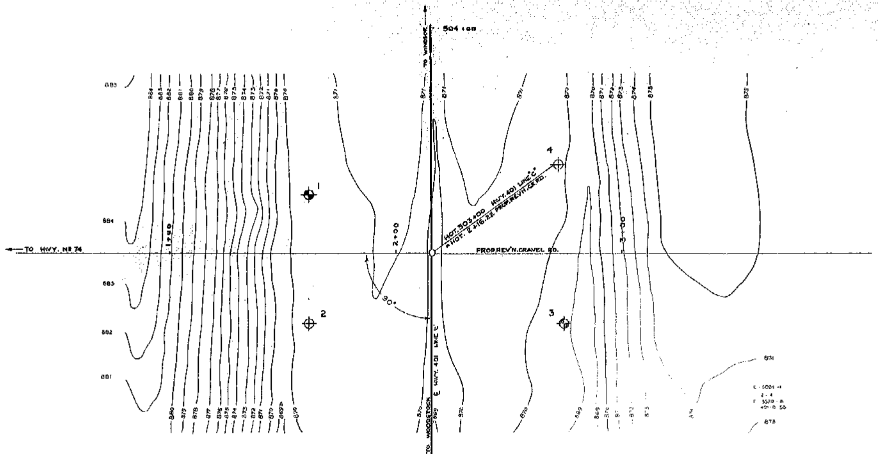
W.O. 57-F-47A

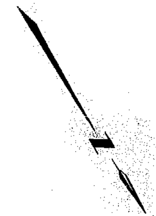
HWY. 401

LINE 'C'

401 IL-85

TRIM LINE



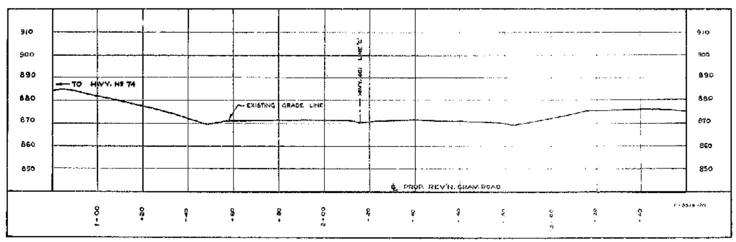


PLAN

SCALE - 1 in. = 20 ft.

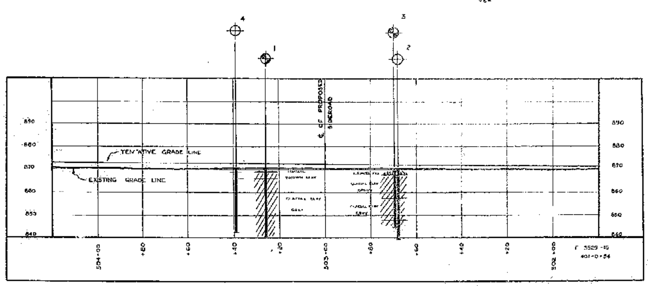
LEGEND	
BORE HOLES	
PENETRATION HOLES	
BORE & PENETRATION HOLES	

HOLE NO.	ELEVATION	STATION	DISTANCE FROM EXIS. ℄
1	870.3'	503+26	54' LT.
2	870.0'	502+68.5	54' LT.
3	869.3'	502+68.5	58' RT.
4	870.1'	503+39	56' RT.



PROFILE
FOR
GRAVEL ROAD REVISION

SCALE HOR. VER. 1 in. = 20 FT.



PROFILE
FOR
HWY. NO. 401 LINE 'C'

40114-085
GEOCRES No.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION - DOWNSVIEW

GRAVEL ROAD REVISION
PROPOSED CROSSING
5 MILES S. OF LONDON

SHOWING POSITION & ELEVATION OF HOLES

HWY. Nº 401 LINE 'C'	W.P. 98-57	DIV. NO. 2
CO. MIDDLESEX		
TWP. WESTMINSTER	LOT. 21	CON. VI
SCALE: AS SHOWN	SUBMITTED BY	DATE JAN. 31. 1958
DRAWN BY O.M.Z.	APPROVED BY	DRAWING Nº F-57-47A

TRIM LINE

TRIM LINE

TL-129 (REV. 65) H.D.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

401/4-085

GEOCRES No.

DRILL RIG 64-2 OPERATION BORE & PENET'N JOB L-57-42 W.P. 98-57 BORING 1 STA. 503+26 (BWIT)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957
SAMPLER HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY J.L. CHECKED BY J.L. DATE BORING 27 NOV. 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
W.L. - WATER LEVEL IN CASING
W.T. - WATER TABLE IN SOIL
K - PERMEABILITY
C - CONSOLIDATION
CA. - CASING
γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE CONDITION

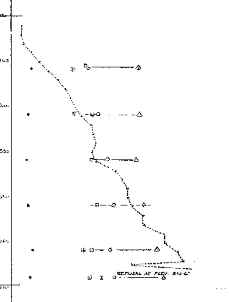
- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION
870.3'		GROUND LEVEL
869.8' 0.5'	W.L.	TOPSOIL
	W.L. AT ELEV. 869.2 ON DEC. 2/57	BROWN CLAY
		GLACIAL CLAY GREY
840.8' 29.5'		END OF BOREHOLE

WATER CONTENT W% — O - NAT., □ - P.W., Δ - L.W.

PENETRATION TEST RESISTANCE BLOWS PER FOOT
AT STANDARD ENERGY (4200 IN. LBS. PER BLOW)
D. CONE PEN. X   STAND. PEN



CASING BLOWS (ACTUAL): BX CASING SHOE; END OF CASING AT ELEV. 842.1'

SAMPLES

OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
					870.3'
					865.3'
γ 1160 pcf		S.S.	1	20	67
					860.3'
γ 1199 pcf		S.S.	2	16	67
					855.3'
γ 1186 pcf		S.S.	3	15	100
					850.3'
γ 1170 pcf		S.S.	4	16	86
					845.3'
γ 1132 pcf		S.S.	5	22	
					842.3'
γ 1142 pcf		S.S.	6	19	78

TRIM LINE

FS-129 (REV. 55) | HDR.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

110 I 14-085
GEOCRES No.

DRILL RIG 54-2 OPERATION PENETRATION JOB F 67-47 WP 98-67 BORING 2 STA. 602+685 (54' LT)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1967
SAMPLER HAMMER WT. 250 LBS. DROP 24 INCHES COMPILED BY P.C. CHECKED BY A.L. DATE BORING NOV. 26, 1967

ABBREVIATIONS

V - INSITU VANE SHEAR TEST; Q - TRIAXIAL QUICK; K - PERMEABILITY
M - MECHANICAL ANALYSIS; S - TRIAXIAL SLOW; C - CONSOLIDATION
U - UNCONFINED COMPRESSION; WL - WATER LEVEL IN CASING; CA - CASING
Q_C - TRIAXIAL CONSOLIDATED QUICK; WT - WATER TABLE IN SOIL; γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK; S.S. - SLEEVE SAMPLE
DO - DRIVE OPEN; PS - PISTON SAMPLE
DF - DRIVE FOOT VALVE; WS - WASHED SAMPLE
TO - THIN WALLED OPEN; RC - ROCK CORE

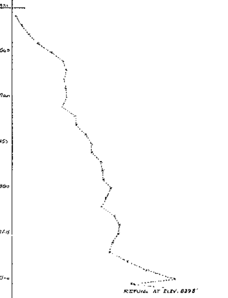
SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE						SAMPLES					
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
870		GROUND LEVEL		870							
5				865							
10				860							
15				855							
20				850							
25				845							
30				840							
35				835							

WATER CONTENT W% — — N - NAT □ PW △ LW

PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS PER BLOW)
D. CONE PEN X----X STAND PEN •——•
50 100 150 200



TRIM LINE

TL-129 (REV. 66)

H.O.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

40 I 14-085
GEOCRES No.

DRILL RIG 54-2 OPERATION BORE & PENET'N JOB F-67-47 W.P. 98-62 BORING 3 STA. 602+68.5 (58' Rt)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1967

SAMPLER HAMMER WT. 350 LBS. DROP 22 INCHES COMPILED BY H.T. CHECKED BY A.L. DATE BORING 6 DEC. 1967

ABBREVIATIONS

V - INSITU VANE SHEAR TEST; Q - TRIAXIAL QUICK; K - PERMEABILITY
M - MECHANICAL ANALYSIS; S - TRIAXIAL SLOW; C - CONSOLIDATION
U - UNCONFINED COMPRESSION; WL - WATER LEVEL IN CASING; CA. - CASING
Q_C - TRIAXIAL CONSOLIDATED QUICK; WT. - WATER TABLE IN SOIL; γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK; S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN; P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE; W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN; R.C. - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

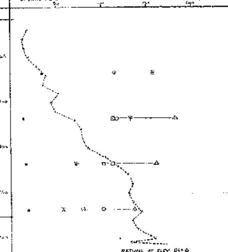
SOIL PROFILE

ELEVATION / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE
864.3'		GROUND LEVEL		
862.8' / 1.5'		GRAVEL (FILL)		
		GLACIAL CLAY / BROWN		860
				855
		GLACIAL CLAY / GREY		850
				845
841.8' / 22.5'		END of BOREHOLE		840

SHEAR STRENGTH IN LBS PER SQ FT: 1000, 2000, 3000, 4000

WATER CONTENT W%: O - NAT.; □ - P.W.; △ - L.W.

PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS PER BLOW)
D. CONE PEN. X----X; STAND. PEN. •——•



CASING BLOWS (ACTUAL): BX CASING PUSHED; END of CASING AT ELEV. 844.3'

SAMPLES

ELEV.	OTHER TESTS	CONDITION	TYPE	NO	PENETRATION RESISTANCE	RECOV. %
864.3'						
859.3'	[illegible] pcf		T.T.	1	38	100
854.3'	[illegible] pcf		T.T.	2	14	78
849.3'	[illegible] pcf		T.T.	3	18	80
844.3'	[illegible] pcf		T.T.	4	19	61

TRIM LINE

TL-199 (REV. 56) HOR.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

40 I 14-085
GEOCRES No.

DRILL RIG 64-B OPERATION PENETRATION JOB E 57-47 WP 98-57 BORING 1 STA. 303+39 (36' RT)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957

SAMPLER HAMMER WT. 250 LBS. DROP 27 INCHES COMPILED BY H.L. CHECKED BY A.L. DATE BORING 4 DEC. 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
N - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_c TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL
K - PERMEABILITY
C - CONSOLIDATION
CA - CASING
γ - UNIT WEIGHT

