

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 9, 1966

OUR FILE REF.

IN REPLY TO

MAR 15 1966

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Millen Road (Revised)
Underpass at Q.E.W., Township of
Saltfleet, District #4 (Hamilton)

W.J. 66-F-7

--

W.P. 208-63

*Included with
W.P. 206-63-2.*

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 feel free to contact our Office.

AGS/MdeF
Attach.

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

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 ✓

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FOUNDATION INVESTIGATION REPORT
For
Proposed Millen Road (Revised)
Underpass at Q.E.W., Township of
Saltfleet, District #4 (Hamilton)
W.J. 66-F-7 -- W.P. 208-63

1. INTRODUCTION:

A request to carry out a foundation investigation for the proposed underpass at the crossing of Q.E.W. and the relocated Millen Rd., was received from the Bridge Location Section - (memorandum from Mr. W. A. Melinyshyn, dated December 1, 1965). Subsequently, an investigation consisting of four sampled boreholes and two dynamic cone penetration tests, was carried out by the Foundation Section.

Presented in this report are the results of this investigation, together with our recommendations for the foundation design of the proposed structure and approaches.

The site is a portion of the Niagara Fruit Belt lying between the Niagara Escarpment and Lake Ontario, in the Township of Saltfleet, about 2 miles east of the easterly limits of the City of Hamilton. During the Pleistocene period the site was inundated by Lake Iroquois which carved the present relatively flat topography from the underlying glacial deposit. The glacial deposit extends to the bedrock (Queenston shale).

2. SUBSOIL CONDITIONS:

2.1) General:

Subsoil at the site generally consists of 43 to 60 ft. of clayey silt with sand and occasional gravel followed by shale bedrock. In B.H. #3 only, a shallow deposit of silty sand was observed to occur both at the upper and lower extremities of the main clayey silt deposit.

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

2.2) Clayey Silt with Sand and occasional Gravel:

This material was encountered right below the topsoil in each borehole, except in B.H. #3 where it was overlain by some 5 ft. of dense silty sand. The upper 10 ft. of the desiccated crust was found to be very stiff to hard with 'N' values ranging from 22 to 52 blows/ft. Immediately below the desiccated crust, the non-oxidized stratum of clayey silt was encountered.

The shear strength of the non-oxidized deposit varied from a minimum value of 900 p.s.f. at about elev. 247, generally increasing with depth, with a maximum of 3,600 p.s.f. around elev. 225. Below elev. 225 the cohesive deposit is essentially hard with 'N' values ranging from 31 to 195 blows/ft., generally increasing with depth.

The following is a summary of the physical properties of the clayey silt deposit:

Liquid Limit	(W _L %)	=	23% - 35%
Plastic Limit	(W _p %)	=	14% - 22%
Moisture Content	(W %)	=	8% - 23%
Bulk Density	(γ)	=	130 - 137 p.c.f.
Shear Strength (Elev. 247 - Elev. 225)			
Unconfined comp. tests		915 -	3,620 p.s.f.
In-situ vane tests		1,200 -	>2,000 p.s.f.

2.3) Silty Sand with Gravel:

This deposit was observed in B.H. #3 only, above and below the clayey silt deposit. This stratum extends from ground surface down to elev. 252.8 and also between elev. 214.8 and elev. 207.6. The material is in a dense to very dense state with 'N' values being in the order of 43 to 100 blows/2".

cont'd. /3

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

2.4) Bedrock - Red Shale:

Bedrock contact was estimated to be between elev. 209 and elev. 200. The upper contact of the bedrock was somewhat difficult to determine due to the presence of weathered material which resembled the overlying clayey material very closely.

3. WATER CONDITIONS:

Observations carried out during the time of the field investigation, indicated that the water level in the boreholes was approximately between elev. 257 and elev. 255. The exact water levels are shown on the borehole logs.

4. 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 Millen Road revision and Q.E.W., an underpass structure is proposed. Present proposals call for a six-span (38'-58'-78'-58'-38') structure with approach fills having a maximum height of about 22 ft. above existing ground level.

Since the upper 10 to 12 ft. of the subsoil consist of 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. 253 with an allowable pressure of 2.5 t.s.f.

The proposed abutments may be constructed within the approach fills and supported on 12" Ø displacement piles driven to, but not beyond elev. 250.0. A 12" Ø pile could carry an allowable load of 30 tons per pile. Care should be taken to ensure that no bouldery fill is placed at the locations through which piles have to be driven.

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

Due to the fact that the subsoil consists of some 30 ft. of compressible material, settlements below the embankments and below the footings, must be anticipated. The type of structure to be built will be dependent on the magnitudes of the differential settlements between the piers and abutments. Settlements due to the proposed imposed loads, have been computed and are as follows:

- 1) Ultimate settlements at the end pier location
Induced by the footing pressure of 2.5 t.s.f.
(footing size 36' x 8') \approx 2.0"
Induced by embankment load (22-ft. height) \approx 1.2"
Total \approx 3.2"

- 2) Ultimate settlement at the centre and intermediate pier locations \approx 2.0"
- 3) Ultimate settlement at the abutment locations
(Induced by embankment) \approx 4.5"

These figures, of course, represent long-term settlements, the time being estimated to be about 50 years.

