

MEMORANDUM

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

ATTENTION: Mr. S. McCombie

OUR FILE REF:

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

DATE: November 28, 1968

IN REPLY TO

DEC - 9 1968

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For

Proposed Crossing at the Realigned
Welland Co. Rd. #11 (East Main St.)
Proposed Canadian National Railway
Crowland Township - Welland County
District No. 4 (Hamilton)
W.J. 68-F-74-1 -- W.P. 240-66-4

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/MdeF
Attach.

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

Foundations Files
Gen. Files

Alfred Sternas
A. G. Sternas
PRINCIPAL FOUNDATION ENGINEER

TABLE OF CONTENTS

1. INTRODUCTION.
 2. DESCRIPTION OF THE SITE AND GEOLOGY.
 3. FIELD AND LABORATORY WORK.
 4. SUBSOIL CONDITIONS:
 - 4.1) General.
 - 4.2) Clayey Silt to Clay.
 - 4.2.1) General.
 - 4.2.2) Strength and compressibility of Clayey Silt to Clay.
 - 4.3) Heterogeneous Mixture of Clay, Silt, Sand and Gravel - Glacial Till.
 - 4.4) Bedrock.
 5. GROUNDWATER CONDITIONS.
 6. HISTORY OF EXISTING M.C.R.R. OVERHEAD STRUCTURE.
 7. DISCUSSION AND RECOMMENDATIONS:
 - 7.1) General.
 - 7.2) Foundations.
 - 7.2.1) Pier Foundations.
 - 7.2.2) Abutment Foundations.
 - 7.3) Settlement Considerations.
 - 7.4) Approach Embankments.
 8. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT

For

Proposed Crossing at the Realigned
Welland Co. Rd. #11 (East Main St.)
Proposed Canadian National Railway
Crowland Township - Welland County
District No. 4 (Hamilton)
W.J. 68-F-74 -- W.P. 240-66-4

1. INTRODUCTION:

The Foundation Section was requested to carry out a subsurface investigation at the site of the realigned Welland County Rd. #11 (easterly extension of Main St. E., Welland) and the proposed C.N.R. alignment in Crowland Twp., County of Welland. The request was contained in a memo from the Bridge Division (Mr. B. S. Richardson, Regional Bridge Project Engineer) dated September 23, 1968. Subsequently, an investigation was carried out by this Section to determine the subsoil conditions at the site. The results of this investigation are presented in this report, together with our recommendations for the design of structure foundations and the stability of the approach fills.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located some 2 miles east of the City of Welland in an area bounded on the north by the twin tracks of the Michigan Central Railroad (M.C.R.R.) and on the south by Welland County Rd. #11. A partially developed residential area surrounds the site. The ground surface in this area is generally flat.

Some 1200 ft. west of the proposed structure is the existing overhead which carries Welland County Rd. #11 over the M.C.R.R. The existing structure is some 270 ft. long (3 spans of about 88'-86'-83') with approach fills some 30 ft. in height above the surrounding ground level. According to available information, the existing structure was built in 1962, and appears to be in a satisfactory condition without any noticeable settlements.

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

The general area is situated in the "Haldimand Clay Plain" physiographic region. In this area, the overburden consists of stratified silts and clays believed to have been deposited in glacial Lake Warren in the geologic past. These lacustrine deposits are underlain by a thin stratum of glacial till followed by shale and limestone bedrock.

3. FIELD AND LABORATORY WORK:

A total of six boreholes was carried out at the site during the course of the field investigation. In addition, one dynamic cone penetration test was carried out adjacent to one of the boreholes. The boreholes were advanced by means of a Penn Drill auger machine to depths of 50 to 70 ft. Deeper boreholes were then advanced by means of a standard 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, or in 2-inch and 3-inch I.D. Shelby tubes which were manually or hydraulically pushed into the soil. The method of driving the split-spoon sampler conformed to the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration test. Field vane tests were carried out, where possible, in the cohesive material of the overburden in order to determine the undrained shear strength of the subsoil. Bedrock was proven by obtaining BXL size rock core samples in 2 boreholes. 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 soil types encountered.

The locations and elevations of all the boreholes are shown on Drawing 68-F-74A, together with the estimated stratigraphical profile across the site. Surveying at the site was carried out by the personnel from the Central Region Engineering

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

Surveys Section; however, the borehole locations were established by Foundation Section personnel. The elevations given in this report are referred to a geodetic datum.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples to determine the following physical properties of the overburden:

- Natural Moisture Contents
- Atterberg Limits
- Bulk Densities
- Grain-Size Distributions
- Undrained Shear Strengths
- Consolidation Characteristics

The results of these tests are plotted on the Record of Borelog sheets as well as on the Figures in the Appendix.

4. SUBSOIL CONDITIONS:

4.1) General:

The predominant stratum across the site is a very stiff to stiff clayey silt to clay with a trace of sand, about 124 feet thick. The upper 13 to 17 feet of this cohesive stratum is desiccated, having a consistency in the hard to very stiff range. Occasional silt seams and layers are present throughout the stratum. Underlying the cohesive deposit is a thin deposit of competent glacial till followed, in turn, by bedrock, the surface of which is at a depth of about 125 feet below ground surface.

4.2) Clayey Silt to Clay:

4.2.1) General:

Underlying a surficial layer of topsoil is the predominant stratum across the site, composed of a brown clayey silt to clay with a trace of sand and gravel. The stratum extends to a depth of

cont'd. /4 ...

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

4.2) Clayey Silt to Clay: (cont'd.) ...

4.2.1) General: (cont'd.) ...

about 123 feet below ground surface. The upper 13 to 17 feet of the deposit is desiccated and fissured, being mottled brown in colour. The gradation of the stratum was determined by carrying out laboratory tests on selected samples. The results of this testing, shown in envelope form on Figure #3, indicate that the granular component of the deposit (material retained on the #200 sieve size) varies from 5 to 15 percent. Thin discontinuous silt and sand seams or lenses are present throughout the stratum. Silt layers, between 2 and 4 feet in thickness, were encountered within or immediately below the desiccated crust in B.H.'s #1, 2, 4 and 6 - i.e., between elevations 590 and 594. An additional silt layer of comparable thickness was encountered at about elevation 545 in B.H. #2.

The Atterberg Limit tests, carried out on representative samples of the clayey silt to clay, are summarized on Figure #2; the results are summarized below:

	<u>Desiccated Zone</u> <u>Range (Average)</u>	<u>Lower Zone</u> <u>Range (Average)</u>
Natural Moisture Content (W) $\%$	16 - 24 (19)	15 - 39 (30)
Liquid Limit (W _L) $\%$	26 - 45 (28)	26 - 40 (35)
Plastic Limit (W _p) $\%$	12 - 15 (18)	14 - 19 (18)
Liquidity Index (I _L) $\%$	0 - 0.3 (0.1)	0.1 - 0.6 (0.4)

These results indicate that, in general, the cohesive stratum is inorganic and of low to intermediate plasticity. The liquidity indices in the lower portion of the deposit are considerably higher than those obtained in the desiccated crust. The total - i.e., wet unit weight of the clayey silt was found to vary from 111 to 137 p.c.f. being on the average about 132 p.c.f.

cont'd. /5 ...

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

4.2) Clayey Silt to Clay: (cont'd.) ...

4.2.2) Strength and Compressibility of Clayey Silt to Clay:

The undrained shear strength of the stratum was measured by in situ vane testing and by laboratory unconfined compression tests. The results are summarized on Figure #1. Based on this testing, it is considered that the undrained shear strength in the desiccated crust is consistently greater than 2,000 p.s.f. Below the desiccated zone the undrained shear strength varies from 750 to 2,000 p.s.f., being typically about 1,200 p.s.f.

The shear strength testing carried out gave a considerable scatter of results. The majority of the lower values recorded on samples tested in the laboratory are probably attributable to disturbance occurring during sampling; such disturbance would tend to remould the material. This is not surprising since the majority of the Shelby tube samples had to be either hydraulically or manually pushed into the soil by the application of large non-uniform pressures. An indication of the disturbance is the consistently high strains at failure (generally in excess of 15 percent). In addition, some of the results obtained on i) samples tested in the laboratory, and ii) by in situ vane testing, were probably adversely affected by the discontinuous silt and sand seams and lenses located throughout the stratum. Taking into consideration the above, it is estimated that the consistency of the clayey silt varies from very stiff to stiff. The upper desiccated zone, however, is in the hard to very stiff range.

The sensitivity of the clayey silt to clay, as measured by several field vane tests, is about 2 to 3; the stratum is thus moderately sensitive.

Standard penetration tests were also carried out within the deposit; the results are summarized on Figure #1. In general, the pattern of 'N' values with depth, corroborates the shear strength profile discussed previously.

cont'd. /6 ...

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

4.2) Clayey Silt to Clay: (cont'd.) ...

4.2.2) Strength and Compressibility of Clayey Silt to Clay: (cont'd.) ...

