

41-61-1

Mr. A. H. Toye,
 Bridge Engineer.
 Materials & Research Division,
 (Foundation Section)

January 18, 1962.

FOUNDATION INVESTIGATION REPORT

By: H.C. Acres & Co., Ltd.

Attention: Mr. A. McCosbie.

Re: Proposed Revisions to Crossing of
 Niagara Street over the Queen Elizabeth Way,
 City of St. Catharines, District No. 4,
 W.P. 41-61.

Attached, we are sending you the above-mentioned report submitted by the Consultant, H. C. Acres & Company, Ltd., Niagara Falls, Ont.

We have reviewed the report and have found the factual information well presented. We are also in agreement with the conclusions and recommendations contained therein.

We believe the information provided by the Consultant will be adequate for your future design work. However, should there be any questions, or additional data required, please feel free to call on our Office.

AGS/MSF
 Attach.

A. G. Sternac
 A. G. Sternac,
 PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. H. Toye (2)
 H. A. Tregaskes
 H. D. McMillan
 I. C. Campbell
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 Foundations Office
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ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION INVESTIGATION

PROPOSED REVISIONS TO CROSSING OF
NIAGARA STREET OVER
THE QUEEN ELIZABETH WAY
CITY OF ST. CATHARINES, DISTRICT NO. 4
W.P. 41-61

H.G. ACRES & COMPANY LIMITED
Consulting Engineers
Niagara Falls, Canada

November 1961

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

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FOUNDATION INVESTIGATION

PROPOSED REVISIONS TO CROSSING OF
NIAGARA STREET OVER
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W.P. 41-61

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THE QUEEN ELIZABETH WAY
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Introduction

At the request of the Ontario Department of Highways, soil explorations were carried out by H.G. Acres & Company Limited at the above site. Proposed modifications to the highway facilities involve the construction of two service roads parallel to the Queen Elizabeth Way. The centrelines of the service roads will be approximately 80 feet from the centreline of the Queen Elizabeth. These service roads will be excavated through the approach embankments of the existing bridge and two new bridges, each with a clear span of about 40 feet will be constructed to carry Niagara Street traffic and the single track of the Niagara - St. Catharines - Toronto Railway over the service roads. In addition, widening of the existing bridge and embankments will be undertaken in order to accomodate one

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or two additional lanes of traffic on Niagara Street. A plan of the site showing the principal features of the existing structures and the proposed revisions is shown on Plate 1.

The F.E. Johnston Drilling Company Limited performed the drilling and soil sampling operations under the supervision of Mr. H.W. Ryder of H.G. Acres & Company Limited. The field work started on October 17, 1961 and was completed on November 3, 1961. Soil testing in the laboratory of H.G. Acres & Company Limited was completed in November 1961.

The results of the field and laboratory work are presented in this report, together with an interpretation of the data obtained, and recommendations concerning foundation conditions and design criteria.

Geology of the Site

The site of the proposed structures is located in the narrow strip of low land extending from Hamilton to the Niagara River and lying between the Niagara Escarpment and Lake Ontario. The area is characterized by a broad flat till plain covered in places by sand and gravel beach deposits and overlying red Queenston shales of Upper Ordovician age.

During the Pleistocene epoch this area was covered by glaciers several times, resulting in the deposition of a compact till complex composed primarily of clay and silt

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with some sand and gravel. Lenses and beds of stratified waterlain clay, silt, sand, and gravel are commonly encountered. In addition, the stiffness of the present surface zones of the till complex has generally been increased by weathering and desiccation.

Exploratory Work

The modified wash boring method of drilling was employed and NX or BX casing was used to advance the holes. In holes 955-1 and 955-2, two-inch diameter thin-walled Shelby tube samples were taken at five-foot intervals wherever possible, and in holes 955-3 and 955-4 three-inch diameter fixed piston samples were recovered. With the exception of hole 955-1, vane tests were performed 18 inches below the lower elevations of the tube samples. Where the soil was too stiff for vane tests, standard penetration tests were carried out and the split-spoon samples retained for inspection and identification.

The location of the four holes are shown on Plate 1 and the program of work is outlined in Appendix A.

Site Conditions and Soil Properties

The elevation of the Queen Elizabeth Way at the site is approximately 332 feet. The original ground elevation varies from approximately 329 feet to 339 feet. The existing embankment stands approximately 18 feet above the

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road level, the side slopes being approximately 2.5 to 1 and 4 to 1 on the eastern and western sides respectively.

The soils which were encountered in the exploratory holes are described in the attached drilling reports, Plates 2 to 5 inclusive. Simplified sections of the bore holes are shown on Plate 1.

(a) - Embankment Fill - The fill material probably consists of gravel, sand and silt. However, the exploratory holes were located at the toes of the embankment and very little fill material was encountered.

(b) - Silty Clay Crust - Directly below the embankment fill lies a stiff brown silty clay containing varying proportions of gravel and sand. The natural undrained shear strength of this layer, from the results of the field vane tests and laboratory unconfined compression tests shown on Plates 6 to 9, generally exceeds 1500 psf. The thickness of this layer varies, as interpreted from the shear strengths, from five feet in hole 955-4 to about 20 feet in hole 955-2. The transition from the stiff crust to the softer underlying soil is fairly gradual.

(c) - Silty Clay - The softer soil underlying the silty clay crust is also a silty clay and probably part of the same deposition. This silty clay is brownish grey in colour and contains some sand and gravel. The natural undrained shear strength of this layer varies little with

- 5 -

depth down to approximately elevation 275 feet, the average shear strength being approximately 1300 psf.

The liquid limit of this soil ranges from 33 to 38 per cent with an average of 35 per cent. The plastic limit ranges from 17 to 20 per cent with an average of 19 per cent. The results of the six Atterburg limits tests on samples from holes 955-1, 955-2 and 955-4 plotted on the Casagrande's plasticity chart are shown on Plate 10. The liquidity index ranges from 0.5 to 0.57 for this layer.

The grain size distribution curves for two samples at depths of 15 and 35 feet in hole 955-3 are shown on Plates 11 and 12 respectively. These two curves are almost identical. The activity of this soil, defined as ratio of the plasticity index to the percentage of clay, averages 0.32.

Consolidation tests were performed on samples recovered at elevations 318.0 feet and 297.0 feet in hole 955-3. The results of these two tests presented on Plates 13 and 14 respectively, show that this silty clay deposit is overconsolidated, the preconsolidation pressure for both samples being approximately 7000 psf. The compression index, C_c is approximately 0.42 for the sample at elevation 318.0 feet and 0.27 for the sample at elevation 297.0 feet.

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(d) - Clay Till - A heterogeneous mixture of sand, gravel, silt and clay was encountered immediately under the silty clay stratum in holes 955-1 and 955-2, the two deeper holes. The silty clay matrix of this material appears to be very similar to the silty clay overlying it. The natural undrained shear strengths, as determined by field vane tests, exceed 2000 psf. However, these results are unreliable because of the presence of a large proportion of coarse grained particles in the soil.

All the laboratory tests are summarized in Appendix B and the field vane tests are tabulated in Appendix C.

Performance of Existing Structures

The existing reinforced concrete bridge has two spans of approximately 50 feet. It was built in 1939 and carries two lanes of traffic and a single track of the Niagara - St. Catharines - Toronto Railway located centrally between the two lanes. The bridge abutments support the bridge deck and retain the approach embankments. The wing walls are in the same plane as the abutment walls but are structurally separated from them by construction joints. These wing walls extend to the toes of the embankments.

An inspection of the structure shows that it is free from serious cracking. No damage attributable to differential settlement was observed.

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Design Considerations

As described in the introduction to this report, the proposed modifications at this site will involve the excavation of a 60-foot length of the existing embankment at either end of the existing bridge to accomodate service roads, and the widening of the existing bridge and approach embankments to accomodate one or two additional lanes of traffic. The widening of the existing bridge could be effected by demolishing the wing walls and constructing extensions to the abutment walls.

The considerations governing the selection of allowable pressures for the foundation soils beneath the abutments and wing walls of the new bridges and the extensions to the existing structure will be reviewed in the following sections of this report.

(a) - Bearing Capacity - A method of supporting the structures would be with spread footings. The ultimate bearing capacity of a shallow spread footing is given by the formula:

$$q_{ult} = 5 \left(1 + 0.2 \frac{D}{B} \right) \left(1 + 0.2 \frac{B}{L} \right) S_u \dots (1)$$

Where: q_{ult} denotes the ultimate bearing capacity of the foundation soil.

D denotes the depth of the base of the footing below the surface of the overburden.

B denotes the width of the footing.

L denotes the length of the footing.

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Su denotes the natural undrained shear strength of the soil beneath the foundation.

