



ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye, **Date** November 10, 1960.
Bridge Engineer, **Subject** FOUNDATION INVESTIGATION REPORT
From Materials & Research Section. **By:** Wm. A. Trow & Associates, Inc.

Attention: Mr. S. McCombie.

Re: C.N.R. Overhead and Hwy. 403 Underpass
Lemonville Rd., Aldershot, Ont., Dist. 4.
W.P. 197-58.

We have reviewed the above mentioned report and on the basis of the presented factual data and calculations carried out, agree with the conclusions contained in the report.

The calculation of the stability of the cutting slopes was carried out under the supposition of a certain ground water table. It seems to us that even more unfavourable ground water conditions can occur (after long rainfalls and during the spring thaw season) and a 2-1/4:1 slope, which is similar to the existing natural ground slope, is the logical recommendation.

Should there be any queries in connection with the above project that you would like to discuss, please feel free to call on our Office.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.
Per:

(A. G. Stermac,
FOUNDATIONS OFFICE ENGR.)

AGS/MdeF

Attach.

cc: Messrs. A. M. Toye (2)

H. A. Tregaskes

D. G. Ramsay

I. C. Campbell

R. E. Richardson

T. J. Kovich

A. Watt

Foundations Office

Gen. Files.

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

BA 1152

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J 582

October 27, 1960

60-F-281C

Mr. A. Rutka,
Acting Materials and Research Engineer,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ontario

Attention: Mr. L.G. Goderman, P.Eng.
Principal Soils and Foundation Engineer

Foundation Conditions - C.N.R. Overhead and Hwy. 403 Underpass
Lemonville Rd., Aldershot Ont. W.P. 197 - 58

Dear Sirs:

We have completed the investigation of foundation conditions at the above-noted bridge sites and our report on the subject is enclosed. It has been prepared by G. Wheeler, P.Eng. who supervised the field and laboratory testing programs.

No foundation problem exists at the relocation site of the C.N.R. Overhead. Queenston shale bedrock lies very close to the surface here and the ground is quite dry and impermeable. The existing bridge structure is in good condition.

A similar situation will exist at the Hwy. 403 Underpass location after excavation to final grade has been completed. All footings except the north abutment will be down to bedrock. Hwy. 403 cuts into the Lake Iroquois shoreline at this location and the maximum depth of the cutting will be in the order of 40 feet. The bottom approximately 12 feet of soil consists of silty clay till which exists in a medium stiff to stiff condition. The north abutment will bear on this material unless support on end-bearing piles is provided. This latter alternative is recommended. Piles about 15 feet long will be required.

In order to ensure that the cutting is permanently stable with a factor of safety of 1.3 against failure this north cutting should be made at $2\frac{1}{4}$ to 1. The existing hillside falls approximately at this slope.

We shall be pleased to discuss any matters that may occur to you after you have reviewed the contents of this report.

Yours very truly,

W. Trow

William A. Trow, P.Eng.

WAT/gc
Encls.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH SECTION
PARLIAMENT BUILDINGS, TORONTO, ONT.

FOUNDATION INVESTIGATION
HWY. 403, C.N.R. CROSSINGS ON LEXINGTON RD.,
ALDERSHOT, ONTARIO
W.P. 197 - 58

Project J 582

William A. Trow and Associates Ltd.

Oct. 27, 1960

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FOUNDATION INVESTIGATION
HWY. 403, C.N.R. CROSSINGS ON LEMONVILLE RD.,
ALDERSHOT, ONTARIO
W.P. 197 - 58

This report contains the results of foundation investigations completed recently for proposed underpass and overhead structures on a revised section of Lemonville Road, near Aldershot, Ontario. The location of the investigated area is shown on Dwg. No. 1.

The project has been divided into two parts. Section A deals with the County Road crossing of Highway No. 403, while Section B covers the foundation investigation for the overhead bridge site at the Canadian National Railway line immediately to the south. The two structures will be connected by an embankment 100 feet long and up to 25 feet high.

Recommendations are made concerning safe bearing values and suitable foundation depths of spread footings for each structure. The stability of approach embankments and cuts are considered in detail.

Field investigation methods at both sites are discussed in an Appendix.

SECTION AFOUNDATION INVESTIGATION
LEMONVILLE ROAD - HIGHWAY NO. 403 UNDERPASSSite Description

The site for the proposed County Road crossing over Hwy. 403 is located some 100 feet west of the existing Lemonville Road. The intersection of the two roads lies near the crest of a steep hill. From this point the ground falls to the south-east uniformly, on a grade of approximately $2\frac{1}{2}:1$, to a level field which lies on the north side of the railway. The elevation of this flat area is about 353 feet, and that of the intersection point is 382 feet, approximately. The ground rises at a much flatter gradient along the route of Hwy. 403, until it reaches a tableland elevation of 402 feet about 100 feet to the west of the proposed underpass structure.

At the time of the investigation, water was seeping along a ditch beside the existing Lemonville Road. This water is believed to originate from the water-bearing sand stratum encountered during the investigation immediately to the west of this road. No springs were noted along the south-eastern slope of this high ground.

Soil Types Encountered

A detailed description of the various soil types encountered at this site, is presented on the three borehole logs, Dwgs. Nos. 4-6. The estimated stratigraphical profile on Dwg. No.2 has been prepared on the basis of this information.

Borehole No. 1 was located at the base of the steep hill and has a similar soil profile to that encountered in the borings at the Railway overhead site. Beneath one foot of topsoil lies approximately 7 feet of hard dry red to dark brown silty clay, which has been derived from the underlying Queenston shale. The latter material was intersected at elevation 346.2 ft. A ten foot BX core, taken at this location, gave recoveries of 63.5% for the first 5 feet, and 100% for the lower 5 feet.

The remaining two boreholes were located on the crest of the hill. Immediately below the topsoil, a loose to medium dense brown silty fine sand was intersected. This material extends to a depth of 10 to 11 feet, or between Elev. 375.4 and 371.5 feet. Penetration resistances in this material ranged between 8 and 31 blows per foot. The stable water table was established at approximately 4 feet above the lower interface of this material. Water flowing in the ditch parallel to Lemonville Road originates from this source.

Below the sand lies a medium stiff to stiff silty clay with some sand and fine gravel sizes. Fragments of shale were noted in the material below 22 feet in borehole 3. This clay extends to bedrock which was intersected at Elev. 357.0 and 360.4 feet in boreholes 2 and 3 respectively. The clay has plastic and liquid limits of the order of 16% and 31%, and a moisture

content which decreases with depth from approximately 27.5% to 19.5%. Close agreement between field vane and triaxial compression tests was obtained. The shear strength increases linearly with depth from 700 to 1500 p.s.f.

Bedrock was proven in Hole 3 by drilling 13 feet into rock. Refusal to driving of the split spoon sampler has been assumed to indicate bedrock in borehole 2. Pieces of Queenston shale were recovered.

Discussion on Foundation Conditions

(1) Bearing Capacity

Spread footings for the southern abutment and the three central piers of the overpass structure will bear directly on the shale bedrock. Bedrock should be intersected at approximate Elev. 346 feet adjacent to the southern abutment, and about Elev. 360 feet at the northern pier. The safe bearing value to apply in this instance is 25 tons per square foot.

Support for the northern abutment may be provided by short piles to bedrock, which is at approximate Elev. 360 feet. Alternatively, spread footings bearing on the medium stiff silty clay could be utilized. The safe net bearing value in this case is of the order of 1800 p.s.f. Short pile foundations are preferable since they eliminate the possibility of differential movements between this abutment and the remainder of the bridge structure.

(2) Stability of Embankments and Cuttings

No embankment stability problem exists for the approaches to the bridge. The southern embankment fill will be approximately 25 feet high, but it will bear on the extremely competent hard red silty clay. The northern embankment will be only 2 to 3 feet high and will cause only a negligible increase in stress in the underlying soil.

A less satisfactory condition exists along the cutting which will be made by Hwy. 403 into the hillside at and to the west of the Lemonville Road crossing. This cutting has a maximum depth of about 40 feet and the bottom 12 feet is in clay having a shear strength ranging from 500 to 1500 p.s.f.

An analysis of this highway cutting was made to examine both its short and long term stability. The results of this analysis are shown on Dwg. 13 and 14, and typical calculations and assumptions are presented in Appendix 2. The minimum factor of safety for the short term, after-construction case, was found to be $F = 1.27$. This value applies for a slope of 2 horizontal to 1 vertical.

A more critical condition was found for the long term stability. The minimum factor of safety in this circumstance, assuming slopes of 2:1, was $F = 1.20$. If the slopes are cut to $2\frac{1}{2}:1$, the minimum factor of safety increases to $F = 1.43$. According to the contours of the existing hillside, the present slopes have stabilized close to a value of $2\frac{1}{2}$ to 1.

In both analyses, a water table perched 4 feet above the surface of the clay was assumed. The level of this water can be expected to vary slightly depending upon the amount of rainfall over the area. This change in level will not affect the stability computations since the soil was assumed to be saturated up to Elev. 402 feet. Some minor sloughing of the sand may be experienced at its point of outcrop just above the clay layer, but this can be controlled by plant growth. As stated in an earlier section, there was no evidence of springs or unstable conditions on the existing slopes.

SECTION BFOUNDATION INVESTIGATION
LEMONVILLE RD. - C.N.R. OVERHEADSite Description

The ground surface in the immediate vicinity of the proposed County Road Overhead falls gently towards the south-east. The double tracks of the Canadian National Railway run from south-west to north-east, and lie in a cutting about 10 feet below the general ground elevation of 350 feet. Some 200 feet north-west of the site, the ground rises steeply to an elevation of 400 feet. The adjacent bridge, which crosses the railway line, appears to be in good condition.

Soil Types Encountered

Four borings were put down at this site in the locations shown on Dwg. 3. Detailed descriptions of the soil types encountered are given on the borehole profiles, Dwg. 7 to 10. These logs show that uniform conditions exist at each test location.

Below approximately 1 foot of topsoil lies a hard dry dark brown to red silty clay. This stratum has been derived from the underlying Queenston Shale in geologically recent times. This material extends to elevations ranging between approximately 344 and 341 feet, at which depths sound bedrock was encountered. For 1 to 2 feet above bedrock, the clay material contains thin bands of unweathered shale.

Bedrock was proven in two test locations, namely boreholes 4 and 7, by drilling 10 feet into it. The percentage of shale recovered from each core is shown on the borehole logs.

An inspection of the holes, several days after drilling had been completed, showed that each hole was dry to full depth. The elevation of ground surface adjacent to each hole has been related to a D.H.O. Bench Mark on Lemonville Road, at the location shown on Dwg. 2.

Discussion of Foundation Requirements

The foundation conditions at the Lemonville Road crossing of the C.N.R. are considered to be excellent. Support for abutments and piers may be provided by spread footings bearing directly on the Queenstone Shale, at approximate Elev. 342 feet on the north side of the railway tracks, and Elev. 340 feet on the south side. The maximum safe bearing value which may be applied to the footings at this depth, is of the order of 25 tons per square foot.

No embankment stability problem exists at this site because of the competent nature of the underlying soil and rock.

The walls of footing excavations in the red silty clay material should stand unsupported during the construction period. No ground water problems are anticipated.

ConclusionsSection A - Lemonville Rd., Highway No. 403 Underpass

1) The foundation investigation revealed that most of the site is underlain by loose to medium dense silty sand and then by medium stiff to stiff silty clay deposits. As a considerable proportion of this material will be removed during construction of the new highway, support for the southern abutment and the three centre piers will be provided by the Queenston shale underlying these strata. The safe bearing value to apply for spread footings bearing directly on bedrock is 25 tons per sq. ft. The northern abutment should be supported by piles driven to bedrock if differential settlement is to be avoided. If about 1 inch of differential settlement can be tolerated this abutment can be supported on the upper levels of the clay stratum. A safe net bearing value of 1800 psf has been recommended.

2) On the basis of short and long term stability analyses of the cutting to be formed by the construction of Highway No. 403 beneath Lemonville Rd., a side slope of about $2\frac{1}{4}$ to 1 should be provided in order to ensure a factor of safety of 1.3.

3) No stability or settlement problems will arise following the placement of embankment fills on the approaches to the bridge structure.

Section B - Lemonville Rd., C.N.R. Overhead

1) Foundation conditions are excellent for the overhead structure at this site. Bedrock is found 5 to 10 feet below present ground surface, and the overlying material consists of a hard red silty clay.

2) The safe bearing value to apply for spread footings founded on the shale bedrock is 25 tons per square ft.

3) No embankment stability, settlement or ground water problems are anticipated.

GHW/gc
Oct. 27, 1960
J 582



G. H. Wheeler
G.H. Wheeler, P.Eng.

APPENDIX 1FIELD INVESTIGATION METHODS

A total of 7 borings were put down in the investigation of foundation conditions for these structures. Three borings were made at the site of the Lemonville Rd. - Hwy. No.403 Underpass, using continuous flight auger equipment. Holes were uncased and approximately 5 inches in diameter. The remaining 4 borings were put down at the proposed C.N.R. crossing, using Department of Highways wash bore equipment. The locations of the boreholes are shown on Dwg's. Nos. 2 and 3.

All holes were drilled to refusal either to augering or to washing, depending on the equipment used. To prove bedrock, 10-foot rock core samples were taken from two of the borings at each site, namely boreholes Nos. 1, 3, 4 and 7.

Disturbed samples of the hard red silty clay and loose sand materials were recovered using a conventional 2-inch O.D. split spoon. This sampler was driven into the ground under an energy of 350 ft.lbs.per blow. The number of blows for the last foot of penetration has been recorded on the borehole logs, Dwg's. No.4 to 10. Samples were classified and sealed in polythene bags, a small sample of the hard silty clay material being first wrapped in aluminum foil for moisture content determination.

Undisturbed samples of the medium stiff silty clay were obtained using a fixed piston sampler. The ends of the shelby tubes were wrapped in aluminum foil and waxed to prevent loss of moisture.

Standard cone penetration tests were performed adjacent to boreholes No.2 and 3. The results of these tests are included on the borehole logs.

Water level observations were taken in each boring to determine the elevation of the water table. Holes 1 and 4 to 7 were found to be dry to full depth, during the observation period of several days. All holes were backfilled at the completion of the investigation.

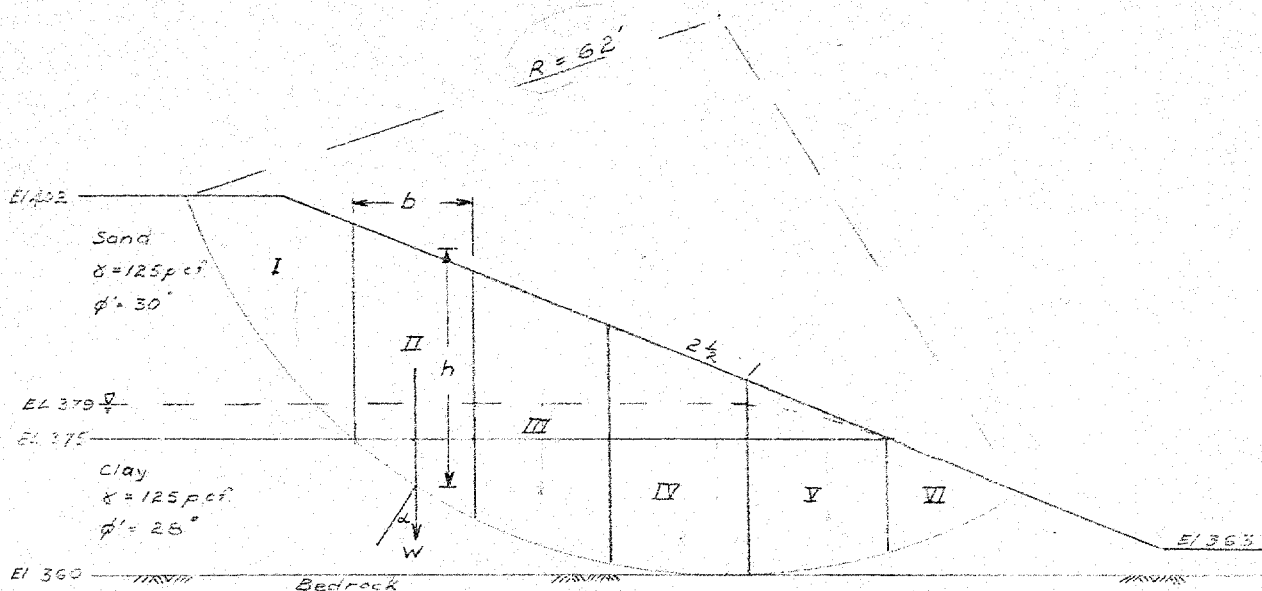
Ground elevation at each boring has been related to datum utilizing a bench mark on Lemonville Road, for which an elevation of 380.29 feet has been given.

APPENDIX 11TYPICAL CALCULATIONS FOR STABILITY ANALYSISCutting of Highway 403 into HillsideAssumed Conditions

Top of Slope	= Elev. 402 feet
Base of Slope	= Elev. 363 "
Top of clay	= Elev. 375
Bottom of clay- Bedrock	= Elev. 360
Water table	= Elev. 379
Properties of Sand:	
Unit weight	= 125 p.c.f.
Apparent and effective angle of friction	= 30°
Properties of Clay:	
Unit weight	= 125 p.c.f.
(a) Short term analysis- Apparent cohesion	= 500 p.s.f. at Elev. 375 ft., increasing linearly to 1500 p.s.f. at Elev. 360 ft.
(b) Long term analysis- Effective cohesion	= 0
Effective angle of friction	= 28°

(1) Long Term Stability ~ side slope of cutting = $2\frac{1}{2} : 1$.

$$\text{Factor of Safety given by: } F = \frac{1}{W \sin \alpha} \left((W - bu) \tan \phi' \left(\frac{\sec \alpha}{\tan \phi' \tan \alpha} \right) \right) *$$



* - The Use of the Slip Circle in the Stability Analysis of Slopes -
A. W. Bishop - Geotechnique, March 1955.

Pore Pressure u assumed to be static water pressure above element of failure area.

(9) x	(12)
F=1.43	F=1.5
20900	21200
19100	19300
18700	18800
15090	15100
10490	10470
4070	4040
88350	88910

$$F_1 = \frac{88350}{61950}$$

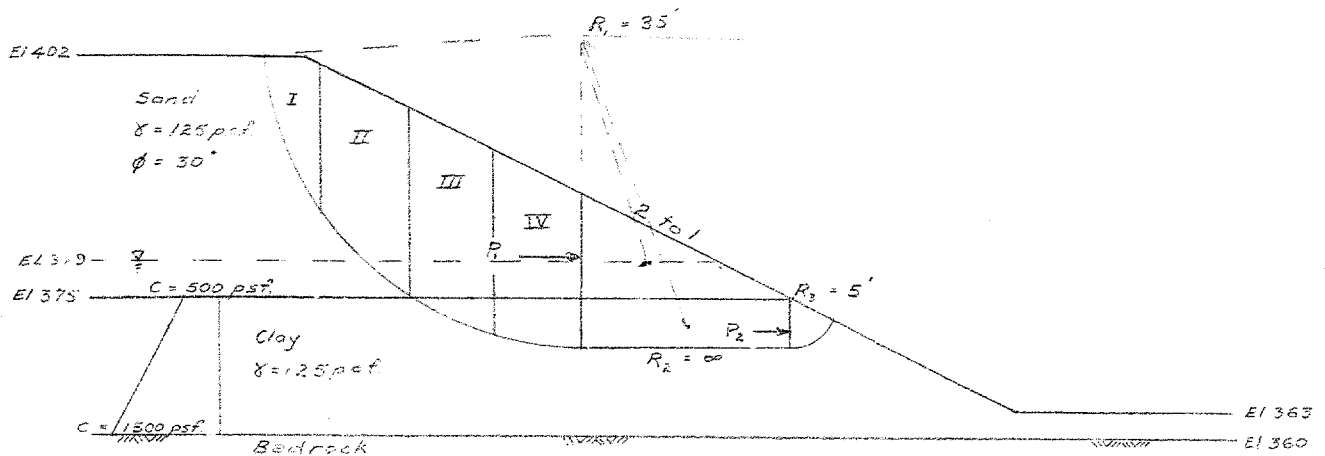
$$F = \frac{88910}{61950}$$

$$F = 1.43$$

$$F = 1.44$$

Therefore: Factor of Safety = 1.43

(2) Short Term Stability - side slope 2 : 1



APPENDIX 11 - Cont.

Circle 1 - Radius 35 feet

Disturbing MomentSection

1	$\frac{1}{2} \times 6 \times 15 \times 125 \times 32$	= 180,000 ft.lbs.
2	$19.5 \times 10 \times 125 \times 25$	= 610,000
3	$21.5 \times 10 \times 125 \times 15$	= 404,000
4	$20 \times 10 \times 125 \times 5$	= 125,000
		<hr/>
		1,319,000 ft. lbs.

Resisting MomentSection

1	Weight = 5620 normal component = 2280 lbs.
2	" = 24400 " " = <u>17180</u>

$$\text{Moment in sand} = \frac{19460 \tan 30^\circ}{F} \times 35 = \frac{\text{Total} = 19460}{F} \times 35 = \frac{394,000 \text{ ft. lbs.}}{F}$$

$$\text{Moment in clay} = \frac{(11.5 \times 600 + 10.5 \times 800) 35}{F} = \frac{535000}{F} \text{ ft. lbs.}$$

$$P_1 = \frac{1,319,000 - \frac{1}{F}(394,000 + 535,000)}{29.33}$$

$$P_1 = 45000 - \frac{31600}{F}$$

.....(1)

APPENDIX 11 - Cont.Circle 2 - $R_2 = \infty$

$$\text{Resisting force} = \frac{800}{F} \times 24$$

$$P_2 = P_1 - \frac{19200}{F} \quad \dots\dots\dots(2)$$

Circle 3 - $R_3 = 5 \text{ ft.}$

$$\text{Resisting moment due to weight} = 15 \times 125 \times 2 = 3750 \text{ ft. lbs.}$$

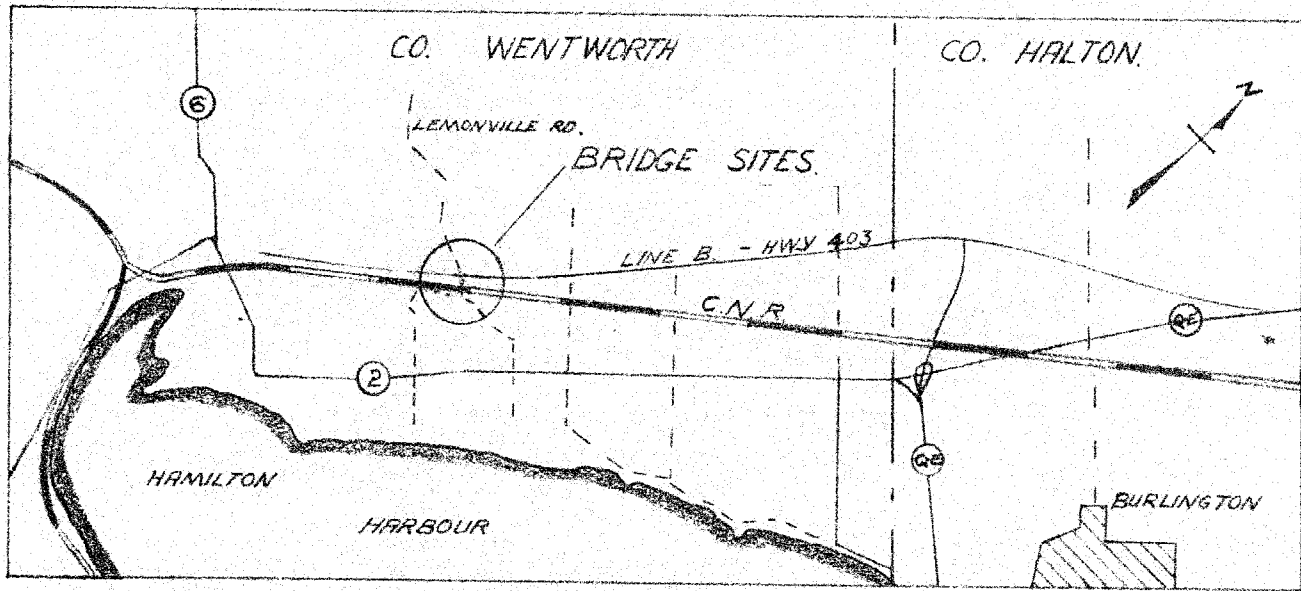
$$\text{" " " Cohesion} = 5 \times \frac{700}{F} \times 5 = \frac{17500}{F} \text{ ft. lbs.}$$

Equating moments

$$P_2 = \frac{3750 + \frac{17500}{F}}{3.33}$$

$$= 1130 + \frac{5200}{F} \quad \dots\dots\dots(3)$$

Elimination of P_1 and P_2 from equations 1, 2 and 3 gives $F = 1.27$



LOCALITY PLAN

Scale : 1" = 1 mile.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 4
PROJECT No. J582

PENETRATION RESISTANCE

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) ⊕

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT

PLASTIC LIMIT

SAMPLE TYPE

2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3 O.D. SHELBY TUBE

2" I.D. SHELBY TUBE

3 O.D. SHELBY TUB

BOREHOLE No. 1

PROJECT Lemonville Rd. - Highway 403 Underpass

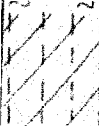

LOCATION Aldershot, Ont.

HOLE LOCATION See Dwg. 2

NOLE ELEVATION 353.9 ft.

DATUM D.H.O. Benchmark = El 380.29 ft.

See Dwg. 2 for location.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40				
	Ground surface	353.9	0						
	1 ft. topsoil then red to dark brown hard silty clay - very weathered Queenston shale	346.2						1	
	Red Queenston shale							2	
	BX rock core drilled from 8'-2"-18'-2"		10					3	
	8'-2"-13'-2" Recovery = 63.5%								
	13'-2"-18'-2" Recovery = 100%								
	End of Hole	335.7	20						
<p>Notes: 1) Bore put down with continuous flight auger. Hole uncased.</p> <p>2) Hole dry at completion of drilling.</p>									
			30						
			40						

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
 2" I.D. SHELBY TUBE —x—x—x—x—
 2" DIA. CONE —————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
 UNCONFINED COMPRESSION ⊗
 VANE TEST AND SENSITIVITY (S) +^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX LI
 X

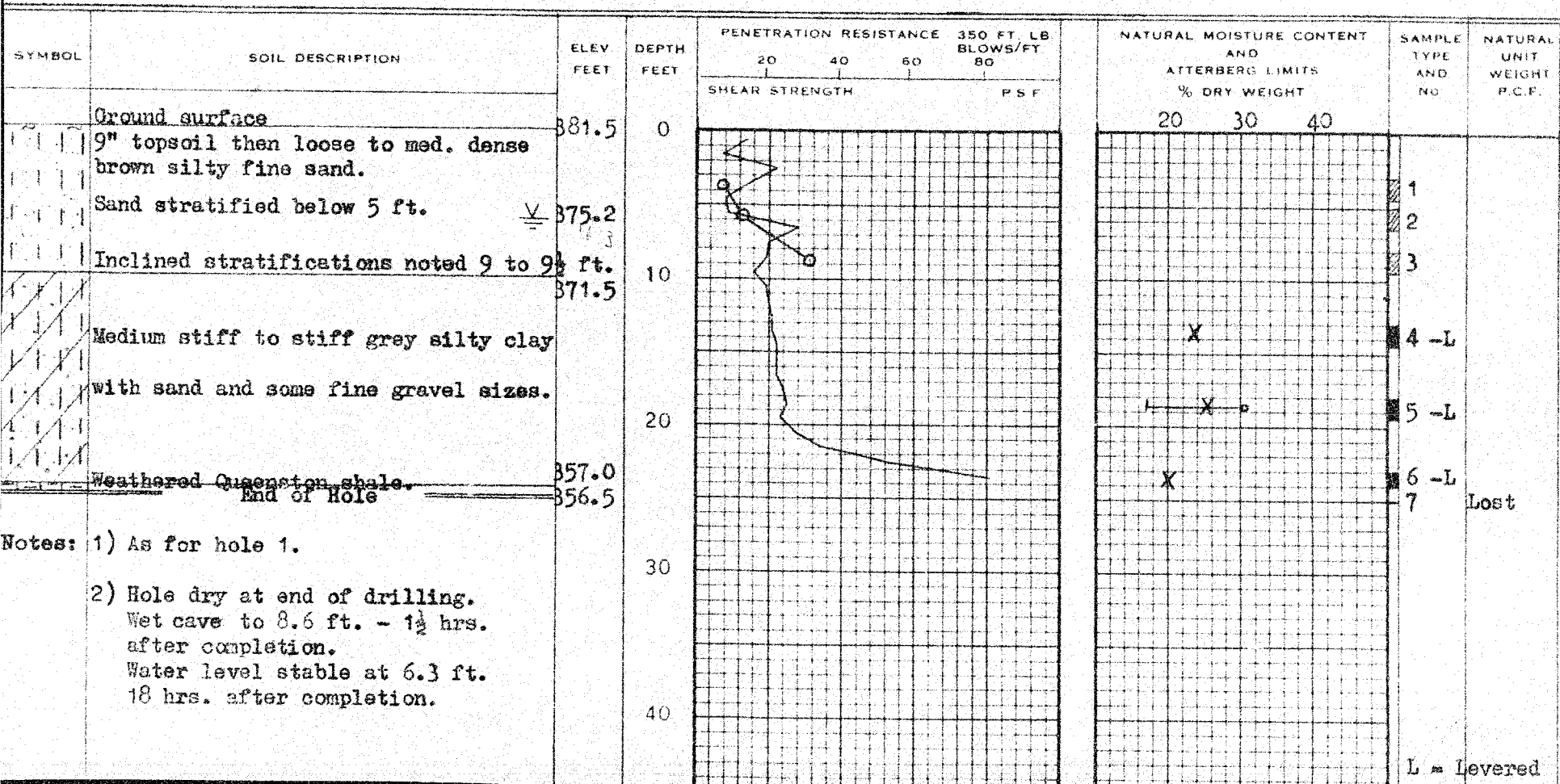
ATTERBERG LIMITS

LIQUID LIMIT —○—
 PLASTIC LIMIT ———

SAMPLE TYPE

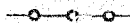

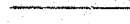
2" O.D. SPLIT TUBE □
 2" I.D. SHELBY TUBE ■
 3" O.D. SHELBY TUBE ▨

BOREHOLE No 2
 PROJECT Lennoxville Rd., Highway 403, Underpass
 LOCATION Aldershot, Ont.
 HOLE LOCATION See Dwg. 2
 HOLE ELEVATION 381.5 ft.
 DATUM As for hole 1






LEGEND

PENETRATION RESISTANCE



2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
 UNCONFINED COMPRESSION 
 VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX

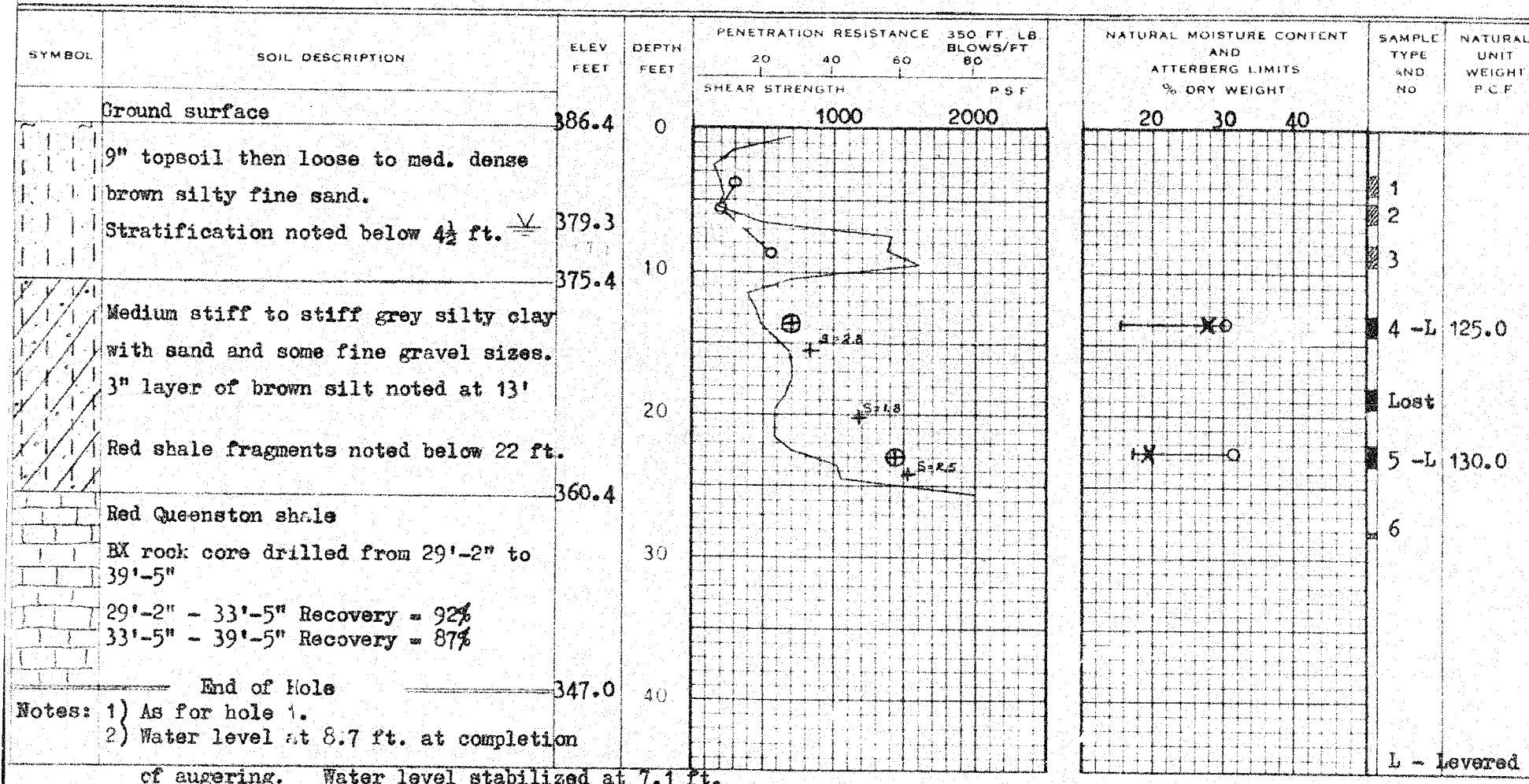
ATTERBERG LIMITS

LIQUID LIMIT PLASTIC LIMIT 

SAMPLE TYPE

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 3" O.D. SHELBY TUBE 




BOREHOLE NO. 3
 PROJECT Lemonville Rd. - Highway 403 Underpass
 LOCATION Aldershot, Ont.
 HOLE LOCATION See Dwg. 2
 HOLE ELEVATION 386.4 ft.
 DATUM As for Hole 1






LEGEND

BOREHOLE No. 4
 PROJECT Lemonville Rd. - C.N.R. Overhead
 LOCATION Aldershot, Ont.
 HOLE LOCATION See Dwg. 3
 HOLE ELEVATION 350.7 ft.
 DATUM D.H.O. Benchmark = El 380.29 ft.
See Dwg. 2

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL
 AT OVERBURDEN PRESSURE 
 UNCONFINED COMPRESSION 
 VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT
 AND LIQUIDITY INDEX

LI
 X

ATTERBERG LIMITS




LIQUID LIMIT

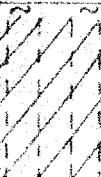
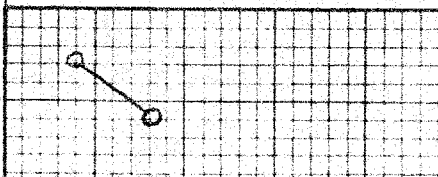

—○

PLASTIC LIMIT

—

SAMPLE TYPE

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 3" O.D. SHELBY TUBE 

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE				350 FT. LB. BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40	60					
	Ground surface			SHEAR STRENGTH				P S F			
	12 ft. black topsoil then red. to dark brown silty clay - very weathered shale.	350.7	0							1	
	Red Queenston shale	340.7	10							2	
	Rock core taken from 10'-2½" to 20 ft. 10'-2½" - 15'-0" Recovery = 52% 15'-0" - 20'-0" Recovery = 92%									3	
	End of Hole	330.7	20								
Notes:	1) Bore put down with conventional diamond drill equipment. Casing 3" O.D.										
	2) Hole dry to bottom at 10 ft. 5 days after completion.										
			30								
			40								

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

2" DIA. CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE

UNCONFINED COMPRESSION

VANE TEST AND SENSITIVITY (S) +

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT

PLASTIC LIMIT

SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE

X²

100

SAMPLE TYPE AND NO.	DATE	TIME	LOCATION	REMARKS

NATURAL
UNIT
WEIGHT
P.C.F.