The consolidation characteristics of the stratum were determined by carrying out three laboratory tests, the results of which are shown as Void Ratio vs. Pressure curves, on Figure #4. The results of this testing indicates that the cohesive stratum is preconsolidated by as much as 5 t.s.f. in excess of existing overburden pressure, within the upper desiccated zone. In the lower non-desiccated portion of the stratum it is estimated that the degree of preconsolidation ranges from about 3 t.s.f., immediately below the "crust", decreasing with depth to as little as 0.5 t.s.f. in excess of existing overburden pressure. It is most difficult to assess the preconsolidation pressure of the stratum by laboratory techniques due to the characteristic curvilinear shape of the oedometer curves (see Figure #4). Because of this the numerical values given previously will, therefore, only indicate the approximate range of the preconsolidation pressure of the cohesive deposit. The initial void ratio (e_0) and compression index (C_c) of the deposit vary from 0.54 to 0.89 and 0.14 to 0.22, respectively.

4.3) Heterogeneous Mixture of Clay, Silt, Sand and Gravel - Glacial Till:

The silty clay to clay stratum is generally underlain by a thin deposit of glacial till, composed of a heterogeneous mixture of clay, silt, sand and gravel. The deposit is essentially non-cohesive in nature; from past experience in the area, however, it is known that random cohesive seams and layers are present throughout. This deposit was encountered at B.H. #2, which was one of the two borings which extended into bedrock. At this boring location the glacial till is about 5 feet thick.

Diamond drilling techniques were required to advance the boring through this deposit, consequently no split-spoon samples

cont'd. /7 ...

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

4.3) Heterogeneous Mixture of Clay, Silt, Sand and Gravel -
Glacial Till: (cont'd.) ...

were obtained. From this it is inferred that the relative density of the glacial till is very dense.

4.4) Bedrock:

Bedrock was established at B.H.'s #2 and 5 by obtaining up to 5 feet of BXL rock core. The bedrock was encountered at a depth of about 125 feet below ground surface - i.e., between elevation 478 and 480.

The bedrock is composed of a grey shale, the upper 3 to 4 feet of which is in a weathered and fractured condition. Numerous mud seams, up to 1/2 inch in thickness, were randomly located throughout this weathered zone. Previous investigations indicated that the bedrock in this area is generally composed of dolomitic limestone with inclusions of gypsum and random interbeds of shale. It is possible, therefore, that the shale bedrock encountered in B.H.'s #2 and 5 is a cap or marker zone overlying limestone bedrock.

5. GROUNDWATER CONDITIONS:

The groundwater level conditions across the site, during the period of the investigation (October, 1968), were observed by taking readings in i) two sealed piezometers installed in B.H. #5 and ii) the open holes at the remaining boring locations. The results of the readings, summarized on Drawing No. 68-F-74A, are tabulated on the following page.

cont'd. /8 ...

5. GROUNDWATER CONDITIONS: (cont'd.) ...

<u>B.H. No.</u>	<u>Ground Surface Elevation</u>	<u>Tip Elevation of Piezometer</u>	<u>Groundwater Level Elevation</u>
1	603.3	-	598.3
2	603.0	-	597.0
5	604.0	546	595.6
5	604.0	477	Dry to *
			580.0
6	603.5	-	591.1 **

* Piezometer installed in glacial till stratum, immediately above bedrock.

** Insufficient time available (during period of investigation) for water level to stabilize in open borehole.

Based on these readings, it is concluded that the piezometric groundwater level in the upper cohesive portion of the overburden is generally within 6 to 8 feet of ground surface.

The deepest of the piezometers installed in B.H. #5 is located in the basal till deposit. This piezometer was dry to a depth of at least 25 feet below ground surface. This would seem to indicate that the piezometric groundwater level in the basal till and upper weathered and fractured portion of the bedrock is considerably lower than that in the overlying cohesive stratum. Previous subsurface investigations in the area corroborate this inference. Further, it has previously been concluded that this decrease in piezometric level with depth is indicative of downward drainage from the overlying cohesive stratum, which is in hydraulic communication with the lower basal glacial till and bedrock. The glacial till and upper weathered and fractured zone of the bedrock can, in fact, be considered as a confined aquifer.

cont'd. /9 ...

6. HISTORY OF EXISTING M.C.R.R. OVERHEAD STRUCTURE:

The existing overhead structure carrying Welland County Rd. No. 11 over the Michigan Central Railway (M.C.R.R.) is located approximately 1,200 feet south-west of the proposed overhead at the crossing of realigned County Rd. #11 and the C.N.R. The structure and approaches were constructed by the County of Welland in 1962.

The subsurface investigation for the existing structure was carried out in November and December, 1961, by W. A. Trow and Associates Limited. The factual data obtained from this investigation, together with their engineering recommendations pertaining to design of foundations and related earthworks, were presented in Report No. J 608, submitted in January, 1961. It is pertinent to note that the subsoil conditions at the site of the existing structure are very similar to those at the site of the proposed overhead, namely: there is an extensive deposit of clayey silt to silty clay, the continuity of which is occasionally interrupted by layers of silt of irregular thickness. Further, the consistency and compressibility characteristics of the cohesive strata at either site are similar.

The soil consultant carried out detailed settlement analyses based on empirical laboratory test values obtained on representative, relatively undisturbed, samples. From the results of these analyses it was predicted that: 1) the embankment could be subjected to settlements of up to 23 inches in areas where the fill heights are of the order of 30 feet, and 1i) the maximum differential settlement occurring between the abutments and piers, to be installed immediately following placement of the approach fills, could be about 5 inches. This Section, in a letter (dated January 30, 1961) to De Leuw Cather and Co. Ltd., who were acting as consultants to the County of Welland, suggested a construction scheme. It was recommended the approach fills, plus a nominal surcharge, be constructed and allowed to remain in place in order to induce a considerable portion of the settlement prior to installation of a multi-span, simply-supported structure. The sequence of construction employed is discussed in the following paragraph.

6. HISTORY OF EXISTING M.C.R.R. OVERHEAD STRUCTURE: (cont'd.) ...

The approach embankments, with the addition of the surcharge fill, were brought up to full height within two weeks commencing on or about May 23, 1962. The maximum height of fill was about 44 feet, with the surcharge accounting for about 12 feet. The majority of the fill was composed of the locally available clayey silt. The surcharge was left in place for a period exceeding 5 months prior to removal and subsequent construction of the sub-structure elements. The piers were founded on spread footings located, at a shallow depth, in the upper desiccated "crust" of the cohesive stratum. The abutments were constructed on spread footings perched in the approach fills; the footings were founded on and within a well compacted zone of granular material. An allowable bearing pressure of 2.5 t.s.f. was used in the design of all footings.

The settlement of the approach fills, together with the surcharge, was monitored by the D.H.O. in accordance with a request from the County of Welland. The results of these observations indicate that combined settlement of the compacted fill and the foundation subsoil was of the order of 4 to 5 inches; settlement of the fill itself accounts for about 2 inches of the total given above. The results obtained seem to indicate that the majority of the settlement was realized within a very short period of time following placement, nominally within about 1-1/2 to 2 months. It should be noted that the magnitude of the observed settlements are considerably less than those theoretically computed.

Based on information provided by Welland County authorities, as well as observations made by us at the time of the present field investigation, it appears that the structure complex is performing satisfactorily without any visible signs of distress due to either instability of the approach fills, or excessive differential settlements between the various structure elements.

cont'd. /11 ...

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General:

It is proposed to construct a 3-span (45'-50'-45') structure at this site. The maximum height of the associated approach fills will be of the order of 30 feet.

The predominant deposit across the site is a stratum of stiff to very stiff clayey silt to clay approximately 123 feet in thickness, the upper 13 to 17 feet of the deposit being desiccated with the consistency in the hard to very stiff range. Random silt layers, up to 4 feet in thickness, are interbedded within the cohesive stratum. The clayey silt to clay is underlain by a thin glacial till deposit, which in turn, is followed by shale bedrock.

7.2) Foundations:

7.2.1) Pier Foundations:

The upper desiccated zone of the cohesive stratum is a competent foundation subsoil. Based on this, it is recommended that the piers be founded on spread footings founded in the "crust" at or above elevation 598. A minimum soil cover of 4 feet should be provided to satisfy the frost protection requirements in the area. Spread footings so founded, may be designed using a safe bearing pressure of 2.5 t.s.f. Settlement of the foundation subsoil at and below footing level, will occur due to the induced footing pressure; the magnitude of these settlements are discussed in Sub-section 7.3.

Pier footing excavations will extend some 4 to 5 feet below existing ground surface. No major dewatering problems are anticipated, since the subsoil is relatively impervious.

7.2.2) Abutment Foundations:

The abutments of the proposed structure could be constructed within the approach fills on spread footings. The fill material, below the spread footings, should consist of well compacted G.B.C. 'A' material and should extend for a horizontal

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

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

7.2.2) Abutment Foundations: (cont'd.) ...

distance of at least 10 feet from the footing edges in the plane of the footing tops. This portion of the fill should be constructed with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for at least a distance of 50 feet behind the abutments before re-excavation for the footings. A design load of 2.0 t.s.f. may be used in design of spread footings.