The base of the footings may be founded in stiff silty clay crust but should be given at least four feet of soil cover. Assuming the base of the footings at elevation 330 feet, the average shear strength of the silty clay within the zone of influence of the footing may be taken to be 1500 psf. Since the shear strength decreases with depth to approximately 1300 psf at about elevation 310 feet the allowable bearing pressure for deeper footings would also decrease apart from its dependence on D/B and B/L (see equation (1)). However, for spread footings with a width of 14 feet or less founded above elevation 320 feet, the allowable bearing pressure would be 2800 psf considering a factor of safety of 3 against a bearing capacity failure.

In the case of eccentric loading, the width of the footing should be increased to limit the maximum contact pressure at the base of the footing to the allowable bearing pressure given above.

(b) - Settlement - Because of the limited amount of relative settlement which can be tolerated between the extensions and the existing bridge, settlement considerations may govern the foundation design, despite the low compressibility of the foundation soils.

The results of the consolidation tests on samples from hole 955-3 at depths of 15 and 35 feet are shown on

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Plates 13 and 14. Both these tests indicate that the preconsolidation pressure of the silty clay is approximately 7000 psf. At depths less than 15 feet in the weathered and desiccated silty clay crust the preconsolidation pressures are probably even higher. Therefore, with the allowable bearing pressure limited to 2800 psf on the basis of bearing capacity considerations, all the stress changes imposed by the new footings will act within the preconsolidation portion of the compression curve. Calculations of settlement in this range are unreliable. The average compressibility (M_v) from the test results for the pressure range of 1000 psf to 4000 psf is 5.4×10^{-3} feet ²/kip. However, this is a maximum value because these samples, like all samples, were subject to some disturbance and disturbance tends to increase compressibility especially in this preconsolidated portion of the compression curve. Nevertheless, the average compressibility (M_v) from the test results can be used in the settlement calculations to obtain an upper limit for the magnitude of the settlements.

(i) - Settlement of Footings Supporting Service Road Bridges - The locations of the footings supporting the service road bridges are shown on Plate 1. For purposes of discussion these four footings have been designated A, B, C, and D, as shown on Plate 1. All of these footings are located beneath the existing embankment. However, for

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practical purpose no preloading effect can be attributed to the presence of the embankment which will be removed, because the pressure due to the embankment is much less than the preconsolidation pressure of the foundation soils.

The footing loads will consist of the live and dead loads due to the bridge, which for a 40-foot span will be about 10 kips per linear foot. In addition, footings A and D will be subjected to some loading imposed by the approach embankments. Each of these footings will be about 100 feet long. The widths of the footings can only be established by detailed design including considerations such as eccentric loading which are beyond the scope of this report. However, the settlements will vary with the footing width and, therefore, a procedure will be outlined by which the designer can estimate the probable settlement for varying widths of footing and varying average base pressures.

In computing the settlement relationship which is given below, a somewhat arbitrary but still conservative estimate of the actual compressibility of the foundation soils has been made by taking 75 per cent of the average compressibility as measured in the laboratory tests.

$$\text{SETTLEMENT} = \frac{\text{AVERAGE BASE PRESSURE}}{\text{(psf)}} \times \frac{\text{WIDTH OF FOOTING}}{\text{(feet)}} \times (6 \times 10^{-5}) \dots (2)$$

(inches)

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This relationship is valid for any footing on the order of 100 feet long with widths between 4 and 10 feet.

For the loading of 10 kips per linear foot, the settlement of a footing would be only about 0.6 inches. Therefore, no settlement problems are anticipated for the bridges over the service roads.

(ii) - Settlement of Footings Supporting Widening of the Existing Bridge - The locations of the footings supporting the widening of the existing bridge are shown on Plate 1. There will be six footings in all if centre piers are used. Assuming there will be centre piers, these footings would be subject to the maximum loads, which for a total span of 120 feet would be approximately 26 kips per linear foot. The abutment footings would be subject to approximately 13 kips per linear foot.

As in the case of the service road footings the final dimensions will depend on the details of the design. However, the relationship given below will allow the designer to estimate the probable settlement for varying widths of footing and varying average base pressures. The relationship is based on the same arbitrary estimates as the relationship for the service road footings.

$$\begin{array}{ccccc} \text{SETTLEMENT} & \times & \text{AVERAGE BASE} & \times & \text{WIDTH OF} & \times & (4.4 \times 10^{-5}) & \dots (3) \\ \text{(inches)} & & \text{PRESSURE} & & \text{FOOTING} & & & \\ & & \text{(psf)} & & \text{(feet)} & & & \end{array}$$

This relationship is valid for any footing in the order of 22 feet in length and between 4 and 14 feet in width.

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If the settlements are computed using the approximate loadings suggested in this section, the settlements of the central piers are 1.2 inches and the abutments 0.6 inches. Therefore, the differential settlement between the central pier and the abutments of the extension to the existing bridge could be as much as 0.6 inches. This must be considered in the details of the design as the tolerable values of differential settlement depend almost entirely on the nature of the structure. With the exception of minor cracking, the existing structure is free from damage due to differential settlements.

Despite the magnitude of these possible settlements a still more serious settlement problem exists. The relative settlement between the new extension and the existing bridge will be in the same order of magnitude as the total settlement of the extension. For this reason care should be exercised in design to ensure that the structure is so articulated that these relative settlements can take place without impairing the performance of the bridge.

Conclusions and Recommendations

(a) - The borings show that the overburden at this site extends to at least 76 feet below the surface of the Queen Elizabeth Way. The overburden is composed predominantly of a silty clay with varying contents of sands

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and gravels. The thickness of the stiff weathered crust of the silty clay layer varies from 5 to 20 feet. Below this crust the silty clay is fairly homogeneous and contains some sand and gravel. The gravel content of the soil increases between elevations 275 and 265 feet and the soil becomes very compact and stiff.

(b) - The properties of the foundation soils are summarized on Plates 6 to 12.

(c) - The new service road bridges and the extensions to the existing bridge may be supported on spread footings founded at least four feet below the ground surface.

(d) - An allowable bearing pressure of 2800 psf may be used for spread footings with a width of 14 feet or less founded above elevation 320 feet. For some footing dimensions within the above limits it may be possible to increase this bearing pressure slightly. For this purpose, the allowable bearing pressure may be computed from the equation on Page 7 of this report, using a value for the natural undrained shear strength, S_u , of 1300 psf and a factor of safety of 3.

(e) - The silty clay on which the spread footings for the bridge foundations will be founded is overconsolidated and has low compressibility in the range of pressures to which it will be subjected by these footings.

The settlements of the foundations for the new service road bridges will probably be less than one inch and no problems resulting from settlements are anticipated.

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However, the settlements of the foundations for the extensions of the existing bridge do present problems. The central pier footings will probably settle about twice as much as the abutment footings resulting in differential settlements in the order of 0.6 inches. This must be considered in the details of the design as the tolerable values of differential settlement depend almost entirely on the nature of the structure. With the exception of minor cracking, the existing structure is free from damage due to differential settlements. The relative settlement between the new extension and the existing bridge will be in the same order of magnitude as the total settlement of the extension. For this reason care should be exercised in design to ensure that the structure is so articulated that these relative settlements can take place without impairing the performance of the bridge.

APPENDIX AProgram of Work

October 17, 1961	Diamond drill arrived at the site. Hole 955-1 commenced.
October 18, 1961	Hole 955-1 was completed.
October 26, 1961	Hole 955-2 was commenced.
October 30, 1961	Hole 955-2 was completed.
October 31, 1961	Hole 955-3 was commenced.
November 1, 1961	Hole 955-3 was completed.
November 2, 1961	Hole 955-4 was commenced.
November 3, 1961	Hole 955-4 was completed.

