

MEMORANDUM

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

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

Attention: Mr. S. McCombie

DATE: November 9, 1966.

OUR FILE REF.

IN REPLY TO: NOV 16 1966

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

E.W. - South Service Rd. Crossing  
of 15 Mile Creek

District #4

W.J. 66-F-68

W. P. 213-63

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.

*Ag 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  
W. S. Melnysh

Foundation Office  
General Files

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# FOUNDATION INVESTIGATION REPORT

For

QEW - South Service Road Crossing  
of 15 Mile Ck.

W.J. 66-F-68 Dis.#4 W.P. 213-63

## 1. INTRODUCTION:

A request, dated June 17, 1965, to carry out a foundation investigation at the proposed crossing of 15 Mile Creek by the South Service Road 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. Catharines and to construct service roads north and south of the main road. This program necessitates the construction of a two-lane multispan structure to carry the south service road over 15 Mile Creek. The north service road has been constructed previously.

Subsequently a foundation investigation was carried out 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 3 miles east of Jordon Harbour, in the township of Louth, County

cont'd /2.....

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, 15 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.

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 backfill was about 3 to 5 feet in height and the approach embankments, built on this fill were about 15 feet in height.

At present the area between the barrier beach and the Q.E.W. right-of-way is occupied by a recreational area. The high land on either side of the creek otherwise is generally being used for orchard cultivation. A small rock fill dam has been placed across the narrowed watercourse by local farmers to control the upstream pond level for purposes of irrigation. The dam and existing backfilled areas are between 2 and 3 feet above the pond level. Water flow is to the north into Lake Ontario.

### 3. FIELD AND LABORATORY WORK:

Using conventional diamond drilling equipment adapted for soil sampling purposes together with a raft where necessary, fourteen sampled boreholes and fourteen dynamic core 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 material 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 given according to the specifications of the Standard Penetration Test. Insitu shear strength was established where possible with a field vane test.

BXL size rock core was 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:

Subsoil at the site consists mainly of a deposit of organic silt-clay underlain by a deposit of clayey silt with sand and gravel (glacial till), and then shale bedrock. A portion of the site has been covered by a fill about 3 to 5 feet deep which is clayey silt to silty clay with some sand and gravel.

The boundaries between the different deposits are shown on the attached Record of Borehole Sheets. The estimated stratigraphical

profiles shown on Drawing No. 66-F-68A are based upon this information.

From ground level downwards the different soil deposits are described as follows:

4.2) Clayey Silt to Silty Clay

This fill material was placed during the construction of the existing bridge. The deposit is generally about 3 to 5 feet thick and consists of clayey silt to silty clay with some sand, gravel and organics. The nature and consistency of this deposit vary from point to point.

4.3) Organic Silt-Organic Clay

This deposit occurred over the entire site either below the fill or immediately below ground surface. Within the area of the pond it was generally 35 feet thick but towards the eastern edge of the pond it decreases to 17 feet thick.

The material was brown to grey brown in colour and relatively uniform. It was highly organic with organic pieces visible throughout the deposit. Well decayed pieces of roots and wood were not uncommon particularly in the lower part of the deposit, in the upper portion occasional samples were fibrous. The samples were prone to expand and occasionally some gas bubbles formed after the shelby tubes were sealed.

The physical properties of this material as determined from field and laboratory tests are summarized as follows:

Bulk density	82-131 p. c. f.
Liquid limit	(22-130% (air dried) (23-74% (oven dried)
Plastic limit	(15-65% (air dried) (16-52% (oven dried)
Moisture content	12-132%
Organic content	1% - 11% (by dry weight)

The shear strength of this material exhibited one of two patterns. Beneath the pond and marshy areas not overlain by fill the shear strength based on field vane tests increases uniformly with depth from approximately 300 lb/sq. ft. at ground surface to 800 lb/sq. ft. at the 30 foot depth. However, beneath the old fill area, this material is distinctly stronger. At the top of the deposit the strength is approximately 1400 lb/sq. ft. It decreases to a minimum of 700 lb/sq. ft. at 8 feet depth and then increases uniformly to 1600 lb/sq. ft. at 40 feet depth.

The laboratory quick triaxial and unconfined compression tests gave values of shear strength which were up to approximately 400 lb/sq. ft. less than the field vane tests. This is attributed to sampling disturbance. The shear strength values used for design purposes are based on the envelope of the lowest undrained shear strengths obtained using the field vane. The shear strength profiles with depth are presented in the appendix to this report.

#### 4.4) Clayey Silt with Sand and Gravel (Glacial till)

This is a heterogeneous mixture of stiff grey silty clay, fine-coarse sand and fine-coarse gravel. It immediately underlies the organic silt-organic clay. The thickness of this deposit ranges from 4 to 11 feet.

The N values obtained from the Standard Penetration test ranged from 8 blows/foot to 84 blows/foot, indicating a stiff to hard consistency.

#### 4.5) Bedrock

This consists of red shale containing layers of green shale.

This was proven in boreholes number 1, 2, & 6 by taking cores of 7, 5, and 8 feet respectively. The rock surface was weathered to a depth of 1.5 to 3 feet. The surface of the weathered rock was proven at other locations by obtaining split spoon samples, and by driving penetration cones to refusal.

Average depth to bedrock over the whole site is about 45 feet.

5. GROUNDWATER

The water table is generally at elevation 246.3 which is the same as the waterlevel in the creek at the location of the proposed structure.

6. DISCUSSION AND RECOMMENDATION:

It is proposed to construct a new structure to carry the South Service Road of the Q.E.W. over 15 Mile Creek. The presently proposed structure is a three span bridge having three equal spans, each 45 feet long. The approach fills are generally 17 feet higher than the existing ground level.

Subsoil conditions at this site consist essentially of a 35 feet thick deposit of soft to stiff organic silt and clay underlain by till and then bedrock.

The organic silt and clay deposit has given rise to difficulties in constructing the adjacent dual bridges which were built in 1939 to carry the Q.E.W. over 15 Mile Creek, and also recently in constructing the bridge which carries the North Service Road over this creek.



No details are available as to the exact difficulties experienced in 1939. However, from the available drawings it can be seen that a three span structure (36'-6", 40'-0", & 36'-6"), with approach fills having a slope of  $1\frac{1}{2}$ :1 in the longitudinal direction was originally proposed, (Drawing No. D 2538-1, July 8, 1938).

The drawing shows the structure supported on wooden piles which no doubt were founded in the till or on bedrock. However, it would appear that difficulties occurred during construction since Drawing No. 2538-7, December 30, 1938, shows details of footing struts for 15 & 16 Mile Creek Bridge. These struts brace the footings in the longitudinal direction. They were to be installed not higher than elevation 244'-0" which is below present creek bottom level, so that they would not be visible today. The presence of these struts or otherwise has not yet been proven at the site. Drawing No. D-2538-8 shows 15 Mile Bridge with two additional spans 20' long at each end of the bridge supported on steel H-piles and with the approach fill slope in the longitudinal direction was reduced to approximately 4:1. This drawing shows no details of the pile foundations of the original piers and abutments so that no conclusions can be drawn from this drawing as to whether the longitudinal strutting was actually installed.

The bridge carrying the North Service Road over the creek is a three span structure each 44 feet long supported on steel H-piles to bedrock, with slopes in the longitudinal direction of 2:1. Initially, no measures were taken to ensure lateral stability of the pier foundations in the longitudinal direction. However, on completing the pier and foundations,

slight movements of the piers were observed (3 to 5 inches at bearing level) and recourse was again made to strutting the foundations in the longitudinal direction. In addition the approach fill for a distance of 40 feet behind each abutment was excavated, removed, and replaced by light-weight fill to improve the stability of the abutment.

In view of the previous experience gained at this site the following alternatives are recommended for the new structure:

6.1) The Bridge as Originally Proposed

Structure Foundations

The organic silt and clay deposit at this site cannot satisfactorily support spread footings and it is therefore recommended that the bridge piers and abutments should be supported on steel piles driven to bedrock or on lined concrete caissons to bedrock at approximate elevation 210. The design load for the steel H-piles can be the maximum for the section. A safe load 150 tons per caisson can be used for the 36"  $\phi$  concrete caisson on bedrock.

It appears from the experience obtained from the previously constructed bridges at the site that the embankment fill placed on organic soils have a tendency to squeeze the underlying organic silt and clay against the pile and caps. The piles should therefore be strutted between the pile caps to resist the lateral forces which will occur, or if concrete caissons are used these can be made integral with the bridge deck and designed as moment carrying members.

An alternative solution would be to sub-excavate all material within the area between the bridge piers and abutments together with an additional 5 feet wide perimeter area, down to approximate elevation 238.0' and replace this material with suitable granular fill. The final side slopes to this excavation should not be steeper than 1:1. This excavation

and backfilling should be carried out prior to constructing any other part of the approach embankment.

Since 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 approximate elevation 237, ensuring that this results in a penetration of at least 5 ft. below the base of the excavation.

#### Structure Approaches

The approach fills for this bridge extend into the area of the pond where they would overly soft organic silt and clay. It is anticipated that should the proposed east approach fill be constructed failure may occur in the lateral direction.

