

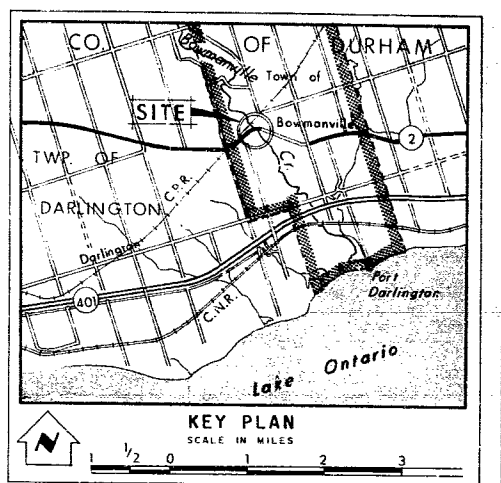
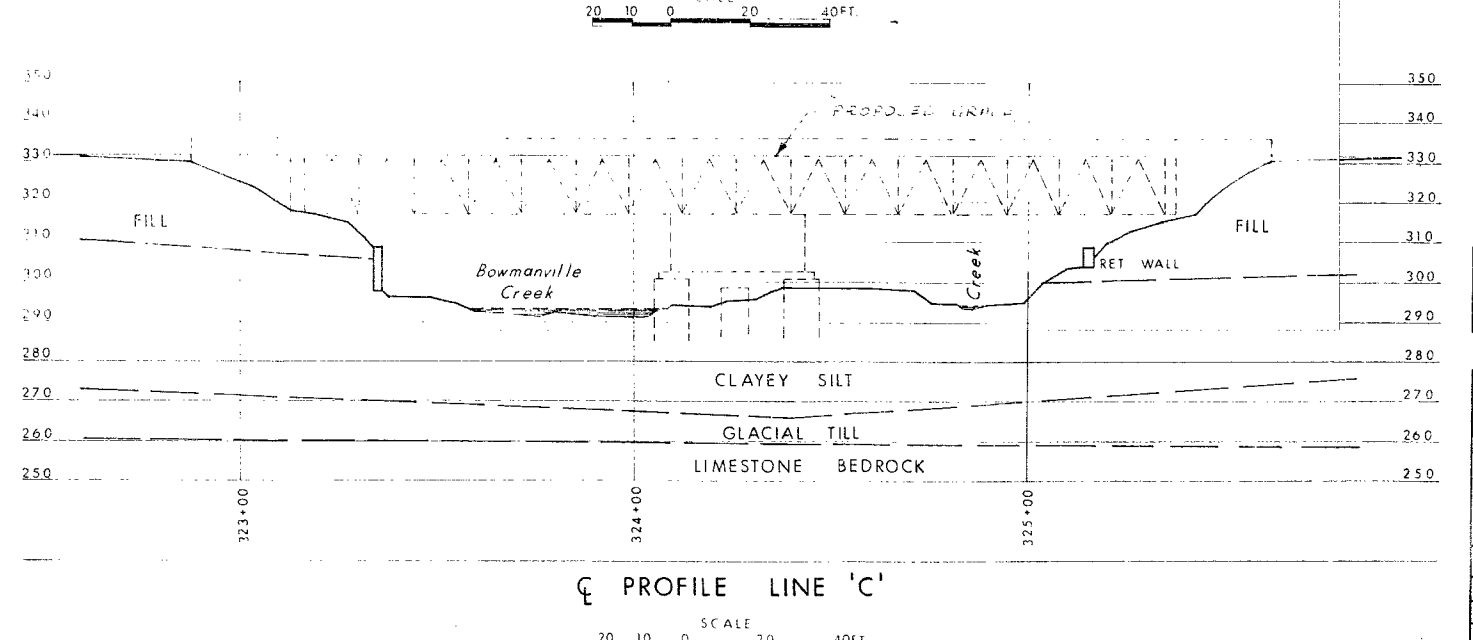
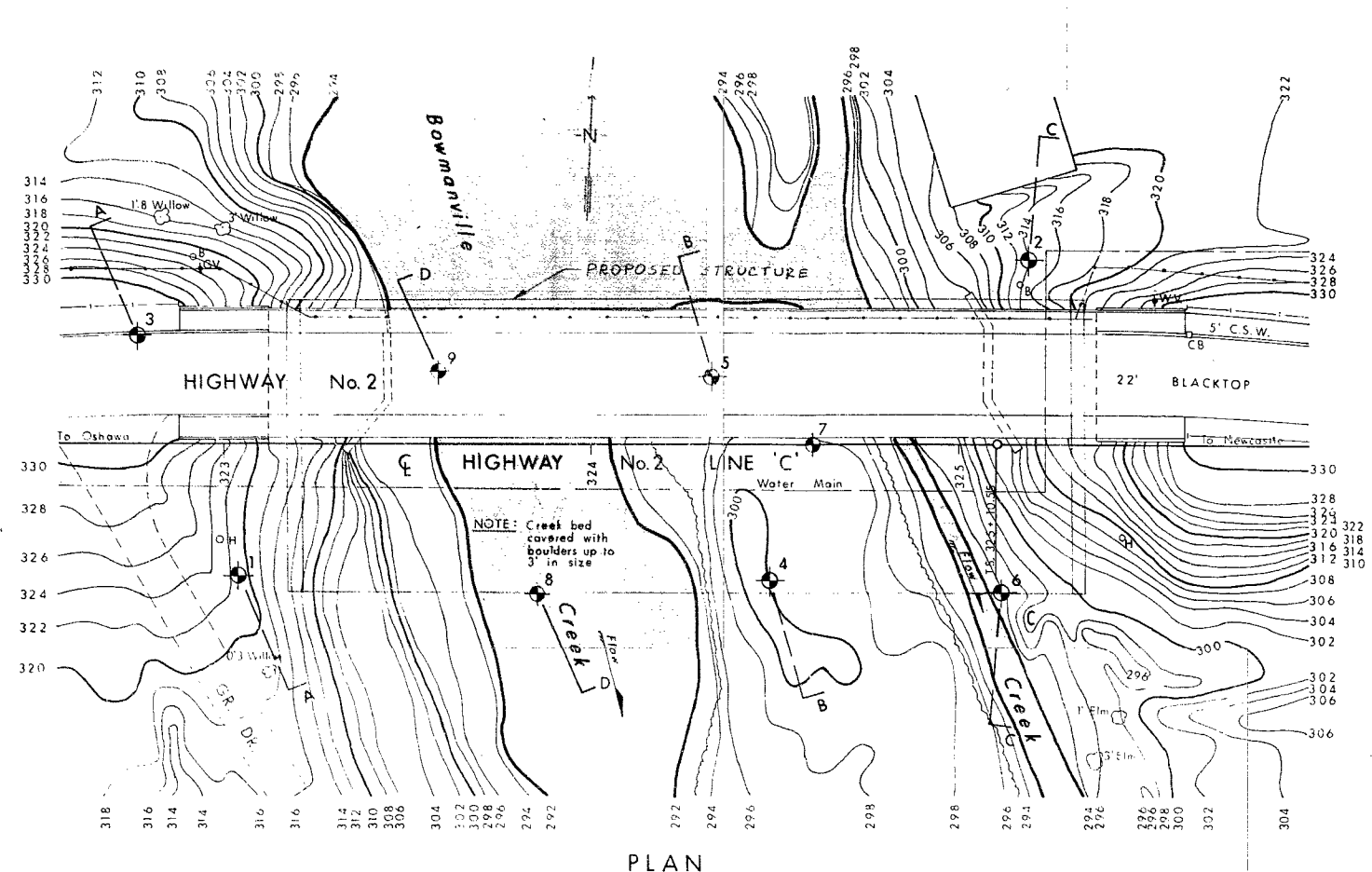
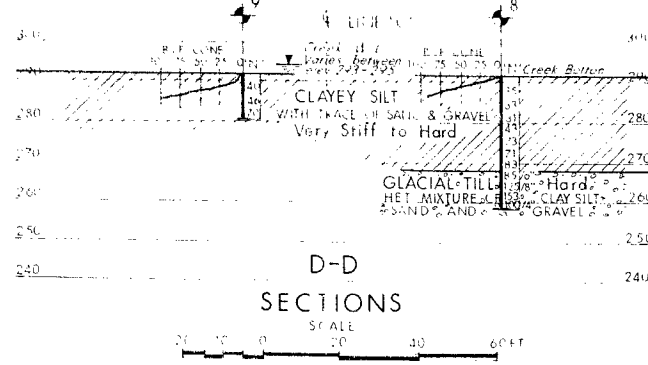
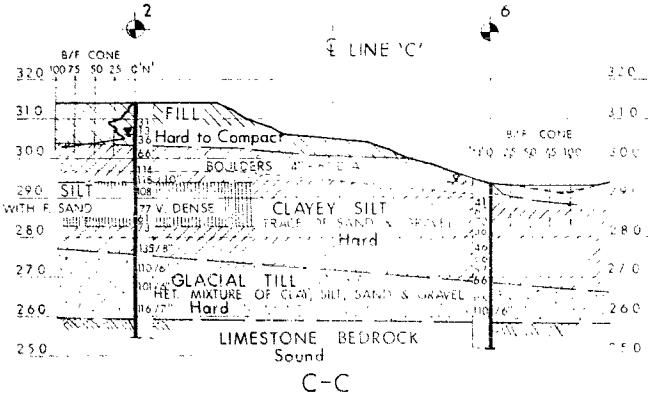
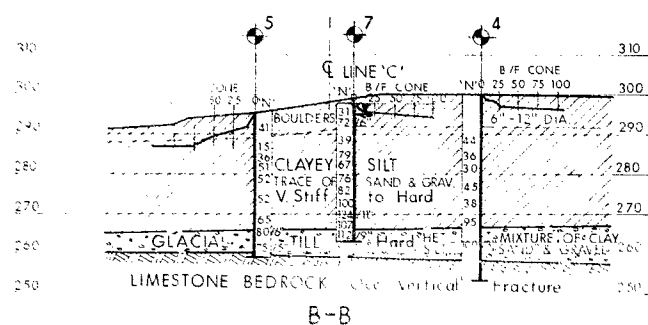
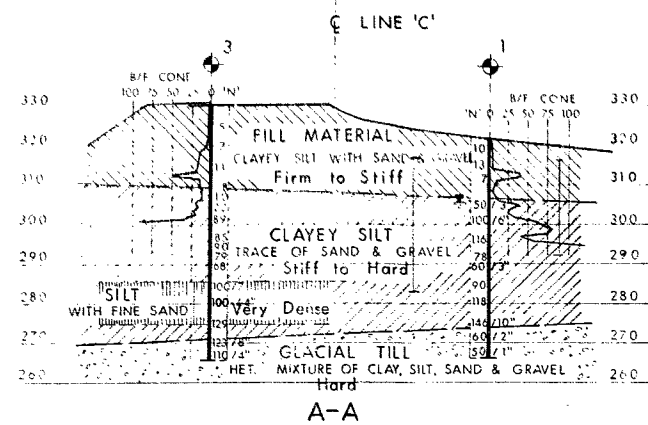
#68-F-49