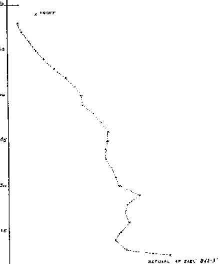
SAMPLE TYPES

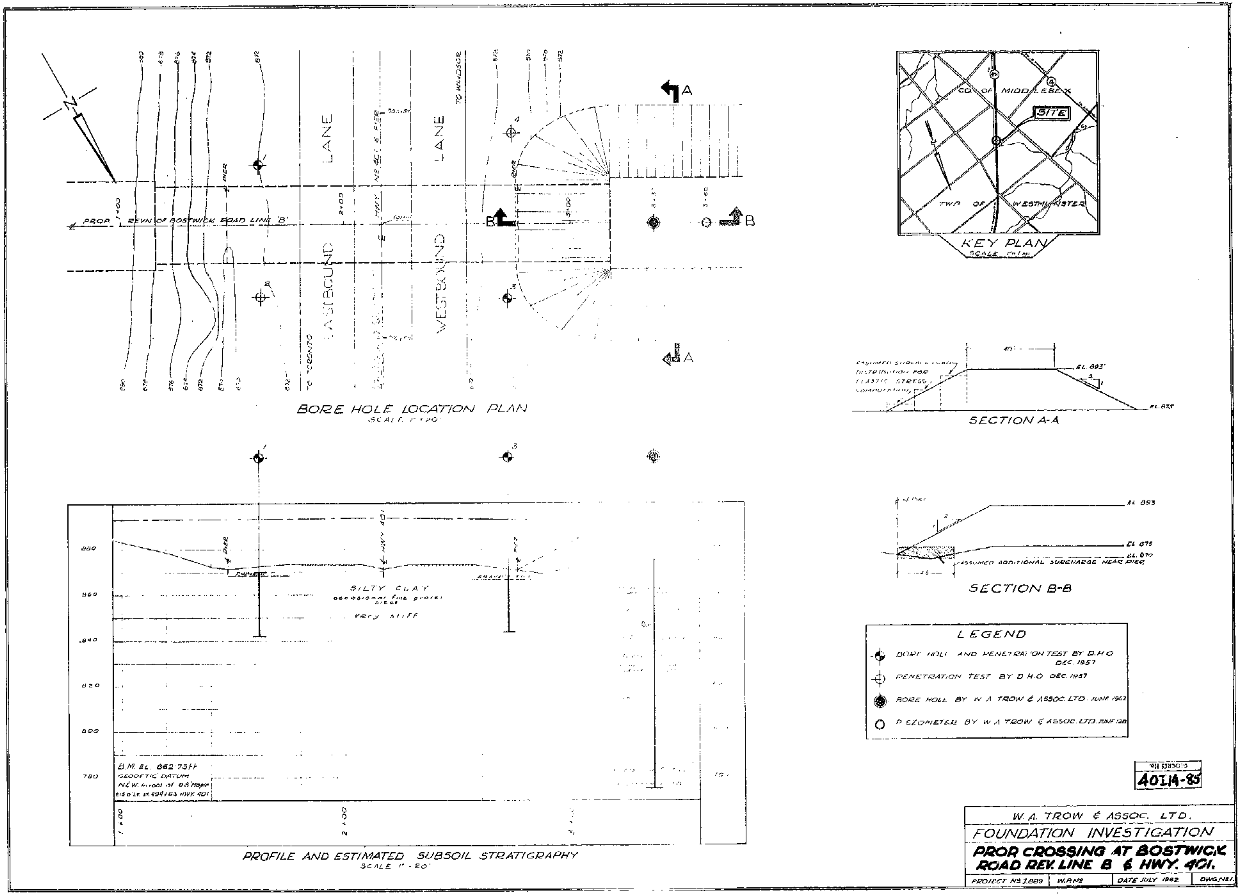
C.S. - CHUNK
D.O. - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
PS - PISTON SAMPLE
WS - WASHED SAMPLE
RC - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE						SAMPLES					
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% — O - NAT □ - PW Δ - LW / PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS PER BLOW) D. CONE PEN. X———X STAND PEN ●———● 50 100 150 200	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
870.1'		GROUND LEVEL		870							
5'				865							
10'				860							
15'				855							
20'				850							
25'				845							
30'				840							




PROP. REVN OF BOSTWICK ROAD LINE 'B'
EASTBOUND LANE
WESTBOUND LANE
TO WINDSOR
A
B
BORE HOLE LOCATION PLAN
KEY PLAN
SITE
SECTION A-A
EL. 893'
SECTION B-B
EL. 893
EL. 875
ASSUMED ADDITIONAL SURCHARGE NEAR PIER
SILTY CLAY
Very stiff
B.M. EL. 862.73FT
GEODETIC DATUM
PROFILE AND ESTIMATED SUBSOIL STRATIGRAPHY
LEGEND
BORE HOLE AND PENETRATION TEST BY D.H.O DEC. 1957
PENETRATION TEST BY D.H.O DEC. 1957
BORE HOLE BY W. A. TROW & ASSOC. LTD. JUNE 1962
PIEZOMETER BY W. A. TROW & ASSOC. LTD. JUNE 1962
GEOCRES No.
40I14-85
W. A. TROW & ASSOC. LTD.
FOUNDATION INVESTIGATION
PROP. CROSSING AT BOSTWICK ROAD REV. LINE B & HWY. 401.
PROJECT No. 1.889
W.P. No.
DATE JULY 1962
DWG. No. 1.

BA 721

ONTARIO

DEPARTMENT OF HIGHWAYS

1

Memo to Mr. A. Toye.
Bridge Engineer.

From Materials & Research.

Date March 31st, 1958.

Subject Re: Foundation Report. Hwy#40 and Boswick Road. W.P.98-57. W.J.F. 57-47.

WP 721

We are forwarding herewith the above mentioned Foundation Report for your information. This proposed structure appeared on the earlier Programmes, and has since been cancelled. It does not as yet appear on the 1959-1960 Programme.

F. C. Brownridge.
Materials & Research Engineer.

per: A. Rutka

A. RUTKA.
Principal Soils Engineer.

c.c. Mr. A. Toye.
Mr. H. Tregaskes.
Mr. D.G. Ramsay.
Mr. W.L.Fraser.
Mr. A. Watt.
Dr. P. Karrow.
Foundation Section.
File.

Department of Highways
COPY
For the Information of

Mr. ~~J. L. Keen,~~
Sr. Bridge Project Eng.

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

G. Scott

May 14, 1962

Bridge Site #20-365 W.P. 98-57
Bostwick Road Underpass
Hwy. 401 District #2

The above structure is to be designed to allow for a revised highway 401 cross section.

Attached herewith please find a print of site plan E 3024-2 showing tentatively proposed locations of the foundations for a 4 span type structure.

We would be pleased if you will review the foundation report BA 721 and inform us what changes will be required in connection with the revised design.

GS/et

Gavin Scott,
Bridge Location Engineer.

cc. S. McCombie
cc. J. L. Keen
cc. N. D. Smith
cc. R. Fitzgibbon

D. H. O.
TORONTO
RECEIVED
MAY 1962
OFFICE
BRIDGE

BA 721

FOUNDATION REPORT

ON

NEW BRIDGE AT HIGHWAY 401 LINE "C" UNDERPASSING THE PROPOSED GRAVEL ROAD REVISION, ABOUT FIVE MILES SOUTH OF LONDON, TOWNSHIP OF WESTMINSTER.

40 I 14-85
GEOCRES No.

Plan No: F-3529-18

Station No: 503+00

Distribution:

Mr. A. Toye
Bridge Engineer (2)

Mr. H. Tregaskes
Construction Engineer (1)

Mr. D. G. Ramsay
Design Engineer (1)

Mr. W. L. Fraser
Dist. Engr. London (1)

Mr. A. Watt
Water Resources Commission (1)

Mr. P. Karrow
Department of Mines (1)

Foundation Section (1)

File (1)

W.P. 98-57

W.J. F-57-47

INTRODUCTION.

This report covers the subsoil investigations carried out at this site to determine the bearing capacity of layers for supporting the foundations of the proposed bridge.

The location of the site is about five miles south of London, where Highway 401 line "C" will underpass the proposed Gravel Road Revision, in the Township of Westminster (Con. VI).

The field work started on November 21, 1957, and was completed on December 6, 1957.

DESCRIPTION OF THE SITE AND FIELD WORK:

The site is in the area referred to as Mount Elgin Ridges characterized by clay moraines of gentle slopes. The clays in the area are believed to be of glacial origin deposited during the late lake Whittlesey and overlying the basic till underneath.

The subsoil explorations were carried out by means of a skid mounted coredrill machine. In the course of investigations two boreholes with adjacent dynamic cone penetrations and two separate dynamic cone penetrations were made. The boreholes were advanced by alternately driving and washing the BX casing. During this operation, where possible, undisturbed samples were extracted and tested at the laboratory. By driving the 2" cone from ground surface down to refusal the dynamic cone penetration profiles were established.

The explorations revealed that the stratigraphy at the site was topsoil underlain by a fairly uniform layer of clay. The colour in the upper zone is brown changing to grey at a depth of some 4 ft. in borehole No. 1, and some 11 ft. in borehole No. 3 below the ground surface. Due to the high resistance offered by the skin friction of the clay layer, casings had to be drilled down,

The location of the boreholes is shown on drawing No. F-57-47A, and their elevations on log sheets under Appendix I.

ANALYSIS OF FIELD AND LABORATORY FINDINGS.

The undisturbed samples were tested in the laboratory and the following results obtained. The clay in the layer is of medium to heavy texture and referred to be of glacial origin. Its liquid limit is about 32%, plastic limit 18% and moisture content 20%. Its average density is 135 p.c.f. The clay is inorganic and of low plasticity. Its moisture content being close to its plastic limit would indicate its preloaded condition. On the other hand the presence of gravel stones in the samples handicapped measuring its compressibility by consolidation tests.

The shear strength of the clay material was determined from the results of unconfined compression tests. Some of the samples tested, due to the presence of gravel, gave unreliable results. These figures were discarded. On the other hand judging from the rest of the unconfined compression test results the layer in a depth of 25 ft. below the ground surface provides a cohesion value of about 1700 p.s.f. The standard penetration tests registered 18 blows per foot penetration.

APPRAISAL OF BEARING VALUES.

Assuming the structure here will be supported on 7 ft. wide (B) continuous footings, and taking depth factor $\frac{D}{B} = 1$, then at elevation 862 ft. the bearing value of the layer will be

$(q = CN_c + \gamma D)$ (I) 5.75 T.S.F. Using a safety factor of 3, the safe bearing value will be about 2 T.s.f.

CONCLUSIONS AND RECOMMENDATIONS:

From the above discussion it will follow that:

1. The stratigraphy at the site presents one fairly uniform clay layer down to the end of boreholes.
2. The layer is competent to provide about 2 T.s.f. safe bearing value at elevation about 862 ft. and possibly more at greater depth.
3. It will be convenient to support the overpass bridge on spread footing type foundations placed at elevation 862 ft. or lower.
4. The approach fills to the new structure do not present any stability problem.

(I). Meyerhof G.G., "Recent Studies of Foundation Behaviour", The Engineering Journal, Vol. 37, No. 2 (February 1954), p. 123.

NR 24/9/59

LCS checked Allowable [illegible]

For Spread footing [illegible]

OK for 3T/ft²

V. Korlu

Foundation Engineer.

APPENDIX I.

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

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

5A 1474

40I 14-85
GEOCRES No.

Project: J889

July 20, 1962

40I 14-85

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

Attention: Mr. A.G. Stermac, P.Eng.

Re:

Settlement Study
Bostwick Road Overpass
W.P. 98-57, Highway 401
District No. 2, London

Dear Sirs:

The enclosed report presents the findings of a settlement study for the proposed bridge crossing at the above noted site.

Very stiff silty clay or clayey silt extends to the full tested depth of 100 feet at this location. A thin dense fine sand layer intersects the clay between depths of 32 to 37½ feet.

Immediate and consolidation settlements have been computed under the centre of the west abutment and the adjacent pier. It is understood that these positions will be instrumented.

The total settlements at these two positions have been computed as 2¾ inches and 1 inch respectively.

