, Ontario on December 5, 1966 at 1 p.m.

These projects bear the following Job Number:

66-F-108 Queensway & Moody Drive
66-F-109 Queensway & C.N.R.
66-F-110 Queensway & Acres Rd.

Yours truly,

MD:mt

M. Devata
Supervising Foundation Engr.
for A. G. Stermac
Principal Foundation Engr.

cc: H. Konings

Foundations Office
General Files

MEMORANDUM

Mr. B. R. Davis,
Bridge Engineer,
Bridge Division.

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

Attention: Mr. S. McCombie

DATE: January 26, 1967

FILE REF.

IN REPLY TO:

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

The Proposed Overhead Structures at
The Crossing of Ottawa Queensway
Extension (Line 'D') and C.N.R.,
District No. 9 (Ottawa)
W.J. 66-P-109 (R) -- W.P. 100-65

1. Introduction:

A REQUEST to conduct a foundation investigation at the above mentioned site, was contained in a memo dated November 18, 1966, from the Bridge Planning Section (Mr. G. Scott). The site is located about 2-1/2 miles west of the junction of Hwy's 7 & 15 and the Ottawa Queensway in Lots 7 & 8, Concession II, Ottawa Front, in the Township of Nepean, County of Carleton.

Due to the urgency of this project, we have been requested to submit our written recommendations as soon as the field and laboratory work have been completed. The final report will be submitted after the completion of drawings and borehole logs. A brief review of soil conditions, together with our recommendations for the structure foundations and approach fills, follows:

2. Subsoil:

Subsoil over the site area consists of 31 to 56 ft. of firm to stiff clayey silt to silty clay, followed by a 1 to 4 ft. thick stratum of silt, sand and gravel (glacial till). Underlying this, sound sandstone bedrock was encountered between elev. 230 and elev. 234.

3. Recommendations:

It is proposed to construct two separate structures at this site to carry the Ottawa Queensway over the C.N.R. tracks. The investigation has revealed the presence of a firm to stiff

3. Recommendations: (cont'd.) ...

deposit of cohesive soil immediately below the ground surface. The presence of such a layer precludes the possibility of a conventional spread footing type foundation; the proposed structure must, therefore, be supported on a piled foundation. It is recommended that a piled foundation utilizing steel H-piles driven to bedrock, be constructed, and that the maximum load for the particular pile section be assumed for design purposes. Piles should be designed to resist all lateral forces induced by the rock fill embankments on the structure.

Due to the existence of the firm to stiff clay layers, fill in excess of 32 ft. in height, constructed with standard 2:1 slopes, will not be stable. For fills in excess of 32 ft. in height, berms should be constructed. A graph showing the relationship between berm lengths and fill heights, are shown on Dwg. No. 66-F-109A. The assumption of using rock fill ($\gamma = 110$ p.c.f., 2:1 side slopes) was obtained from the Regional Road Design Section, Kingston.

The underlying firm to stiff silty clay to clayey silt layers will undergo settlement due to consolidation over a long-term period under the weight of the approach embankments. This will be discussed in detail, in our final foundation report.

The complete foundation report for this project will be forwarded to you as soon as possible. If you have any further queries, or if any of the foregoing requires clarification, please do not hesitate to call us.

