

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

Attention: Mr. S. McCombie

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

DATE: March 7, 1966

OUR FILE REF.

IN REPLY TO

MAR '9 1966

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

The Proposed Gray's Side Road
Underpass, Q.E.W., in the City
of Hamilton, District No. 4

W.J. 66-F-1 --

W.P. 207-63

Included with
206-63-02

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

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

AGS/MdeF
Attach.

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

Foundations Office
Gen. Files

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

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FOUNDATION INVESTIGATION REPORT
For
The Proposed Gray's Side Road
Underpass, Q.E.W., in the City
of Hamilton, District No. 4
W.J. 66-F-1 -- W.P. 207-63

1. INTRODUCTION:

In a memo dated December 1, 1965, the Bridge Location Section requested a foundation investigation for the above structure.

The investigation was carried out by our Section. Presented in this report are the results of the completed field and laboratory studies as well as recommendations pertaining to the foundations of the proposed structure.

2. DESCRIPTION OF THE SITE:

The proposed bridge site is located in the City of Hamilton, some 2 miles east of the Burlington Bay Skyway at the Q.E.W. and Gray's Road crossing. The location is a relatively flat, built-up area. The Q.E.W. at this location runs parallel to the shoreline of Lake Ontario and is approx. 0.2 - 0.3 miles south of it.

Physiographically, the region is known as the Iroquois Plain. The Plain is built on shallow sand soils underlain by clayey deposits of low permeability, which in turn, overlay red-coloured layers, derived from the underlying Queenston shale bedrock.

3. FIELD INVESTIGATION:

The field investigation consisted of six sampled boreholes and five cone penetration tests. Borehole #6 which was drilled in March 1965, is also incorporated in this report. Borings were carried out by a continuous flight "Penn. Auger", while the bedrock was proved using a conventional diamond drill rig. For the standard penetration tests a driving energy of 350 ft. lb. was used. In the cohesive

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

deposits "undisturbed" Shelby tube samples were recovered by pushing the tubes into the soils manually. Field vane tests were performed to determine the shear strength of the clayey silt soils according to D.H.O. procedures.

The locations and elevations of the boreholes may be seen on Drawing No. 66-F-1A accompanying this report.

4. SOIL CONDITIONS:

4.1) General:

Soil samples were visually examined and identified in the field as well as in the laboratory. Standard laboratory tests were done on representative samples in order to define the shear strength characteristics, moisture contents, grain size distributions, densities and the plasticities of the deposits. The results of all tests together with the soil strata, are plotted on the Borelog sheets appended to this report. The estimated stratigraphical profile on Drawing No. 66-F-1A is based on the borelogs.

A brief description of the various soil strata follows:

4.2) Clayey Silt with traces of Sand and Gravel:

This material was found in every borehole, extending from ground elevation down to a depth of approx. 60 ft. (El. 195.0' - 199.0') in the deeper holes, and to the bottom of the holes shallower than 60 ft. The upper 12 - 15 ft. portion of the stratum is brown-coloured, becoming grey below that depth. The consistency is firm to very stiff and occasionally hard. There is a slight increase of the shear strengths with depth. The average laboratory unconfined shear strength down to elevation 235 ft. was found to be 1,000 p.s.f. and approx. 1,500 p.s.f. below this elevation. The results of the field vane tests are somewhat higher than the unconfined tests, especially in the lower elevations. The average value of the plastic limit of the layer is 18%, the liquid limit being approx. 30%. The bulk densities, as determined in the

cont'd. /3

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

4.2) Clayey Silt with traces of Sand and Gravel: (cont'd.) ...
laboratory, vary between 125.0 p.c.f. and 136.0 p.c.f. The material has low sensitivity, averaging roughly $S = 1.5$.

