

MEMORANDUM

cc: GEN. FILES ✓ 23-69-151

W.P. 217-62-2

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

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

Attention: Mr. S. McCombie

DATE: November 14, 1966.

OUR FILE REF.

IN REPLY TO

NOV 30 1966

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Q.E.W. Crossing of 16 Mile Creek
District #4

W.J. 66-F-62

W.P. 214-63-2

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:sm
Attach.

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

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

Foundations Office
General Files ✓

TABLE OF CONTENTS

1. INTRODUCTION
2. TOPOGRAPHY AND GEOLOGY
3. FIELD AND LABORATORY WORK
4. SUBSOIL CONDITIONS
 - 4.1) General
 - 4.2) Fill Material
 - 4.3) Organic Clay-Organic Silt
 - 4.4) Silty Sand
 - 4.5) Clayey Silt to Silt, Some Sand and Gravel
 - 4.6) Shale Bedrock
 - 4.7) Water Table
5. EXISTING STRUCTURES
6. DISCUSSION AND RECOMMENDATIONS
 - 6.1) General
 - 6.2) 3-Span Structure
 - 6.3) A Multispan Structure
 - 6.4) Flexible Pipe-Arch Culvert
7. SUMMARY
8. MISCELLANEOUS

APPENDIX I

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_s	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
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

FOUNDATION INVESTIGATION REPORT
For
Q.E.W. Crossing of 16 Mile Creek
District #4

W.J. 66-F-62

W.P. 213-63

66-F-63

1. INTRODUCTION:

A request, dated June 17, 1965, to conduct a foundation investigation at the proposed crossing of 16 Mile Creek by the North and South Service Roads of the Q.E.W. was received from Mr. W. S. Melinyshyn, Regional Bridge Location Engineer.

It is proposed to reconstruct the existing Q.E.W. as a controlled access highway from the Stoney Creek traffic circle to St. Catherines and to construct service roads north and south of the main road. This program will necessitate the construction of two 2-lane multispans to carry the service roads over 16 Mile Creek.

Subsequently, a foundation investigation was conducted at the proposed site to determine the subsoil conditions. Field and laboratory test results together with discussion and recommendations for the bridge foundations and embankment designs, are reported herein.

2. TOPOGRAPHY AND GEOLOGY:

The site is located within the Niagara Fruit Belt lying between the Niagara Escarpment and Lake Ontario and is approximately 2.5 miles east of Jordan Harbour, in the township of Louth, County of Lincoln.

During the Pleistocene period this area was inundated by Lake Iroquois which carved the relatively flat general topography from the underlying glacial till. As the lake level

receded much below the present level of Lake Ontario, 16 mile Creek cut a valley about 600 feet wide through the till. Later, the rise in Lake Ontario water level to approximately its present level, drowned the lower portion of the creek and created a lagoon and marsh cut off from the lake by a barrier beach. Water flow is to the north into Lake Ontario.

During construction of the Q.E.W., fill was placed across part of this lagoon leaving a narrow water course which was spanned by the existing bridge. The fill was about 5 to 6 feet in height; the approach embankments, built on this fill were about 13 feet in height.

At present the barrier beach and the area between it and the Q.E.W. right-of-way is being used as a recreational area. The high land on either side of the creek otherwise, is generally being used for orchard cultivation.

3. FIELD AND LABORATORY WORK:

Using conventional diamond drilling equipment adapted for soil sampling purposes together with a raft where necessary, 23 sampled boreholes and 25 dynamic cone penetration tests were carried out at the site. A driving energy of 350 ft.-lbs per blow was used for the dynamic cone penetration tests.

In cohesive materials 2-inch I.D. Shelby tube samples were obtained by manually pushing the tubes into the soil if possible. Otherwise, samples of cohesive and non-cohesive materials were obtained using a 2-inch O.D. split-spoon sampler driven according to the specifications of the Standard Penetration Test. In-situ shear strength was established where possible with a field vane test.

BXL size rock core samples were obtained from some of the boreholes to prove the bedrock.

Samples were visually examined and identified in the field and subsequently in the laboratory. Laboratory tests were conducted on selected representative samples to determine, where applicable, atterberg limits, bulk density, grain-size distribution, natural moisture content and shear strength. The shear strength was determined by means of laboratory vane, quick triaxial and unconfined compression tests.

Results of the laboratory and field tests together with the location and elevations of the boreholes are presented in the appendix of this report.

4. SUBSOIL CONDITIONS:

4.1) General:

The natural subsoil at the site consists mainly of a deposit of organic clay-organic silt underlain by a deposit of clayey silt to silt with some sand and gravel (till-like) and then shale bedrock. A portion of the site has been covered by a fill of about five feet in depth which is predominantly clayey-silt with some sand and gravel.

The stratigraphical profiles shown on drawings 66-F-62A and 66-F-62B are estimated from the field data which is contained on the attached Borehole Log sheets. The different soil deposits are described below.

4.2) Fill Material:

This material was placed prior to the construction of the existing bridge and is generally about 5 feet thick. The material is predominantly clayey-silt with some sand and gravel but in places varied to silt or silty sand to sandy silt. The consistency is variable from soft to stiff.

4.3) Organic Clay-Organic Silt:

This material was encountered in all boreholes (except borehole 18) immediately beneath the fill material or at ground surface outside the limits of the old fill. The deposit is 35 to 40 feet thick over most of the site and tapers out near the valley's edge. In the lagoon north of the Q.E.W. a 3 to 8 foot thick layer of very loose to compact silty sand was encountered at a depth of about 7 feet otherwise the deposit was fairly uniform. The material as sampled, varied from brown to grey-brown, to grey occasionally, and was highly organic with organic pieces generally visible. Well-decayed pieces of roots and wood were not uncommon particularly in the lower portion of the deposit. The samples were prone to expand and occasionally some gas bubbles formed under the shelby caps after the shelby tubes were sealed.

The shear strength of the deposit appeared to have been influenced by the old fill. Beneath the fill area the shear strength decreased with depth to a minimum at about elevation 240' (about 10 feet depth) and then increased with increasing depth. Within the bottom 5 to 10 feet of the deposit there was a tendency for the shear strength to decrease slightly. (Shear strength profiles are included in the appendix of this report).

Outside the influence of the fill area the shear strength started at a minimum and increased with increasing depth except in the bottom 5 to 10 feet of the deposit where there was a slight decrease.

From the shear strength profiles it will be noted that the field vane shear strengths are generally considerably higher than shear strengths obtained from laboratory vane tests, quick triaxial or unconfined compression tests. This is attributed in part to

sampling disturbance and to subsequent swelling of the samples as noted above.

Other physical properties of the deposit were quite variable as can be seen on the attached borehole log sheets.

4.4) Silty Sand:

This deposit was encountered only in the area of the lagoon north of the Q.E.W. It occurred within the organic silt-organic clay deposit at a depth of about 7 feet varied from 3 to 8 feet in thickness.

The relative density of this material varied from very loose to compact, being generally loose. 'N' value ranged from 3 to 12 blows per foot.

4.5) Clayey Silt to Silt, Some Sand and Gravel:

This material underlies the organic silt-organic clay deposit and varies in thickness from about 3 to 11 feet over most of the site. On the extreme east and west this deposit becomes thicker as the organic silt-organic clay peters out. The deposit is generally till-like in appearance. Usually the lower 1-2 feet of the deposit was red-brown in colour and very dense. Occasional boulders were encountered at the bottom of the deposit, just above the shale bedrock.

4.6) Shale Bedrock:

Bedrock consisted of red Queenston shale and was encountered at about elevation 196' to 205'. (ie. at about 45' to 50' depth).

It was proven in 8 borehole by diamond drilling to obtain BXL-size core and was indicated in 5 other holes from split-spoon samples into the upper weathered zone. The deposit was extensively weathered for about 2 feet. Beneath the weathered zone the rock was sound although recovery was low because of the tendency for shale to grind and break up during drilling.

4.7) Water Table:

The water table was observed in the boreholes during the drilling program and varied from about elevation 251' to creek level of about elevation 246.5'.

5. EXISTING STRUCTURES:

The existing structures are twin 2-lane, 7-span reinforced concrete bridges which were constructed about 1938-1939. The spans are 20'-20'-36.5'-40'-36.5'-20'-20'.

From bridge drawing D 2539-1, July 8, 1938, it is evident that the bridges were designed as 3-span (36.5-40-36.5) structures supported on timber piles presumably driven to refusal through the organic clay-silt material. Proposed forward slopes were $1\frac{1}{2}$ to 1.

This design was modified by drawing D 2539-7, December 30, 1938 to include footing struts across the 40' span at elevation 244' which is below the creek level. A further revision was given in Drawing D2539-8, January 15, 1939, in which the additional 20' spans were added supported on steel 'H' piles presumably driven to bedrock. The forward slopes were changed to 4 to 1. No details are available of the reasons for these modifications.

Observation of the existing bridges indicate that considerable settlement of the embankments has occurred as well as some settlement of the structures. It would appear that the central spans have settled more than the spans supported on the steel piles. The existing forward slopes are now about 5 to 1. It is not evident whether or not the footing struts are in place.

Similar observations may be made of the structures existing at 15 Mile Creek with similar subsoil conditions. A new 3 span structure north of the present Q.E.W. at 15 Mile Creek and intended

ultimately for the north service road also required struts and light-weight fill near the abutments as remedial measures to control a small inward movement of the pier footings presumably caused by settlement of the approach fill.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct a 2-lane, 3-span bridge with about 17 feet approach embankments, both north and south of the existing Q.E.W. to carry the proposed north and south service roads over 16 Mile Creek. Subsoil at the site is generally a deep deposit of organic clay-organic silt underlain by a clayey silt to silt with sand and gravel deposit and then bedrock. A portion of the site is covered by a fill of about a 3 to 5 foot thickness.

The subsoil is not suitable for the use of spread footing-type foundations and some type of piled foundation will be required for the structure. Any embankment established on this type of subsoil can be expected to settle considerably.

The following three proposals will be discussed:

- 1) A 3-span bridge (45'-45'-45') with approach embankments.
- 2) A multispans structure (approximately 550' long)
- 3) A flexible pipe-arch culvert and associated embankments.

6.2) 3-Span Structure:

The bridge piers and abutments should be supported on a pile-type foundation to the bedrock at approximately elevation 196' to 205'. Either steel 'H' piles (eg. 14 BF 74 may be designed for 90 tons) or lined concrete caissons (eg. 36" ϕ may be designed for 150 tons) may be used.

It is further recommended that some precautions be taken to resist any horizontal forces placed on the pile groups by the effects of the embankment settlement. Three alternative solutions are proposed:

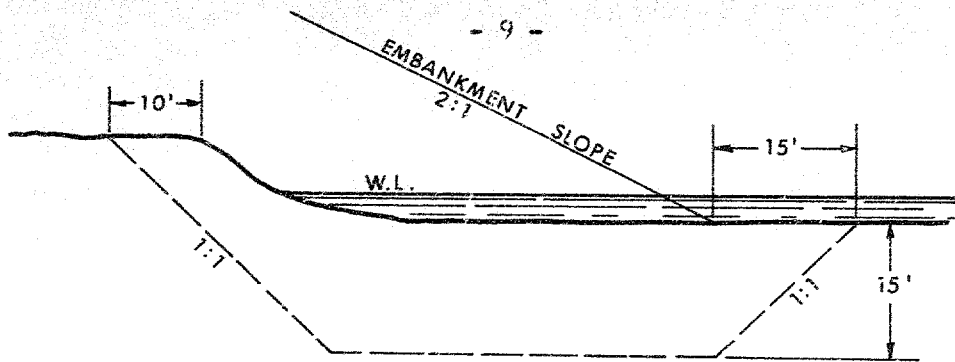
- 1) Struts may be placed between the pile caps.
- 2) If concrete caissons are used for the foundations they could be designed as moment carrying members and be incorporated in the bridge design.
- 3) Prior to the construction of the approach embankments all material lying in the area between the bridge abutments with an additional 5 foot width around this area should be sub-excavated to about elevation 238.0 and replaced with suitable granular material. Side slopes of the excavation should not be steeper than 1 to 1.

Since the pile caps for the piers (and the struts between the pile caps if this method is adopted) will be constructed below the ground water level, a dewatering scheme will be necessary. A suitable scheme would be to drive temporary sheeting around the excavations to a depth of at least 5 feet greater than the base of the excavation.

Subsoil conditions are somewhat different north and south of of the existing Q.E.W. therefore, the approach embankments must be discussed separately.

For the north service road the approach embankments may be constructed with standard 2 to 1 side slopes. Where the toe of the fill extends into the pond area, the upper 6 to 8 feet of soft organic material should be sub-excavated and replaced with a granular fill.

For the south service road the approach embankments may be constructed with standard 2 to 1 side slopes. Where the toe of the fill extends into the pond area, the material beneath the toe should be subexcavated to a depth of 15 feet and replaced with suitable granular material as indicated below.



The sub-excavations for both embankments will improve the slope stability, but will not affect the settlements. The embankments should be constructed as much prior to the structure construction as possible; however, even so, large settlements may be expected to occur.

6.3) A Multispan Structure:

Multispan structures could be utilized to span the entire creek valley. This would require structures some 550 feet long which could be founded on either steel 'H' piles or lined concrete caissons driven to bedrock.

There would be no settlement or stability problems. Consideration should be given to future developments of the Q.E.W. Any additional fill placed for the widening of the main lanes could cause lateral movement of the multispan structure.

6.4) Flexible Pipe-Arch Culvert:

Provided that hydrological and navigational requirements could be met, a flexible pipe-arch culvert could be used in lieu of the proposed 3-span structure. The culvert could be installed on a 3 foot granular pad with adequate camber to compensate for the anticipated settlement.

Embankment construction details are the same as for the 3-span structure outlined in section 6.2).

7. SUMMARY:

The results of a foundation investigation for the proposed north and south service road crossings of 16 Mile Creek by the Q.E.W. are presented.

The subsoil at the site consists of a deep deposit of organic clay-organic silt underlain by a deposit of clayey silt to silt with some sand and gravel and then shale bedrock. Part of the site is covered with a fill some 3 to 5 feet thick. Large settlements are anticipated for embankments on this type of subsoil; hence, performance of the existing structures is reviewed.