To ensure lateral stability of the fill sub-excavation should be carried out to a depth of 15 feet below the bottom of the pond as indicated below. The excavated material should be replaced with suitable granular fill.

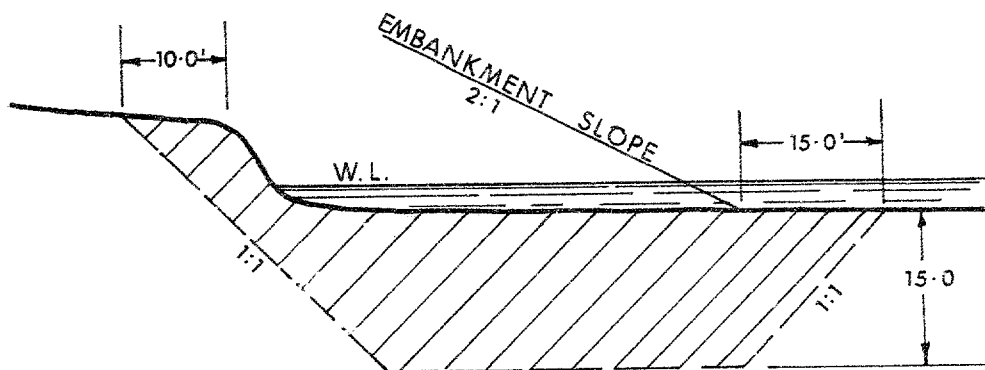


Diagram showing subexcavation requirements at toe of embankment

Sub-excavations will be required for all of the east approach fill where the toe of the embankment spills into the pond.

It is considered that the west approach fill will be stable. However, between the limits as defined by the intersection of the centre line bearing of the west abutment and west pier with the pond this same sub-excavation procedure is recommended.

If these recommendations are carried out, no stability problems are anticipated. However it is anticipated that settlements of the approach fills will occur. It is therefore recommended that the approach fills be completed and allowed to stand for one year before they are paved. Nevertheless, settlement will occur for some time afterwards and the pavement will have to be maintained periodically.

#### 6.2) A 500 Feet Long Structure

The previously described stability and settlement problems could be obviated by constructing a long trestle type structure founded on steel H-piles or lined concrete caissons to bedrock. This structure would commence at approximately Sta. 197+75, and finish at Sta. 203+50.

Consideration should be given to future developments of the C.E.W. Any additional fill placed for the widening of the main lanes could cause lateral movements of the trestle type of structure.

#### 6.3) Flexible Pipe-Arch Culvert

Consideration should be given to using a flexible pipe arch culvert as an alternative to the proposed structure with the full expectation of considerable settlement. In this case the culvert should be supported on a pad 2 to 3 feet thick of suitable granular fill. Sub-excavation for the toe of the east approach fill should be carried out as outlined previously.

If the proposal is adopted the Foundation Section will supply details of the necessary camber to be provided to compensate for the expected settlement of the culvert.

The classification as to whether the creek is to be used for navigation purposes may influence this proposal.

## 7. SUMMARY

A foundation investigation for a proposed new structure to carry the South Service Road of the Q.E.W. over 15 Mile Creek is reported.

Subsoil conditions at this site consist essentially of a 35 feet thick deposit of soft to stiff organic silt and clay underlain by till and then bedrock. The organic silt and clay deposit has given rise to difficulties in constructing existing adjacent bridges and these have been reviewed herein.

Three alternative proposals have been put forward for the new structure. 1. The bridge as originally proposed. 2. A 500 feet long structure. 3. A flexible pipe arch culvert.

For the bridge structures it has been recommended that the new structure be founded on steel H-piles driven to bedrock or on lined concrete caissons to bedrock. Procedures for construction and dewatering have been outlined in this report.

Recommendations have been made for the construction procedure to be followed for the approach fills. No stability problems are anticipated if these recommendations are carried out. However the approach fills are expected to undergo large settlements and they will therefore have to be maintained periodically. It has been recommended that the pavements to the approach fills should not be constructed until

a year after the fills have been completed.

The classification of 15 Mile Creek for navigational purposes may influence the proposal for a flexible pipe arch culvert.

8. MISCELLANEOUS

The field work for this project was carried out during the period July 18 to August 15, 1966, under the supervision of Mr. L. Palmer, Project Foundation Engineer.

The equipment used was owned and operated by Canadian Longyear Ltd.

This report was written by Mr. L. Palmer and Mr. A. C. Calder, Project Foundation Engineer, and was reviewed by Mr. M. Devata, Supervising Foundation Engineer.

November, 1966.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66 - F - 68

LOCATION GEW &amp; 15 Mile Crk; S. Service Rd, Sta 200 + 76, o/s 14' RT

ORIGINATED BY L.P.

W.P. 213 - 63

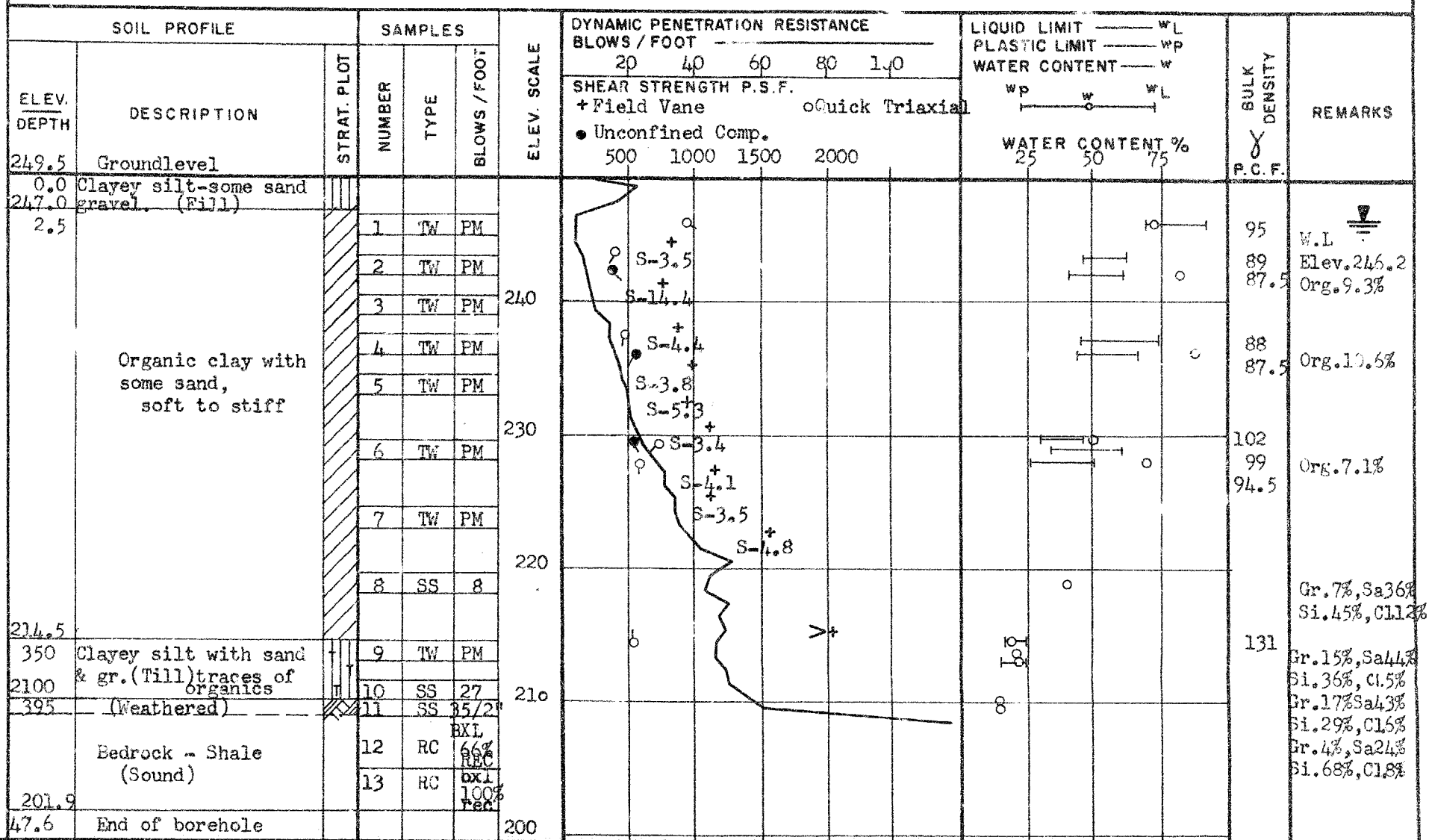
BORING DATE July 18 1966

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing; Cone.

CHECKED BY D.K.