As an alternative, the proposed abutments may be constructed within the approach fills and supported on 12-3/4 inch O.D. x 1/4 inch wall thickness, closed-end steel tube piles driven 10 feet into the desiccated crust; in no case should the final pile tip elevation be lower than 583. The piles can be designed for a safe capacity of 20 tons/pile.

Care should be taken to ensure that rock or bouldery fill is not placed within the areas in which piles have to be driven.

The differential settlement between the various structure elements is the governing factor in determining the type of structure to be employed at this location. The Sub-section to follow will discuss this aspect in detail.

7.3) Settlement Considerations:

Computations, based on Schmertmann's* method, have been carried out to determine the consolidation settlement of the foundation subsoil due to the pier footing and embankment loading.

*Schmertmann, J. H. -

"The Undisturbed Consolidation Behaviour of Clay" -
American Society of Civil Engineers" - 1955.

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

7.3) Settlement Considerations: (cont'd.) ...

Results of the analyses are summarized in tabular form below:

1) Ultimate settlements of the end pier locations -		
Induced by the footing pressure of 2.5 t.s.f. (footing size 10' x 60')	≈	2-1/2"
Induced by embankment loading (30-ft. height)	≈	1/2"
Total	≈	3"
2) Ultimate settlement at the abutment locations (Induced by embankment)	≈	10"

These values represent the total long-term consolidation settlement, the majority of which should occur within 1 year, with about 50% occurring within 3 months.

Because of the predicted relatively rapid rate of settlement it would be extremely advantageous to construct the approach fills prior to construction of the structure footings in order to reduce the differential settlements between the various elements. In this regard it is recommended that, if scheduling allows, the fills be constructed and remain in place for a period of between 3 and 6 months. For example, it is estimated that, if a staging period of 6 months is allowed, the differential settlement between the abutments and end piers would be of the order of 2 to 2-1/2 inches.

As discussed in Section 7, the actual settlements of the existing structure were considerably less than that theoretically computed. It is most probable, therefore, that the computed differential settlements are the upper limiting range of those likely to take place.

7.4) Approach Embankments:

The approach embankments will have a maximum height of about 30 feet. For fills of this magnitude, no stability problems

cont'd. /14 ...

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

7.4) Approach Embankments: (cont'd.) ...

are anticipated, provided standard 2:1 slopes are adopted. The maximum computed consolidation settlement of the embankment will be of the order of 10 inches. The time rate of settlement will be similar to that discussed previously.

8. MISCELLANEOUS:

The field work, performed during the period October 8 - 18, 1968, was undertaken by Mr. C. Mirza, Project Foundation Engineer. Equipment used was owned and operated by F. E. Johnston Limited.

The preparation of the report was carried out by Messrs. C. Mirza and B. T. Darch, Senior Foundation Engineer, under the general supervision of Mr. M. Devata, Supervising Foundation Engineer.

November, 1968

APPENDIX I

RECORD OF BOREHOLE NO. 1

FOUNDATION SE

MATERIALS & TESTING DIVISION

JOB 68-7-74

LOCATION Sta. 68+00 E Prop. County Rd. #11

ORIGINATED BY CM

W.P. 240-66-4

BORING DATE Oct. 9-10, 1968

COMPILED BY CM

DAYUM Geodetic

BOREHOLE TYPE Cont. flight auger - Penn Drill

CHECKED BY

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL		BULK DENSITY	REMARKS				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	SHEAR STRENGTH P.S.F.			PLASTIC LIMIT — WP		WATER CONTENT — W	
							o Unconfined			wp — w — wl			
							500 1000 1500 2000 2500	WATER CONTENT %		15 30 45			
603.3	Ground Level												
0.0	Desiccated & Fissured		1	SS	28	600							
			2	SS	25								
	Hard to very stiff		3	TW	PH			σ					132
	Mottled Brown		4	TW	PH	590		6250					135
586.3			5	TW	PH			δ					137
17.0			6	TW	PH								
	Clayey silt to silty		7	TW	PH								
	clay trace of sand		8	TW	PH			δ					133
	and gravel.		9	TW	PH								
			10	TW	PH								
	Stiff to firm		11	SS	11								137
	Brown		12	TW	PH								112
	Occ. silt seams		13	TW	PH								
533.3													
70.0	End of Borehole												

0
150 % Strain at failure
10

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 66-P-74

LOCATION Sta. 67-22 1/2 Prop. County Rd. #11 o/s 35' RL.

ORIGINATED BY CM

W.P. 240-66-4

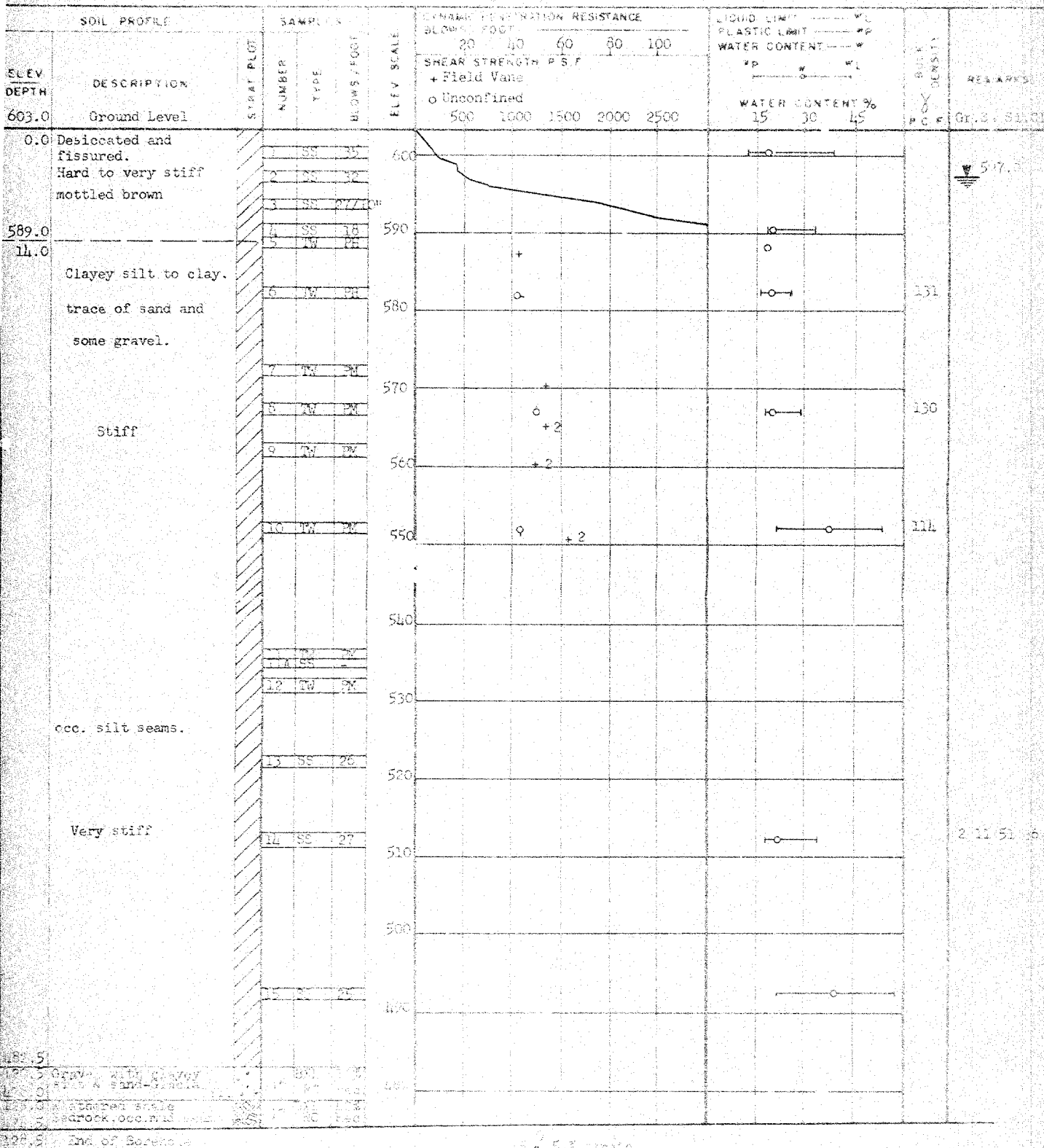
BORING DATE October 6-17 1968

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Cont. Flight Auger & Washboring - BX Casing

CHECKED BY



RECORD OF BOREHOLE NO.