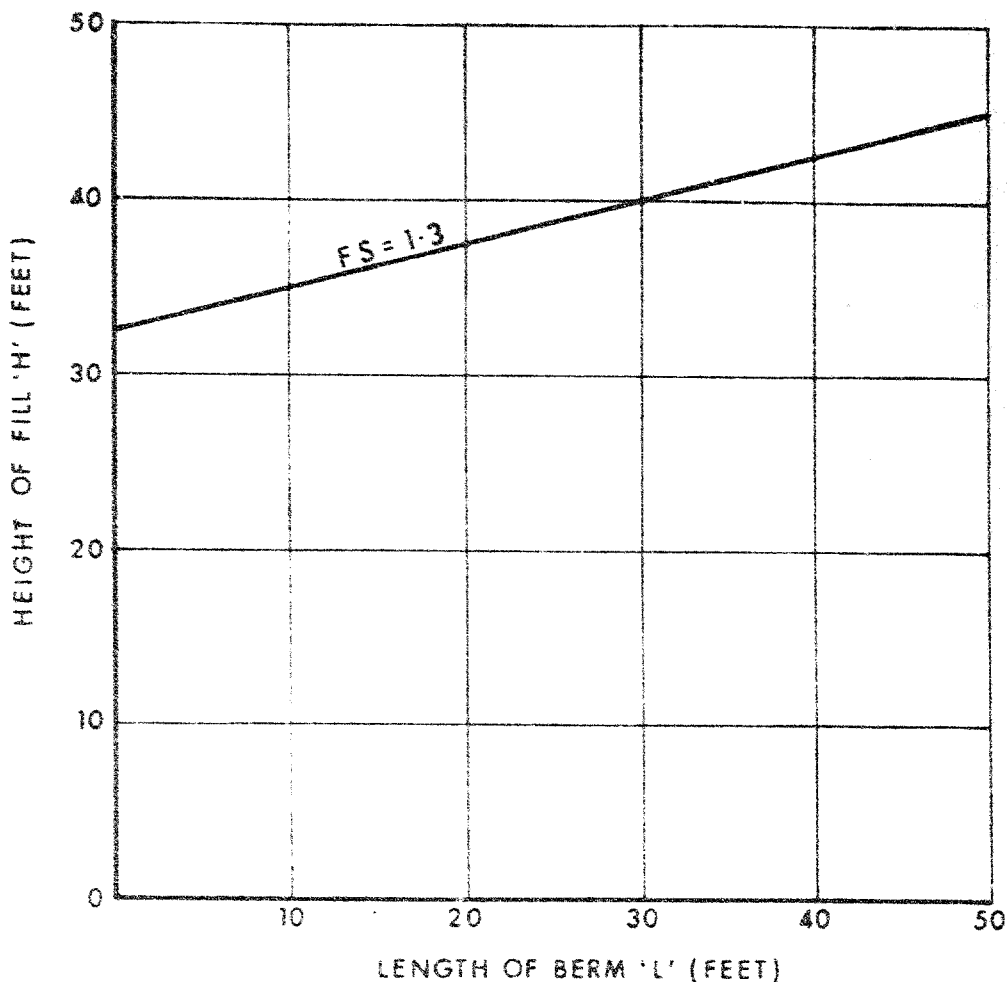
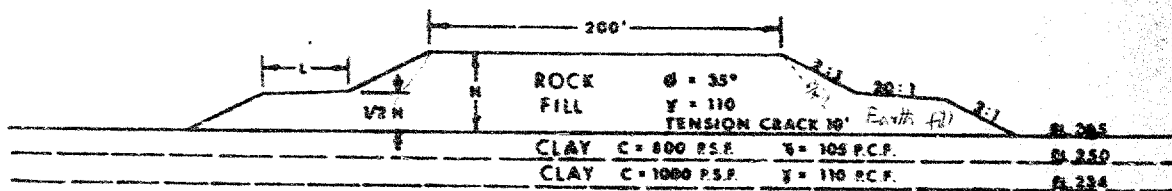
MD/MdeP

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Parren
S. J. Markiewicz
C. Scott
C. R. Robertson
J. E. Gruspier
B. A. Singh

M. Devata

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

Foundations Office ✓
Gen. Files



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

THE HEIGHT OF FILL VS BERM LENGTH OTTAWA QUEENSWAY & C.N.R.

W.P. NO. 108-65

DATE 27 JAN. 1967

APPROVED *M. Swata*

DRAWING NO. 66-F-109B

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

Mr. M. Devata,
Senior Foundation Engineer,
Foundation Section,
Room 107, Lab. Building

FROM: Bridge Division,
Downsview, Ontario

DATE: February 23, 1967


FILE REF.