BOREHOLE No. 5



PROJECT Lemonville Rd. - C.N.R. Overhead

LOCATION Aldershot, Ont.



HOLE LOCATION. See Dwg. 3

HOLE ELEVATION 348.3 ft.

DATUM AS FOR Hole 1

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB. BLOWS/FT. 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40				
	Ground surface	348.3	0						
	12" topsoil then red hard silty clay -very weathered shale.	344.3						1	
	Red weathered Queenston shale	342.6						2	
	End of Hole								
Notes: 1) As for hole 1.									
2) Hole dry to bottom at 5 ft. 4 days after completion.									
3) Refusal to driving of split spoon at 5' - 9" .									

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) ⊕

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT _____
PLASTIC LIMIT _____

SAMPLE TYPE

2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

BOREHOLE No. 6

PROJECT Lemonville Rd. - C.N.R. Overhead

LOCATION Aldershot, Ont.




HOLE LOCATION See Dwg. 3

HOLE ELEVATION 350.5 ft.

DATUM AS for Hole 1.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
				20	40				
	Ground surface	350.5	0						
	12" sandy topsoil then red to dark brown hard silty clays with grey patches - very weathered shale.	346.0						1	
	Grey weathered Queenston shale with red bands	343.9				1279		2	
Notes: 1) As for hole 1.				10					
2) Hole dry to bottom at 6 ft. 4 days after completion.									
3) Refusal to driving of casing at 6'-7" .				20					
			30						
			40						

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) ⊕

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX



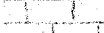

ATTERBERG LIMITS

LIQUID LIMIT _____
PLASTIC LIMIT _____

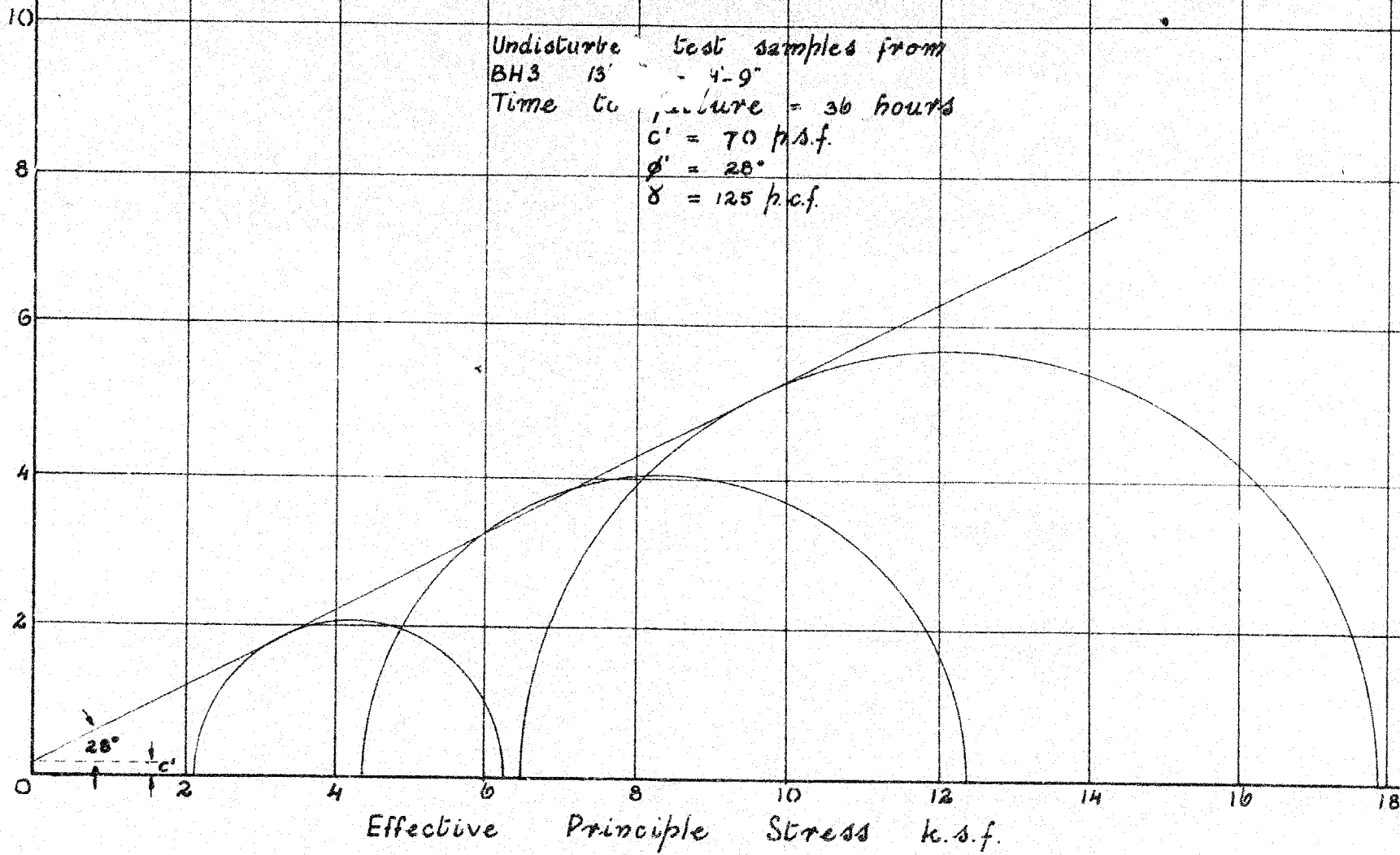
SAMPLE TYPE

2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

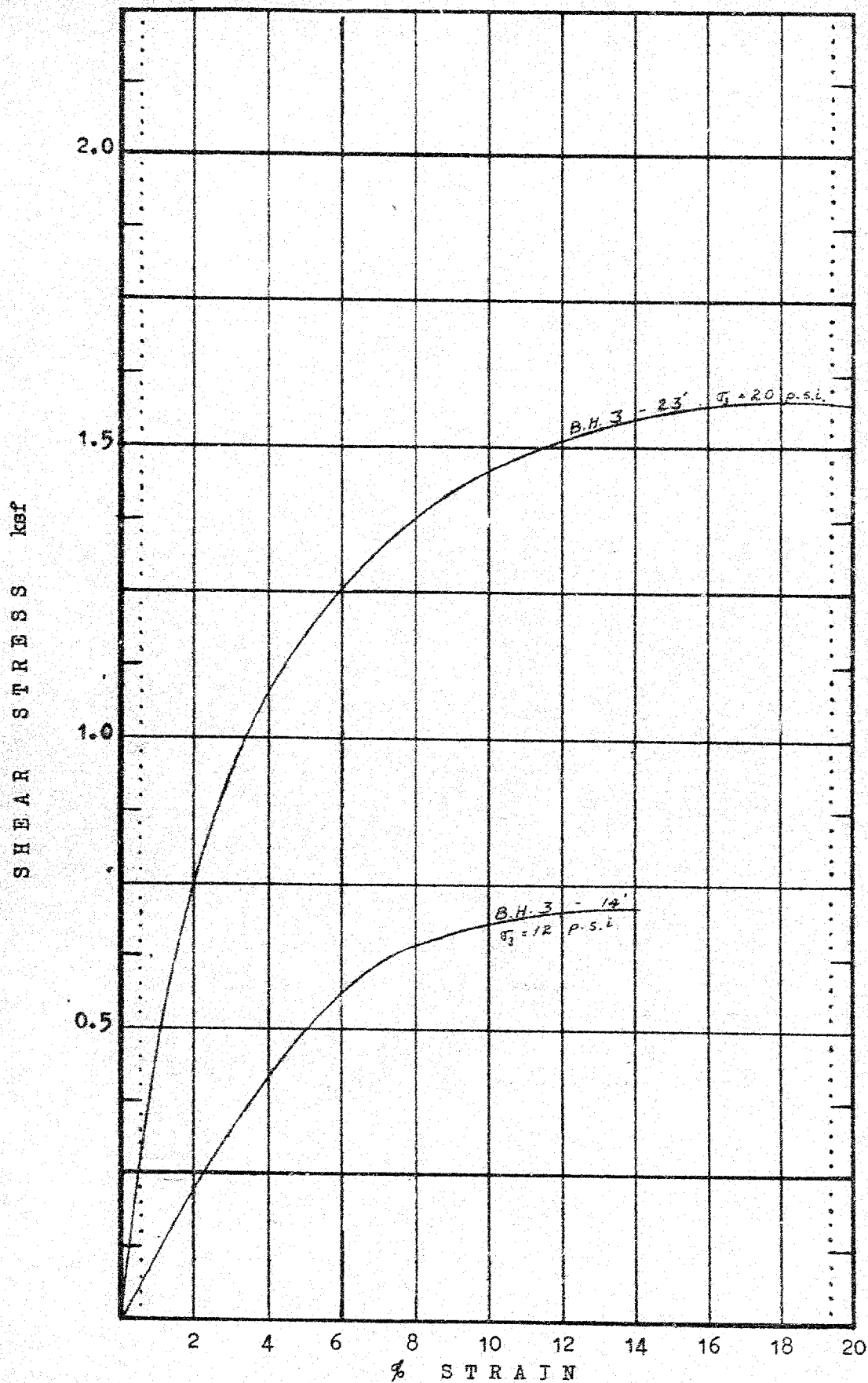
BOREHOLE NO. 7
PROJECT Lemonville Rd. - C.N.R. Overhead
LOCATION Aldershot, Ont.
HOLE LOCATION See Dwg. 3
HOLE ELEVATION 349.2 ft.
DATUM As for Hole 1.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB. BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40				
	Ground surface	349.2	0						
	12" sandy topsoil then red to dark brown silty clay - very weathered shale.	343.7						1	
	Red Queenston shale.					SC FOR 6"		2	
	Rock core drilled from 6'-0" - 16'-0"		10						
	6'-0" - 11'-0" Recovery = 80%								
	11'-0" - 16'-0" Recovery = 85%								
	End of Hole	333.2							
Notes: 1) As for hole 1.				20					
2) Hole dry to bottom at 6 ft. 22 hours after completion.									
				30					
				40					

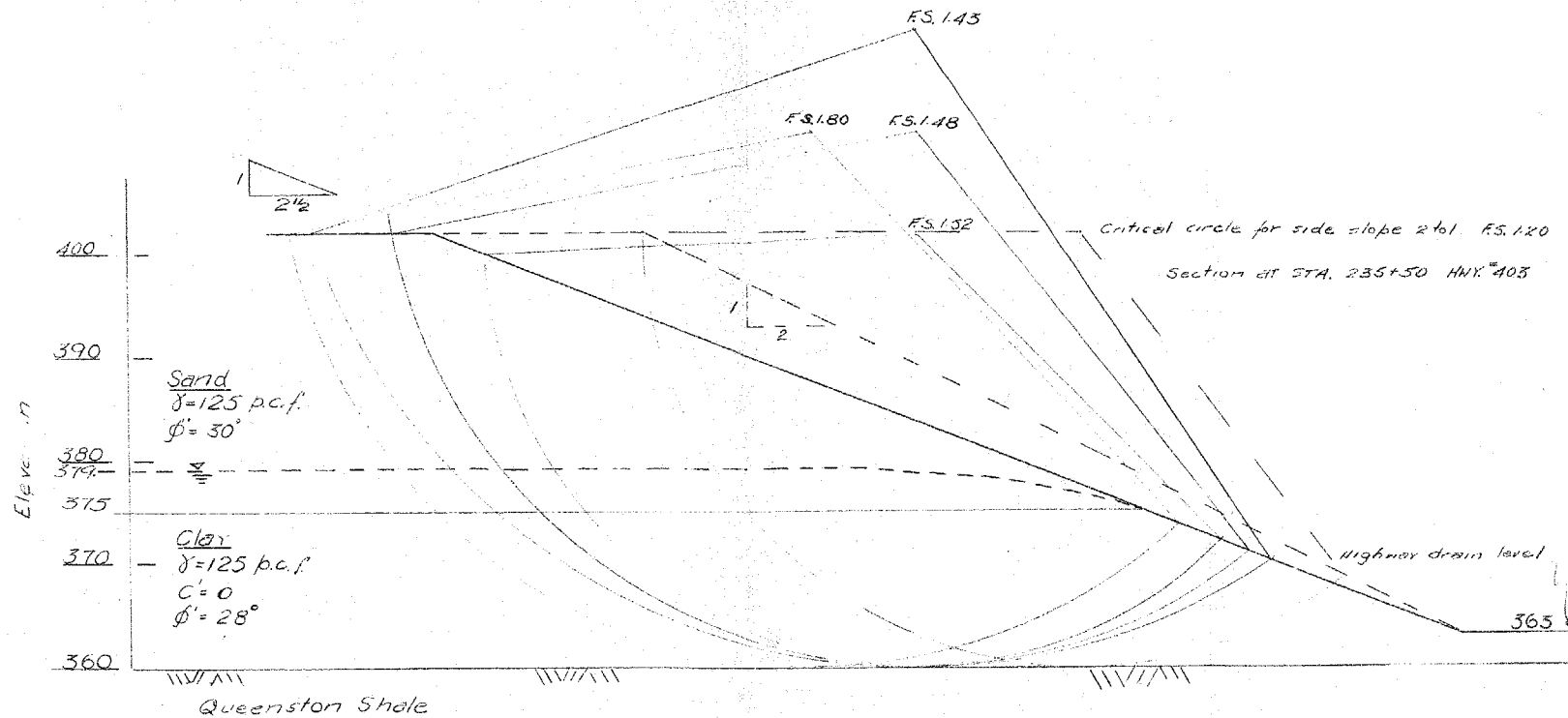
Shear Strength k.s.f.



CONSOLIDATED DRAINED TEST ON MEDIUM STIFF SILTY CLAY



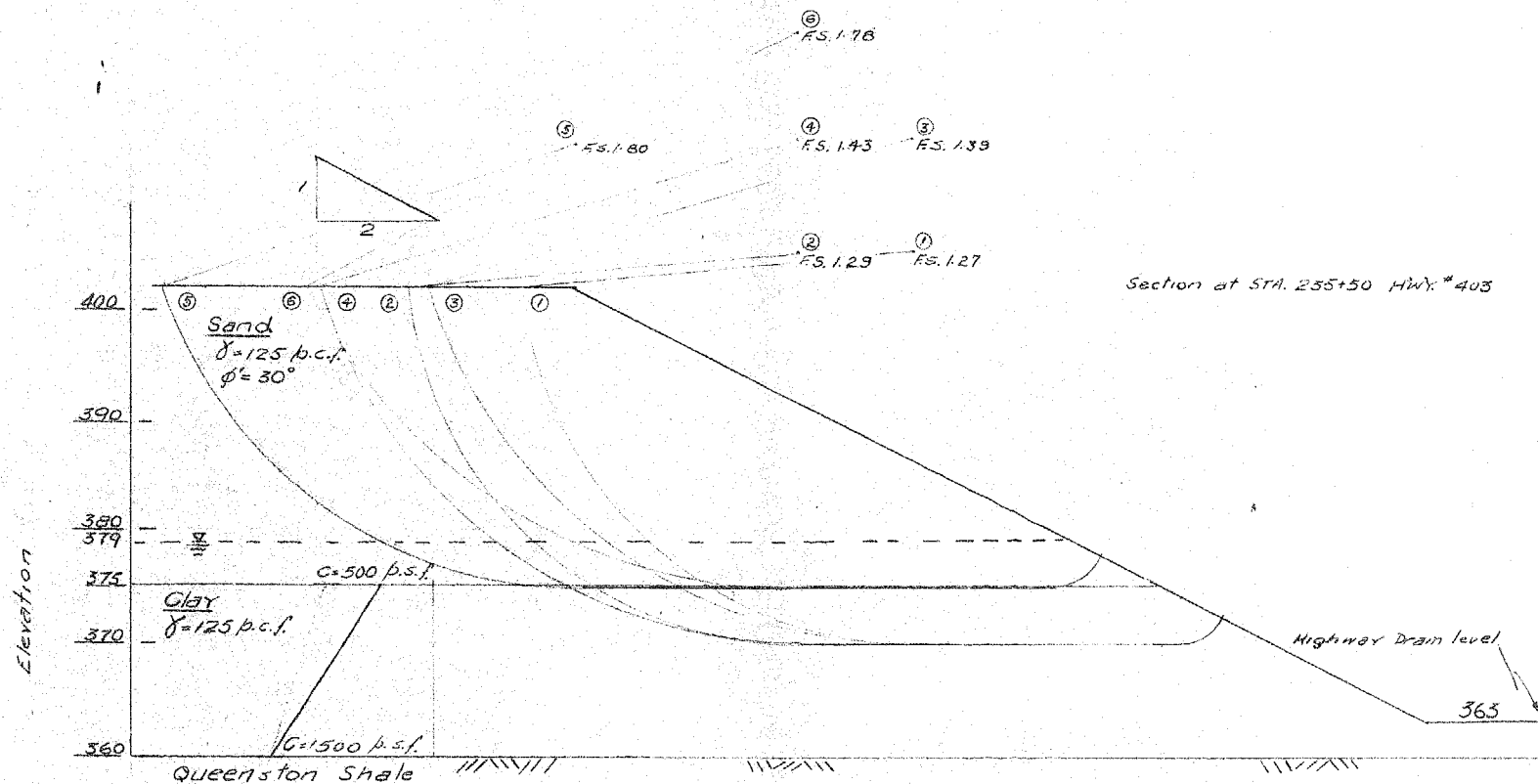
STRESS STRAIN CURVES UNDRAINED TRIAXIAL TEST RESULTS



RESULTS OF LONG TERM SLIP CIRCLE STABILITY ANALYSIS
 PROPOSED CUTTING HIGHWAY N° 403

J582

Drawing #13

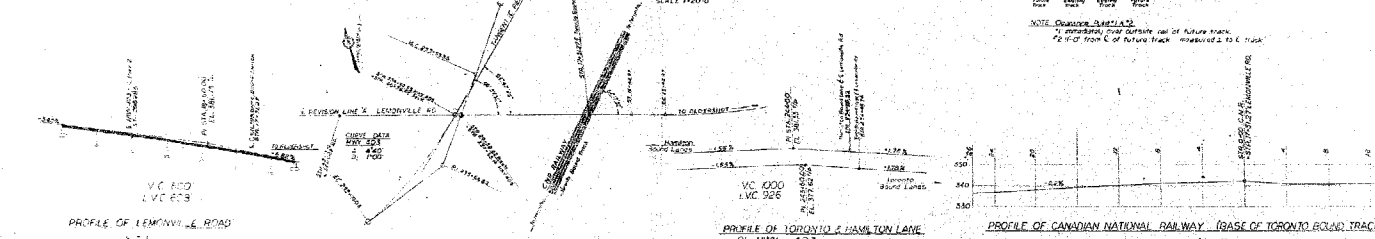
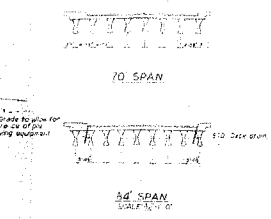
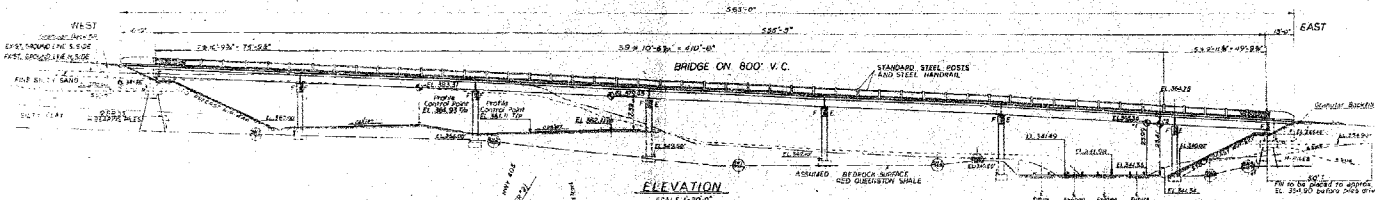
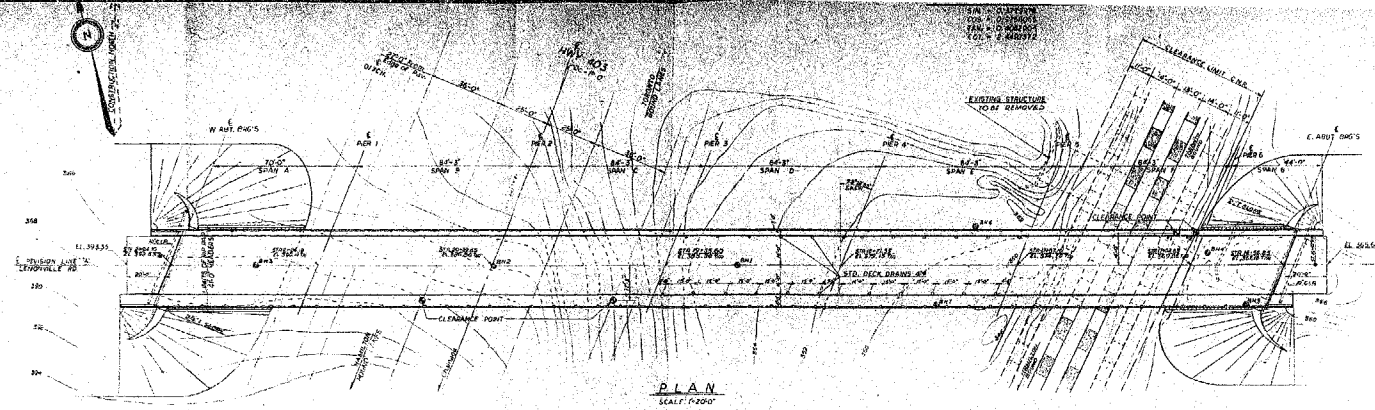
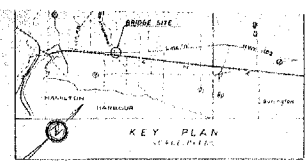


RESULTS OF SHORT TERM SLIP CIRCLE STABILITY ANALYSIS
 PROPOSED CUTTING HIGHWAY N^o 403

J582

Drawing #14

5.00' CLEARANCE
OVER EXISTING
RAILROAD TRACKS
(SEE E. 180077)



W.P. 197-58
DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE, TORONTO

LEMONVILLE ROAD BRIDGE
OVER HWY. 403 & C.N.R.

THE HWSB Highway No. 403
SP. 100-1000-100
TWO EAST LAMBTONSHIRE LOT 10 CON. 1

PRELIMINARY PLAN

APPROVED

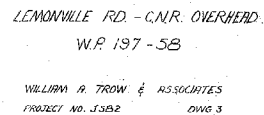
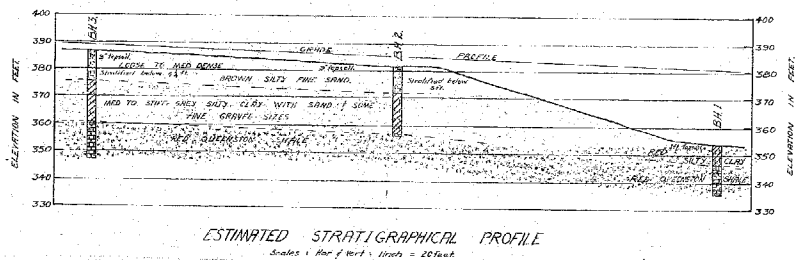
SCALE: 1" = 10' 0"

DATE: 10/1/58

BY: [Signature]

FOR: [Signature]

PROJECT: D-1804-P



MEMORANDUM

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

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

Attention: Mr. S. McCombie

DATE: August 9, 1967

OUR FILE REF.

IN REPLY TO

AUG 22 1967

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed New Bridge
At Bronte Creek & Hwy. #2, Line 'A'
Oakville, Ontario
District #4 (Hamilton)
W.J. 67-F-42 -- W.P. 296-65

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

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

AGS/Mdof
Attach.

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

Foundations Files
Gen. Files

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

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 - 5.3) Organic Clay-Silt with traces of Sand (containing Shells and occasional fragments of Wood).
 - 5.4) Sandy Silt to Silty Sand with Organics and Clay.
 - 5.5) Alluvial Sand and Gravel with traces of Organics, Silt and Clay.
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 - 5.7) Bedrock - Shale.
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FOUNDATION INVESTIGATION REPORT
For
Proposed New Bridge
At Bronte Creek & Hwy. #2, Line 'A'
Oakville, Ontario
District #4 (Hamilton)
W.J. 67-F-42 -- W.P. 296-65

1. INTRODUCTION:

A request for a foundation investigation at the site of the proposed new bridge, was received from the Bridge Location Section in a memo dated May 18, 1967.

A field investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the location of the proposed structure. Presented in this report are the results of this investigation, together with recommendations pertaining to the design of the proposed foundations and approach embankments.

2. DESCRIPTION OF THE SITE:

The bridge is located at the crossing of Bronte Creek and King's Hwy. #2, Line 'A'. The site is within the Town of Oakville, County of Halton. The surrounding area is generally flat. The existing bridge is 124 ft. long, and is a concrete bowstring type.

Physiographically, the site is located in the region referred to as the Iroquois Plain.

3. FIELD INVESTIGATION PROCEDURE:

A total of 15 boreholes, and 6 dynamic cone penetration tests was carried out. Boring was achieved by means of conventional diamond drilling equipment adapted for soil sampling purposes. Most of the work was carried out using a raft. Soil samples were obtained by means of standard split-spoons and Shelby tubes.

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

The bedrock samples were obtained using AXF diamond drill cores. Dynamic cone penetration tests were carried out adjacent to the boreholes.

The locations and elevations of all boreholes are shown on Drawing 67-F-42A, which accompanies this report.

4. LABORATORY TESTS:

Most samples were tested in the laboratory for the water content and the Atterberg limits. Certain samples were tested for determination of the undrained shear strength, consolidation and grain-size distribution. The shear strength tests were undrained triaxial tests, unconfined compression tests, and lab. vane tests. The organic contents were determined in the chemical laboratory.

The results of these tests are plotted on the borehole log sheets which form part of this report.

5. SOIL TYPES AND SOIL CONDITIONS:

5.1) General:

Subsoil at the site consists of different deposits of varying depths, followed by shale bedrock. The boundaries of the different deposits are shown on the accompanying borelog sheets. The estimated stratigraphical profiles and cross section shown on Drawing 67-F-42A, are based upon this information.

From ground level downwards, the various soil types are as follows:

5.2) Fill Material (Clayey Silt with Sand and Gravel):

This material was observed in B.H.'s 1, 11, 12, 14, 8 and 10, and consists of clayey silt with sand and gravel. From the grain-size analyses, the curves show relatively well graded particles except Sample 3 in B.H. 14 which shows a uniform material.

cont'd. /3 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.2) Fill Material (Clayey Silt with Sand and Gravel): (cont'd.)...

Physical properties as determined from field and laboratory tests, are summarized as follows:

Liquid Limit	W _L	21 - 32%
Plastic Limit	W _p	15 - 20%
Moisture Content	W	12 - 20%
Organic Content		0.3 - 2.2%

The consistency of this fill in general, is firm to stiff.

5.3) Organic Clay-Silt with traces of Sand (containing Shells and occasional fragments of Wood):

This deposit was encountered in all boreholes except B.H. 1, and consists of organic clay-silt with traces of sand and shells.

The grain-size distribution curve of Sample No. 1 in B.H. 2, shows a skip-graded material, which is a poorly graded type.

From the plasticity chart, most points lie in CI, OI & CL ranges.

Physical properties as determined from field and laboratory tests, are summarized as follows:

cont'd. /4 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.3) Organic Clay-Silt with traces of Sand (containing Shells and occasional fragments of Wood): (cont'd.) ...

Unconf. Comp. Test		210 - 1100 p.s.f.
Triaxial Test		250 - 1200 p.s.f.
Lab. Vane Test		270 - 730 p.s.f.
Field Vane Test		400 - 1170 p.s.f.
Bulk Density		103 - 119 lb./ft. ³
Liquid Limit	W _L	35 - 50%
Plastic Limit	W _p	22 - 30%
Moisture Content	W	30 - 40%

The organic content ranges from 0.4 - 11.8% by weight as shown in the following table:

cont'd. /5 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.3) Organic Clay-Silt with traces of Sand (containing Shells and occasional fragments of Wood): (cont'd.) ...