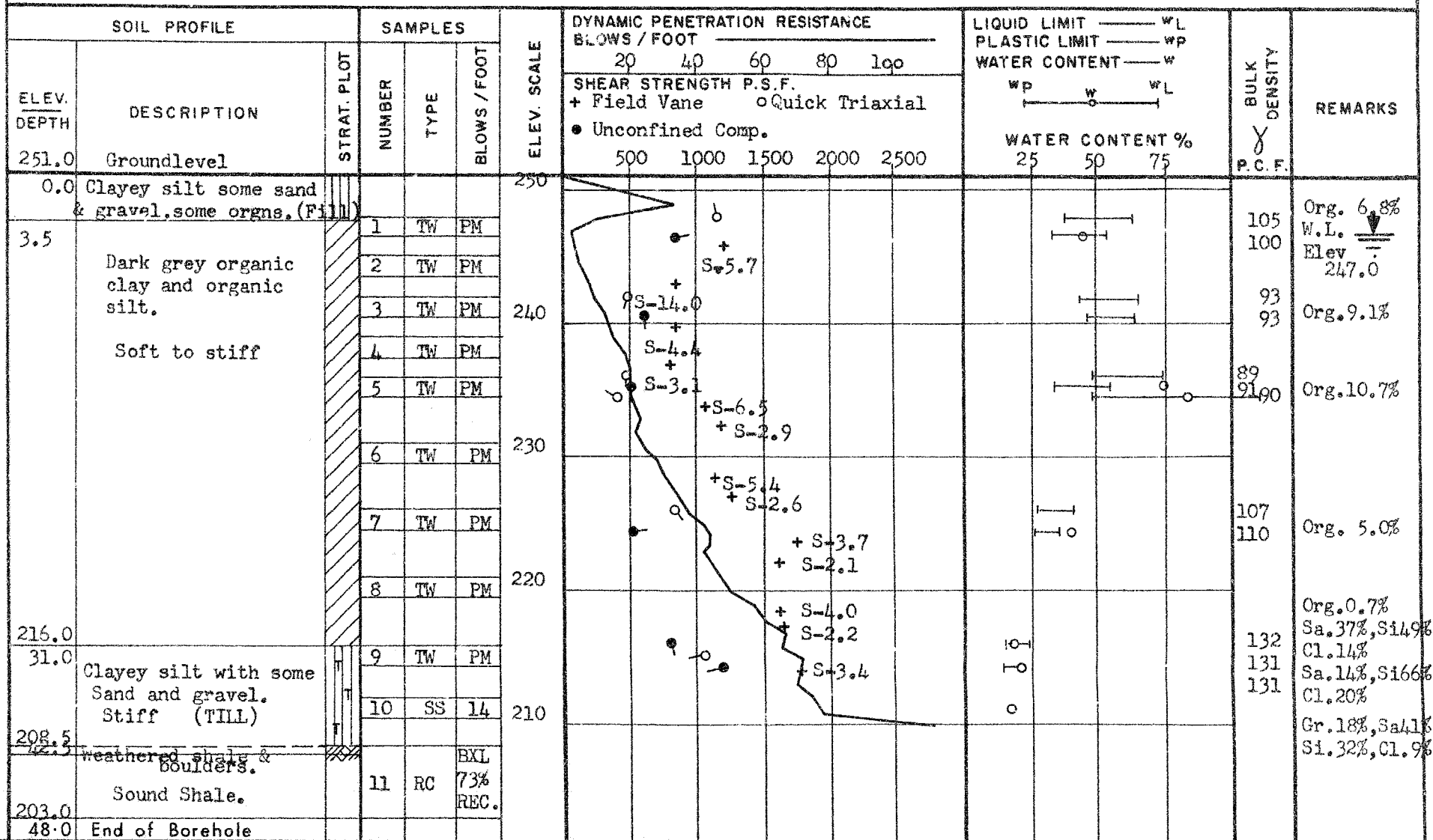


DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-68LOCATION Qew & 15 mile crk; Service Rd., Sta 200 + 45.0/s17' LT.ORIGINATED BY L.P.W.P. 213-63BORING DATE July 19, 20/66COMPILED BY W.T.E.DATUM GeodeticBOREHOLE TYPE Washboring, Nx Casing, Cone.CHECKED BY D.K. *WRE*

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

JOB 66 - F - 68

LOCATION CEW & 15 Mile Crk. Sta. 199 +98. o/s 49' RT.

ORIGINATED BY L.P.

W.P. 213-63

BORING DATE July 20, 21 /66

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing, Cone.

CHECKED BY \_\_\_\_\_ D.K. *[Signature]*

## RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

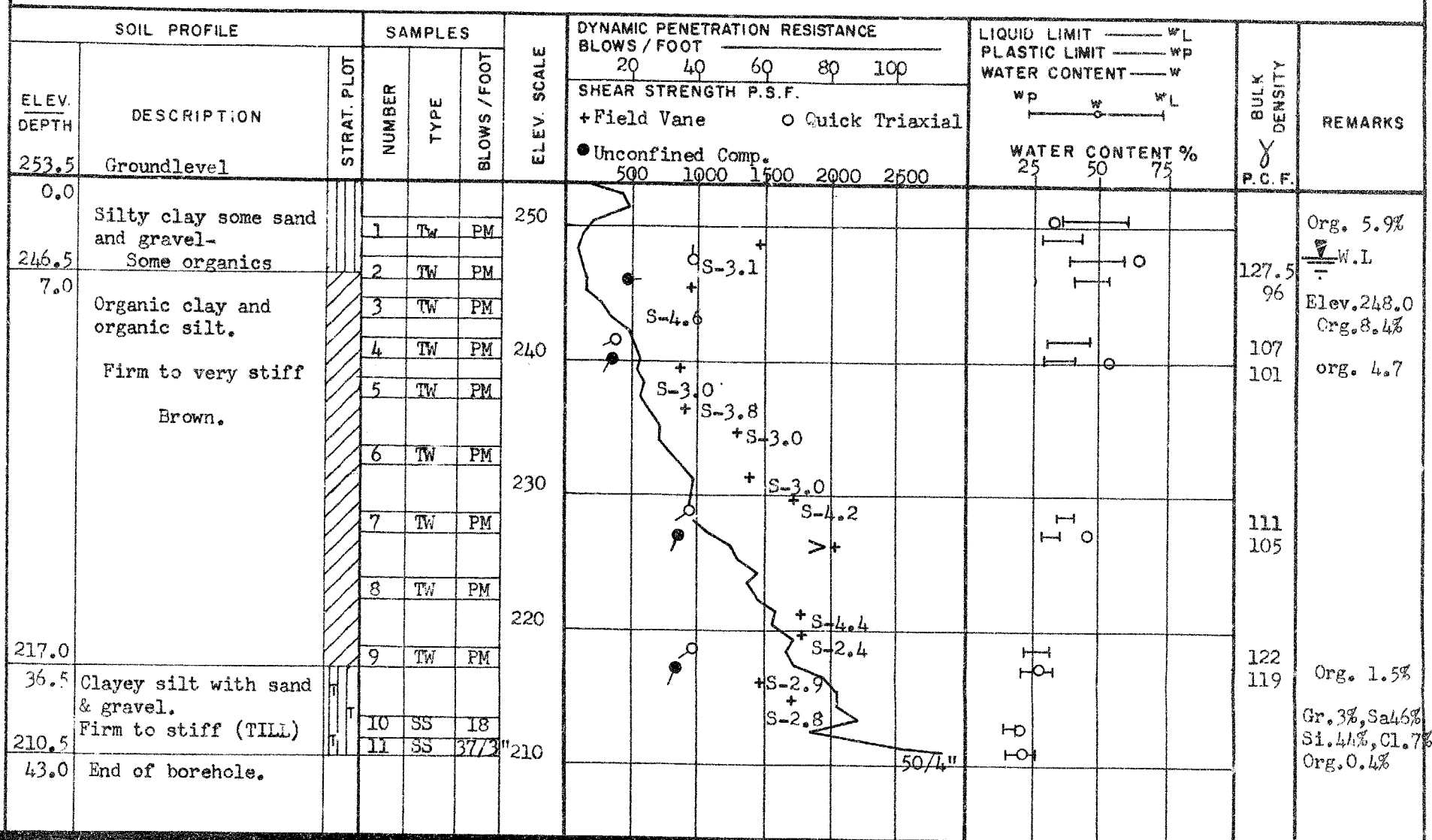
## RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 66-F-68

LOCATION Gow & 15 Mile Crk: Sta. 199 + 40, o/s 15' LT.ORIGINATED BY L.P.

W.P. 213-63

BORING DATE July 21 & 22 1966COMPILED BY W.T.E.DATUM GeodeticBOREHOLE TYPE Washboring, NX Casing, Cone.CHECKED BY D.K.

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 66 - F - 68

LOCATION Oew & 15 Mile Crk: Sta. 198 + 45. O/S 37' RT.

ORIGINATED BY L.P.

W.P. 213- 63

BORING DATE July 22 & 25 1966

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing, Cone.

CHECKED BY D.K.