IN REPLY TO

SUBJECT: W.P. 108-65, Site 3-237 (Your File Job No. 66-F-109)
C.N.R. Overhead Structure
Ottawa Queensway Extension
2.7 Miles West of Jct. Hwy. 15
District No. 9

Please find attached one copy of a "Preliminary Sketch"
drawing no. D6138-PS1 for the above structure site.

Would you kindly advise us of your opinion regarding the
stability of the structure and embankments for the arrangement
shown.



J.L. Keen,
Senior Bridge Project Engineer

JLK:rd

Encl.

c.c. G.S. Saunders, P.Eng.,
DeLeuw, Cather & Co. - Ottawa

RECOMMENDATIONS BY MR DEVATA GIVEN BY PHONE TO MR KEEN

In order to prevent movements of the abutments, the wingwalls should be built longer. The wingwalls should also be supported on end ~~of~~ bearing piles driven to bedrock. One additional pile will be needed at each wingwall. (Total of four)

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Administration Building,
DOWNSVIEW, Ontario.

FROM: Bridge Division,
KINGSTON, Ontario.

DATE: March 14, 1967.

OUR FILE REF.

IN REPLY TO

SUBJECT:

W.P. 108-65, Site 3-257, C.N.R. O'Head
(2.7 Miles West of Jct. Hwy. 15)
Ottawa Queensway, District 9

Herewith please find print of Preliminary Plan
D-6138-P-1. May we have such comments as you wish
to make.



J. A. Fisher

For: G. Scott
REGIONAL BRIDGE LOCATION ENG.

GS/h1

Enc.

Department of Highways Ontario

Copy for the information of

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

Mr. G. Scott,
Regional Bridge Location Engineer,
Kingston Regional Office

Bridge Division,
Downsview, Ontario

March 13, 1967

C.N.R. Overhead
2.7 Miles West of Jct. Hwy. 15
W.P. 108-65, Site 3-257
Ottawa Queensway, District 9

Attached herewith are prints of the Preliminary Bridge
Plan Drawing D-6138-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$319,000.
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

agp

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

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

March 22, 1967

C.M.R. Overhead,
2.7 Miles West of Jct. Hwy. #15,
W.P. 108-65, W.J. 66-P-109, Site 3-257,
Ottawa Queensway, District #9 (Ottawa).

We have reviewed the Preliminary Bridge Plan Drawing D-6133-P1 for the above mentioned structure, and submit the following comments.

Since settlement of the proposed roadway embankments may be as high as 32 inches, some negative skin friction forces can be imposed on the piles supporting the abutments. These forces, combined with movement of subsoil due to strain imposed by the embankment loading, will generally tend to displace the piles laterally. In view of this, we recommend that consideration be given to supporting the extreme ends of the wing walls on piles driven to bedrock. In our opinion, this will improve the stability of the abutment in the longitudinal direction.

10/3def

cc: Messrs. S. McCombie
G. Scott

Foundations Files /
Gen. Files

M. Devata

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

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

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

May 24, 1967

C.N.R. Overhead --
2.7 Miles West of Jct. Hwy. 15,
W.P. 108-65, Site 3-257,
Ottawa Queensway, District 9.

With respect to your memo of May 23, 1967, regarding the above structure, we wish to submit the following comments for your consideration:

In view of the performance of a number of bridges in the Cornwall area, we would recommend that the presently designed vertical pile supporting the abutment be changed to a pile battered in the direction away from the bridge. The abutment would thus be supported by two rows of piles battered in the opposite directions.

We would also recommend that the piles supporting the wing walls be battered in the direction away from the bridge. The connection of the abutment and the wing walls should be adequately reinforced to be rigid enough to allow the abutment and wing walls to act as one unit.

It is agreed that the soil at this site is stronger and, therefore, less compressible than the soil at the Cornwall sites. However, settlements in the order of 30 inches are predicted, and we feel that these could cause movements of the abutments away from the bridge unless the proposed measures are implemented.