B.H. No.	Organic Content in Percentage by Weight												
	S 1	2	3	4	5	6	7	8	9	10	11	12	13
2	4.2					1.7							
3		4.8	1.2	3.8		4.6							
4		3.0	2.8	0.7									
11	2.0	2.4											
12					0.4								
13		4.7		2.9	1.7	1.9	0.8		1.7	1.1			
14								11.0	11.8	2.1		1.8	
8	2.2					1.9	4.9	4.9		0.9			
7		5.1	2.5				4.3						
6													
5				5.2		5.1							
9								6.2		0.7	0.6	2.5	1.0
10					4.9			1.2					

Table showing the percentage of organic material in each borehole for different samples.

These organic matters consist of undecomposed substances of wood fragments, fibrous roots and microorganisms.

The consistency of this deposit varies from very soft to firm material.

cont'd. /6 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.3) Organic Clay-Silt with traces of Sand (containing Shells and occasional fragments of Wood): (cont'd.) ...

The undrained shear strength test results show an extremely wide scatter (which is partly due to variability in material), but by grouping the boreholes conveniently, the scatter in each group can be reduced. In the Appendix, four plots are shown of the shear strength with depth profile of:

- Group (1) B.H.'s 2 & 7 (West bank of creek - not under existing fill)
- Group (2) B.H.'s 3, 4, 5 & 6 (Bed of creek)
- Group (3) B.H.'s 10, 12, 13 & 14 (under east approach fill)
- Group (4) B.H.'s 9 & 11 (East bank of creek - not under existing fill)

The estimated average undrained shear strengths are included on the plots for the four conditions, and are as follows:

Group (1)	El. 240 - El. 220	530 - 800 p.s.f.
Group (2)	El. 240 - El. 220	400 - 800 p.s.f.
Group (3)	El. 240 - El. 225	600 - 800 p.s.f.
Group (4)	El. 244 - El. 220	350 - 800 p.s.f.

5.4) Sandy Silt to Silty Sand with Organics and Clay:

The deposit was observed in B.H.'s 3, 6 and 5. It consists of sandy silt to silty sand with a maximum thickness of 10 ft. at B.H. 3.

The grain-size distribution curves for B.H. 3, samples 1, 3 and 4, are shown in the Appendix.

The plasticity chart for this deposit could not give a clear identification.

Physical properties as determined from field and laboratory tests, are summarized as follows:

cont'd. /7 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.4) Sandy Silt to Silty Sand with Organics and Clay:

Undrained Shear Strength		250 p.s.f.
Bulk Density		95.5 - 103 lb./ft. ³
Liquid Limit	W _L	42 - 55%
Plastic Limit	W _P	40 - 45%
Moisture Content	W	45 - 50%
Organic Content		1.5 - 4.8%

The densification ranges between very loose to loose.

5.5) Alluvial Sand and Gravel with traces of Organics,
Silt and Clay:

This deposit was found in boreholes 2, 4, 6, 7, 11, 12, and 13. The maximum thickness of this layer is about 8 ft. The organic content was found to be 6.2% maximum in Sample 8, B.F. 9, and 0.7% minimum in Sample 10, B.H. 10.

The grain-size distribution curves are shown in the Appendix.

Physical properties as determined from field and laboratory tests, are summarized as follows:

cont'd. /8 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.5) Alluvial Sand and Gravel with traces of Organics,
Silt and Clay: (cont'd.) ...

Undrained Shear Strength		600 - 1000 p.s.f.
Liquid Limit	W _L	30 - 60%
Plastic Limit	W _P	18 - 45%
Moisture Content	W	30 - 50%
Organic Content		0.7 - 6.2%

The densification of this deposit varies from loose to compact.

5.6) Glacial Till - Clayey Silt with Sand, Gravel and Shale
Fragments:

This layer was found in B.H.'s 1, 1A, 4, 11, 8, 5, 9 and 10. It is about 24 ft. thick in B.H. 1, 5 ft. in B.H. 5, and 6 ft. in B.H. 10.

It consists of clayey silt with sand, gravel and shale fragments.

From the grain-size distribution analyses, the sample 5 in B.H. 1 shows a skip-graded material.

In the plasticity chart, the samples lie in the CL zone.

cont'd. /9 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.6) Glacial Till - Clayey Silt with Sand, Gravel and Shale
Fragments: (cont'd.) ...

Physical properties as determined from field and laboratory tests, are summarized as follows:

Undrained Shear Strength		1000 - 4000 p.s.f.
Liquid Limit	W _L	27 - 31%
Plastic Limit	W _P	18 - 20%
Moisture Content	W	20 - 40%

The consistency varies from very stiff to hard.

5.7) Bedrock - Shale:

The shale bedrock was encountered at all boreholes. Its level ranges between 223.0 at B.H. 1, to 215.0 at B.H. 10. It consists of grey clayey shale, soft and hard, with a relatively low recovery.

From the geological point of view, a report from Mrs. Z. Dunikowska, Geologist, Special Projects, is as follows:

"Results of the investigation show: that borings penetrated through Pleistocene deposits filling up the valley, and went into bedrock of the Upper Ordovician System.

"Holes No. 1 and 8 on the West bank of Bronte Creek were drilled in an area where the Queenston Formation overlies the Medford Formation. Hole No. 1 at a depth of approx. 34 ft. and Hole No. 8

cont'd. /10 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.7) Bedrock - Shale: (cont'd.) ...

at approx. 31 ft., after penetrating the Queenston Formation (Hole No. 1, 4 ft., Hole No. 8, 7 ft.), enter the Medford Formation. In both holes (1 and 8) the Queenston Formation is represented by red and greenish, soft shale with layers of harder, sandy shale or thin seams of argillaceous, often sandy limestone. Some of the shales have a mottled appearance, abundant fossils in limestone seams and in shale. The remaining core of Hole No. 1 (5 ft.) and Hole No. 8 (7 ft.) was drilled in the Medford Formation and consists mainly of grey and greenish-grey, soft shale with layers of up to 3 inches slightly harder arenaceous grey shale. Sporadic thin layers and seams (up to 3 inches) of grey, argillaceous, often sandy limestone. The core is highly fossiliferous, shales have visible ripple marks and lensey character as a result of shallow water sedimentation.

"The holes 2, 3, 4, 5, 6, 7, 9, 10, 11, 12, 13 and 14, were drilled in the Medford Formation and all show uniform succession of grey-greenish or grey-blueish shale, often arenaceous interbedded with seams or thin layers of grey limestone.

"In all borings the amount of shale prevails over the harder layers."

6. GROUNDWATER CONDITIONS:

The groundwater level in the boreholes on both sides of the embankments were measured. It was at El. 247.6 in B.H. 14 - i.e., 1 ft. higher than the water level of the creek in this period of time. It may be assumed, however, that within the limits of the site, the groundwater level could be assumed as 247.0.

cont'd. /11 ...

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General:

It is proposed to construct a new bridge at the crossing of Hwy. 2 and Bronte Creek to replace the present 120-ft. single-span concrete bowstring bridge.

The new centre-line (Line 'A') will be almost coincident with the existing centre-line of Hwy. 2, but the new profile grade will be some 11 to 12 ft. higher than the existing grade.

Present proposals call for either a single, 124-ft. span bridge, or a three-span bridge having a centre span of 72 ft. and approach spans of 63 ft. These latter proposals, of course, are to be regarded as preliminary only, and are subject to change. The new bridge will be about twice the width of the existing bridge which is about 40 ft. wide. The maximum height of the new approach embankments will be about 53 ft. above the river bed, and the most critical slopes from a stability point of view, will therefore be in the longitudinal direction.

The various aspects of the proposed project are discussed below under appropriate headings.

7.2) Structure Foundations:

The field investigation has revealed the presence of an extensive deposit of soft to firm organic clay-silt extending from the west limits of the new bridge to some two or three hundred feet east of the east limits. The nature of this deposit is such as to completely preclude the possibility of conventional spread footing type foundations. It is recommended, therefore, that the new bridge be supported on piled foundations, but in order that these may perform satisfactorily, it will be necessary to construct the bridge approaches in such a way as to preclude the possibility of even slight lateral earth movements. This aspect is dealt with in detail, as follows, under the heading: "Structure Approaches."

cont'd. /12 ...

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

7.2) Structure Foundations: (cont'd.) ...

Insofar as the piled foundations are concerned, it would seem that steel H-piles driven to bedrock, would be the most economical and practical solution; due to the nature of the bedrock, it should be possible to key this type of pile a few inches into the bedrock. The piles should be designed so as to resist all lateral forces induced on the structure by the approach embankments. Design loads may be the maximum allowable for the particular section adopted.

Consideration should be given to the use of cast-in-place concrete caissons installed some 3 - 4 feet into sound bedrock for the proposed piers (3-span structure). This type of foundation should be much simpler to construct than the conventional type of footing or pile cap which will require a dewatering scheme to enable concrete to be poured in the dry. Such caissons should be capable of withstanding safely, design loads of up to 35 t.s.f. provided they are founded within sound shale bedrock.

As to whether a single-span or 3-span structure is constructed, this question can be decided entirely on the basis of economical or, possibly, hydrological considerations. The abutment footings of a single-span structure will require a dewatering scheme, since they will have to be constructed below river water level, whereas the footing of a perched abutment can be constructed entirely in the dry.

7.3) Structure Approaches:

As discussed above, it is important to construct approach embankments at this site in such a way as to rule out the possibility of lateral earth movements, and ensure the satisfactory performance of the future bridge structure. From the west bank of the creek to several hundred feet east of the east bank, the upper subsoil consists

cont'd. /13 ...

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

7.3) Structure Approaches: (cont'd.) ...

of a deposit of very soft to firm organic clay-silt with extremely variable properties, both as regards shear strength and material composition. The existing embankment is composed of about 11 feet of fill material at the east approach to the bridge, and appears to be stable at the present time, although differential settlements have occurred along the junction of a widening carried out some six years ago, with consequent cracking of the pavement. It is believed that the addition of a further 12 feet of material on the existing fill will result in unstable conditions which will give rise not only to possible failure of the embankment in the forward direction where the total height will be 33 ft. above the creek bed, but also, to differential settlements on that part of the approach east of the structure.

In view of the foregoing, the following steps are recommended:

1) Excavate all organic soil above El. 232.0 from the west interface with the glacial till (Sta. 414+20⁺) to a line 30 ft. east of the back of the east abutment footing. From this point easterly, the base of the excavation should slope at 4 horizontal to 1 vertical upwards to El. 240.0, then continue east at El. 240.0. The east limits of the excavation will be decided upon by the Soils Section when they have completed their field investigation. The width of the excavation base should be about 10 ft. wider than the bridge deck or road bed (5 ft. each side), and should have side slopes of 1:1.

2) The excavated trench should be backfilled with suitable granular material up to a convenient height above water level from which point on, suitable earth fill may be used. The upper two feet of fill within the stream bed should be coarse material suitable for scour protection. Standard 2:1 slopes may be constructed on the approach embankments.

cont'd. /14 ...

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

7.3) Structure Approaches: (cont'd.) ...

3) East of the line where the 4:1 sloping excavation base intersects El. 240.0 (Sta. 412+22⁺, single-span structure and Sta. 411+86⁺, three-span structure), the existing fill material may be left in place; the organic material above El. 240.0 should be excavated for the new widening only. It is believed that the resulting section will be stable, but it should be borne in mind that differential settlements are likely to occur under the future embankment due to consolidation of unequal thicknesses of organic soil. For this reason, paving should be delayed as long as possible to allow consolidation to take place.

8. SUMMARY:

A foundation investigation at the site of the proposed new bridge at Bronte Creek and Hwy. 2, is reported.

Subsoil at the site was found to consist of up to 20 feet of very soft to firm organic clay-silt, followed by up to 10 feet of compact to very dense alluvial sand and gravel, followed by shale and limestone bedrock, partially covered by dense glacial till.

It is recommended to partially subexcavate the organic soil within the limits of and to about 65 feet east of the new bridge and replace with suitable granular fill prior to constructing the new bridge.

It is recommended to found the new bridge on steel H-piles driven a few inches into the shale bedrock. Consideration should be given to the use of cast in-situ concrete caissons to support the piers in the case of a multispan structure.

The decision to construct a single, or multispan structure should be based on economical or hydrological reasons.

cont'd. /15 ...

8. SUMMARY: (cont'd.) ...

Partial subexcavation should be carried out for the proposed widening of Hwy. 2 east of the bridge. Existing fill material may be left in place east of a line some 65 feet behind the east abutment. Standard 2:1 slopes may be constructed on the new approach embankments.

Excavation for structure footings carried out below the water level, will require a dewatering scheme.

9. MISCELLANEOUS:

The field investigation was carried out during the period May 30 to June 23, 1967. Equipment used on the site was owned and operated by Dominion Soil Investigation Ltd. The supervision of the field work and preparation of this report, were undertaken by Dr. S. Nassif, Project Foundation Engineer.

The report was reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

August 1967

APPENDIX I

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-42

LOCATION Sta. 413 + 66, Offset 26' Rt.

ORIGINATED BY SN

W. P. 296-65

BORING DATE June 1 & 2, 1967

COMPILED BY AMS

DATUM Geodetic

BOREHOLE TYPE NX Casing, Washbore & AXT Rock Core

CHECKED BY *SR*

SOIL PROFILE

SAMPLES

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOTLIQUID LIMIT — WL
PLASTIC LIMIT — WP
WATER CONTENT — WELEV.
DEPTH

DESCRIPTION

STRAT. PLOT

NUMBER

TYPE

BLOWS / FOOT

ELEV. SCALE

SHEAR STRENGTH P.S.F.

o Unconfined

• Triaxial

+ Field Vane

x Lab Vane

WATER CONTENT %

BULK
DENSITY
P.C.F.

REMARKS

246.6

Water Level

245.5

GROUND LEVEL

0.0

Organic clay - silt
with traces of sand
(containing shells
& occasional pieces
of wood)

1

SS

1

+ 3.5

10

9 9 73 9
Org. 4.2%

2

TW

PM

+ 2.7

10

104

Org. 4.0%

3

TW

PM

x •

+ 4.0

10

119

4

TW

PM

+ 3.5

10

118

5

TW

PM

+ 6.0

10

114

6

TW

PM

+ 4.0

10

114

Org. 1.7%

111

7

TW

PM

10

115

219.5

Alluvial sand & gravel
with traces of org.
silt and clay.

8

TW

PM

10

100

218.7

Loose to Compact

9

SS

50/1"

26.8

Bedrock

10

AXT

Rec.

Grey

RC

40%

Shale

11

AXT

Rec.

Bedrock

RC

62%

12

AXT

Rec.

13

AXT

Rec.

14

AXT

Rec.

200.5

45.0

End of Borehole

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 67-F-42

LOCATION Sta. 413 + 34, Offset 33' Rt.

ORIGINATED BY SN

W. P. 296-65

BORING DATE June 6, 1967

COMPILED BY AMS

DATUM Geodetic

BOREHOLE TYPE NX Casing, Washbore, AXT Rock Core

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. PLOT	NUMBER TYPE	BLOWS / FOOT	SHEAR STRENGTH P.S.F.		
					+ Field Vane	WP W WL	
246.6	Water Level				400 600 800 1000 1200	WATER CONTENT % 20 40 60	P.C.F.
238.6	GROUND LEVEL						
0.0	Sandy silt to silty sand with organics.						
	Very loose to loose.		1 TW FM		+ 8.0		
			2 TW FM				
			3 SS				
230.1			4 SS 1 1/8"				
8.5	Organic clay - silt with traces of sand.		5 TW FM		+ 3.3		
			6A SS 7				
			7 SS 31				
218.3			8 SS 29/10"				
20.3	Bedrock Grey Shale Bedrock		9 AXT RC				
			10 AXT RC Rec 50%				
			11 AXT RC Rec 80%				
208.6							
30.0	End of Borehole						

FOUNDATION SECTION

ORIGINATED BY SN

COMPILED BY _____ SN

CHECKED BY

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 67-F-42 LOCATION Sta. 412 + 99, Offset 32' Lt. ORIGINATED BY SN
W.P. 296-65 BORING DATE June 9, 1967 COMPILED BY SN
DATUM Geodetic BOREHOLE TYPE Core Drill CHECKED BY SK

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. • Triaxial + Field Vane					Wp — W — WL 20 40 60				
246.6	Water Level						400	600	800	1000	1200					
242.6	Ground Level															
0.0	Sandy silt to silty sand with organics. Very Loose to Loose.		1	TW	PM	240	•	3.3							103	
238.6	Organic Clay-silt traces of sand and gravel. Very soft to firm.		2	SS	2											
4.0			3	TW	PM	230										
			4	SS	3											
			5	SS	2											
			6													
			6A	SS	2											
222.6	Clayey silt (Glacial Till) Hard		7	SS	4	220										
20.0			8	SS	26											
217.6			9	SS	30/8"											
25.0	Shale Bedrock					210										
204.6																
18.0	End of Borehole															

Gr. 28, Sa. 29
Si. 38, Cl. 5

FOUNDATION SECTION

ORIGINATED BY SN

COMPILED BY _____ SN

CHECKED BY

[illegible]

FOUNDATION SECTION

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 %				
251.9	Ground Level					400	600	800	1000	1200	WP	WL	WL		
0.0	Clayey silt with sand and gravel.		1	SS	4										
	Firm to stiff. (Fill)		2	SS	8										
246.6			3	SS	19										
4.3	Organic clay-silt with traces of sand (contains shells and occ. pieces of wood)		4	SS	16										
	Very soft to firm.		5	SS	8										
			6	SS	10										
			7	SS	5										
			8	SS	6										
			9	SS	8										
236.7			10	SS	10										
15.2	Clayey silt with sand, gravel & shale frgts.		11	SS	19										
	Very stiff to hard. (Fill)		12	SS	60										
			13	SS	94/1"										
			14	SS	109										
227.4			15	SS	84/6"										
24.5	Bedrock Grey shale														
213.9															
28.0	End of Borehole														

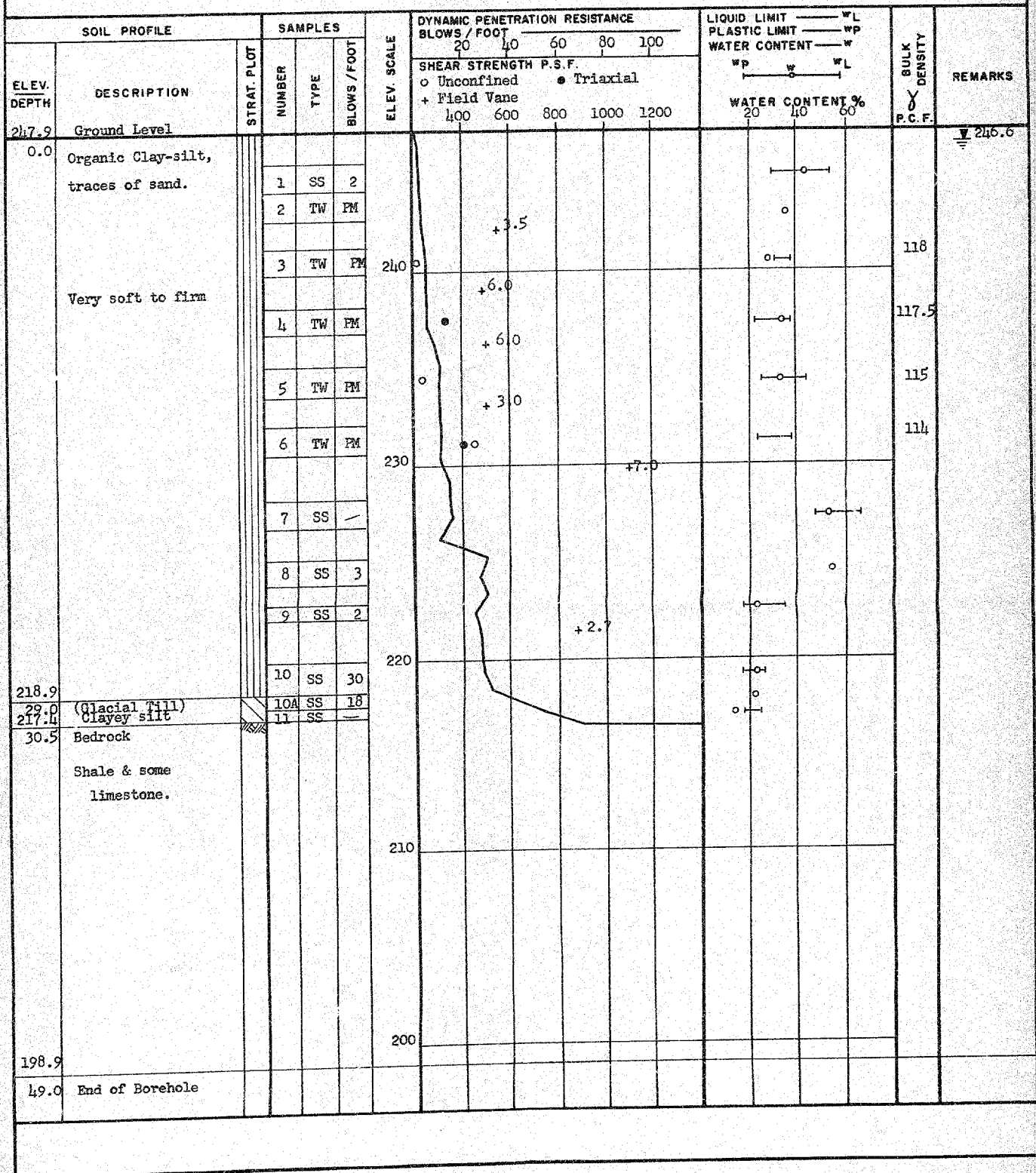
DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-42 LOCATION Sta. 412 + 70, Offset 30' Lt. ORIGINATED BY SN
W.P. 296-65 BORING DATE June 22, 1967 COMPILED BY SN
DATUM Geodetic BOREHOLE TYPE Core Drill CHECKED BY SK



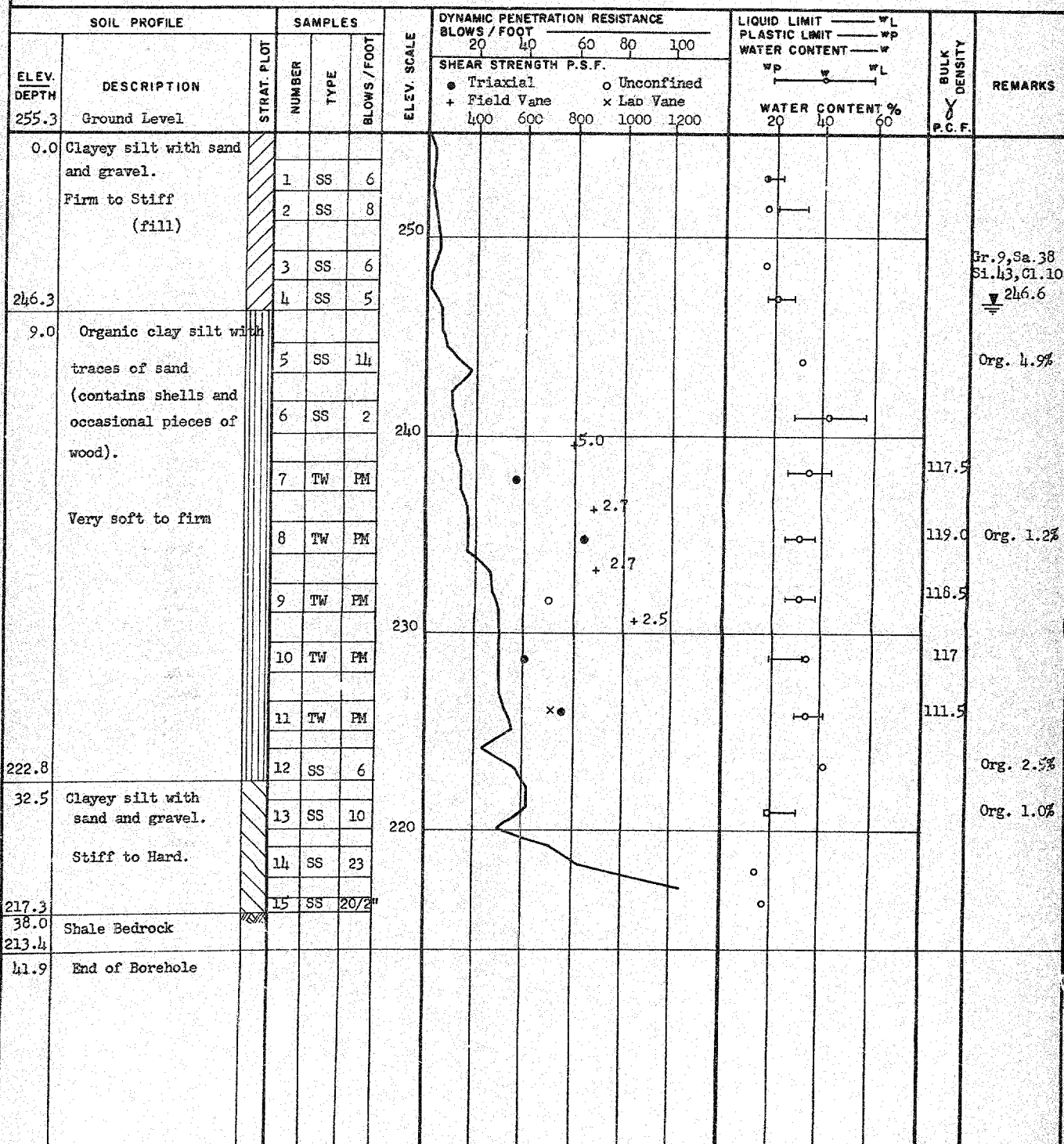
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO.10

FOUNDATION SECTION

JOB 67-F-42 LOCATION Sta. 412 + 28, Offset 26' Lt. ORIGINATED BY SN
W.P. 296-65 BORING DATE June 23, 1967 COMPILED BY SN
DATUM Geodetic BOREHOLE TYPE Core Drill CHECKED BY SK



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

JOB 67-F-42 LOCATION Sta. 412 + 50, Offset 33' Rt. ORIGINATED BY SN
W.P. 296-65 BORING DATE June 11, 1967 COMPILED BY SN
DATUM Geodetic BOREHOLE TYPE Core Drill CHECKED BY SR

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	20	40	60	80	100	WP	WL		
248.6	Ground Level														
0.0	Clayey silt with sand & gravel. (Fill)		—	SS	1										
			—	SS	7										
	Firm to stiff		1	SS	11										
			—	SS	5										
239.1			2	SS	4	240									
9.5	Organic clay-silt traces of sand		3	SS	3										
	Very soft to firm		4	TW	PM										
			5	TW	PM	230									
226.6			6	TW	PM										
22.0	Sand & Gravel		—	SS	8										
	Loose to comp.		7	SS	6										
			8	SS	6										
218.6			9	SS	19	220									
30.0	Clayey silt		10	SS	90/5"										
216.9	(Glacial Till)		11	SS	100/6"										
31.7	Shale Bedrock					210									
203.6															
45.0	End of Borehole														

Gr. 7, Sa. 62
Sl. 28, Cl. 3Gr. 17, Sa. 45
Sl. 26, Cl. 8

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO.12

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-42

LOCATION Sta. 412 + 09, Offset 24' Rt.

ORIGINATED BY SN

W.P. 296-65

BORING DATE June 20, 1967

COMPILED BY SN

DATUM Geodetic

BOREHOLE TYPE Core Drill

CHECKED BY *HL*

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					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	20	40	60	80	100	WP	WL		
257.6	Ground Level														
0.0	Clayey silt with sand and some gravel (Fill)		1	SS	16										
	Firm to stiff		2	SS	4										Gr. 8, Sa. 37
			3	SS	—	250									Sl. 40, Cl. 15
248.6			4	SS	2										
9.0	Organic Clay-silt with traces of sand.		5	SS	6										
			5A	SS	12										
	Very soft to firm.		6	SS	5										
			7	TW	PM	240									
			8	TW	PM										115
			9	TW	PM										117.5
						230									119
			10	TW	PM										112
225.1															113
32.5	Sand & silt. Loose to comp.		11	TW	PM	220									108
219.6															
38.0	Bedrock														
	Shale														
212.6															
45.0	End of Borehole					210									Gr. 0, Sa. 49, Sl. 47, Cl. 4

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 13

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

67-F-42

JOB LOCATION Sta. 411 + 46, Offset 63' Rt.