SECTION

OFFICIALS & TESTING DIVISION

DATE: 10-1-54 LOCATION: ROUTE 100, 1000 Yds. County: Boone Co. #100-100-100 ORIGINATED BY: J. H. HARRIS
 BORING DEPT: 015-100-100 COMPLETED BY: J. H. HARRIS
 NAME: Geodetic BOREHOLE TYPE: 100-100-100 (Hand Drill) CHECKED BY: J. H. HARRIS

TIME OF DAY	SAMPLES	PENETRATION RESISTANCE					WATER CONTENT	
		100	200	300	400	500	10	20
10:00 AM	1	100	100	100	100	100	10	10
10:05 AM	2	100	100	100	100	100	10	10
10:10 AM	3	100	100	100	100	100	10	10
10:15 AM	4	100	100	100	100	100	10	10
10:20 AM	5	100	100	100	100	100	10	10
10:25 AM	6	100	100	100	100	100	10	10
10:30 AM	7	100	100	100	100	100	10	10
10:35 AM	8	100	100	100	100	100	10	10
10:40 AM	9	100	100	100	100	100	10	10
10:45 AM	10	100	100	100	100	100	10	10
10:50 AM	11	100	100	100	100	100	10	10
10:55 AM	12	100	100	100	100	100	10	10
11:00 AM	13	100	100	100	100	100	10	10
11:05 AM	14	100	100	100	100	100	10	10
11:10 AM	15	100	100	100	100	100	10	10
11:15 AM	16	100	100	100	100	100	10	10
11:20 AM	17	100	100	100	100	100	10	10
11:25 AM	18	100	100	100	100	100	10	10
11:30 AM	19	100	100	100	100	100	10	10
11:35 AM	20	100	100	100	100	100	10	10
11:40 AM	21	100	100	100	100	100	10	10
11:45 AM	22	100	100	100	100	100	10	10
11:50 AM	23	100	100	100	100	100	10	10
11:55 AM	24	100	100	100	100	100	10	10
12:00 PM	25	100	100	100	100	100	10	10
12:05 PM	26	100	100	100	100	100	10	10
12:10 PM	27	100	100	100	100	100	10	10
12:15 PM	28	100	100	100	100	100	10	10
12:20 PM	29	100	100	100	100	100	10	10
12:25 PM	30	100	100	100	100	100	10	10
12:30 PM	31	100	100	100	100	100	10	10
12:35 PM	32	100	100	100	100	100	10	10
12:40 PM	33	100	100	100	100	100	10	10
12:45 PM	34	100	100	100	100	100	10	10
12:50 PM	35	100	100	100	100	100	10	10
12:55 PM	36	100	100	100	100	100	10	10
1:00 PM	37	100	100	100	100	100	10	10
1:05 PM	38	100	100	100	100	100	10	10
1:10 PM	39	100	100	100	100	100	10	10
1:15 PM	40	100	100	100	100	100	10	10
1:20 PM	41	100	100	100	100	100	10	10
1:25 PM	42	100	100	100	100	100	10	10
1:30 PM	43	100	100	100	100	100	10	10
1:35 PM	44	100	100	100	100	100	10	10
1:40 PM	45	100	100	100	100	100	10	10
1:45 PM	46	100	100	100	100	100	10	10
1:50 PM	47	100	100	100	100	100	10	10
1:55 PM	48	100	100	100	100	100	10	10
2:00 PM	49	100	100	100	100	100	10	10
2:05 PM	50	100	100	100	100	100	10	10
2:10 PM	51	100	100	100	100	100	10	10
2:15 PM	52	100	100	100	100	100	10	10
2:20 PM	53	100	100	100	100	100	10	10
2:25 PM	54	100	100	100	100	100	10	10
2:30 PM	55	100	100	100	100	100	10	10
2:35 PM	56	100	100	100	100	100	10	10
2:40 PM	57	100	100	100	100	100	10	10
2:45 PM	58	100	100	100	100	100	10	10
2:50 PM	59	100	100	100	100	100	10	10
2:55 PM	60	100	100	100	100	100	10	10
3:00 PM	61	100	100	100	100	100	10	10
3:05 PM	62	100	100	100	100	100	10	10
3:10 PM	63	100	100	100	100	100	10	10
3:15 PM	64	100	100	100	100	100	10	10
3:20 PM	65	100	100	100	100	100	10	10
3:25 PM	66	100	100	100	100	100	10	10
3:30 PM	67	100	100	100	100	100	10	10
3:35 PM	68	100	100	100	100	100	10	10
3:40 PM	69	100	100	100	100	100	10	10
3:45 PM	70	100	100	100	100	100	10	10
3:50 PM	71	100	100	100	100	100	10	10
3:55 PM	72	100	100	100	100	100	10	10
4:00 PM	73	100	100	100	100	100	10	10
4:05 PM	74	100	100	100	100	100	10	10
4:10 PM	75	100	100	100	100	100	10	10
4:15 PM	76	100	100	100	100	100	10	10
4:20 PM	77	100	100	100	100	100	10	10
4:25 PM	78	100	100	100	100	100	10	10
4:30 PM	79	100	100	100	100	100	10	10
4:35 PM	80	100	100	100	100	100	10	10
4:40 PM	81	100	100	100	100	100	10	10
4:45 PM	82	100	100	100	100	100	10	10
4:50 PM	83	100	100	100	100	100	10	10
4:55 PM	84	100	100	100	100	100	10	10
5:00 PM	85	100	100	100	100	100	10	10
5:05 PM	86	100	100	100	100	100	10	10
5:10 PM	87	100	100	100	100	100	10	10
5:15 PM	88	100	100	100	100	100	10	10
5:20 PM	89	100	100	100	100	100	10	10
5:25 PM	90	100	100	100	100	100	10	10
5:30 PM	91	100	100	100	100	100	10	10
5:35 PM	92	100	100	100	100	100	10	10
5:40 PM	93	100	100	100	100	100	10	10
5:45 PM	94	100	100	100	100	100	10	10
5:50 PM	95	100	100	100	100	100	10	10
5:55 PM	96	100	100	100	100	100	10	10
6:00 PM	97	100	100	100	100	100	10	10
6:05 PM	98	100	100	100	100	100	10	10
6:10 PM	99	100	100	100	100	100	10	10
6:15 PM	100	100	100	100	100	100	10	10

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 68-P-74 LOCATION Sta. 70 + 24 @ Prop. County Rd. #11, o/s 35' Rt. ORIGINATED BY CM
W P 240-66-4 BORING DATE October 16, 1968 COMPILED BY CM
DATE Geodetic BOREHOLE TYPE Cont. Flight Auger - Penn Drill CHECKED BY ✓

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	Liquid Limit — WL Plastic Limit — PL Water Content — W	BULK DENSITY	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYP.	BLOWS / FOOT	SHEAR STRENGTH P.S.F. + Field Vane o Unconfined	WATER CONTENT % WP — WL	P.C.F.	
603.3	Ground Level					500 1000 1500 2000 2500	15 30 45		
0.0	Desiccated and Fissured Hard to very stiff		1	SS	31	600			
	Mottled Brown		2	SS	41				
591.3			3	SS	28				
12.0			4	TW	PH	590	+		130
	Clayey silt to silty clay		5	SS	41				
			6	SS	LL		+2		134
	Stiff to Firm		7	TW	PH				
	Trace of sand & gravel		8	SS	10	580	+2		
			9	TW	PH				
	Occ. silt seams		10	TW	PH	570	+2		
			11	SS	11		+2		
			12	TW	PH	560	+2		131
	Brown		13	SS	8	550	+3		
			14	TW	PH	540			127
532.0			15	TW	PH	530			
71.3	End of Borehole					0 15-5% strain 10			

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 68-E-74

W P 240-66-4

DATUM Geodetic

LOCATION Sta. 71+03 @ Prop. County Rd.#11 o/s 35' Lt.

BORING DATE Oct. 9-15, 1968

WOREHOLE TYPE Washboring - NX- BX Casings

* ORIGINATED BY: CM

COMPILED BY _____ CM

CHECKED BY

SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W		PULV. DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	SHEAR STRENGTH P.S.F. + Field Vane o Unconfined x Lab Vane	W.P. W.L.	WATER CONTENT % 15 30 45		
604.0	Ground Level									
0.0	Desiccated and Fissured		1	SS	70					
	Hard to very stiff		2	SS	95					
	Mottled Brown		3	TW	84					
			4	SS	57					
588.0			5	SS	53					
16.0			6	PM	34					
			7	TW	PM					
			8	SS	29					
			9	SS	21					
			10	TW	PM					
			11	TW	PM					
			12	SS	15					
			13	TW	PM					
	Clayey silt to silty clay with trace to some sand & gravel, occ. silt seams		14	TW	Lost					
			14A	SS	-					
			15	TW	PM					
			16	TW	PM					
	Very stiff to firm.		17	TW	PM					
	Red brown to brown.		18	SS	17					
			19	TW	PM					
			20	SS	15					
			21	SS	16					
	Pockets of sand and gravel.		22	SS	27					
			23	SS	30					
			24	SS	30					
125.0	Weathered shale bedrock w/occ. mud seams.		25	SS	131					
473.5			26	HXL	252					
130.5	End of Borehole			NC	Rec.					