ACS/ndef

A. G. Sternac
A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. S. McCombie
C. Scott

Foundations Files
Gen. Files

Department of Highways Ontario

Copy for the information of
Mr. A. Stermac, Principal Foundation Engineer

Mr. G. Scott,
Reg. Bridge Location Engineer,
London Regional Office,
London, Ontario

Bridge Division,
Downsview, Ontario

May 23, 1967

C.N.R. Overhead
2.7 Miles West of Jet. Hwy. 15
W.P. 108-65, Site 3-257
Ottawa Queensway, District 9

Attached herewith are prints of the Preliminary Bridge
Plan Drawing D 6138-P2 for the above-mentioned structure.

The estimated cost of the proposed structure is \$300,000.
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. Grebaki,
Bridge Design Engineer

Attach.

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

Mr. W. R. Kinnear,
Scheduling Engineer,
Program Division.

Mr. A. Crowley,
Expediter.

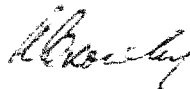
December 13, 1968.

66-F-109

W. P. Nos. 108-65, 423-64 & 430-64, Ottawa Queensway Extension

I have been advised by Mr. Stermac, Foundations Engineer, that the date for the completion of the Foundation Reports on the above structures will be delayed by one month, to February 15th, 1969. The reasons for this delay are due to the facts: (1) A Preliminary Report on these structures was carried out earlier in 1966. Since that time the location of these structures has been changed. This requires more soundings be taken than was originally scheduled.

(2) Poor soils conditions in the area of the structures necessitates a more extensive investigation which will require more time for completion.



A. Crowley,
Expediter.

AC/gar

c. c. W. G. Wigle
A. G. Stermac

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To:

Mr. A. Stenosa,
Principal Foundation Engineer,
Room 107, Laboratory Building,
BOWENSVILLE, Ontario.

From: Mr. G. Scott,
Regional Bridge Location Eng.,
Bridge Office,
KINGSTON, Ontario.

Date:

November 18, 1966.

Our File Ref.

IN REPLY TO:

SUBJECT:

W.P. 108-64 O.Q.W. & C.W.R. Crossing, Site #
Hwy. Ottawa Queensway Ext, District #9, Ottawa

We are sending you herewith two prints of preliminary plan #2-4629-1 on which we have marked in red the proposed location of the above noted structure.

Please make the necessary arrangements for foundation soils investigation, and receiving your report to us in due course.