Summary of Time

<u>Work Type</u>	<u>Number of Holes</u>	<u>Total Length (feet)</u>	<u>Total Time (hours)</u>
Modified Wash Boring	4	244.8	74.5

APPENDIX B

Summary of Laboratory Test Results

Hole No.	Sample No.	Elevation (Feet)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Liquidity Index	su _n (Psf)	ef (%)	su _r (Psf)	Sensitivity
955-1	5	310.0	26.0	-	-	-	1780	4	-	-
	6	305.0	30.3	38.7	20.4	0.54	800	8	270	3.0
955-3	34	317.7	27.3	34.8	19.8	0.50	1310	9	-	-
	38	297.7	26.3	32.7	18.0	0.56	1830	2	-	-
955-4	27	323.0	25.7	37.7	19.3	0.40	1870	7	-	-
	29	312.0	26.0	33.4	17.6	0.53	1340	8	-	-
	31	301.0	27.6	35.3	17.2	0.57	1690	5	-	-

Note: ef - failure strain for undisturbed sample

su_n - Undrained shear strength from unconfined compression test on undisturbed samples

su_r - Undrained shear strength from unconfined compression tests on remoulded samples

APPENDIX CResults of In-Situ Vane Tests

Hole No.	Elevation (Feet)	Undrained Shear Strength (Psf)		Sensitivity
		Undisturbed	Remoulded	
955-2	326.5	2520	1070	2.4
	321.5	2930	1730	1.7
	316.5	1610	760	2.1
	311.5	1450	540	2.7
	306.5	1040	410	2.5
	301.5	1230	380	3.2
	296.5	1640	760	2.2
	291.5	1640	570	2.8
	286.5	1640	760	2.2
	281.5	1640	470	3.5
	276.5	1060	250	4.2
	271.5	1450	280	5.2
	266.5	2330	1900	1.2
	256.5	4200+	3120	1.4
955-3	324.7	2330	1040	2.2
	319.2	1450	690	2.1
	314.7	1290	690	1.9
	309.7	1290	610	2.1
	304.7	1290	570	2.3
	299.7	1290	440	2.9
	294.7	1230	500	2.6
955-4	325.0	1770	650	2.6
	320.0	1450	600	2.4
	315.0	1450	660	2.2
	310.0	1290	470	2.7
	305.0	1230	530	2.3
	300.0	1230	470	2.6

APPENDIX D

List of Plates

- 1 - Exploratory holes, plan and section
- 2 - Drilling report, Hole 955-1
- 3 - Drilling report, Hole 955-2
- 4 - Drilling report, Hole 955-3
- 5 - Drilling report, Hole 955-4
- 6 - Summary of drilling and test results, Hole 955-1
- 7 - Summary of drilling and test results, Hole 955-2
- 8 - Summary of drilling and test results, Hole 955-3
- 9 - Summary of drilling and test results, Hole 955-4
- 10 - Summary of drilling and test results, Plasticity Chart
- 11 - Summary of drilling and test results, Grain Size Curve
- 12 - Summary of drilling and test results, Grain Size Curve
- 13 - Summary of drilling and test results, Consolidation Test
- 14 - Summary of drilling and test results, Consolidation Test

DRILLING REPORT

CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS JOB No. 955
 PROJECT: W.P. 41-61 HOLE No. 955-1
 SITE: QEW and Niagara Street Interchange SHEET No. 1 OF 3
 CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 10:00 A.M. October 17 1961
 FINISHED 5:00 P.M. October 18 1961
 METHOD OF DRILLING: SOIL Modified wash boring CASING DIAM. BX
 ROCK CORE DIAM.
 LOCATION: LATITUDE Sta 127 + 24 ELEVATIONS: DATUM G.S.C.
 DEPARTURE 102.6 Right Centreline DRILL PLATFORM -
 BEARING QEW GROUND SURFACE 340.2
 INITIAL DIP 90 degrees ROCK SURFACE -
 OTHER DIPS BOTTOM OF HOLE 263.7
 WATER TABLE 7-

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST*
			NO	TYPE*	SIZE	DEPTH	RET'D	
0.0	Asphalt				In.	Ft.	In.	Blows
0.5	Silty Sand & Gravel	Backfill material, reddish brown						
5.5	Sand	Brownish medium size dense	1	AQ	2	5.0 5.5 6.0 6.5	18	4 7 11
7.8	Gravel Sand Silt & Clay	Brownish grey with some mottled patches and gravel size particles, very stiff	2	AQ	2	7.5 8.0 8.5 9.0	18	4 6 9
			3	BO	2	9.5 11.0	18	Pushed 1300 lb
			4	BO	2	12.5 14.0	17	Pushed 1300 lb
			5	BO	2	14.0	Lost	Pushed 1300 lb

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLEIOFILM BAG
 Z — DISCARDED

INSPECTOR: H.W. Ryder
 LOGGED BY: H.W. Ryder

APPROVED

DATE

November 1961

DRILLING REPORT

CLIENT ONTARIO DEPARTMENT OF HIGHWAYS

JOB No. 955

PROJECT W.P. 41-61

HOLE No. 955-1

SITE QEW and Niagara Street Interchange

SHEET No. 2 OF 3

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST *
			NO	TYPE	SIZE In.	DEPTH Ft.	RET. D. In.	
15.0	Silty Clay	Brownish grey stiff	6	AQ	2	15.0 16.5	14	
			7	BO	2	17.5 17.9 18.0 18.5 19.0		Pushed 1300 lb 2 11 14
			8	BO	2	20.0 21.5	18	Pushed 1300 lb
			9	BO	2	25.0 26.5	18	Pushed 1300 lb
		After 30 feet depth, it became progressively easier to push the sampler	10	BO	2	30.0 31.5	18	Pushed 1300 lb
			11	BO	2	35.0 36.5	18	Pushed 1300 lb
			12	BO	2	40.0 41.5	18	Pushed 1000 lb
			13	BO	2	45.0 46.5	Lost	Pushed 1000 lb
			14	AQ	2	46.0 47.0	12	
			15	BO	2	50.0 51.5	18	Pushed 1000 lb
			16	BO	2	55.0 56.5	16	Pushed 1000 lb
			17	BO	2	60.0 61.5	17	
		A 6-inch resistive layer at 65.5 feet	18	BO	2	65.0 65.5 66.0 66.5	14	Pushed 1300 lb 9 14
			19	BO	2	70.0 70.5 71.0 71.5	12	 6 22 21

DRILLING REPORT

CLIENT ONTARIO DEPARTMENT OF HIGHWAYS
 PROJECT W.P. 41-61
 SITE QEW and Niagara Street Interchange

JOB No. 955
 HOLE No. 955-1
 SHEET No. 3 OF 3

DEPTH Feet	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST #
			NO.	TYPE	SIZE In.	DEPTH Ft.	RET'D In.	
75.0	Silty Clay	There was some very coarse washed material on top of sample #19. The material appeared same as before with a larger percentage of coarse gravel particles	20	AQ	2	75.0 75.5 76.0 76.5	18	13 14 21
76.5		End of Hole						
		<u>*Penetration Test</u> This is the number of blows of a 140-pound weight falling 30 inches required to advance the tube sampler or split-spoon to the depth indicated						

DRILLING REPORT

CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS
 PROJECT: W.P. 41-61
 SITE: QEW and Niagara Street Interchange
 CONTRACTOR: F.E. Johnston Drilling Company Limited
 METHOD OF DRILLING: SOIL Modified wash boring
 LOCATION: LATITUDE Sta 126 + 76
 DEPARTURE 102 Left Centreline QEW
 BEARING
 INITIAL DIP 90 degrees
 OTHER DIPS

STARTED 4:00 P.M. October 25 1961
 FINISHED 11:45 A.M. October 30 1961

JOB No. 955
 HOLE No. 955-2
 SHEET No. 1 OF 4

CASING DIAM. NX
 BX
 CORE DIAM.

DATUM G.S.C.
 DRILL PLATFORM -
 GROUND SURFACE 339.5
 ROCK SURFACE -
 BOTTOM OF HOLE 247.2
 WATER TABLE

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST ¹ Blows
			NO.	TYPE*	SIZE In.	DEPTH Ft.	RET'D In.	
0.0	Humus	Black organic						
0.5	Gravel Sand Silt & Clay	Brownish, dry, very stiff and weathered	1	AQ	2	5.0 5.4	5	86
			2	BO	2	10.0 10.5 11.0 11.5	10	Pushed 1300 lb 9 10
					Vane Test	13.0		
15.0	Silty Clay	Brownish grey, stiff showing some brown and grey mottling with occasional gravel size particles	3	BO	2	15.0 15.5 16.0 16.5	18	Pushed Hard 1300 lb
					Vane Test	18.0		
			4	BO	2	20.0 20.5 21.0 21.5	16	Pushed Hard 1300 lb
					Vane Test	23.0		

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
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 Q — GLASS JAR

R — CLOTH BAG
 S — PLYFILM BAG
 Z — DISCARDED

INSPECTOR: H.W. Ryder
 LOGGED BY: H.W. Ryder

APPROVED

DATE

November 1961

DRILLING REPORT

CLIENT
 PROJECT
 SITE

ONTARIO DEPARTMENT OF HIGHWAYS
 W.P. 41-61
 QEW and Niagara Street Interchange

JOB No. 955
 HOLE No. 955-2
 SHEET No. 2 OF 4

DEPTH Feet	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TESTS ¹ Blows
			NO.	TYPE	SIZE In.	DEPTH Ft.	RET'D In.	
			5	BO	2	25.0 25.5 26.0 26.5	18	Pushed 1200 lb
				Vane Test		28.0		
		Material the same but getting softer with depth	6	BO	2	30.0 30.8 31.0 31.5	16	Pushed Easy 500 lbs
				Vane Test		33.0		
			7	BO	2	35.0 35.5 36.0 36.5	18	Pushed 400 lbs
				Vane Test		38.0		
			8	BO	2	40.0 40.5 41.0 41.5	18	Pushed 400 lbs
				Vane Test		43.0		
			9	BO	2	45.0 45.5 46.0 46.5	18	Pushed 400 lbs
				Vane Test		48.0		
			10	BO	2	50.0 50.5 51.0 51.5	18	Pushed 400 lbs
				Vane Test		53.0		
			11	FQ		50.0 55.0		
			12	BO	2	55.0 55.5 56.0 56.5	16	Pushed 400 lbs