4.3) Silt to Sandy Silt with traces of Gravel:

Underlying the clayey silt in borehole No's 1, 4 and 6, a reddish-brown silt to sandy silt layer was observed with traces of gravel. The upper surface of the stratum lies at the approx. elevation of 195.0 ft., extending to the bedrock at approx. 178.5 ft. The relative density of this material is very dense, corresponding to standard penetration 'N' values much in excess of 100 blows/ft. The soil is predominantly of a granular nature; however, occasionally it exhibits a slight plasticity.

4.4) Queenston Shale Bedrock:

The bedrock was proved in borehole No. 6, by drilling with a diamond core barrel from El. 178.5 ft. to El. 168.5 ft., a thickness of 10 ft. The bedrock displays a reddish-brown colour, so typical of the Queenston shales. It is stratified with greenish, shaley carbonate. The recovery at the upper 5 ft. was found to be 70%, while 100% recovery was achieved below, indicating some weathering of the rock at the upper portion.

5. GROUND WATER CONDITIONS:

Ground water level observations were carried out at every borehole location each day during the field investigation. The observed ground water levels at the end of the field work, are listed as follows:

cont'd. /4

5. GROUND WATER CONDITIONS: (cont'd.) ...

No. of Borehole	El. of Groundwater (ft.)
1	251.6
2	245.5
3	252.8
4	254.0
4A	252.8
5	252.5
6	258.0

It is to be noted, that in B.H.'s #1, 2, 3 and 4A, the water levels were still rising at the end of the observation period, so that water levels in excess of the ones marked may be anticipated.

6. DISCUSSION AND RECOMMENDATIONS:

It is proposed to reconstruct the existing Q.E.W. as a controlled access highway from Stoney Creek traffic circle to St. Catharines. In addition, two-lane service roads are proposed to be built on both sides of the Q.E.W. This reconstruction program necessitates the construction of several underpass structures.

At the crossing of Gray's side rd. and Q.E.W., an underpass structure is proposed. Present proposals call for a six-span (38'-57'-77'-77'-57'-38') structure with approach fills having a maximum height of about 22' above existing ground level.

Since the upper 12 to 15 ft. of the subsoil consist of very stiff to hard brown clayey silt with traces of sand and gravel, conditions are favourable for spread footing support, and in the case of the proposed piers, it is recommended that footings be placed at approximate elev. 255 with an allowable pressure of 2 t.s.f.

cont'd. /5

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

The proposed abutments may be constructed within the approach fills and supported on 12" \emptyset displacement piles driven to but not beyond elev. 245.0. A 12" \emptyset pile could carry an allowable load of 30 tons/pile. During construction of the approaches, care should be taken to ensure that no bouldery fill is placed at the locations through which piles have to be driven.

An analysis based on Skempton and Bjerrum's* method has been made to estimate the settlement under the pier footings and embankment load. It is assumed in the calculations that $\mu = 0.6$.

Results of the analysis are as follows:

- 1) Ultimate settlement at the end pier locations
Induced by the footing pressure of 2 t.s.f. -
(footing size 36' x 3') $\approx 3.0"$
Induced by embankment load (22-ft. height) $\approx 2.0"$
Total $\approx 5.0"$
- 2) Ultimate settlement at the intermediate and
centre pier locations -
(footing size 36' x 8') $\approx 3.0"$
- 3) Ultimate settlement at the abutment location
Induced by embankment load (22-ft. height) $\approx 9.0"$

In addition to the aforementioned consolidation settlements, there will be elastic or immediate settlements. It can be assumed that the elastic settlements will take place during and immediately after construction.

If the footings and embankments are constructed at the same time, the maximum differential settlements between the end and intermediate piers will be in the order of 2". The differential settlements between the end piers and the abutments will be in the order of 4".

* Skempton, A. W. and Bjerrum, L. -- "A Contribution to the Settlement Analysis of Foundations on Clay" -- Geotechnique 1958, p. 168.