W.P. #421-65

HWY #2 (LINE C)

BOWMANVILLE

CREEK



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation, JUNE 1968

NO.	ELEVATION	STATION	OFFSET
1	321.6	323+04	36' RT.
2	313.5	325+20	50' LT.
3	330.6	322+77	29' LT.
4	300.0	324+49	37' RT.
5	295.3	324+33	18' LT.
6	293.4	325+12	40' RT.
7	298.0	324+60	Q
8	292.0	323+85	40' RT.
9	292.0	323+58	20' LT.

NOTE

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

REVISIONS

NO.	DATE	BY	DESCRIPTION
1	23 Dec 68	G.P.	Bore Holes 7, 8 & 9, Section D-D Added

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION - FOUNDATION SECTION

BOWMANVILLE CREEK

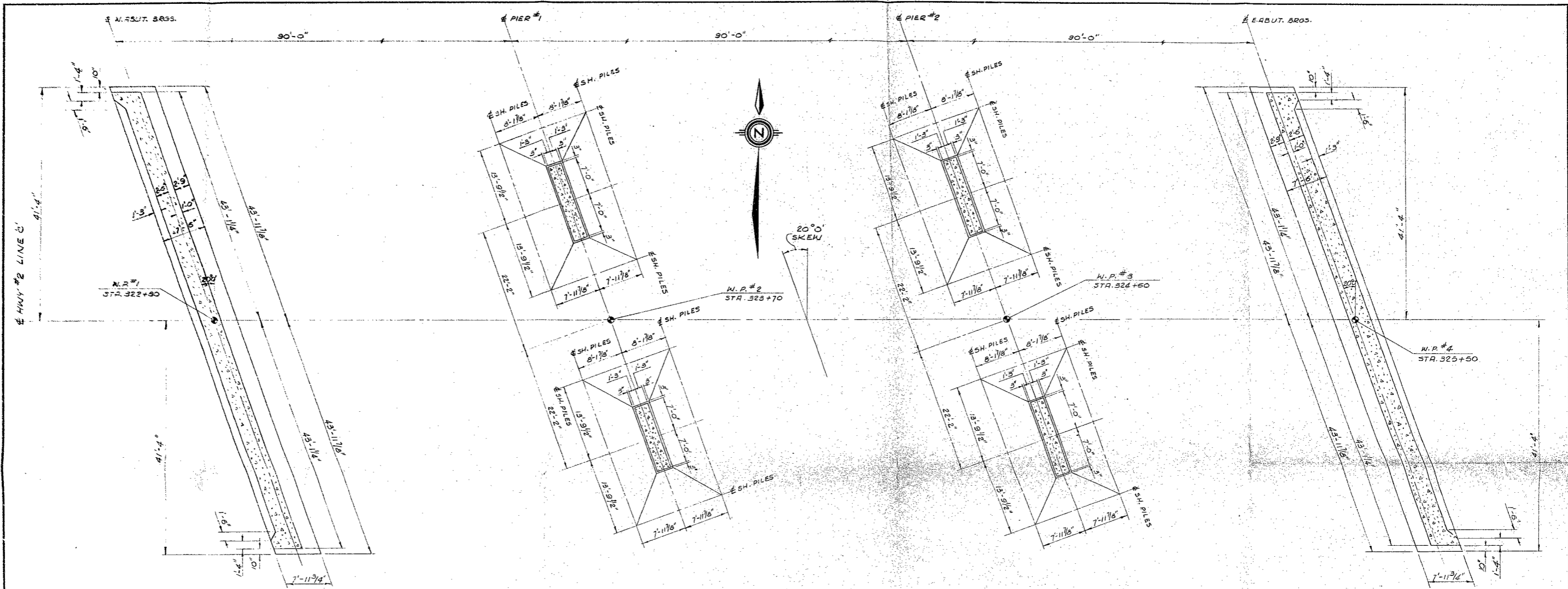
KING'S HIGHWAY NO. 2 LINE 'C' DIST. NO. 7

CO. DURHAM TOWN OF BOWMANVILLE

TWP. DARLINGTON LOT CON.

BORE HOLE LOCATION & SOIL STRATA

SUBM'D W. H.	CHECKED	WP. NO. 421-65	M.B.T. DRAWING NO.
DRAWN A. N.	CHECKED	JOB NO. 68-F-49	68-F-49A
DATE AUG. 1/1968	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>R. R. R.</i>	CONT. NO.		



FOOTING LAYOUT
SCALE: 1/8" = 1'-0"

NOTE:
FOR DETAILS & REINFORCEMENT
SEE DWG. D-6564-4

PRINT RECORD		
No.	FOR	DATE

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
68-F-49			
BOWMANVILLE CREEK BRIDGE			
IN THE TOWN OF BOWMANVILLE			
KING'S HIGHWAY No. 2	DIST. No. 7		
CO. DURHAM			
TWP. DARLINGTON	LOT 13	CON. 2	
FOOTING LAYOUT			
APPROVED		SITE No. 21-169	W.P. No. 421-65
DESIGN J. S.	CHECK R. K.	CONTRACT No.	
DRAWING B. S.	CHECK R. K.	DRAWING No.	D-6564-3
DATE MAR/1968	LOADING H320-44		

✓

15

HAMMER TYPE D-12 WEIGHT 1.375 T ENERGY 22,500 FT. LBS

[illegible]

BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 7 CONTRACT NO. 72-82 STRUCTURE BOWMANVILLE CREEK BRIDGE
 CONTRACTOR ARMSTRONG BROS. DESIGN LOAD OF PILE R=3x70 = 210 TONS
 HAMMER DETAILS: TYPE D12 WEIGHT 1.375 T. HEIGHT OF FALL OR ENERGY 22500 IN-T.
 TYPE OF ANVIL OR CAP 14" DRIVING HEAD WEIGHT OF ANVIL OR CAP 1014 LB
 PILE DETAILS STEEL HP12x53 @ 22.48/FT., 17016 BATTER, 1/2" x 11" x 12" STEEL DRIVING SHOE
 PILE NO. 2 LOCATION SOUTH HALF OF WEST ABUT. DATE DRIVEN NOV 28/72

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.
39	1	2	31	26	175		51			76	
44	2	5	37	27	125		52			77	
49	3	8	37	28	150		53			78	
54	4	10	37	29	175		54			79	
59	5	12	37	30	200		55			80	
64	6	15	37	31	250		56			81	
69	7	20	37	32	225		57			82	
74	8	22		33			58			83	
79	9	24		34			59			84	
84	10	26		35			60			85	
89	11	28		36			61			86	
94	12	29		37			62			87	
99	13	30		38			63			88	
104	14	32		39			64			89	
109	15	33		40			65			90	
114	16	34		41			66			91	
119	17	35		42			67			92	
124	18	36		43			68			93	
129	19	37		44			69			94	
134	20	100		45			70			95	
139	21	100		46			71			96	
144	22	100		47			72			97	
149	23	127		48			73			98	
154	24	135		49			74			99	
159	25	135		50			75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	20	21	18	20	21	20
MEASURED REBOUND IN INCHES	20	21	18	20	21	20
FINAL LENGTH OF PILE	33'					
FINAL CUT OFF ELEVATION	316.50					

REPORT TO BE SENT TO: - PRINCIPAL FOUNDATION ENGINEER
 MATERIALS & TESTING DIVISION
 DEPARTMENT OF HIGHWAYS
 DOWNSVIEW, ONTARIO