In order to reduce the differential settlements between the piers and abutments, consideration should be given to constructing the fill for as long a period as possible, in advance of the bridge construction.

No major dewatering problems are anticipated during 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.

cont'd. /5

5. MISCELLANEOUS:

The field work, performed during the period January 17 to January 28, 1966, was undertaken by Mr. L. Palmer, Project Foundation Engineer, and Mr. H. Szymanski.

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

The general supervision of the project and the preparation of the report were carried out by Mr. M. Devata, Senior Foundation Engineer.

March 1966

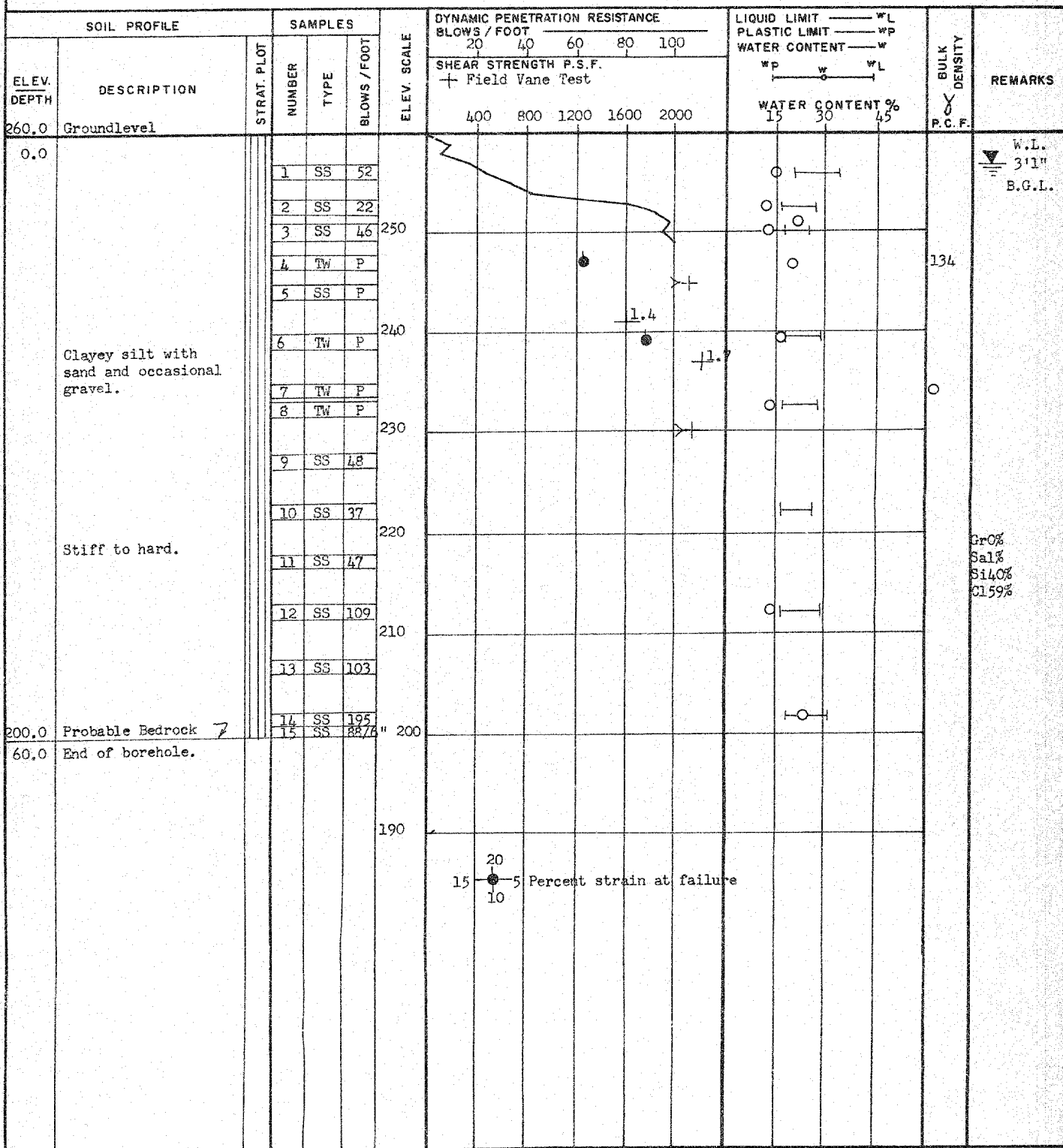
APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-7 LOCATION C.E.W. & Millen Rd. Rev'n Sta. 31+73 17' Lt. ORIGINATED BY L.P.
W.P. 208-63 BORING DATE Jan. 17, 1966. COMPILED BY _____
DATUM Geodetic BOREHOLE TYPE Dynamic Cone & Penn Auger CHECKED BY AK



FOUNDATION SECTION

ORIGINATED BY H.S.