[illegible]



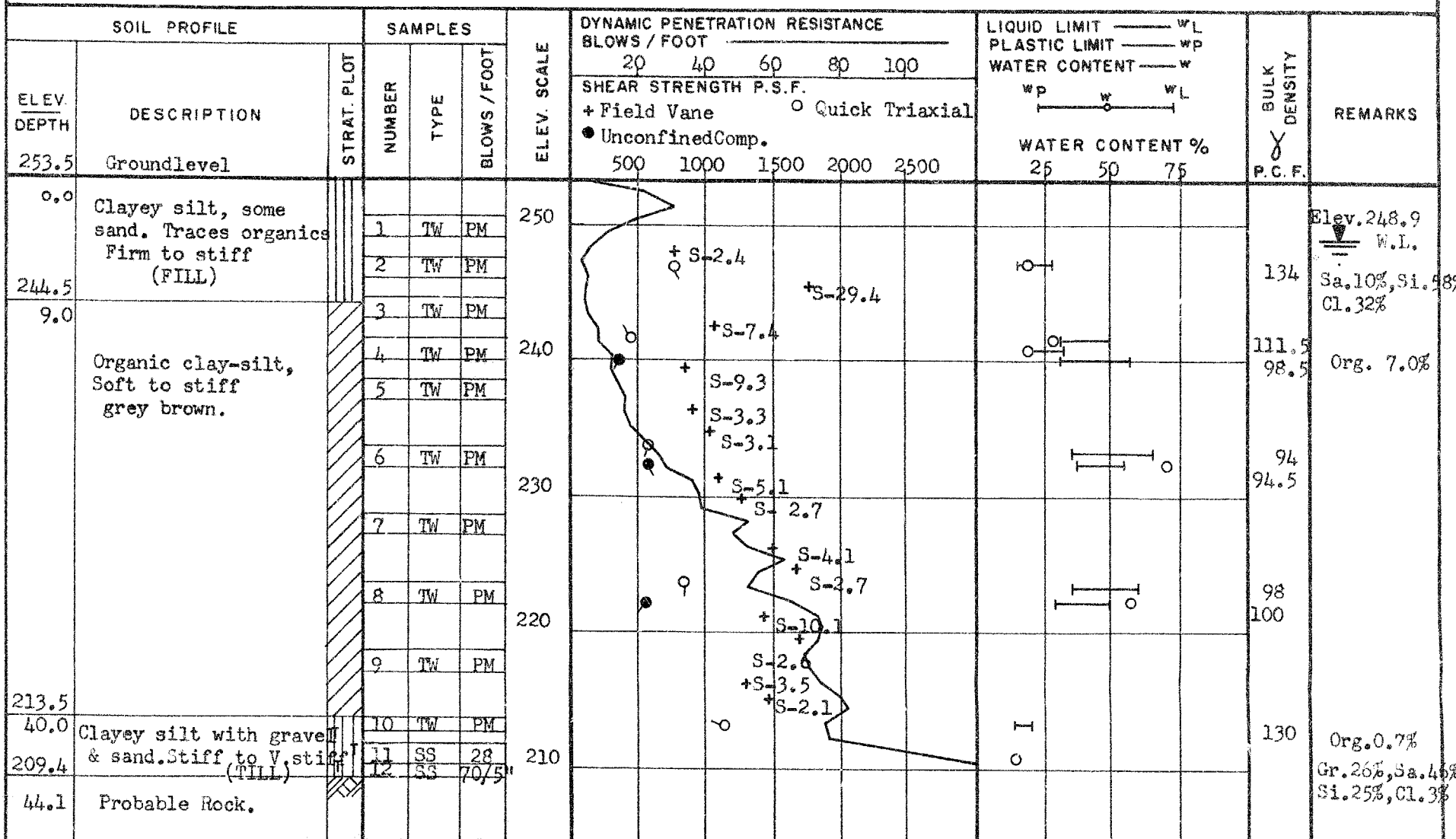
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 66 - F - 68 LOCATION Cow & 15 Mile Crk; Sta. 202 + 46.5, o/s 16' LT. ORIGINATED BY L.P.  
W.P. 213 - 63 BORING DATE July 26, 1966 COMPILED BY W.T.E.  
DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing; Cone. CHECKED BY D.K. H.R.



DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

JOB 66 - F - 68

LOCATION C.E.W. & 15 Mile Crk; Sta. 203 + 03 32' RT.

ORIGINATED BY L.P.

W.P. 213 - 63

BORING DATE July 27 & 28 1966

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, Nx Casing: Cone.

CHECKED BY D.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LQUID LIMIT ——— WL	BULK DENSITY  P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20    40    60    80    100	PLASTIC LIMIT ——— wp		
							SHEAR STRENGTH P.S.F. + Field Vane      Quick Triaxial ● Unconfined Comp.	WATER CONTENT ——— w		
							500   1000   1500   2000   2500	wp                  w                  WL		
								WATER CONTENT % 25    50    75		
247.0	Groundlevel									
0.0	Brown organic silt and organic clay		1	TW	PM	240	-o		99	W.L. Elev. 246.3
	Firm to stiff		2	TW	PM		+ S-3.6			
			3	TW	PM		+ S-4.5			
			4	TW	PM		+ S-6.6			
			5	TW	PM		+ S-8.0			
229.5			6	SS	30	230	> +		85	
17.5	Clayey silt, some sand, grey brown to grey. Hard.		7	SS	33	220			82	
25.5	Silt, Grey, Compact.		8	SS	47					
28.0	Occasional thin silt seams.		9	SS	75	210				
211.0			10	SS	45/4"					
36.0	Clayey silt, Sa & Gr (T.M.)		11	SS	50/6"					
207.9	Hard weathered shale									
39.1	End of borehole.					200				

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

JOB 66 - F- 68

LOCATION Q.E.W. & 15 Mile Crk; Sta. 202 + 49, o/s 60' RT.

ORIGINATED BY L.P.

W.P. 213 - 63

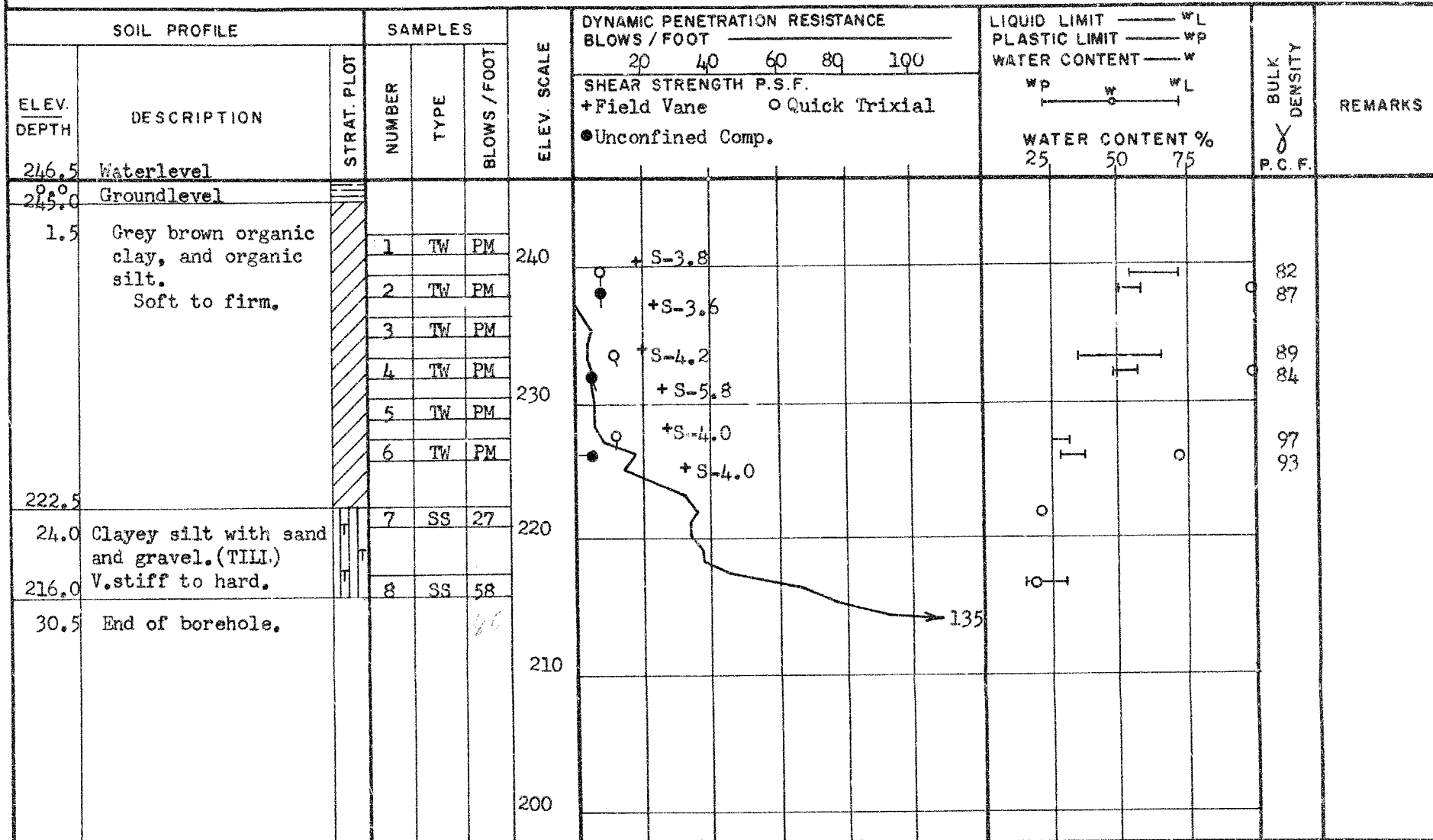
BORING DATE August 8 & 9 1966

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing; Cone.