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

If spread footings are adopted, the approach embankments should be built at least 6 months or more prior to the construction of the structure foundations. This sequence will take care of the elastic settlements and part of the consolidation settlement induced by the approach fills at the outer pier and abutment locations.

As an alternative, the entire structure can be supported on steel H-piles driven to practical refusal some 10 ft. into the very dense silt stratum. For 14 BP 73, a safe design load of 80 tons/pile can be used.

No major dewatering problems are anticipated during the construction of footings, in view of the low permeable nature of the subsoil; however, care should be taken to prevent softening of the subsoil at the footing levels by the surface run-off.

No stability problems are anticipated provided that standard 2:1 slopes are constructed.

7. MISCELLANEOUS:

The field work, performed during the period January 4 to 17, 1966, was undertaken by Mr. P. L. Wang, Project Foundation Engineer, under the general supervision of Mr. M. Devata, Senior Foundation Engineer, who also prepared the report.

Equipment used was owned and operated by Dominion Soil Investigation Ltd.



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-P-1 LOCATION E.W. & Gray's Rd Sta. 28+15 10' Lt.ORIGINATED BY P.L.W.W.P. 207-63 BORING DATE Jan. 4 - 9, 1966.COMPILED BY P.L.W.DATUM Geodetic BOREHOLE TYPE Penn Auger.CHECKED BY M.D. C.J.D.

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		20	40	60	80	100	WP	W	WL		
255.5	Groundlevel														
243.5 12	Brown (Grey) Grey clayey silt with traces of sand and gravel. Firm to very stiff.		1	SS	12	250									
			2	SS	37										
			3	SS	20										
			4	TW	PH	240								133	
			5	TW	PH									133	
			6	TW	PH									128	
			7	TW	PH	230								133	
			8	TW	PH										
			9	TW	PH									132	
			10	TW	PH	220								136	
			11	TW	PH									138	
			12	TW	PH									127	
			13	TW	PH	200								134	
			14	TW	PH									134	
			15	SS	100/2"										
195.5 60	Reddish brown silt to sandy silt with traces of gravel. Very dense.		16	SS	120/4"	190									
			17	SS	100/1"										
			18	SS	150/2"										
180.5	End of borehole.														
75															

15 20 45 Percent strain at failure

S= Sensitivity

Gr2%Sa5%
Si&Cl 93%

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-1 LOCATION C.E.W. & Gray's Rd Sta. 28+62 17' Rt. ORIGINATED BY P.L.W.
W.P. 207-63 BORING DATE Jan. 9, 1966. COMPILED BY P.L.W.
DATUM Geodetic BOREHOLE TYPE Penn Auger CHECKED BY M.D. *W.D.*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 — SHEAR STRENGTH P.S.F. + Field Vane Test • Unconfined Compression Test 1000 2000 3000	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W Wp — W — WL WATER CONTENT % 15 30 45	BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT						
256.5	Groundlevel										
0	Brown (Gray) Clayey silt with traces of sand and gravel. Firm to very stiff.		1	SS	13						
			2	SS	22		250				
			3	SS	44						
245.0			4	SS	25						
11.5			5	TV	PH		240				132
			6	TV	PH						131
233.5											
23.0	End of borehole.					230					

20
15+5 Percent strain at failure.
S=Sensitivity

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-1

LOCATION S.E.W. & Gray's Rd. Sta. 29+23 12' Lt.

ORIGINATED BY P.L.W.