The differential movement, therefore, is 1¾ inches, 50 percent of which will occur upon placing of the fill.

The remaining approximate 7/8 inch of differential consolidation settlement represents an angular displacement of only about 1/600 of the span length. Thus, if the placing of the approach fill precedes the girder and deck construction, it would appear unnecessary to make special provision for this movement.

The computations of these settlement values, and engineering reasoning involved in this work is presented in the main body of the report.

We hope that this information assists you in the design of these structures and in the interpretation of the field settlement observations.

Yours very truly,

W. Trow

William A. Trow, P.Eng.

WAT/gc
Encls.

WILLIAM A. TROW AND ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH BRANCH
PARLIAMENT BUILDINGS, TORONTO, ONTARIO

SETTLEMENT STUDY
BOSTWICK ROAD OVERPASS
W.P. 98-57, HIGHWAY 401
DISTRICT NO. 2, LONDON

Project: J889 July, 1962

William A. Trow and Associates Ltd.

TABLE OF CONTENTS

ENCLOSURES

SETTLEMENT STUDY
PROPOSED OVERPASS, BOSTWICK RD. OVER HWY. 401
DISTRICT NO. 2, LONDON, ONTARIO

Project

The original plans for this crossing incorporated a single span, closed abutment bridge, with approach embankments approximately 20 feet high.

A foundation investigation for this earlier project was performed by Department of Highways of Ontario personnel in December, 1957. The findings of this work, presented in a report dated March, 1958, indicated that very stiff glacial silty clay underlies the site to a depth of at least 30 feet. It was recommended that the bridge structure should be founded on spread footings at about El 862 feet. The design net bearing pressure was 2 tsf.

No study was made at that time of possible settlement under the additional weight of the approach fill. Due to the approximately similar stress conditions prevailing around and under each abutment, the construction would have settled fairly uniformly with little or no differential movement.

Recently revised plans for this crossing, however, call for a 4 span structure with spill-through type abutments. The differing stress conditions under the abutments and piers in this layout are sources of potential differential movement under these components of the structure.

In order to assess the probable magnitude of this differential settlement, as well as the overall settlement under the weight of this overpass, one additional boring was made at this site.

This report presents the findings of this field work, together with the results and a discussion of the settlement computations. It is understood that this information will be used in the interpretation of actual measurements of fill and abutment settlement.

Reference is made in this submission to a report on a settlement study for the Dingman Creek Road crossing over Highway 401, located about 4 miles to the east of this site.* The field work, laboratory tests and analyses for both projects were carried out simultaneously.

* "Settlement Study, Dingman Creek Road Overpass, WP 22-59-1, Hwy. 401, District No. 2, London" - Report of W.A. Trow & Associates Ltd., July, 1962

Site Description

The site of the proposed overpass is located some 720 feet southwest of the existing gravel road intersection with Highway 401. At this proposed location, the Highway is cut a few feet below the surrounding ground surface, through one of the many surface undulations in this area. The presence of this small cutting reduces the height of the required approach fill.

This location is some 3 to 4 miles southwest of the Dingman Creek Road, - Highway 401, intersection. As noted in the report for that overpass, the successive strata in this area are believed to represent deposits of the glacial Lakes Maumee and Whittlesey.

Soil Types Encountered

In this recently completed investigation, one borehole was put down a short distance beyond the proposed west abutment in the position shown on drawing 1. This location, on the west side of the highway was chosen because the embankment fill will be highest under this approach.

The result of this boring is shown in the log, presented as drawing 2 of this report. The relevant data of this work and of the preceding investigation is summarized in stratigraphical form on drawing 1.

Reference to these drawings shows that very stiff glacial silty clay underlies the site. A hard crust above a depth of 10 to 15 feet is apparent; otherwise the clay is reasonably uniform to a depth of 32 feet. It is generally of low to medium plasticity with the natural moisture much nearer to the plastic limit than to the liquid limit. Occasional fine to medium gravel sizes throughout the clay indicate its glacial origin.

A stratum of dense medium sand, grading to silty very fine sand with depth, intersects the clay body between a depth of 32 to 37½ feet. Upon penetrating this layer, water rose in the borehole and stabilized almost immediately at the depth of about 29 feet.

Immediately below this granular layer, a silty clay of similar characteristics to the upper stratum was encountered extending to a depth of about 43 feet.

Below this depth, the soil changes to a slightly clayey cohesive silt of low plasticity. Irregular intrusions of both clay and fine sand were noted throughout the stratum. Below a depth about 89¼ feet, a significant increase in the penetration resistance of the soil was noted, although this condition is relatively unimportant for this settlement study.

In the laboratory, consolidation tests were performed on three samples from depths of 20, 41 and 80 feet respectively. Samples of the clayey silt from both 60 feet and 80 feet were prepared. Although some unavoidable sample disturbance was suspected at this time, the sample from 80 feet was considered to be satisfactory for test. However, the results of these tests, presented on drawings 3 and 4 of this report, indicate that severe sample disturbance to the silty soil from 80 feet has occurred. The remaining two tests indicate that, above a depth of 40 feet, the soil has been heavily preconsolidated under loads of about 20,000 psf in excess of the present overburden pressure. A similar result was obtained for tests on samples from the Dingman Creek Overpass study.

The selection of compressibility values from these results will be discussed more fully in the following sections of this report.

In order to determine the "elastic" modulus of the soil, under stress conditions approximating those in the ground, two K_o consolidated undrained triaxial tests with pore pressure measurement were performed on samples from 20 and 50 feet. The results of these tests, presented on drawings 5 and 6, indicate that the undrained shear strength is approximately 2800 psf. This strength may be slightly in excess of the true undisturbed strength since some drainage from the sample occurred during the consolidation stage of the test. The value of the pore pressure coefficient, A, was also determined in this test, and was found to lie between 0.14 and zero for the range of stresses to be applied to the soil.

Grain size analyses results for these samples are presented on drawing 7. In addition, Atterberg Limits tests were performed on samples from various levels. These latter tests show that the silty clay above 45 feet is of low to medium plasticity, with clayey silt of low plasticity below this depth.

An open stand pipe piezometer was installed at a depth of 39 feet at the commencement of the field work. The purpose of this installation was to provide a reliable means of determining the final water table level if it did not stabilize during the field work. However, since this piezometer penetrated the previously noted wet sand stratum, the water level stabilized in the standpipe almost immediately at the depth of 29.3 feet or El 846.5 feet. Subsequent observations showed minor fluctuations in the level as the main borehole was advanced below the sand.

Settlement Analyses

1) General

It is understood that the approach embankment will attain a level of about El 893 feet at the west abutment, or an approximate height of 18 feet above the general ground level of El 875 feet. The approach fill, spilling through the abutment and extending toward the pier, covers somewhat lower ground near the pier where the average ground level is approximately El 870 feet.

In the settlement analyses of this report, the following assumptions regarding the embankment have been made.

i) The unit weight of the approach fill has been assumed to be 130 pcf.

ii) The slopes of the embankment, both along the sides and at the spill-through, have been assumed at a 2:1 angle.

iii) The weight of an additional 5 feet thickness of fill under the spill-through has been included in the stress calculations for this study.

These assumptions are summarized on drawing 1.

The magnitude of the total settlement, S_t, under the applied weight of the approach fill is normally determined from the following expression:

$$S_t = S_i + \mu . S_c \; *$$

where: S_i is the immediate settlement due to the elastic compression of the soil,

S_c is the computed "oedometer" settlement determined from the consolidation test,

and μ is a coefficient relating the field consolidation to the laboratory value

The selection of suitable soil properties and the computation of these respective portions of the total settlement are discussed in the following subsections.

2) Immediate Settlement

Immediate or elastic settlement will take place in both the silty clay and fine sand underlying the site, and will occur simultaneously upon the application of the surcharge weight.

The magnitude of this settlement, S_i, can be determined from the expression:

$$S_i = 0.5 \sum \frac{\Delta p . \Delta H}{E} \; **$$

* "A Contribution to Settlement Analysis of Foundations on Clay" Skempton & Bjerrum - Geotechnique - Dec., 1957

** $S_i = \int_0^H \left(\frac{\sigma_1}{E} - \frac{u\sigma_2}{E} - \frac{u\sigma_3}{E} \right) dH = 0.5 \sum \frac{\sigma_1 \Delta H}{E}$ for $u = 0.5$ & $\sigma_2 = \sigma_3 = \sigma_1/2 = \Delta p/2$

where: Δp is the average increment of vertical stress in any depth, ΔH,

and E is the "elastic" modulus of the soil

The stress increment, Δp, has been determined by using a Newmark influence chart, with the assumed surface distribution of load as shown on drawing 1. In this study, no reduction in stress has been made for any possible hard surface crust condition, for similar reasons as outlined in the settlement study report for the Dingman Creek Road overpass.

The value of E for the clay has been determined from the stress-strain curves of k_o consolidated undrained triaxial tests with the vertical consolidation pressure approximately equal to the overburden pressure. The results of these tests on samples from 20 and 50 feet, presented on drawings 5 and 6, indicate that the secant moduli, over the appropriate range of shear stresses, are 480 ksf and 1360 psf respectively. The shear stress profile under the abutment due to the weight of the surcharge has been determined on drawing 8 by approximate methods.

The lower value obtained for the sample at the 20 feet depth is consistent with values obtained in the previously noted Dingman Creek Road overpass settlement study. In testing the sample from 50 feet, the first cycle of loading had to be terminated, due to a fault in the machine. The higher value of 1360 psf was determined from the second cycle of loading, and therefore may be too high on account of this hysteresis effect. For purposes of calculation, therefore, the lower value of 480 ksf is used. The value of E equal to 1360 psf has been determined for the much lower shear stress at a depth of 10 feet.

The value of the "elastic" modulus for the silt stratum is determined from a semi-empirical relationship with the penetration resistance of the soil*. For an average N value of 48, the modulus is estimated as 550 ksf.

The immediate settlement calculations are presented on drawing 9.

3) Consolidation Settlement

Consolidation, or recompression settlements, due to the expulsion of pore water, will be confined to the strata of silty clay or clayey silt.

The magnitude of this movement, μS_c, can be determined from the expression:

$$\mu S_c = \sum_1 \mu . m_v . \Delta p . \Delta H$$

* Imperial College Notes - 1959-60

where: μ has been previously defined,

m_v is the modulus of compressibility, as discussed below,

and Δp is the average increment of vertical stress in any depth ΔH, as discussed in the previous section on immediate settlements.

The value of the coefficient, μ , partially depends upon the pore pressure coefficient A of the clay, which was determined approximately as 0.14 in the two k_o consolidated undrained triaxial tests performed in this study. If a conservative value of A equal to 0.2 is assumed, as in the previous study, similar values of μ can be used for both projects. These values are summarised on drawing 10.

Laboratory consolidation tests were performed on samples from 20, 41 and 80 feet. The results of these tests, presented on drawings 3 and 4, show the values of the compressibility coefficients as determined over the range of stresses equivalent to the applied surcharge. In view of the apparent disturbance to the sample from 80 feet, the value of m_v for this depth has been determined from the consolidation curve for the sample from 41 feet. This procedure, used in the Dingman Creek Road overpass study, is considered to be permissible in view of the similar initial void ratio and consolidation history of both samples.