The proposed 3-span structure together with alternative proposals of either a multispan structure some 550 feet long or a flexible pipe-arch culvert are discussed.

Either of the structures could be supported on a pile-type foundation using either steel 'H' piles or lined concrete caissons supported to bedrock. The pipe-arch culvert could be installed on a 3-foot thick granular pad.

The construction of the approach fills has been discussed with details of sub-excavations as required for slope stability. Large settlements should be anticipated and as much time as possible should be left between the construction of the approach embankments and the completion of the final grade. All fill used beneath the prevailing water level should be granular.

Construction of pile caps or struts below the ground water table will require a dewatering scheme. Temporary sheeting driven to a depth of 5 feet below the anticipated base of the excavation should be adequate.

Consideration should be given to the future development of the Q.E.W. as it could influence the type of structures chosen. In addition navigational and hydrological requirements must be considered.

8. MISCELLANEOUS:

The field investigation was carried out during the period June 17 to August 26, 1966. The equipment was owned and operated by Canadian Longyear Limited under the supervision of Mr. L. Palmer, Project Foundation Engineer, who subsequently prepared this report.

The entire project was under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who also reviewed this report.

November, 1966.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-62

LOCATION C.E.W. & 16 Mile Ck., N. Service Rd. Sta. 184+44 17'lt.

ORIGINATED BY LP

W.P. 214-63

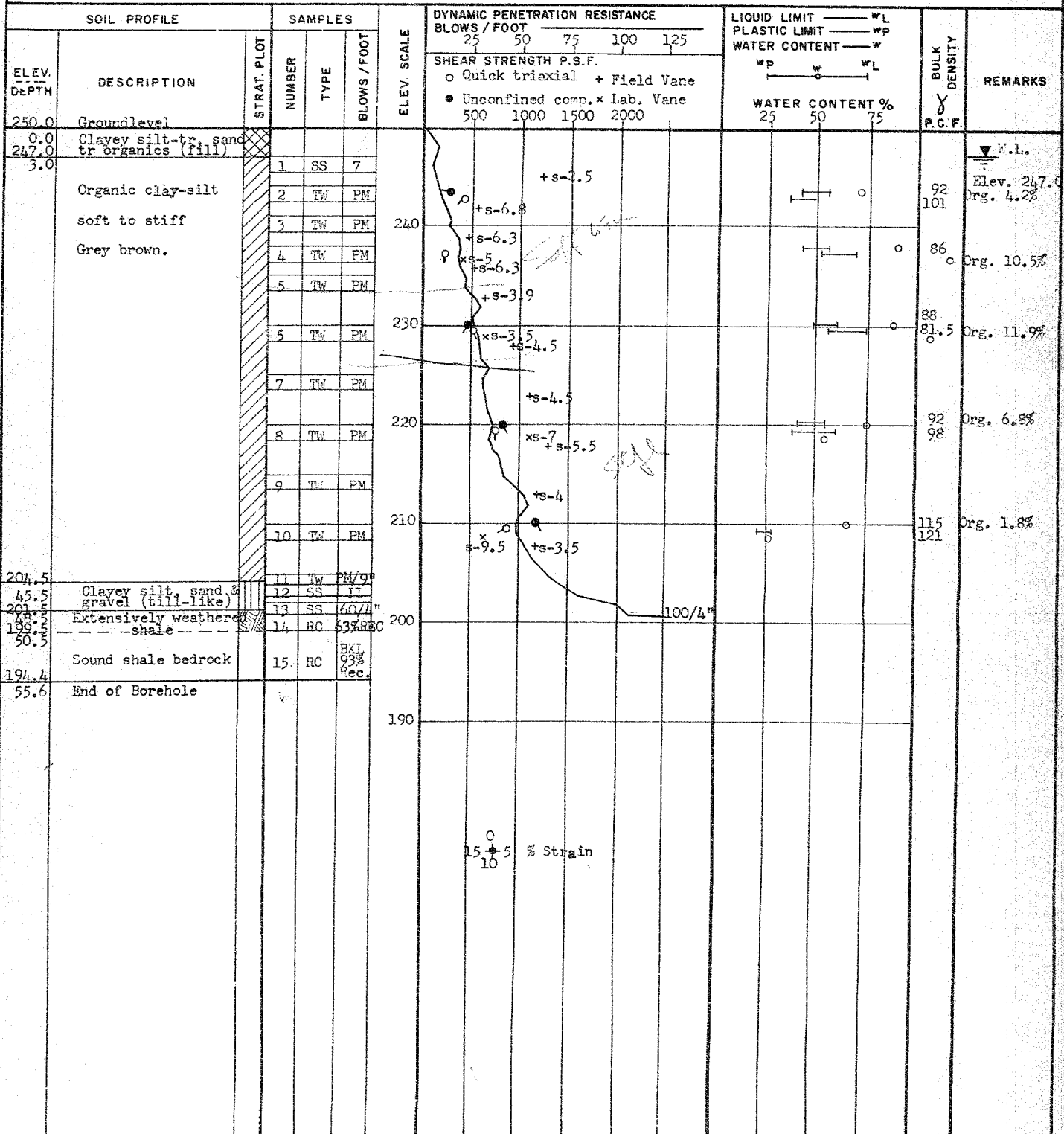
BORING DATE June 17, 20/66

COMPILED BY WTE,LP

DATUM Geodetic

BOREHOLE TYPE Washboring, Nx Casing: Cone

CHECKED BY



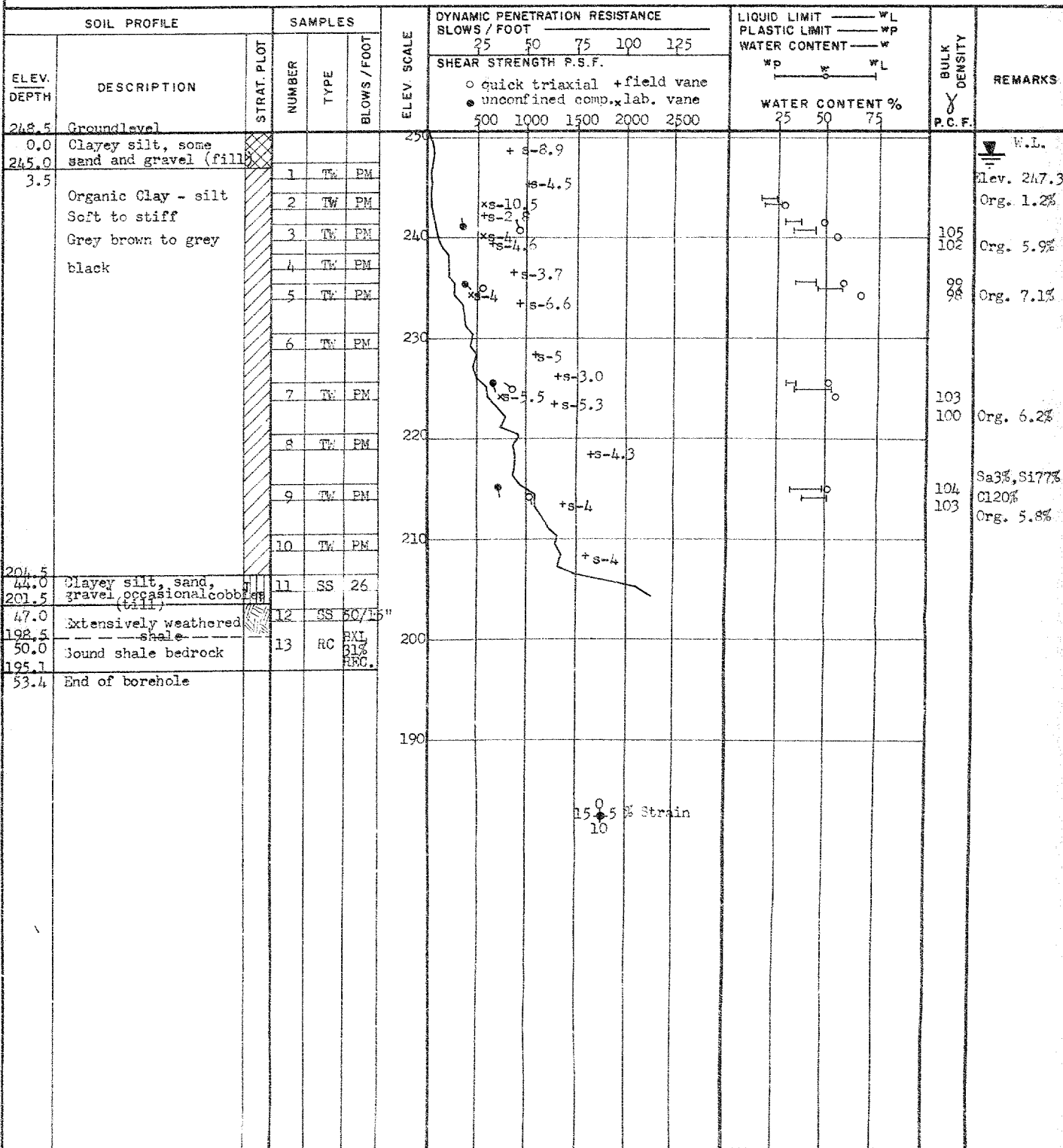
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-62 LOCATION E.E.W. & 16 Mile Ck., N. Service Rd. Sta. 184+85 16' Rt. ORIGINATED BY LP
W.P. 214-63 BORING DATE June 21/66 COMPILED BY WTE, LP
DATUM Geodetic BOREHOLE TYPE Washboring, Nx Casing: Cone CHECKED BY _____

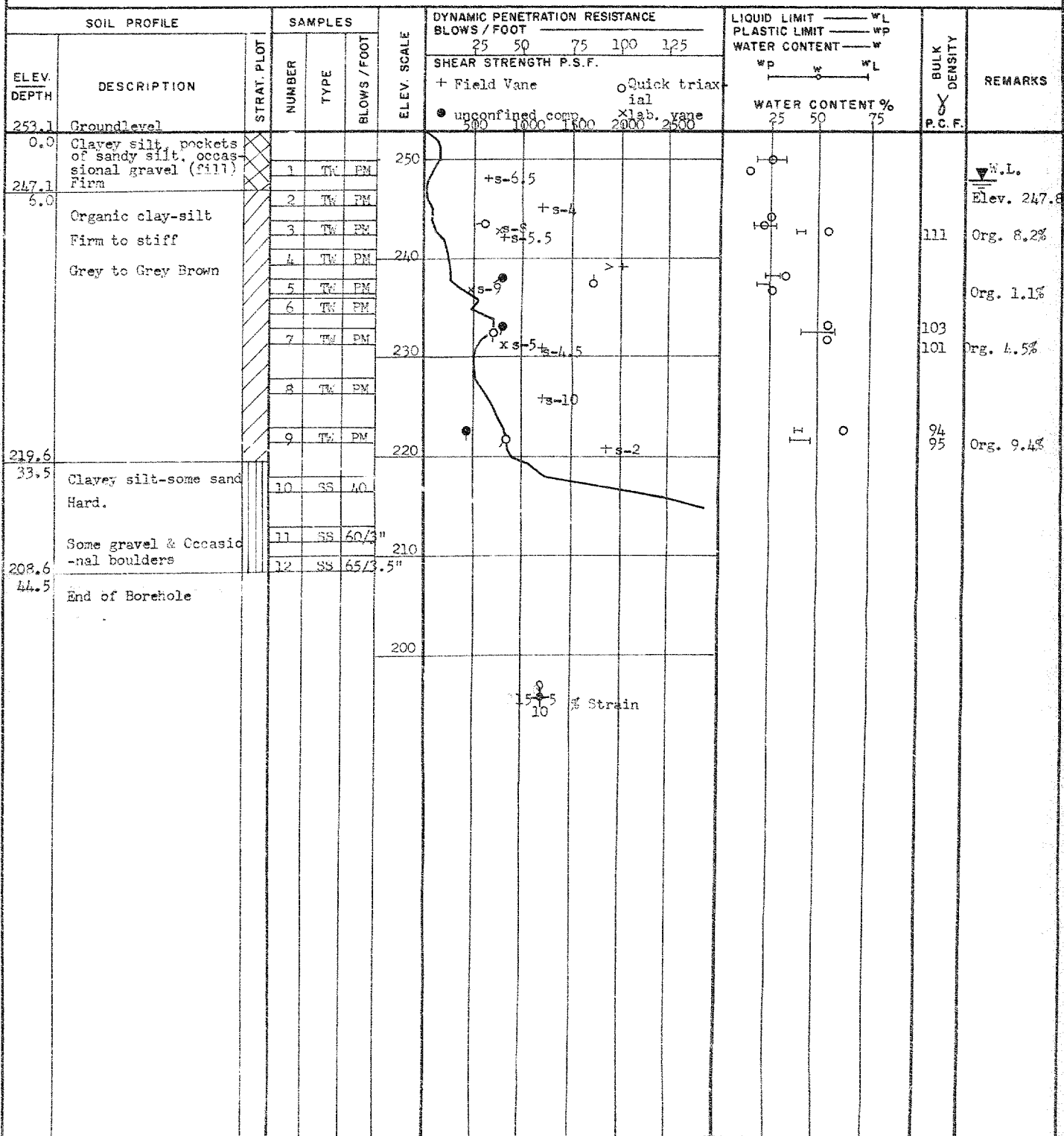


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-62 LOCATION S.E.W. & 16 Mile Crk., Sta. 183 + 00 on C. N. Service Rd. ORIGINATED BY LP
W.P. 214-63 BORING DATE June 22, 23, 1966 COMPILED BY WTE LP
DATUM Geodetic BOREHOLE TYPE Washboring, Mx Casing; Cone CHECKED BY _____