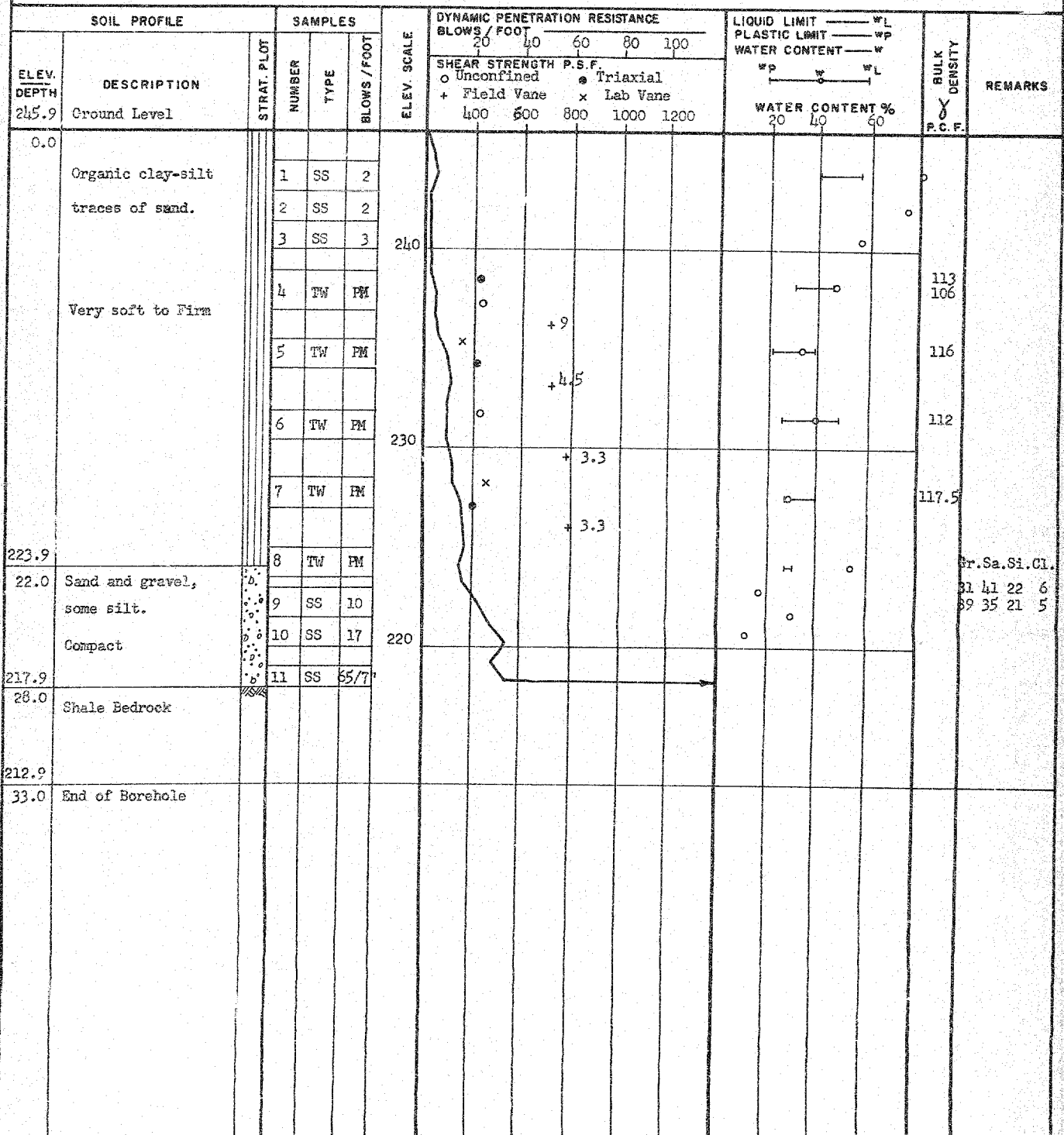
ORIGINATED BY SN

W.P. 296-65 BORING DATE June 22, 23, 1967

COMPILED BY SN

DATUM Geodetic BOREHOLE TYPE Core Drill

CHECKED BY *LL*



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

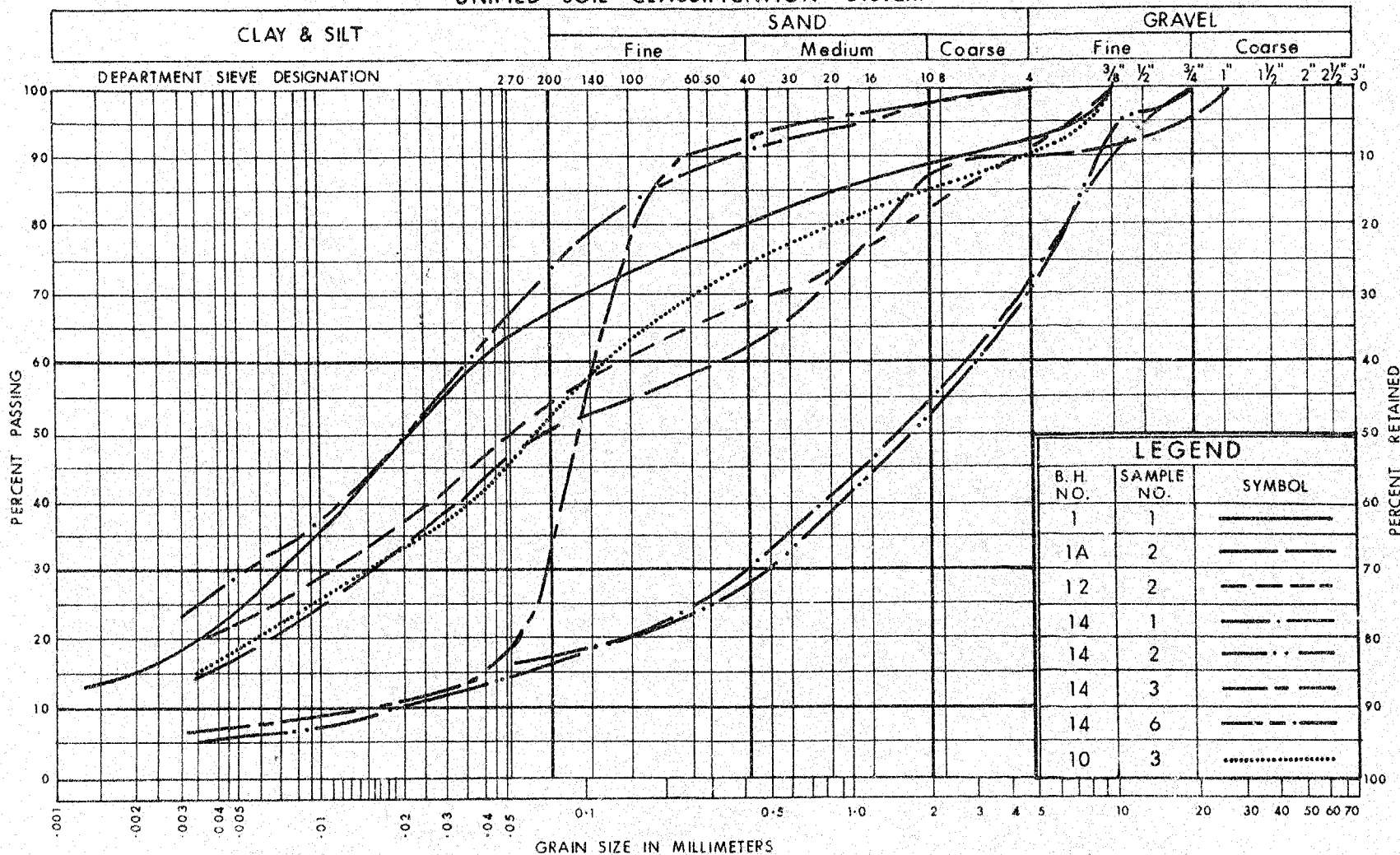
RECORD OF BOREHOLE NO. 14

FOUNDATION SECTION

JOB 67-F-42 LOCATION Sta. 110 + 73, Offset 37' Rt. ORIGINATED BY SN
 W.P. 296-65 BORING DATE June 23, 1967 COMPILED BY SN
 DATUM Geodetic BOREHOLE TYPE Core Drill CHECKED BY SL

[illegible]

UNIFIED SOIL CLASSIFICATION SYSTEM



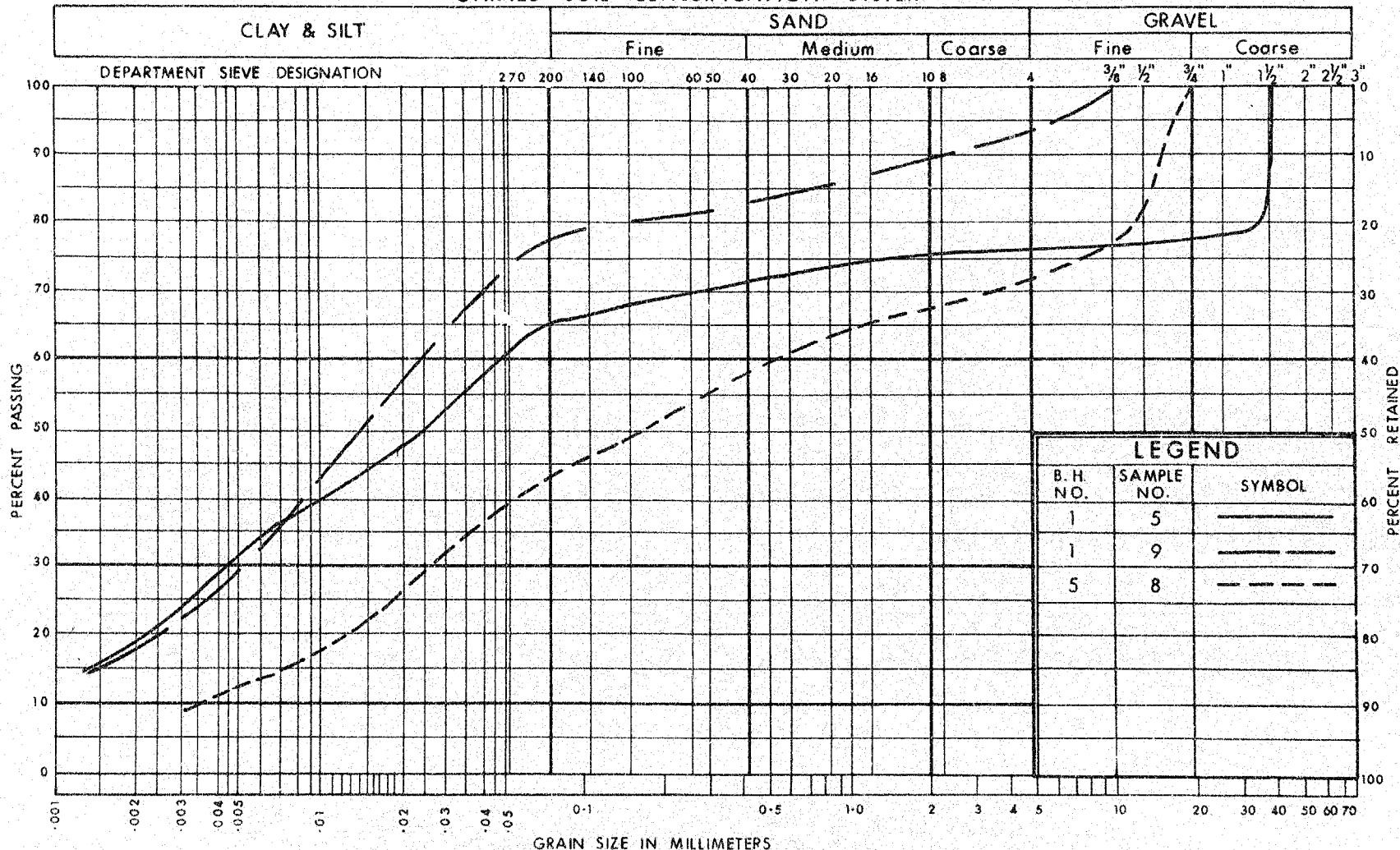
GRAIN SIZE DISTRIBUTION
CLAYEY SILT WITH SAND & GRAVEL

W.P. No. 296-65
JOB No. 67-F-42



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND

B. H. NO.	SAMPLE NO.	SYMBOL
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1	9	—————
5	8	- - - - -



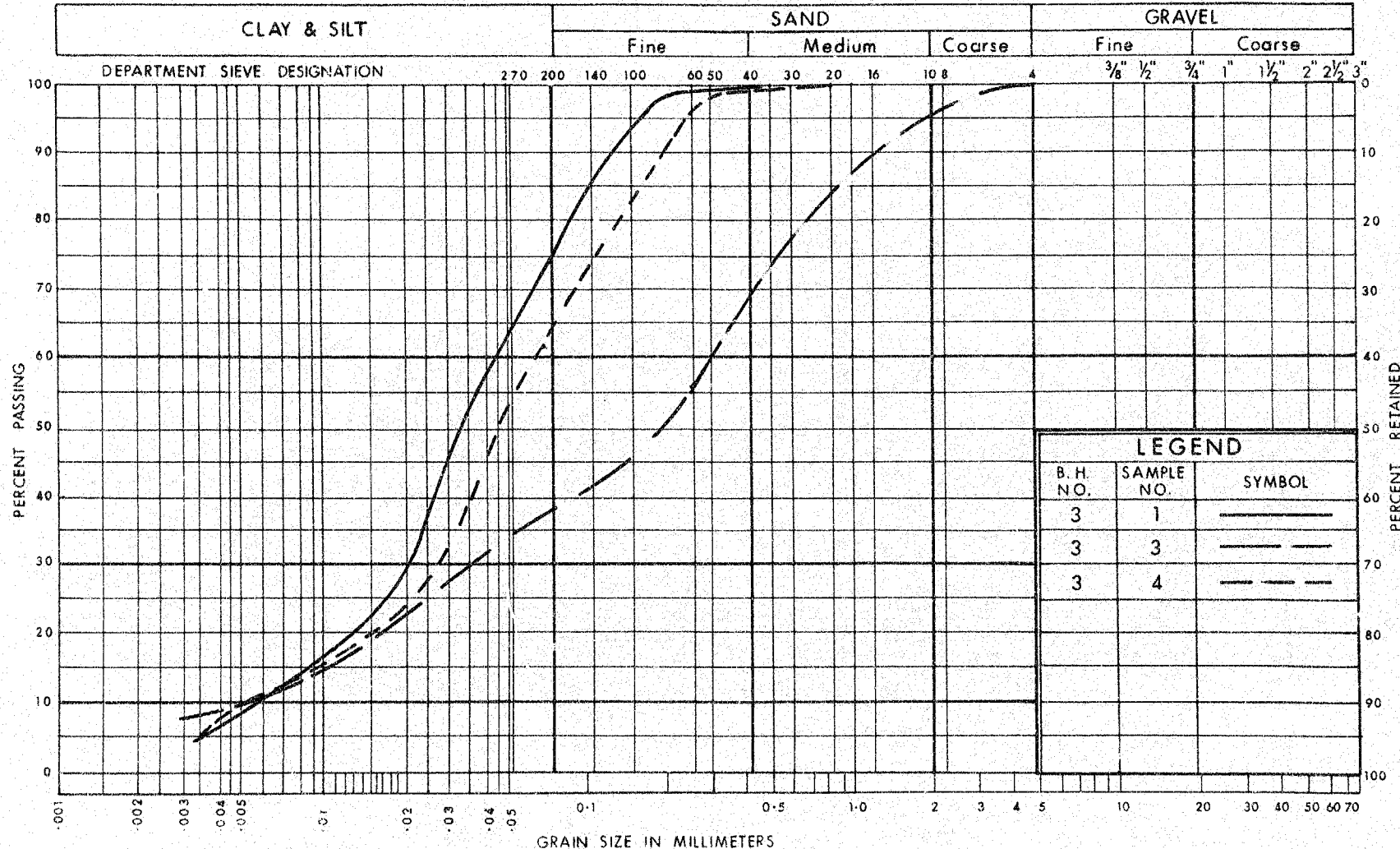
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
CLAYEY SILT WITH SAND, GRAVEL & SHALE FRAGMENTS
(GLACIAL TILL)

W.P. No. 296-65

JOB No. 67-F-42

UNIFIED SOIL CLASSIFICATION SYSTEM



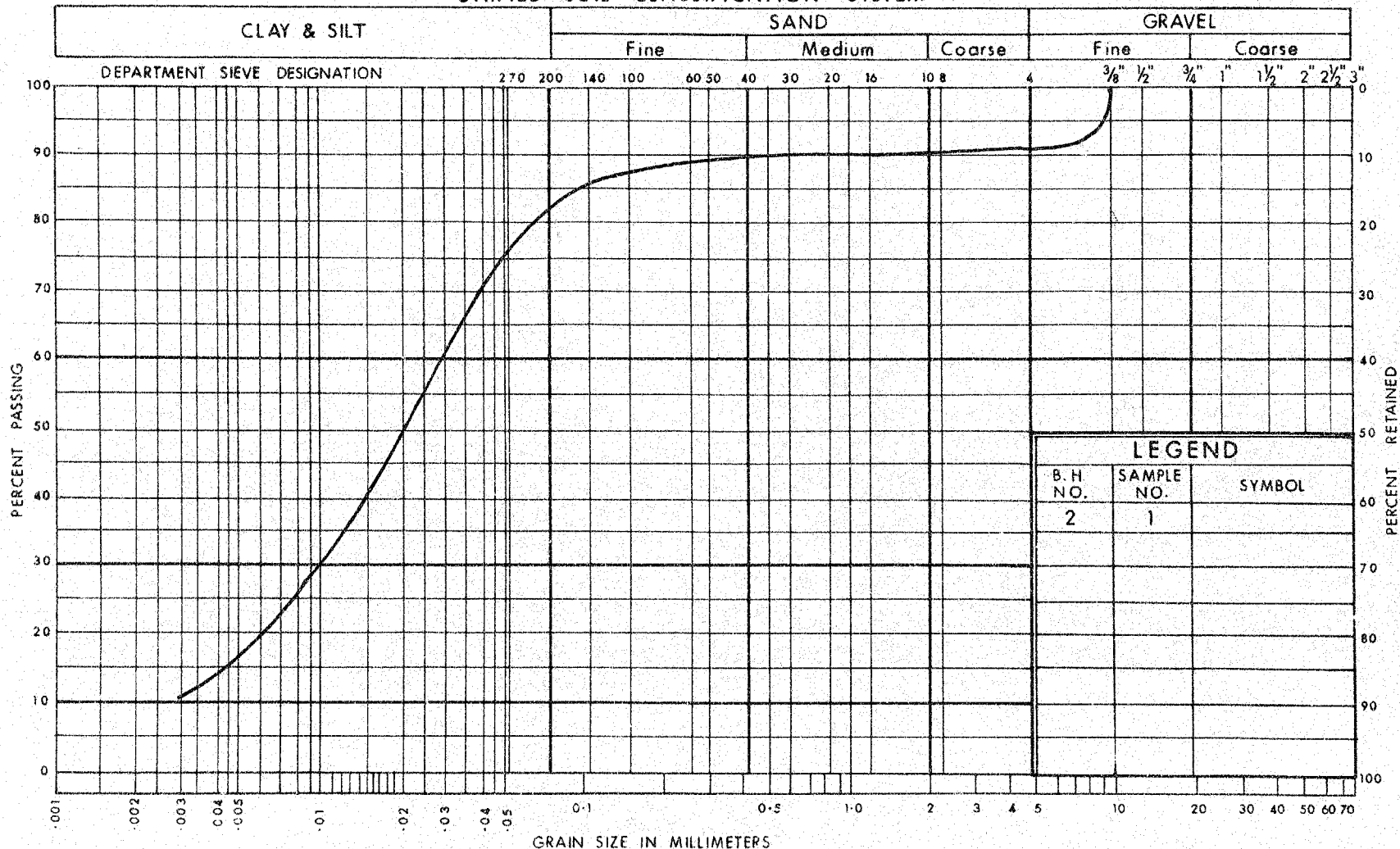
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILTY SAND WITH ORGANICS
AND CLAY

W.P. No. 296-65

JOB No. 67-F-42

UNIFIED SOIL CLASSIFICATION SYSTEM



ONTARIO

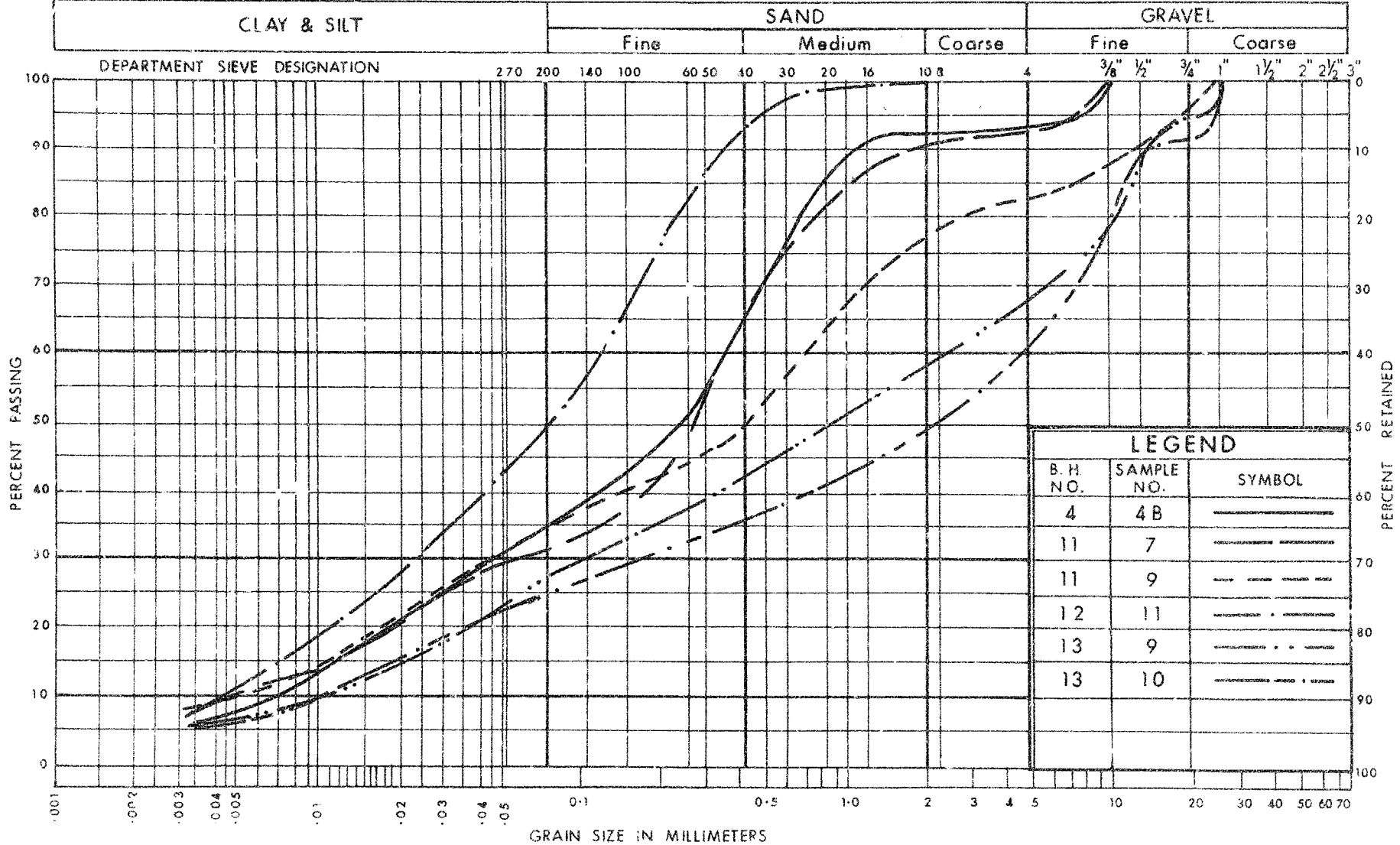
DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

GRAIN SIZE DISTRIBUTION
ORGANIC CLAY SILT WITH TRACES OF SAND

W.P. No. 296-65

JOB No. 67-F-42

UNIFIED SOIL CLASSIFICATION SYSTEM



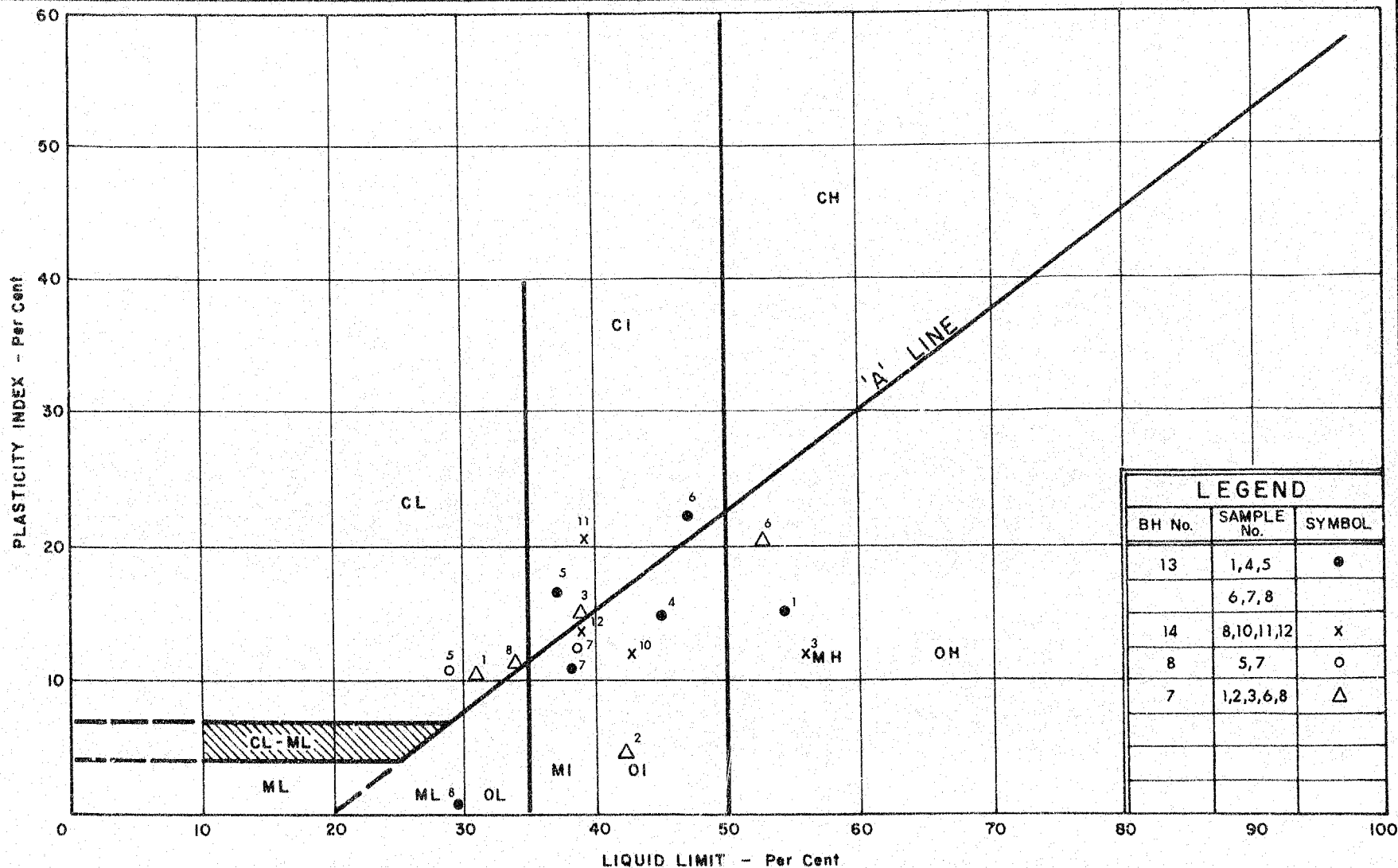
ONTARIO

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
ALLUVIAL SAND & GRAVEL WITH SILT & TRACES
ORGANICS & CLAY

W.P. No. 296-65

JOB No. 67-F-42

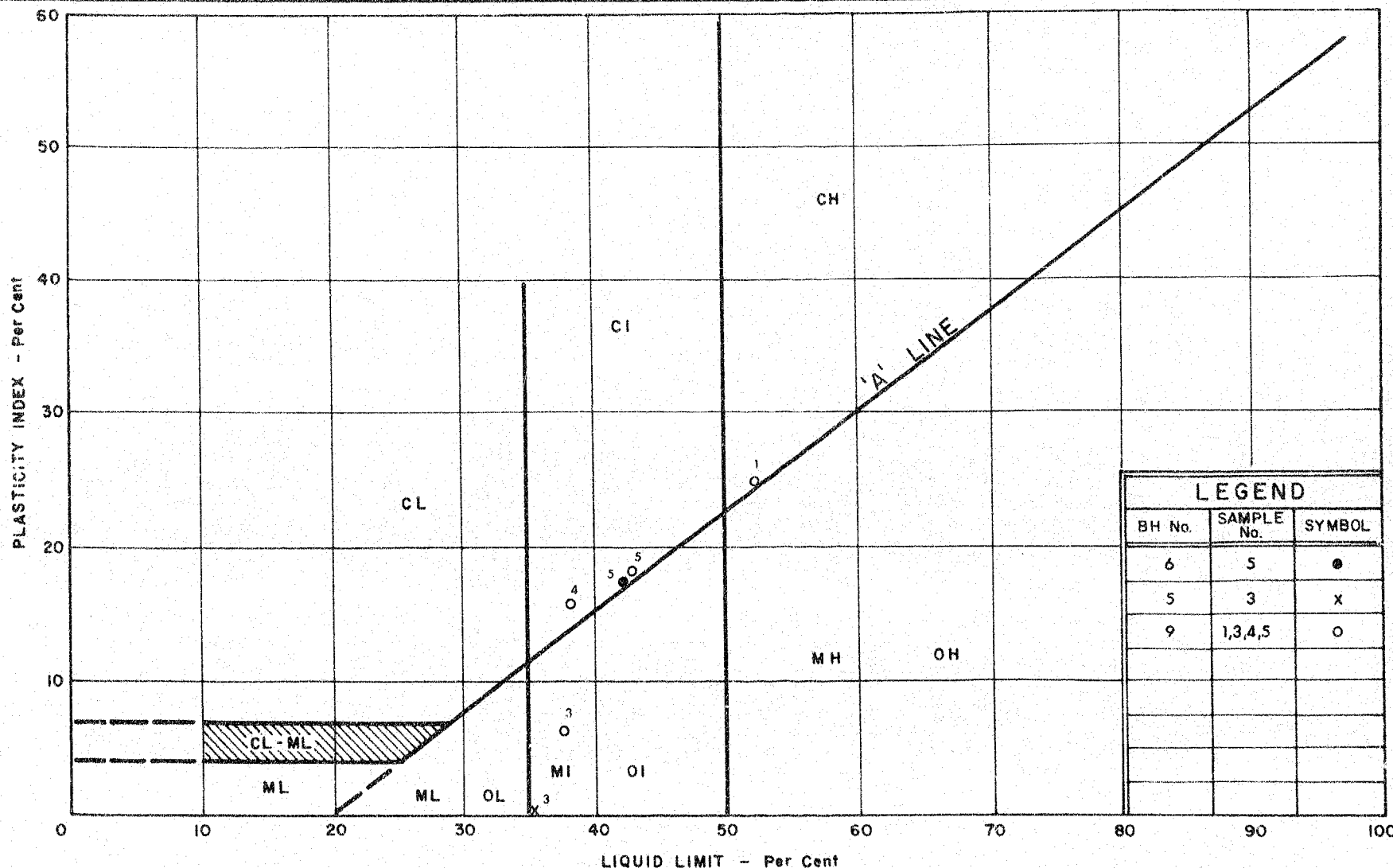


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART ORGANIC CLAY SILT WITH TRACES OF SAND

W.P. No. 296-65

JOB No. 67-F-42



LEGEND		
BH No.	SAMPLE No.	SYMBOL
6	5	●
5	3	x
9	1,3,4,5	○

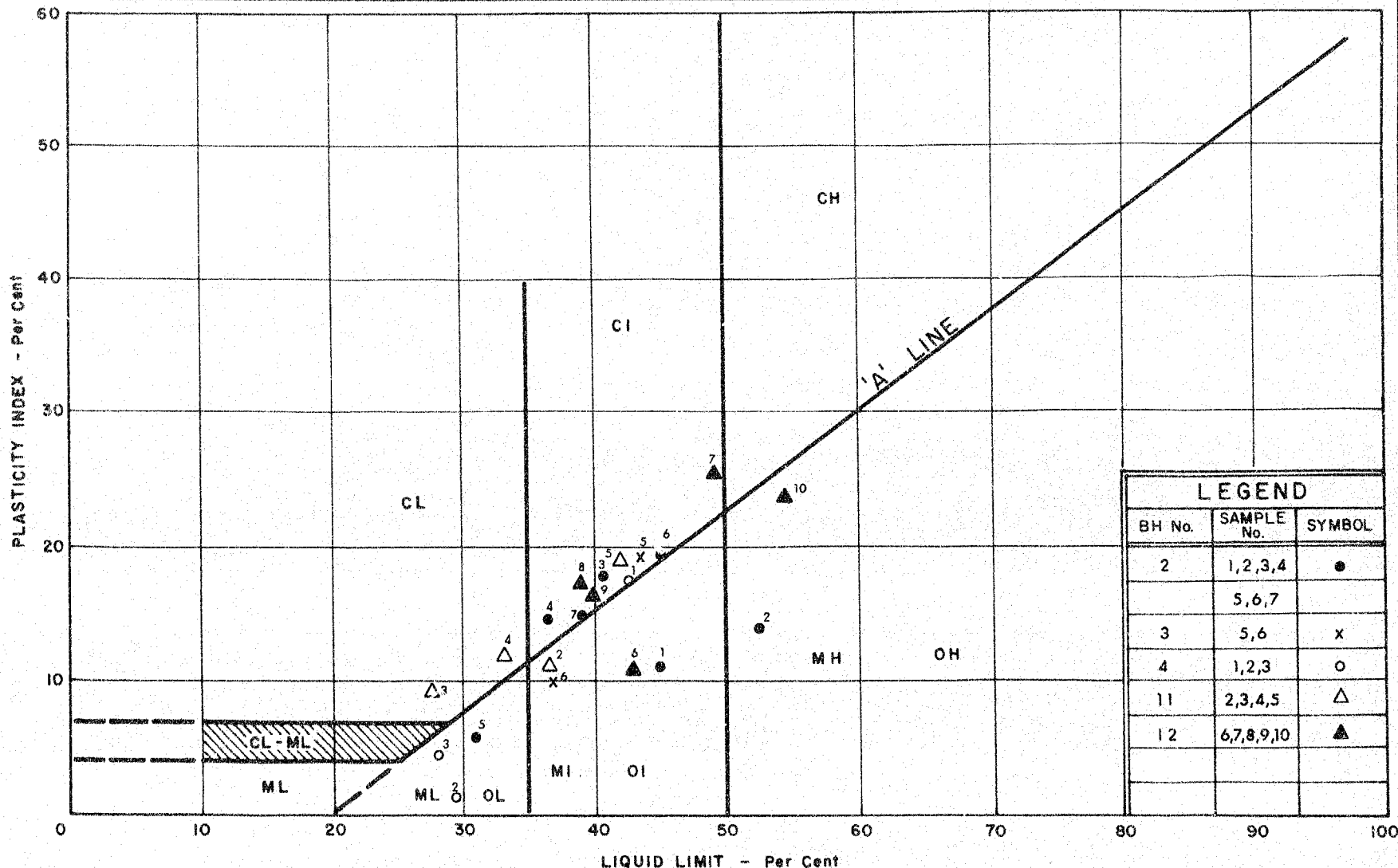


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART ORGANIC CLAY SILT WITH TRACES OF SAND