W.P. 207-63

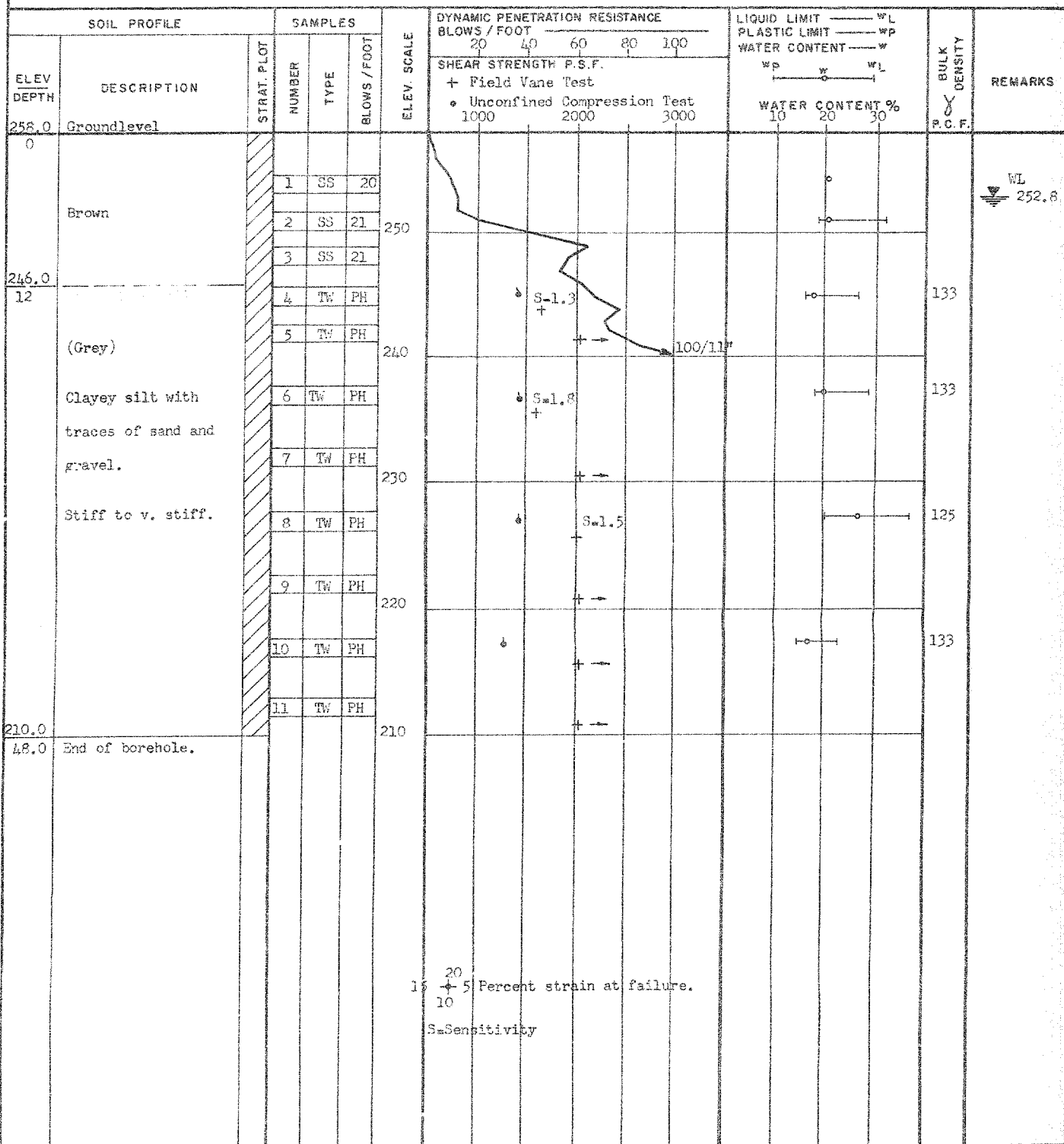
BORING DATE Jan. 11 - 12, 1966.

COMPILED BY P.L.W.