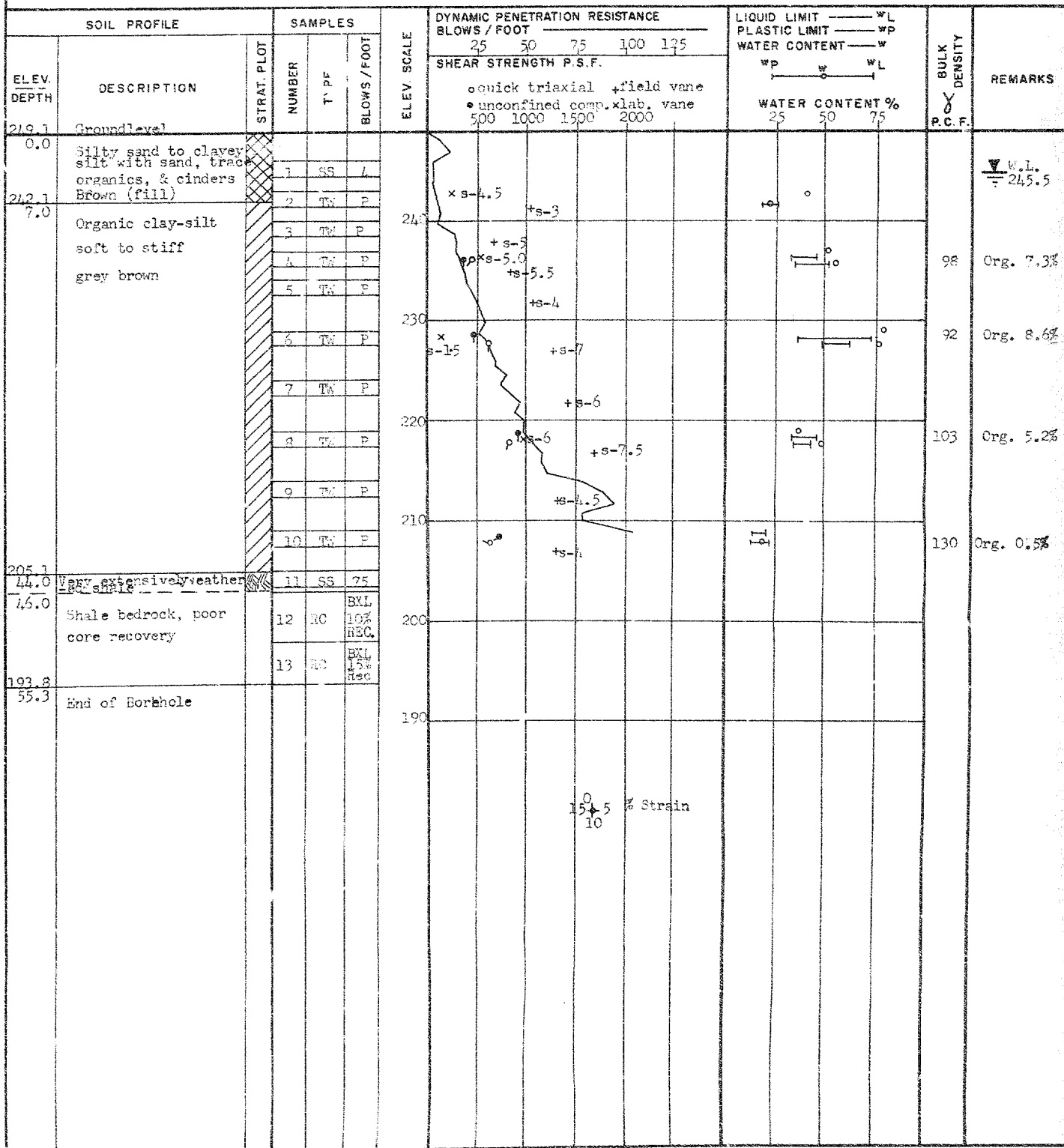


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 66-P-62 LOCATION E.W. & 16 Mile CR., 184/80, 0/S 11.5' L., South Service Rd. ORIGINATED BY LP
W.P. 214-63 BORING DATE June 28, 29/66 COMPILED BY WTE., LP
DATUM Geodetic BOREHOLE TYPE Washboring, Nx Casing: Cone CHECKED BY _____

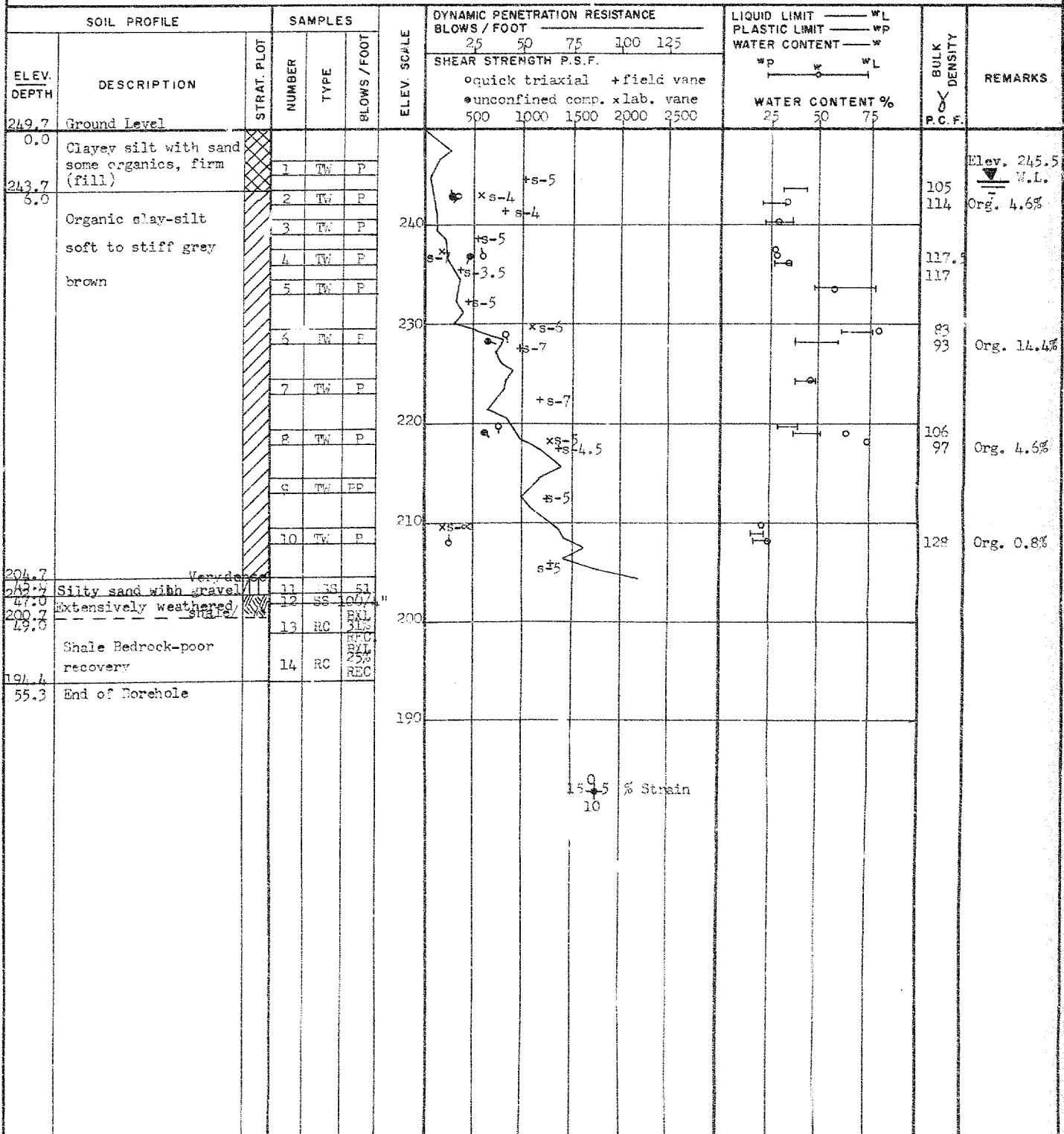


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 66-F-62 LOCATION S.E.W. 316 Mile Cr., S. Service Rd., 184/35, C/S 27' Rt. ORIGINATED BY L.P.
W.P. 314-63 BORING DATE June 29, 30/66 & July 4/66 COMPILED BY Wte LP
DATUM Geodetic BOREHOLE TYPE Washboring, Nx-Bx; Cone CHECKED BY _____



FOUNDATION SECTION

ORIGINATED BY LP

COMPILED BY AMS? LP.

CHECKED BY _____

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

JOB 56-F-62

LOCATION G.E.W. & 16 MI. CK. 183/49.5.0/s 1.5' Rt. S. Service Rd.