COMPILED BY

CHECKED BY

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. PILOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. + Field Vane Test • Unconf. Comp. Test 400 800 1200 1600 2000			WATER CONTENT % 15 30 45					
258.7	Groundlevel														
	Clayey silt with sand and occasional gravel.		1	SS	50	250								▽ W.L. 2'11" B.G.L.	
			2	SS	41										
			3	SS	36										
				4	TW	P	240								136
				5	TW	P									134
				6	TW	P									130
				7	TW	P	230								135
				8	TW	P									142
				9	SS	48									
				10	SS	31	220								
				11	SS	60									
				12	SS	150									
202.2	Probable Bedrock	7	13	SS	123/6"	200									
56.5	End of borehole.														
						190									
						20									
						15									
						10									

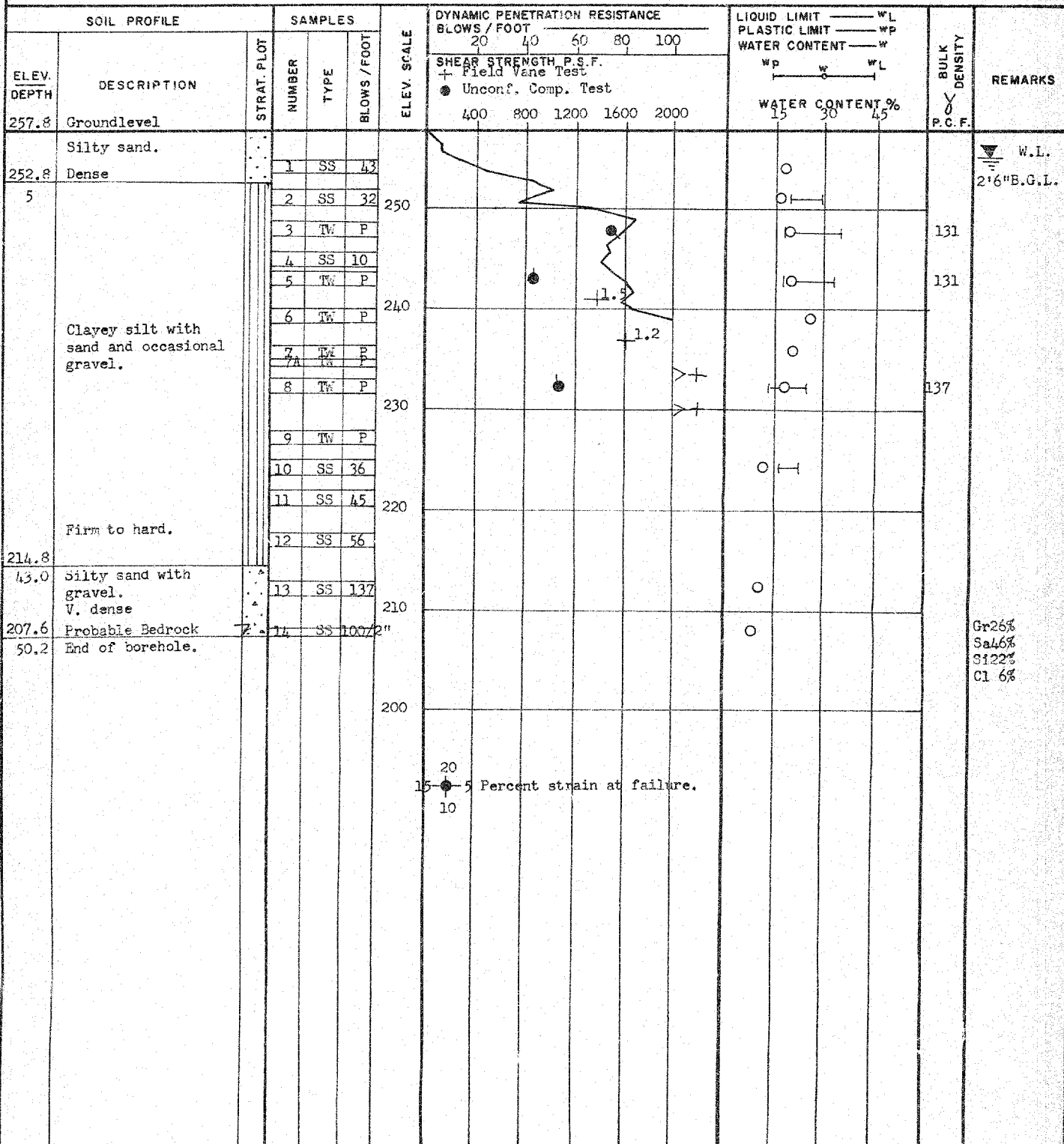
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-7 LOCATION S.E.W. & Millen Rd. Rev'n Sta. 28+26 15' Rt. ORIGINATED BY P.L.W.
W.P. 208-63 BORING DATE Jan. 27, 1966. COMPILED BY _____
DATUM Geodetic BOREHOLE TYPE Dynamic Cone & Penn Auger CHECKED BY HR