The coefficients of compressibility determined in this manner are summarised on drawing 10, and appear to follow a reasonably consistent trend. It should be noted that the Schmertmann correction* to the consolidation curves for sample disturbance has not been applied. A preliminary study of a corrected curve for the sample from 20 feet showed that little decrease in the value of m_v is obtained. Accordingly, it was decided that this correction was unimportant for the stress range applicable to this project.

In the report on the settlement study for the Dingman Creek Road Overpass, it was argued that, because of negative pore pressures existing in the desiccated surface soil above the water table, this hard soil is subject to high effective stresses at the present time. It is considered, therefore, that the addition of the surcharge will cause little consolidation. Hence, it was assumed that recompression, as calculated in the normal manner, would only occur in the lower half of the zone of soil above the water table.

* "Estimating The True Consolidation Behaviour of Clay From Laboratory Test Results" - Schmertmann, ASCE Separate No. 311, October, 1953

Applying the above assumption to this project, it is seen that zero reconsolidation takes place in the upper 15 feet of soil. The neglect of this appreciable depth of soil may result in an under-estimation of the consolidation movement, but it is considered that any error will be small.

The computations of consolidation settlement are presented on drawing 10.

4) Summary of Computations

Settlements have been computed for points located below the centre of the west abutment, and the centre of the adjacent pier respectively. It is understood that these locations will be instrumented.

Settlements of the footings due to the dead weight of the bridge have been neglected. Any movement from this source will be confined to the upper layer of desiccated soil and will be small.

Computations of immediate and consolidation settlements are presented on drawings 9 and 10, respectively. The calculated movements are summarized below.

	ABUTMENT	PIER
Immediate Settlement	1.38	0.49
Consolidation Settlement	1.33	0.46
	2.71 ins.	0.95 ins.

The maximum differential settlement is therefore computed as 1.76 inches.

5) Discussion of Results

As discussed in the previous study, the accuracy of the computed settlements are mainly dependent upon the correct selection of the appropriate soil properties. Also, in this project, the assumption of zero recompression above a depth of 15 feet is open to question. The error involved in this latter assumption is believed to be small. It is reasoned, however, that it is most unlikely that values of $1/m_v$ and E are too high, since sample disturbance should always give conservatively low values of these coefficients. Hence, the computed settlements probably overestimate the actual movement.

The differential immediate settlement of 0.89 inches or 51 percent of the total relative movement between piers and abutment will occur immediately upon application of the surcharge load. The remaining 49 percent, or 0.87 inches of differential settlement will occur over a period of time.

Computations of the rate of consolidation are presented on drawing 11. The results of these calculations show that the above differential consolidation settlement of 0.87 inches will be essentially complete after about 5 years, with about 33 percent of the movement complete by 6 months. As mentioned in the associated study on the Dingman Creek Road overpass, these estimates of consolidation rate can be considered as a guide only, and actual movement will probably occur much faster.

The readings from the proposed settlement plate and piezometer instrumentation at this site will give a much more reliable guide to the rate of consolidation movement.

It is noted, however, that even if the superstructure is constructed immediately after placement of the fill, the remaining differential movement of 3/4 to 7/8 inches represents an angular displacement of only about 1/600 of the span. If the bridge girders are designed to accommodate this low angular movement, it should be possible to dispense with any form of jacking arrangement at the abutments.

It should be noted that a much lower differential movement and hence angular displacement will occur at the east abutment where the surcharge weight is considerably less.

Permissible Bearing Pressure

In the previously noted initial report prepared for this site, a bearing pressure of 2 tsf at a depth of 10 feet, or El 862 feet was recommended for the design of the bridge footings. This permissible stress was determined on the basis of an average undrained shear strength in the clay of 1700 psf, as measured in unconfined compression tests.

The results of the single recent boring appear to indicate that footings could be conveniently founded at a higher level if desired. Competent soil exists at this borehole location at a depth of about 4 feet, or El 571.5 feet. The hard desiccated material below this level appears to extend to at least a depth of 10 to 15 feet. The average penetration resistance in this zone was measured as over 30 blows per foot. A similar value was measured in hole 3 of the previous work, although a somewhat lower value of 20 was evident in hole 1. At this latter borehole location, an estimated 10 to 14 feet of the original desiccated soil has been removed during construction of Highway 401. Hence somewhat lower strengths and penetration resistance can be expected in this area. However, the presence of the lower levels of a desiccated soil zone at these boring locations is evident from the natural moisture content and Atterberg limit determinations presented in the initial report.

It is considered that the above noted shear strength of 1700 psf is not consistent with this apparent desiccated condition. This low strength measurement could result from the unconfined method of testing, in which there was no confining pressure to prevent premature failure of the soil by "bursting". Accordingly, one triaxial compression test was performed on a sample from a depth of 8 feet in the recent borehole. The result of this test, presented on drawing 12, shows a strength in excess of 8000 psf, which is consistent with the apparent desiccated nature of the soil.

This extremely high strength certainly does not apply to the lower ground just east of the Highway, where the original desiccated crustal soil has been removed. However, it would appear entirely reasonable to assign a strength of 3000 psf to the zone of stressed soil under footings founded at El 866 feet or higher. This value is slightly in excess of the strength determined at a depth of 20 feet in the recent boring. It is believed, in fact, that, if additional samples were available for testing, a strength of at least 4000 psf could be assigned to the soil below anticipated footing level.

The safe bearing pressure, q, for use in design of the footings is determined from the expression:

$$q = cN_c/F$$

where: c is the undrained shear strength, assumed equal to 3000 psf as noted above,

N_c is a coefficient; approximately equal to 6 for shallow footings,

and F is the required factor of safety; assume equal to 3.

Substituting these values, a net safe bearing capacity of 6000 psf is computed. This permissible stress is believed to be quite conservative at the west abutment location, but is probably more truly applicable near the borehole 1 location of the original investigation.

These remarks concerning the allowable bearing pressure of the soil are presented since it is believed that the competent nature of the soil encountered during this recent work, is not consistent with the original recommendation of the relatively low bearing value of 2 tsf.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
P. G. M. IMRIE
PROVINCE OF ONTARIO

PGMI/gc
J889
July, 1962

P.G.M. Imrie, P.Eng.

APPENDIX A

Field and Laboratory Investigation Methods

The one boring at this site was put down using continuous flight auger equipment. The hole was 5 inches in diameter, and was uncased. Sampling at 5 feet intervals was continued to 70 feet, and at 10 feet intervals thereafter.

Disturbed samples of the soil were recovered by driving a 2 inch O.D. split spoon into the soil ahead of the boring. The number of 350 ft. lbs. hammer blows required to extend the penetration of the spoon from 6 to 18 inches was recorded as the penetration resistance of the soil. Upon withdrawal of the spoon, the soil was identified, and retained in a moisture proof plastic bag. In addition, a portion of any clay sample was wrapped in tinfoil to prevent any drying out.

Undisturbed samples were recovered by pressing or driving 2 inch or 3 inch I.D. shelby tube samplers into the soil ahead of the boring. Upon recovery, the tubes were sealed for transportation to the laboratory after preliminary identification of the soil.

An open standpipe piezometer was also installed to measure the ground water level. A 1½ inch diameter, 18 inch long porous plastic piezometer tip with ½ inch O.D. polyethylene tubing was used. Initially, a hole was drilled to a depth of 39 feet. About 6 inches of tamped granular material was placed in the boring and the piezometer lowered into the hole. This was surrounded and covered by about 2 feet of tamped granular soil. Slightly plastic bentonite balls were then placed in the hole and tamped to form a seal about 2½ feet thick. This was followed by another 3 feet of tamped plastic clay originally recovered from the boring. The hole was then filled to the surface and the tubing surrounded by a steel pipe with a screw cap to prevent rain entering the standpipe. A small hole in the cap permitted the escape of air as the water rose in the tubing.

As noted in the report, the water level stabilised almost immediately in the standpipe. Subsequent readings taken by D.H.O. personnel after an elapsed time of about 2 weeks, confirmed the stabilised level.

In the laboratory, consolidation tests were performed on three samples from selected depths. Particular care was taken in the preparation of these samples to avoid disturbance. If small stones at the edge or ends of any sample prevented accurate trimming, that specimen was discarded and the consolidometer ring jacked into another block of soil.

During the test, the pressure increment ratio was about 50 percent over the range of stresses applicable to the project. It is believed that this procedure more accurately defines the consolidation curve.

Two K_o consolidated undrained triaxial tests with pore pressure measurement were performed on samples from 20 and 50 feet. The consolidation stage of this test was performed under conditions approximating zero lateral strain by adjusting the lateral or axial consolidation load so as to maintain the correct relationship between the volume of water expelled and the axial strain. The vertical consolidation pressure should have equalled the overburden pressure, although in practice it was not possible to accurately reach and maintain this value without excessive adjustments to the consolidation pressures. At the completion of the consolidation stage it was noted that the vertical consolidation pressure was somewhat less than the existing overburden pressure in both tests.

Before commencing the second stage of the test, a back pore pressure of 15 psi was applied to the sample to dissolve any air trapped between membrane and sample. The test was then performed at a strain rate of 0.017%/min., which rate is believed to be slow enough to permit equalization of pore pressures throughout the sample.

Natural moisture and Atterberg Limits determinations, and grain size distributions analyses were also performed on selected samples.

The record of these field and laboratory measurements are recorded on drawings 2 to 7 of this report.

Department of Highways of Ontario personnel from London assisted in setting out the centreline of the proposed overpass. The elevation of the borehole was referred to a nail and washer in the root of a maple tree 215 feet left of station 494 + 63 on Highway 401. The level to this bench mark is taken as El 862.73 feet.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 2
PROJECT NO. J889

LEGEND

BOREHOLE NO. 1
PROJECT Proposed Overpass, Bostwick Road over Hwy. 401
LOCATION District No. 2, London
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 875.5 ft.
DATUM See Dwg. 1.