SIGNED J. W. Bradley
 NAME (PRINT) J. W. BRADLEY
 DATE Nov. 28/72

DEFECTS IN NEGATIVE DUE TO ATTACH SKETCH OF PILE NUMBERING SYSTEM
 CONDITION OF ORIGINAL DOCUMENT

cat. 316.5
 length 33.0
 d. tips 283.5

Notes:-

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 $\frac{1}{2}$ " O.D. steel tube x 0.251" @ 33 lbs. per ft. Vertical. 12 $\frac{1}{2}$ " x $\frac{1}{2}$ " steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 7 CONTRACT NO. 72-82 STRUCTURE BOWMANVILLE CREEK BRIDGECONTRACTOR HAMSTRONG BROS. PARTNERS DESIGN LOAD OF PILE R=70 x 3 = 210 TonsHAMMER DETAILS: TYPE D12 WEIGHT 1375 Tons HEIGHT OF FALL OR ENERGY 22500 FT-LBTYPE OF ANVIL OR CAP 14" DRILLING HEAD WEIGHT OF ANVIL OR CAP 1014 LBPILE DETAILS STEEL HP12x53 @ 23 LB/FT, 17010 BATTER, 1/2" x 11" x 12" STEEL DRIVING SHANKPILE NO. 9 LOCATION SOUTH 1/2 OF WEST ABUTMENT DATE DRIVEN NOV 29/72

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.
39	1	0	26			51			76		
	2	0	27			52			77		
	3	1	28			53			78		
	4	2	29			54			79		
	5	2	30			55			80		
	6	3	31			56			81		
	7	4	32			57			82		
	8	4	33			58			83		
	9	4	34			59			84		
	10	7	35			60			85		
	11	6	36			61			86		
	12	12	37			62			87		
	13	7	38			63			88		
	14	13	39			64			89		
	15	26	40			65			90		
	16	25	41			66			91		
	17	15	42			67			92		
	18	17	43			68			93		
	19	33	44			69			94		
	20	61	45			70			95		
	21	80	46			71			96		
	22		47			72			97		
	23	149	48			73			98		
	24	400	49			74			99		
	25		50			75			100		

DETAILS FOR FINAL <u>3</u> INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	10	87	115			
MEASURED REBOUND IN INCHES	0.08	0.10	0.12			
FINAL LENGTH OF PILE	24.5'			FINAL CUT OFF ELEVATION 316.50		

REPORT TO BE SENT TO: - PRINCIPAL FOUNDATION ENGINEER
MATERIALS & TESTING DIVISION
DEPARTMENT OF HIGHWAYS
DOWNSVIEW, ONTARIOSIGNED J. W. BradleyNAME (PRINT) J. W. BRADLEYDATE Dec. 29/72

ATTACH SKETCH OF PILE NUMBERING SYSTEM

cut off
 316.5'
 24.5'
 292.0'

11' 3L : 13L
 1 INCH X 1 INCH

1:115 = 0.009"/BLW

OVER

Notes:-

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 $\frac{3}{4}$ " O.D. steel tube x 0.251" @ 33 lbs. per ft. Vertical. 12 $\frac{3}{4}$ " x $\frac{1}{2}$ " steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

DRIVEN H-PILES FOR
SOUTH HALF OF WEST ABUT.
OF BOWMANVILLE CREEK STR.

PILE NO.	FINAL MEASURED $S + \frac{C}{2}$	APPROX. LENGTH OF PILE IN GROUND.
1		37.5'
2	$0.12 + 0.5(0.23) = 0.24"$	29.0'
3	$0.11 + 0.5(0.14) = 0.18"$	36.5'
4	$0.21 + 0.5(0.16) = 0.29"$	32.0'
5	$0.18 + 0.5(0.18) = 0.27"$	33.0'
6	$0.13 + 0.5(0.20) = 0.23"$	33.0'
7	$0.10 + 0.5(0.38) = 0.29"$	26.5'
8	$\frac{2}{1'}$	23.0'
9	$0.16 + 0.5(0.12) = 0.22"$	23.5'
10	$0.25 + 0.5(0.10) = 0.30"$	24.0'
11		
12		

PILE #2

S + $\frac{C}{2}$ FOR LAST 6"

$$1'' \quad 0.21 + 0.5(0.20) = 0.31''$$

$$2'' \quad 0.21 + 0.5(0.21) = 0.32''$$

$$3'' \quad 0.20 + 0.5(0.18) = 0.29''$$

$$4'' \quad 0.22 + 0.5(0.20) = 0.32''$$

$$5'' \quad 0.16 + 0.5(0.21) = 0.27''$$

$$6'' \quad 0.12 + 0.5(0.23) = 0.24''$$

Pile # 9

5 + 1/2 FOR LAST 3"

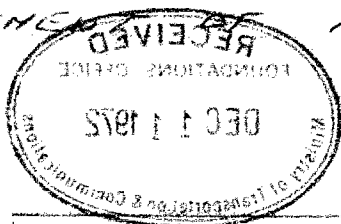
$$1 \quad 0.19 + 0.5(0.08) = 0.23"$$

$$2 \quad 0.18 + 0.5(0.10) = 0.23"$$

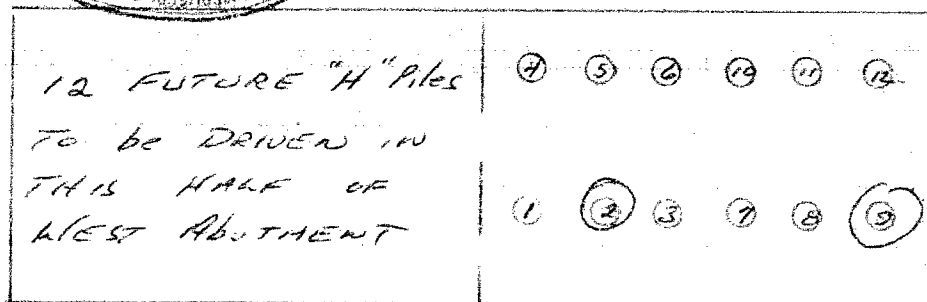
$$3 \quad 0.16 + 0.5(0.12) = 0.22"$$

← 2

"H" PILE LAYOUT FOR SOUTH HALF OF WEST
ABUTMENT OF BOWMANVILLE CREEK STR.



HWY #2



SOUTH HALF OF
WEST ABUTMENT

[Handwritten signature]

MEMORANDUM

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

FROM: C.S. Grebski,
Bridge Office

ATTENTION:

DATE: April 15, 1969

OUR FILE REF.

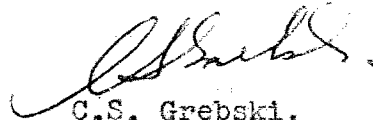
IN REPLY TO

SUBJECT: Bowmanville Creek Bridge
in the Town of Bowmanville
W.P. 421-65, Site 21-169
Highway 2, District 7

68-F-49

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.



C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.c. Foundation Section

N. L. Latta
April 17/69

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 2, 1968

OUR FILE REF.

IN REPLY TO

AUG - 8 1968

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For the
Proposed Crossing At Bowmanville Creek
And
Revised King's Hwy. #2 - Line "C"
Town of Bowmanville Co. of Durham
District No. 7 (Port Hope)
W.J. 68-F-49 W.P. 421-65

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

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

AGS/sf
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
D. P. Collins
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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-

FOUNDATION INVESTIGATION REPORT
For the

Proposed Crossing At Bowmanville Creek
And
Revised King's Hwy. #2 - Line "C"
Town of Bowmanville Co. of Durham
District No. 7 (Port Hope)

W.J. 68-F-49

W.P. 421-65

1. INTRODUCTION:

The Foundation Section was requested to carry out a subsurface investigation at the site of the present crossing of Bowmanville Creek and Hwy. #2 in the Town of Bowmanville, County of Durham. The request was contained in a memo from the Bridge Division (Mr. W. S. Melinyshyn, Regional Bridge Location Engineer), dated May 24th, 1968.

At this location the existing structure will be demolished and replaced with a new wider structure. The centre line of Hwy. #2 will be relocated about 20 ft. south of the existing centre line.

Subsequently an investigation was carried out by this Section to determine the subsoil conditions at the site.

The report contains the results of the investigation together with recommendations pertaining to the foundations of the proposed structure and the stability of the approach fills.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located near the western limits of the Town of Bowmanville, where Hwy. #2 crosses Bowmanville Creek. At this location the creek flows southerly towards Lake Ontario. Bowmanville Creek is in a valley about 38 ft. deep with a valley floor about 200 ft. wide. About 100 ft. north of the existing bridge there is an old dam with a large mill pond behind it. The dam is about 25 ft. high. The water level in the dam is controlled by a stop log system channelling the water

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd) ...

through a wooden apron. Below this apron, and extending to the north edge of the bridge, is a basin of water about 12 ft. deep formed by the waterfall. At the north east corner of the existing bridge there is a feed and flour mill which is still operating on water power. The tail race from the mill flows along the east side of the valley while the main stream flows along the west side, leaving an island in the middle. The creek and tail race bottom are surficially covered with boulders. The water level in both fluctuates in accordance with the mill operations.

The stone pier and abutments of a previous bridge are located beneath the existing structure. The abutments form retaining walls for the fill located in front of the existing bridge abutments. On the west side, the retaining wall has been extended northerly by driving closed - steel sheeting; the top of which is at about mid-height of the old stone abutment. This sheeting retains the highway embankment.