W.P. No. 296-65

JOB No. 67-F-42

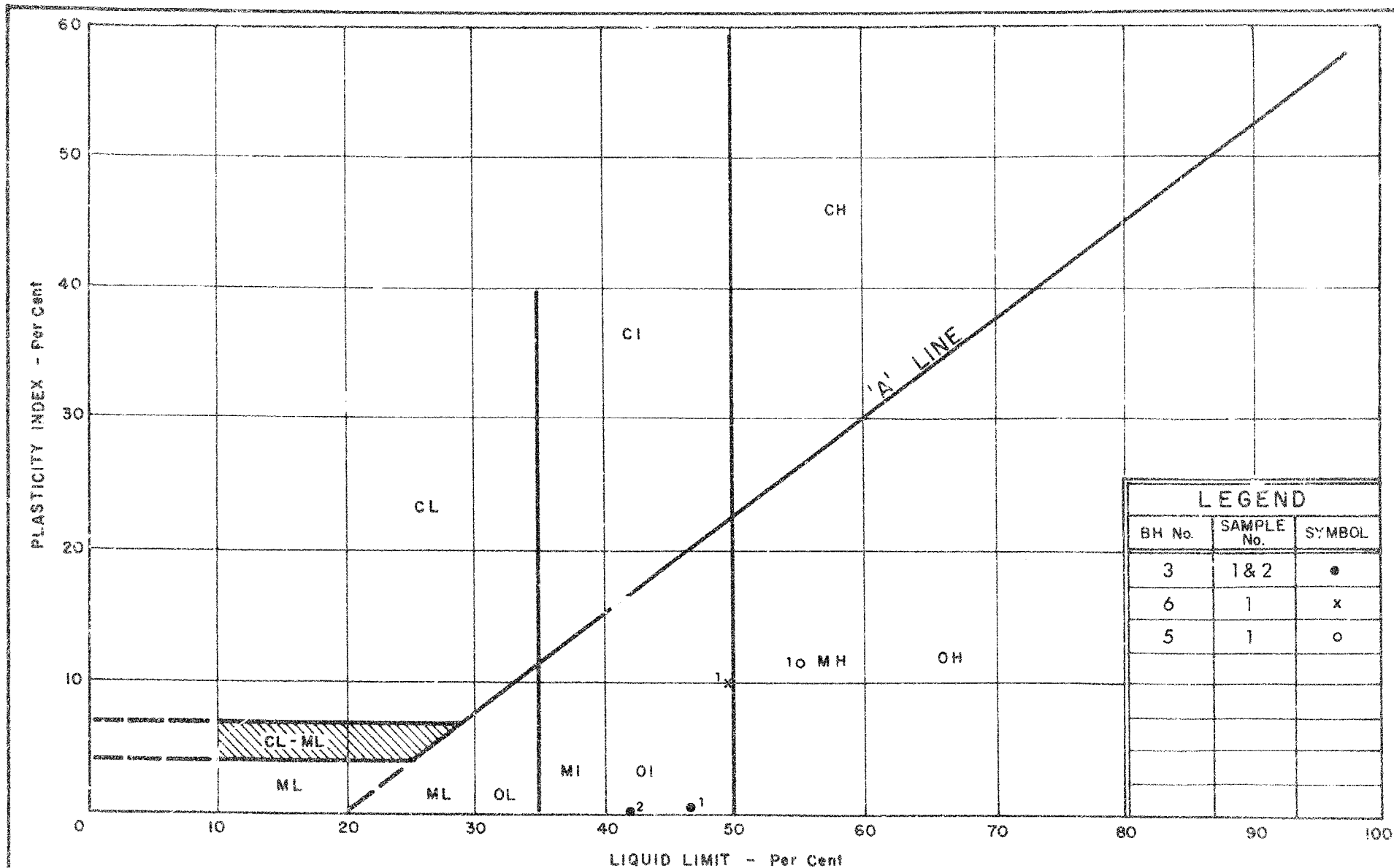


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART ORGANIC CLAY SILT WITH TRACES OF SAND

W.P. No. 296-65

JOB No. 67-F-42

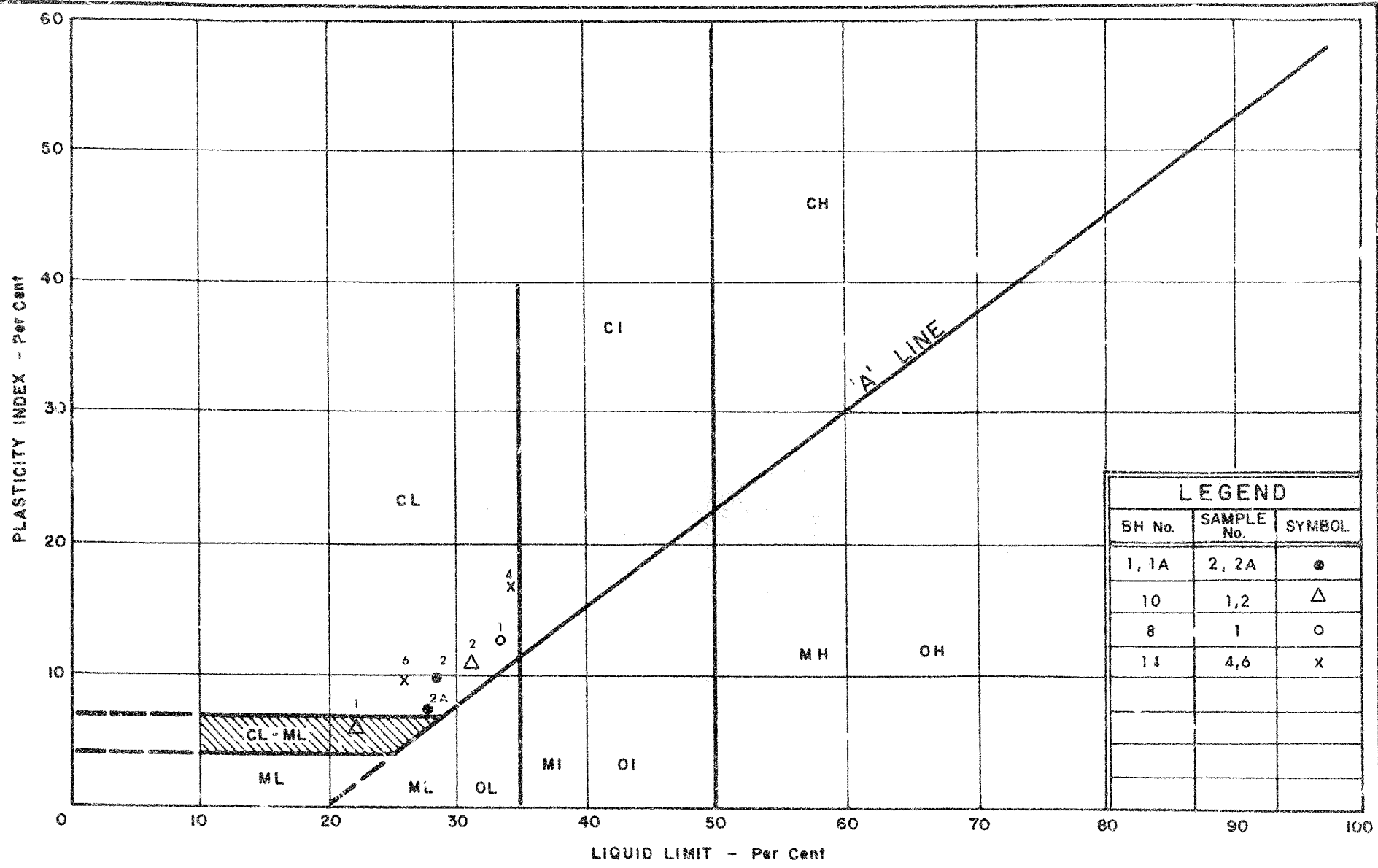


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART SANDY SILT TO SILTY SAND WITH ORGANICS AND CLAY

WP. No. 296-65

JOB No. 67-F-42



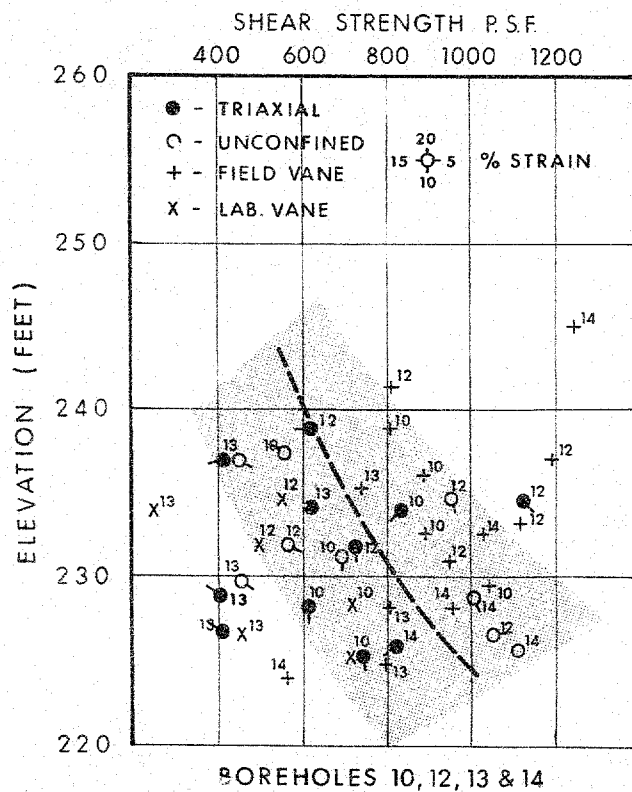
LEGEND		
BH No.	SAMPLE No.	SYMBOL
1, 1A	2, 2A	●
10	1,2	△
8	1	○
14	4,6	x



DEPARTMENT OF HIGHWAYS
 MATERIALS and
 TESTING
 DIVISION

PLASTICITY CHART CLAYEY SILT WITH SAND & GRAVEL

W.P. No. 296-65
 JOB No. 67-F-42

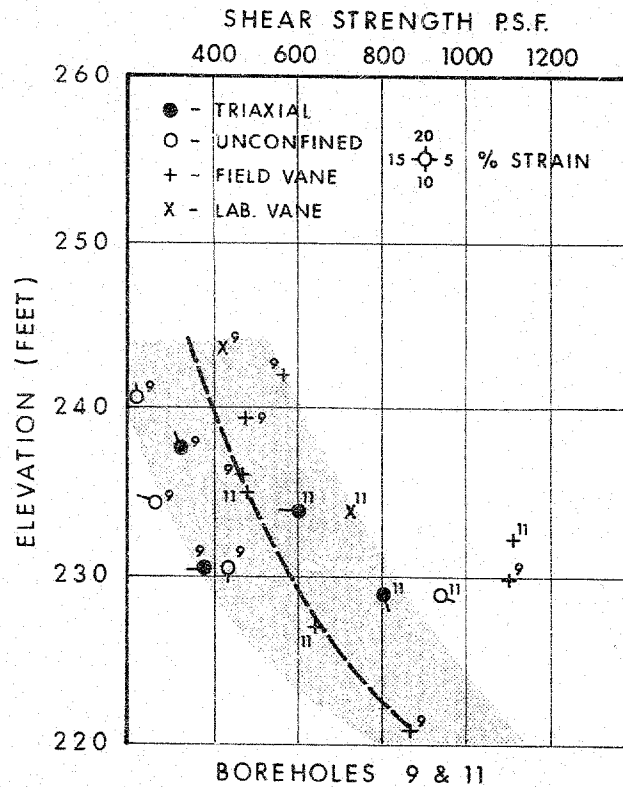


VALUES WITH GREATER PROBABLE DEGREE OF ERROR

HIGHER
VALUES

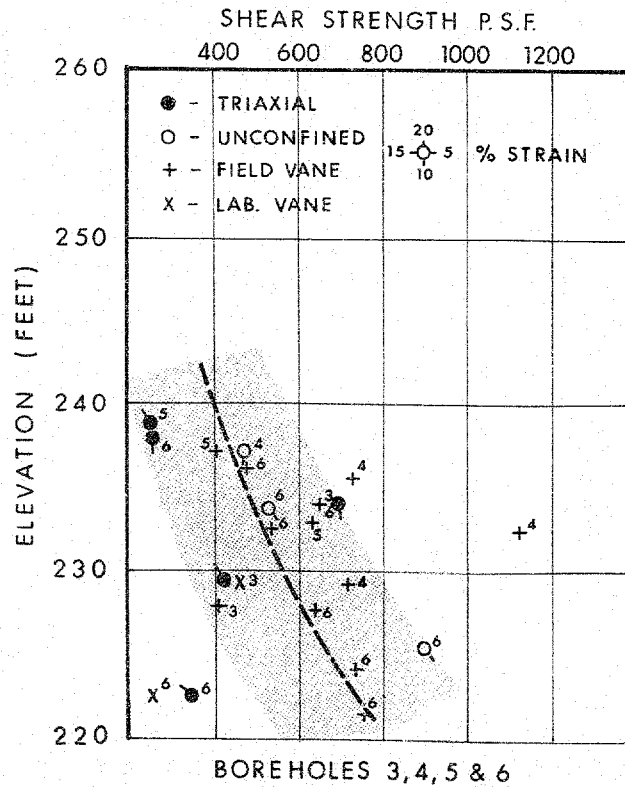
LOWER
VALUES

TYPE OF TEST	BH NO.	POSITION (ELEV.) ±	SHEAR STRENGTH P.S.F.	REASON
+	14	245	1280	GRAVEL IN SAMPLE
+	12	242	800	" " "
+	12	237	1200	ROOTS IN SAMPLE
●	12	235	1135	STRAIN 9%
+	12	233	1120	ROOTS IN SAMPLE
X	13	234	256	ORGANIC SILT
○	13	229	434	ORG. CLAY SILT, HI W
●	13	229	404	CL. SI. - SI., STRAIN 16%
X	13	227	458	ORG. SILT DISTURBED
●	13	227	416	" " "
+	14	224	560	" " , HIGH W



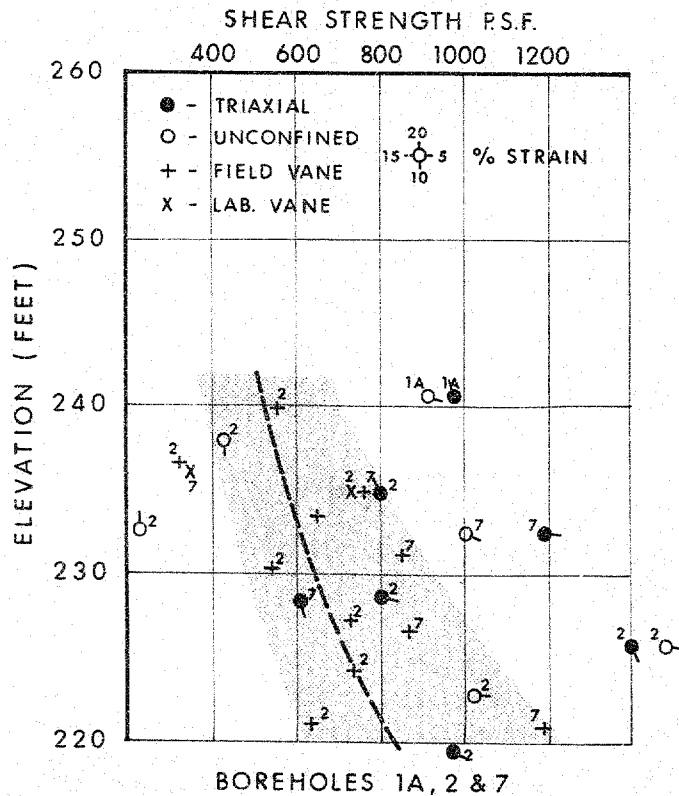
VALUES WITH GREATER PROBABLE DEGREE OF ERROR

	TYPE OF TEST	BH NO.	POSITION (ELEV. ±)	SHEAR STRENGTH P.S.F.	REASON
HIGHER VALUES	+	11	2 3 2	1120	CLAYEY SILT STIFF
	+	9	2 3 0	1120	CLAYEY SILT STIFF
	Q	11	2 2 9	940	ORG. SI. CL., STRAIN 6%
LOWER VALUES	○	9	2 3 4	260	ORG. CL. SI., STRAIN 16%



VALUES WITH GREATER PROBABLE DEGREE OF ERROR

	TYPE OF TEST	BH NO.	POSITION (ELEV. ±)	SHEAR STRENGTH P.S.F.	REASON
HIGHER VALUES	+	4	236	720	ROOTS IN SAMPLE
	+	4	232	1120	GRAVEL IN SAMPLE
LOWER VALUES	X	6	223	253	ORG. SILT DISTURBED
	●	6	223	364	" " "



VALUES WITH GREATER PROBABLE DEGREE OF ERROR

	TYPE OF TEST	BH NO.	POSITION (ELEV. ±)	SHEAR STRENGTH P.S.F.	REASON
HIGHER VALUES	○	1A	241	902	ORG. CLAYEY SILT WITH SILT POCKETS LOW W
	●	1A	241	995	AS ABOVE
	○	7	232	1010	ORG. SILT, STRAIN 6%
	●	7	232	1205	" " " "
	●	2	226	1430	" " LOW W
	○	2	226	1485	" " " "
LOWER VALUES	+	2	237	320	SOFT ORG. CLAY SILT
	X	7	236	355	DISTURBED
	○	2	232	216	ORG. SILT, STRAIN 20%

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario

ATTENTION:

DATE: July 8, 1968

OUR FILE REF:

IN REPLY TO

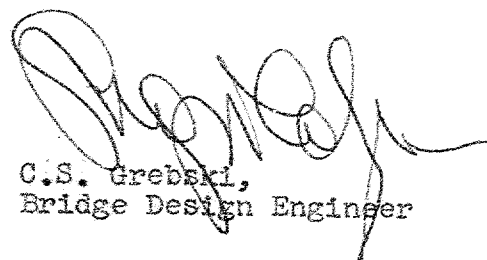
SUBJECT: Twelve Mile Creek Bridge
W.P. 296-65, Site 10-155
Highway 2, District No. 4

67-F-42

Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure.

Kindly give us your comments at your earliest convenience.

CSG:rd

for 
C.S. Grebski,
Bridge Design Engineer

NO Comments
July 10th 1968

W.L. Duly

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. H. Greenland,
District Engineer,
District No. 4,
HAMILTON, Ontario.

ATTENTION: Mr. D. Thrasher,
Constr. Engr.

OUR FILE REF

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

DATE: November 28, 1969.

IN REPLY TO

SUBJECT: 12 Mile Creek Bridge on Highway 2
West Abutment Foundation for -
Bailey Bridge, District #4 (Hamilton)
Cont. 69-13, W.P. 296-65, W.J. 67-F-42

This memo summarizes the main points of discussions recently held between yourself, Mr. A. G. Stermac, Mr. W. Birch and the writer regarding the Bailey Bridge foundations and east approach embankment presently under construction on the above mentioned contract.

(1) Deposits of soft organic silt exist under the east approach fill and east abutment foundation of the Bailey Bridge. In our view there is a definite risk of a base failure within the subsoil if the present design is proceeded with. This view has been somewhat reinforced by the experience at the west abutment. (See memo to H. Greenland - Nov. 7th, 1969).

(2) It is recommended that remedial measures be taken as soon as possible in order to ensure stability and future satisfactory performance of the bridge and approach embankment.

(3) A stable condition should be achieved if either of the two following methods are adopted:

(a) Increase the span of the bridge by at least 30 feet on the east side and found the abutment on piles driven to bedrock (el. 218+). Reduce the height of the existing embankment within the 30 feet, by half to act as a stabilizing berm.

(b) Construct the bridge as presently designed after removing the organic soil in the vicinity of the abutment down to el. 232+ and replacing it with suitable granular material. The width of the excavation base should be at least equal to the width of the detour roadbed and should extend from at least 10 feet forward of the front of the timber crib base to at least 10 feet behind the back of the crib base. The sides at the excavation should be sloped at 1:1. Existing fill should be partially removed to ensure stability during the excavation stage.

KGS:rk

c.c. Messrs. H. A. Tregaskes
D. M. Hopper
T. J. Kovich
W. D. Birch
Foundation Files
General Files

K. G. Selby,
Supervising Foundation Engr.
for.
A. G. Stermac,
Principal Foundation Engr.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. H. Greenland,
District Engineer,
District No. 4,
HAMILTON, Ont.

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

ATTENTION:

DATE: November 7, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT:

12 Mile Creek Bridge on Highway #2
West Abutment Foundation for -
Bailey Bridge, District #4 (Hamilton)
Cont. 69-13, W.P. 296-65, W.J. 67-F-42

The above mentioned site was visited by the writer and Mr. D. Smith, Project Soils Supervisor, on November 6, 1969 in response to a request by Mr. R. Brooks, Project Supervisor. It had been observed by Mr. Brooks that, the foundation being prepared for the Bailey Bridge West abutment was extremely soft, and vibrated considerably when compacting equipment passed over the area. During our visit, similar observations were made as a result of which, the following recommendations were given on site and to yourself by phone later in the day.

(1) The problem is the presence of soft organic silt as shown on Foundation Drawing #67-F-42A. This material should be removed down to el. 232.0 and replaced with suitable granular material.

(2) The width of the excavation base should be at least equal to the width of the detour roadbed and should extend from at least 10 ft. forward of the front of the timber crib base to at least 5 ft. behind the back of the crib base. The sides of the excavation should be sloped at about 1:1.

KGS/MdeF

cc: Messrs. H. A. Tregaskes
D. M. Hopper
T. J. Kovich
W. D. Birch

Foundations Files ✓
Gen. Files

K. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM

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

From: Bridge Division,
Downsview, Ontario.

Date: May 18th, 1967.

Our File Ref.

In Reply To

Subject: Bronte Creek Crossing,
Hwy. 2, Line 'A',
W.P. 296-65, District 4.

Enclosed please find 2 prints of the site plan E-4775-1 for the above structure. Probable footing locations are shown in red for a single span structure, and in blue for a 3 span spill through structure.

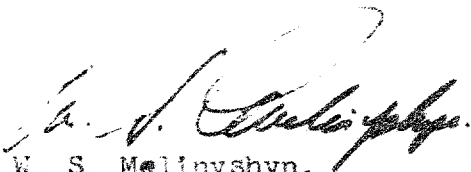
Would you please undertake a foundation investigation of sufficient scope to enable us to design the bridge.

Enclosed for your convenience is a site report. Would you also cover in your report the use of $1\frac{1}{2}$:1 rip-rapped slopes on the spill through embankments for the three span scheme.

WSM/cew

Encl.

cc R. Forrest
A. Crowley


W. S. Melnyshyn,
Regional Bridge Location Engineer

67-F-42

401 & Keele St.
Downsview, Ontario

June 20, 1967

Dominion Soil Investigation Ltd.
77 Crockford Blvd.
Scarborough, Ontario

Dear Sirs:

This is to confirm our request of April 28, 1967 for the supply of a Core Drill together with all necessary equipment, as specified under the terms of our Contract Agreement, at Bronte, Ontario.

This project bears Job Number 67-F-42.

Yours truly,



KS:mt

R. Selby
Supervising Foundation Engineer
for A. G. Stermac
Principal Foundation Engineer

401 & Keele St.
Downsview, Ontario

July 11, 1967

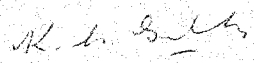
Master Soil Investigation
104 Kenhar Drive
Weston, Ontario

Dear Sirs:

This is to confirm our request of May 29, 1967 for the supply of a Diamond Drill together with all necessary equipment, as specified under the terms of our Contract Agreement, at Hwy.#2 & Bronte Creek, Oakville, Ontario, on May 30, 1967.

This project bears Job Number 67-F-42.

Yours truly,



KGS:mt

K. G. Selby
Supervising Foundation Engineer
for: A. G. Starnac
Principal Foundation Engineer

Mr. B. E. Davis,
Bridge Engineer,
Bridge Div., Admin. Bldg.
Attn: Mr. J. Harris, Hydrology.

2.

November 18, 1967

We would appreciate it if you would advise us as to your final hydrology recommendations for this project. If further discussion is required, please feel free to contact this Office.

H. G. Selby

ECB/maef

H. G. Selby,
SUPERVISING FOUNDATION ENGR.
PORT
A. G. Starnes,
PRINCIPAL FOUNDATION ENGR.

cc: MEMPHIS. B. E. Davis (2)
B. A. Trappesee
D. W. Parvett
C. E. Hunter (2)
B. Greenland
K. B. Melinashyn
T. J. Kovich
B. A. Singh
Foundations Files ✓
Gen. Files

Mr. W. E. Davis,
Bridge Engineer,
Bridge Division,
Main. Bldg.

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

November 16, 1967

Attention: Mr. J. Harris,
Senior Bridge
Hydrology Engr.

Brooks Creek and Hwy. 72, Line 18'
S.D. 206-65 -- S.D. 67-8-60

We have reviewed the Hydrology Report for the above mentioned project by James E. Amelgren Ltd., with particular regard to the comments relating to the stability of the proposed approach embankments, and to the proposed subs excavation of the organic material within the stream bed. As a result of our review, we feel that certain recommendations contained in our foundation report require re-emphasizing and enlarging.

(1) Our recommendations pertaining to the stream bed specify replacing the existing material by the same amount of granular fill and rip-rap. It was not recommended to fill to a higher level than the present stream invert.

(2) It is understood that the 'effective' stream invert is that level to which the stream would scour during flood conditions. It is from this level that the cross sectional area of the waterway required is calculated by the Hydrologist. If this is the case, we would recommend that subs excavation be carried out to a depth of at least 4 feet below the 'effective' invert level so as to provide at least 4 feet of new fill material, the top 2 feet of which should be rip-rap suitable for scour protection. At locations other than the actual stream bed, subs excavation and filling should be carried out according to our Foundation Drawing 67-7-425.

(3) Rip-rap should be continuous from the stream bed up the slopes of the approaches so as to permit no scouring out of fill material. This is important from the point of view of slope stability.

(4) The Hydrology Consultant's comments in (ii) Page 31, appear to have arisen from a misunderstanding by him of our intentions, and of the purpose of the fill material. This matter has now been clarified in a telephone discussion between the writer and Mr. E. Chisholm.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario.

ATTENTION: Mr. K. G. Selby.
OUR FILE REF.

DATE: November 20th. 1967.

IN REPLY TO

SUBJECT: Bronte Creek at Hwy. #2,
W.P. 296-65 Site No. 10-155,
District #4 - BW 1527.

In answer to your request of November 16th. 1967 regarding the Consultant's hydrology recommendations for the bridge waterway, I have summarized these below for your convenience.

1. Required waterway area 1650 square feet measured at right angles to channel between elevations 229 and 244.6. This is equivalent to a span of 106' (measured at el. 237± if trapezoidal section).
2. Location and skew approximately as shown on foundation report drawing 67-F-42B.
3. New stream bed to be at elevation 229 or lower if a non-erodible invert is to be provided as per Foundation Section recommendations.

These recommendations are concurred with, although the new stream bed could be placed at up to el. 232 if required, provided that the span is increased correspondingly to maintain the 1650 square foot opening.

We will be glad to discuss the project further if required.

JDH/ss

cc. B. Davis
James F. MacLaren Ltd.
H. A. Tregaskes
D. W. Farren
G. K. Hunter
H. Greenland
W. S. Melinyshyn
T. J. Kovich
B. A. Singh


J. D. Harris,
Bridge Hydrology Engineer.

MEMORANDUM

Telephone: 248-3415

To:

Mr. A.G. Stermac,
Principal Foundation Engineer,
Materials and Testing Division,
Laboratory Building

FROM:

Mr. A.G. Kelly,
Toronto Regional Road Design

DATE:

January 2nd, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

Re: Work Project 296-65, Proposed New Bridge at
Bronte Creek and Highway 2, Oakville,
District 4, Hamilton

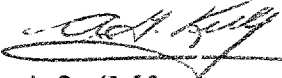
Road Design is presently preparing Contract Documents for the above work and are concerned with the proposed detour.

Having reviewed the recommendations of your Foundation Investigation Report together with the Soils Report, it appears that in order to employ a detour on the north side it would be necessary to either span 400 feet with a series of Bailey Bridges founded on piles, or to use approach fills with a shorter span Bailey Bridge which, due to the instability of the east detour fill and the proximity of the excavation for the new highway fill, would probably require sub-excavation of and selected backfilling of the east detour approach. Either scheme will probably be costly.

On the other hand, a southerly detour will have better alignment, more shallow fills and probably reduced foundation problems. This side was never considered for a detour due to the proximity of a restaurant at Station 415⁺ and a frame shed at Station 412⁺. The Municipality owns the moveable frame shed and the restaurant will have to be relocated due to entrance difficulties.

A visual observation in the field leads me to believe that the southerly detour would not require extensive foundation work.

I solicit your comments as to the foundation treatment required for both the north and south detour.


A.G. Kelly,
Senior Project Design Engineer,
FOR: G.K. Hunter,
Regional Road Design Engineer

AGK/bap
encl.

c.c. W.S. Melinyshyn
T. Kovich
W. Birch

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac

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

Bridge Division,
Downsview, Ontario

January 16, 1968

Twelve Mile Creek Bridge
W.P. 296-65, Site 10-155
Highway 2, District No. 4

Attached herewith are prints of the Preliminary Bridge Plan Drawing D-6331-F for the above-mentioned structure.

The estimated cost of the proposed structure is \$200,000. This cost includes tender, materials, engineering and sundry construction.

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

CSO:rd

C.S. Greboki,
Bridge Design Engineer

Attach.

c.c. S. McCombie
A. Stermac (2)
J. Anderson

Letter sent to C. Greboki.
Jan. 31st 1968

W. L. Sullivan

MEMORANDUM

To: Mr. G. K. Hunter,
Regional Road Design Engineer,
Toronto Region,
Central Bldg.

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

Attention: Mr. A. G. Kelly,
Senior Project
Design Engr.

DATE: January 18, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

Proposed Detour, Bronte Creek & Hwy. 2
District No. 4 (Hamilton)

W.P. 296-65 -- W.J. 67-F-42

PLEASE INCLUDE THIS MEMO WITH YOUR COPY(S)
OF FOUNDATION REPORT W.J. 67-F-42.

We have reviewed your proposals for a temporary detour at the above mentioned site. We are of the opinion that the south line would be the most economical proposition. Borings recently carried out in the field by the Regional Materials Engineer, show that soil conditions from the east bank of the river back to Sta. 411+00, correspond approximately to B.H. 9 (Report 67-F-42) apart from the existence of 6 to 7 feet of fill on the surface.

We recommend that the east abutment of the Bailey Bridge be located at or east of Sta. 412+30, and that the height of the new fill be kept to a maximum of 10 feet or lower, if possible. Settlements of the east approach are likely to occur and frequent maintenance will be required to ensure a satisfactory road surface during the life of the detour.

The west approach should present no problems regarding stability or settlement if the existing river bank on the west side is not steepened and new fill is not placed so as to spill forward into the river. Both abutments should be founded on end-bearing piles driven to rock. Assuming that the west abutment will be located at approximate Sta. 414+30, refusal for the piles should be at El. 227½. At the east abutment, refusal should be met at El. 217½.

If it is desired to construct the detour on the north side of Hwy. 2, the east approach to the Bailey Bridge should be constructed according to the recommendations contained in Report 67-F-42, pertaining to the new portion of Hwy. 2, since the site is virtually a swamp and much less favourable than the south side which is at least partially consolidated. There, too, the bridge should be supported on end-bearing piles and refusal should be met at roughly the same elevations as for the bridge on the south detour. No problems should be encountered for the west approach, and our recommendations are the same as for the south detour.

Mr. G. K. Hunter
Regional Road Design Engr.,
Toronto Region,
Central Bldg.

2.

Attn: Mr. A. G. Kelly,
Sr. Project Design Engr.

January 18, 1968

If you have any further queries concerning this matter,
please contact this Office.

KGS/MdeF

H. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

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

Foundations Files ✓
Gen. Files

WJS

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

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

February 1, 1968

-- TWELVE MILE CREEK BRIDGE --
W.P. 296-65 -- W.J. 67-F-42
Hwy. 2 - District #4 (Hamilton)

We have reviewed Preliminary Bridge Plan D-6331-P
for the above structure.

We note that the drawing does not show the subexcavation
of organic material within the stream bed and under the East
abutment as recommended in our foundation report, and later revised
in our memo to Mr. J. Harris, dated November 16, 1967. The latter
revision followed our review of the Hydrology Report by James F.
MacLaren Ltd.

Regarding the penetration of the closed end 18" Ø tube
piles 6 inches into the rock by hammering, we believe that this
may be extremely difficult unless a special shoe with a fairly
sharp cutting edge is employed. The shoe itself, may penetrate
but it is doubtful if the tube pile itself, could be socketed into
the sound rock. More than likely it would penetrate an inch or
two into rock shattered by the shoe. We have had experience in
the past where attempts to hammer 12-3/4 O.D. steel tubes into
shale bedrock resulted in buckling of the pile tips and no
penetration. On the other hand, we believe that H-piles could be
driven satisfactorily into the rock using the same driving
technique as for Oslo Points. In the shale bedrock, of course,
no Oslo Points need be fitted. Our experience in the past -
(Gibson Lake & #406) has shown that H-piles can be driven 6 - 8
inches into harder rock than shale, relatively simply, without
damage to the pile tips.

K. G. Selby

KGS/maef

cc: Messrs. S. McCombie
W. S. Melinyshyn
Foundations Files
Gen. Files

K. G. Selby,
SUPERVISING FOUNDATION ENGINEER
For:
A. G. Stermao,
PRINCIPAL FOUNDATION ENGINEER

Department of Highways Ontario
Copy for the information of
Mr. A. Stermac

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

Bridge Division,
Downsview, Ontario

February 19, 1968

Twelve Mile Creek Bridge
N.P. 295-65, Site 10-195
Highway 2, District No. 4

Attached herewith are prints of the revised Preliminary Bridge Plan Drawing B-6331-P2 for the above-mentioned structure.

The estimated cost of the proposed structure is \$231,000. This cost includes tender, materials, engineering and sundry construction.