CHECKED BY D.K.





DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

## MATERIALS &amp; TESTING DIVISION

JOB 66 - F - 68

LOCATION CEW & 15 Mile Crk: Sta. 202 + 51, o/s 100' RT.

ORIGINATED BY           L.P.          

W.F. 213 - 63

BORING DATE August 9 1966

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing, Cone.

CHECKED BY                      D.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	Liquid Limit — WL	BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. P. LOT	NUMBER	TYPE	BLOWS / FOOT		20 40 60 80 100	PLASTIC LIMIT — WP		
							SHEAR STRENGTH P.S.F. + Field Vane      ° Quick Triaxial • Unconfined Comp. 500 1000 1500 2000 2500	WATER CONTENT % WP — W — WL 25 50 75		
246.5	Waterlevel									
245.0	Groundlevel									
1.5	Brown organic clay soft to firm.		1	TW	PM	240	+ S-4.7		85	Org. 6.5%
			2	TW	PM		+ S-6.3		87	
			3	TW	PM		+ S-6.5		88	
			4	TW	PM					
			5	TW	PM	230	+ S-1.9		130	Org. 1.0%
228.0			6	SS	18				129	
18.5	Clayey silt with sand and gravel, V.stiff to hard (TILL)		7	SS	20	220				Org. 0.14% Gr. 1%, Sa. 5% Si. 80%, Cl. 1.1%
216.0			8	SS	41					
30.5	End of borehole.					210				

## OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

JOB 66 - F- 68LOCATION Q.E.W. & 15 Mile Crk; 202 1/2 MI o/s 701 Rt.ORIGINATED BY L.P.W.P. 213 - 63BORING DATE August 10 1966COMPILED BY W.T.E.DATUM GeodeticBOREHOLE TYPE Washboring, NX Casing, Cone.CHECKED BY D.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	SAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WP	W	WL		
246.8	Waterlevel						500	1000	1500	2000	2500	25	50	75		
244.8	Groundlevel															
2.0	Dark brown organic clay & organic silt. Soft to firm.		1	TW	PM	240									134	90
			2	TW	PM											79
			3	TW	PM											104
			4	TW	PM											92
			5	TW	PM	230										104
			6	TW	PM											104
			7	TW	PM	220										
216.3			8	TW	PM											125
30.5	Brown clayey silt with sand and gravel. (TILL)															123
210.8	Firm to hard.		9	SS	35/6"	210										
36.0	Probable shale.															

Org. 0.5%  
Gr. 27%, Sa. 40%  
Si. 29%, Cl. 4%  
Bouncing

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 12

FOUNDATION SECTION

JOB 66 - F - 68

LOCATION C.E.W. & 15 Mile Crk; Sta. 202 / 00, o/s 100' RT

ORIGINATED BY L.P.

W.P. 213 - 63

BORING DATE August 10 & 11 1966

COMPILED BY W.T.E.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing; Cone.

CHECKED BY D.K.

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	W		
246.5	Waterlevel															
244.5	Groundlevel															
2.0	Brown grey organic clay and organic silt		1	TW	PM	240									108	86
			2	TW	PM											
			3	TW	PM											
			4	TW	PM											
			5	TW	PM	230									97	98
			6	TW	PM											
			7	TW	PM	220									100	96
217.5																
29.0	Clayey silt with sand and gravel and organics		8	TW	PM											
			9	SS	9	210										
210.0	Firm (TILL)		10	SS	6											
36.5	Probable weathered shale.															
	End of cone test.															

SHEAR STRENGTH P.S.F.  
+ Field Vane      o Quick Triaxial  
• Unconfined Comp.

WATER CONTENT %  
25    50    75

Org. 2.7%  
Gr. 49%, Sa. 30%  
Si. 17%, Cl. 4%

50/3"

## FOUNDATION SECTION

ORIGINATED BY L.P.

COMPILED BY W.T.E.

CHECKED BY D.K.

[illegible]

MATERIALS &amp; TESTING DIVISION

JOB 66-F-68

LOCATION Q.E.W. & 15 Mile Crk: Sta. 198 + 77, o/s 50' RT.

ORIGINATED BY L.P.

W.P. 213 - 63

BORING DATE August 12, & 15 1966

COMPILED BY W.T.E.

DATUM Geodetic

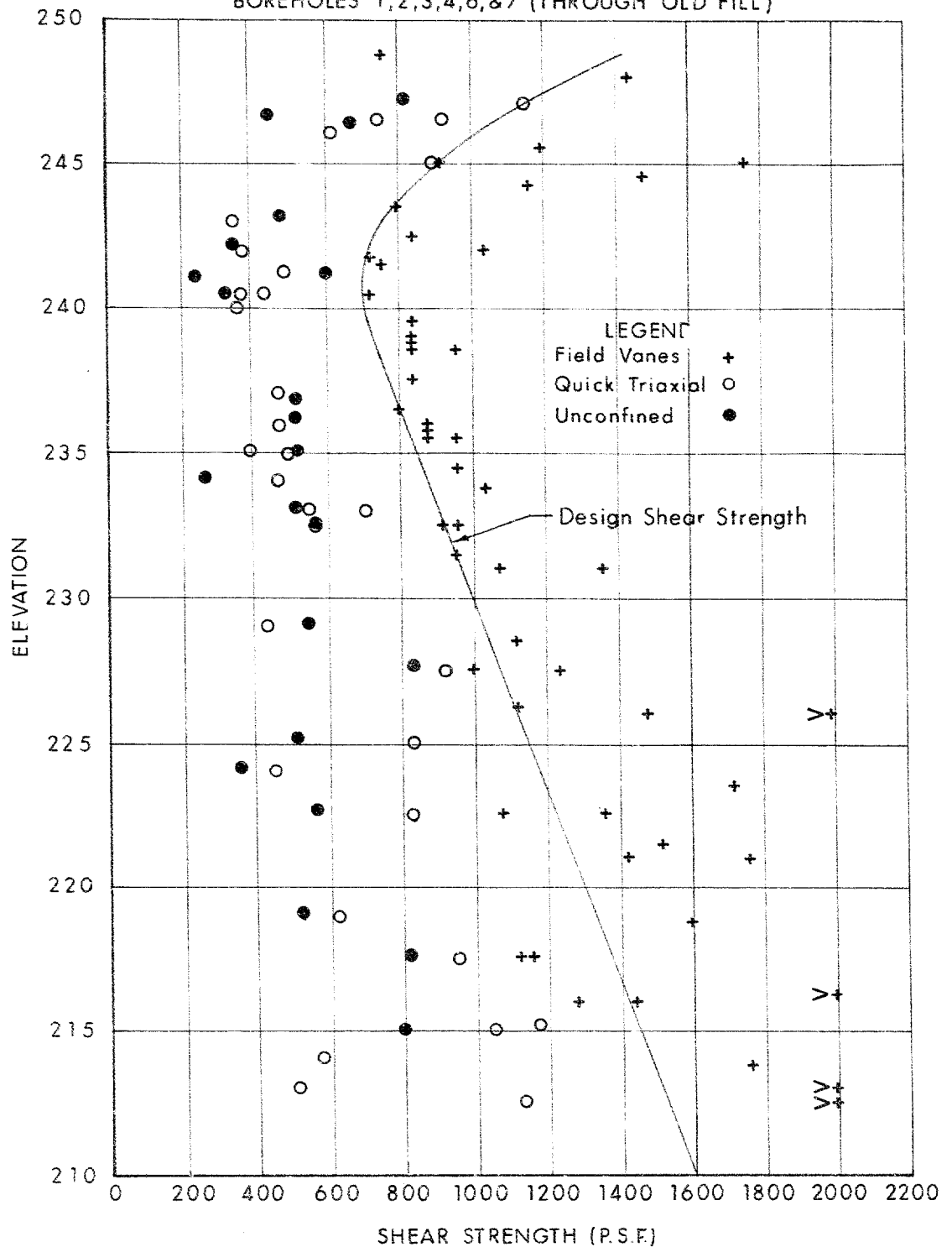
BOREHOLE TYPE Washboring, NX-RX;Cone.