DRILLING REPORT

CLIENT ONTARIO DEPARTMENT OF HIGHWAYS
 PROJECT W.P. 41-61
 SITE QEW and Niagara Street Interchange

JOB No. 955
 HOLE No. 955-2
 SHEET No. 3 OF 4

DEPTH Feet	SOIL TYPE	DESCRIPTION COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST *
			NO	TYPE	SIZE In.	DEPTH Ft.	RETD In.	
					Vane Test	58.0		
			13	B0	2	60.0 60.5 61.0 61.5	18	Pushed 300 lbs
					Vane Test	63.0		
			14	B0	2	65.0 65.5 66.0 66.5	18	Pushed
					Vane Test	68.0		
			15	B0	2	70.0 70.5 71.0 71.5	18	Pushed
					Vane Test	73.0		
74.5	Gravel Sand Silt & Clay	Reddish brown and stiff		B0	2	75.0		Unable to push the tube
			16	AQ	2	75.0 75.5 76.0 76.5	8	13 14 18
					Unable to insert vane into soil			
			17	AQ	2	80.0 80.5 81.0 81.5	14	8 10 13
					Vane Test	83.0		Pushed in hard 1000 lbs

DRILLING REPORT

CLIENT ONTARIO DEPARTMENT OF HIGHWAYS JOB No. 955
 PROJECT W.P. 41-61 HOLE No. 955-2
 SITE QEW and Niagara Street Interchange SHEET No. 4 OF 4

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUC- TURE, WATER CONTENT, PLASTICITY, COM- PACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST *	
			NO.	TYPE	SIZE In.	DEPTH Ft.	RET'D In.	Blows	
85.0	Gravel Sand Silt & Clay	Brown and grey, mottled	18	AQ	2	85.0	6		
						85.5		11	
						86.0		15	
						86.5		16	
			19	AQ	2	90.0	12		
						90.5		21	
						91.0		28	
						91.2		100	
			At 88.0 the wash water turned very reddish and larger percentage of gravel was washed up						
			Beyond 88.0 the driving was very difficult						
92.3	End of Hole		AQ	2	92.0				
					92.3	0	112		
<u>* Penetration Test</u> This is the number of blows of a 140-pound weight falling 30 inches required to advance the tube sampler or split-spoon to the depth indicated									

DRILLING REPORT

CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS JOB No. 955
 PROJECT: W.P. 41-61 HOLE No. 955-3
 SITE: QEW and Niagara Street Interchange SHEET No. 1 OF 2
 CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 1:00 P.M. October 30 1961
 FINISHED 5:00 P.M. November 1 1961
 METHOD OF DRILLING: SOIL Modified wash boring CASING DIAM. 4" dia.
 ROCK CORE DIAM.
 LOCATION: LATITUDE Sta. 128 + 00.5 ELEVATIONS: DATUM C.S.C.
 DEPARTURE 53.0 Left Centreline DRILL PLATFORM -
 BEARING QEW GROUND SURFACE 332.7
 INITIAL DIP 90 degrees ROCK SURFACE -
 OTHER DIPS BOTTOM OF HOLE 294.7
 WATER TABLE

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC	SAMPLE					PENETRATION TEST * Blows
			NO	TYPE *	SIZE	DEPTH	RET'D	
					In.	Ft.	In.	
0.0	Humus	Lawn						
0.5	Silty Clay	Yellowish brownish grey with occasional gravel particles. Very stiff and weathered	1	AQ	2	5.0	18	
						5.5		4
						6.0		5
						6.5		7
					Vane Test	8.0		
			2	CO	3	10.0	20	Pushed
						11.8		
13.5	Silty Clay	Brownish grey with occasional gravel size particles, stiff			Vane Test	13.5		
			3	CO	3	15.0	20	Pushed
						16.8		
					Vane Test	18.0		

SAMPLING METHOD

* A - SPLIT TUBE
 B - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 F - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR H.W. Ryder
 LOGGED BY H.W. Ryder

APPROVED

DATE

November 1961

DRILLING REPORT

CLIENT	ONTARIO DEPARTMENT OF HIGHWAYS	JOB No.	955
PROJECT	W.P. 41-61	HOLE No.	955-3
SITE	QEW and Niagara Street Interchange	SHEET No.	2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST *
			NO.	TYPE	SIZE In.	DEPTH Ft.	RET'D In.	Blows
			4	CO	3	20.0 21.8	20	Pushed
					Vane Test	23.0		
25.0	Silty Clay	Brownish grey with occasional gravel size particle. Stiff with some layering of silt and clay	5	CO	3	25.0 26.8	12	Pushed
					Vane Test	28.0		
			6	CO	3	30.0 31.8	20	Pushed
					Vane Test	33.0		
			7	CO	3	35.0 36.8	20	Pushed
38.0		End of Hole			Vane Test	38.0		
<p><u>* Penetration Test</u> This is the number of blows of a 140-pound weight falling 30 inches required to advance the tube sampler or split-spoon to the depth indicated</p>								

DEPTH Feet	SOIL TYPE	DESCRIPTION; COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY COMPACTIONNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST #
			NO	TYPE *	SIZE	DEPTH	RET'D	Blows
					In.	Ft.	In.	
0.0	Humus	Grass and topsoil						
0.5	Gravel Silt and Sand	Backfill material reddish brown						
4.0	Silty Clay	Brownish grey with some gravel size particles weathered slightly, quite stiff	1	BO	2	5.0 6.5	18	Pushed 800 lbs
					Vane Test	8.0		
			2	BO	3	10.0 11.5	18	Pushed 800 lbs
					Vane Test	13.0		
			3	BO	3	15.0 16.7	18	Pushed 800 lbs
		Stiffness decreasing with depth			Vane Test	18.0		
			4	BO	3	20.0 21.7	20	Pushed 800 lbs

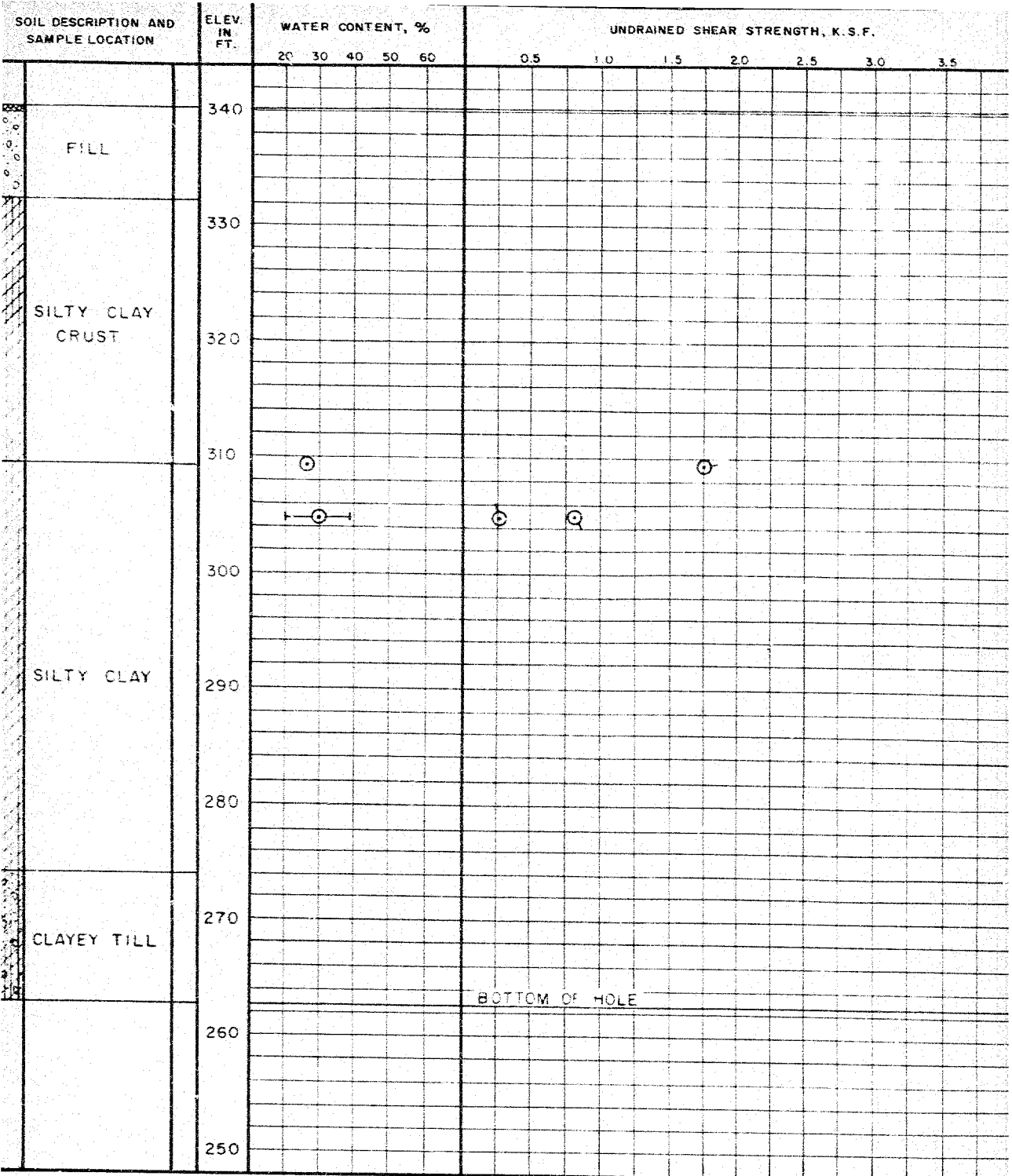
R - CLOTH BAG
S - PLIOFILM BAG
Z - DISCARDED