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

CAG:rd

C.E. Grehald,
Bridge Design Engineer

Attach.

C.C. S. McCombie
A. Stermac (2)
J. Anderson

Feb. 27th 1968

Comments:-

Excavation and backfill of organic material should be completed prior to constructing the new bridge.
(Refer to Fdn. Report Page 14)

Your note on bottom of Drawing implies that excavation may be carried out after building the bridge.

Otherwise no comments

W. L. Gully

MEMORANDUM

Telephone: 248-3415

TO: Mr. W.D. Birch,
Bridge Maintenance Engineer,
Maintenance Division,
Laboratory Building

FROM: Mr. A.G. Kelly,
Toronto Regional Road Design

DATE: March 4th, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

Re: Proposed Bailey Bridge Detour,
Bronte Creek, Highway 2, Oakville,
Work Project 296-65

67-F-42

Attached are a plan, a profile, and an alignment drawing for the proposed Bailey Bridge Detour at the above site.

The following is Road Design's Design Data:-

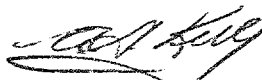
Grade Shown - Top of Pavement

Approach Standards:-

- Pavement 22'
- Shoulder 5' (including 2' rounding)
- granular depth - 12" sand cushion
 - 6" GBC 'A'
- Pavement Depth - 2" HL-6

A letter to the undersigned from K.G. Selby, dated January 18th, 1968, outlines the Foundation Section's recommendations. (NOTE: The Stations stated in the Foundation's letter refers to Highway 2. The new detour alignment changes these Stations to detour Station 412+50 and 414+50).

Road Design requests the preparation of Bailey Bridge Drawings, Quantities, and Estimates for inclusion in the Contract Drawings.



A.G. Kelly,
Senior Project Design Engineer,
FOR: G.K. Hunter,
Regional Road Design Engineer

AGK/bap
attach.

c.c. A.G. Stermac ✓
T. Kovich
W. Melinyshyn

J. MARCH 6P

OK.

A.K.B

DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS

Form
SB-OS-62

ACTION SLIP

DATE March 10, 1972

TO Mary Topolski - Room 109
Central Bldg.

FROM

Amand

- | | |
|--|---|
| <input type="checkbox"/> NOTE AND FILE | <input type="checkbox"/> PREPARE REPLY FOR MY SIGNATURE |
| <input type="checkbox"/> NOTE AND RETURN TO ME | <input type="checkbox"/> TAKE APPROPRIATE ACTION |
| <input type="checkbox"/> RETURN WITH MORE DETAILS | <input type="checkbox"/> PER YOUR REQUEST |
| <input type="checkbox"/> NOTE AND SEE ME | <input type="checkbox"/> FOR YOUR SIGNATURE |
| <input type="checkbox"/> PLEASE ANSWER | <input type="checkbox"/> FOR YOUR INFORMATION |
| <input type="checkbox"/> FOR YOUR APPROVAL | <input type="checkbox"/> INVESTIGATE AND REPORT |
| <input type="checkbox"/> RETURN WITH YOUR COMMENTS | <input type="checkbox"/> AS DISCUSSED |

COMMENTS

This is all I
found so far.

Cheryl

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

RE: W. J. B. B. B. B.
S. P. (BAILEY BR.)
RECOMMENDATIONS
12 MILE C.R. BR.
ON HIGHWAY 2

To: Mr. H. Greenland,
District Engineer,
District No. 4,
HAMILTON, Ont.

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

ATTENTION:

DATE: November 7, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT:

12 Mile Creek Bridge on Highway #2
West Abutment Foundation for -
Bailey Bridge, District #4 (Hamilton)
Cont. 69-13, W.P. 296-65, W.J. 67-F-42

The above mentioned site was visited by the writer and Mr. D. Smith, Project Soils Supervisor, on November 6, 1969 in response to a request by Mr. R. Brooks, Project Supervisor. It had been observed by Mr. Brooks that, the foundation being prepared for the Bailey Bridge West abutment was extremely soft, and vibrated considerably when compacting equipment passed over the area. During our visit, similar observations were made as a result of which, the following recommendations were given on site and to yourself by phone later in the day.

(1) The problem is the presence of soft organic silt as shown on Foundation Drawing #67-F-42A. This material should be removed down to el. 232.0 and replaced with suitable granular material.

(2) The width of the excavation base should be at least equal to the width of the detour roadbed and should extend from at least 10 ft. forward of the front of the timber crib base to at least 5 ft. behind the back of the crib base. The sides of the excavation should be sloped at about 1:1.

KGS/MdeF

cc: Messrs. H. A. Tregaskes
D. M. Hopper
T. J. Kovich
W. D. Birch

Foundations Files
Gen. Files

K. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

OK

MEMORANDUM

W.P.-296-65-2
Re Proposed Detour
Bronte Creek
Hwy 2

To: Mr. G. K. Hunter,
Regional Road Design Engineer,
Toronto Region,
Central Bldg.

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

Attention: Mr. A. G. Kelly,
Senior Project
Design Engr.

DATE: January 18, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

Proposed Detour, Bronte Creek & Hwy. 2
District No. 4 (Hamilton)
W.P. 296-65 -- W.J. 67-F-42

PLEASE INCLUDE THIS MEMO WITH YOUR COPY(S)
OF FOUNDATION REPORT W.J. 67-F-42.

We have reviewed your proposals for a temporary detour at the above mentioned site. We are of the opinion that the south line would be the most economical proposition. Borings recently carried out in the field by the Regional Materials Engineer, show that soil conditions from the east bank of the river back to Sta. 411+00, correspond approximately to B.H. 9 (Report 67-F-42) apart from the existence of 6 to 7 feet of fill on the surface.

We recommend that the east abutment of the Bailey Bridge be located at or east of Sta. 412+30, and that the height of the new fill be kept to a maximum of 10 feet or lower, if possible. Settlements of the east approach are likely to occur and frequent maintenance will be required to ensure a satisfactory road surface during the life of the detour.

The west approach should present no problems regarding stability or settlement if the existing river bank on the west side is not steepened and new fill is not placed so as to spill forward into the river. Both abutments should be founded on end-bearing piles driven to rock. Assuming that the west abutment will be located at approximate Sta. 414+30, refusal for the piles should be at El. 227±. At the east abutment, refusal should be met at El. 217±.

If it is desired to construct the detour on the north side of Hwy. 2, the east approach to the Bailey Bridge should be constructed according to the recommendations contained in Report 67-F-42, pertaining to the new portion of Hwy. 2, since the site is virtually a swamp and much less favourable than the south side which is at least partially consolidated. There, too, the bridge should be supported on end-bearing piles and refusal should be met at roughly the same elevations as for the bridge on the south detour. No problems should be encountered for the west approach, and our recommendations are the same as for the south detour.

cont'd. /2 ...

Mr. G. K. Hunter,
Regional Road Design Engr.,
Toronto Region,
Central Bldg.

2.

Attn: Mr. A. G. Kelly,
Sr. Project Design Engr.

January 18, 1968

If you have any further queries concerning this matter,
please contact this Office.

KGS/MdeF

K. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

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

Foundations Files
Gen. Files ✓

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

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

π	$= 3.1416$
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

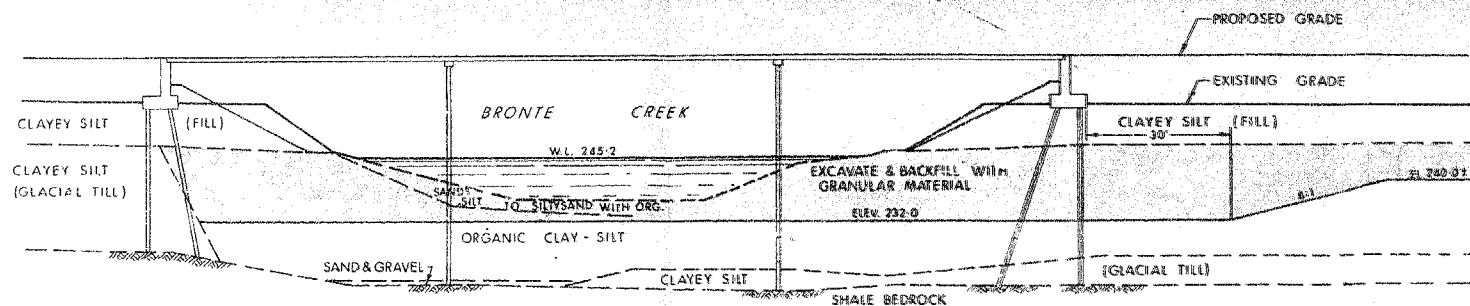
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

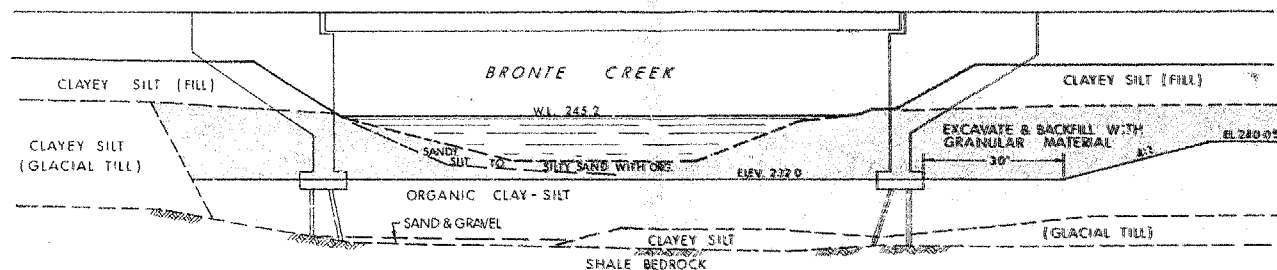
SLOPES

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



☐ PROFILE - LINE 'A' (3 SPAN STRUCTURE)

SCALE 1" = 20'

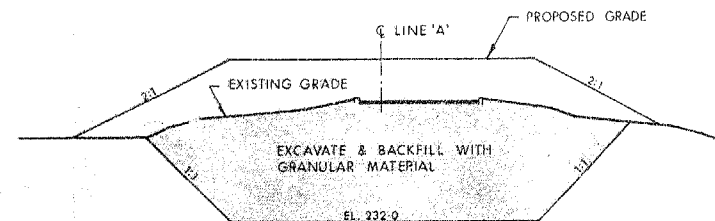


☐ PROFILE - LINE 'A' (1 SPAN STRUCTURE)

SCALE 1" = 20'



TYPICAL CROSS-SECTION
☐ STRUCTURE STA. 413+19
BOTH STRUCTURES



TYPICAL CROSS-SECTION
STA. 412+00

3 SPAN STRUCTURE

	DEPARTMENT OF HIGHWAYS	
	MATERIALS and TESTING DIVISION	
ONTARIO	DATE 10 AUGUST 1967	APPROVED <i>A. J. Kermack</i>

HIGHWAY NO 2 & BRONTE CREEK		
PROPOSED SUB - EXCAVATION		
W. P. 296-65	DIST. 4	JOB 67-F-42
DRAWING NO. 67-F-42 B		

REPORT TO THE
DEPARTMENT OF HIGHWAYS
OF ONTARIO
ON HYDROLOGIC INVESTIGATIONS
OF BRONTE CREEK BRIDGE
HIGHWAY 2 DISTRICT 4

INDEX

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SECTION 2	Hydrology
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SECTION 8	Acknowledgements

REFERENCE NO. 870

October 27, 1967.

Mr. J. Harris, P. Eng.,
Senior Bridge Hydrology Engineer,
Bridge Division,
Department of Highways of Ontario,
Downsview, Ontario.

Report on
Hydrologic Investigations
WP 296-65 Bronte Creek Bridge
Highway #2 - District A

Dear Sir:

On May 17, 1967, our Firm was authorized, on behalf of the Department of Highways of Ontario, to undertake a hydrologic study for the proposed bridge over Bronte Creek at Highway #2 in the Town of Oakville.

We now take pleasure in reporting our findings and recommendations resulting from this study. Our investigations were conducted in accordance with the terms of reference outlined in the agreement between the Department and our Firm, dated May 26, 1967. These are as follows:

1. Field hydrologic and hydraulic investigation of the watershed, existing bridges, special features of the river and the proposed crossing, including the collection of flood data.
2. Hydrologic and hydraulic studies and computations, including,
 - (a) collection of necessary records,
 - (b) calculation of design flood,
 - (c) study of foundation report and recommendations,
 - (d) hydraulic calculations for bridge waterway, including consideration of scour, erosion and backwater insofar as these are applicable to the site in question,

October 27, 1967.

Mr. J. Harris - 2

- (e) consideration of scour protection required for foundations,
 - (f) determination of most suitable location of bridge, angle of skew, minimum soffit elevation, dimensions of required river diversion, and necessity for relief structures, and
 - (g) determination of rip-rap and other miscellaneous hydraulic requirements.
3. Submission of calculations and other relevant material to the employer for review and discussion prior to finalization of the Report.
 4. Preparation of Bridge Hydrology Report, with final recommendations.
 5. Such other services as may be required in connection with and subsequent to the final report.

We trust that the information and recommendations submitted herein will be adequate for these requirements. Should additional information be required in this regard, we shall be pleased to discuss the matter with you at your future convenience.

Yours very truly,

P. S. Chisholm:GS

H. Pennerty
Executive Vice-President.

SECTION 2 - HYDROLOGY

(a) General - Bronte Creek Watershed

(i) Drainage Area Definition

The watershed of Bronte Creek includes portions of Halton, Wentworth and Wellington Counties. It falls within the jurisdiction of the Halton Region Conservation Authority.

The principal drainage axis of the study basin is 22 miles long oriented generally in a north-west south-east direction. Bronte Creek enters Lake Ontario at the Village of Bronte. The lower eight mile reach of the water course is contained within a narrow $3/4$ mile wide valley which extends upstream to the Village of Zimmerman. Above that location, the basin broadens rapidly to an average width of $6\frac{1}{2}$ miles. The maximum width of $8\frac{1}{2}$ miles occurs at the upper limit of the basin between the Towns of Darbyville and Strabane. The study area may be approximated by the triangular area of land between these two towns and the Town of Bronte. The watershed comprises 135 square miles and the main stream length is 32 miles.

(ii) Topography

The Bronte Creek watershed is separated into two distinct topographical regions by the Niagara Escarpment. This feature traverses the basin in a general north-east south-west direction approximately

at the mid-point of the principal axis. Due to the irregular shape of the basin almost 70% of the watershed area lies above the escarpment (elevation 825.00) while the remaining 30% comprises the side slopes of the escarpment itself and the area between the escarpment and the mouth of the creek at Lake Ontario.

The source of the main stream of Bronte Creek is about 2 miles north-east of Morriston at an elevation of about 1050 feet above sea level, approximately 800 feet above Lake Ontario. The area above the escarpment consists of a well defined drumlin field over the northern extent, evident in frequent and well rounded hills. A smooth, gently sloping limestone plain extends from the southern extent of the drumlins to the top of the escarpment ridge. Within this area, the main stream and its tributaries wind through the valleys between the drumlins and meander across the flatter plain area. The average river gradient above the escarpment is approximately 15 feet per mile.

Downstream of the escarpment the water course falls over the ridge for a short distance at a steep gradient, and then levels out as it traverses the clay till plains, and till moraine areas between the escarpment and Lake Ontario. The average gradient in the main stream over the lower reach is approximately

25 feet per mile.

(iii) Lag Time

For this study, lag time is defined as the time interval between the centroid of rainfall excess for a given storm and the resulting flood peak at the mouth of the creek at Lake Ontario.

From the preceding discussion, it may be seen that the gradients of Bronte Creek and its tributaries are reasonably high with only localized slack water reaches. In general, these steep slopes provide high velocity flows with strong erosive capacity.

Unfortunately, existing streamflow records for Bronte Creek are not adequate to define a unique lag time for this basin. However, upon comparing the study area to other watersheds in Southern Ontario for which lag time data are available, a representative lag time of 12 hours was selected for the Bronte Creek watershed and subsequent analyses.

(iv) Physiography and Land Use as it Affects Runoff

As discussed in section (ii), the area above the escarpment has developed on a limestone plain. The northern extent of this portion is complicated by the presence of drumlins while the southern portion has a gently rolling topography. The major soil developed over this area is a Grey Brown Podzolic, stony, sandy, loamy till. This soil is well drained internally and generally has high

lower zone water storage capacity. The presence of stones within the characteristic till has discouraged widespread crop farming and led to a high percent of pasture land. Where such grass lands have been over worked or unprotected, severe erosion has occurred and high degree of runoff is often experienced. Where crop farming has been attempted, ploughing practice is generally poor. Severe gullying is evident throughout the study area indicating uncontrolled surface drainage.

Below the escarpment, the soil has been developed on clay, till plains, and till moraine. It is again a clay loam with fewer stones and is in general well drained. Orchards are common in this area and the land use reflects more careful management.

(b) Hydrologic Analyses

(1) Background

Where river control structures are concerned, it is not the normal or average discharges of the stream, but the unusual or less frequent flows that have occurred in the past or may reasonably be expected to occur in the future that define the discharge design criteria.

In Southern Ontario, high flows and associated flooding conditions in the watercourses and their flood plain areas result primarily from two independent combinations of hydrologic phenomena.

In the first case, spring freshet flood peaks from the melting of snow each spring, combine with spring rainfall. At this time of the year, either saturated or frozen ground conditions prevail, and a high percentage of both the melting snow and the spring rainfall contributes to direct runoff. Due to the frequent alternating periods of freezing and thawing each spring, several significant peak freshet floods occur each year for each stream. However, for this study only the maximum average day for each year of data for a given stream has been considered.

The second category of significant flooding conditions is produced by summer thunderstorms. These storms may be of variable intensity and duration.

On the basis of the foregoing, a peak flow-frequency relationship was synthesized by the unit-hydrograph method for summer thunderstorms over the Bronte Creek watershed, and compared with the characteristics of likely spring freshet peak floods for Bronte Creek.

(ii) Spring Freshet Peak Flow - Frequency Relationship

As discussed in section (a), available stream flow data for Bronte Creek are not complete enough to establish flood frequency relationships. However, this problem was overcome by transposing data from gauged watersheds in Southern Ontario, selected on the basis of period of record and climatological similarity to that of the study area. Initially, twenty-two basins of similar drainage area were investigated. However, with the further considerations of period of record, general physiography, and climate, data from only three of these were selected for transposition to Bronte Creek. These basins and their respective gauging station locations are as follows: The Thames River at Woodstock, The Middle Thames River at Thamesford and The Ganaraska River at Dale.

Unfortunately, while these stations provided upwards to 20 years of Maximum Mean Daily Flow data, continuous gauge recorders have not been in use over the entire period of record. Instantaneous peak flow values have been recorded only since 1963. Accordingly, a correlation between maximum mean daily flow values and corresponding instantaneous peak flows for these data was undertaken. On the basis of the limited data available, it was concluded

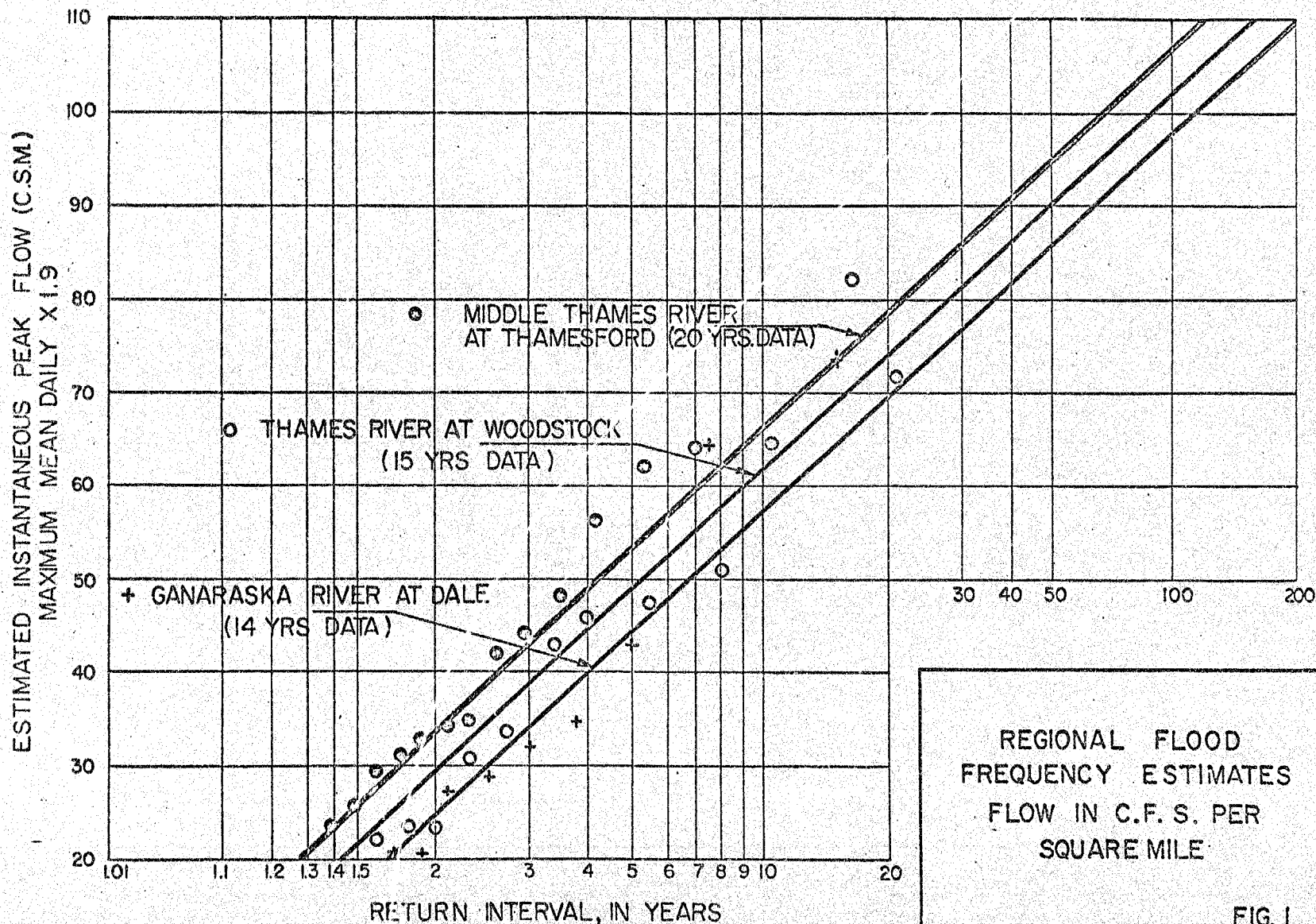


FIG. 1

that the instantaneous peak discharges for freshet floods were approximately $1.90 \times$ maximum mean daily flows. Applying this factor to the data of the above noted basins, Figure 1 was developed. This provided a representative basis for estimating the spring freshet peak flow - frequency relationship for the ungauged Bronte Creek watershed.

(iii) Synthesis of Summer Flood Peak Flow - Frequency Relationship

As discussed in section (i) above, summer thunderstorms may lead to severe flooding over a watershed such as that of Bronte Creek. To forecast peak discharges produced by such a storm, it is first necessary to establish what proportion of a given rainfall amount will comprise the direct runoff. The runoff component, referred to as "excess rainfall" forms the bulk of the flood volume. The corresponding peak flow is dependent upon the rate of generation of this excess and the temporal distribution of the excess during the storm under consideration.

For the purposes of this section of the study, total precipitation resulting from six hour duration storms were investigated. Storm totals of this nature, expected to occur with an average frequency of once in fifty years and once in one hundred years were selected from regional rainfall frequency data and

extrapolations thereof. The total rainfall excess and hourly distribution throughout each six hour period were determined according to the methods of Chapter II - Flood Studies of the publication "Design of Small Dams".

The rainfall excess sequences so defined were applied to synthetic one hour unit hydrographs to produce peak flow estimates for each storm. The return period of these peak flows is assumed to be the same as the return period of the associated rainfall.

It is acknowledged that this assumption is not completely justifiable. However, the investigations associated with establishing the authority of this practice are not within the scope of the present study.

(c) Comparison of Peak Flow Estimates

The results of the spring freshet peak flow-frequency investigations are illustrated in Figure 1. The maximum range in the unit flow predictions of the frequency curves for the three rivers is 8 csm (cubic feet per second per square mile) of tributary area.

On the basis of this large range in unit flow predictions, a secondary and more detailed investigation of the physiographic similarity of those drainage basins to that of the Bronte Creek basin was undertaken. This investigation concluded that the two branches of the Thames River were

adequately similar for the purposes of transposing flow data, to the Bronte Creek basin, while the Ganaraska River was not. Accordingly, it is recommended that the Ganaraska River data not be considered in applying the results of the flood frequency analysis illustrated in figure 1. On this basis, the forecast range is reduced from 8 csm to 3 csm.

In the absence of data pertaining to the particular basin under investigation, a range in peak flows of 3 csm is not unreasonable. Since there is no justification of recommending one of the Thames branches over the other on the basis of physiographic similarity, it is suggested herein that the curve giving the more conservative values, in particular, the curve associated with the Middle Thames River be considered representative of the Bronte Creek drainage basin.

The results of the comparison of peak flow estimates established on the above basis are presented in Table 1:

Table 1 Seasonal Flood Frequency

<u>Return Period</u>	<u>Spring freshet Peak Flow c.f.s.</u>	<u>Summer Storm Peak Flow c.f.s.</u>
50 years	12,700	3,900
100 years	14,500	4,100

It is concluded from these data, that the design of the proposed structure should be based upon the hydraulic considerations associated with a spring freshet peak flow of from 12,000 to 15,000 c.f.s.

SECTION 3 - FIELD INVESTIGATIONS

(a) Background

Subsequent to our receipt and review of the foundation report a field reconnaissance was carried out for the following purposes:

- (i) to obtain additional field data on factors relating to the hydraulic characteristics of the site;
- (ii) to obtain additional information, from local residents, on the characteristics of past flooding experience for the purpose of complementing limited available stream flow data.

(b) General

The site for the proposed bridge is located within the Town of Oakville, at the existing crossing of Highway No. 2 and Bronte Creek. A broad delta marsh area extends upstream of the crossing for approximately seven tenths of a mile. The marsh area is generally free of encroachment except for limited fill observed on both stream banks in the vicinity of the existing bridge. Three bridge crossings of the Bronte Creek were investigated in the course of our review. These include:

- (i) the existing Highway No. 2 bridge;
- (ii) the former railway crossing (now abandoned) located approximately one mile upstream of Highway No. 2;
- (iii) the crossing of Bronte Creek at the Queen Elizabeth Highway.

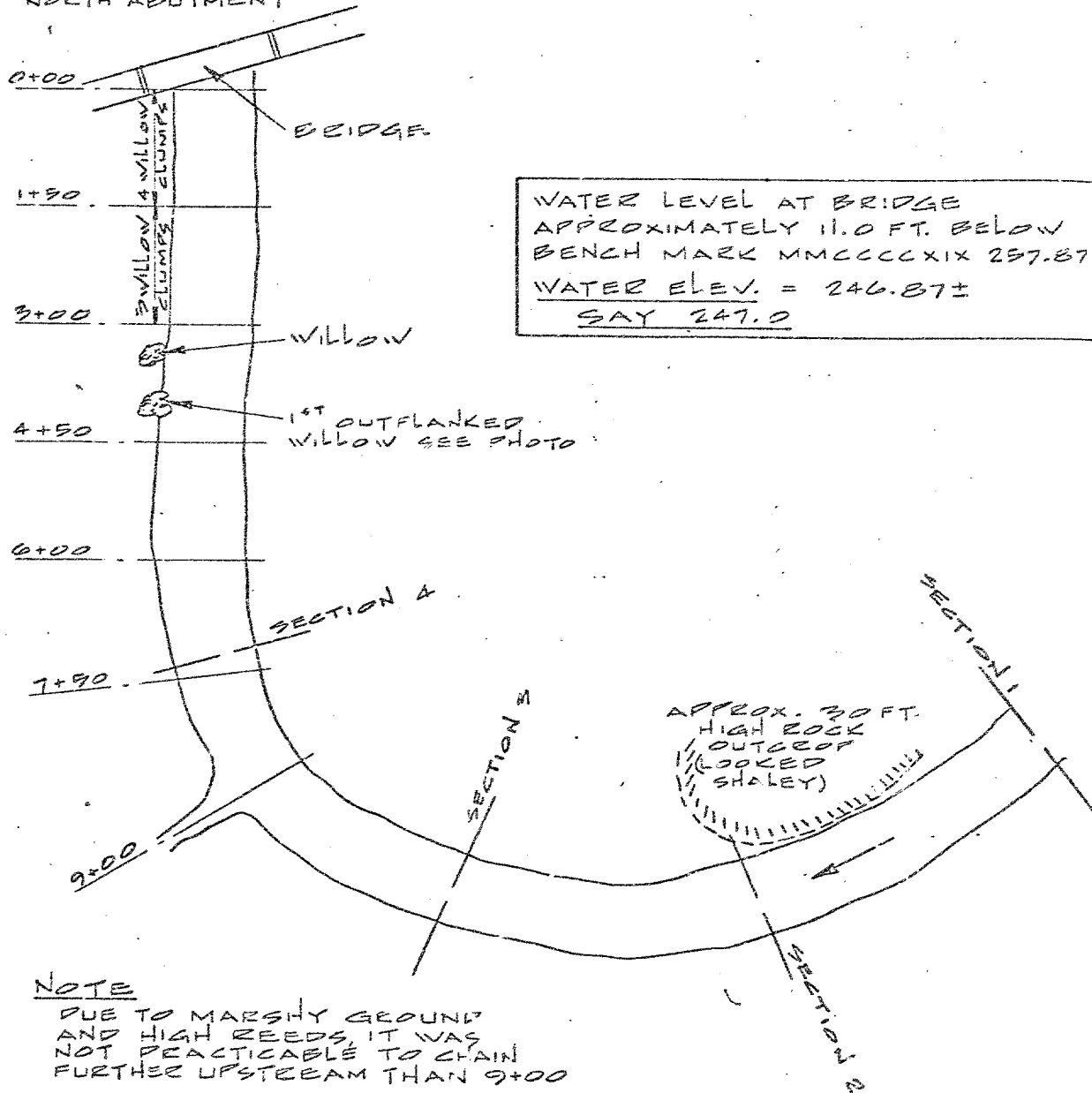
Immediately downstream of Highway No. 2, the northflood plain has been filled to an average elevation of approximately 250+ and is occupied by the marina of the Metro Marine Limited, the Bronte Yacht Club and buildings of the local fire department. This development extends approximately two tenths of a mile downstream of Highway No. 2 to Bronte Road. The outlet channel to Lake Ontario is contained within the Bronte Road embankment works on the north and the harbour breakwater on the south.

Interviews were held with Mr. W. G. Sargent, Bronte Harbour Master, and E. Vandermolen, manager of Metro Marine Limited in the course of our studies. Subsequent to these discussions, our detailed site investigation described in Section 3(c) was completed.

(c) Details of Site Investigation

A sketch plan of the general area upstream of the proposed bridge is presented on Figure 2.

APPROXIMATE
CHAINAGE
UPSTREAM OF
NORTH ABUTMENT



SKETCH PLAN OF
AREA UPSTREAM
OF BRIDGE

APPROX. SCALE 1" = 200'

FIG. 2

FILE 870-A-1754

The following information was obtained due to its significance to the present study.

- (i) A centreline profile of the creek to try to locate any peculiarities, particularly an outcrop near the bend upstream of the bridge.
- (ii) Cross-sections in the bend to locate the thalweg.
- (iii) Provide a description of the north bank (see Figure 2) to include evidence of ice scars on trees, and the number of clumps of mature willows on this bank.
- (iv) To locate lines and spacing of the old wooden piles under the existing bridge.
- (v) To detect any evidence of undermining of the abutments of the existing bridge.
- (vi) To investigate the erosion around the existing corrugated metal pipes located at the downstream side of the north abutment

(d) Centreline Profile

Chainage stations were established on the north bank at intervals of approximately 150 feet. Water depths were observed in the near mid-stream and were referred as closely as practicable to the chainage stations. Random soundings were taken to assure that the depths so recorded were representative for each particular location. The results of this survey are recorded in Table 2 below.

Table 2 - Approximate Streambed Profile

<u>Chainage</u>	<u>Depth at Midstream</u>	<u>Streambed Elevation</u>
1+50	10'-6"	236.5
3+00	10'-6"	236.5
4+50	11'-6"	235.5
6+00	10'-0"	237.0
7+50	9'-6"	237.5
9+00	8'-6"	238.5
Between 9+00 and Section 1 (See sketch previous page)		Generally between 8'-6" and 9'-6" decreasing fairly uniform to about 8 ft. towards Section 1. No sign of outcrop.