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 68-F-74

LOCATION

Sta. 72+20 @ Prop. County Rd. #13

ORIGINATED BY CM

W.P. 240-66-1

BORING DATE

Oct. 11-15, 1968

COMPILED BY CM

DATUM Geodetic

BOREHOLE TYPE

Cont. flight auger - Penn Drill

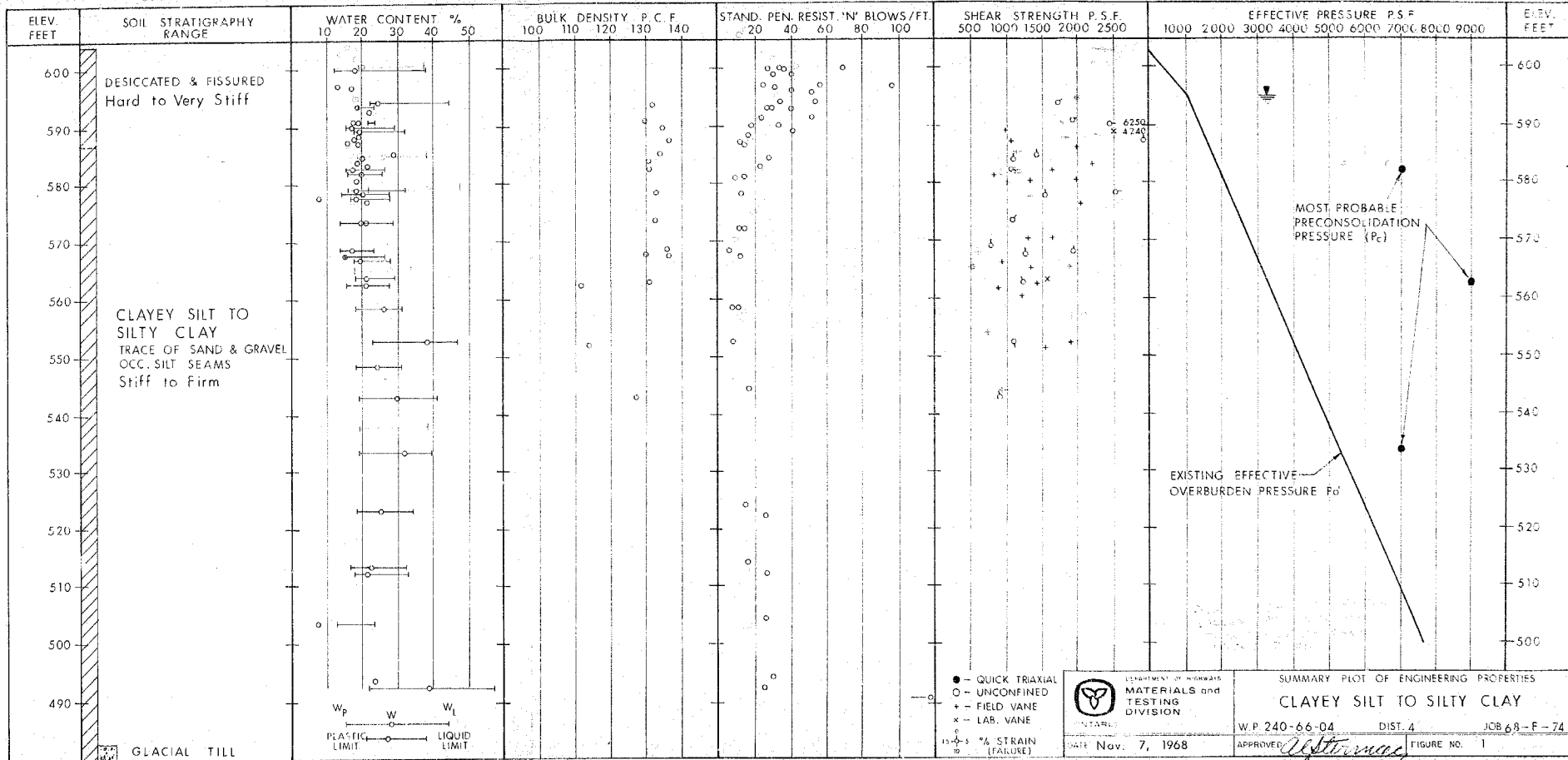
CHECKED BY

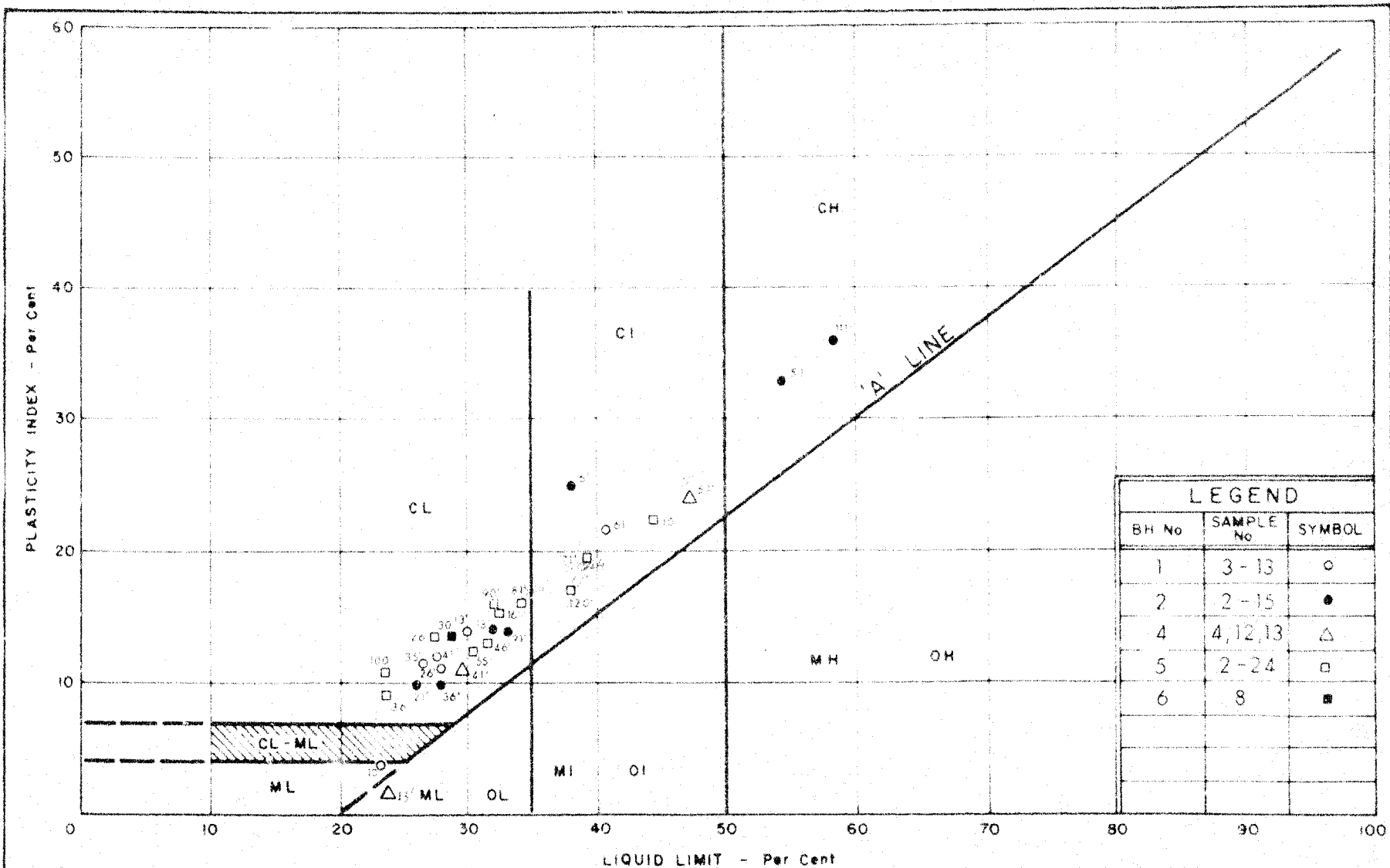
SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT						LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P C F	REMARKS	
ELEV DEPTH	DESCRIPTION	STRAT. PLCT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P S F + Field Vane x Lab Vane					WATER CONTENT % 15 30 45					
603.5	Ground Level						500	1000	1500	2000	2500						
0.0	Desiccated and Fissured Hard Mottled Brown		1	SS	36	600											
			2	SS	57												
			3	SS	35												
590.0			4	SS	25	590											
13.5			5	SS	17												
			6	TW	PH												
	Clayey silt with trace of sand & gravel		7	SS	13	580											
			8	TW	PH												
	Stiff		9	SS	7	570											
	Brown		10	TW	PM												
	Occ. silt seams		11	SS	9	560											
			12	TW	PM												
548.5					550												
55.0	End of Borehole																
						540											