November 1961

DRILLING REPORT

CLIENT	ONTARIO DEPARTMENT OF HIGHWAYS	JOB No.	955
PROJECT	W.P. 41-61	HOLE No.	955-4
SITE	QEW and Niagara Street Interchange	SHEET No.	2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION. COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST*
			NO	TYPE	SIZE In.	DEPTH Ft.	RET'D In.	Blows
						Vane Test	23.0	
			5	B0	3	25.0 26.7	20	Pushed 800 lbs
						Vane Test	28.0	
			6	00	3	30.0 31.8	20	Pushed
						Vane Test	33.0	
			7	00	3	35.0 36.7	20	Pushed
38.0		End of Hole				Vane Test	38.0	
		* Penetration Test This is the number of blows of a 140-pound weight falling 30 inches required to advance the tube sampler or split-spoon to the depth indicated						



H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

SUMMARY OF DRILLING AND TEST RESULTS

HOLE 955-1

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

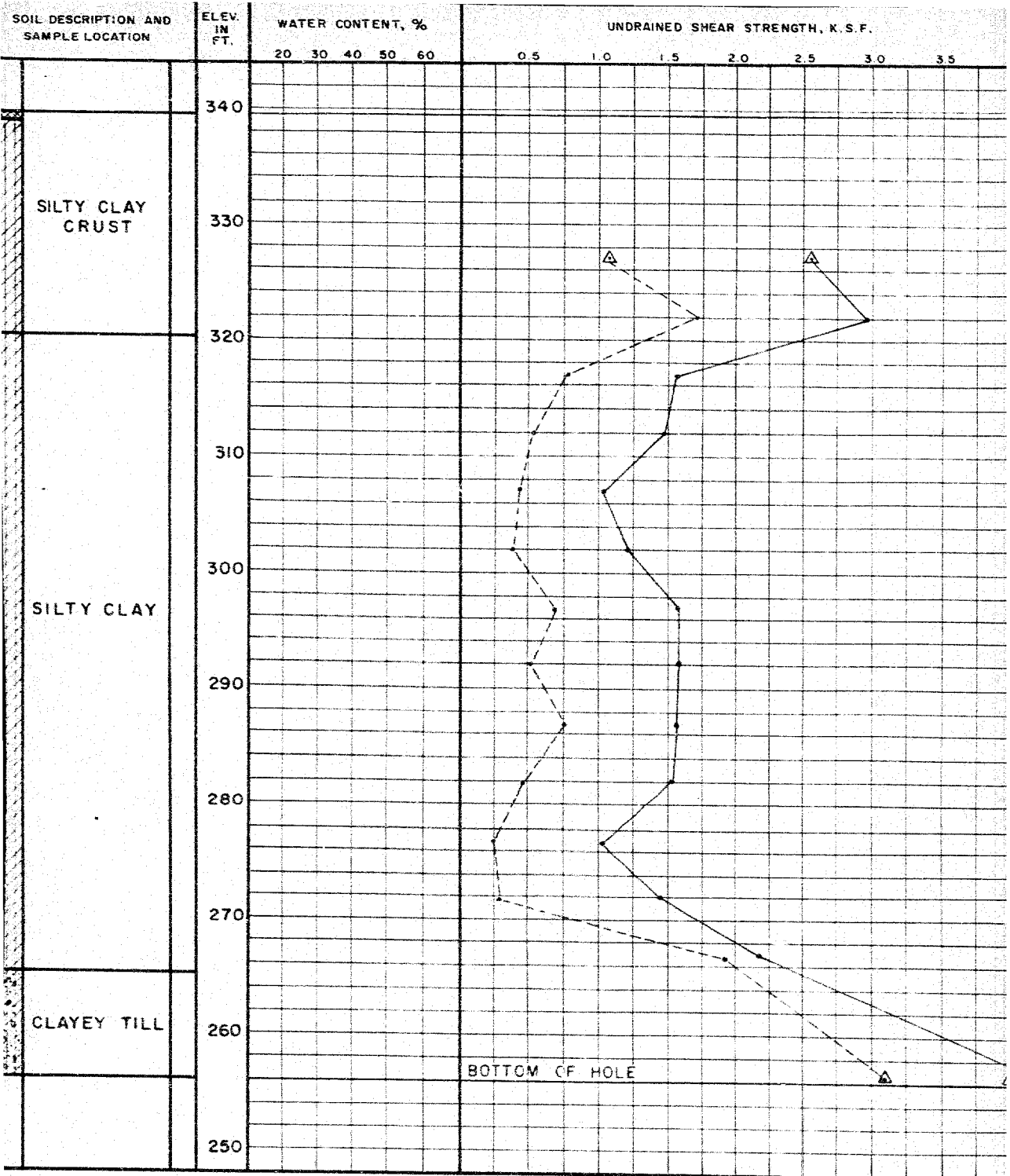
DATE: DECEMBER 1961

W. P. 41-61

H. G. Acres
H. G. ACRES & COMPANY LTD.

JOB No. 955

PLATE 6



3 SOIL SAMPLE
O NATURAL WATER CONTENT
L LIQUID LIMIT
P PLASTIC LIMIT

O UNDRAINED COMPRESSION TEST
Δ FIELD VANE TEST
— NATURAL STRENGTH
--- REMOULDED STRENGTH

0
15—O—5
10
FAILURE STRAIN