DATUM Geodetic

BOREHOLE TYPE Penn Auger

CHECKED BY M.D. 152



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 66-F-1 LOCATION C.B.W. & Gray's Rd. Sta. 30467 16' Lt. ORIGINATED BY P.L.W.
 W.P. 207-63 BORING DATE Jan. 12 to 14, 1966 COMPILED BY P.L.W.
 DATUM Geodetic BOREHOLE TYPE Penn Auger CHECKED BY M.D. *MD*

[illegible]

FOUNDATION SECTION

CHECKED BY M.D. 400

SOIL PROFILE			SAMPLES			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	SHEAR STRENGTH P.S.F. + Field Vane Test • Unconfined Compression Test			WATER CONTENT %				
						1000	2000	3000	10	20	30		
259.0	Groundlevel												
0													
	Brown		1	SS	32								
			2	SS	27								
			3	TW	PH								
	(Grey) Clayey silt with traces of sand & gravel Stiff to very stiff.		4	TW	PH								
236.0													
23.0	End of borehole.												

W.L.
252.8

δ + S=1.2

δ + S=1.2

133

132

20
15 + 5 Percent strain at failure.
10

S=Sensitivity

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 66-F-1

LOCATION Q.E.W. & Gray's Rd Sta. 30+76.28' Rt.

ORIGINATED BY P.L.W.

W.P. 207-63

BORING DATE Jan. 17, 1966.

COMPILED BY P.L.W.

DATUM Geodetic

BOREHOLE TYPE Penn Auger

CHECKED BY M.D. *l.d.*

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	WL		
260.0	Groundlevel														
0	Brown		1	SS	26										
			2	SS	27										
			3	SS	33	250									
			4	TW	PH										
248.0			5	TW	PH										
12.0	(Grey) Clayey silt with traces of sand and gravel. Stiff to very stiff.		6	TW	PH										
			7	TW	PH	240									
237.0															
23.0	End of borehole.														

20
15 + 5 Percent strain at failure
10
S= Sensitivity

WL 252.5
GrO%Sa3%
Si56%Cl 41%

+ >2000
+ >2000
100/9" = 9050
+ S=1.4
+ S=1.7

FOUNDATION SECTION

CHECKED BY M.D. 122

[illegible]

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
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. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: December 1, 1965.

Our File Ref.

In Reply To

66-5-1

SUBJECT: Gray's Side Road Underpass, W.P. 207-63,
Millens Road Underpass, W.P. 208-63,
Fruitland Road Underpass, W.P. 209-63,
Glover Road Underpass, W.P. 210-63,
Hwy. Q.E.W. - Dist. 4.

Attached please find a plan showing the proposed underpasses with the probable location of footings shown in red.

Would you kindly arrange foundation investigations at the above sites and provide us with the information necessary to design the structures.

All structures will carry the side-roads over the existing Q.E.W. with approximately 20' of fill on the approaches.

Your office has completed a preliminary foundation report (Q.E.W. from Stoney Creek Traffic Circle to St. Catherines) under W.J. 65-F-28.

WSM/sp

W. S. Melinyshyn

W. S. Melinyshyn,
Regional Bridge Location Engineer.

cc. A. Crowley
R. Forrest

COMPLETION DATE FEB 22, 1966

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario

DATE: June 21, 1966

OUR FILE REF.

IN REPLY TO:

SUBJECT:

W.P. 207-63, Site 36-203
Gray's Road Underpass
W.P. 208-63, Site 36-204
Millens Road Underpass
W.P. 210-63, Site 36-206
Glover Road Underpass
Q.E.W. District 4

bb-f-1 ✓

bb-f-2

bb-f-3

Herewith is one print each of our preliminary drawings D-5907-P1, D-5912-P1 and D-5903-P1 for the above structures. Please note that it is proposed to construct the abutments on spread footings on granular fill. Is this satisfactory to your department?

JFW/pr
Encl.


W. S. Melinyshyn,
Regional Bridge Location Engineer

Mr. W. S. Melinyshyn,
Regional Bridge Location Engr.,
Bridge Division, Admin. Bldg.