Existing Hwy. #2 is a two-lane paved roadway about 22 feet wide. The associated approach have been constructed with 1½:1 side slopes.

The surrounding terrain is light to heavily wooded with a heavy growth of underbrush.

Physiographically the site is situated in the "Iroquois Sand Plain". Based on available geological information it is known that the overburden in the area is composed of glacial till sheets, lacustrine clay deposited in glacial lake Iroquois is, however, often encountered between till sheets, particularly in the upper 30 to 40 feet. The bedrock is composed of limestone of the Trenton formation.

cont'd /3 ...

3. FIELD AND LABORATORY WORK:

Six boreholes, each with an accompanying dynamic cone penetration test were carried out during the course of the field investigation. The borings were advanced by means of a conventional diamond drill rig adapted for soil sampling purposes.

Samples were obtained at required depths in a 2 inch O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. In addition to the split-spoon samples, two samples were obtained by manually pushing 2-inch I.D. Shelby tubes into the soil. Bedrock was proven in 4 of the boreholes by obtaining AXT size rock core samples. During sampling and drilling operations, detailed logs of the borings were made, these logs contain a record of the drilling and sampling techniques used, together with the soil types encountered.

The location and elevation of all the borings were surveyed in the field by personnel from the Central Region Engineering Surveys Section, and are shown on Drawing 68-F-49A, together with the estimated stratigraphical profile. All elevations in the report are referenced to a Geodetic datum.

All samples were visually examined and identified in the field and later in the laboratory. Laboratory tests were carried out on selected representative samples to determine the physical properties of the overburden namely:

Natural Moisture Contents

Atterberg Limits

Grain-Size Distributions

Organic Contents.

3. FIELD AND LABORATORY WORK: (Cont'd.) ...

On completion of these tests, the various soil samples were classified as to type and consistency in accordance with Unified Soil Classification System (October 1963).

The results of the laboratory tests are plotted on the Record of Borelog sheets and summarized on Figures 1 to 6, inclusive, all contained in the Appendix of the report.

4. SUBSOIL CONDITIONS:

4.1) General

Subsoil at the site generally consists of a 25 to 39 foot thick stratum of clayey silt with a trace of sand and gravel followed by a glacial till deposit composed of a heterogeneous mixture of clay, silt, sand and gravel. The glacial till varies from 6 to 16 feet in thickness. The overburden is underlain by sound limestone bedrock. At the toe of the valley banks, in the vicinity of the existing bridge abutments, the natural subsoil is overlain by 10 to 22 ft. of fill material.

The boundaries between the various soil strata are shown on the Record of Borelog sheets contained in the Appendix of the report. The estimated stratigraphical sections shown on Drawing 68-F-49A, are based on this information.

From ground level downwards, the different soil types are described in detail as follows:

4.2) Fill Material

In BH's 1, 2 and 3, put down on the sides of the valley banks near the existing bridge abutments, up to 22 ft. of fill material was encountered. The fill was placed during construction of the previous

cont'd /5 ...

4. SUBSOIL CONDITIONS: (Cont'd.) ...

and existing bridges. It is predominantly composed of a brown clayey silt with sand and gravel. Occasional boulders up to 7 inches in diameter are located throughout the fill. The fill is quite homogeneous in composition except in BH #2 where occasional layers of silt, up to 6 inches thick, are present. Grain-size distribution curves obtained on two samples of the fill are shown on Figure 1.

Atterberg limit tests carried out on samples of the fill are plotted on the Record of Borelog sheets and summarized on the "Plasticity Chart", Figure 4. These tests gave values for the liquid and plastic limit ranging from 17 to 27 and 15 to 17, respectively; with the corresponding natural content being between 10 and 21%. These results indicate that the fill is primarily inorganic and of low plasticity.

Standard penetration resistance values, carried out within the fill, gave "N" values ranging from 7 to 36 blows/ft. Based on these results it is estimated that the consistency of the basically cohesive fill randomly ranges from firm to hard.

4.3) Clayey Silt with Trace of Sand and Gravel

Underlying the fill material in BH's 1, 2 and 3 and a thin layer of topsoil in BH's 4, 5 and 6, is the predominant overburden stratum across the site, a brown to grey clayey silt with a trace of sand and gravel. The thickness of this deposit varies from 25 to 39 feet. In BH's #1, 2 and 3, a trace of organic matter was encountered within the upper 2 to 3 feet of the stratum. Below the organic zone and extending down to depths of between 5 and 16 feet below ground surface, occasional seams and pockets of silt to sandy

4. SUBSOIL CONDITIONS: (Cont'd) ...

silt are present within the clayey silt stratum. In all borings, with the exception of BH. #3, boulders up to 12 inches in diameter are randomly located throughout the stratum, particularly above elevation 289 i.e., within the upper 3 to 10 feet of the clayey silt. In BH's #2 and 3 a distinct layer of silt with fine sand some 10 feet in thickness was encountered within the stratum. In addition a few silty clay zones, up to 4 inches in thickness, are randomly located throughout the clayey silt stratum, particularly in BH's #2 and 3. Grain-size distribution curves obtained in samples of the stratum are plotted on Figure 2.

Atterberg limit tests, carried out on representative samples of the stratum are plotted on the Plasticity Chart, Figure 5. The results are presented in tabular form below:

		<u>Range</u>
Liquid Limit	(WL)	15-43
Plastic Limit	(WP)	11-21
Natural Moisture Content	(W)	5-27%

The natural water content is generally about 4 to 5 percent above the liquid limit in the upper portion of the stratum decreasing to about 3 to 4 percent below the plastic limit with depth. Based on these results it is estimated that the stratum is basically inorganic and of low plasticity.

Standard penetration tests carried out within the stratum are plotted on the Record of Borelog sheets. This testing indicates that the "N" values within the stratum generally range from 30 blows/ft. to 100 blows/6" with the exception of BH. #3. "N" values of 10 and 22

h. SUBSOIL CONDITIONS: (Cont'd.) ...

blows/ft. were obtained in the upper 5 feet of the clayey silt at this boring location. These lower values seem to indicate that this may be a reworked material. Based on these results it is estimated that the consistency of the stratum varies from very stiff to hard, with the upper "reworked" zone at BH. #3 being in the stiff range.

4.4) Glacial Till (Het. Mixture of Clay, Silt, Sand and Gravel.)

Underlying the clayey silt stratum at all the boring locations is a hard glacial till deposit, ranging from 6 to 16 ft. in thickness. The glacial till is composed of a grey heterogeneous mixture of clay, silt, sand and gravel. Typical grain-size distribution curves for the glacial till are shown on Figure #3.

Atterberg limit tests, carried out on representative samples of the glacial till, are plotted on Figure 6. The results are summarized in tabular form below:

Liquid Limit	(WL)	14-20%
Plastic Limit	(WP)	10-17%
Moisture Content	(W)	6-11%

Based on this testing it is estimated that a portion of the glacial till is cohesive in nature, exhibiting a low plasticity, while other zones are non-cohesive. It is considered that these zones are oriented in a random fashion throughout the deposit.

4.5) Limestone Bedrock

Bedrock was proven in 4 of the wash borings by obtaining from 1 to 9 feet of AXT rock core. The depth at which bedrock was encountered ranged from elev. 253 to 260 i.e., from 35 to 54 feet below

4. SUBSOIL CONDITIONS: (Cont'd.) ...

ground surface. A slight flow of natural gas was noticed just at the surface of the bedrock; this terminated after a short period of time.

The bedrock is composed of a grey limestone of the Trenton formation. It is generally in a sound condition as evidenced by the high core recovery (70-100%). In BH. #5 the bedrock seemed to be fractured.

5. GROUND WATER CONDITIONS:

Groundwater level observations were made in the open holes during the period of the investigation. These observations, which are recorded on the borelog sheets and summarized on Drawing 68-F-49A indicate that the groundwater level in the valley banks was at about elev. 306, or some 12 ft. above creek level. On the valley floor, however, the groundwater level was at about elevation 293 to 295 which corresponds closely with the water level in the creek.

6. EXISTING STRUCTURE:

The existing bridge structure is a two span steel truss structure approximately 220 feet long. The bridge deck, which is 22 feet wide, is composed of concrete. The deck rests on the truss which in turn is supported by a group of 4 piers at its centre line and concrete abutments at either end. The abutments have associated wing walls on either end. The bridge provides a clear span of the order of 25 feet above creek level.

The bridge structure is in a poor state of repair. Severe rusting is noted at various locations along the structure, further some of the steel beams show signs of flaking. In the centre of the structure two small beams are bent about 6 to 8 inches out of line.

cont'd /9 ...

6. EXISTING STRUCTURE: (Cont'd.) ...

There has been some differential settlement at the abutment locations; this movement, has been minor. The north wing walls on both sides of the bridge, however, are cracked and tilted towards the north indicating that some differential settlement has occurred at these locations.

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General

It is proposed to construct a two span (109'-109') structure to replace the existing bridge at the crossing of Hwy. #2 and Bowmanville Creek. The new structure will be 80 feet wide with the centre-line about 20 feet south of the existing highway centre-line. There will be no change in the highway grade at the crossing.