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

SUMMARY OF DRILLING AND TEST
RESULTS
HOLE 955-2

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

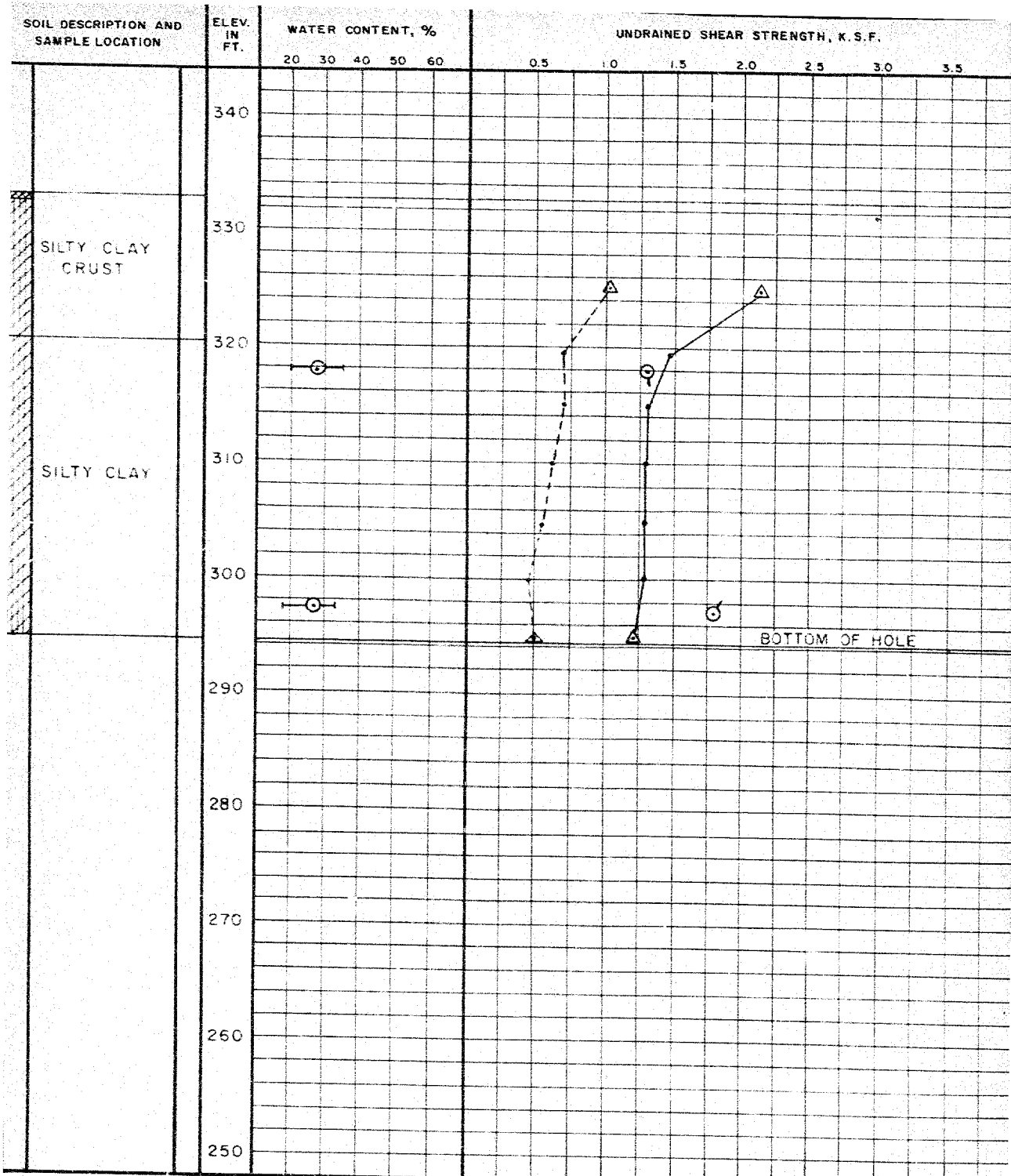
DATE: DECEMBER 1961

W.P. 41-61

H.G. ACRES & COMPANY LTD.

JOB No. 955

PLATE 7



3 SOIL SAMPLE
O NATURAL WATER CONTENT
— LIQUID LIMIT
— PLASTIC LIMIT

O UNDRAINED COMPRESSION TEST
△ FIELD VANE TEST
— NATURAL STRENGTH
--- REMOULDED STRENGTH

0
15 — 5
10
FAILURE STRAIN

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

SUMMARY OF DRILLING AND TEST RESULTS

HOLE 955-3

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

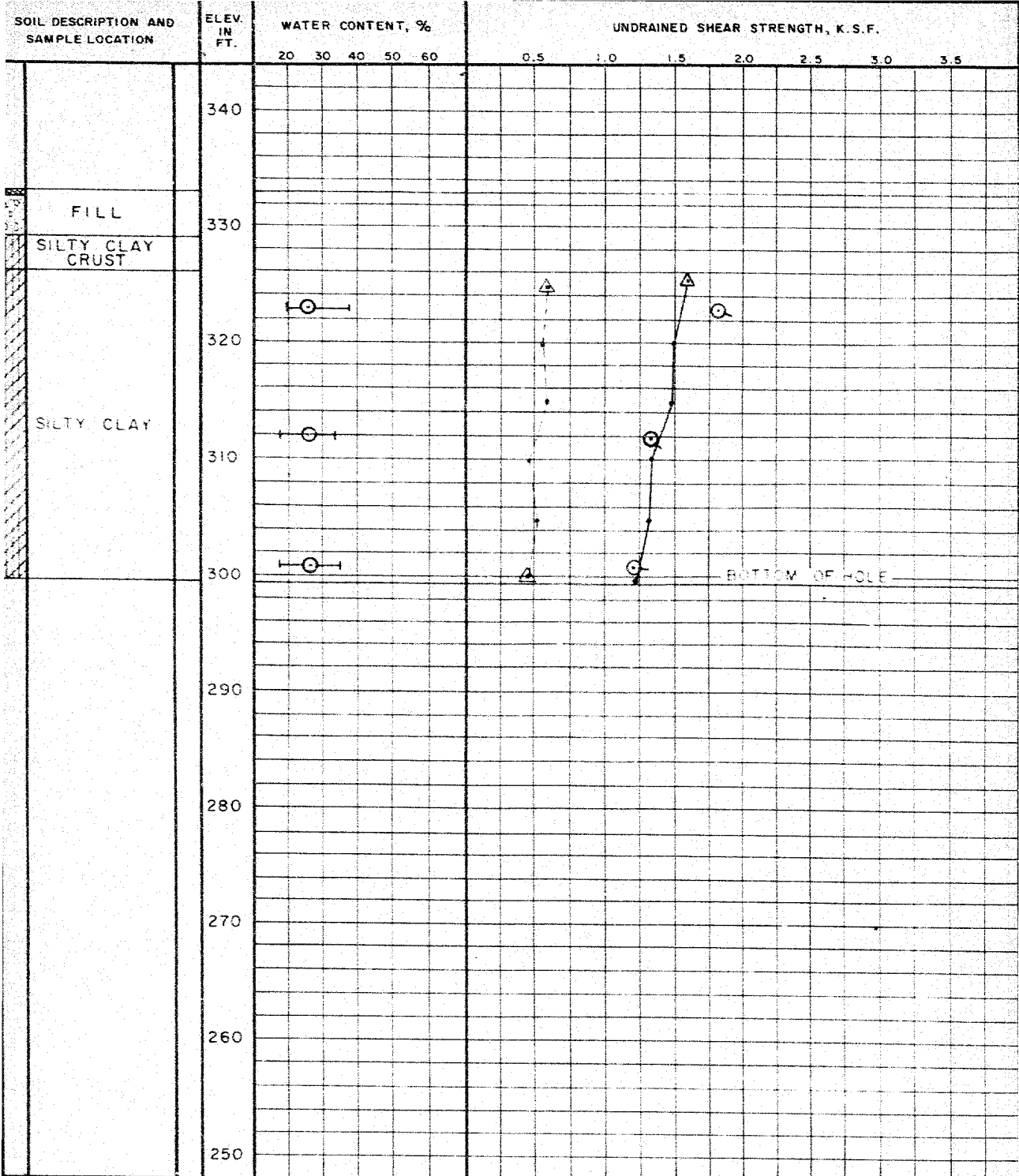
DATE: DECEMBER 1961

W.P. 41 - 61

H. G. Acres
H. G. ACRES & COMPANY LTD.

JOB No. 955

PLATE 8



3 SOIL SAMPLE
 ○ NATURAL WATER CONTENT
 — LIQUID LIMIT
 — PLASTIC LIMIT

○ UNDRAINED COMPRESSION TEST
 △ FIELD VANE TEST
 ——— NATURAL STRENGTH
 --- REMOULDED STRENGTH

15 — 5
 10
 FAILURE STRAIN

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

SUMMARY OF DRILLING AND TEST RESULTS

HOLE 955-4

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

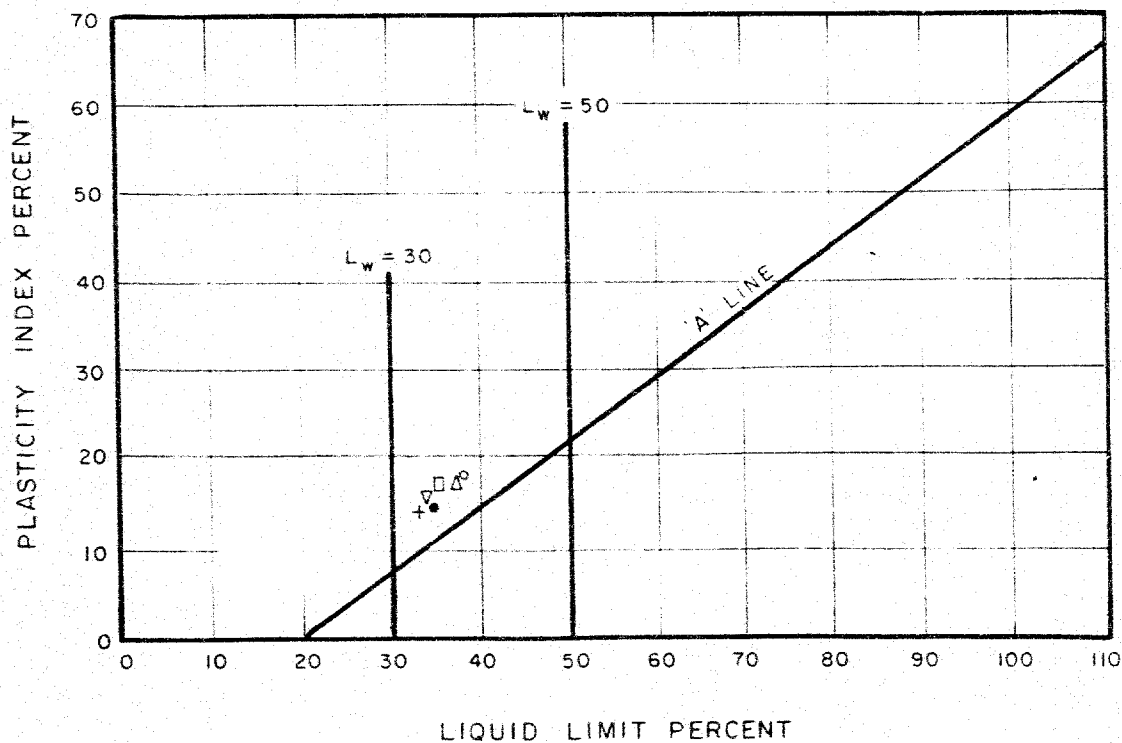
DATE: DECEMBER 1961

W.P. 41-61

H.G. ACRES & COMPANY LTD.