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

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

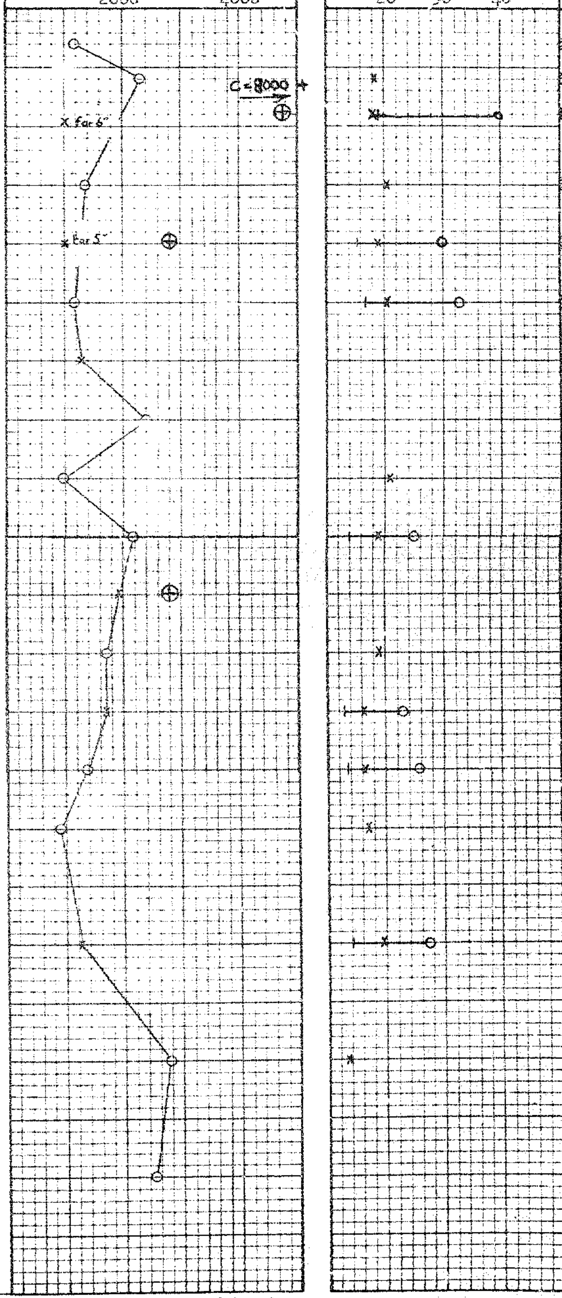
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
Ground Surface	875.5	0		
~ 6 ins. topsoil				
SILTY CLAY-red brown, oxidized and hard above ~ 8 ft., grey and very stiff below 8 ft.			1	
			2	
-occasional fine gravel sizes throughout, becoming more frequent with depth		10	3 P D	136 = 8" = 6"
			4	
		20	5P D	137 = 11" = 5"
-relatively clear silty clay below ~ 24 ft.			6	
	846.5	30	7D	
	843.5			
SAND-stratified dense med. sand, -changing to silty fine sand, and sandy silt with depth, wet			8	
	838			
-very stiff silty clay to ~ 43 ft.		40	9 9a	134 P=12"
CLAYEY SILT-very stiff to hard -silt content increasing with depth, becoming slightly clayey cohesive silt of low plasticity			10	= 12"
-odd irregular intrusions of clay		50	11D	134
-some fine to medium gravel sizes throughout, slightly more frequent with depth.			12	
-4 in. thick fine sand seams at 50 ft. and 60 ft.		60	13D	
			14	
-gravel sizes very infrequent below ~ 68 ft.		70	15	
		80	16D	
-very stiff, varved clay at 89 ft.	786.2	90	17	
-very stiff to hard clayey silt glacial till with frequent fine to coarse gravel sizes.				
End of Hole	775.0	100	18	
		110		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. — SHEAR STRENGTH P.S.F. — NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

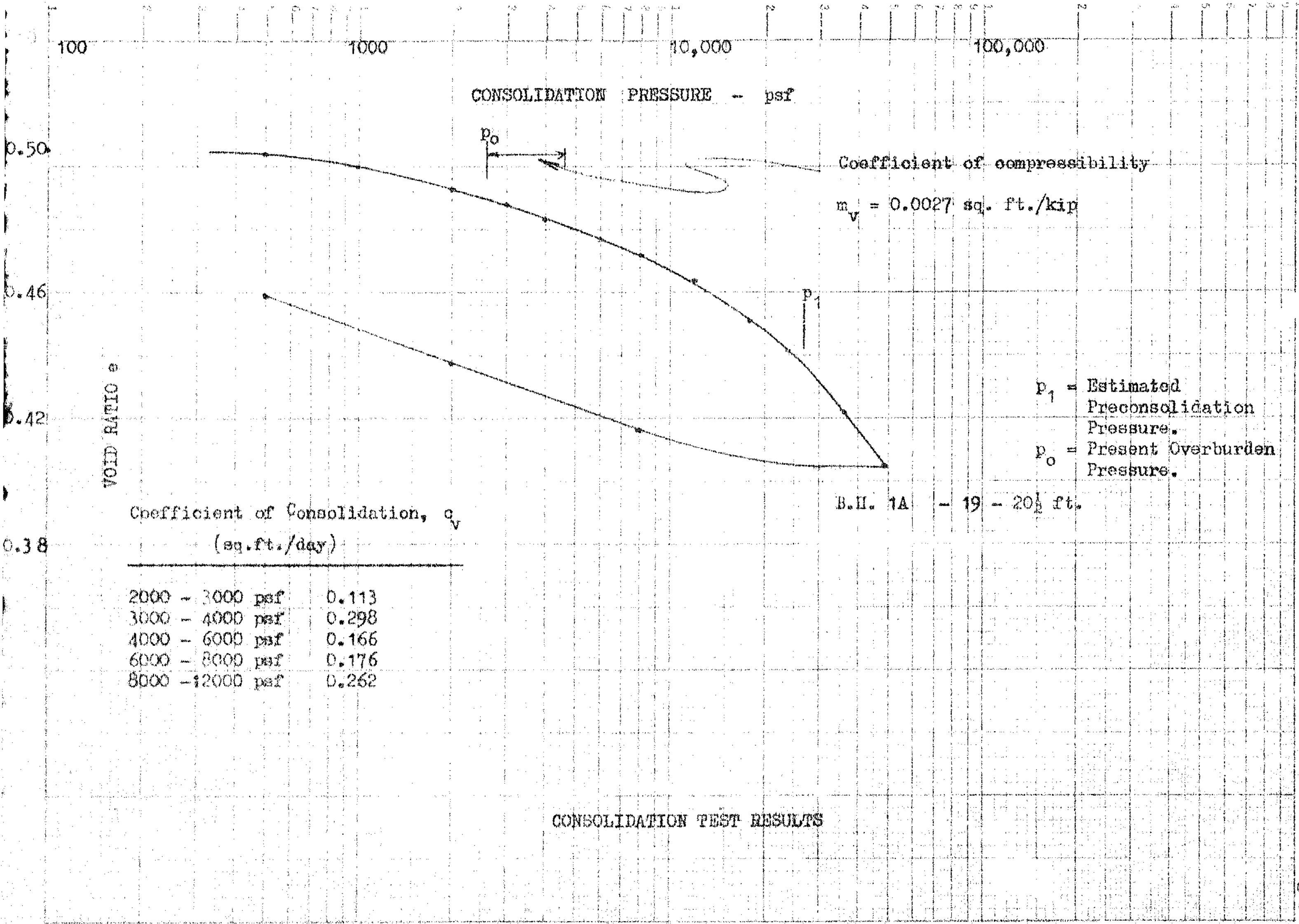


D = Driven
P = Pushed

Notes:
1) Hole augered & uncased to full depth.
2) Piezometer installed 23 ft. north of hole. Water stabilised almost immediately at 29.3 ft. below ground surface.
3) Water stabilised in borehole at completion at 29.2 ft. below ground surface.
4) Shear strengthd at 20 & 41 ft. determined from k_o consolidated undrained tests.

K&E SEMI-LOGARITHMIC 359-81G

J889



Coefficient of Consolidation, c_v (sq.ft./day)

2000 – 3000 psf	0.113
3000 – 4000 psf	0.298
4000 – 6000 psf	0.166
6000 – 8000 psf	0.176
8000 – 12000 psf	0.262

CONSOLIDATION TEST RESULTS

Dwg.3

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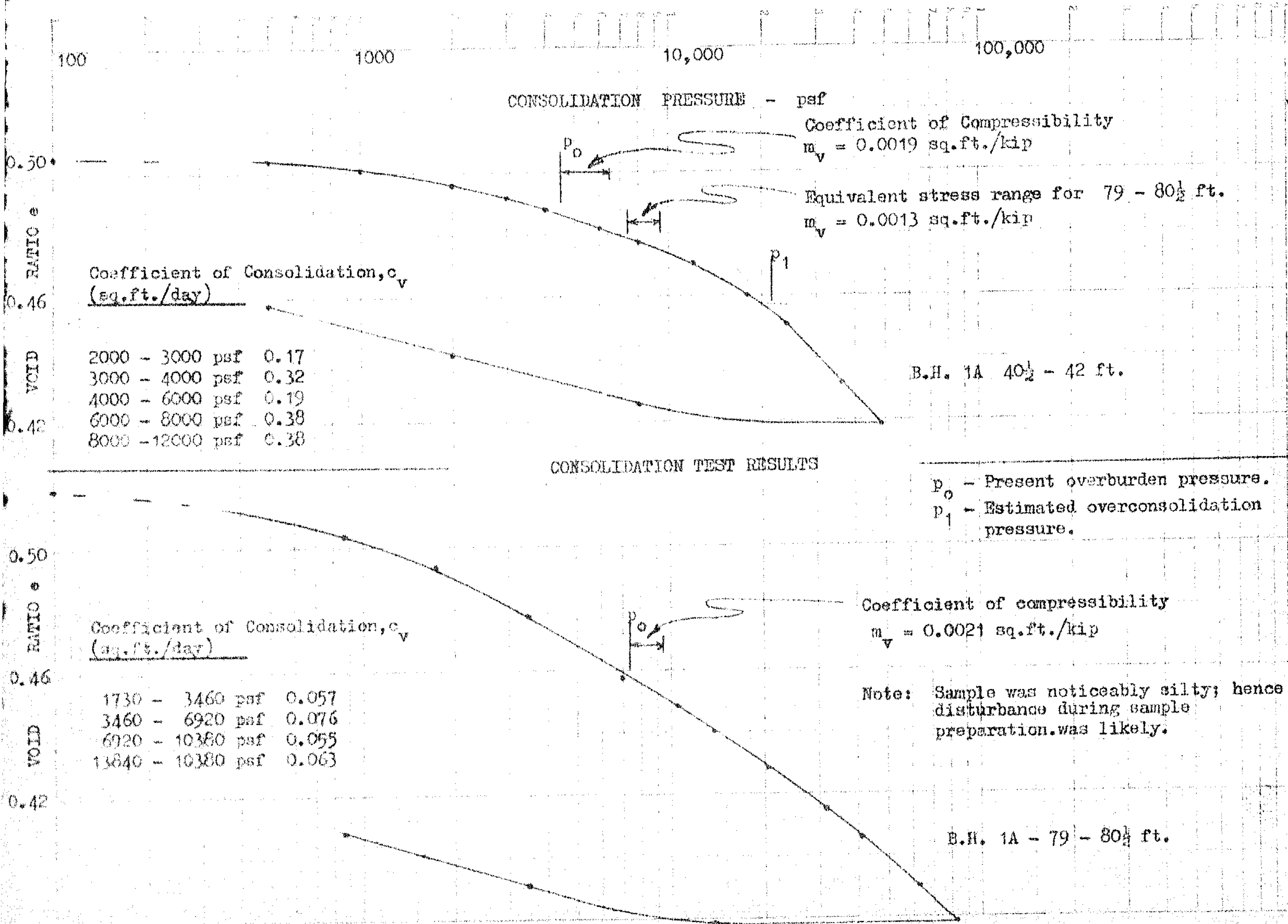