ORIGINATED BY LP

W.P. 214-63

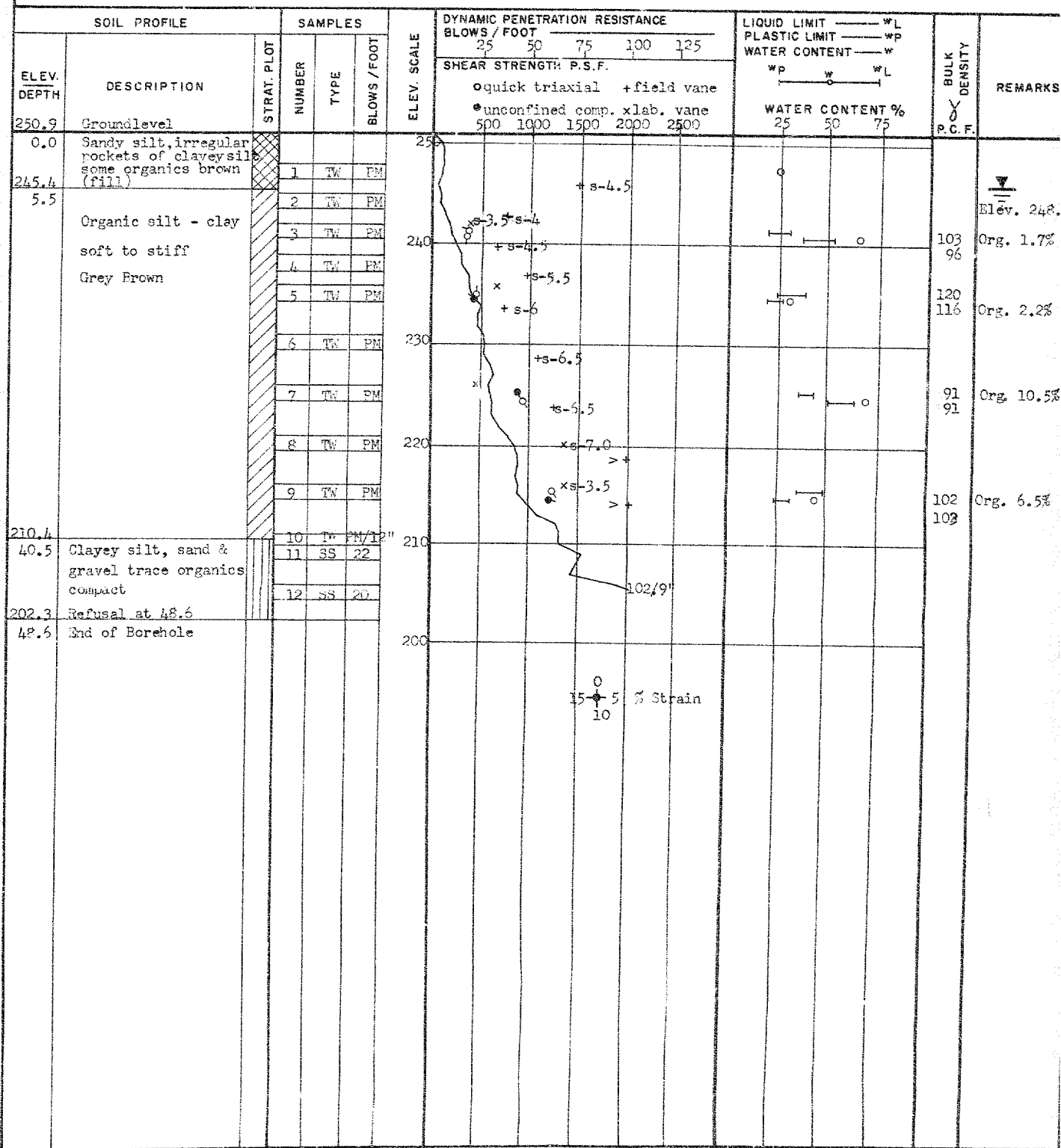
BORING DATE July 4 & 5/66

COMPILED BY AMS LP

DATUM Geodetic

BOREHOLE TYPE Nx-Bx Casing, Cone

CHECKED BY



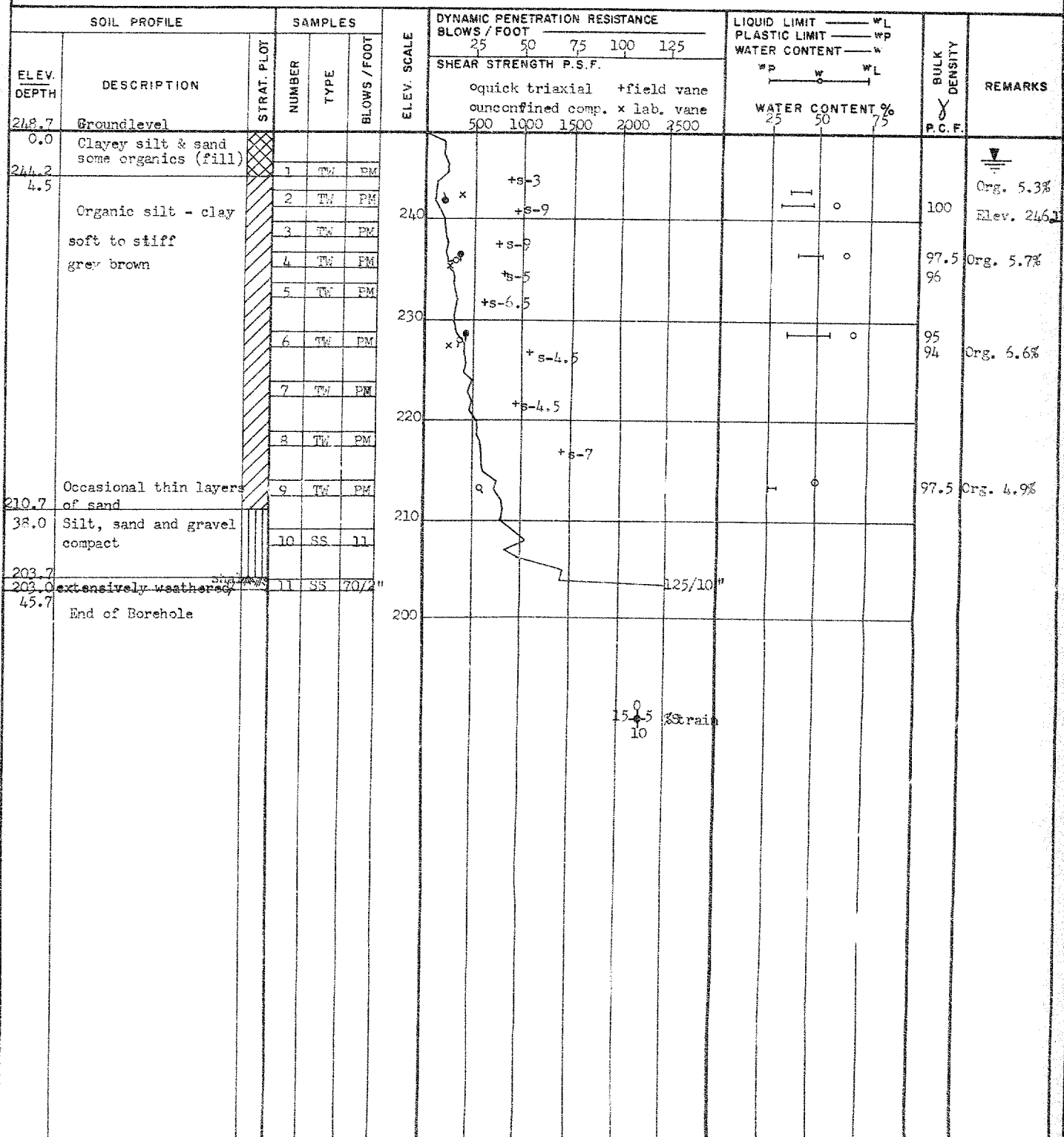
DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 66-F-62 LOCATION C.E.W. & 16 Mi. Cr. South Service Rd. 183/52, c/s 55.51 Mt. ORIGINATED BY LP
W.P. 214-63 BORING DATE July 6, 1966 COMPILED BY AMS LP
DATUM Geodetic BOREHOLE TYPE Nr Casing & Cone CHECKED BY _____

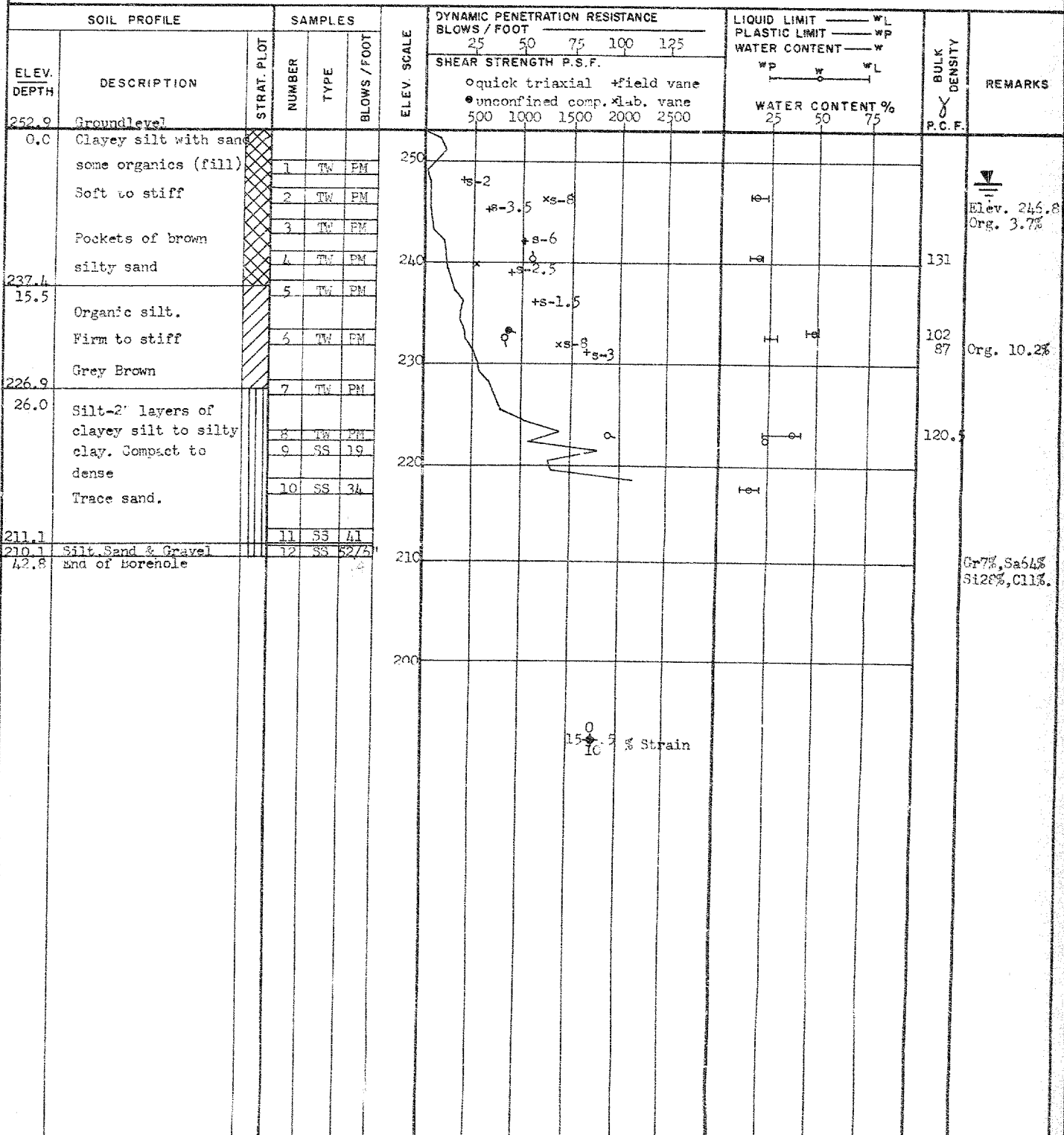


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

JOB 66-E-62 LOCATION S.W. & 16 Mile Ck., South Service Rd. 182°75.0/s1'11" ORIGINATED BY LP
W.P. 214-53 BORING DATE July 7, 1966 COMPILED BY AMS LP
DATUM Geodetic BOREHOLE TYPE Nx Casing, Washbore & Cone CHECKED BY _____

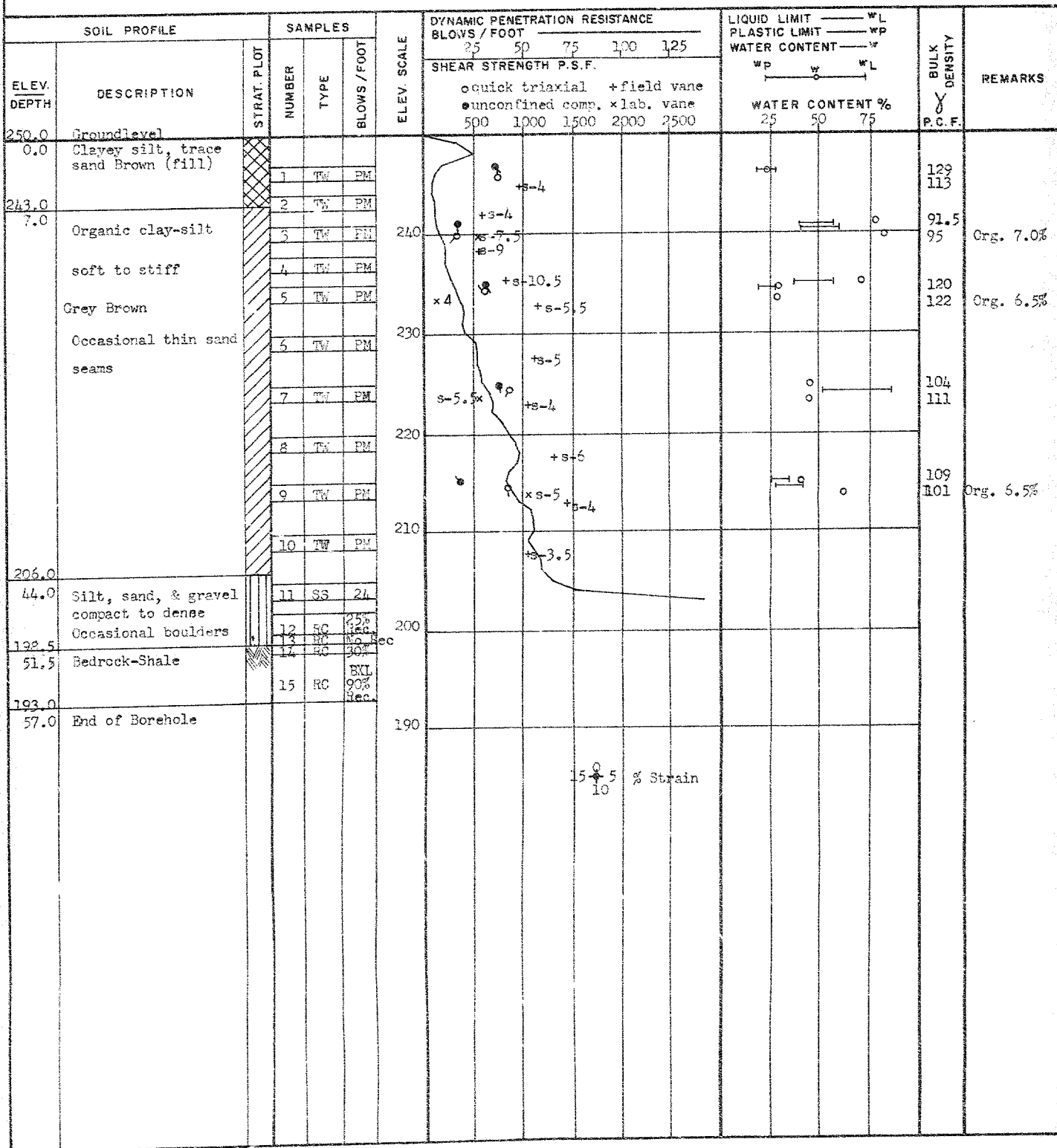


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION
JOB 66-F-62
W.P. 214-63
DATUM Geodetic

RECORD OF BOREHOLE NO. 12

FOUNDATION SECTION

LOCATION C.E.W. & 16 MI. Ck. South Service Rd. 185/34.0/s 12' Rd. ORIGINATED BY LP
BORING DATE July 8, 1966 COMPILED BY AMS LP
BOREHOLE TYPE Nx-Pv Casing, Cone & BVL Core CHECKED BY _____



FOUNDATION SECTION

DATUM Geodetic BOREHOLE TYPE Washboring, Mx-Ex Casing, Cone 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. PLOT	NUMBER	TYPE	BLOWS / FOOT	SHEAR STRENGTH P.S.F.					WATER CONTENT %						
						Quick triaxial + field vane ● unconfined comp. *lab. vane											
						500 1000 1500 2000 2500					25 50 75						
252.6	Groundlevel																
0.0	Clayey silt, some sand & gravel. Stiff to firm (fill). Occasional pockets of organics and organic silt		1	TW	P												
			2	TW	P												
			3	TW	P												
241.6			4	TW	P												
11.0	Organic clay - silt soft to stiff		5	TW	P												
	Pieces of wood		6	TW	P												
	Occasional sand seams and pockets		7	TW	P												
			8	TW	P												
			9	TW	P												
			10	TW	P												
			11	TW	P												
205.1			12	SS	50												
47.5	Silt, some sand & gravel, very dense Occasional boulders		13	SS	60												
			14	SS	60/71												
195.8			15	RC	233												
56.8	Bedrock - Shale		16	RC	EXL												
192.1																	
60.5	End of Borehole																

FOUNDATION SECTION

LOCATION E.W. & 16 Mi. S. South Service Rd. 185479 o/s 15.5186 ORIGINATED BY LP

BORING DATE July 11 & 12, 1966

COMPILED BY AMS LP

BOREHOLE TYPE Nx Casing, BKL Core & Cone

CHECKED BY _____

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 15

FOUNDATION SECTION

JOB 66-F-52 LOCATION G.B.W. & 16 W. Ck. N. Service Rd. Sta. 186/93, o/s 214 ORIGINATED BY LP
W.P. 214-63 BORING DATE July 13, 1966 COMPILED BY WTE LP
DATUM Geodetic BOREHOLE TYPE Washboring, Nx Casings Cone CHECKED BY _____