591.1

x6

+2





DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART CLAYEY SILT TO CLAY

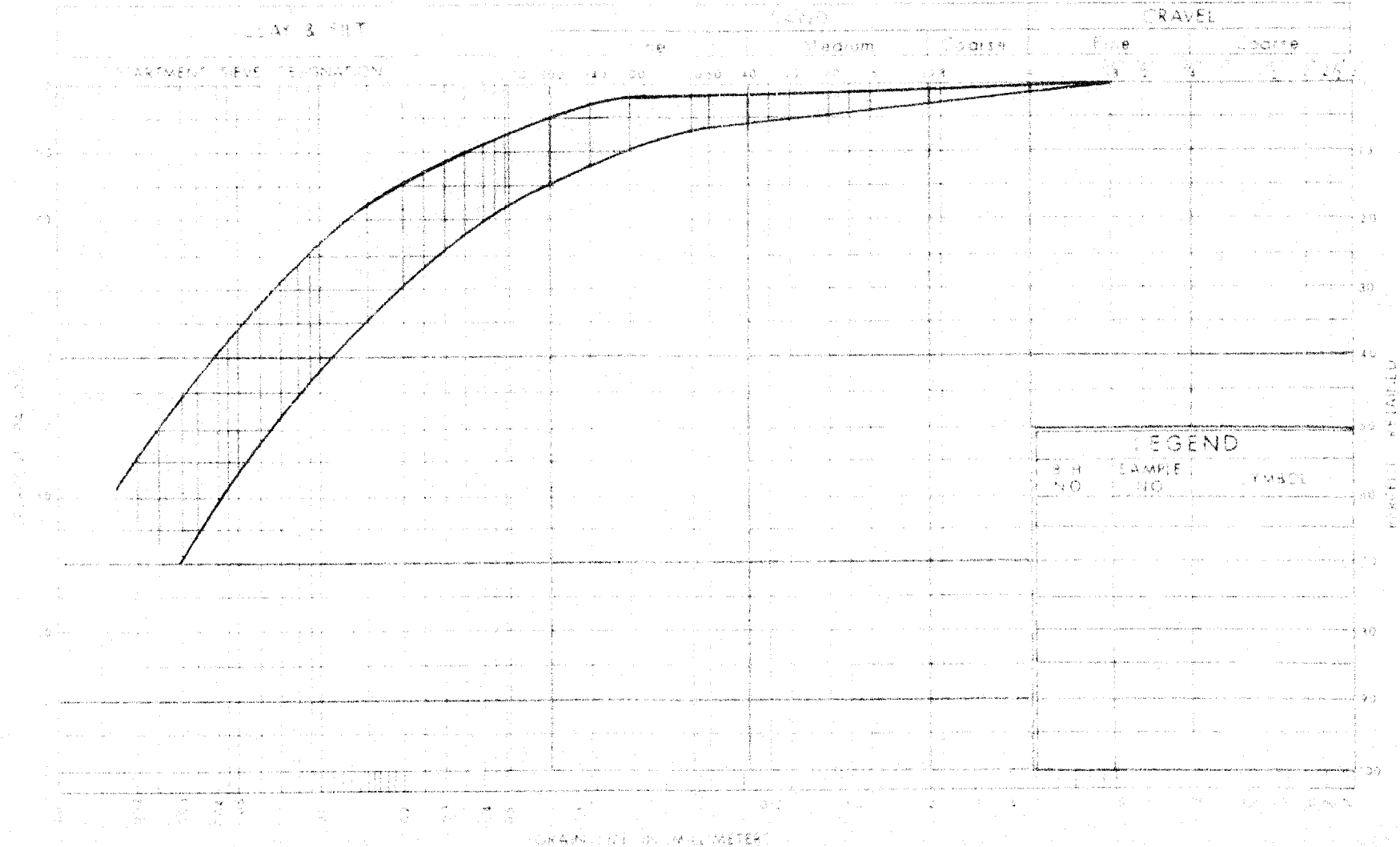
WP No. 240-66-4

JOB No. 68-F-74

FIG NO 2

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION CLAYEY SILT TO CLAY

DEPARTMENT OF HIGHWAYS
MATERIALS AND
TESTING
DIVISION

W.P. No. 240-66-4

JOB NO. 68-F-74

FIG. NO. 3

VOID RATIO - PRESSURE CURVES

JOB NO. 68-F-74

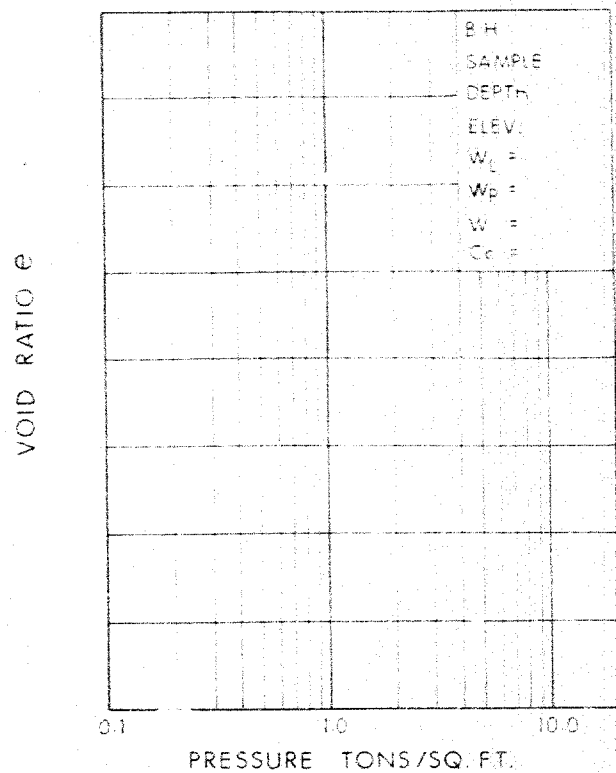
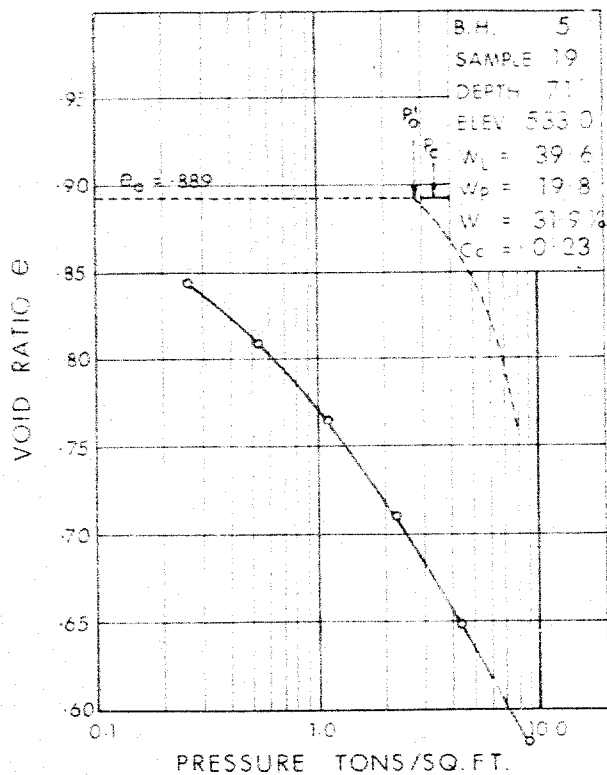
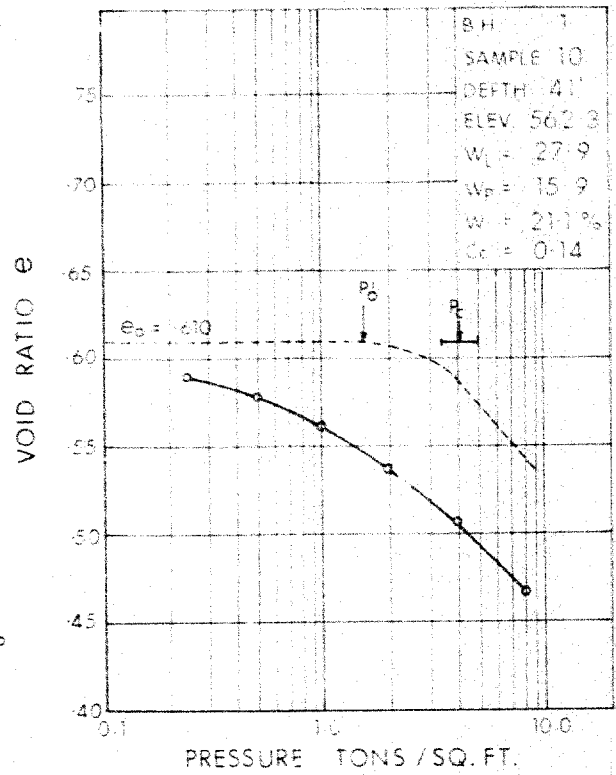
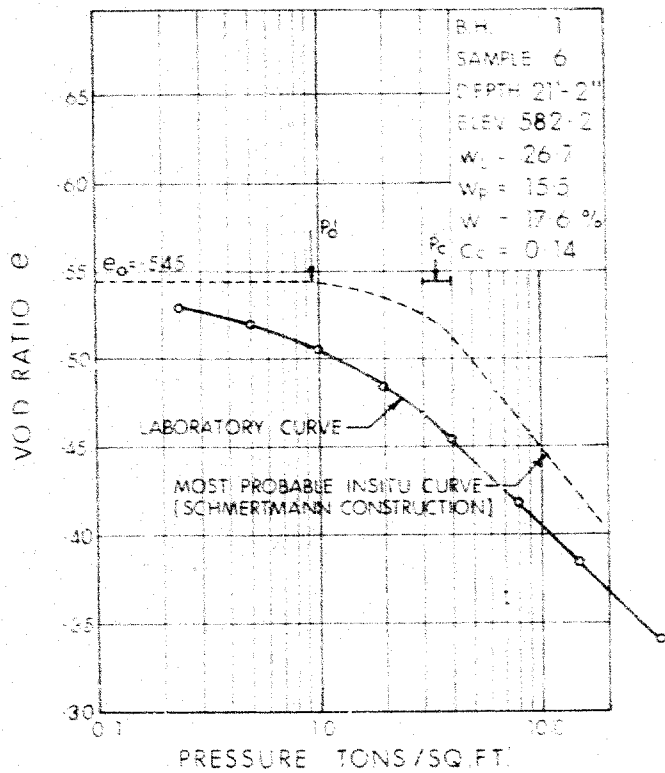
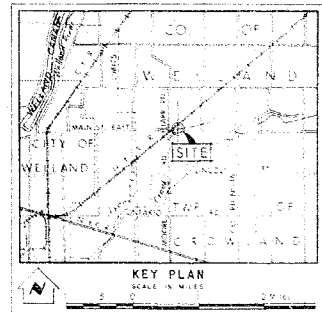
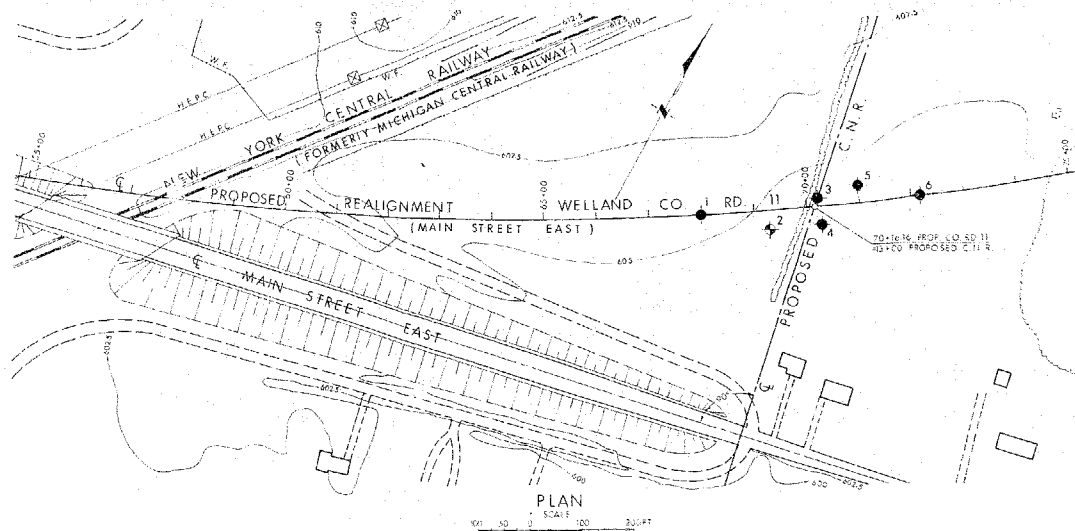
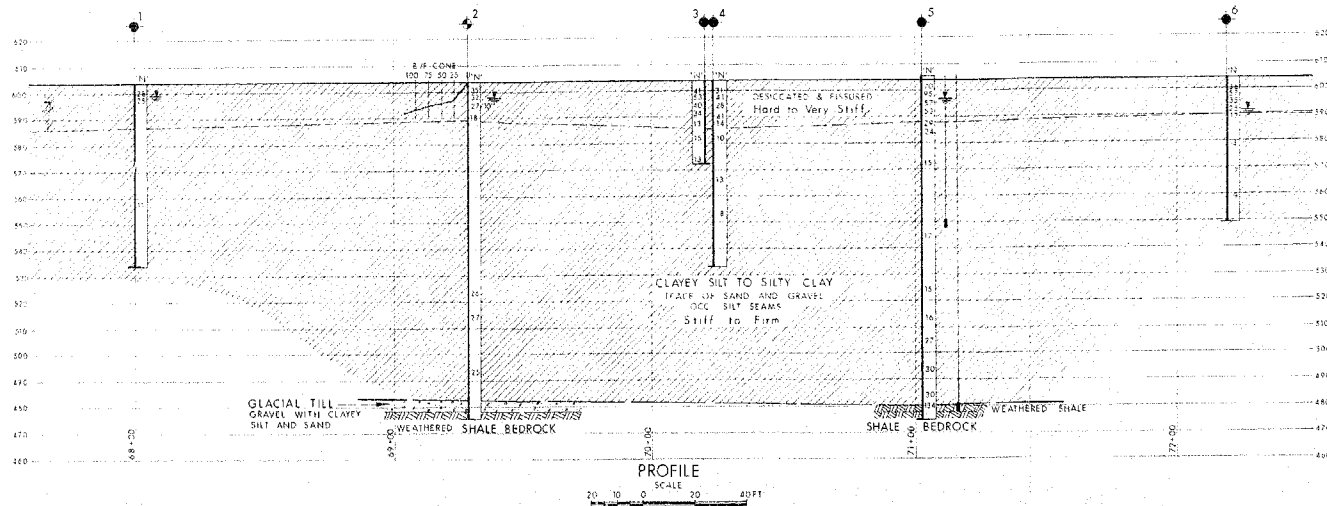


FIG. 4



LEGEND			
●	Bore Hole		
⊕	Cone Penetration Test		
⊗	Bore & Cone Penetration Test		
—	Water Levels established during field investigation		
I	Piezometer		
NO.	ELEVATION	STATION	OFFSET
1	603.3	68+00	0
2	603.0	69+20	20 FT
3	601.8	70+20	14 FT
4	603.3	70+24	35 FT
5	604.0	71+23	28 FT
6	603.5	72+00	0

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.



DATE	BY	REVISION

DEPARTMENT OF HIGHWAY - ONTARIO
MATERIALS & TESTING DIVISION

CANADIAN NATIONAL RAILWAYS OVERHEAD

PROPOSED CO. RD. 11 (MAIN ST. EAST) DIST. NO. 4
CO. WELLAND
TWP. CROWLAND LOT 10 CON. 10

BORE HOLE LOCATIONS & SOIL STRATA

SUBNO. C. M. CHECKED WP NO. 240 5-6-74
DRAWN G. P. CHECKED J. J. JOHNSON 68-F-74A
DATE Nov. 8, 1968 SITE NO. 68-F-74A
APPROVED 68-F-74A DIST. NO. 4

PRINT RECORD	NO.	DATE

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

SS	SPLIT SPOON	TW	THINWALL OPEN