In the vicinity of the existing bridge abutments, from 10 to 22 ft. of fill material, primarily composed of clayey silt with sand and gravel is present. Underlying this fill material on the banks, and from the surface down on the valley floor, is a 25 to 39 ft. thick deposit of very stiff to hard clayey silt with a trace of sand and gravel, the stratum contains occasional boulders and seams or layers of silt and sandy silt throughout. The clayey silt is followed by a glacial till layer, from 6 ft. to 16 ft. in thickness consisting of a heterogeneous mixture of clay, silt, sand and gravel. The overburden deposits are underlain by sound limestone bedrock.

7.2.1.) Pier Foundation

The proposed pier can be founded on a spread footing located within the hard clayey silt stratum, at or below elevation 289 using a safe bearing pressure of 2.5 t.s.f. in design. In order to provide adequate frost protection the underside of the footing should be covered

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

by 5 feet of earth. Settlement of the pier footing will be negligible, providing the foundation soil at and below footing level is not softened due to surface run-off or construction traffic. In this regard it is recommended that as soon as the excavation reaches footing level a working mat of lean concrete or, alternatively, a granular blanket be placed on the base. The pier footing should be protected against scour; the scheme employed will be dependent on hydrological requirements to be provided by the Bridge Hydrology Section.

The pier footing excavation will extend down some 4 to 5 feet below the recorded groundwater level. Groundwater seepage into the excavation should be minor due to the impermeable nature of the clayey silt. If, however, granular water bearing seams and pockets are encountered, as discussed in Section 4.3) of this report, a dewatering scheme may be required.

7.2.2) Abutment Foundations

The proposed abutments can be constructed within the approach fills supported on end bearing piles driven to practical refusal within the hard clayey silt stratum. Pile capacities will be dependent upon the structural strength of the pile section chosen. For example a 14 BP 73 steel "H" pile may be designed for 90 tons per pile. In view of the presence of occasional boulders in the overburden it is recommended that the steel "H" piles be fitted with a driving shoe.

For estimating purposes it can be assumed that the piles may penetrate to an approximate tip elevation of 290-285 at the west abutment location and to an approximate elevation of 280 at the east abutment location.

cont'd /11 ...

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

Due to the competence of the clayey silt stratum at and below tin elevation, settlement of the abutments will be negligible.

7.3) Approach Fills.

The proposed grade of Hwy. #2 will be at the same elevation as the existing roadway (i.e. elev. 330). Construction of the proposed bridge will necessitate widening of the existing highway some 40 ft. on the south side to accommodate the proposed 4 lane highway. The maximum height of approach fills in the widened portion will be of the order of 30 ft. above existing ground surface. No stability problems are anticipated for embankments constructed with standard 2:1 slopes, provided that all top soil is removed on the existing side slopes prior to placing any fill.

Settlement of the foundation subsoil will take place due to the embankment surcharge loading. This settlement will be negligible and elastic in type i.e. - take place during the construction period.

8. SUMMARY:

The results of a foundation investigation for the proposed crossing of Bowmanville Creek and Hwy. #2 in the Town of Bowmanville, County of Durham are reported.

Subsoil at the site consists of a 25 to 39 foot thick stratum of very stiff to hard clayey silt with a trace of sand and gravel followed by a glacial till deposit (6 to 16 feet thick) composed of a heterogeneous mixture of clay, silt, sand and gravel. The overburden is underlain by sound limestone bedrock. In the vicinity of the existing abutments, up to 10 to 22 feet of fill overlies the natural

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

deposits. The groundwater level on the valley floor is at about creek level i.e., elev. 294, while the levels on the valley banks are about 10 to 12 feet higher.

The pier can be founded on a spread footing located at or below elevation 289 using a safe bearing pressure of 2.5 t.s.f. in design. The pier footing excavation will be carried out some 4 to 5 feet below the recorded groundwater level, further the excavation is within close proximity of the creek, for these reasons a dewatering scheme may be required, as discussed in the report.

Abutments, located within the approach fills, can be founded on end - bearing piles driven to practical refusal within the competent clayey silt stratum.

No stability or settlement problems are anticipated for the approach fills of the height contemplated, provided 2:1 slopes are employed, and all organic material is removed prior to the construction of approach fills.

9. MISCELLANEOUS:

The field work, performed during the period of June 17, to July 4, 1968, was supervised by Mr. W. Hutton, Project Foundation Engineer, who also wrote this report.

The investigation was carried out under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who reviewed the report.

Equipment used was owned and operated by F.E. Johnston Drilling Co. Ltd.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOE 68-F-49

LOCATION Sta. 323+04 @ Hwy.#2 Line 'C' o/s 36' Rt.

FOUNDATION SECTION

W. P. 421-65

BORING DATE June 17, 1968

ORIGINATED BY WH

DATUM Geodetic

BOREHOLE TYPE Washboring - Diamond Drilling

COMPILED BY _____ TC

CHECKED BY

[illegible]

FOUNDATION SECTION

ORIGINATED BY WFF

COMPILED BY _____ TO _____

CHECKED BY

[illegible]

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 68-F-49

LOCATION Sta. 322+77 @ Hwy.#2 Line 'C' o/s 39' Lt.

ORIGINATED BY WH

W P 421-65

BORING DATE June 24, 1968

COMPILED BY _____ TO _____

DATUM Geodetic

BOREHOLE TYPE Washboring - Diamond Drill

CHECKED BY

SOIL PROFILE			SAMPLES	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LQUID LIMIT ——— W L	PLASTIC LIMIT ——— W P	WATER CONTENT ——— W	BULK DENSITY	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER TYPE BLOWS / FOOT	ELEV SCALE	20 40 60 80 100		% P O L	P C F	
330.6	Ground Level				400 800 1200 1600 2000		10 20 30		Gr.Sa.Sl.Cl.
0.0	Fill Material			330					
	Clayey silt with sand and gravel.		1 SS 5						
	Firm to stiff Brown		2 SS 7	320					
309.1			3A SS 13						
21.5			4 SS 8	310					Hole caved to elev. 307.6
	Clayey silt to silty clay with trace of sand & gravel		5 SS 10						
	(occ. pockets or layers of silt & sandy silt above El.304)		6 SS 22						
	Stiff to hard.		7 SS 89	300					1 7 52 40
			8 SS 85						
			9 SS 90						
			10 SS 79						
285.1	Brown to grey		11 SS 68	290					
			12 SS 100/7"						
45.5	Silt with fine sand. Very dense.		13 SS 100/4"	280					0 43 54 3
			14 SS 129						
274.6	Grey								
56.0									
270.6									
60.0	Glacial Till		15 SS 123/8"	270					4 40 35 21
265.3	Het.mix. of clay,silt, sand & grav.Hard.Grey		16 SS 100/4"						
65.3	End of Borehole			260					

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 68-F-49

LOCATION Sta. 324+49 @ Hwy. #2 Line 'C' o/s 37' Rt.

ORIGINATED BY WH

W P 421-65

BORING DATE June 28, 1968

COMPILED BY TC

DATUM Geodetic

BOREHOLE TYPE Washboring - Diamond Drill

CHECKED BY _____

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 68-F-49

LOCATION Sta. 324 + 33 @ Hwy.#2 Line 'C' o/s 18' Lt.

FOUNDATION SECTION

W. P. 421-65

BORING DATE July 3, 1968

ORIGINATED BY WH

DATUM Geodetic

BOREHOLE TYPE Washboring - Diamond Drilling

COMPILED BY _____ TC

CHECKED BY

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 68-F-49

LOCATION Sta. 325+12 @ Hwy.#2 Line 'C' o/s 40' Rt.