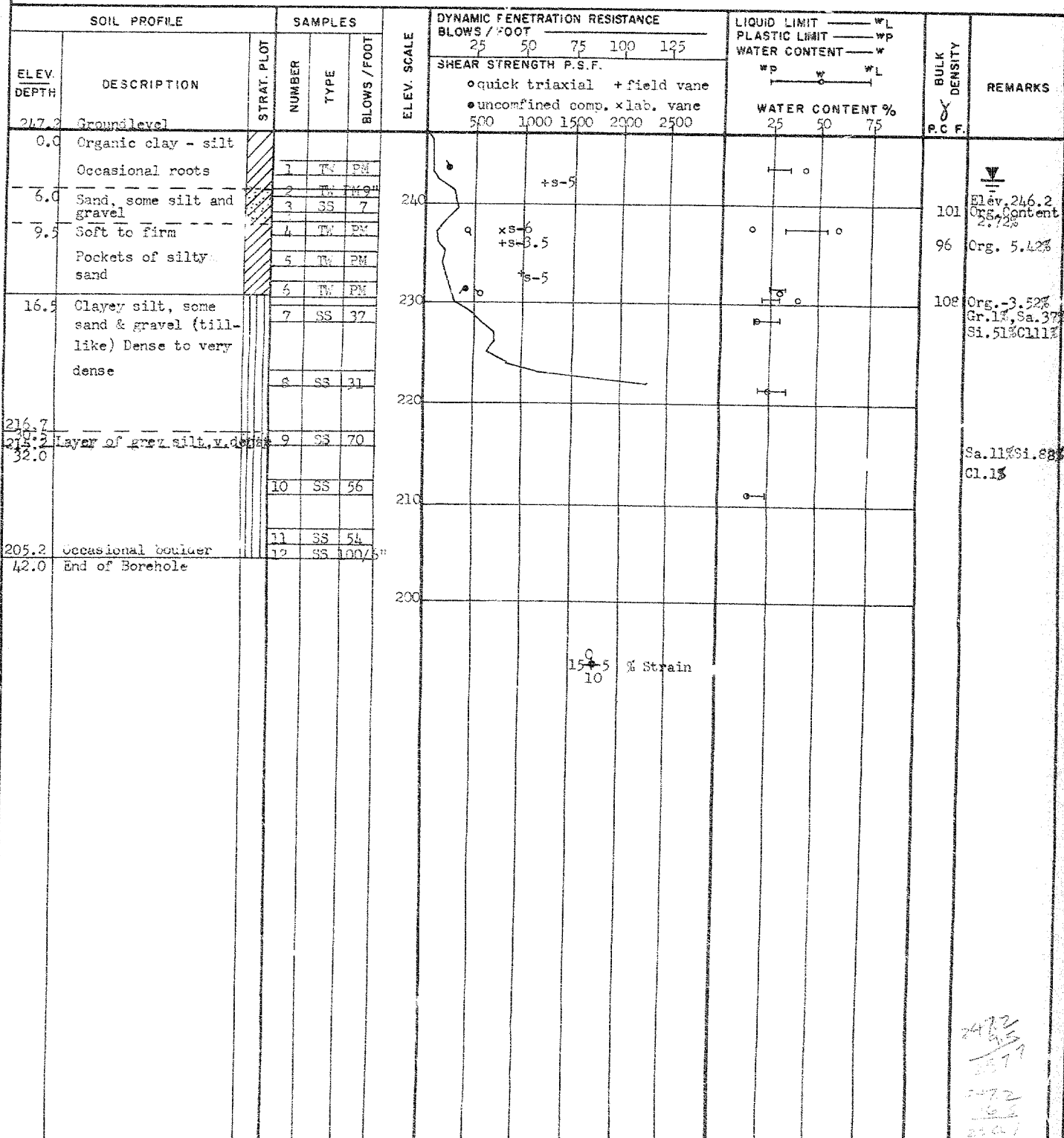
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DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 17

FOUNDATION SECTION

JOB 66-F-62 LOCATION C.B.W. & 16 MI. Ck. S. Service Rd., 186/62, o/s 26.5' Rt. ORIGINATED BY L.P.
W.P. 214-63 BORING DATE July 13. & 14, 1966 COMPILED BY ANS LP
DATUM Geodetic BOREHOLE TYPE MX Casing & Washboring CHECKED BY _____



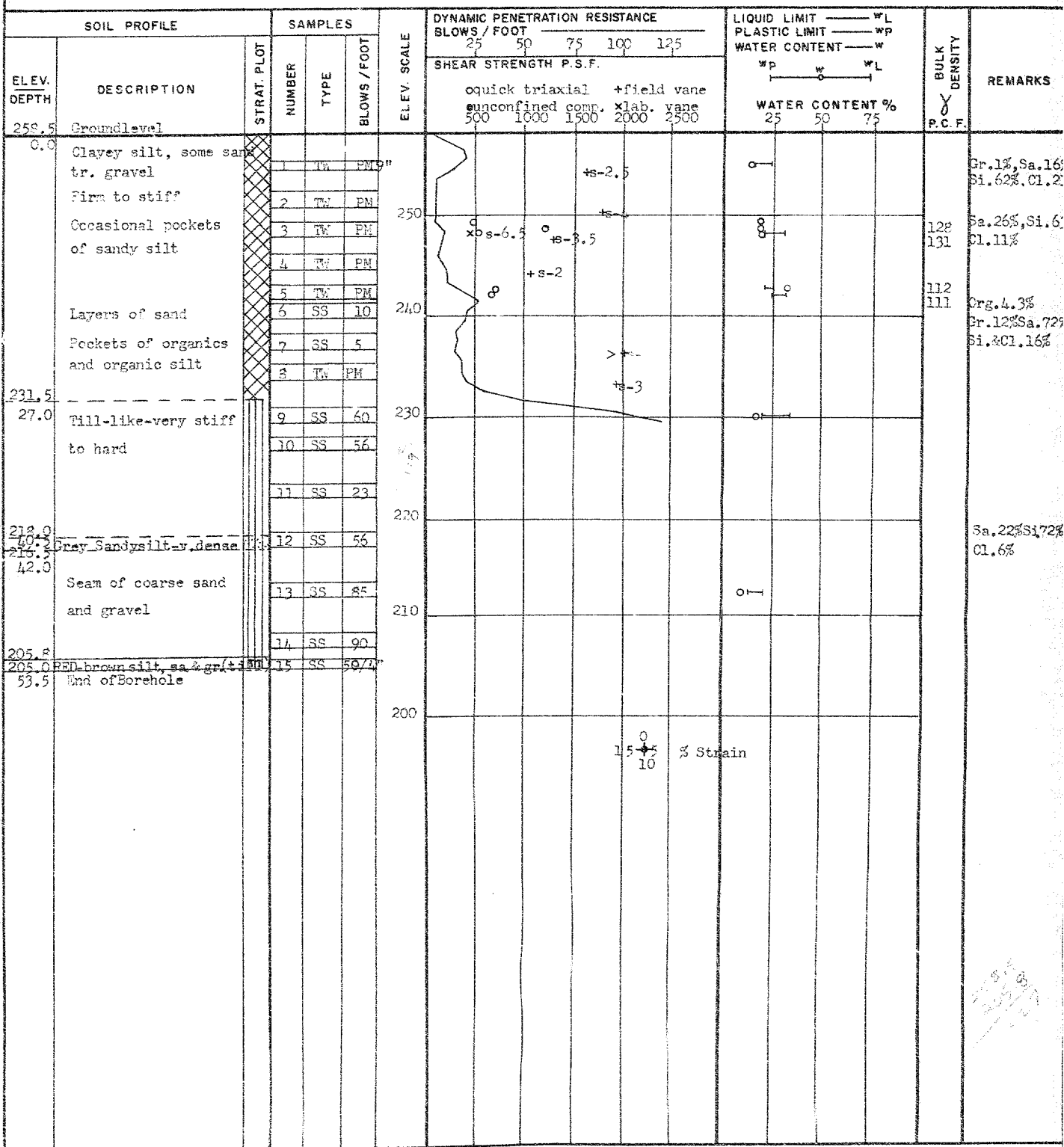
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 18

FOUNDATION SECTION

JOB 65-F-62 LOCATION E.E.N. & 16 MI. Ch. S. Service Rd. 186/61, o/s 16' Lt. ORIGINATED BY LP
W.P. 214-63 BORING DATE July 14 & 15, 1966 COMPILED BY AMS LP
DATUM Geodetic BOREHOLE TYPE Nx Casing & Washbore CHECKED BY _____



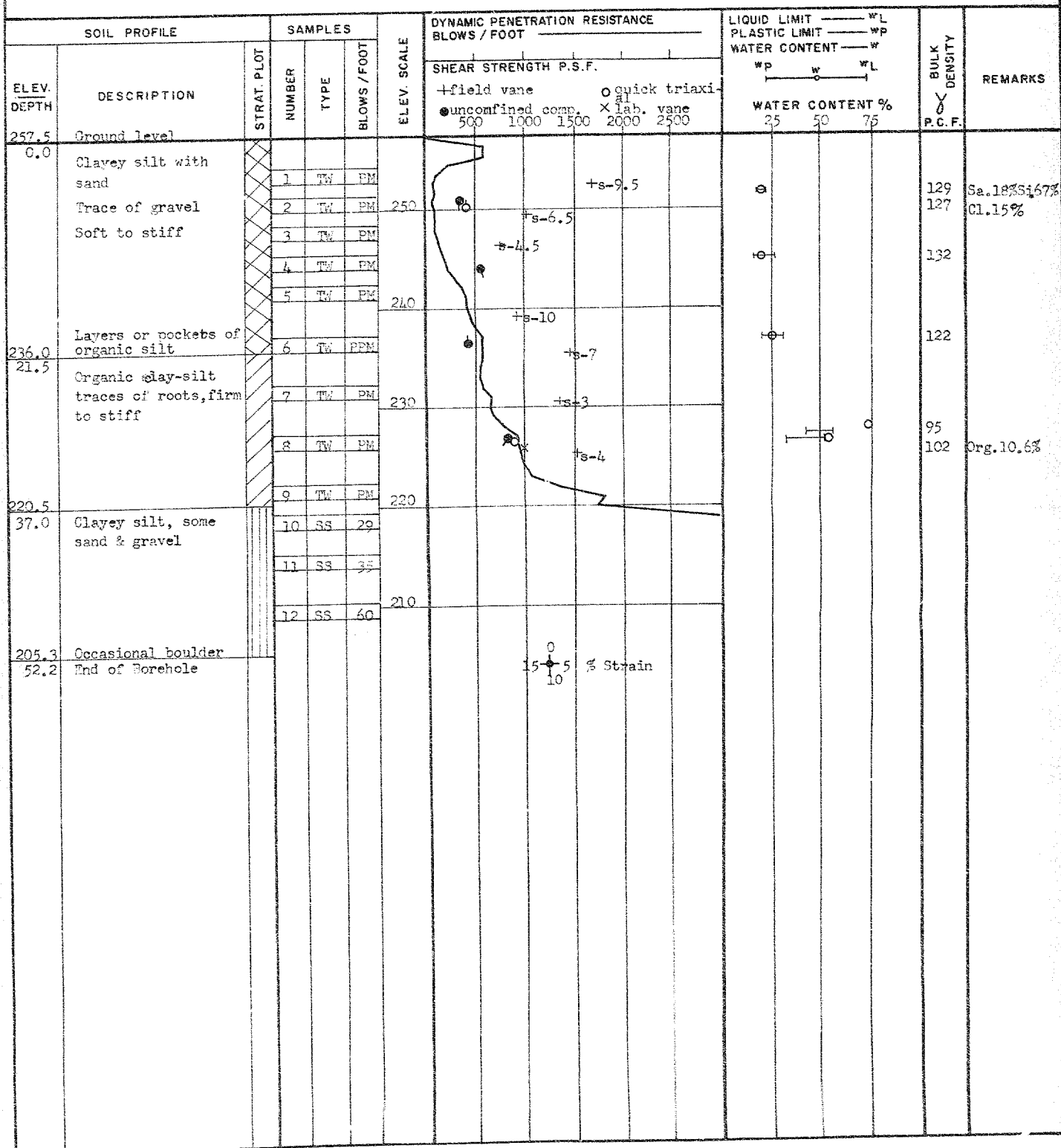
DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 19

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 66-P-62 LOCATION C.E.W. & 16 Mi. Ck. N. Service Rd. 184/38.5 o/s 22' 5ft. ORIGINATED BY LP
W.P. 214-63 BORING DATE July 14, 1966 COMPILED BY DK LP
DATUM Geodetic BOREHOLE TYPE Washboring, Nx Casing: Core CHECKED BY _____



FOUNDATION SECTION

JOB 66-F-62 LOCATION C.E.W. & 16 Mi. Ck. N. Service Rd. Sta. 183/00, o/s 201 ORIGINATED BY LP
W. P. 214-63 BORING DATE August 17, 18, 1966 COMPILED BY WTE LP
DATUM Geodetic BOREHOLE TYPE Washboring, Nx Casing; Cone 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. PLT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	25	50	75	100	125	WP	WL		
						SHEAR STRENGTH P.S.F. + field vane Quick triax- 									

FOUNDATION SECTION

JOB 66-P-62 LOCATION C.E.W. & 16 Mile Crk. 182+25 70 LT. ORIGINATED BY LP
W. P. 214-53 BORING DATE August 17, 1966 COMPILED BY DK LP
DATUM Geodetic BOREHOLE TYPE Washboring Nx Basing Cone CHECKED BY _____

[illegible]

FOUNDATION SECTION

JOB 55-F-52 LOCATION A.E.W. & 16 Mi. Ck. N. Service Rd. 186+00, s/s75' Lt. ORIGINATED BY LP
 W.P. 214-53 BORING DATE August 22, 1966 COMPILED BY DK LP
 DATUM Geodetic BOREHOLE TYPE Washboring; Nx Casing; Cone 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. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	WATER CONTENT %					
216.5	Water Level						+ field vane O quick triaxial	25 50 75 100 125					
215.0	Ground Level						● unconfined comp. X lab. vane	500 1000 1500 2000 2500					
1.5	Organic clay-silt soft to firm		1	TW	PM	240	+s-8	25 50 75	34	Org. 9.1%			
238.5			2	TW	PM								
8.0	Silty sand to sandy silt, very loose to compact		3	SS	12								
232.5			4	SS	17								
14.0			5	SS	12								
	Grey Brown to grey		6	SS	2	230	+s-5		90	Sa. 37% Si. 61% Cl. 2% Org. 6.7%			
			7	TW	PM				90				
			8	TW	PM		+s-4						
			9	TW	PM	220	+s-4		96	Org. 5.6%			
			10	TW	PM		+s-5		102				
			11	TW	PM	210	+s-4.5		99	Org. 5.5%			
			12	TW	PM		+s-2		96				
203.5													
43.0	Silt, Sand & Gravel		13	SS	29	200							
196.5	0.2' weathered shale		14	SS	28								
50.0	End of Borehole												
						190							
							15 0 5 10 % Strain						

65-1931

MATERIALS & TESTING DIVISION

FOUNDATION SECTION

JOB 66-F-62 LOCATION L.H. & 16 Mi. E. S. Service Rd. 186-90, o/s 70' Rt. ORIGINATED BY LF
W.P. 214-63 BORING DATE August 25, 1966 COMPILED BY AMS LP
DATUM Geodetic BOREHOLE TYPE MX Casing & Washbore CHECKED BY _____