WS	WASHED SAMPLE	TP	THINWALL PISTON
SB	SCRAPER BUCKET SAMPLE	OS	OSTERBERG SAMPLE
AS	AUGER SAMPLE	FS	FOIL SAMPLE
CS	CHUNK SAMPLE	RC	ROCK CORE
ST	SLOTTED TUBE SAMPLE		
	PH	SAMPLE ADVANCED HYDRAULICALLY	
	PM	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Qu	UNCONFINED COMPRESSION	LV	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	FV	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

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

MEMORANDUM

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

FROM: C.S. Grebski,
Bridge Office

ATTENTION:

DATE: November 24, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT: C.N.R. Overhead Bridge
East Main Street Relocation
W.P. 240-66-04, Site 34-235
District No. 4

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

Attach.

c.c. Foundation Section


C.S. Grebski,
Bridge Design Engineer

Mr. C. B. Ortbai,
Bridge Design Engineer,
Bridge Office,
Admin. Bldg.

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

December 3, 1967

C.R.R. Overhead Bridge
East Main Street Relocation
Site 34-238
W.P. 240-66-84, W.J. 66-2-74

We have reviewed the final bridge drawings and submit the following comments:

The final drawings indicate piled foundations for abutment support instead of spread footings on compacted granular fills. We believe this scheme is more economical than constructing fills consisting of granular material; if this is so, the timber piles should not be driven any deeper than elev. 350.0. A note to this effect should be made on the Contract drawings.