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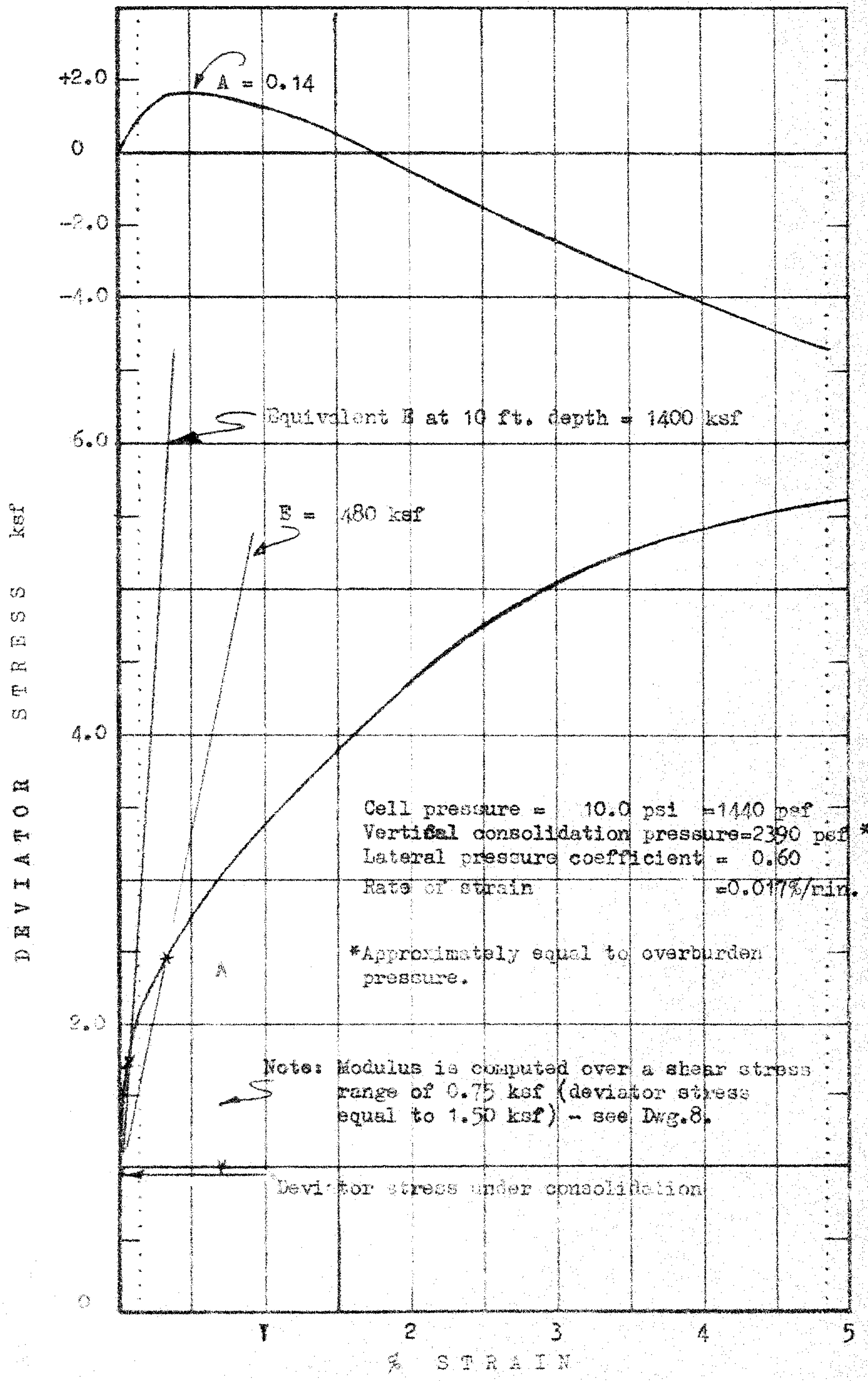
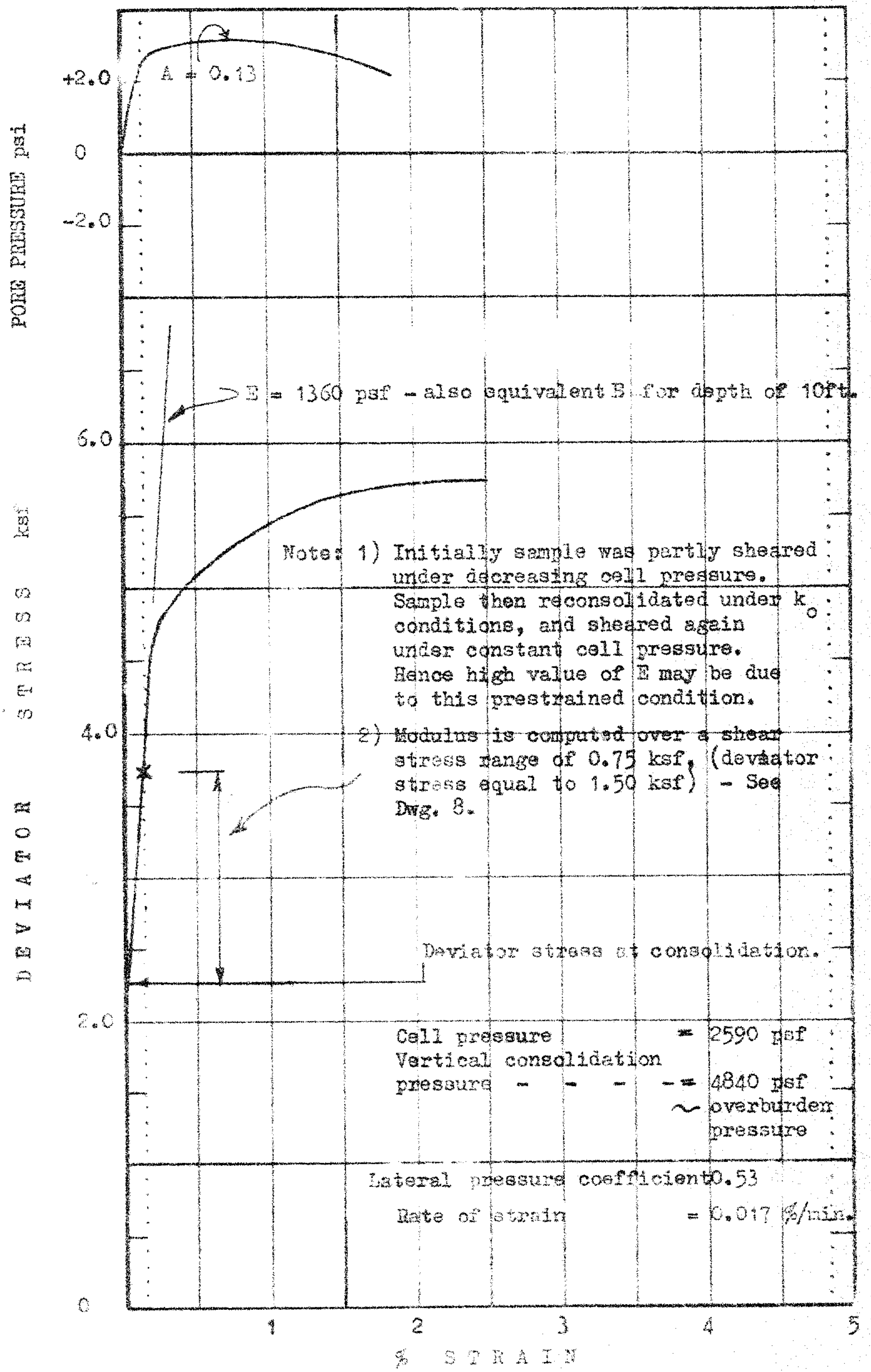


STRESS-STRAIN CURVE FOR k_o

CONSOLIDATED UNDRAINED TRIAXIAL TEST

BH 1A 19-20½ ft.



STRESS-STRAIN CURVES FOR k_o CONSOLIDATED UNDRAINED TRIAXIAL TEST

BH 1A, 49 - 50½ ft.

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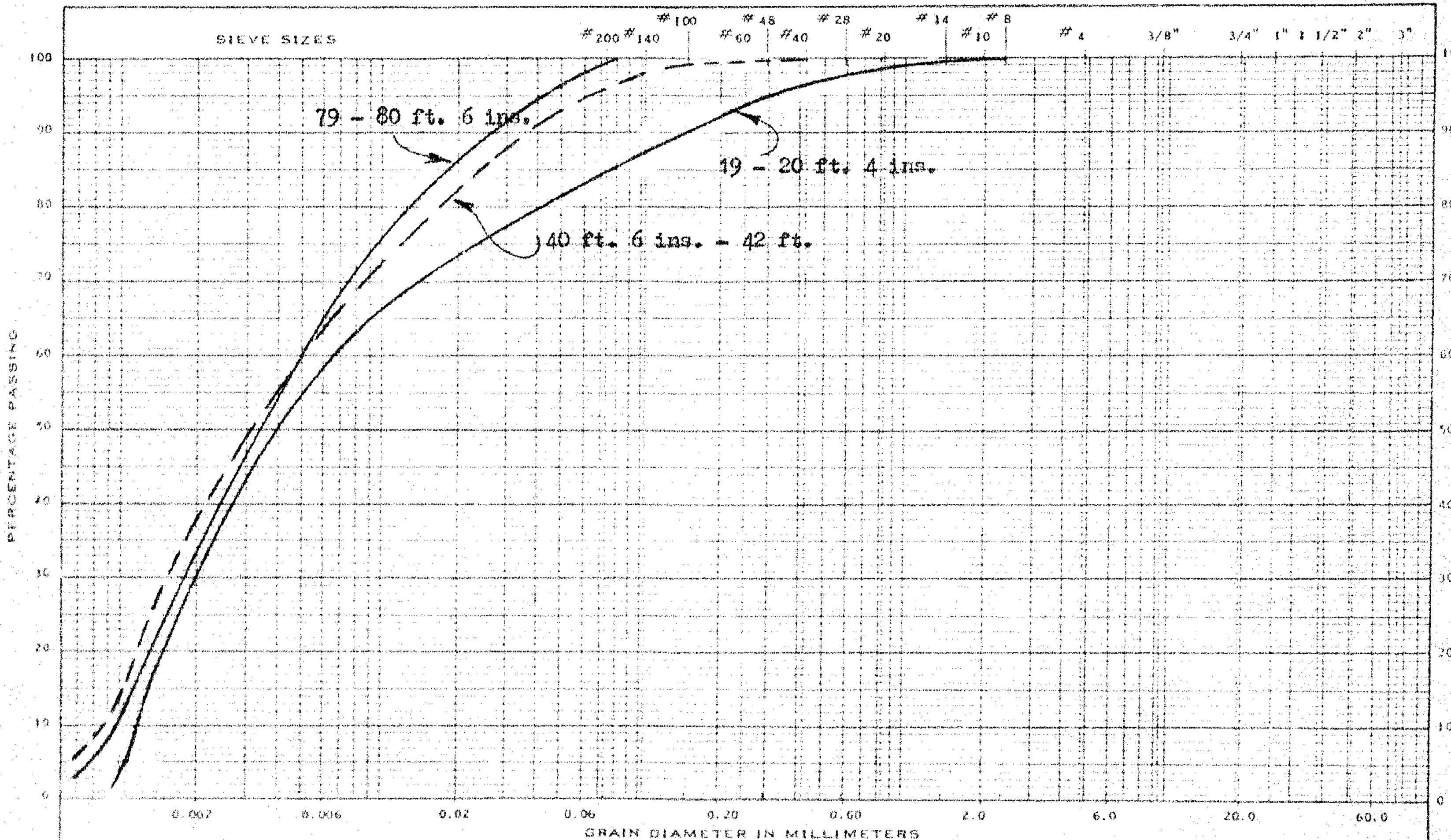
MECHANICAL ANALYSIS