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT						LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT.	NUMBER	TYPE	BLOWS / FOOT										P.C.F.	
246.5	Water Level															
245.5	Ground level															
1.0	Organic clay - silt Soft to firm Grey Brown		1	TW	PM	240	+s-7								89	Org. 8%
			2	TW	PM		+s-6									
			3	TW	EM		+s-5.5									
			4	TW	PM		+ . 5								87 91	Org. 7.1%
	0.5' Layer Sand to Silty Sand		5	TW	PM	230	+s-6									Sa. 61% Si. 33%
226.0			6	TW	PM		+s-4.5									C.I. 6%
20.5	Occasional sand pockets		7	TW	PM	220	+s-4.5									
			8	TW	PM		+s-6								105 107	Org. 4.5%
			9	TW	PM	210	+s-2.5									
39.8	Silt, sand, gravel (till)		10	TW	PM											
204.7	Redbrown, 0.2' weathered shale		11	SS	53/A"											
41.8	End of Borshole					200										

FOUNDATION SECTION

DATUM Geodetic BOREHOLE TYPE Nx Casing & Washbore CHECKED BY _____

[illegible]

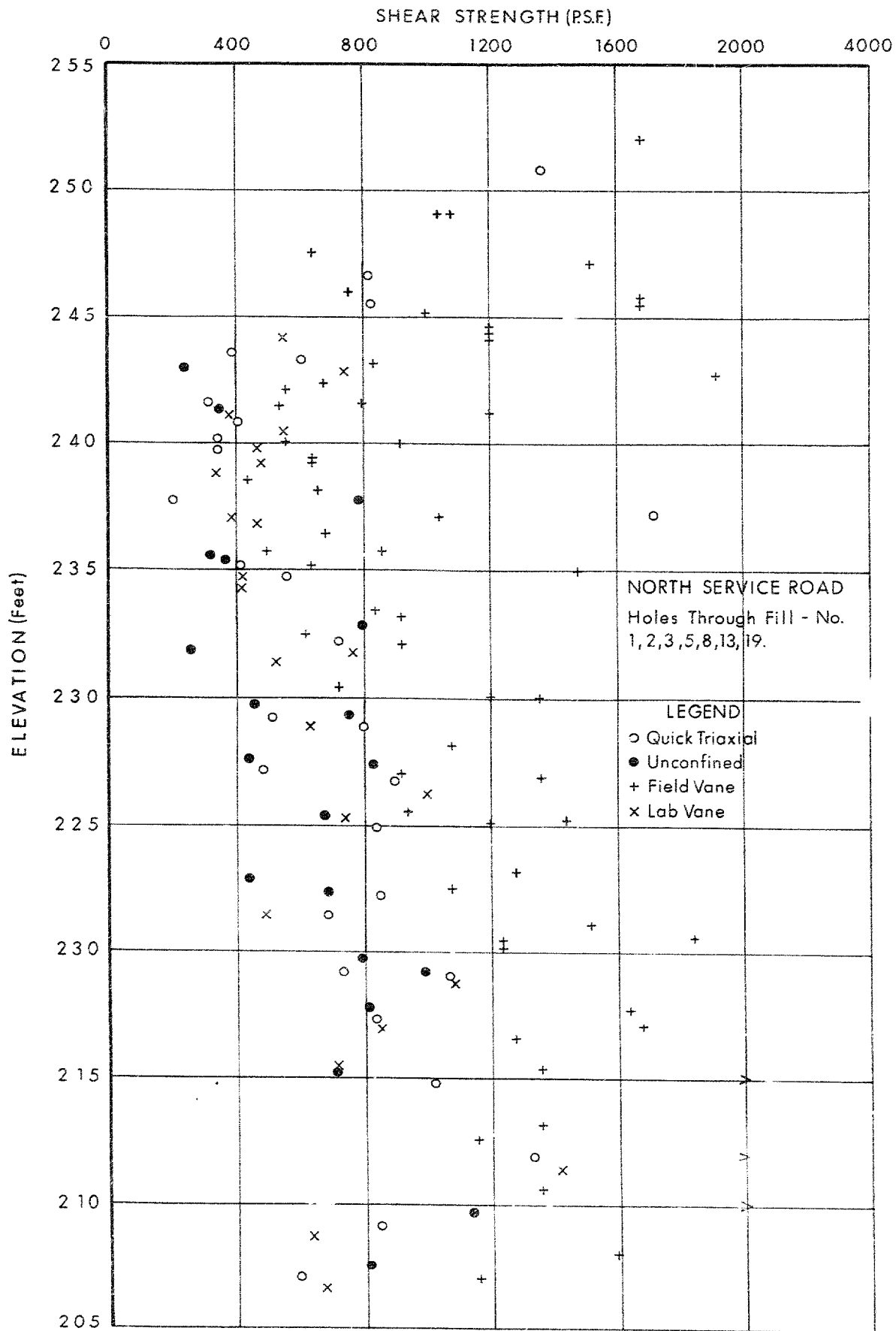
FOUNDATION SECTION

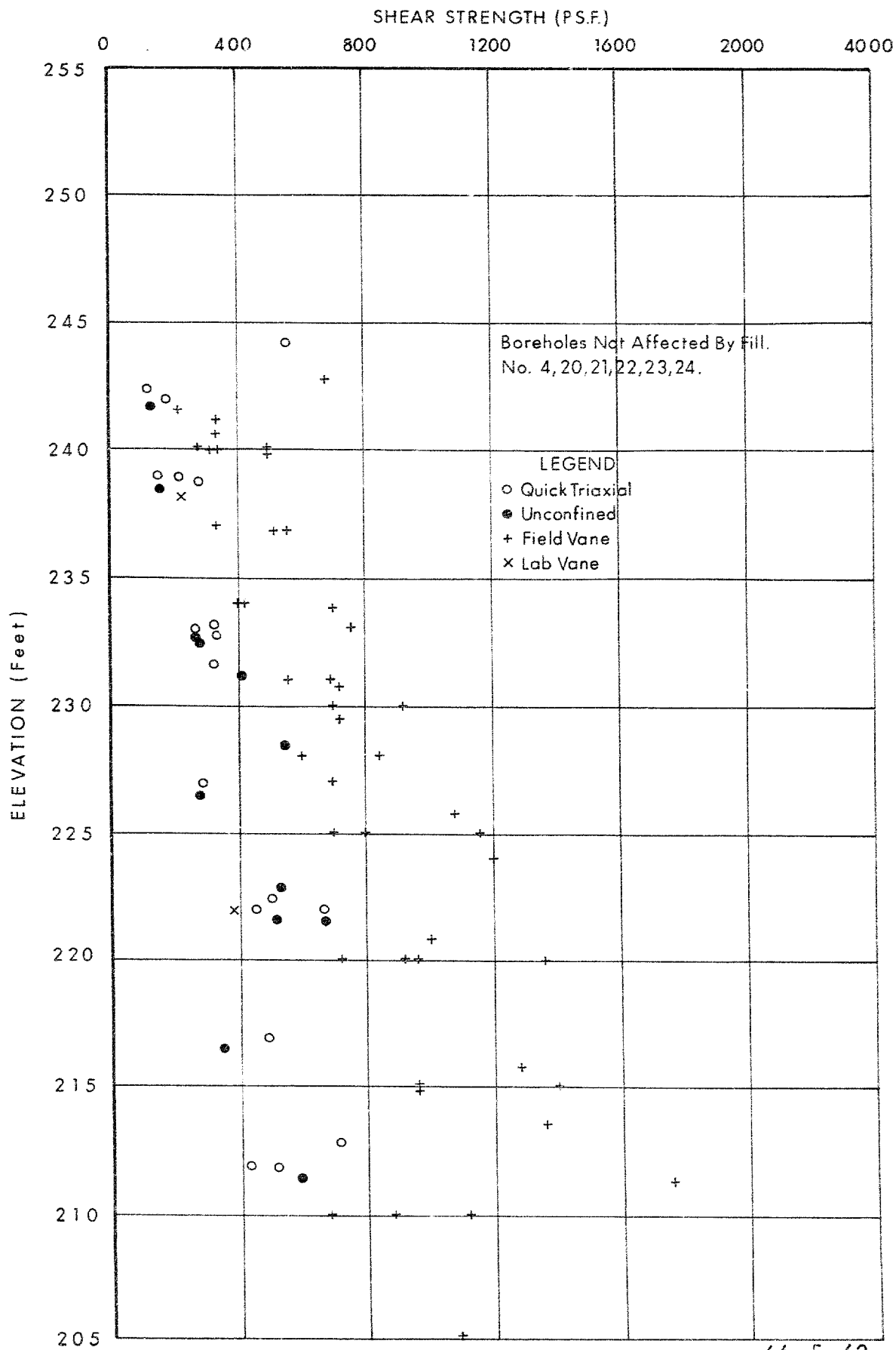
ORIGINATED BY LP

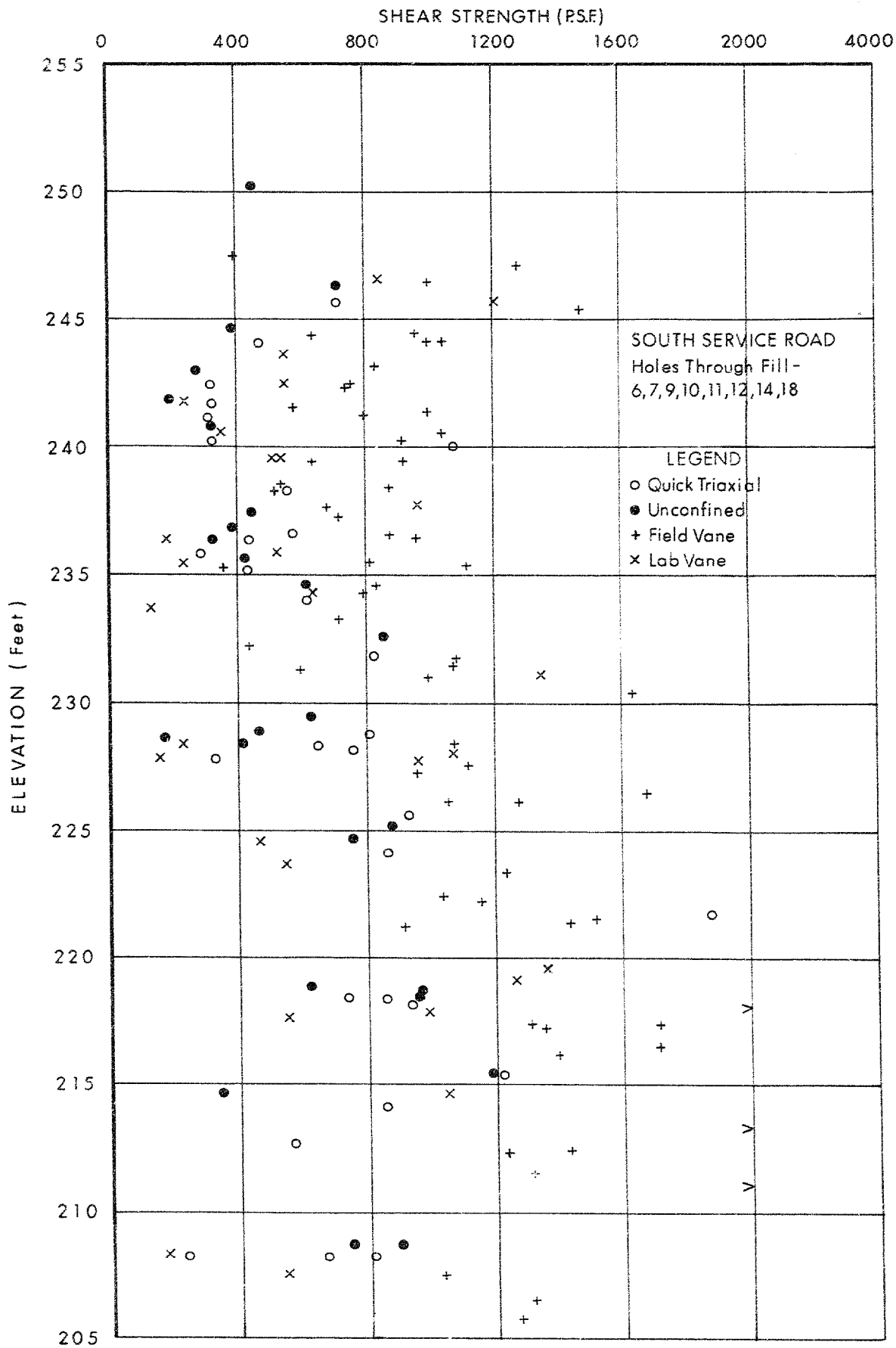
COMPILED BY WTE LP

CHECKED BY

[illegible]







MEMORANDUM

*Proposed North Service
Road Structure.*

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

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

Attn: Mr. C. S. Grebski,
Bridge Design Engr.

DATE: August 18, 1967

OUR FILE REF.

IN REPLY TO

SUBJECT:

Proposed North Service Road Structure
-- Revised Profile Grade --
District No. 4 (Hamilton)
W.J. 66-F-62 -- W.P. 214-63

Since the original foundation report was prepared for the above mentioned project, a grade revision has been proposed which would in effect, increase the structure approaches to a maximum height of 36 ft. in the longitudinal direction. An increase of this magnitude necessitates a complete re-evaluation of the stability of the structure and embankments. As a result of this, we have carried out further stability analyses, and our new recommendations pertaining to structure foundations and approach embankments, are as follows:

Two alternative proposals can be considered, taking into account the revised proposed grade, but the final choice should be based on economical considerations:

- (1) Approximately 500 ft. Long Structure Spanning Most of the Problem Area:

For this scheme, half-height berms 60 ft. long, will be required in the forward direction only, without any subexcavation for the approach embankments. This would require a structure 500 ft. long between Sta. 181+50 and Sta. 186+50.

- (2) Approximately 300-ft. Long Structure Spanning About Half of the Problem Area:

For this scheme, in addition to the subexcavation of the organic material, half-height berms will be required as follows:

cont'd. /2 ...

Mr. B. R. Davis, Bridge Engr.