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

June 28, 1966

W.P. 207-63, Gray's Rd. Underpass (W.J. 66-P-1) ✓
W.P. 208-63, Millens Rd. Underpass (W.J. 66-P-7)
W.P. 210-63, Clover Rd. Underpass (W.J. 66-P-6)

We have reviewed the preliminary drawings D-5907-P1, D-5912-P1, and D-5903-P1, for the above mentioned structures and submit the following comments pertaining to abutment foundations on granular fills:

1) In our opinion, constructing the proposed perched abutments on granular fills for the above mentioned projects, is quite satisfactory. In this case, 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. A design load of 2 t.s.f. may be used for the abutment foundations.

2) It is believed that the designer has taken into account the cost of G.B.C. class 'A' material for these projects at the approach fill locations. Mr. T. J. Kovich, Regional Materials Engineer, indicated to us that the estimated cost of the material will be in the order of \$2.50/ton.

3) In order to reduce the differential settlements between the piers and abutments, consideration should be given to constructing the fills for as long a period as possible, in advance of the bridge construction. This Section would like to install settlement plates at all of the above sites prior to the commencement of approach fill operations.

MD/MdeF

cc: Foundations Office
Gen. Files

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

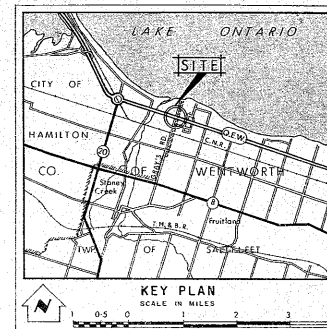
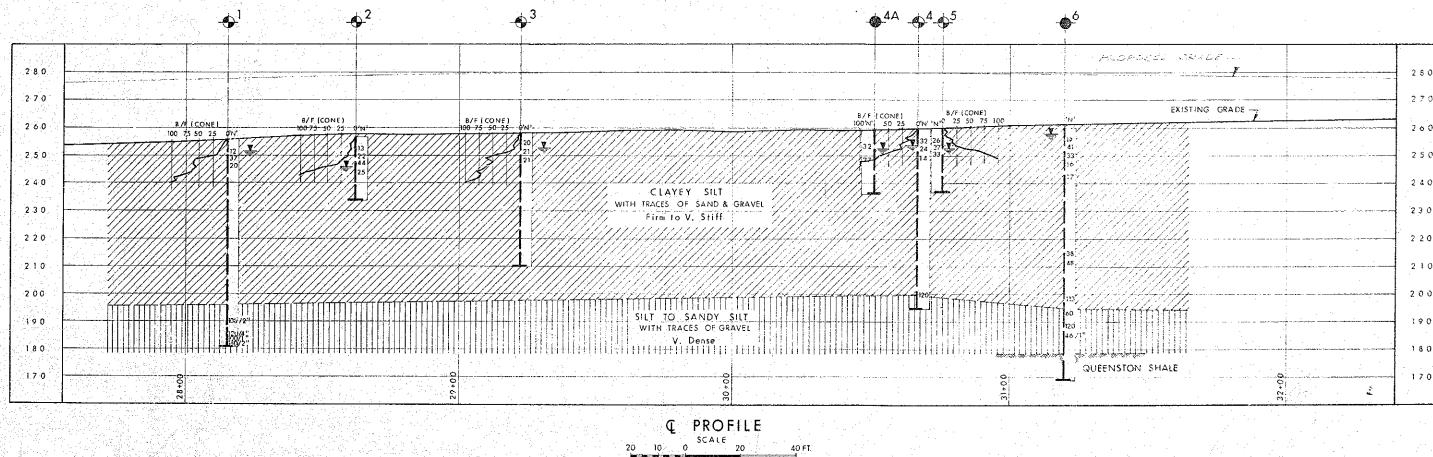
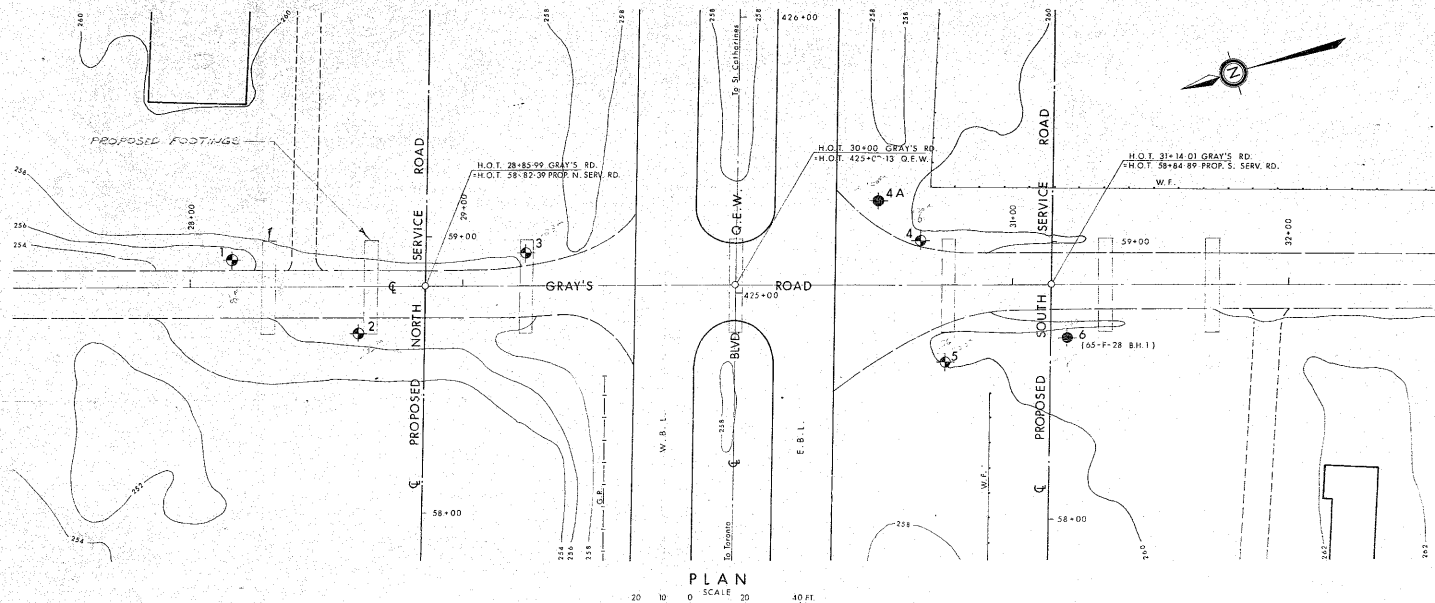
#66-F-1

W.P. #207-63

G.E.W. &

GRAY'S SIDE

ROAD



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation. JAN. 1966		

NO.	ELEVATION	STATION	OFFSET
1	255.5	28+15	10' LT.
2	256.5	28+62	17' RT.
3	258.0	29+23	12' LT.
4	259.5	30+67	16' LT.
4A	259.0	30+52	30' LT.
5	260.0	30+76	28' RT.
6	260.0	31+20	19' 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.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO
WATERWAYS & TESTING DIVISION - FOUNDATION SECTION

GRAY'S ROAD

KING'S HIGHWAY NO. Q.E.W. DIST. NO. 4
CO. WENTWORTH CITY OF HAMILTON
TWP. _____ LOT _____ CON. _____

BORE HOLE LOCATIONS & SOIL STRATA

SUBNO. P.W. CHECKED ☒ W.P. NO. 207-63 E.A.T. DRAWING NO. _____
DRAWN S.O. CHECKED ☒ JOB NO. 66-F-1-A **66-F-1A**
DATE 15 FEB. 1966 SITE NO. _____ BRIDGE DRAWING NO. _____
APPROVED *[Signature]* FOR NO. _____

REF. NO. E-4725-1

PRINT RECORD	NO.	FOR	DATE