Project: J889



← CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION

GRAIN SIZE DISTRIBUTION CURVES

BOSTWICK ROAD - BH 1A

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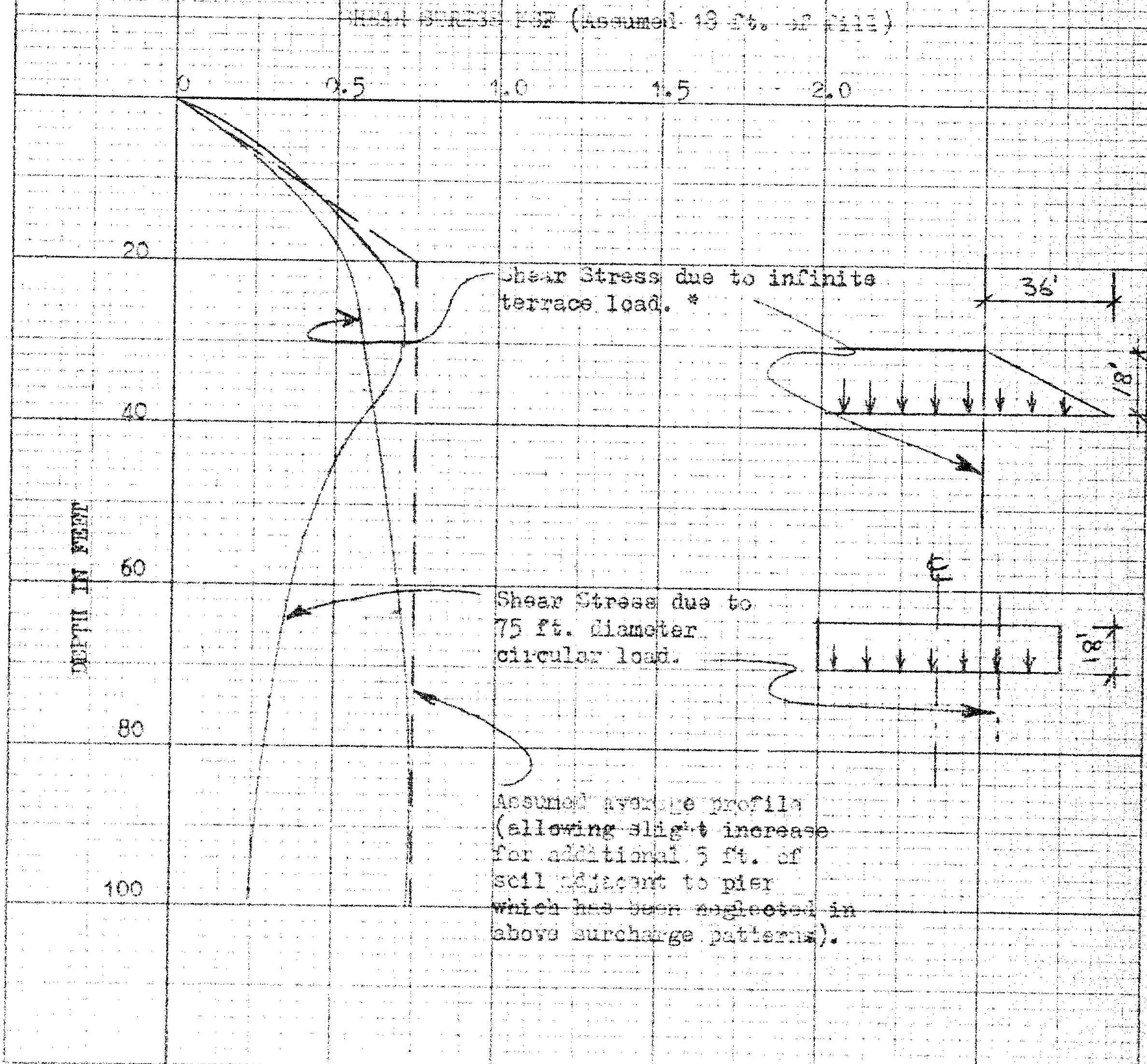
Dwg. 7



*"Theories of Elasticity and Plasticity" - L. Jurgenson
Journal of Boston Soc. of C.E. 1934

SHEAR STRESS PROFILE
-under CL Abutment

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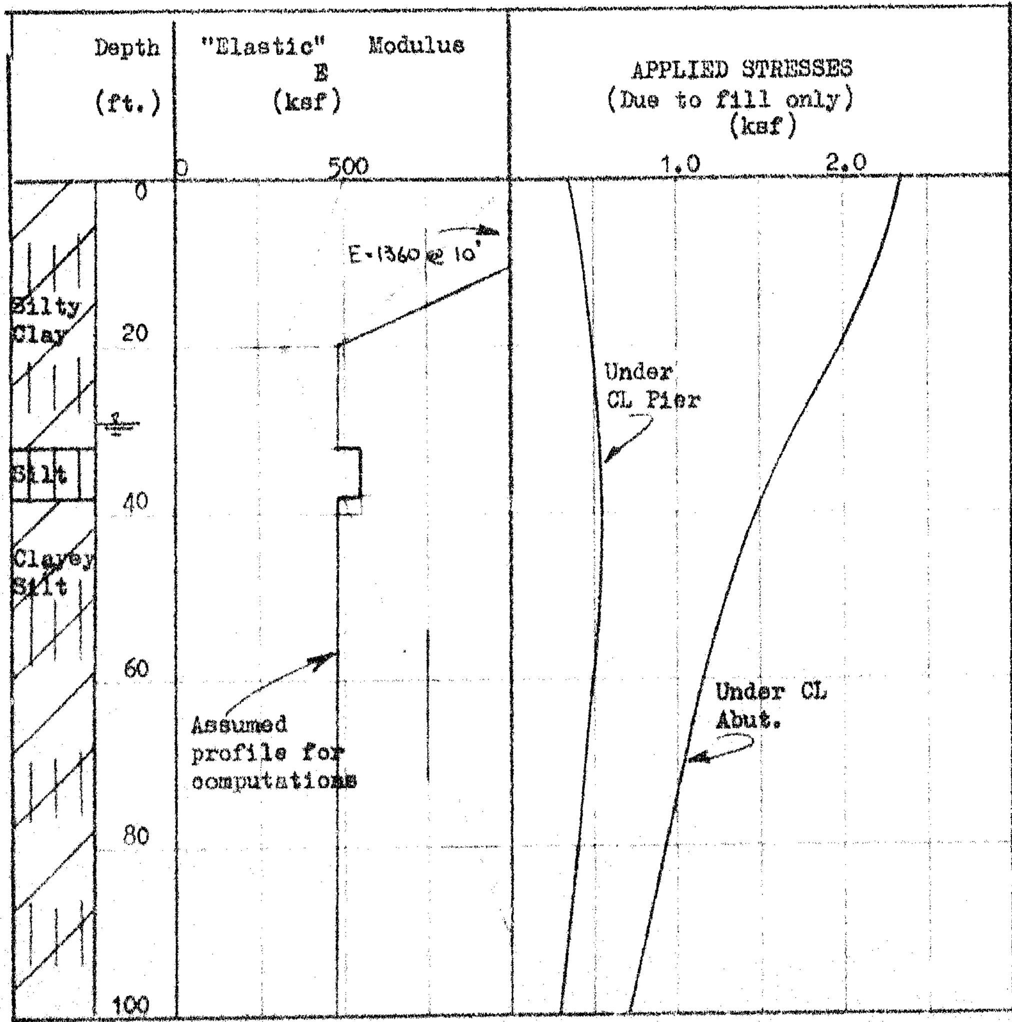
Project: J889



$s_i = \sum 0.5 \cdot \Delta p \cdot \Delta H/E$	
Under CL Abut. (ins.)	Under CL Pier (ins.)
0.5x2.20x240/1360 = 0.19	0.5x0.42x240/1360 = 0.04
0.5x1.72x168/480 = 0.30	0.5x0.53x168/480 = 0.09
0.5x1.60x72/550 = 0.10	0.5x0.54x72/550 = 0.04
0.5x1.30x240/480 = 0.32	0.5x0.51x240/480 = 0.13
0.5x1.03x240/480 = 0.26	0.5x0.44x240/480 = 0.11
0.5x0.82x240/480 = 0.21	0.5x0.34x240/480 = 0.08
1.38 ins.	0.49"

IMMEDIATE SETTLEMENT COMPUTATIONS
-under CL Abutment and CL Pier

Dwg. 9

Depth (ft.)	Coefficient of Compressibility m_v (sq.ft./kip) 0 — 0.005	μ – Assumed Values	Applied Stresses (due to fill only) (ksf) 0 — 1.0 — 2.0	$S_c = \sum \mu . m_v . \Delta p . \Delta H$ Under CL Abut. (ins.)	Under CL pier (ins.)
Silty Clay	Assume incompressible	0.65	Under CL pier	0.65x0.0024x2.05x60 = 0.19	0.65x0.0024x0.46x60 = 0.04
Silt		0.70		0.70x0.0022x1.72x168 = 0.45	0.70x0.0022x0.53x168 = 0.14
Clayey Silt		0.65		0.65x0.0018x1.30x240 = 0.36	0.65x0.0018x0.52x240 = 0.14
	Assume profile for computations	0.6	Under CL Abut.	0.60x0.0015x1.03x240 = 0.22	0.60x0.0015x0.44x240 = 0.10
		0.5		0.50x0.0011x0.82x240 = 0.11	0.50x0.0011x0.34x240 = 0.04
				1.33 ins.	0.46 ins.

CONSOLIDATION SETTLEMENT COMPUTATIONS

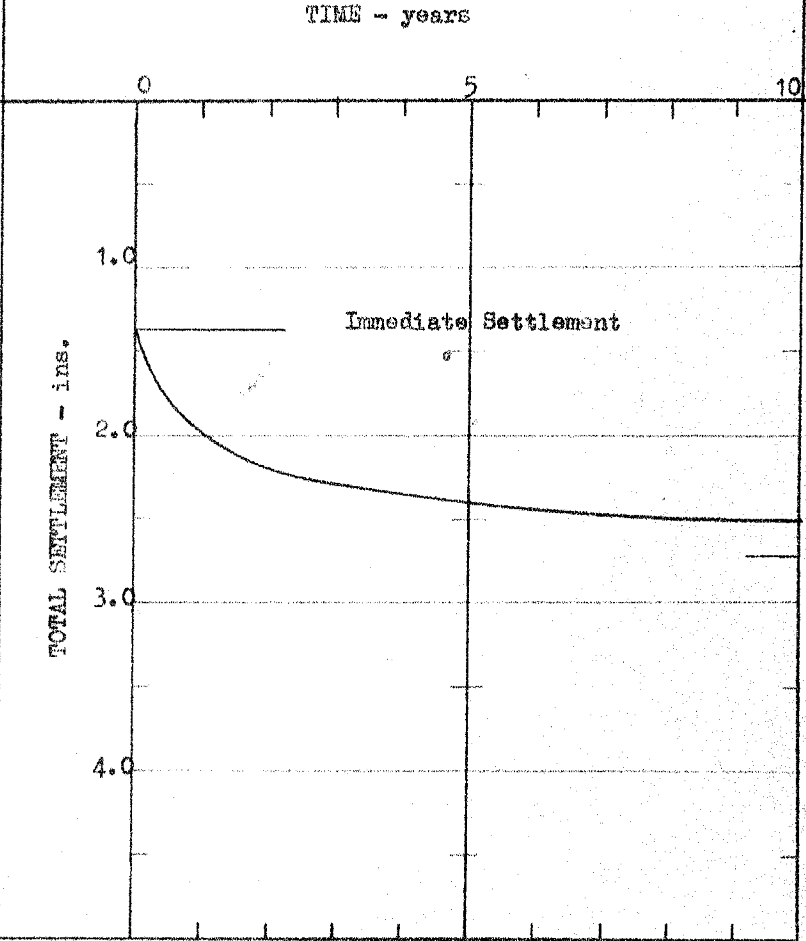
– under CL of Abutments and Piers

Project: J889

Dwg. 10

Project: J889

Depth (ft.)	Drainage Thickness (ft.)	Coeffic. of Consol. c_v (sq.ft./day)	For x=50% t = 0.197	For x=90% t = 0.84
0	2H=32' H=16'	0.2	$t_{50} = \frac{0.197 \times 16^2}{0.2}$ =252 days for 0.5x0.64 =0.32 ins.	$t_{90} = \frac{0.84 \times 16^2}{0.2}$ = 2.9 yrs. for 0.9x0.64 = 0.58 ins.
40 – 60	2H=22'	0.2	$t_{50} = \frac{0.197 \times 11^2}{0.2}$ =118 days for 0.5x0.36 =0.18 ins.	$t_{90} = \frac{0.84 \times 11^2}{0.2}$ = 1.40 yrs. for 0.9x0.36 = 0.32 ins.
80 – 100	H=40'	0.07	$t_{50} = \frac{0.197 \times 40^2}{0.07}$ =12.3 yrs. for 0.5x0.33 =0.16 ins.	$t_{90} = \frac{0.84 \times 40^2}{0.07}$ = 53 yrs. for 0.9x0.33 = 0.30 ins.