ORIGINATED BY WH

W.P. 421-65

BORING DATE June 26 - 27, 1968

COMPILED BY WH

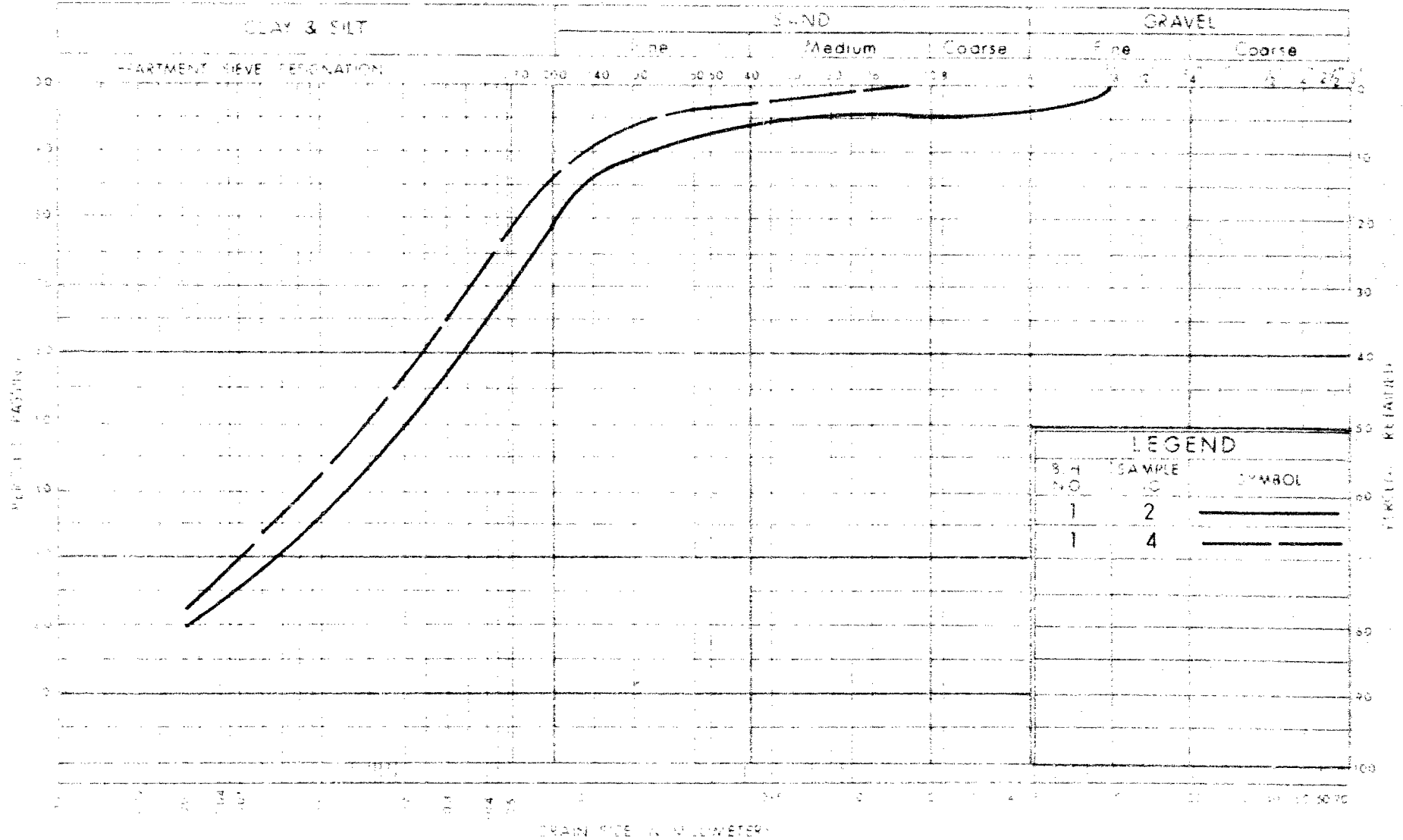
DATUM Geodetic

BOREHOLE TYPE Washboring - Diamond Drilling

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— W _L			BULK DENSITY	REMARKS				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					PLASTIC LIMIT ——— W _P								
							20	40	60	80	100	WATER CONTENT ——— W								
							SHEAR STRENGTH P S F					W _P ——— W ——— W _L			WATER CONTENT %			P C F		
												10 20 30								
293.4																Gr.Sa.Si.Cl				
0.0	Clayey silt with trace of sand & gravel.		1	SS	41	290														
	Occ. boulders up to 4" in diam. above elev. 291.		2	SS	67															
			3	SS	70															
			4	SS	56	280														
	Hard.		5A	SS	46															
			6	SS	66															
	Grey		7	SS	57	270														
268.4			8	SS	66															
25.0	Glacial till.		9	SS	115															
	Het.mix. of clay,silt, sand and gravel.		10	SS	110/6"															
258.4	Hard. Grey.		11	AXT		260														
35.0	Limestone Bedrock			RC	80% Rec.															
251.9	Sound. Grey																			
41.5	End of Borehole					250														

UNIFIED SOIL CLASSIFICATION SYSTEM

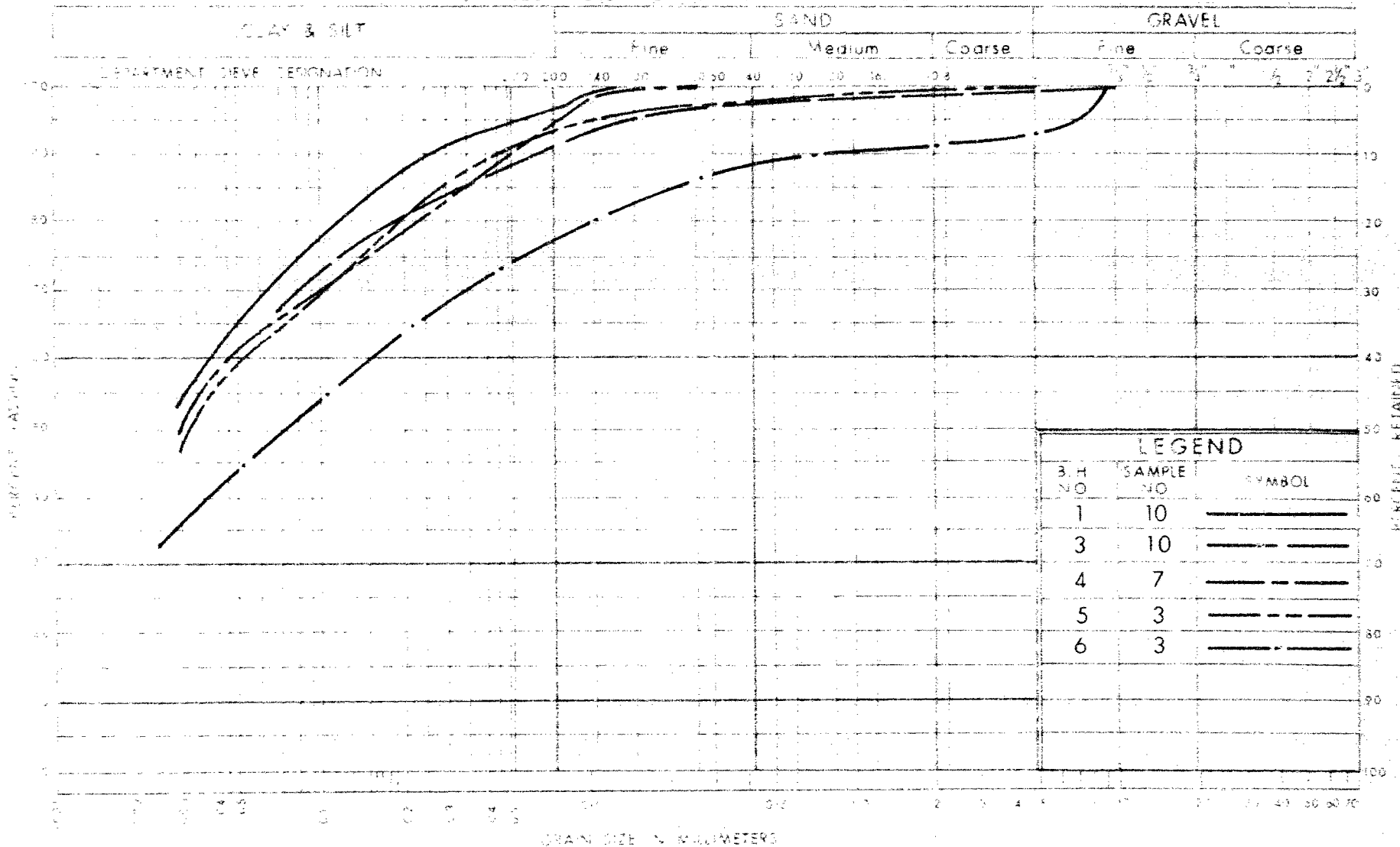


GRAIN SIZE DISTRIBUTION FILL MATERIAL

WP No 421-65
JOB No 68-F-49
FIG. 1

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION
DEFECTS IN NEGATIVE DUE TO CLAYEY SILT WITH SAND & GRAVEL
CONDITION OF ORIGINAL DOCUMENT