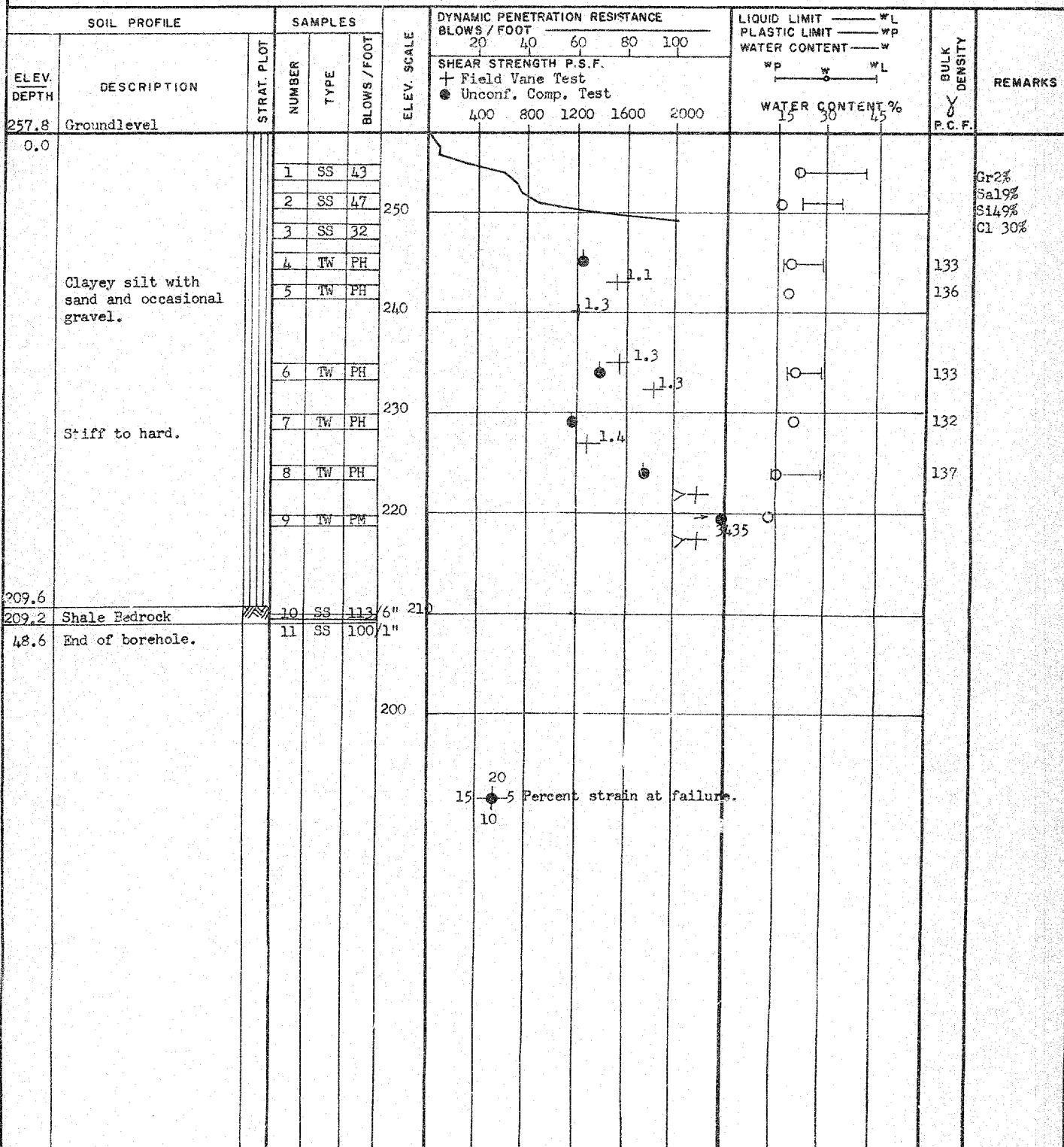


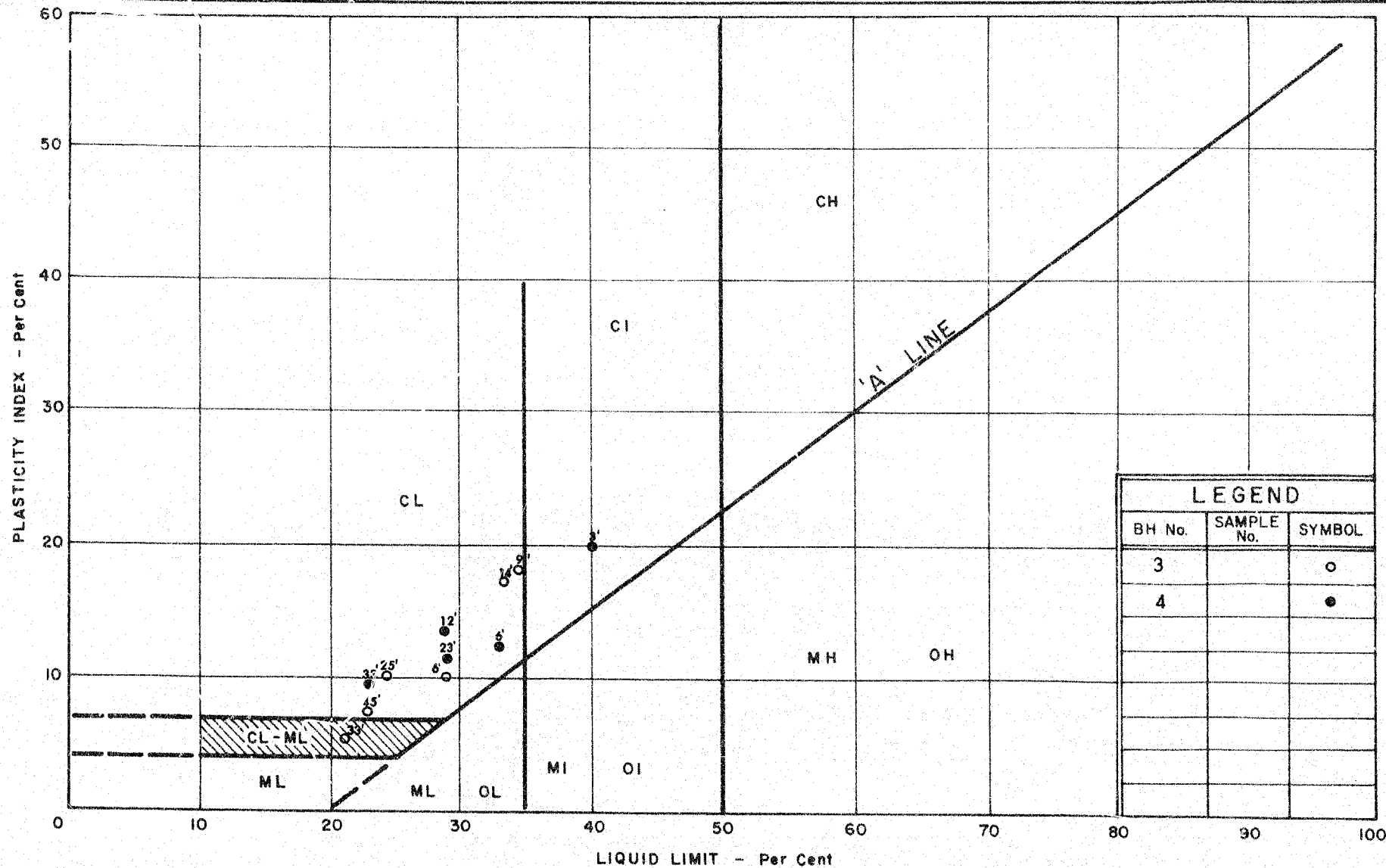
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 66-F-7 LOCATION Q.E.W. & Millen Rd. Rev'n Sta. 29/22 17' Lt. ORIGINATED BY H.S.
W.P. 208-63 BORING DATE Jan. 20, 1966. COMPILED BY _____
DATUM Geodetic BOREHOLE TYPE Dynamic Cone & Penn Auger CHECKED BY SP





DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

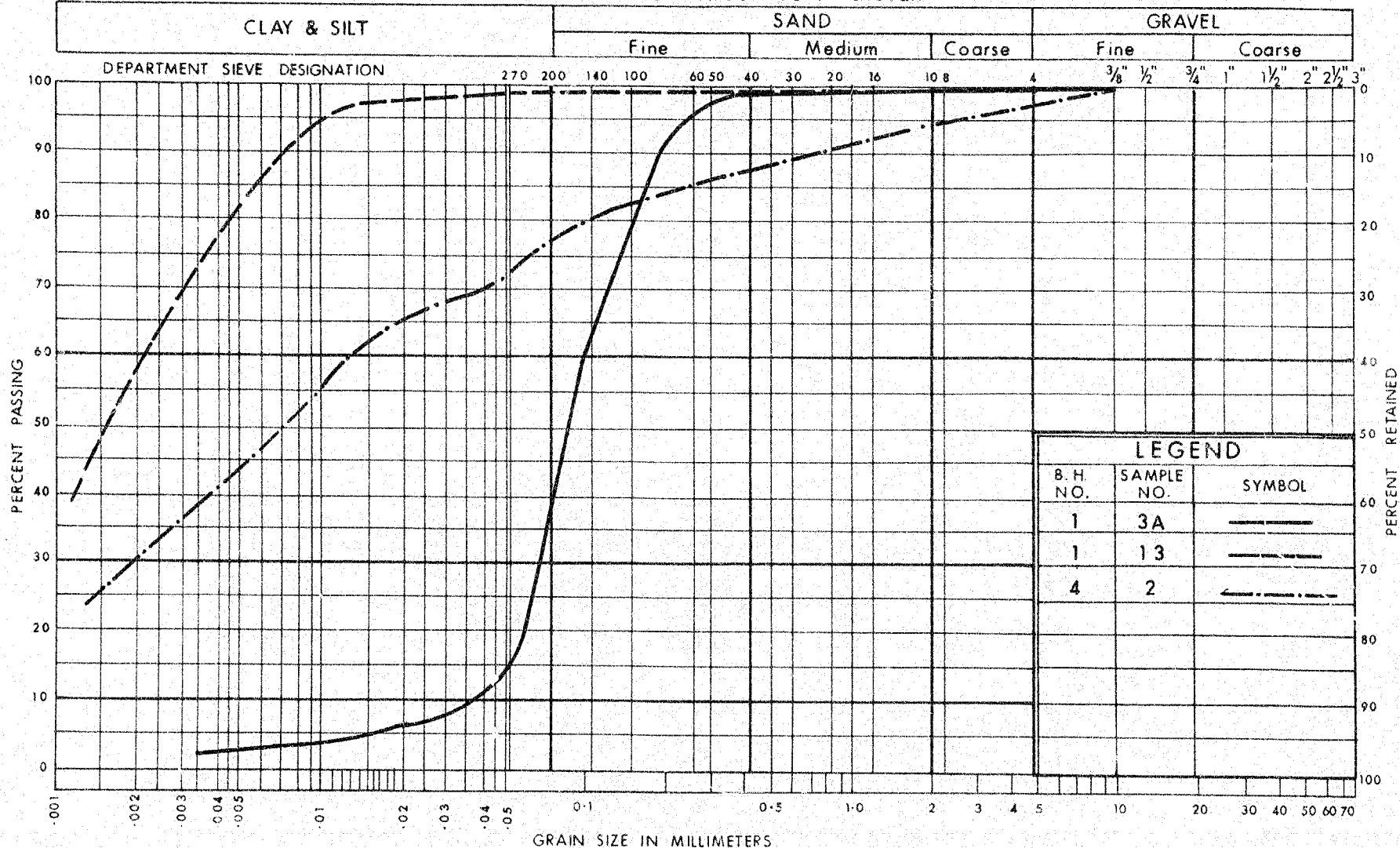
PLASTICITY CHART

Q.E.W. & MILLEN RD.