MD/MSF

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

cc: Messrs. S. McConble
W. B. Melinsky
Foundations Files
Gen. Files

ags

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

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

July 8, 1969

C.N.R. Overhead Bridge --
East Main Street Relocation
W.P. 240-66-04, Site 34-235
Co. Rd. 11, District No. 4

(Foundation Report W.J. 68-P-74)

We have reviewed the Preliminary Bridge Drawing D-6653-P for the above structure and have the following comments to offer:

1. We have recommended in the Foundation Report, that the pier footings should be located as high as possible in the desiccated crust. Footings located at or above elevation 598 can be designed for a safe allowable bearing pressure of 2.5 TSP. On Section A-A of Dwg. D-6653-P, the pier footings are shown to be located at elevation 596. We would, therefore, like to draw your attention to the aforementioned.
2. The existing ground lines shown in "Section A-A" and "South Elevation" of Dwg. D-6653-P, do not appear to have the same elevations.
3. According to our present information, the approach fills will be surcharged prior to the construction of the structure footings. Also, this Section will instrument these fills. Therefore, a note should be added to this effect on the Contract Documents.

RD/WdeP

cc: Messrs. S. McCombie
W. S. Melingshyn
R. Greenland

Foundations Files ✓
Gen. Files

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



GIBB, ALBERY PULLERITS & DICKSON

CONSULTING PROFESSIONAL ENGINEERS

PARTNERS:
ALBERY PULLERITS DICKSON & ASSOC., LTD.
SIR ALEXANDER GIBB & PARTNERS

29 GERRAIS DRIVE
DON MILLS, ONTARIO, CANADA
TELEPHONE 429-2920

May 28, 1969

Ref: #343-201

Mr. A.G. Stermac, P.Eng.,
Principal Foundation Engineer,
Department of Highways Ontario,
Laboratory Building,
Downsview, Ontario.

Dear Sir:

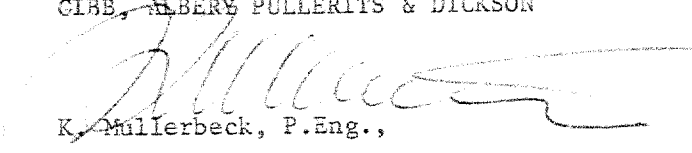
WELLAND COUNTY ROAD #11
OVERPASS OVER N.Y.C. RR

In response to your phone call today, we take pleasure in forwarding a copy of part of the General Arrangement Drawing of the above overpass.

Upon the xerox, we have marked in red pencil a few spot elevations as designed, and in green pencil corresponding spot elevations as we have measured on May 26, 1969. Our measurements are based on B.M 613.81 - top of painted nut mounted on bolt of hydro tower 227 feet left of Station 60+97, East Main Extension, as established by the Department.

Yours very truly,

GIBB, ALBERY PULLERITS & DICKSON


K. Mullerbeck, P.Eng.,

Encl:

KM/hstc

W.P. 240-66-4

W.D. 68-F-74

NOTE:

THE STRUCTURE WILL BE DESIGNED AS A
THREE SPAN CONTINUOUS BRIDGE FOUNDED
ON SPREAD FOOTINGS

ABUTMENTS ON SPREAD FOOTINGS ON FILL
(GRANULAR 'B'). FILL WILL BE SURCHARGED
(15-20 FT) AND SURCHARGE LEFT FOR
APPROX. 6 MONTHS.

INSTRUMENTATION FOR

- (a) MEASUREMENT OF FILL SETTLEMENT
- (b) - " - " PIER "

ONCE STRUCTURE IS BUILT SETTLEMENTS
(DIFFERENTIAL) IN EXCESS OF 1 INCH
SHALL BE REPORTED TO THE BRIDGE
OFFICE.

CONCLUSIONS REACHED AT MEETING, MAY 27, 1969

PRESENT MESSRS K. PULLERITZ
K. KULLERBACH
B. RICHARDSON
A.G. STERMAC

640.22 - ELEVATION AS DESIGNED
640.08 - AS MEASURED MAY 26/69

640.22 - ELEVATION AS DESIGNED
640.08 - AS MEASURED MAY 26/69

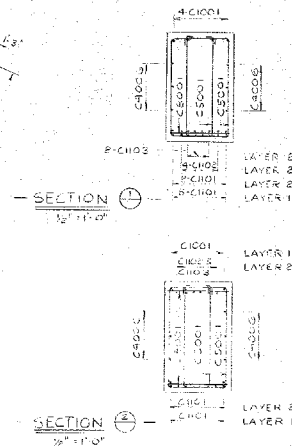
— 700.00 P102 635.54
635.59

Page 10

68537
68537
68537

78. 2004 6.25.29
63512

STILL UNDER REVIEW
NOISE AS SUBJECT CA 100



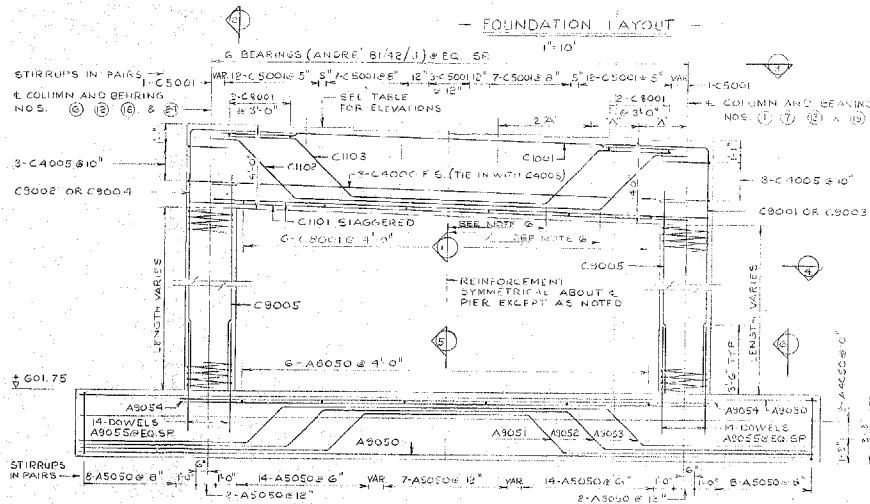
BEARING SEAT ELEVATIONS							
NO.	ELEV.	NO.	ELEV.	NO.	ELEV.	NO.	ELEV.
1	632.31	7	633.37	13	631.37	19	632.06
2	632.43	8	633.49	14	632.10	20	633.12
3	632.56	9	633.61	15	632.22	21	633.32
4	632.68	10	633.74	16	632.35	22	633.45
5	632.81	11	633.86	17	632.48	23	633.58
6	632.93	12	633.95	18	632.61	24	633.71

ABBREVIATIONS

E F	EACH FACE
EQ. SP	EQUAL SPACES
FR F	FR. FACE
NR F	NR. FACE
*	INDICATES INTERMEDIATE BAR LOCATIONS

NOTES

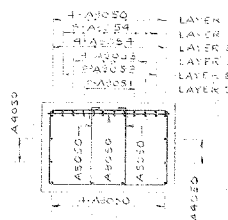
1. FOR GENERAL NOTES SEE DWG NO. BRIDGE-1
2. TOLERANCE FOR PARALLEL TO L
3. PROPOSED C.N.R.
4. (1) DENOTES BRIDGE PIER BEARING AND REINFORCEMENT TO BE AT EQUAL SPACES EXCEPT AS NOTED.
5. PILES SHALL BE 30 FT LONG-SIZE 12. 3.0 IN. DIA. 100% STEEL
6. REQUIRE TWENTY-ONE WITH COAL TAIL CEMENTS TO 8 LB. RETENTION. THE ELEVATION SHALL BE 585.0 AND 581.0 ELEVATION AS PER ADJUSTMENT DRAWING.
7. SUB. REINFORC. CARRYING LOAD 120 TONS
8. 2.0 PERCENT MINIMUM STEEL
9. ALLOW 4. UNLESS OTHERWISE NOTED



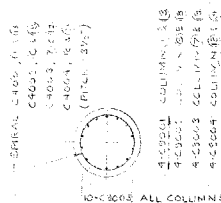
- TYPICAL PIER ELEVATION -
- 4 THUS -
3/8" = 1'-0"



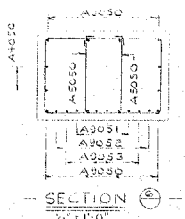
--- PART PLAN (3)



SECTION 5

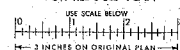


- SECTION 4 -



SECTION 6

FOR REDUCED PLAN



REVISIONS			
	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

KING'S HIGHWAY No. E. MAIN ST. (CO. RD 11) DIST. No. 4
CO. OF WELLAND
TWP. OF CROWLAND LOT CON.

FOUNDATION LAYOUT & PIER DETAILS

APPROVED _____	SITE No.	W.P. No.
	34-235	240-66-4

DESIGN	S. R. S.	CHECK	<i>[Signature]</i>	CONTRACT			
DRAWING	S. R. S.	CHECK	<i>[Signature]</i>	No.			

DATE	Nov 69	LOADING	1520-19	DRAWING No.	D-6653-2
------	--------	---------	---------	-------------	----------