UNIFIED SOIL CLASSIFICATION SYSTEM



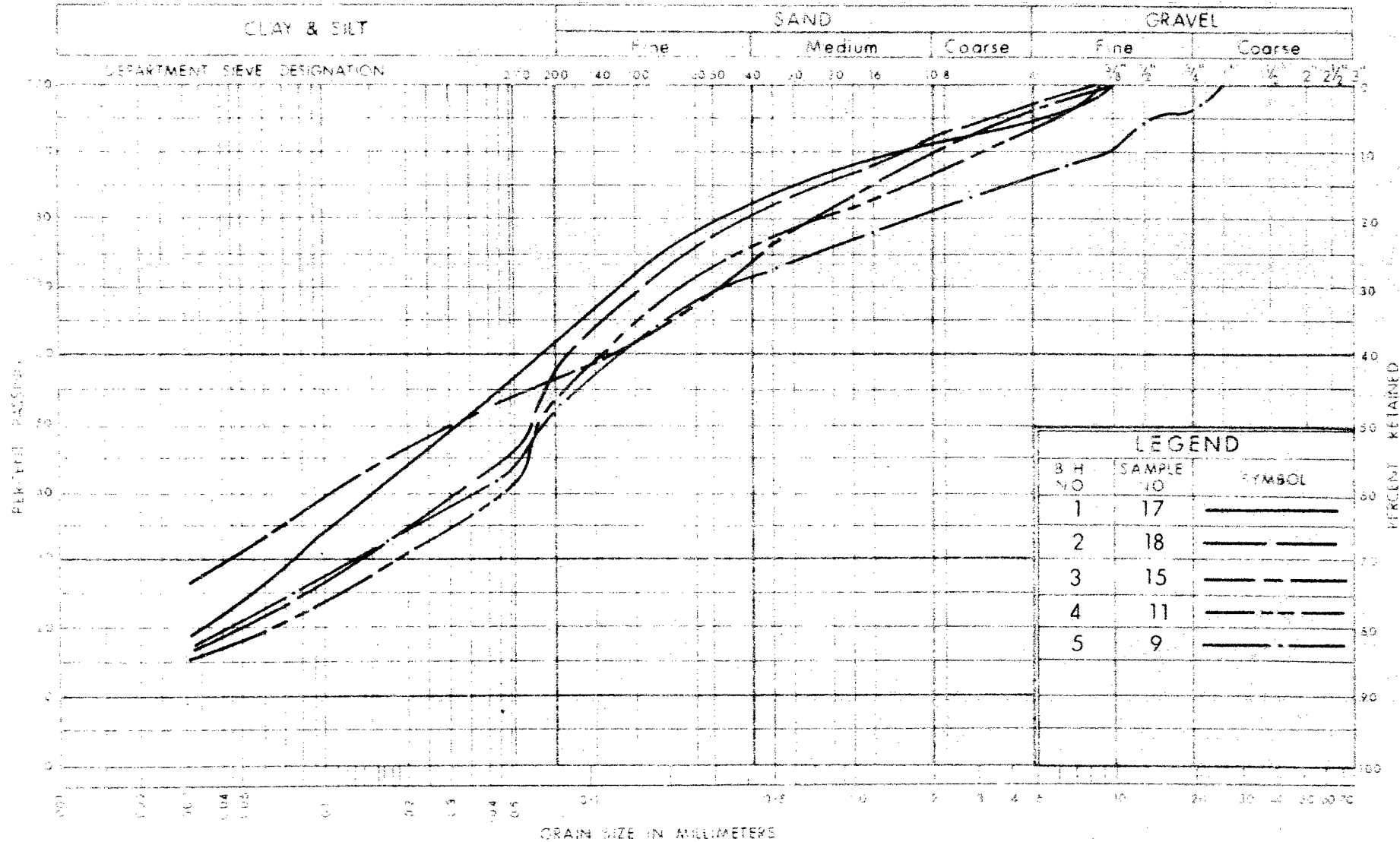
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
WITH TRACES OF SAND & GRAVEL