Attn:

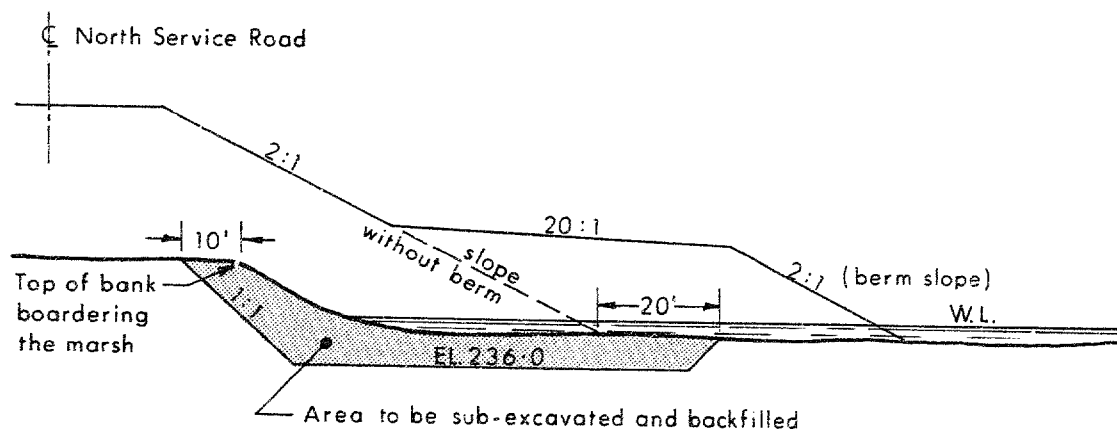
Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division.

- 2 -

August 18, 1967

<u>Location</u>	<u>Berm Length</u>
a) East Approach:	
Forward Slope - Full width of embankment.	60 ft.
North Side Slope - Abutment \bar{Q} bearing.	60 ft.
" " " - Sta. 187+50	Zero
South Side Slope	No berms required
b) West Approach:	
Forward Slope - Full width of embankment.	60 ft.
North Side Slope - Sta. 184+00	70 ft.
" " " - Sta. 183+00	50 ft.
" " " - Sta. 181+00	Zero
South Side Slope	No berms required

Prior to the construction of the approach embankments with berms, subexcavation of the organic soil down to Elev. 230.0 will be necessary over an area 10 ft. wider than the bridge (5 ft. each side) extending from a line 5 ft. west of the west abutment footing to a line 5 ft. east of the east abutment footing. The sides of the excavation should slope at 1:1. Subexcavation will also be required on the north side of the embankment as illustrated below:



cont'd. /3 ...

Mr. B. R. Davis, Bridge Engr.

Attn:

- 3 -

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

August 18, 1967

All fill material used beneath the prevailing water level should consist of a granular type of material. It should be noted that the aforementioned subexcavation procedures and construction of berms will only improve the slope stability, but will not affect the settlements. The embankments should be constructed as much prior to the structure construction as is possible; however, even so, excessive settlements may be expected to occur.

Structure Foundations:

As previously outlined in our Foundation Report (W.J. 66-F-62) for the above mentioned two alternatives, the entire structure should be supported on a pile-type of foundation to the bedrock at approximate Elev. 196.0 to Elev. 205.0. Either steel H-piles (14 BP 73 may be designed for 90 tons), or lined concrete caissons (e.g., 30" \emptyset socketed 4 ft. into shale, may be designed for 300 tons) may be used. Consideration should be given to future developments of the Q.E.W. Any additional fill placed on the organic soils (for the future widening of the main lanes) will have a tendency to squeeze the underlying organic silt and induce lateral thrust on the piles and caps. Therefore, the piers should be designed to resist the induced lateral forces for any future widening of the main lanes of the Q.E.W.

For your information, we are enclosing a copy of the letter we received from Western Caissons Ltd. outlining the cost of 24 in., 30 in., and 36 in. \emptyset concrete caissons. In addition, we have requested Franki of Canada Ltd. for similar information, and it will be submitted to you as soon as we receive it.

If you require further information pertaining to this project, please feel free to contact this Office.

MD/MdeF

Encl.

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

Foundations Files
Gen. Files ✓

M. Devata

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

WESTERN CAISSONS LTD.

W.P. 514-62
Re: Proposed Bridge
Sixteen Mile Creek
**FOUNDATION
SPECIALISTS**

46 Credit Stone Road,
Maple, Ontario.

August 9, 1967.

Department of Highways of Ontario,
Material and Research Division,
Foundation Section,
DOWNSVIEW, Ontario.

Attention: Mr. M. Devata, P. Eng.

Dear Sir:

Re: Proposed Bridge,
Sixteen Mile Creek and
Q.E.W. North Service Road

Further to our meeting of August 3, 1967, we have reviewed the information submitted to us and would like to submit the following for your consideration.

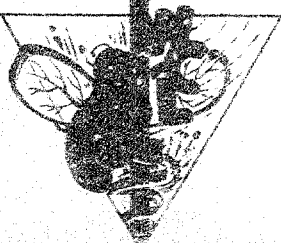
It is understood that it is proposed to construct a bridge at the above location with a total load of approximately 600 tons per bent. It is estimated that eight bents will be required to cross the creek. Based on piles being 50 feet in length the following information was requested on a per pile basis using a design criteria of:-

- a) End Bearing in the shale - 25 tons per sq. ft.
- b) Skin Friction in the shale - 100 lbs. per sq. in.
- c) Length of Socket - 4 feet.
- d) Concrete - 3,000 p.s.i.

(SEE ATTACHED SUMMARY)

Since the loading is 600 tons per bent, even two 24" diameter caissons could be designed to carry 300 tons each by doing the following:-

- 1. Length of socket is increased from 4 feet into shale to 5 feet.
- 2. Increase the strength of concrete.
- 3. Increase the quantity of steel per pile.



Department of Highways of Ontario

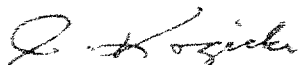
August 9, 1967.

In analysing the above prices one should add \$1.00 per foot for handling and installation of the sonotube and the 16 gauge corrugated pipe. In addition to the above, an allowance of \$500.00 should be made for move-in and move-out.

We trust the above information is sufficient, however, should additional information be required, please feel free to contact us.

Yours very truly,

WESTERN CAISSONS LIMITED



P. Kozicki, P. Eng.

PK/bt

Pile Diameter	Capacity in tons per ft. socket in shale	Pile Capacity (tons)	Cost of Heavy Wall Sonotube/ft.	Cost of 16 gauge corr.pipe/ft.	Cost of 3/8" pipe/ ft.	Cost of caisson/ft. 8 #7 bars #4 ties @12"o.c. (for reinforcing steel)	Cost of caisson/ft. less 8 #7 bars #4 ties @12" o.c.
24"	45	260	\$2.55	\$4.45	\$14.50	\$3.40	\$ 8.50
30"	56	348	\$3.63	\$5.27	\$18.00	\$3.40	\$10.40
36"	68	458	\$4.61	\$6.49	\$21.60	\$3.40	\$12.85

W.P. 214-63

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

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

August 23, 1967

Attn: Mr. C. S. Grebaki,
Bridge Design Engr.

Proposed North Service Road Structure
-- Revised Profile Grade --
District No. 4 (Hamilton)
W.J. 66-F-62 -- W.P. 214-63

As mentioned in our recent memo (August 18th)
on the above subject, attached please find additional
information on the cost of drilled-in caissons, supplied
by Franki of Canada Ltd.

MD/Eden
Attach.

Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farrer
G. E. Hunter (2)
H. Greenland
W. S. Melinyshyn
T. J. Kovich
Foundations Files
Gen. Files

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



FRANKI

CANADIAN INSTITUTE

THORNCLIFFE SQUARE, UNIT NO. 3
P.O. BOX 190 STATION "B"

TELEPHONE
421-7710
CABLES
TRANSFOR

Our Reference:
X. 10067

TORONTO 17, ONT.

August 21, 1967

Department of Highways, Ontario,
Materials and Testing Division,
Foundation Section,
Room 107, Lab Building,
Hwy. 401 & Keele Street,
Downsview, Ontario

Attention: Mr. S. Murty Devata, P.Eng.
Senior Foundation Engineer

Dear Sir:

Re: Bridge, Sixteen Mile Creek

As requested, we have carried out a preliminary study into the cost of drilled-in-caissons for the above project.

For budget purposes, we estimate the cost to supply and install 16 . 24" diameter x 0.50 inch wall steel encased caissons, socketed 3 feet into the shale bedrock to be FORTY-FOUR THOUSAND DOLLARS (\$44,000.00).

Our budget price is based on the following:

(1) There will be 8 piers with 2 caissons per pier, each carrying a design load of 300 tons.

(2) The caissons will extend to the underside of the deck at approximately elevation 278.

(3) The cost of concrete and the steel shells
are included.

Yours very truly,
FRANKI CANADA LIMITED



AWM/eh
Encl

A. W. Millard, P.Eng.
Project Engineer

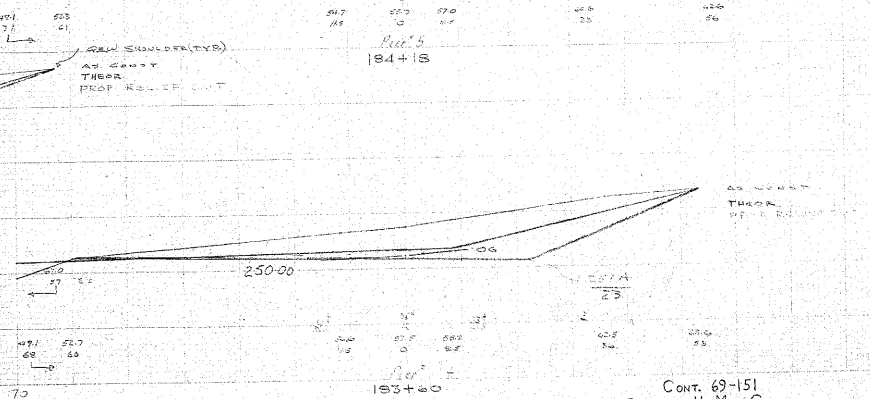
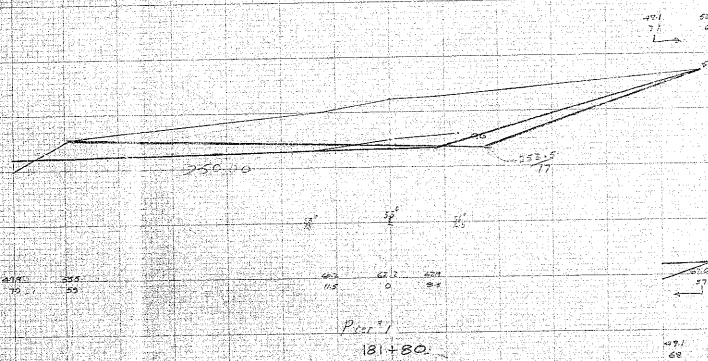
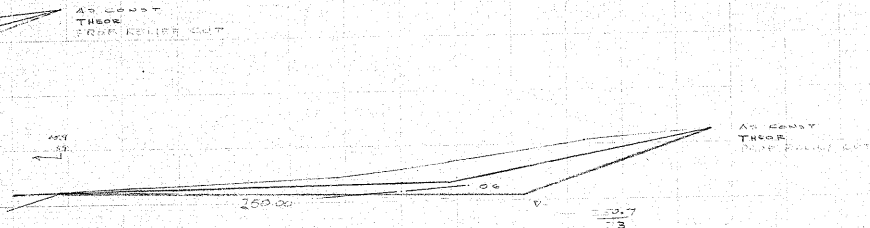
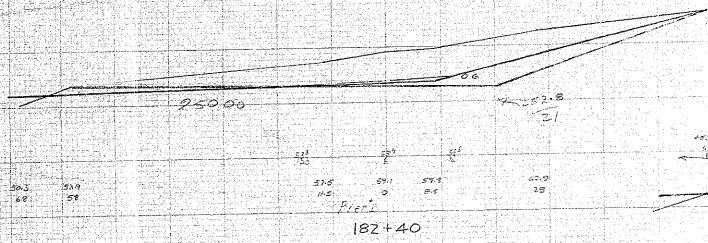
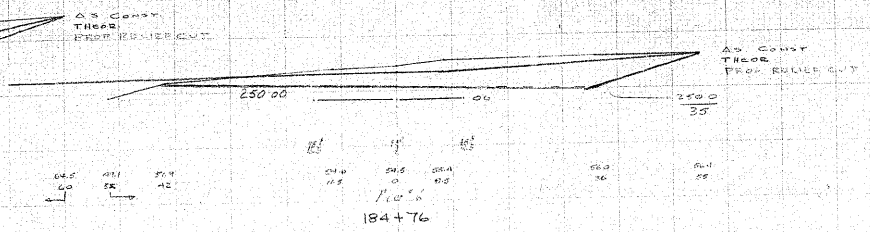
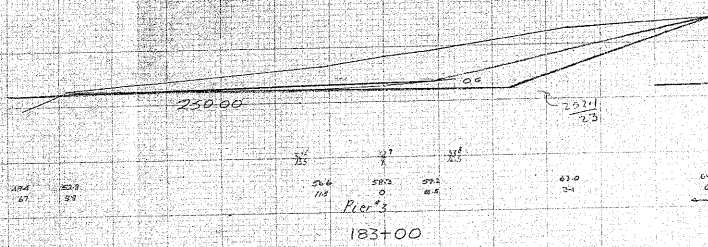
W.P. 214-63-02

Q.E.W.