(e) Cross Sections at Bend to Locate Thalweg

Four cross sections were established approximately, as shown on the sketch plan Figure 2. The positions of the soundings were estimated only across the section, since there was insufficient variation in depth to warrant more careful location. A brief description of each cross-section follows.

(1) Section 1 - Approximately 150 ft. upstream
of shaly outcrop

Cross section fairly uniform.

Width about 90 ft.

Max. depth at midstream of about 8 ft.

Depth remained 7 ft. for some way toward north bank.

Bottom slopes out uniformly from the south bank.

(ii) Section 2 (at shaly outcrop)

Section is approximately uniform with maximum depth of 8'-6" at midstream. In contrast to Section 1 deep section extends towards the south bank.

(iii) Section 3

Very uniform section (i.e. symmetrical). Width about 100 ft. with maximum depth of approximately 10 feet at midstream.

(iv) Section 4

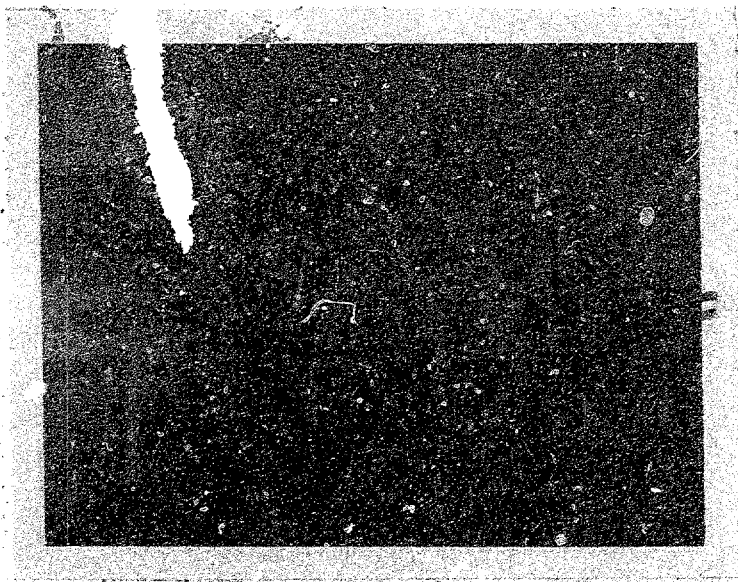
Again symmetrical.

Width about 110 ft.

Maximum depth of approximately 10 feet at midstream.

(e) Description of North Bank

A careful investigation of the numerous mature trees failed to produce any evidence of ice scarring. Detailed notes of observations made at particular locations have been summarized according to the indicated chainage intervals.

(1) Chainage 0+00 to 1+50

View of North
Abutment Looking
Downstream

The abutment is exposed here, in contrast to the south abutment which is protected by the river bank.

Note the exposed tree roots and the poor condition of the unrelieved concrete apron.

The concrete armouring shown in the photo ends just upstream of the abutment. The creek bank continues low, soft and spongy with grass nearly down to water level. There are 4 clumps of willows, and these are outflanked, jutting from 5-10 ft. into the waterway. The roots are exposed and serious undermining is taking place. There is no direct evidence of undermining or even erosion, between the willows (i.e. the banks are not undercut and they have fairly sound grass cover) but the outflanking indicates that erosion is going on, and that it is quite powerful. In these reaches from 0+00 to 1+50 ft. there is little dumped concrete, in contrast to the reaches 1+50 to 3+00.

(1) Chainage 0+00 to 1+50



View of North
Abutment Looking
Downstream

The abutment is exposed here, in contrast to the south abutment which is protected by the river bank. Note the exposed tree roots and the poor condition of the unrelieved concrete apron.

The concrete armouring shown in the photo ends just upstream of the abutment. The creek bank continues low, soft and spongy with grass nearly down to water level. There are 4 clumps of willows, and these are outflanked, jutting from 5-10 ft. into the waterway. The roots are exposed and serious undermining is taking place. There is no direct evidence of undermining or even erosion, between the willows (i.e. the banks are not undercut and they have fairly sound grass cover) but the outflanking indicates that erosion is going on, and that it is quite powerful. In these reaches from 0+00 to 1+50 ft. there is little dumped concrete, in contrast to the reaches 1+50 to 3+00.

(11) Chainage 1+50 to 3+00



View of outflanked
clump of willows
on north bank,
looking upstream

Beyond this clump the water comes in about as far as it has in the foreground.

Note the dumped concrete rubble. The depth where the willows jut out the farthest is 4-5 ft. typically.

There are five clumps of willows in this reach and between each there is a pile of dumped concrete rubble. These are large slabs about six inches thick (See photo next page).

The condition of the banks near the water line therefore could not be seen, nor could the effectiveness of the stone be measured.

(11) Chainage 1+50 to 3+00



View of outflanked
clump of willows
on north bank,
looking unstream

Beyond this clump the water comes in about as far as it has in the foreground.

Note the dumped concrete rubble.

The depth where the willows jut out the farthest is

4-5 ft. typically.

There are five clumps of willows in this reach and between each there is a pile of dumped concrete rubble. These are large slabs about six inches thick (See photo next page).

The condition of the banks near the water line therefore could not be seen, nor could the effectiveness of the stone be measured.



Typical view in this reach showing the dumped rubble between two outflanked clumps of willows.

Note also the rubble here, which has been dumped to protect the next clump.

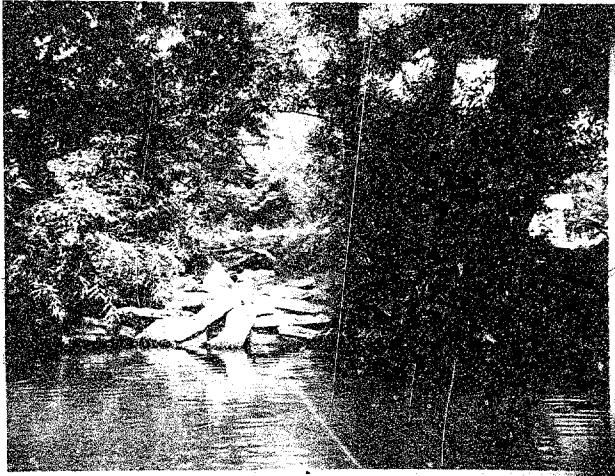
This photo illustrates how the stone prevents examination of the bank beneath.

(111) Chainage 3+00 to 4+50

There are fewer willows in this reach, and they are not near the water's edge, except that the first outflanked willow is approximately at chainage 4+00.

Banks are low and soft and the bed material is very spongy. There is no visible undermining of the banks except near the outflanked willow. In places the banks at the water's edge are nearly vertical, but are very low. (This is illustrated in the next photo).

There is no large rubble dumped here, or anywhere else upstream, but there has been smaller rip-rap (say 1 cu.ft. and less) dumped in random locations.



Typical view in this reach showing the dumped rubble between two outflanked clumps of willows.

Note also the rubble here, which has been dumped to protect the next clump.

This photo illustrates how the stone prevents examination of the bank beneath.

(iii) Chainage 3+00 to 4+50

There are fewer willows in this reach, and they are not near the water's edge, except that the first outflanked willow is approximately at chainage 4+00.

Banks are low and soft and the bed material is very spongy. There is no visible undermining of the banks except near the outflanked willow. In places the banks at the water's edge are nearly vertical, but are very low. (This is illustrated in the next photo).

There is no large rubble dumped here, or anywhere else upstream, but there has been smaller rip-rap (say 1 cu.ft. and less) dumped in random locations.

The first really
outflanked willow
looking upstream



Note the boat
which shows how far
the river is behind
the willow. The
surveyor is standin
on the water's edge
and his left hand
is on the 10 ft. mark
on the rod.

The chainage is approximately 4+00 and the depth at
the point jutting right into the river is about 5'-6".

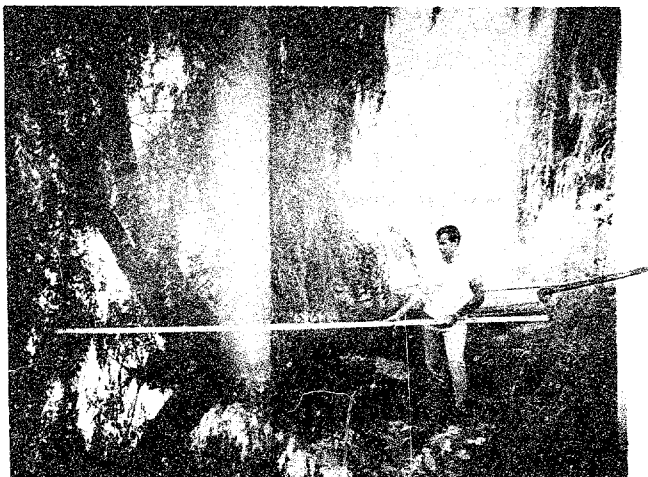
This photo illustrates how low the banks are near
the water's edge. This is typical. The banks near
the waterline are not much more than this anywhere
between 0+00 and 9+00.

(iv) Chainage 4+50 to 6+00

Between these stations coniferous shrubs with dense
growth down to water level cover the banks. There
appears to be minor erosion, but there is no outflanking
at all, and this reach looked stable.

Upstream of this reach the banks are grass covered.
There are no willows on the water's edge and no

The first really
outflanked willow
looking upstream



Note the boat
which shows how far
the river is behind
the willow. The
surveyor is standing
on the water's edge
and his left hand
is on the 10 ft. mark
on the rod.

The chainage is approximately 4+00 and the depth at
the point jutting right into the river is about 5'-6".

This photo illustrates how low the banks are near
the water's edge. This is typical. The banks near
the waterline are not much more than this anywhere
between 0+00 and 9+00.

(iv) Chainage 4+50 to 6+00

Between these stations coniferous shrubs with dense
growth down to water level cover the banks. There
appears to be minor erosion, but there is no outflanking
at all, and this reach looked stable.

Upstream of this reach the banks are grass covered.
There are no willows on the water's edge and no

outflanking of any trees. The water depth at the bank is about 13" to 2'-6" and the banks are about 1'-6" high. In places these low banks are vertical and show minor signs of undercutting.

(v) Chainage 6+00 to 7+50



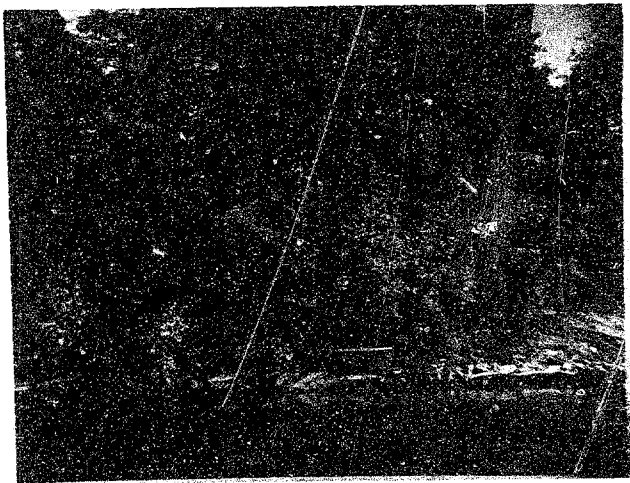
The picture is slightly upstream of station 6+00.

The low, almost vertical banks described in previous section start just downstream of picture. This picture shows the worst undercutting detected.

Upstream of this location the banks are soft and spongy near the water's edge and rise to about 5-6 ft. high further back (see extreme right of photo for example). The bed shelves out quite gently (say 2-3 ft. deep 8 ft. out).

outflanking of any trees. The water depth at the bank is about 13" to 2'-6" and the banks are about 1'-6" high. In places these low banks are vertical and show minor signs of undercutting.

(v) Chainage 6+00 to 7+50



The picture is slightly upstream of station 6+00.

The low, almost vertical banks described in previous section start just downstream of picture. This picture shows the worst undercutting detected.

Upstream of this location the banks are soft and spongy near the water's edge and rise to about 5-6 ft. high further back (see extreme right of photo for example). The bed shelves out quite gently (say 2-3 ft. deep 8 ft. out).

At about chainage 7+00 the river banks give way to reeds and rushes which continue from there upstream. The erosion of the riverbank starts at about chainage 7+00 i.e. as soon as it is exposed to the flow and not protected by reeds and rushes.

(f) Supplementary Soundings Along North Bank

A line of soundings was established at the locations indicated below in Table 3. These particular soundings were taken to attempt to identify the extent of the reach affected by the bridge contraction.

Table 3 - Supplementary Soundings North Bank

<u>Chainage</u>	<u>Depth at Centreline</u>	<u>Depth 10 to 15 Feet Out From North Bank</u>
0+00 to 0+25		About 9 ft.
0+25 to 1+50	10'-6"	8'-6" to 5'-6"
1+50 to 1+75	12' @ 275	5 ft. - 7 ft.
1+75 to 3+00	10'-6" @ 300	7'-6" - 9 ft.
3+00 to 4+50	11'-0" @ 450	decreases to 5 ft.
4+50 to 6+00	10'-0" @ 600	about 6 ft.

It appears that as the bridge is approached, the depth 15 ft. from the bank increases.

(g) Presence of Wooden Piles

Two lines of wooden piles were observed directly under the present bridge. The piles were located on lines parallel to the direction of flow at a distance of approximately twelve feet from each abutment. It is assumed that these piles were used as under-

pinning during construction of the present
bridge in 1918.

SECTION 4 - INTERVIEW WITH LOCAL RESIDENTS

(a) Interview with E. Vandermolen, Manager
Metro Marine Limited

Mr. Vandermolen has been working with Metro Marine since 1960. Information obtained with his cooperation is summarized under the following items.

(1) Navigation Channel Depth

The depth of water in the navigation channel varies from eight feet in the vicinity of the Marina to twelve feet along the breakwater. The channel has been dredged on several occasions between the downstream limits of the Marina and the channel outlet at the end of the breakwater.

(11) Occurrence of River Ice

Ice thickness in the navigation channel is estimated to be normally two feet during the late winter. The occurrence of ice in the form of ice cover as opposed to ice jamming seems to be characteristic for the Bronte Creek outlet.

Mr. Vandermolen indicated that in his experience, the relief of ice out of the Creek mouth has never been complicated or prevented by ice cover on Lake Ontario. The navigation channel usually clears of ice between the Marina and Lake Ontario prior to the spring freshet.

(iii) Local Flooding

According to Mr. Vandermolen, the Marina site (elevation 250+) has never been flooded. Normal creek rise during spring freshet is in the order of one to one and a half feet to a maximum stage of approximately 248.5 feet.

(b) Interview with W. G. Sargent

Mr. Sargent is 63 years old, having lived all his life in Bronte. His knowledge of flooding on Bronte Creek is extensive on the basis of his long experience as a commercial fisherman in the area and current responsibilities as Harbour Master. A summary of information obtained from Mr. Sargent is provided in the following notes.

(1) Navigation Channel Depth

Mr. Sargent's comments confirmed generally those opinions provided by Mr. Vandermolen. In addition, Mr. Sargent indicated that river depth is adequate for small craft navigation over a substantial channel reach upstream of Highway No. 2. This was confirmed by our own surveys described in Section 3. It is his opinion that if the new bridge provides the proposed clearance, the flood plains upstream of the new bridge location will be developed for additional Marina facilities.

(ii) Occurrence of River Ice

It is Mr. Sargent's recollection that river ice has not been a problem either at the proposed bridge location or in the navigation channel downstream of that point. He did recall however, that on two occasions it was necessary to remove an ice jam by blasting, from the river bend investigated and described above in Section 3. He was not able to recall specific dates in this regard.

It was confirmed that an annual ice cover is experienced but that it is not generally troublesome.

(iii) Local Flooding

Mr. Sargent confirmed that the site now occupied by the Metro Marine has never flooded to the best of his recollections. He did recall however that flooding of the delta marsh area upstream of the present bridge location is common. Further, it was common prior to the construction of the present 120 foot clear span bridge which replaced the original steel truss structure of similar clear span sometime between 1916 and 1918.

The most severe flooding event remembered by Mr. Sargent occurred prior to the construction of the existing bridge. He did not recall the date, but was able to confirm that the flood waters over-

flowed the approach embankments to the old bridge. It was not possible to locate a road profile for the original approaches, consequently it was not possible to establish maximum flood stage for this event.

It is noted, however, that the crown of the bridge deck of the original structure was identical to that of the existing crossing which is at elevation 256.0. It is recalled by Mr. Sargant that the original approaches were not as high as the present embankments but he was not able to provide an estimate of the difference in elevation.

(c) Closure

Mr. Sargant recalled during a follow-up telephone call that Mr. D. MacDonald, who was the carpenter's foreman during construction of the present bridge is still living in Bronte. As such, he may have been able to provide some further qualitative opinion on the relative elevations of the current and original bridge approaches.

However, due to the considered adequacy of information received from Mr. Sargant, an interview with Mr. MacDonald was not pursued.

SECTION 5 - HYDRAULICS OF PROPOSED WATERWAY ALTERNATIVES(a) Site Description

As discussed in Section 3, a major change in flow direction exists approximately 900 feet upstream of the present crossing. At that bend the creek enters the lower flood plain, flowing in a generally north-east direction. It is redirected to the south east by the north bank, which is badly eroded as discussed.

The Creek crosses Highway No. 2 at the bottom of its last complete meander across the original flood plain prior to entering Lake Ontario. The original flood plain is estimated to have been at least 700 feet wide. The proposed site for the new structure is coincident with that of the present crossing which is located adjacent to the south creek bank. As such it is exposed to the possibility of outflanking to the north, a matter discussed in more detail in Section 6.

The reported subsoil conditions indicate the presence of highly erodible material within the flood plain, bounded on the bottom by shale bedrock located generally at elevation 218₊. The shallowness of the layer of parent glacial till overlying portions of the bedrock sampled, suggest that past incidents of severe scour have intruded downwards at least twenty to twenty-five feet through the flood plain alluvium to the underlying bedrock.

The presence of decomposed wood fragments, fibrous roots and micro-organisms reported in all boreholes except BH 1 indicate that such intrusions are of contemporary occurrence. The clear waterway of the existing bridge is 120 feet measured between vertical abutment faces which are skewed at approximately 25 to 30 degrees to the direction of flow. This waterway represents only 16 per cent of the unobstructed width of the natural watercourse.

The waterway for the two alternatives proposed for the new bridge is essentially similar to that provided in the existing structure from the point of view of contraction ratio and bulk flow area provided.

Due to this high contraction ratio, and proximity to Lake Ontario, zones of rapidly varying flow are to be expected upstream, through, and downstream of both the existing and proposed new structures. The river bed and subsurface soils existing at the crossing will scour when exposed to average velocities of 1+ f.p.s., a condition expected to obtain annually for both present and proposed new structures.

(b) New Structure

The present proposals of the new bridge are understood to be either:

- (i) a single 124 foot span bridge, or;
- (ii) a three span bridge having a centre span of 72 feet, two approach spans of 63 feet, all supported on two intermediate piers aligned to the direction of bulk flow.

It has been suggested that the invert of the channel through the new structures be armoured with scour protection.

Due to the relative similarity of the waterway provided by these two alternatives, and the absence of downstream backwater, each will function as an uncontrolled spill-through outlet when passing high discharge. This characteristic will be aggravated by the provision of a non-scouring waterway which will further throttle stream discharge and intensify non-uniformity of flow in the vicinity of the crossing.

On the basis of the information taken from Figure 67-F-42B of the Foundation Report, a non-scouring invert elevation of 235₊ would provide an available depth of flow through the proposed opening of approximately 12.5 feet with Lake Ontario at elevation 247.5. Unstable flow would result downstream of the crossing at a bridge discharge of approximately 16,000 c.f.s. Under these conditions the bulk or average velocity through the opening would be in excess of 20 feet per second.

SECTION 6 - CONCLUSIONS

(a) Introduction

The conclusions reached in the course of our investigations may be summarized under two basic categories, those of a hydrologic nature, and those of a hydraulic nature. Particular considerations in each regard are outlined below.

(b) Hydrologic

Such information as is available on past flooding of the Bronte Creek indicate that spring freshet conditions have caused the most severe flood events in past history. This position is indicated by our studies on the synthesis of flood estimates corresponding to regionally severe summer thunderstorm precipitation.

Ice jamming does not appear to have aggravated past flood stage either at the project site or Creek mouth. This situation is supported by five observations;

- (1) the wooden piles beneath the present bridge are still sound after approximately fifty years of exposure to such ice jam conditions as do obtain at this crossing;
- (11) there is no evidence of ice scarring of existing mature trees upstream of the present crossing;

- (iii) the low lying area downstream of the crossing has never been flooded;
- (iv) no recollection of notable ice jams at the crossing was registered by Mr. Sargant.
- (v) the creek mouth enters the north shore of Lake Ontario which is in the lee of ice driving winter winds.

It would appear that infrequent severe ice jamming has occurred at the channel bend immediately upstream of the site. Should the hydraulic characteristics of this natural obstruction be deliberately improved to eliminate ice jamming by deepening, widening or increasing the radius of curvature of the bend, it is possible that the risk of ice jamming may be introduced at the subject crossing in the future.

In light of the possible upstream extension of marina activity in the area some improvement of the present channel geometry is a definite possibility. There is an argument in this regard, to provide as wide a clear span at the new bridge as is economically practicable.

Some recognition is due the fact that the existing bridge has stood fifty years. The probability is very high (60%) that this structure has been exposed to a flooding event of frequency less than once in fifty years. In this regard, there is a forty

percent chance that a flood of as low a frequency of once in one hundred years has been imposed upon it. In the absence of any indication of costly or unusual maintenance requirements due to flood damage, the geometry of the present waterway appears to constitute a sound design basis for future structures.

(c) Hydraulic

Our investigations have led to the conclusion that the proposed site for the new bridge is subject to possible outflanking to the north in the event that future erosion should occur in the badly damaged north bank. Such erosion would be accompanied by the production of large debris as existing trees became completely undermined. It would further reduce the effectiveness of the proposed alignment of the intermediate piers for the three span alternative by changing the intended angle of attack.

Severe scouring at the location of the new bridge is to be expected. It is not possible to accurately estimate likely scoured depths through either of the proposed new waterways. Sub-surface data indicate past scoured intrusions have extended twenty to twenty-five feet to bedrock.

At the present time, it would appear from available sounding data shown on DHO figure PLAN-E4775-1 that a scour hole of significant proportion can develop during rising flood stages in the apparently

unprotected channel invert. The provision of armouring on the waterway invert will tend to transfer such scouring potential downstream, away from the bridge site and possibly accelerate toe erosion of the south bank of the bend in the Creek immediately downstream of the site.

SECTION 7 - RECOMMENDATIONS

The following recommendations follow from the foregoing considerations:

- (a) The larger willow trees existing along the north bank of the approach channel between chainage 0+00 and 4+50 are badly outflanked and susceptible to complete undermining and eventual wash out. On this basis, it is recommended that they be removed and a definite approach channel be established in the course of the proposed construction.
- (b) It is recommended that the north bank of the approach channel be stabilized against future erosion and possible outflanking of the proposed bridge location. Stabilization should be carried out between chainage 0+00 and 7+00 as shown on Figure 2. It is recommended that rock rip-rap protection consisting of 8-inch minimum size stone underlain by a graded granular filter be provided between these limits. Toe protection for the rip-rap cover will be necessary. It is recommended that the cover be extended at least four feet below existing river bed for this purpose.
- (c) It is recommended that the final design of the new bridge should be based upon the hydraulic requirements associated with a design stream discharge of no less than 15,000 c.f.s. Such a flow is predicted to occur with an average frequency of once in 50 years. The probability of such a streamflow being exceeded during an assumed useful life of 50 years for the new structure is high (approximately 60%) according to available data. Consequently, design to provide for this capacity should be approached on a conservative basis.

Our detailed considerations in this regard are as follows:

- (i) The design of the new bridge should provide a waterway of 1650 square feet below elevation 244.6 G.S.C. (minimum mean monthly water level Lake Ontario). This waterway is to be provided on a plane oriented at right angles to the abutment walls, and is intended to limit average velocity to between 8 and 9 f.p.s. during the design discharge of 15,000 c.f.s.

It is noted in this regard, that it is proposed to place granular backfill within the bridge waterway to the limits shown on drawing 67-F-42B of the Foundation Investigation Report. The backfill so placed, would extend between the elevations 232.0 and 236+ in mid-stream and higher near the abutments. These upper limits on the fill extend up to 3'+ above the present stream invert. As such, some obstruction of the present water-way would result. Further, it is doubtful that scouring of the backfill so placed could be controlled.

- (ii) It is thus recommended that the granular backfill within the waterway be omitted, since it cannot be depended upon either to provide stability for the approach embankments or to protect the underlying organic clay silt. Further, as indicated above the backfill placed to the limits indicated in the Foundation Investigation Report would partially obstruct the waterway.

(iii) Width requirements to provide for the recommended waterway depend upon the scoured depth allowed at the crossing. It is recommended that a basic centre-line width of 120 feet as proposed in the Foundation Investigation Report is adequate provided that a nominal scoured depth of 15'6" measured below elevation 244.6 G.S.C. is tolerable from the point of view of foundation design.

Design on this basis should anticipate that the scoured waterway through the bridge opening would be approximately rectangular being 106' wide measured normal to abutment walls and at least 15'6" deep.

On the basis of the nature of existing sub-surface materials, it is reasonable to expect that actual scoured depth at the crossing will exceed the 15'-6" required to develop the recommended waterway. In the event that scoured depths in excess of 15'-6" are unacceptable from the point of view of stability of abutment and approach embankment as finally designed, it is recommended that the underlying organic clay silt be protected by a minimum 2'+ thick rip-rap apron placed below elevation 229+. The apron so placed should consist of 100% individual stones of minimum 0.3 cubic feet volume and minimum dimension of 8 inches. It should extend downstream of the bridge a distance equal to six times the vertical interval between the top of the rip-rap and shale bed rock.

In the case of the single span structure illustrated on Drawing No. 67-F-42B, the rip rap apron should be extended upstream a distance equal to the length of the side walls of the abutment, and should extend laterally to cover the entire length of the abutment pile cap of the upstream abutment walls.

For the three span structure, the rip-rap apron should continue at least 16 feet upstream of the bridge and should extend laterally an adequate distance to protect the toe of the approach embankment as built.

- (iv) In the event that design proceeds according to the 1 span structure illustrated on drawing no. 67-F-42B, it is recommended that both abutments be provided with either cylindrical or box transition sections on the upstream and downstream sides. It is anticipated that the construction of the abutments for this alternative could be carried out within cofferdams which would be left in place after completion to protect the foundations and end sections of the approach embankments. In such a case, the box abutments as detailed on drawing 67-F-42B are acceptable. Further, with the cofferdam left in place, it would not be necessary to deliberately limit scoured depth for hydraulic reasons. Hence the rip-rap apron discussed above could be eliminated unless required by structural limitations of the cofferdam as left in place.

However, should the cofferdam be removed after abutment construction, it is recommended that the rip-rap apron be provided and the sidewalls of the abutments be extended vertically from the foundation pile cap to the upper limit of the granular backfill and back into the approach embankments. This is recommended in the event that it is necessary to protect the embankments against lateral failure which might be induced by scour within the waterway itself.

The skew angles indicated for the box abutments on drawing No. 67-F-42A are considered to be reasonable for this alternative. The definition of an exact skew angle from the data available is not possible, and therefore skew to conform basically to the geometry of the above-noted drawing is recommended.

- (v) As indicated in paragraph 7(c)(iii), above, scoured depths in excess of 15'-6" measured below elevation 244.6 G.S.C. are not unreasonable for the recommended centre-line waterway width of 120'.

If scoured depths in excess of 15'-6" are unacceptable from the point of view of abutment and approach embankment design for the 3 span alternative of drawing no. 67-F-42B, it is recommended that sheet piling be provided to limit the lateral extent of scoured waterway. The height, location, type and tie-back details for such piling will depend upon structural limitations, and as such will dictate the selection of permissible scoured depth and hence width of the waterway provided for this alternative. Again, the requirement for the rip-rap apron depends primarily upon structural capabilities of the piling. If it is not required to deliberately limit scoured depth from this point of view, the apron may be omitted in this case as well.

As outlined in the case of the single span alternative, skew to conform basically to the geometry of drawing no. 67-F-42A is recommended.

- (d) It is recommended that rip-rap protection as described in recommendation 7(a) above be provided downstream of the site where bridge discharge impinges directly on the present unprotected south bank.

SECTION 8 - ACKNOWLEDGEMENTS

We wish to acknowledge and record our appreciation of the cordial assistance afforded us during our investigations by staff of the Department of Highways of Ontario, Mr. W. G. Sargent Bronte Harbour Master and Mr. E. Vandermolen Metro Marine Limited Bronte.

To all others who have aided us in any way, we wish to extend our sincere appreciation.

LEGEND

REFERENCE POINTS

•	Nail
•	Nail & Washer
✕	Cut Cross
⊙	1 inch Round Iron Bar
⊠	Standard Iron Bar
□	Standard Concrete Monument
○	Iron Pipe
↑	Cut Arrow
V	Cut Crows Foot
⊕	Diameter
△	Hub
⊙	Bearing Tree

CLEARING AND GRUBBING

	Area to be cleared
	Area to be grubbed
	Area to be cleared and grubbed

CULVERTS AND DRAINAGE

	Culvert to be constructed
	Culvert with headwall to be constructed
	Existing Culvert
	Existing Culvert with headwall
	Sewer
□	Manhole
□	Catch Basin
	Catch Basin Setback
	90° Gutter Outlet
	45° Gutter Outlet

FENCES

	Existing Fence
	Fence to be erected
	Fence to be renewed
	Stone Fence

GUIDE RAIL

	Guide Rail to be erected
	Guide Rail to be removed

MATERIALS

	Granular B' Sand Cushion or Granular Borrow
	Granular A'
	Concrete

BUILDINGS

	1 Storey Brick House with Basement
	One - 2 Storey Brick & Frame, Service Station and Restaurant with Basement
	2 Storey Concrete Block Apartment with Basement
	Two - 2 Storey Brick Borne with Basements

RAILWAYS

	Single Track Railway
	Double Track Railway
	Abandoned Railway

MISCELLANEOUS

	Steel Hydro Tower
	Deciduous Tree
	Coniferous Tree
	Hedge
	Hydrant
	Light Standard
	Light Standard & Sign
	Wells or Wells & Windmill
	Traffic Light
	Mail Box
	Swamp on Marsh
	Edge of Swamp
	Vertical Earth
	Vertical Rock
	Sidewalk
	Lake or River Bank
	Earth or Rock Cut
	Earth or Rock Fill

A.

A.S.	Arch Structural Plate Bolted
A.C.	Asphalt Coated
A.C. & A.P.	Asphalt Coated and Asbestos Protected
A.C., A.P. & P.I.	Asphalt Coated, Asbestos Protected and Paved Invert
A.C. & P.I.	Asphalt Coated and Paved Invert

B.

B.W.F.	Barbed Wire Fence
B/R	Base of Rail
B.C.	Beginning of Curve
B.F.S.	Beginning of Full Superelevation
B.S.	Beginning of Superelevation
B.V.C.	Beginning of Vertical Curve
B.	Bell Pole
B.M.	Bench Mark
B.F.	Board Fence

C.

C.N.R.	Canadian National Railway
C.P.R.	Canadian Pacific Railway
C.B.	Catch Basin
C.	Chain Line
C.L.F.	Chain Link Fence
C.P.	Clay Pipe
Con.	Concession
Conc.	Concrete
C.P.	Concrete Pipe
S.P.	Concrete or Corrugated Sewer Pipe
Cont.	Contract
C.A.H.	Controlled Access Highway
C.S.P.	Corrugated Steel Pipe
C.S.P.A.	Corrugated Steel Pipe Arch
Co.	County
Culv.	Culvert
C & G.	Curb and Gutter
C.S.	Curve Spiral

D.

D.	Degree of Curve
D.A.	Delta Angle
D.H.W.	Department of Highways Monument
D.H.O.	Department of Highways of Ontario
Dev. Rd.	Development Road
Dist.	District

E.

E.P.	Edge of Pavement
E.F.	Electric Fence
El. or Elev.	Elevation
E.C.	End of Curve
E.F.S.	End of Full Superelevation
E.S.	End of Superelevation
E.V.C.	End of Vertical Curve
Ent.	Entrance

F.

F.H.	Fire Hydrant
Ft.	Foot or Feet
Fdn.	Foundation
Fr.	Frame

G.

Gar.	Garage
G.M.	Gas Meter
G.V.	Gas Valve
Ga.	Gauge
G.B.M.	Geodetic Bench Mark
G.	Grading
G.D.	Grading and Drainage
Gran.	Granular
G.B.	Granular Base
G.R.	Guide Rail

H.