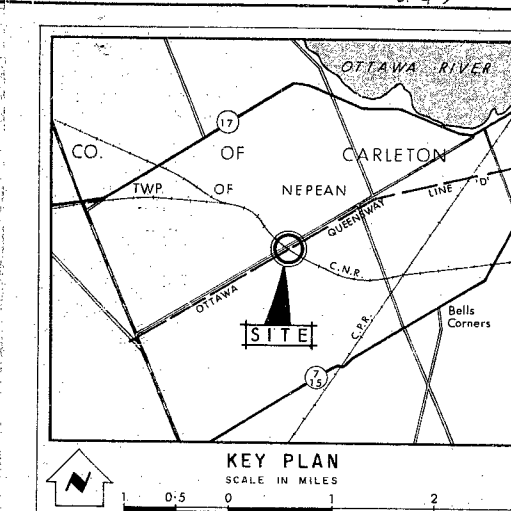
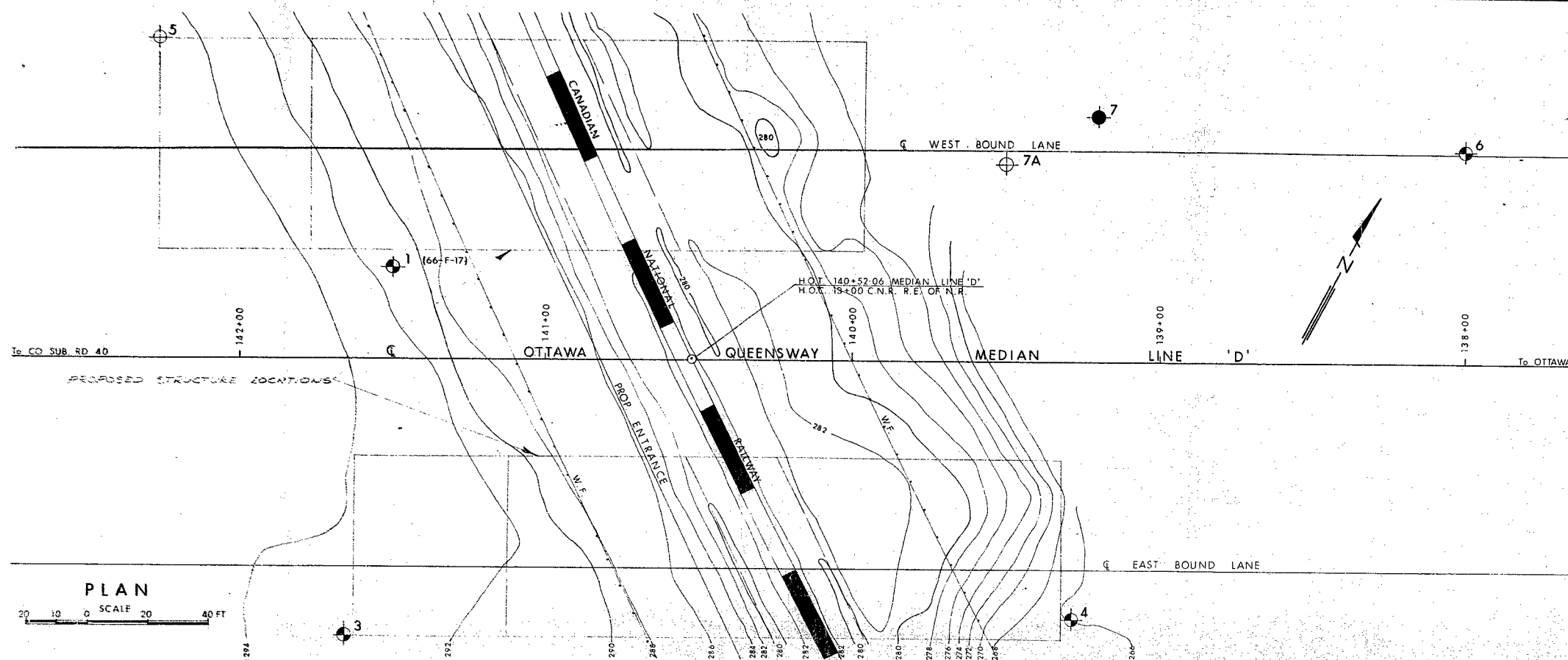
J. A. Fisher

For: G. Scott,

REGIONAL BRIDGE LOCATION ENGINEER.

JAY/GS/lm
Encl.

#66-F-109
W.P. #108-65
OTTAWA
QUEENSWAY
CANADIAN
NATIONAL
RAILWAY



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation, MAR. 1966

NOTE: Bore Holes 1 & 2 taken from job No. 66-F-17, done March 1966.

NO.	ELEVATION	STATION	OFFSET
1	291.0	141+5.0	30' RT.
2	NOT SHOWN ON DRAWING		
3	293.4	141+6.25	90' LT.
4	265.9	139+28	84' LT.
5	294.7	142+27	104' RT.
6	265.1	138+00	68' RT.
7	264.6	139+20	79' RT.
7A	265.2	139+50	64' RT.

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

NO.	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION - FOUNDATION SECTION

CANADIAN NATIONAL RAILWAY

KING'S HIGHWAY NO. OTTAWA QUEENSWAY LINE 'D' DIST. NO. 9

CO. CARLETON OTTAWA-CARLETON

TWP. NEPEAN LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. L.P.	CHECKED	W.P. NO. 108-65	M.B.T. DRAWING NO.
DRAWN S.O.	CHECKED	JOB NO. 66-F-109	66-F-109A
DATE 26 JAN. 1967	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	PRINCIPAL FOUNDATION ENGINEER	CONT. NO.	

REF. NO. E-4629-1