W.P. No. 208 - 63

JOB No. 66 - F - 7

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

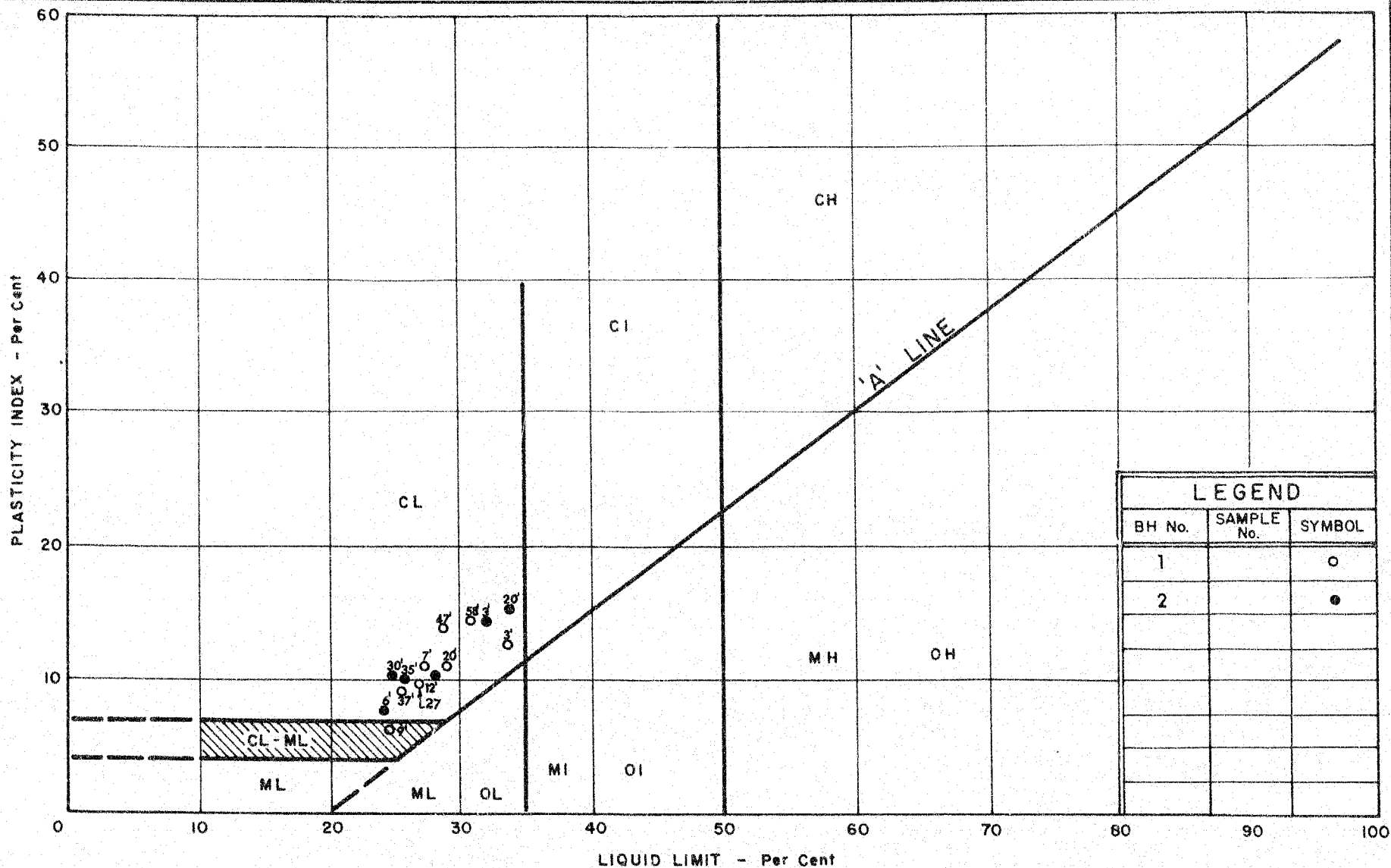
ONTARIO

GRAIN SIZE DISTRIBUTION

CLAYEY SILT WITH SAND AND OCCASIONAL GRAVEL

W.P. No. 208-63

JOB No. 66-F-7



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART

Q.E.W. & MILLEN RD.

W.P. No. 208 - 63

JOB No. 66 - F - 7

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
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_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. Steward,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: December 1, 1963.

Bus File Ref.

IN REPLY TO

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. Catherine's) under W.J. 65-F-28.

WSM/sp

W. S. Melnyshyn
W. S. Melnyshyn,

Regional Bridge Location Engineer.

cc. A. Crossley
R. Forrest

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

COPIES OF THIS REPORT ARE 23

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

Foundation Section,
Materials & Testing Div.,
Room 167, Lab. Bldg.

June 28, 1966

W.P. 207-63,	Gray's Rd. Underpass	(W.J. 64-F-1)
W.P. 208-63,	Hillans Rd. Underpass	(W.J. 66-F-7)
W.P. 210-63,	Clover Rd. Underpass	(W.J. 66-F-6)

We have reviewed the preliminary drawings D-5912-F1, D-5912-F1, and D-5963-F1, 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. Kevich, 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.

ND/MACF

cc: Foundations Office
Gen. Files

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

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

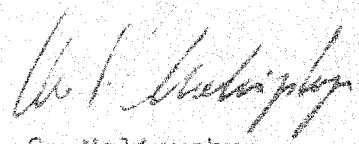
IN REPLY TO:

SUBJECT:

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

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. Melnyshyn,
Regional Bridge Location Engineer

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

MEMORANDUM

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

From: Bridge Division,
Downsview, Ontario.

Date: December 1, 1965. 66-1-7

Our File Ref.

In Reply To

Subject: Gray's Side Road Underpass, W.P. 207-63,
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WSM/sp

cc. A. Crowley
R. Forrest

W. S. Melnyshyn
W. S. Melnyshyn,
Regional Bridge Location Engineer.

COMPLETED DATE FEB 23 1966

AS OF MARCH 2. 1966

W.P.	SITE	DISTRICT	DUE DATE
✓ 207-63	GREY'S SIDE RD U'PASS QEW.	4	✓ FEB. 23. 1966.
✓ 210-63	GLOVER RD. U'PASS QEW.	4	FEB. 23. 1966
333-63	DISTRESS R. BRIDGE SEC. RD. 520.	11	FEB. 9. 1966
417-65-1	RETAINING WALLS AT QEW. & WICKMAN RD OVERPASS	6	URGENT
48-65	LA FONTAIN CR. STRUCTURE HWY 17 E. OF OTTAWA	9	APRIL 16. 1966
430-64	MOODY DRIVE INTERCHANGE 1.8 M. W. OF JCT. HWY 15. OTTAWA	9	MAY 15. 1966
✓ 698-64	BLANCHE R. AT SWASTIKA HWY 66.	14	URGENT
✓ DAY LABOUR	MICKLE CR. BR. SEC. RD. NO 560 6.7 M. W. OF ELK LAKE		URGENT