W.P. No. 421 - 65
JOB No. 68 - F - 49
FIG. 2

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

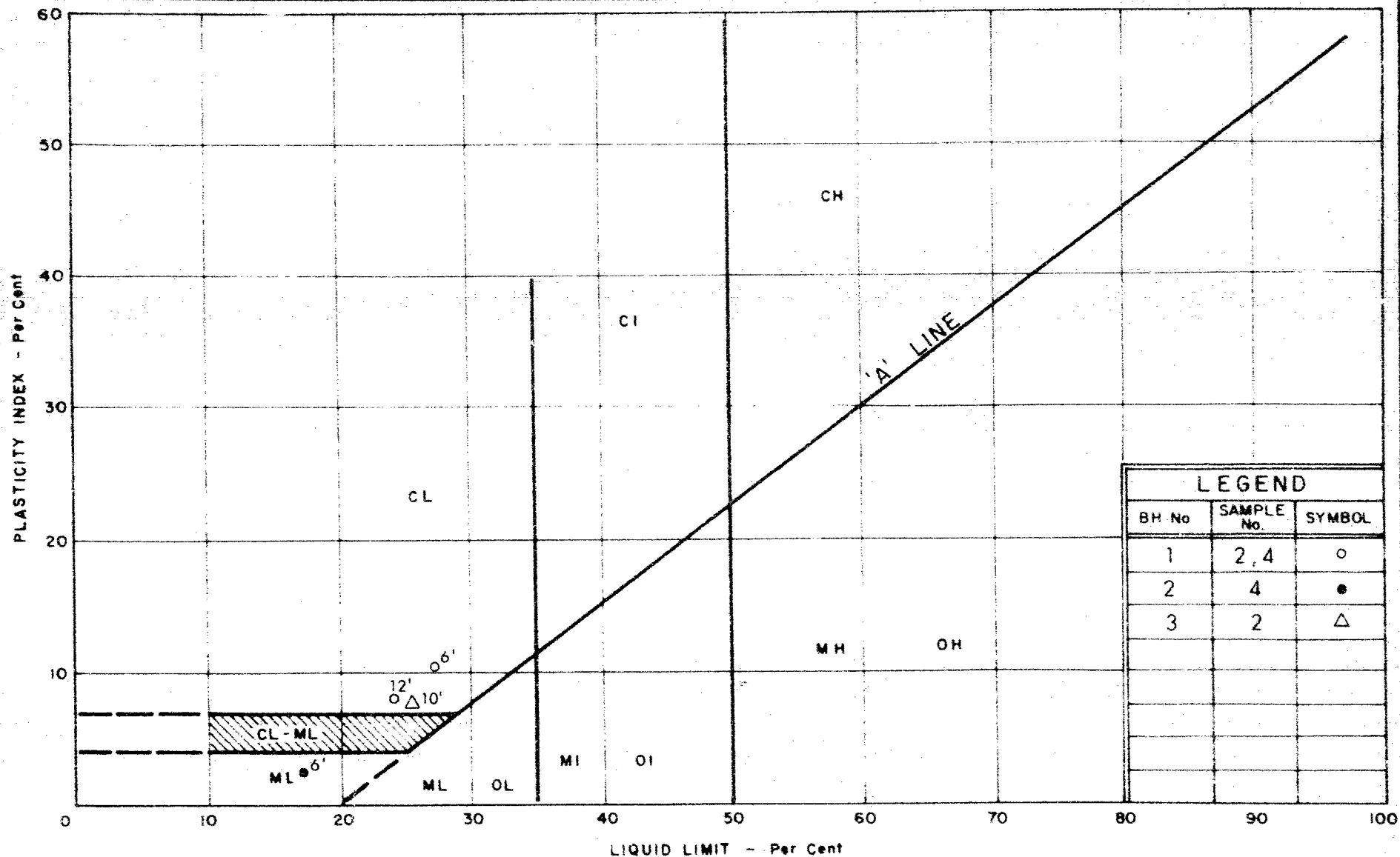
GRAIN SIZE DISTRIBUTION GLACIAL TILL

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

WB No 421-65

JOB No 68-F-49

FIG. 3

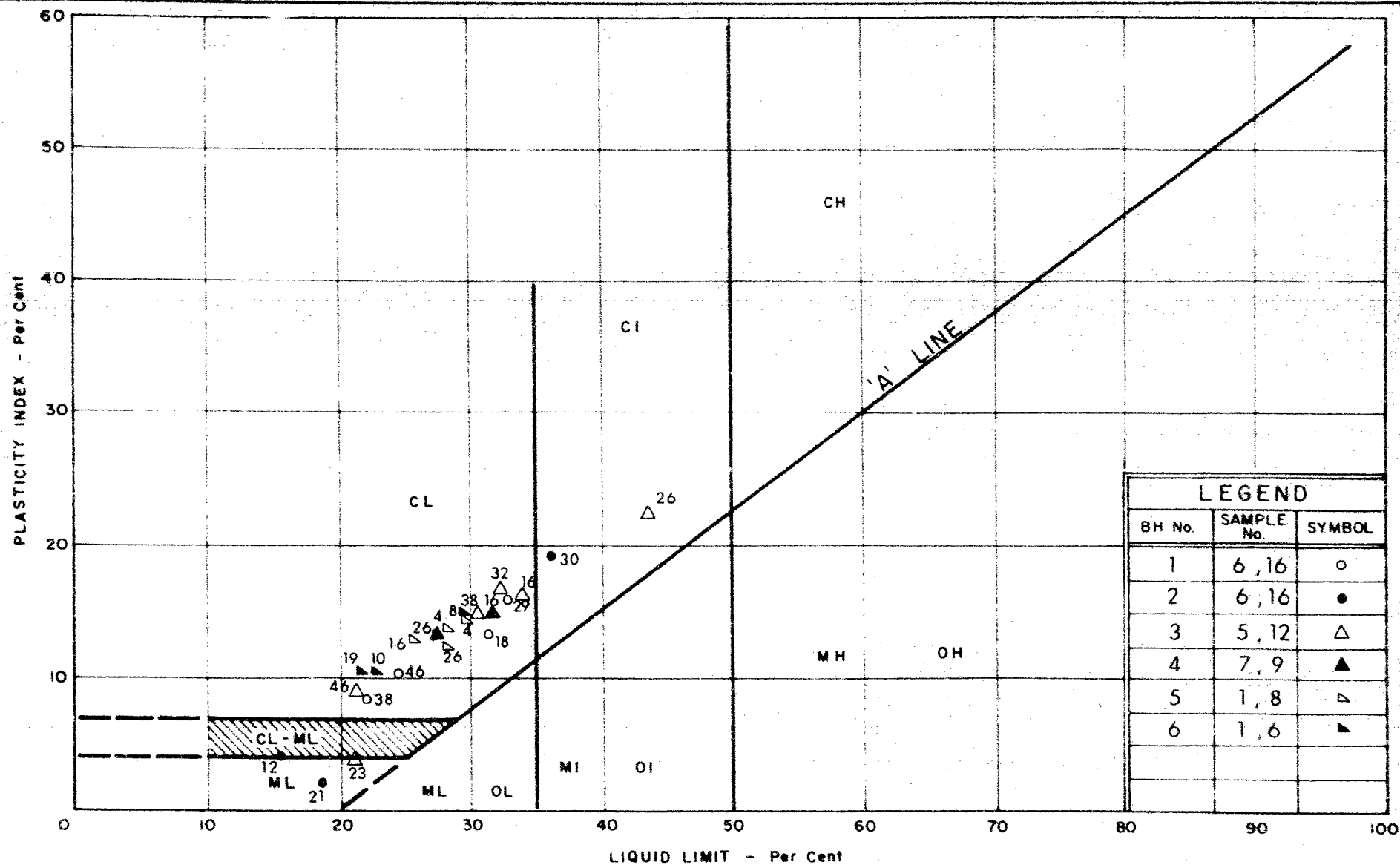


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

PLASTICITY CHART FILL MATERIAL CLAYFY SILT WITH SAND & GRAVEL

WP No. 421-65
JOB No. 68-F-49
FIG. 4

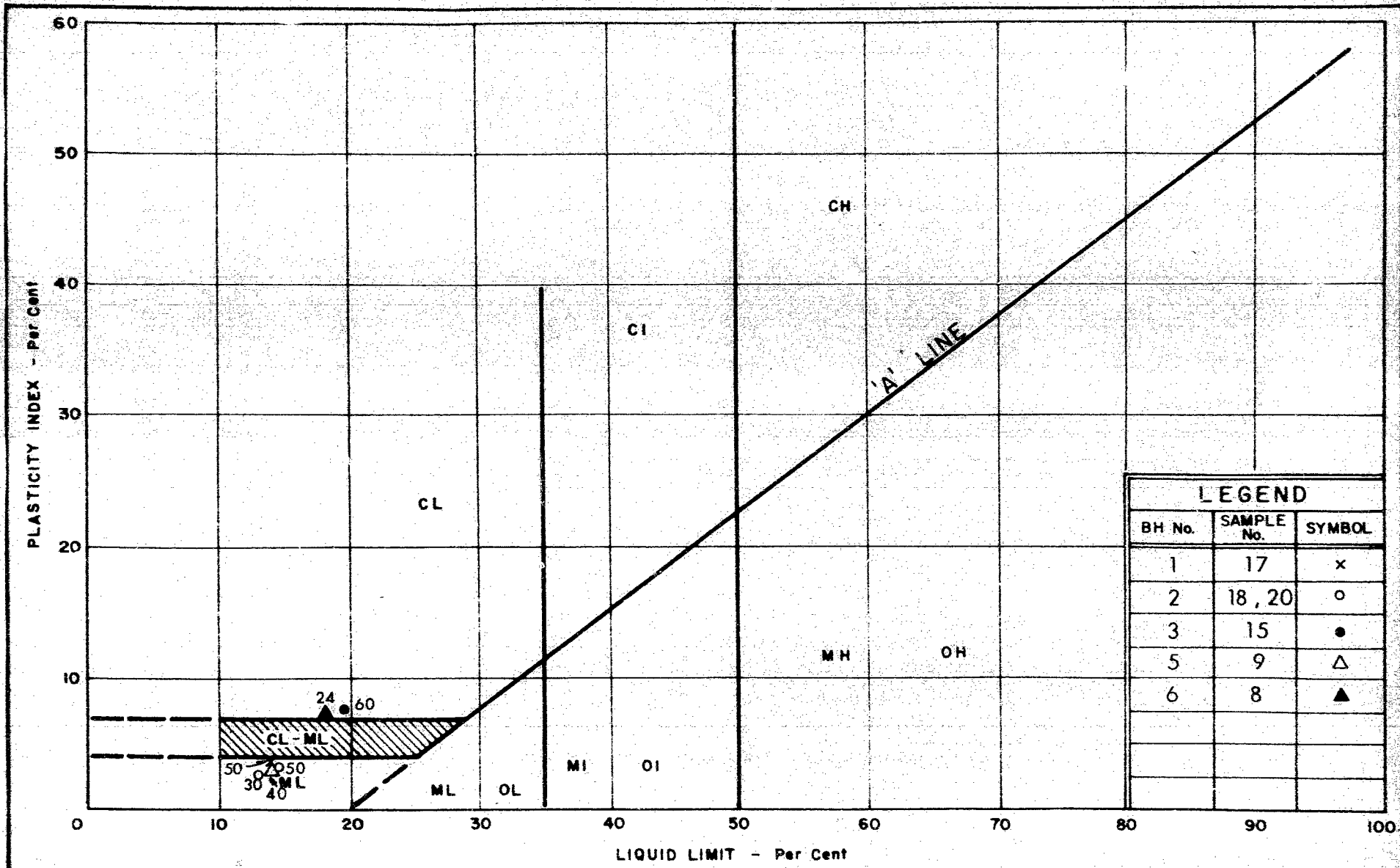


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

PLASTICITY CHART CLAYEY SILT WITH TRACES OF SAND & GRAVEL

W.P. No. 421 - 65
JOB No. 68 - F - 49
FIG. 5



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART
GLACIAL TILL
HET. MIXTURE OF CLAY, SILT, SAND & GRAVEL

W.P. No. 421 - 65
JOB No. 68 - F - 49
FIG. 6

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
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

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

MEMORANDUM

agj

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

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

ATTENTION: Mr. S. McCombie

DATE: January 8, 1969

OUR FILE REF.

IN REPLY TO

JAN - 9 1969

SUBJECT:

ADDENDUM

To

FOUNDATION INVESTIGATION REPORT W.J. 68-F-49

(August 1968)

Proposed New Structure over Bowmanville Creek
Town of Bowmanville -- County of Durham
W.P. 421-65 -- District No. 7 (Port Hope)

The original foundation investigation for the above mentioned site was carried out with the assumption that a two-span structure was contemplated at this location, and the results were submitted in our Foundation Report W.J. 68-F-49. Since then, a change in the design to a three-span structure has been proposed by the Bridge Design Section. As a result of this, we have initiated an additional field investigation to determine the subsoil conditions for the revised three-span structure.

The additional information, consisting of 3 sampled boreholes (B.H.'s #7, 8 & 9), has now been completed and a new drawing incorporating these boreholes, is included with this memo and should be included with your copy(s) of our Foundation Report W.J. 68-F-49.

The recent investigation confirmed our original findings and our recommendations contained in our foundation report are still applicable.

If you have any further queries with respect to this project, please contact our Office.

MD/MdeP

Attach.

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

Foundations Files ✓
Gen. Files

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

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 68-F-49

LOCATION Sta. 324 + 60 @ Hwy. #2 Line 'C'

ORIGINATED BY WH

W P 421-65

BORING DATE December 2, 1968

COMPILED BY HS

DATUM Geodetic

BOREHOLE TYPE MX Casing - washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W _P WATER CONTENT %			BULK DENSITY PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	20	40	60	80	100	W _P	W		
298.0	Ground Level															
0.0	Boulders up to 6" in size. Localized random sandy silt zones.		1	SS	31											
292.0			2	SS	72/6"											
6.0	Clayey silt with a trace of sand and gravel.		3	SS	39											
			4	SS	79											
	Grey		5	SS	67											
			6	SS	76											
			7	SS	82											
	Very stiff to hard.		8	SS	100											
			9	SS	124/11"											
266.0			10	SS	107											
32.0	Glacial Till		11	SS	112/9"											
262.7	Het. mix. of clay, silt, sand & gr. Hard (grey).		12	SS	65/4"											
35.3	End of Borehole															

W.L. in open hole @ Elev 299.5 on Dec. 6/68.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 8

FOUNDATION SECTION

JOB 63-R-49LOCATION 323 + 85 @ Hwy. 2 Line 'C' o/s 40' Rt.ORIGINATED BY WHW P 121-65BORING DATE December 4, 1968COMPILED BY HSDAY/IN GeodeticBOREHOLE TYPE NX Casing - WashboringCHECKED BY /

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS		
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80			100	Wp
292.0	Greek bottom													
0.0	Clayey silt with trace of sand & gravel - deposit intermixed with sand & gravel in upper 3 ft.		1	SS	35	290								
			2	SS	32									
			3	SS	31									
			4	SS	43	280								
			5	SS	73									
			6	SS	71									
268.0	Very stiff to hard. (Grey)		7	SS	83	270								
			8	SS	85/6"									
24.0	Glacial Till Het. mix. of clay, silt, sand & gravel.		9	SS	125/8"									
			10	SS	153									
258.5	Hard. (Grey)		11	SS	100/4"	260								
33.5	End of Borehole													

Creek WL varies between elev. 293 & 295

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

JOB 68-F-49

LOCATION Sta. 323 + 58 @ Hwy. 2 Line 'C' o/s 20' Lt.

ORIGINATED BY WH

W P 421-65

BORING DATE December 6, 1968

COMPILED BY HS

DATUM Geodetic

BOREHOLE TYPE NX Casing - Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— W L		BULK DENSITY	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	20	40	60	80	100	PLASTIC LIMIT ——— W P		
292.0	Creek Bottom												WATER CONTENT ——— W		
													W P ——— W L		
													WATER CONTENT %		
0.0	Clayey silt with a trace of sand and gravel.		1	SS	40	290									
			2	SS	46										
280.5	Hard. (Grey)		3	SS	70										
11.5	End of Borehole					280									

Creek W.L. varies between elev. 293 & 295