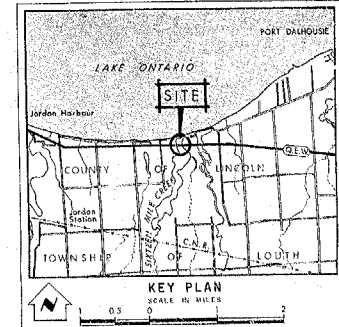
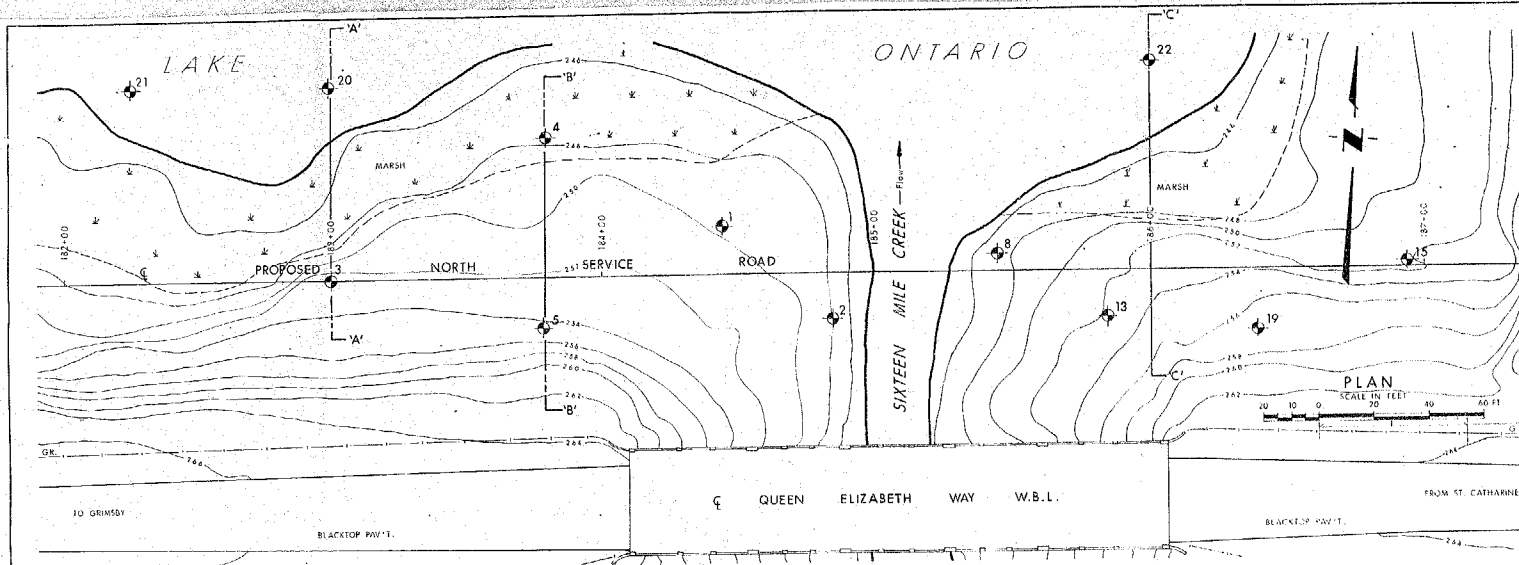
4 18 MILE

CREEK

30M3-12



CONT. 69-151
CROSS SECTIONS 16 Mile Cr.
N.S.R.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore H. Cone Penetration Hole		
	Water Levels established at time of field investigation, JUNE 1966		

NO.	ELEVATION	STATION	OFFSET
1	250.0	184+4.2	12.0' LT
2	248.5	184+4.5	16.0' LT
3	243.1	183+0.0	0
4	238.7	182+7.9	50.0' LT
5	238.0	181+7.6	16.0' RT
8	227.9	185+1.5	6.0' LT
13	252.4	185+8.4	17.0' RT
15	241.2	186+9.3	2.0' LT
19	247.5	186+2.8	22.0' RT
20	246.8	183+0.0	70.0' LT
21	244.0	181+2.5	22.0' LT
22	246.5	186+0.0	75.0' LT

NOTE
The boundaries between bore holes have been established only of Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

FILE NO.	DATE	BY	DESCRIPTION

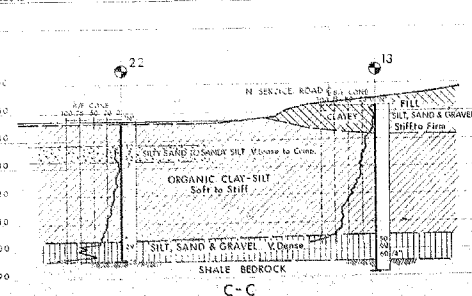
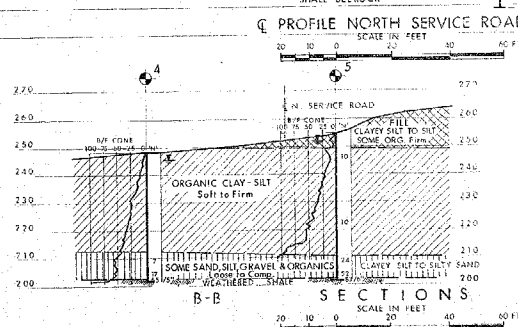
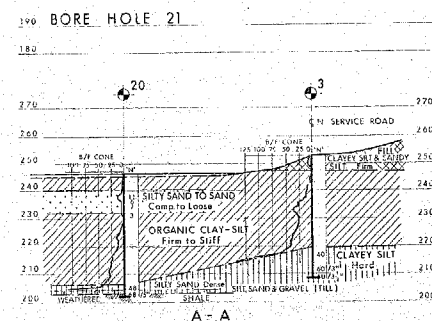
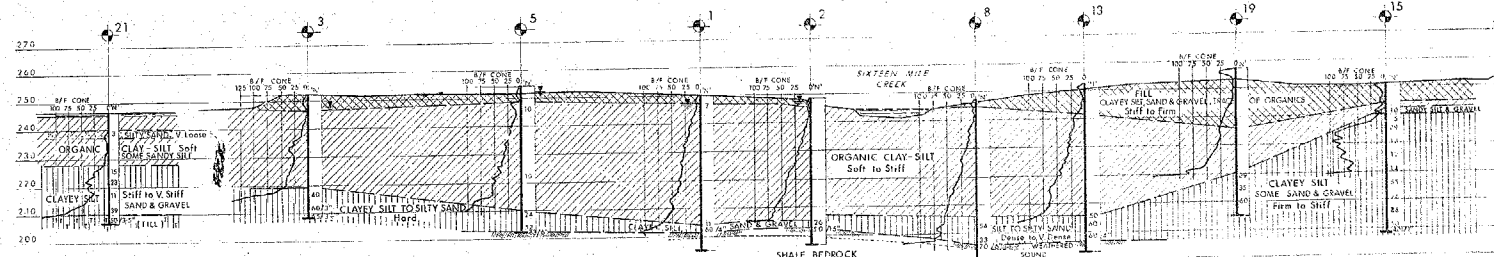
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

SIXTEEN MILE CREEK

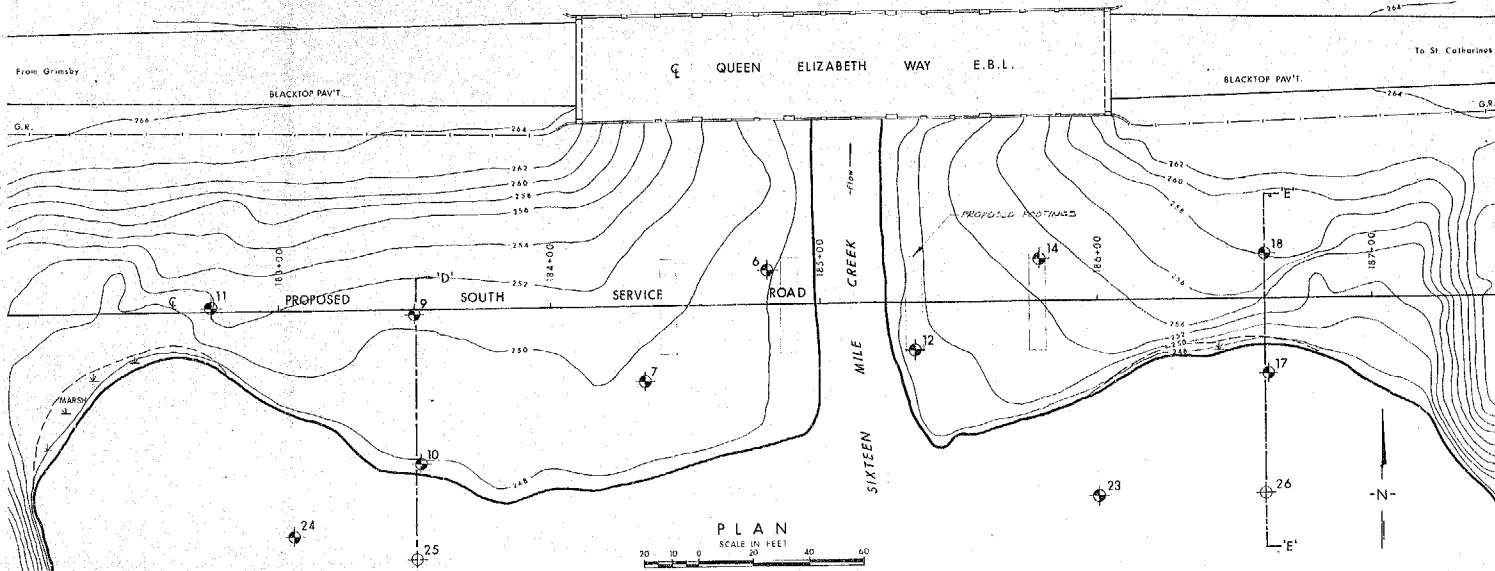
KING'S HIGHWAY NO. Q.E.W. NORTH SERVICE RD. DIST. NO. 4
CO. LINCOLN
TWP. LOUTH LOT 10 CON. 1

BORE HOLE LOCATION & SOIL STRATA

SHEET NO. 1 OF 2	CHECKED	BY NO. 214-43-2	ALST. DRAWING NO.
DRAWN G.T.	CHECKED	JOB NO. 66-F-67	66-F-62A
DATE 19 NOV 1966	SITE NO.		BRIDGE DRAWING NO.
APPROVED	BY		

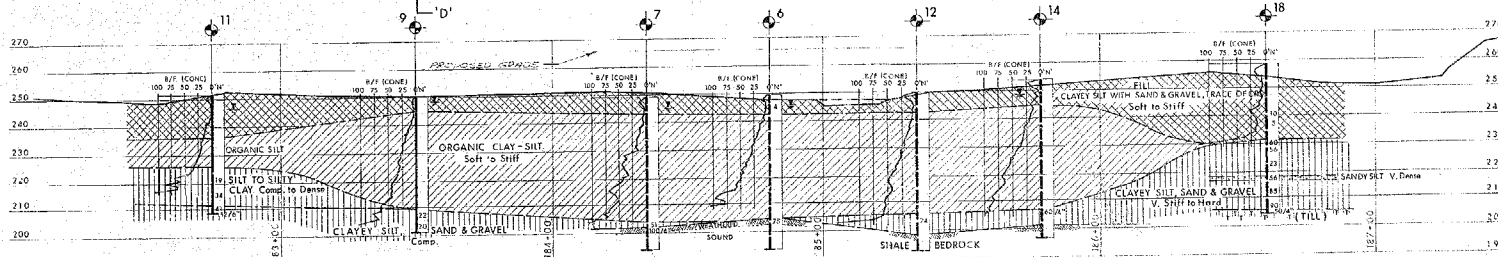


PRINT RECORD	NO.	FOR	DATE



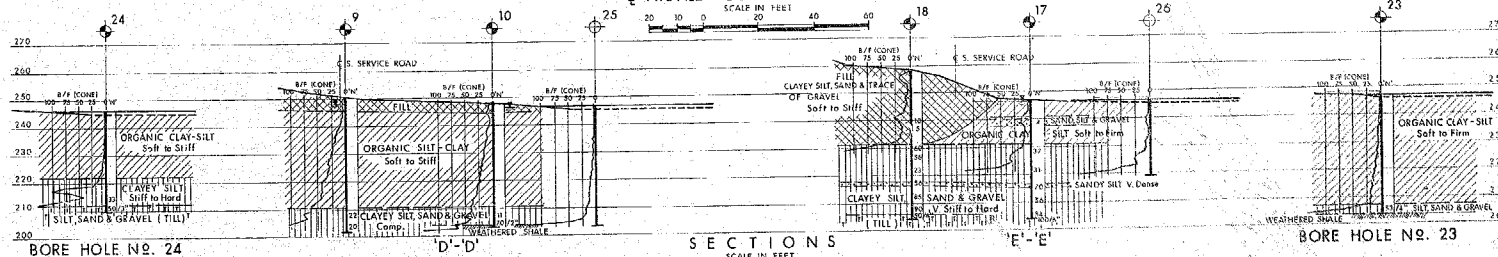
PLAN

SCALE IN FEET



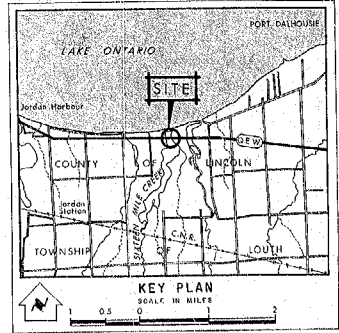
PROFILE SOUTH SERVICE ROAD

SCALE IN FEET



SECTIONS

SCALE IN FEET



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- ⬇ Water Levels established at time of field investigation, JUNE & JULY 1966

NO.	ELEVATION	STATION	OFFSET
6	249.1	184+80	11.5' LT
7	249.7	184+35	27.0' RT
9	250.9	183+49	1.5' ST
10	248.7	183+52	35.5' RT
11	252.9	182+75	1.0' LT
12	250.0	185+24	17.0' RT
14	254.0	185+79	18.5' RT
17	247.2	186+62	26.5' RT
18	258.5	186+41	16.0' LT
23	246.5	186+60	7.0' ST
24	246.5	183+05	81.5' RT
25	246.5	183+50	94.0' RT
26	246.5	186+60	70.0' RT

NOTE

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

NO.	DATE	REVISION
1	19 NOV 1966	1

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

SIXTEEN MILE CREEK

KING'S HIGHWAY NO. Q.E.W. SOUTH SERVICE RD. DIST. NO. 4
CO. LINCOLN TWP. SOUTH LOT 10 CON. 1

BORE HOLE LOCATION & SOIL STRATA

SUBMIT. L.P. CHECKED ☒ W.P. NO. 218-A3-2 M&T DRAWING NO. 66-F-62B
GRAPH. C.T. CHECKED ☒ JOB NO. 66-F-62
DATE 19 NOV 1966 SITE NO. 18 BRIDGE DRAWING NO. 18
APPROVED [Signature] CONT. NO. 18