JOB No. 955

PLATE 9



LEGEND	HOLE	ELEVATION (FEET)
○	955-1	305.0
●	955-3	317.7
+	955-3	297.7
Δ	955-4	323.0
▽	955-4	312.0
□	955-4	301.0

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W. P. 41 - 61

SUMMARY OF DRILLING AND TEST
RESULTS

PLASTICITY CHART

APPROVED

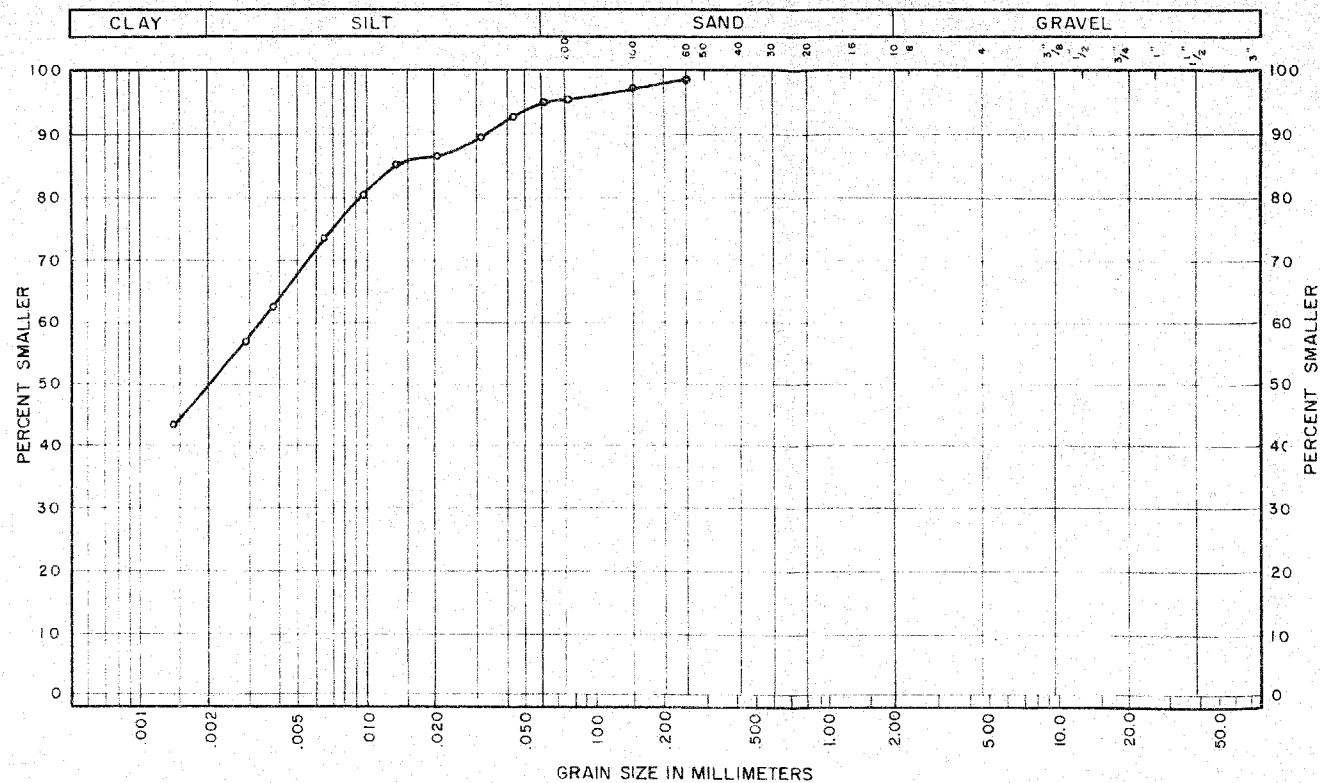
H. G. Acres

H. G. ACRES & COMPANY LTD.

DATE DECEMBER 1961

JOB No. 955

PLATE 10



HOLE No. 955-3

SAMPLE No.

DEPTH 15 FT.

TESTED BY B.H.