CHECKED BY D.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	BLOW PENETRATION RESISTANCE					LIQUID LIMIT ——— WL			BULK DENSITY Y P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					PLASTIC LIMIT ——— WP					WATER CONTENT %
							20	40	60	80	100	WATER CONTENT ——— W					
							SHEAR STRENGTH P.S.F.					wp ——— w ——— WL					
							+ Field Vane      O Quick Triaxial										
							● Unconfined Comp.										
							500    1000    1500    2000    2500					25    50    75			W.L. ▼		
247.5	Groundlevel																
	Grey organic clay Soft to firm		1	TW	PM												
			2	TW	PM	240											
			3	TW	PM												
			4	TW	PM												
			5	TW	PM	230											
			6	TW	PM												
			7	TW	PM	220											
216.5			8	TW	PM												
31.0	Grey clayey silt with sand and gravel. V.stiff to hard. (TILL)		9	SS	11										Org. 0.5%		
			10	SS	47	210											
205.0			11	SS	94										Org. 0.3%		
42.5	(With boulders)		12	SS	76										Gr. 12%, Sa. 4%		
202.8	Weathered shale.		13	SS	60/2"										Si. 30%, Cl. 9%		
44.7	End of borehole.		14	RC	BXL	200											

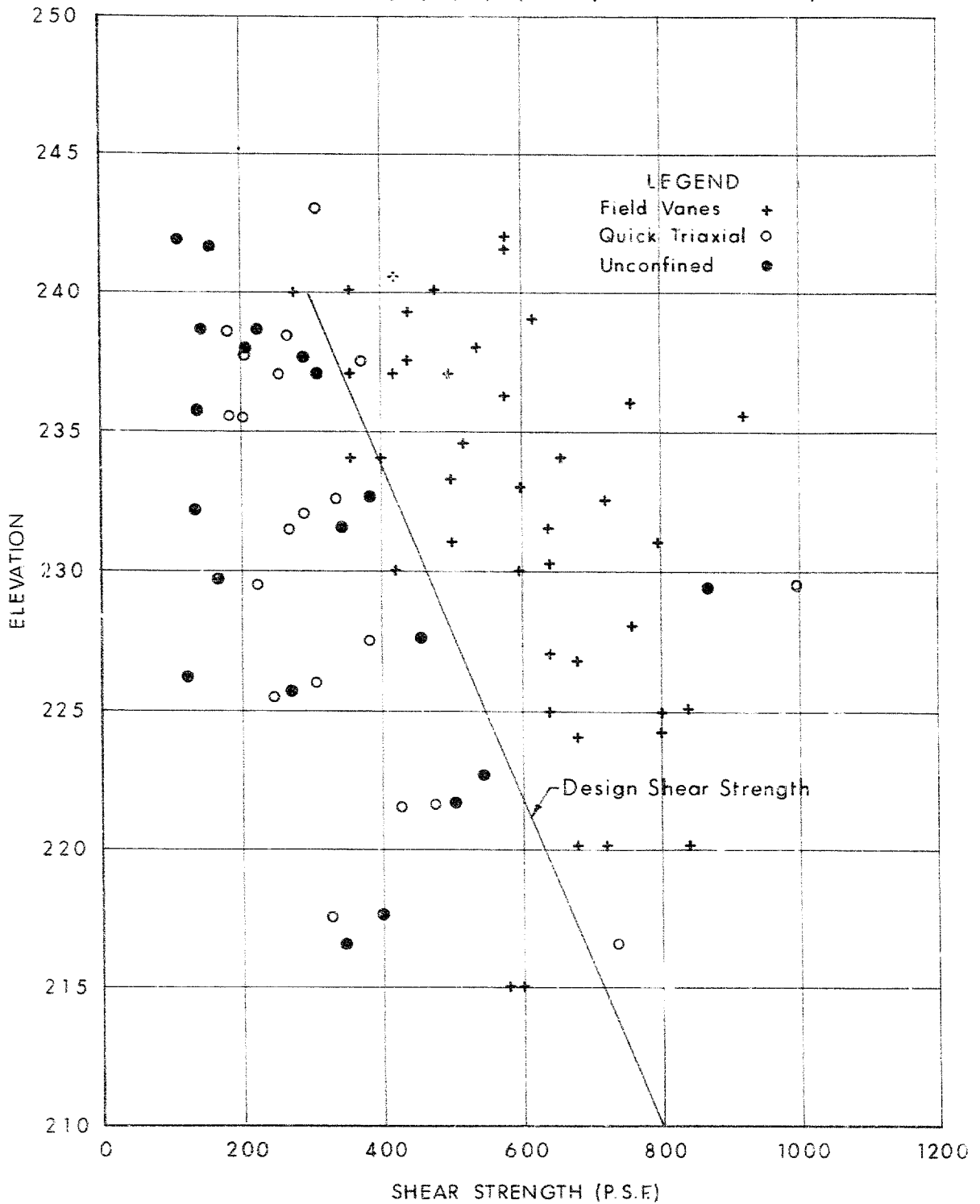
# SHEAR STRENGTH PROFILE

BOREHOLES 1,2,3,4,6,&7 (THROUGH OLD FILL)



# SHEAR STRENGTH PROFILE

BOREHOLES 8, 9, 10, 11, 12, 13, & 14 (IN MARSH & POND)



## 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

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

## GENERAL

$\pi$	= 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
$\sigma$	NORMAL STRESS
$\sigma'$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

## EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	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
$\beta$	ANGLE OF SLOPE TO HORIZONTAL

66-F-68

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Eng., Room 107, Lab. Bldg.

Mr. W.G. Wigle,  
Program Engineer,  
Program Section,  
Admin. Bldg.

Bridge Division,  
Downsview, Ontario.

December 12th, 1966.

Fifteen Mile Creek, W.P. 213-63,  
Sixteen Mile Creek, W.P. 214-63,  
Eighteen Mile Creek, W.P. 286-66,  
G.E.W. Service Roads, District #4.

Reference is made to previous correspondence concerning the above structures and the problems associated with the inclusion of dams as requested by the Hon. R. Welch, M.P.P. for Lincoln.

Having discussed this with the Niagara Peninsula Conservation Authority a solution has been reached which will allow us to shortly undertake the design the structures independent of the Authority's work. (See attached letter)

Investigation of the sites by the Foundation Branch has revealed that embankment stability and settlements of the approaches require special consideration. Their recommendation is to construct the embankments well in advance of the opening of the service roads. (See letter)

We request that this be reviewed with the possibility of calling the grading contract, at least for the approaches, first, followed later by the structures and paving.

WSM/aw

  
W.S. Melnychyn,  
Regional Bridge Location Engineer.

c.c. C.K. Hunter  
A. Stermac  
R. Forrest

66-F-61

Mr. S. McCombie,  
Bridge Planning Engineer,  
Bridge Division, Admin. Bldg.

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

Attn: Mr. W. S. Melinyshyn

October 20, 1966

Recommended Stage Construction - Q.E.W.

66-F-68 ✓ 66-F-63

Our investigation of the three sites on the Q.E.W. (Fifteen Mile Creek, W.P. 213-63; Sixteen Mile Creek, W.P. 214-63, and Eighteen Mile Creek, W.P. 211-63), has disclosed the presence of a very soft, highly compressible, organic layer of silt and clay overlying bedrock or till. The thickness of this layer is in the order of 30 to 40 ft. There is also ample evidence of settlements and slope instability at the above mentioned three creek crossings. 66-F-62

Based on our findings, relatively large settlements of the embankments to be built on the soft and compressible layer, can be expected.

The proximity of the present Q.E.W. and the thickness of the compressible layer, rules out subexcavation and removal of this material. Depending on the final choice of the structures for the crossing of the creeks, partial subexcavation may be given consideration. However, this may be more for embankment stability reasons than settlement reduction. In order to reduce settlements that will have to be contended with after the road has been opened to traffic, we would strongly recommend that the embankments (grading contract) be built as far ahead as possible.

Should you wish to discuss any aspects of this problem, or would require additional information, please feel free to call on this Office.

*A. G. Stermac*

A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER

AGS/ndeF

cc: Foundations Office

Gen. Files

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Eng., Room 107, Lab. Bldg.

Mr. W. C. Friedmann,  
Sr. Project Planning Engineer,  
Planning Branch,  
Central Region.

Bridge Division,  
Downsview, Ontario.

April 6th, 1967.

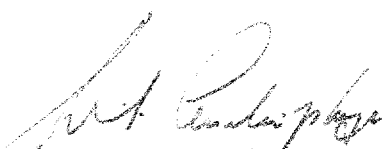
Attention: Mr. L. Schwabl

15 Mile Creek Structure,  
Q.E.W. South Service Road,  
Q.P. 213-63,  
District No. 4.

We request that your office re-investigate the revised alignment at the above site with a view to minimizing the southerly shift of 12 feet.

This shift pushes the road approaches further into the 15 mile creek basin, where soil conditions are very poor. This requires more extensive and costly sub-excavation plus replacement with suitable granular.

WSM/aw

  
W. S. Melnyshyn,  
Regional Bridge Location Engineer.

c.c. G. Hunter  
H. Greenland  
W. Kinnear  
T. Kovich  
A. Stermac

Department of Highways Ontario

Copy for the information of

A.G. Stermac, Principal Foundation Eng., Room 107, Lab. Building.

Mr. W. Wigle,  
Program Engineer,  
Program Section,  
Admin. Bldg.

Bridge Division,  
Downsview, Ontario.

April 3rd, 1967.

W.P. 213-63, Site 18-23,  
Fifteen Mile Creek,  
South Service Road Structure,  
Q.E.W., District 4.

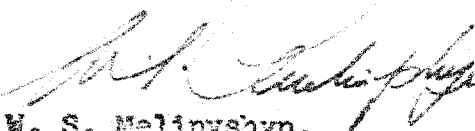
16-F-68

Because of complex traffic movements associated with the Highway 406 - Q.E.W. interchange (W.P. 77-63) just west of St. Catharines, revisions have had to be made to the Seventh Ave. Interchange (W.P. 212-63). These revisions affect the alignment of the south service road structure at Fifteen Mile Creek.

We are instructing our design section to cease work on this project until revised plans are received. Further foundation investigation may also be required depending on the extent of this southerly shift of alignment.