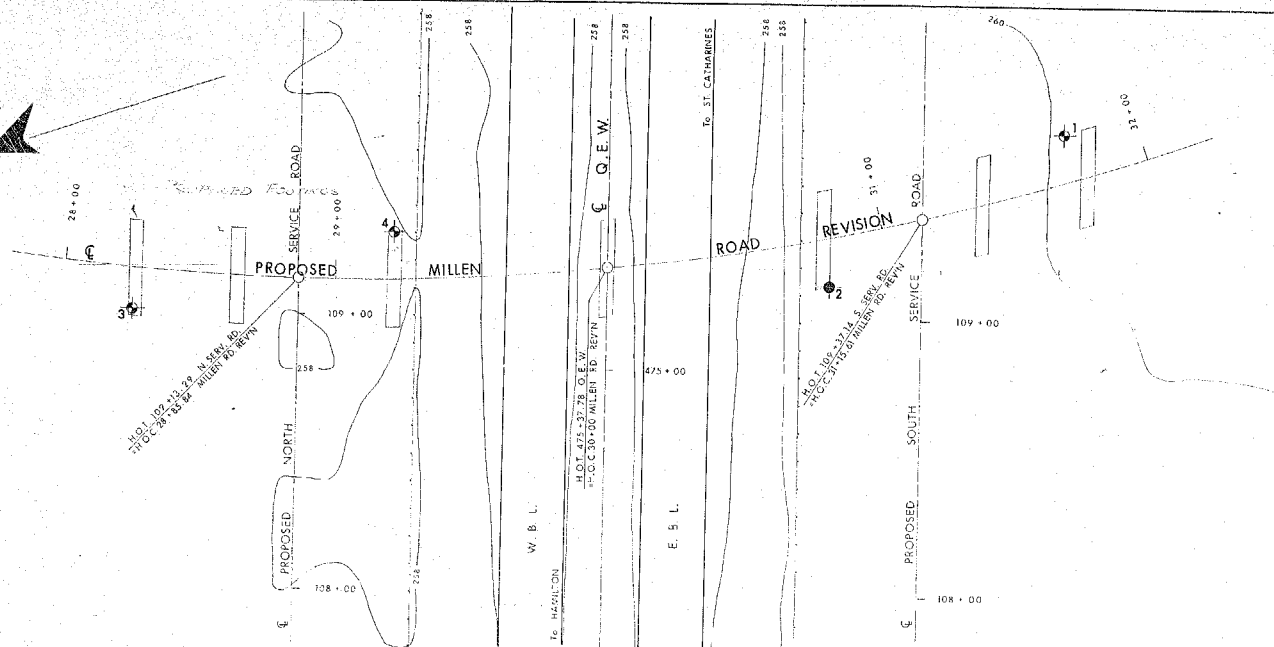
W. P.	SITE	DISTRICT	DUE DATE
909-64	RICHMOND RD. (HWY 17) U'PASS INTERCHANGE - QUEENSWAY	9	URGENT
108-65	CNR O'HEAD 2.7 M. WEST OF JCT HWY 15. OTTAWA	9	MAY 15. 1966
431-64	COUNTY RD 40 INTERCHANGE U'PASS 4.7 M. W. OF JCT HWY 15	9	MAY 16. 1966
85-59-7	SEWER AT DIXON RD, KIPLING AV. 2 HWY 401	6	URGENT
270-62	HWY 63 RELOCATION TROUT LAKE, NORTH BAY	13	QUITE URGENT

#66-F-7

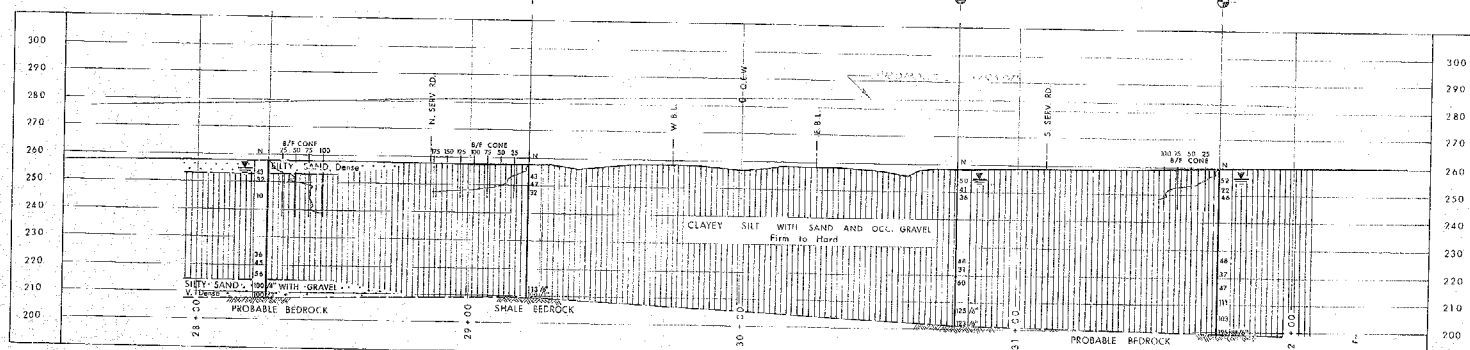
W.P.#208-63

Q.E.W. &

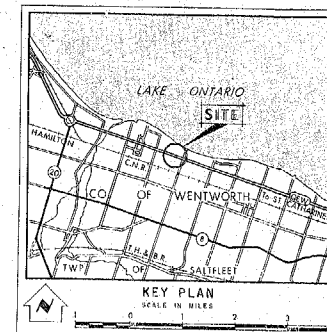
MILLEN RD.



PLAN
SCALE IN FEET
0 20 40 60



PROFILE
SCALE IN FEET
0 20 40 60



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	260	31+73	17'1
2	254.7	30+78	17'1
3	257.8	8+2	15'0
4	257.8	19+22	17'1

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

DATE	BY	DESCRIPTION

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

MILLEN ROAD REVISION

KING'S HIGHWAY NO. Q.E.W. DIST. NO. 4
CO. WENTWORTH
TWP. SALTFLY LOT 19 CON. BF & 2

BORE HOLE LOCATIONS & SOILS STRATA

SUBMIT. LP	CHECKED 477	REF. NO. 208-43	MAT. DRAWING NO.
DRAWN DCH	CHECKED 477	JOB NO. 66-7-3	66-F-7A
DATE 23 FEB 1966	SIC NO.		BRIDGE DRAWING NO.
APPROVED 477	CONT. NO.		

REF. No. E-4726-1