H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS

ONTARIO DEPARTMENT OF HIGHWAYS

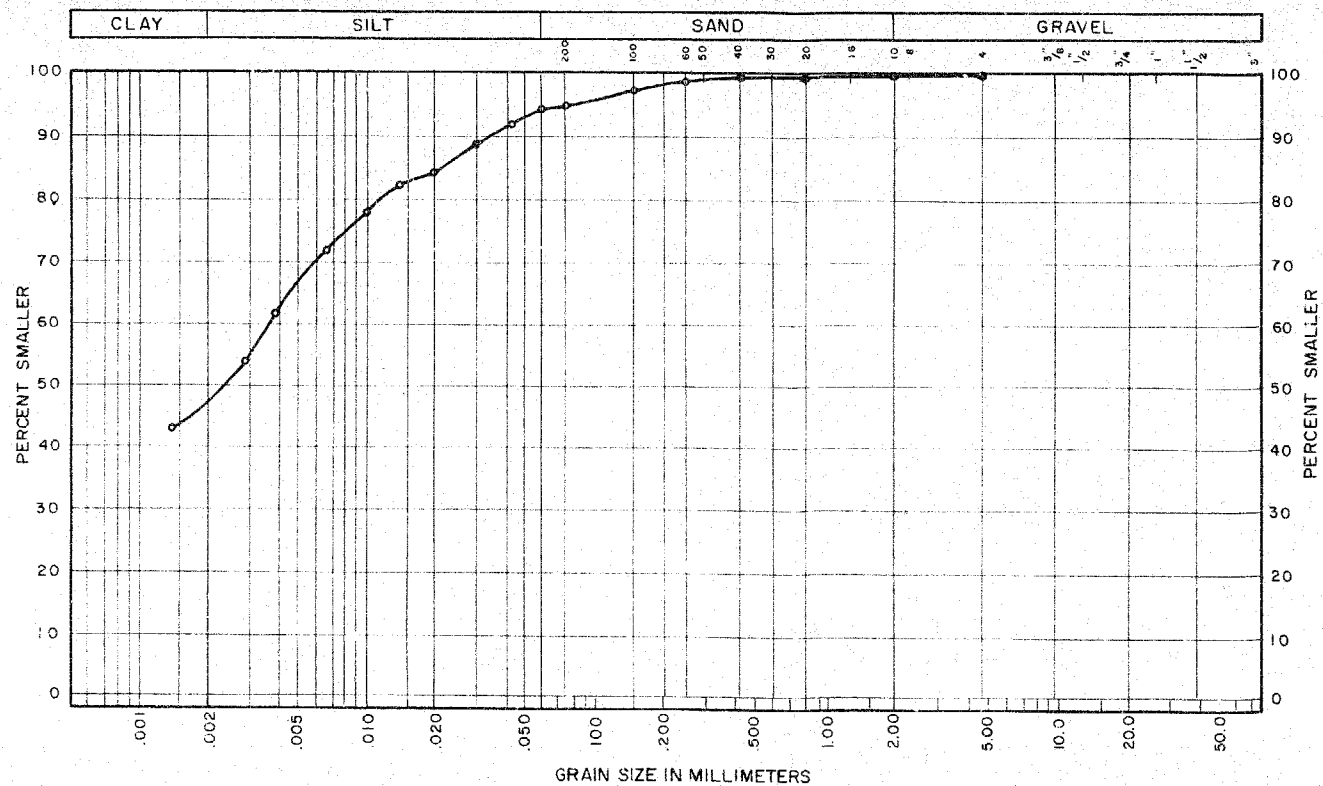
W. P. 41-61

SUMMARY OF DRILLING AND TEST
RESULTS
GRAIN SIZE CURVE

H.G. ACRES & COMPANY LIMITED

DATE DECEMBER 1961

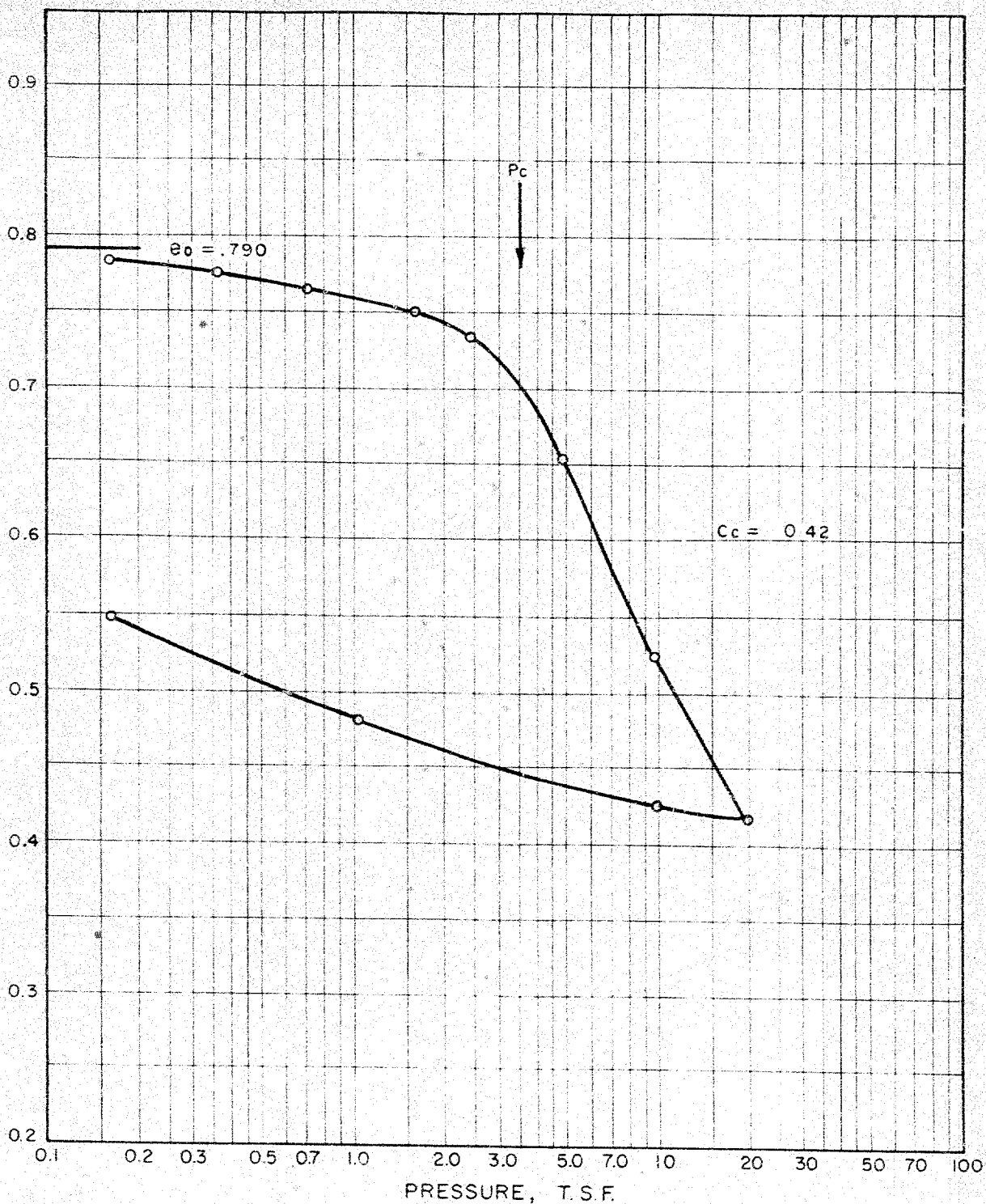
PLATE II



HOLE No. 955-3
 SAMPLE No.
 DEPTH 35 FT.
 TESTED BY B.H.

H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS	
ONTARIO DEPARTMENT OF HIGHWAYS	
W.P. 41-61	
SUMMARY OF DRILLING AND TEST RESULTS	
GRAIN SIZE CURVE	
<i>[Signature]</i>	DATE DECEMBER 1961
H.G. ACRES & COMPANY LIMITED	PLATE 12

VOID RATIO



OVERBURDEN PRESSURE - $P_0 =$
 CONSOLIDATION PRESSURE - $P_c = 3.5$ T.S.F.

NATURAL WATER CONTENT 28.3%
 LOADING INTERVAL 24 HOURS

SAMPLE No. 955-CO-34
 TEST No. 955-9-1

TEST DATE NOV. 4, 1961
 TESTED BY R.L.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W.P. 41-61

CONSOLIDATION TEST

HOLE No. 955-3 SAMPLE ELEV. 318.0 FEET

APPROVED

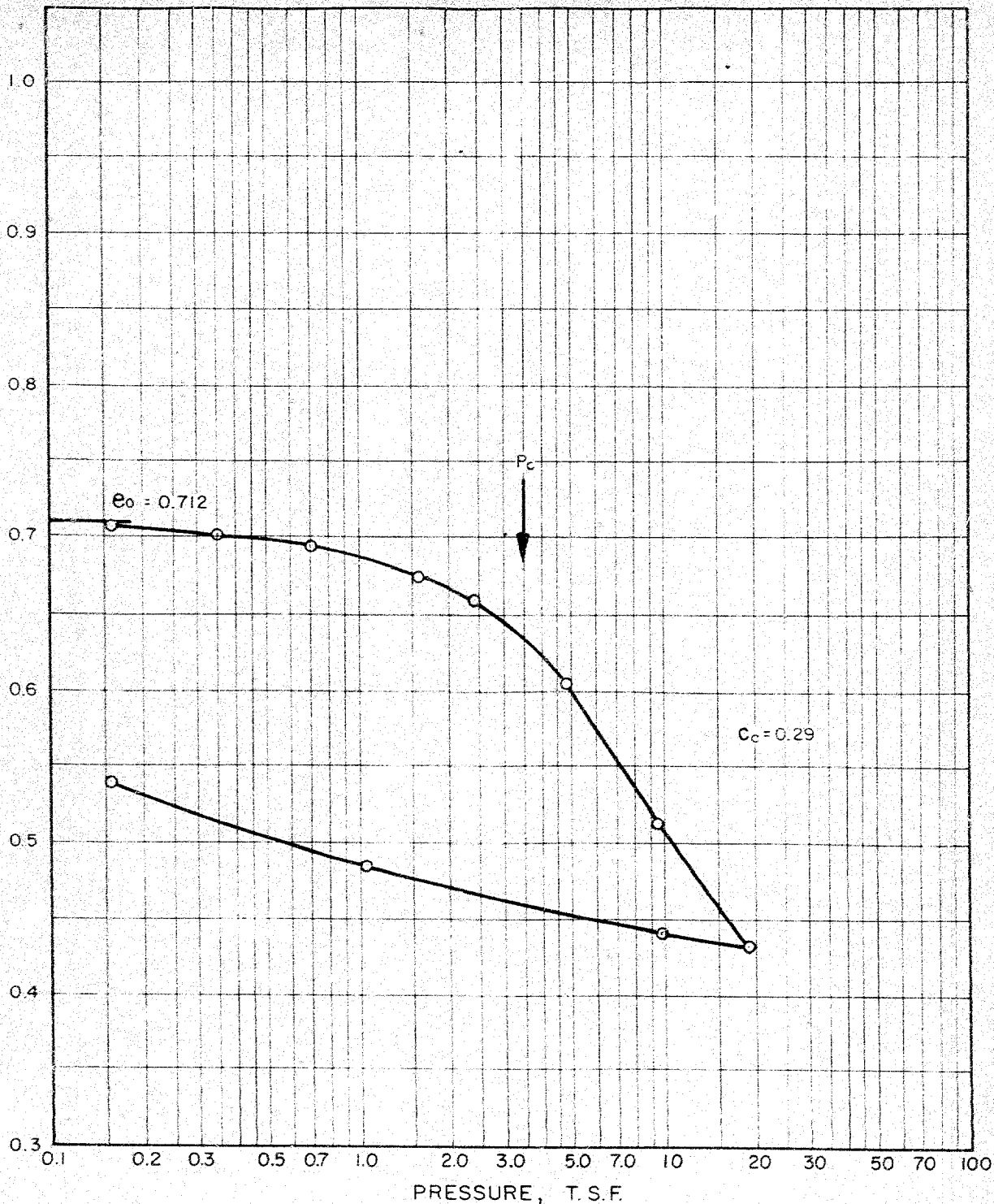
DATE: DECEMBER 1961

JOB No. 955

H.G. ACRES & COMPANY LTD

PLATE 13

VOID RATIO



OVERBURDEN PRESSURE - $P_0 =$
 CONSOLIDATION PRESSURE - $P_c = 3.5$ TSF

NATURAL WATER CONTENT 25.9%
 LOADING INTERVAL 24 HRS.

SAMPLE No. 955 - CO-38
 TEST No. 955 - 9 - 2

TEST DATE NOV. 4 1961
 TESTED BY R.L.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W.P. 41 - 61

CONSOLIDATION TEST

HOLE No. 955-3

SAMPLE ELEV. 297.0 FEET

APPROVED

DATE: DECEMBER 1961

[Signature]
 H. G. ACRES & COMPANY LTD

JOB No. 955

PLATE 14