We are somewhat dismayed that these revisions to 15 Mile Creek, which have been known to be taking place for some time now, should come to our attention purely by accident this morning.

JFW/aw

  
W. S. Melinyshyn,  
Regional Bridge Location Engineer.

c.c. R.G. Burnfield  
C.S. Crebski  
G.K. Hunter  
A.G. Stermac  
S. McCombie  
R. Forrest

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

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

August 1, 1967

Fifteen Mile Creek Bridge  
R.S.W. South Service Road  
N.P. 213-63, Site 18-202

10-F-68

We have re-reviewed the subsoil conditions taking into account the revised alignment of the South service road approximately 3 ft. South (at the structure) farther into the swamp, and submit the following comments:

Since there is no change in the proposed grade, in our opinion, the recommendations contained in the original foundation report should adequately cover the new alignment. We feel there is no necessity for additional borings at the revised location.

RD/Kief

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

cc: Messrs. C. S. Grebski  
S. McCombie  
G. K. Hunter

Foundation Files  
Gen. Files

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Eng., Room 107, Lab. Bldg.

Mr. C. Orobaki,  
Bridge Design Engineer,  
Admin. Bldg.

Bridge Division,  
Downsview, Ontario.

July 20th, 1967.

Fifteen Mile Creek Bridge,  
Q.E.W. South Service Road,  
W.P. 213-63, Site No. 18-202.

We enclose herewith revised survey plans B-138-20-  
Location plan, C-138-15 - Location profile and E-4734-1.  
Site plan as required for the design of the above structure.  
The revision shifts the alignment of the south service road  
approximately 8 feet south (at the structure) further into  
the swamp.

Your office may now resume the design based on the  
revised plans. The structure cross section remains as  
previously issued. We will forward appropriate plans to  
the Foundation Branch in the event that they may wish to  
revise their recommendations especially on the treatment  
of the approach fills.

WEM/co  
Encl.

  
W. S. Melnychuk,  
Regional Bridge Location Engineer.

c.c. R.B. Burnfield  
G.H. Hunter  
H. Greenland  
E. McCombie  
A. Stermac  
W. Forrest  
A. Crowley

P.S.

Mr. Stermac:

Enclosed are copies of the site and location plans  
showing the entire shift of the alignment of the South  
Service Road. Your comments, on the necessity of addition  
borings or revised recommendations, will be appreciated by  
the Bridge and Road Design Offices.

alp

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

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

October 2, 1967

Fifteen Mile Creek Bridge  
W.P. 213-63, Site 13-23  
Q.E.W. - South Service Rd.  
District #4 (Hamilton).

60-1000

We have reviewed the Preliminary Bridge Plan  
Drawing D-6103-P1 for the above mentioned structure,  
and submit the following comments:

The subexcavation between the abutments should  
be extended at least 5 feet longitudinally outside the  
outside extremes of the proposed abutment footings and  
backfilled to the required grade with suitable granular  
material. Side slopes of 1:1 may be used for the  
subexcavation.

MD/WdeP

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

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

Foundations Files  
Gen. Files



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

Mr. W. Melnyshyn,  
Reg. Bridge Location Engineer,  
Central Region,  
Administration Building

Bridge Division,  
Downsview, Ontario

September 25, 1967

Fifteen Mile Creek Bridge  
W.P. 213-63, site 18-23  
Q.E.W. - South Service Rd.  
District No. 4 - Hamilton

Attached herewith are prints of the Preliminary Bridge Plan  
Drawing B-6103-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$80,000.  
This cost includes tender, materials, engineering and sundry  
construction.

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

CSG:rd

C.S. Gretski,  
Bridge Design Engineer

Attach.

c.c. S. McCombie  
A. Stermac (2)  
R. Forrest  
B. Cross

Copy for the information of

S. McCombie, Bridge Planning Engineer, Admin. Building.

Bridge Division,  
Downsview, Ontario,  
October 5th, 1966.

Mr. A. Barnes,  
Director,  
Department of Energy and  
Resources Management,  
Conservation Authorities Branch,  
Box 358,  
Downsview, Ontario.

RE: The Fifteen Mile Creek Bridge, 66-F-68  
W.P. 213-03,  
Sixteen Mile Creek Bridge,  
W.P. 214-03, Highway Q.E.W., 15-165  
District #4. 66-F-68

Dear Sir:

Our Department is presently engaged in the design of service roads and associated structures and interchanges along the Q.E.W. to achieve a complete control of access.

Service road bridges are to be built on Fifteen Mile Creek and Sixteen Mile Creek offset approximately 135' from the Q.E.W. center-line (see attached plans).

Enclosed please find relevant correspondence from the Hon. R. Welch, M.P.P. for Lincoln. He writes on behalf of farmers and residents who are pressing for some type of permanent structures which will effectively raise and maintain water levels in these drainage basins. Irrigation systems are dependent on this water and it seems reasonable that in conjunction with the construction of the service road structure, weirs could be built to conserve the water.

The effects of raising water levels to higher elevations and the results of backwater, however, can best be determined by your Branch. It is conceivable that for as many people as would support such a project there may be some who will object due to inundation of their property.

-- 2 --

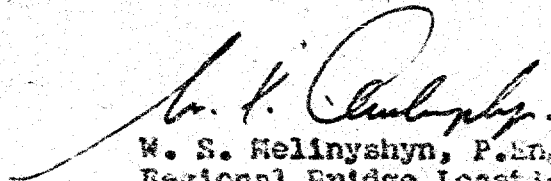
RE: The Fifteen Mile Creek Bridge,  
W.P. 213-53,  
The Sixteen Mile Creek Bridge,  
W.P. 214-53.

Our feeling is that a scheme using weirs, spillways and check dams is feasible. Special design features and certain modifications to our structures will be required if the above are constructed (such as adjustments to size of openings, footing locations, scour protection etc.) Our present schedule calls for design of the bridges immediately to meet the 1908 construction program.

We would appreciate your Office reviewing the proposals as requested by the Hon. R. Welch, M.P.P. and informing our Department of your comments. If feasible, and in order to design our bridges, we would require details as to the desirable water levels which would have to be maintained; your opinion as to who would be responsible for the design, construction and maintenance of the weirs, etc., and your comments regarding contribution towards the cost. No doubt your Branch makes contribution towards projects of this type.

Though not specifically mentioned the same treatment may be required at Eighteen Mile Creek where tentative extensions to the existing twin concrete culverts are contemplated.

Yours very truly,



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

MSK/caw  
Encl.

cc H. Greenland  
C. Hunter  
S. McCombie  
G. Wigle  
J. Harris

# DEPARTMENT OF HIGHWAYS BRIDGE INSPECTION REPORT

Sufficiency Rating..... Index No. Lincoln 9.E.W. 542.3  
 Highway No. Q.E.W. District No. 4..... District) LINCOLN.....  
 Number & Name of Structure... 15 Mile Creek Bridge.....  
 Type of Structure Steel Rigid Frame Dual Bridges.....  
 Inspected By E. Van Nellen..... Date of Inspection April 19 1956  
 Mileage From Jet. Map. 20 and Q.E.W. 23.7 mil.....

Bridge Dimensions Bridge Plan No. D-2538..... (D-2539)  
 Span 20.20.36:6". 40.36:6". 20. Length of Bridge... floor 197!.....  
 Width of Bridge Roadway 20.30! Width of Bridge... 92!.....  
 Clearance..... Date of Construction... 1939.....

## Approach Geometrics

Tangent alignment, gentle uphill grade at each side of bridge.

## Waterway Adequacy

Satisfactory

RETURN TO D.H.O.  
BRIDGE MAINTENANCE  
SECTION

## Condition of Bridge

### Abutments

Concrete bents on 5 piles each with concrete sill on top,  
good condition

### Ballast Walls

Concrete - good condition

### Piers

2 outside piers at each end of bridge consist of concrete piles  
good condition.

2 middle piers - Steel 5 beams - good condition

### Bearings

Steel beams are bearing on sills

### Main Girders, Trusses or Ribs

Good condition

### Floor Beams

None

SUPER IMPOSED DOCUMENT MAY  
APPEAR AS MULTI-FEED ON FILM.

Stringers

None

Deck

Concrete-good condition

Wearing Surface

Good condition

Expansion Joints

Good condition

Handrail Posts

Concrete - good condition  
One post broken on North bridge

Handrails

Steel - good condition

Wingwalls

Concrete - good condition

Curbs

Concrete - fair condition

Sidewalks

Concrete - good condition

Drainage

Catch basins - deterioration of catch basins at bottom

Retaining Walls

None

Embankment Slopes

Good

SUPER IMPOSED DOCUMENT MAY  
APPEAR AS MULTI-FEED ON FILM.

Additional Observations

Other bridge is in same condition

General Condition of Structure

Good

Structural Strength

H - 20

Recommendations

None

Date..April.18th.195 6

Sig.....

Action

Date.....195

Sig.....

Remedial Measures

Plan No. D -

Date Commenced.....

File No.

Date Completed.....

Date.....195

Sig.....

Page 4.

Rough Sketch of Structure or of Part of Structure

66-F-68

Resp. Rep.