H.W.L.	High Water Level
High.	Highway
H.S.	Highway Sign
H.M.	Hot Mix
Ho.	House
H.O.T.	Hub on Tangent
H.E.P.C.	Hydro Electric Power Commission
H.C.	Hydro Cable
H.	Hydro Pole
H & B.	Hydro and Bell Pole
H & L.	Hydro and Lamp Pole

I.

I.B.	Iron Bar
I.P.	Iron Pipe

L.

Li.	Left
L.V.C.	Length of Vertical Curve
L.S.	Light Standard
L.W.L.	Low Water Level

M.

M.B.	Mail Box
M.H.	Manhole

N.

N. & W.	Nail and Washer
---------	-----------------

P.

Pav.	Pavement
P.	Pipe, Round Rivetted
P.A.	Pipe Arch, Rivetted
P.A.S.	Pipe Arch Structural Plate, Bolted
P.G.	Plain Galvanized
P.S.	Pipe Round Structural Plate Bolted

R.

R.	Radius
Rwy.	Railway
R.F.	Rail Fence
Rev'n	Revision
Rt.	Right
R.O.W.	Right of Way
R.I.B.	Round Iron Bar

S.

S.S.	School Section
Sec.	Secondary Road
Ser. Rd.	Service Road
S.R.	Side Road
S.R.F.	Snake Rail Fence
Sp.	Spiral Angle
S.C.	Spiral Curve
S.C.S.	Spiral Curve Spiral
S.T.	Spiral Tangent
Std.	Standard
S.C.M.	Standard Concrete Monument
S.I.B.	Standard Iron Bar
Sto.	Station
Stn	Stone (Foundation)
Sty.	Storey (In Buildings)

T.

Tan	Tangent
T.S.	Tangent Spiral
T.	Telegraph Pole
T.L.C.	Temporary Level Crossing
T/R	Top of Rail
Twp.	Township
T.L.	Traffic Light
T.C.H.	Trans - Canada Highway
T.P.	Turning Point

V.

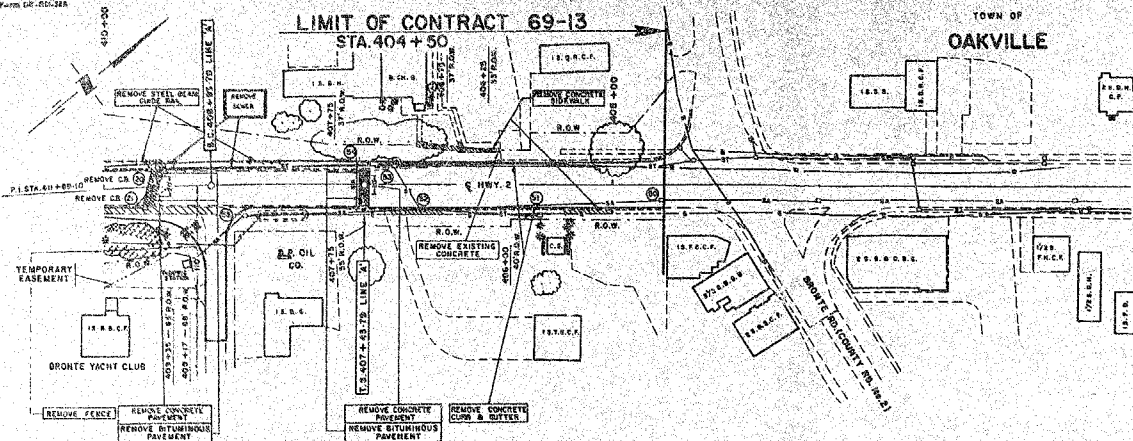
V.P.	Vitrified Pipe
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W.

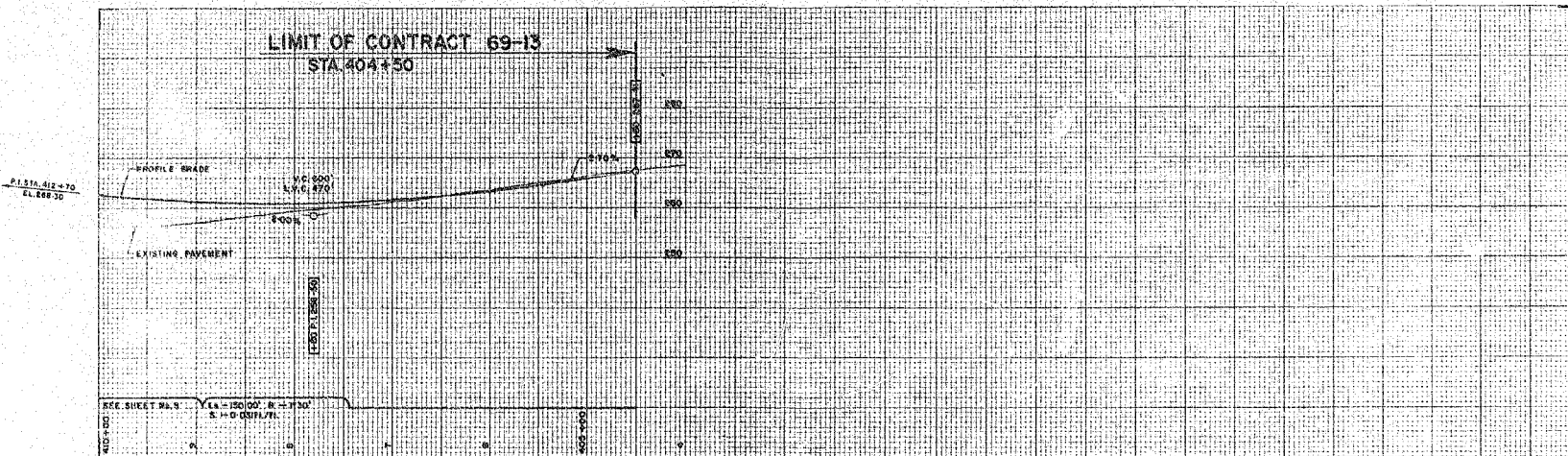
W.M.	Water Meter
W.V.	Water Valve
W.F.	Wire Fence
W.M.F.	Wire Mesh Fence
W.P.	Work Project
W.I.F.	Wrought Iron Fence

MATERIALS

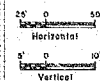
So.	Soil or Sandy
Si.	Silt or Silty
Cl	Clay
Lo.	Loom
Org.	Organic
G.B.C.	Granular Base Course
Asph.	Asphalt
St.	Stone



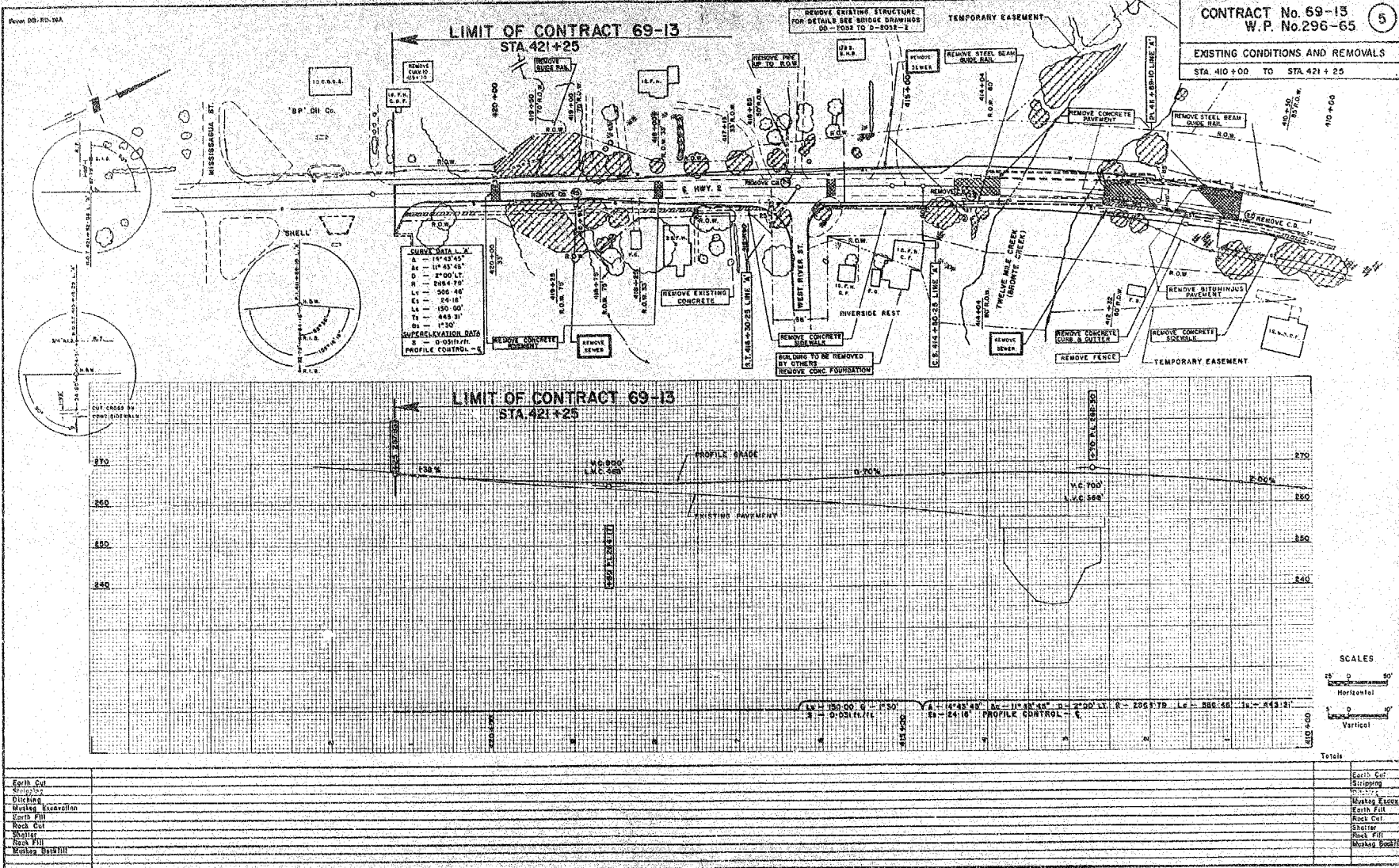
SURVEY DATA LINE 2
 A = 14° 43' 45"
 B = 11° 43' 45"
 C = 2° 00' 00"
 R = 2564.75'
 T = 445.31'
 L = 208.48'
 S = 24.16'
 L = 150.00'
 R = 1° 30'
 SUPERELEVATION DATA
 S = 0.0311 ft./ft.
 PROFILE CONTROL - S



SCALES



Totals		
Earth Cut		Earth Cut
Stripping		Stripping
Ditching		Ditching
Grading		Grading
Earth Fill		Earth Fill
Rock Cut		Rock Cut
Shoring		Shoring
Rock Fill		Rock Fill
Miscellaneous		Miscellaneous



PLACE	UNDER	CONCRETE
STA.	OF	TOTAL
14+70	15+00	15+00

LIMIT OF CONTRACT 69-13 STA 421+25

CONTRACT No. 69-13
W.P. No. 296-65

CONSTRUCTION DETAILS

STA 410+00 TO STA 421+25

NOTE: DETAILS OF STRUCTURE
OF THE GRADE DRAINAGE
SHALL BE AS SHOWN ON
THE DRAWING
SEE SHEET 100-10-11

MISSISSIPPI ST

100+00

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STATION	TO STATION	LOCATION
410+00	415+00	LT & RT
415+00	421+25	LT

GRAD CURVE AT ENTRANCE
AS SHOWN ON 42 DIRECTED
BY THE ENGINEER

FOR TYPICAL CROSS-SECTIONS
SEE SHEET No. 11

PLANT TOPPING AND
SUB-GRANULAR BASE

LIMIT OF CONTRACT 69-13 STA 421+25

THE NEW EXISTING CONDITIONS

EXCAVATION FOR WASHING ONLY

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EXCAVATION FOR WASHING ONLY

SCALES

1" = 40'

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1" = 40'

1" = 40'

Total

276.00

276.00

276.00

276.00

276.00

276.00

276.00

276.00

PAVEMENT ON

APPROACH ROAD

SURFACE COURSE

EL. 11.1

EL. 11.1

EL. 11.1

EL. 11.1

1200 CU YDS

1200 CU YDS

1200 CU YDS

1200 CU YDS

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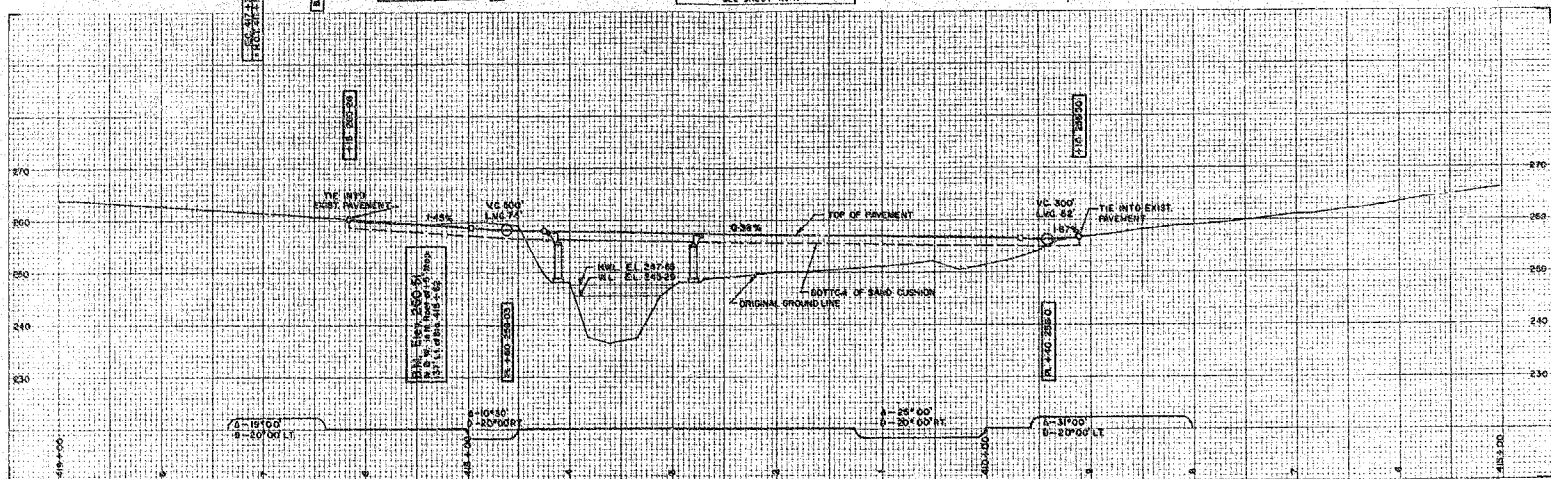
1200 CU YDS

1200 CU YDS

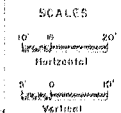
1200 CU YDS

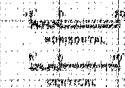
1200 CU YDS

STA. 408 + 00 TO STA. 417 + 34



Earth Cut	60 CU YDS	500 CU YDS	500 CU YDS
Graveling	70 CU YDS	150 CU YDS	220 Y Graveling
Ditching			Ditching
Grading			Grading
Earth Filling	3,400 CU YDS	610 CU YDS	Earth Fill
Rock Cut			Rock Cut
Shallow			Shallow
Rock Fill			Rock Fill
SURFACE COURSE H.I. 6 - 2"		SURFACE COURSE H.I. 6 - 2"	
100% GRANULAR "A" AND SAND			

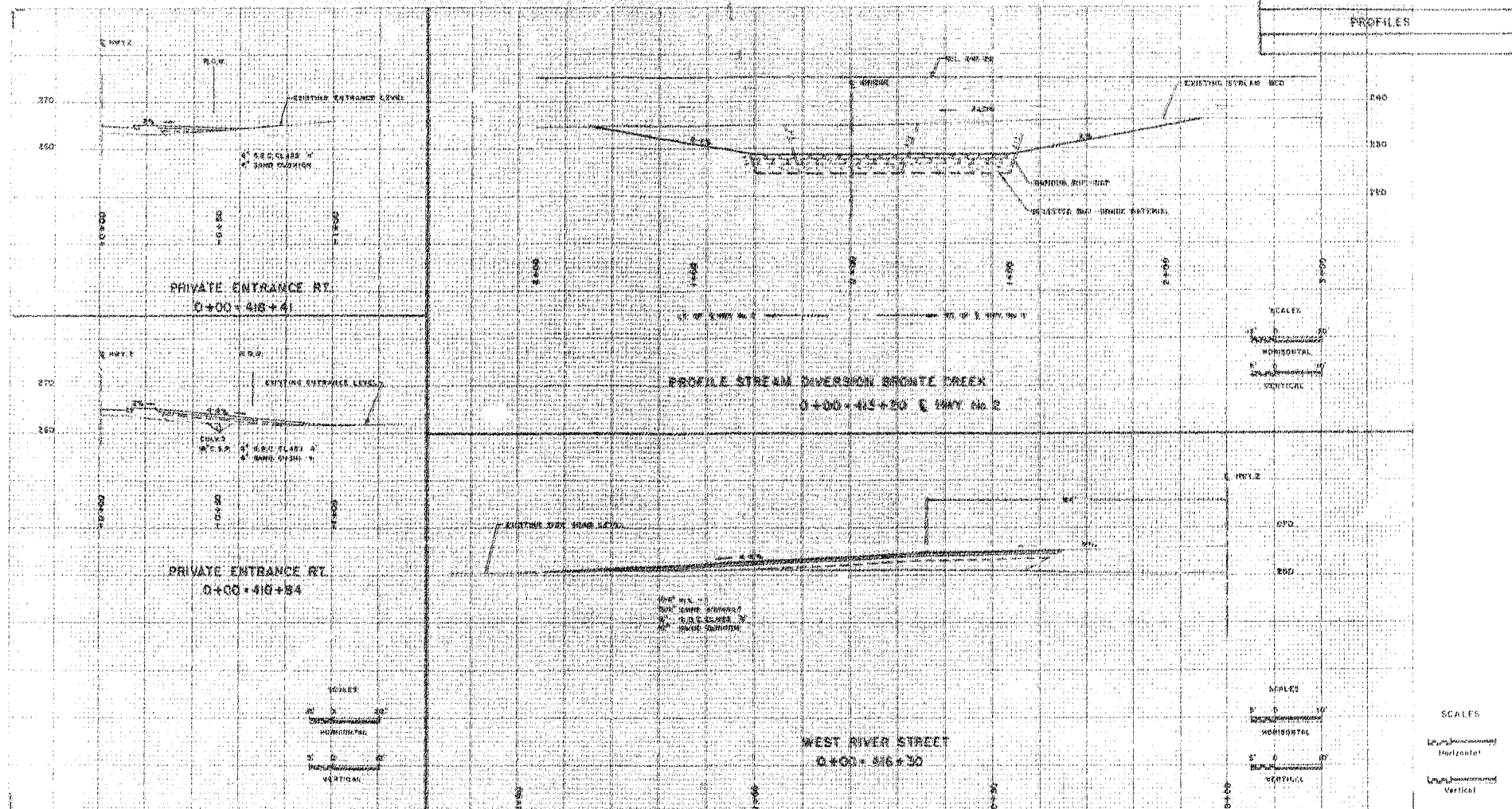
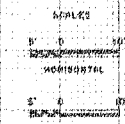
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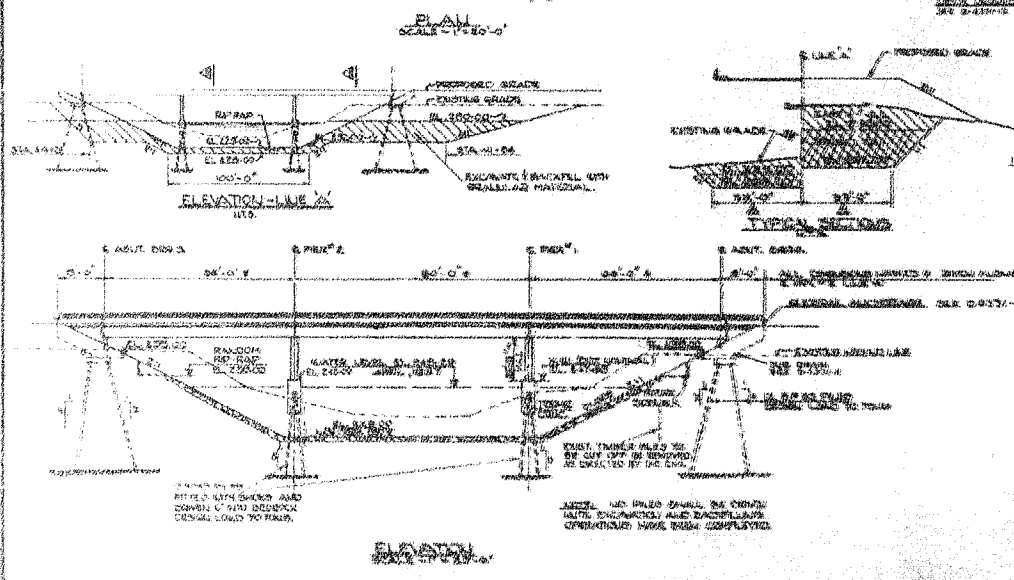
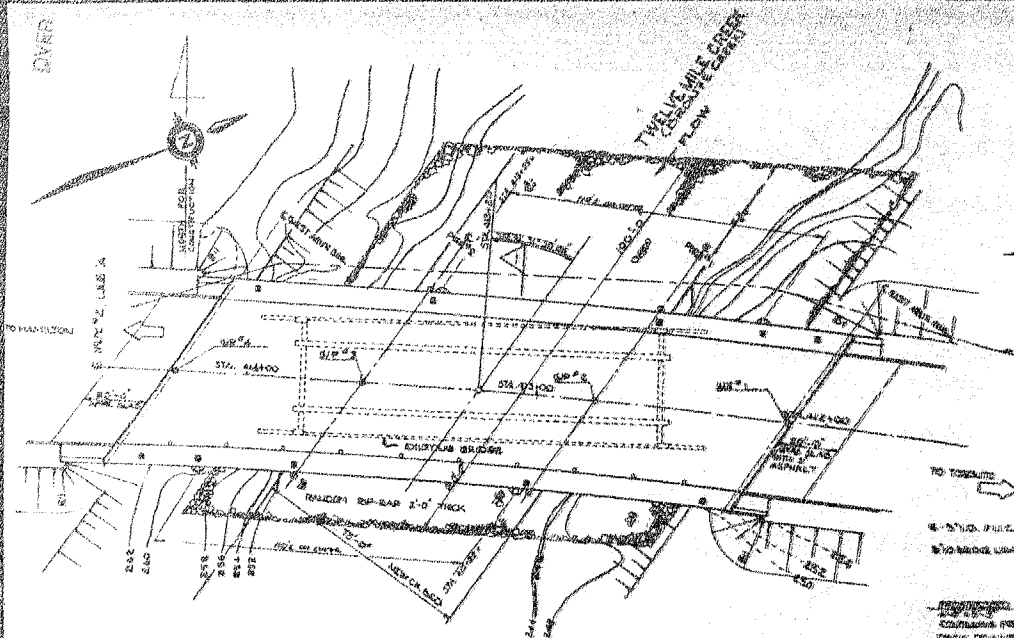
SCALES

Horizont

Vertical



		Total	
6-14-79	80 DU 779		Curtis Cut
6-14-79	80 DU 779		B-1000
6-14-79	80 DU 779		Vernon Evans
6-14-79	80 DU 779		Jerry Gill
6-14-79	80 DU 779		Wendy Cut
6-14-79	80 DU 779		Mari
6-14-79	80 DU 779		Russ Pitt
6-14-79	80 DU 779		Hughes Booth

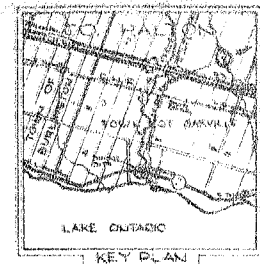
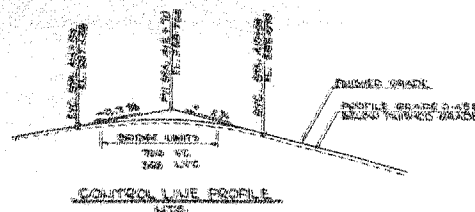


DATA

1.	STA. 414+00
2.	STA. 414+25
3.	STA. 414+50
4.	STA. 414+75
5.	STA. 415+00
6.	STA. 415+25
7.	STA. 415+50
8.	STA. 415+75
9.	STA. 416+00

LEGEND

ALL LIGHTING POLE DATA SEE DETAIL 101
 ALL LIGHTING LIGHT SEE DETAIL 101
 ALL LIGHTING JUNCTION BOX
 NOTE: 10' OF COIL OF CONDUIT FOR
 MUST BE LEFT AT EACH BOX



GENERAL NOTES

CLASS OF CONCRETE	
DECK AND DIAPHRAGMS	4000 psi
PRESTRESSED FOR GIRDERS	6000 "
PARAPET WALL AND SIDEWALKS	4000 "
APPROACH SLAB	3000 "
PIERS	4000 "
TELEPHONE	3000 "
ABUTMENTS	3000 "
ABUTMENT FOOTINGS	3000 "

CURB CENTER TO REINFORCING STEEL	
ABUTMENT & PIER FOOTINGS	3"
ABUTMENT & PIERS	3"
PIERS	2"
PARAPET WALL	1 1/2"
DECK TOP	2"
DECK BOTTOM	1 1/2"

GEODETIC B.M. 1144000.00 - 1144000.00
 CONCRETE TYPICAL BRIDGE OVER BRIDGE CREEK, OLD
 LAND BRIDGE, 1144000.00 - 1144000.00, 1144000.00 - 1144000.00
 FACE OF SOUTHABENT TRUSS, 6 FEET FROM SOUTHWEST
 END OF TRUSS & 11 INCHES ABOVE BRIDGE FLOOR.
 BOLT SET HORIZONTALLY.
 RUN TO TOWN OF CANNONVILLE (BROOKS).

LIST OF DRAWING DETAILS TO BE

GENERAL PLAN	1
PIER, DECK, APPROACH AND SIDEWALK	2
FOOTING LAYOUT	3
ABUTMENT DIMENSIONS AND BENCH	4
PIER, DECK & SIDEWALK	5
GENERAL DETAILS	6
DECK DIMENSIONS AND BENCH	7
APPROACH SLAB	8
PARAPET WALL DETAILS	9
STANDARD STEEL PARAPET BAR	10
STANDARD DETAILS I	11
STANDARD DETAILS II	12
STANDARD DETAILS III	13
PARAPET ELECTRICAL DETAILS	14
SAFETY BRIDGE	15
STEEL BRIDGE FOOTING	16



DEPARTMENT OF HIGHWAYS, ONTARIO
 LANSING OFFICE

TWELVE MILE CREEK BRIDGE

BRIDGE INVENTORY NO. 1111 DATE 11-1-41
 CO. HAMILTON
 TOWN OF CANNONVILLE, ONT.

GENERAL PLAN

APPROX. 1:20
 SCALE 1" = 100'-0"

DATE 11-1-41
 DRAWN BY J. J. HANCOCK
 CHECKED BY J. J. HANCOCK
 APPROVED BY J. J. HANCOCK

Don Thresher

Tom
Muir
Contract
Control

8-12-14 filing

Tom	cust of key
Muir	
Contract	
Control	

Ken Selby
Room 107

Material research
~~test files~~

Roy Runking
Office

Director of
Supply/used
filling

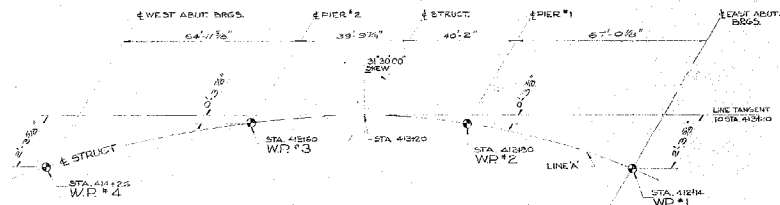
67-F-42

W.P. #296-65

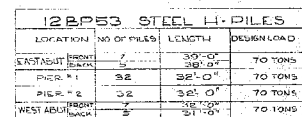
Hwy. #2

BRONTE

CREEK



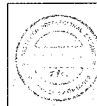
SCALE LONGIT: 1" = 20'-0"
LATERAL: 1" = 2'-0"

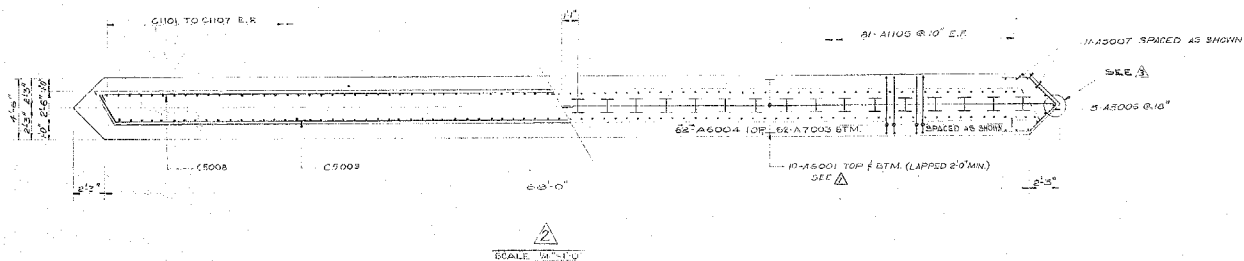
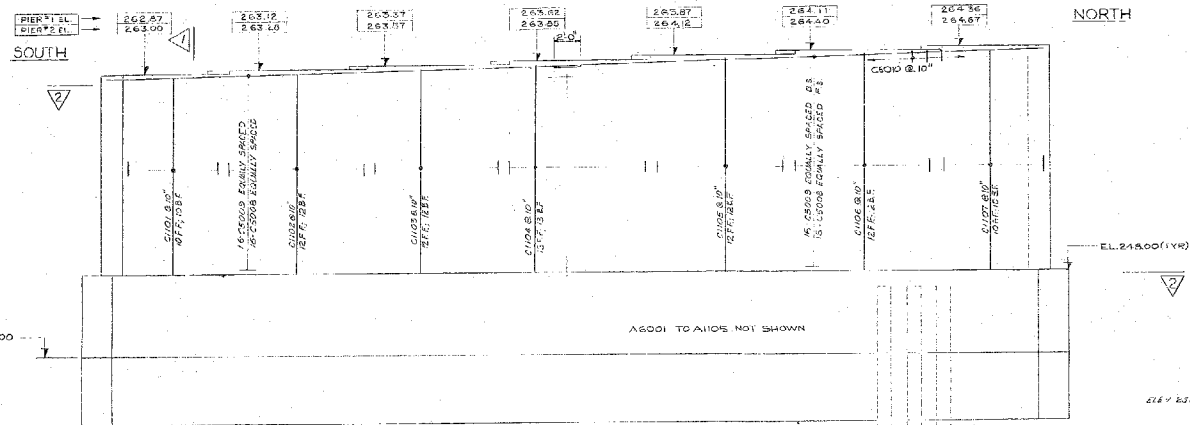


12BP53 STEEL H-PILES			
LOCATION	NO OF PILES	LENGTH	DESIGN LOAD
EAST ABUT	1	39'-0"	70 TONS
PIER #1	32	32'-0"	70 TONS
PIER #2	32	32'-0"	70 TONS
WEST ABUT	1	32'-0"	70 TONS

[illegible]

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
<u>TWELVE MILE CREEK BRIDGE</u>			
KING'S HIGHWAY No. <u>5</u>		DIST. No. <u>4</u>	
CO. <u>DALTON</u>		TWP. <u>TOWN OF OAKVILLE</u>	
LOT <u>101</u>		CON. <u></u>	
<u>FOOTING</u>		<u>LAYOUT</u>	
APPROVED <u></u>		DIST. No. <u>10-155</u>	
REGION <u>W.E.</u>		CONTRACT <u></u>	
DRAWING <u>D-6331-3</u>		DIVISION <u></u>	
DATE <u>12/1/68</u>		D. <u>6331-3</u>	

[illegible]

[illegible]

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO			
BUILDING DIVISION			
TWELVE MILE CREEK BRIDGE			
KING'S HIGHWAY No. 2			
CO. HALTON		DIST. NO. 4	
TWP. TOWN OF ORKWILLE		LOT CON.	
PIER DIMENSIONS & REINFORCEMENT			
APPROVED _____		DATE 10-13-5 SHEET NO. 296-65	
BUILT BY _____			
DESIGN <i>A/E</i>		CONTACT NO.	
DRAWING <i>C/F/L/A</i>		DRAWING NO.	
CHECKED <i>U/S/H/W</i>		D-6331-5	