CONDITION OF CONCRETE IN STRUCTURES

PROJECT 0300

FIFTEEN MILE CK. BR.  
DISTRICT: No. 4  
HWY. Q.E.W.  
LOG NO. 54E

TYPE: Steel rigid frame R.C.  
Deck Slab  
CONSTRUCTED: 1939  
PURPOSE OF SURVEY: Restoration  
Bridge Maintenance July 17/64  
DATE OF SURVEY: October 8, 1964.

A. PAVEMENT AND DECK SLAB

Asphalt through the full width of roadway wearing surface worn out.

Location of Test Area	Depth of Asphalt	BOND	Condition of Concrete under Asphalt	Core W.C.
A. West Abutment b of roadway	1"	Nil	Total disintegration of top fibres of slab 3/4" deep	
B. West abutment gutter area	1 1/2"	"	As above, 1" deep	172
C. East abutment b of roadway	1 1/4"	"	Concrete sound. No traces of deterioration	
D. East abutment gutter area	1 1/4"	"	Mortar in top fibre of slab disintegrated. Deterioration can be qualified as light scaling	173

1. Concrete Under Asphalt

Depth of deterioration in form of disintegration of mortar in top fibres and separation of C.A. varies



over the deck area and is generally heavier in gutter area.

2. Soffit

Transversal cracks. Seepage through the longitudinal construction joints and cracks. White deposit of calcium origin and ettringites formed. Dark paths indicate quite large extension of wetted area.

3. Exterior Vertical Faces

Limited concentrated cracks and disintegration of concrete in form of surface spalls, located at riveted portion of steel girders. Light seepage through the cracks and light corrosion to top flange of steel members.

4. Bearing Area

Light cracks and seepage.

B. SIDEWALKS AND CURBS

Concrete deteriorated locally in form of surface spalls and exposure of reinforcement on 5% of slab area.

C. HANDRAIL POSTS

Generally sound. Top of one post deteriorated in form of cracks caused probably by freezing water in the pocket formed for installation of light post.

D. PIERS

Cracks, seepage (from suspended M.H.) located.

E. GENERAL CONDITION OF CONCRETE

Concrete in nondeteriorated portion of structure is sound. Recovered cores from deck slab represent well compacted and graded material. Good bond between aggregate and mortar. High value of rebound number indicate concrete of ultimate strength in range of 7,000 psi. This is confirmed by lab compressive test ( $f'_c = 6390$ ).

The sonar investigation of core gave rather lower pulse velocity that can be expected for the quality of concrete and can be attributed to disturbances caused by reinforcing steel.

F. CONCLUSIONS

Light repairs to damaged concrete in the slab exterior vertical faces and sidewalk is advisable.

Consider replacement of damaged concrete in the deck slab and waterproofing if resurfacing decided or

remove the deteriorated concrete and replace the asphalt pavement by concrete wearing surfacc.

J. Wawrzynski,  
Project Engineer.

November 4, 1964.

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

DATE: January 15, 1970

IN REPLY TO

**SUBJECT:** Re: Contract 69-151 - Hamilton District  
Notification of Intent to Claim -  
Antici Construction Company Limited

The Contractor makes the following statement regarding the conditions at Piers A and B of the Fifteen Mile and Sixteen Mile Creek structures on the South Service Road:

"The information provided on the drawings and in the specifications regarding the soil conditions at the above mentioned locations was not an accurate representation of the actual conditions."

we disagree with the above statement for the following reasons:

Under the previous Contract 68-108, the organic material within the area of the structure was excavated, removed and replaced. On the Contract Drawing D-6126-1 of Contract 69-151, this area is shown and it is stated that the organic material is replaced with "Selected backfill material". From the previous Grading Contract (68-108), it can be seen that the "Selected backfill material" was Sand Cushion (Special Provisions, p. 15) - and this is exactly what was there, and what the Contractor (Antici Construction Co.) found.

The Contractor proceeds with his statement as follows:

"The result of this was that the specified unwatering alone was not effective in providing an acceptable surface on which to place the pier footings."

The Department generally never specifies unwatering methods. We provide factual information and our assessment of problems that can be encountered. When excavations in permeable soils have to be carried out below the water table, we naturally

January 15, 1970

Re: Contract 69-151 - Hamilton District - Notification of Intent  
to Claim - Antiel Construction Company Limited .....

point out this fact and indicate the need for an appropriate dewatering scheme. Often we give details of a scheme which we know would be satisfactory, but we never specify it. For the Sixteen Mile Creek structure, for example, we made, in our Report W.J. 66-P-62 dated November 14, 1966, the following statement - (page 8):

"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 excavation to a depth of at least 5 feet greater than the base of the excavation."

A similar statement is to be found in our Report W.J. 66-P-63, for the Fifteen Mile Creek structure.

As can be seen, the above statement is positive regarding the need for dewatering, but is only suggestive regarding the method to be used. Consequently, to state that the ... "Specified unwatering alone was not effective ..." - is wrong.

The Contractor's statement reads further, as follows:

"At locations 1) and 2) the drawings indicated that the areas involved were excavated and backfilled with "Select Backfill Material" under a previous contract. In actual fact, the present soil condition is best described as silty sand to sandy silt in a "quick" condition."

Nowhere in Contract 69-151 is the "Select Backfill Material" defined, and it is therefore impossible for the Contractor to argue that the material presently in place is not the one that should be there. There were three ways by which the Contractor could have found out, ahead of time, what is understood as "Select Backfill Material" if he had been in doubt:

- (1) He could have asked the D.H.O., or
- (2) he could have looked through the Contract Documents of the previous Contract (68-108) under which this operation was completed, or
- (3) he could have gone to the site and taken a few samples of the material in question.

Mr. A. Rutka,  
Materials & Testing Engineer,  
Room 102, Lab. Bldg.

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January 15, 1970

Re: Contract 69-151 - Hamilton District - Notification of Intent  
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In addition to the foregoing, the following should be kept in mind: Since the excavation of the organic material and its replacement was to be carried out predominantly under water, it is desirable to use a granular type of material as backfill. The best material would probably be G.B.C. 'A'; the next, G.B.C. 'B', and next, Sand Cushion. In these two particular cases it was decided to use Sand Cushion.

All the three above mentioned materials would become "quick" or "boil" if subjected to a high-enough hydraulic gradient. Boiling of such materials is not a property, but a condition. If an excavation in such materials is to be carried out below the water table, an appropriate dewatering procedure has to be used. Failure to do this, will result in a "quick" or "boiling" condition.

Since the Contractor states that a "quick" condition was observed, we submit that he was using the wrong dewatering procedure and has nobody to blame but himself.

Regarding the rip-rap at Location 3), we have no knowledge of this; it has nothing to do with our work and, therefore, we have no comments.

AGS/MdeF

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

cc: Mr. A. G. Stermac  
Gen. Files

Mr. T. J. Kovich

## NOTIFICATION OF INTENT TO CLAIM

CHIEF ENGINEER,  
DEPARTMENT OF HIGHWAYS ONTARIO.

Date Dec. 22, 19 69.

Against Contract No. 69-151

District Hamilton

Location Q.E.W. Service Roads

Contractor Antici Construction Co. Ltd.

15 and 16 Mile Creek

Sub-Contractor Moir Construction Co. Ltd.

In accordance with Paragraph 2, Sub-section 104-1 of Section 104 "Control of the Work" of the "General Conditions of the Contract" D.H.O. Form 100, I/ we declare my/our intention to file a claim against the above contract due to the following (Give complete details, attaching separate sheets if necessary.)

Locations Involved

- 1- Fifteen Mile Creek, South Service Road., Piers A and B ✓ 66-E-66 W.P. 213-63
- 2- Sixteen Mile Creek, South Service Road., Piers A and B ✓ 66-E-62 W.P. 214-63
- 3- Sixteen Mile Creek, North Service Road., Pier #7

The information provided on the drawings and in the specifications regarding the soil conditions at the above mentioned locations was not an accurate representation of the actual conditions. The result of this was that the specified unwatering alone was not effective in providing an acceptable excavated surface on which to place the pier footings.

At locations 1) and 2) the drawings indicated that the areas involved were excavated and backfilled with "Select Backfill Material" under a previous contract. In actual fact the present soil condition is best described as silty sand to sandy silt in a "quick" condition.

Compensation received under the contract pay items is substantially inadequate to cover the expenses incurred in constructing the footings concerned.

At location 3), the drawings indicated a 2' thick layer of rip rap across the stream bed and up the slope to a point 4' from the edge of the footing for pier #7. In actual fact, rip rap, approximately 8' thick existed over the entire area of the footing.

Again, payment under contract pay items falls far short of actual costs.

The conditions described are beyond the scope of the present contract and we require additional compensation to cover the cost of performing the work.

NOTE: Contractor must give this notice to the Chief Engineer and District Engineer within 7 days of his date of commencement on the work out of which this claim arises—Refer—Section 104-1 "General Conditions of the Contract" D.H.O. Form #100 Revised April 1st, 1958.

Signed

*Walter L. Lint*  
Contractor or Authorized Representative.

TO BE MADE IN QUADRUPLICATE BY THE CONTRACTOR.  
ONE COPY SENT TO DISTRICT ENGINEER—TWO COPIES SENT TO CHIEF ENGINEER.