SETTLEMENT v. TIME COMPUTATIONS

—under CL Abutment

Dwg. 11

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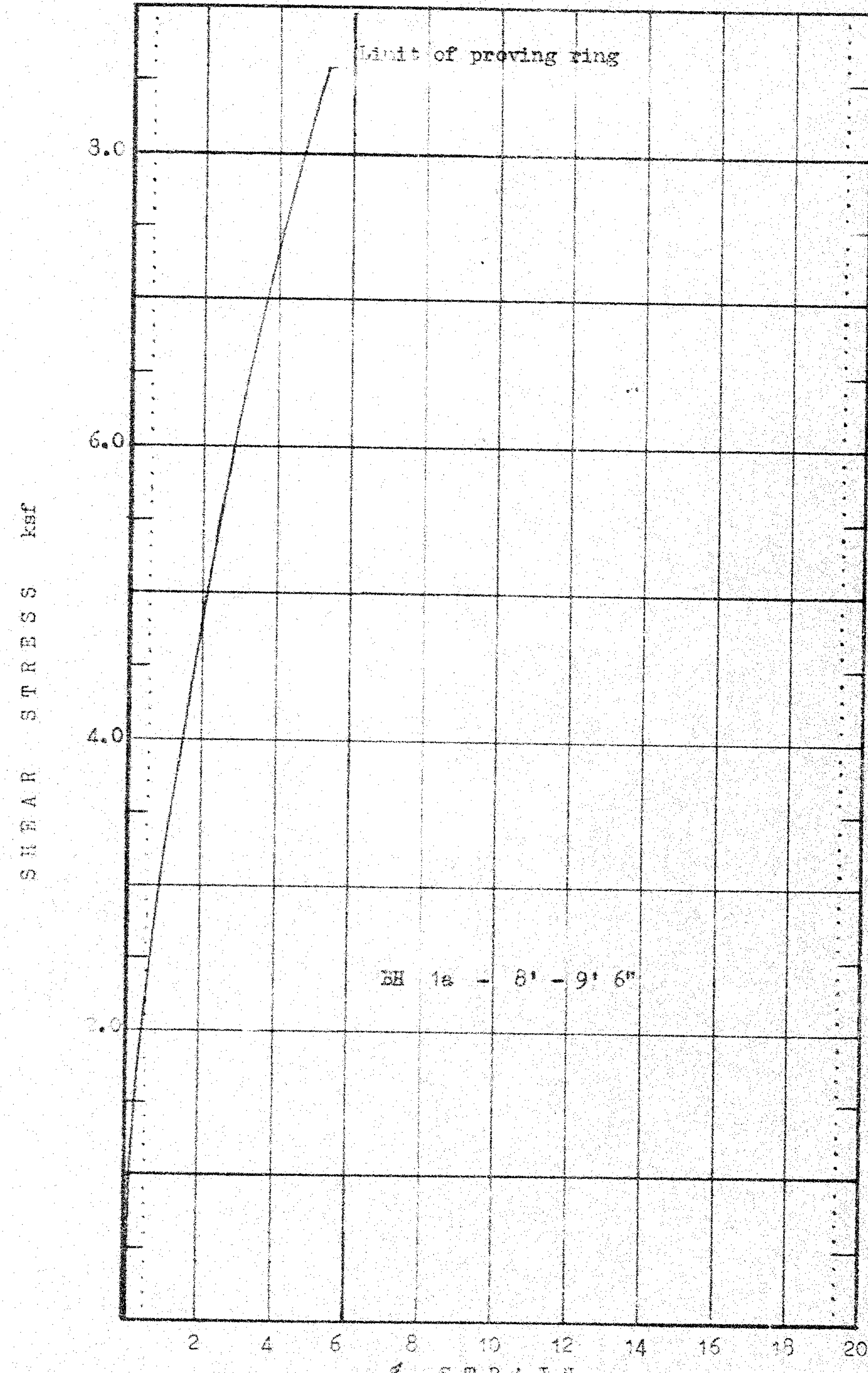


STRESS-STRAIN CURVE - UNDRAINED TRIAXIAL TEST

CELL PRESSURE - 10 psi

WILLIAM A. TROW AND ASSOCIATES

ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye, **Date** Aug. 3, 1962.
Bridge Engineer. **Subject** FOUNDATION INVESTIGATION REPO
From Materials & Research Division. by: Wm. A. Trow & Associates Ltd.

(Foundation Section)

Attention: Mr. S. McCombie.

Re: Settlement Study
Bostwick Road Overpass
W.P. 98-57, Highway 401
District #2.

Attached we are forwarding to you the report for the above-mentioned foundation investigation submitted by William A. Trow & Associates Ltd.

We have reviewed the report, and on the basis of the presented factual data and information, we agree with the conclusions and recommendations contained therein.

We believe that the given recommendations will prove to be adequate for your future design work. However, should there be any other questions in connection with the above project that you would like to discuss, please feel free to call on our Office.

KYLo
K. Y. Lo
Supervising Foundation Engr.

For: A. G. Stermac
Principal Foundation Engineer.

KYL/tt
Attach.
cc: Messrs. A. M. Toye (2) ✓
H. A. Tregaskes
H. D. McMillan
A. Gater
W. L. Fraser
T. J. Kovich
J. R. Roy
J. E. Gruspier
E. R. Saint
F. Norman
A. Watt
Foundations Office, Gen. Files

40I 14-85

ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye,
Bridge Engineer.

Date August 15, 1962.

Subject ABUTMENT FOUNDATIONS --

From Materials & Research Division,
(Foundation Section)

Attention: Mr. C. Bassi.

Re: W.P. 98-57,
Bostwick Rd. ~~Overpass~~, (UNDERPASS)
Hwy. #401, District #2,
London.

With regard to your query concerning the abutment foundations for the above proposed structure, we submit the following comments:-

(1) The abutments may be supported on spread footings placed within the approach fills. The fill material below the tops of the footings should consist of well compacted G.B.C. Class 'A' material and should extend for a horizontal distance of at least 10' from the footing edges in the plane of the footing tops. This portion of the fill should be built with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for a distance of about 50' behind the abutments before re-excavating for the abutment footings. A design load of 2 t.s.f. may be used for the abutment foundations.

(2) If it is desired to support the abutments on piled foundations, 12¾" O.D. steel tubes driven to approx. el. 842.0 should provide a design load of 40 tons/pile. Because of the fine-grained nature of the subsoil, it is difficult to predict accurately the performance of piles driven into it. It would be preferable, therefore, to carry out a pile loading test in order to check the capacity of at least one pile.

If you have any further queries in connection with this matter, please contact this Office.

K. G. Selby

KGS/MdeF

K. G. Selby,
SENIOR FOUNDATION ENGR.

cc: Mr. B. Davis
Foundations Office
Gen. Files.

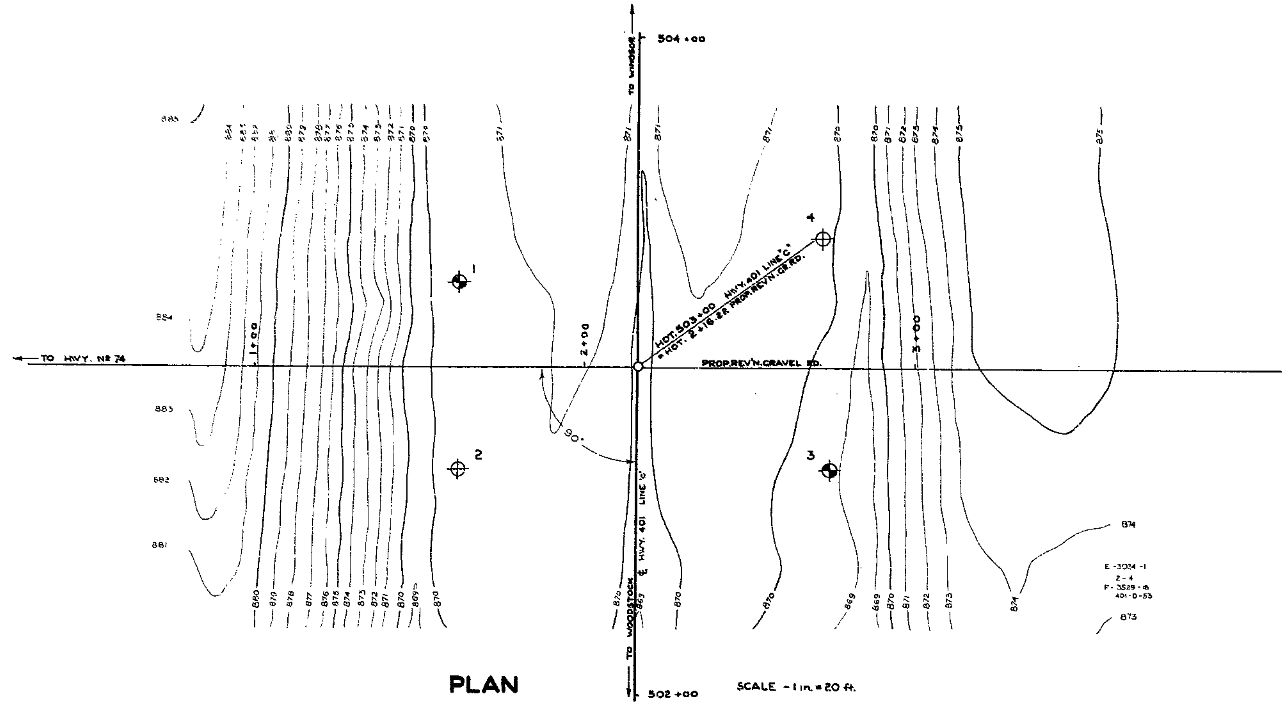
W.O. 57-F-47A

HWY. 401

LINE 'C'

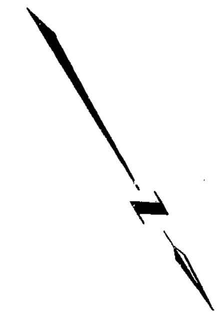
401 14-85

TRIM LINE



PLAN

SCALE - 1 in. = 20 ft.



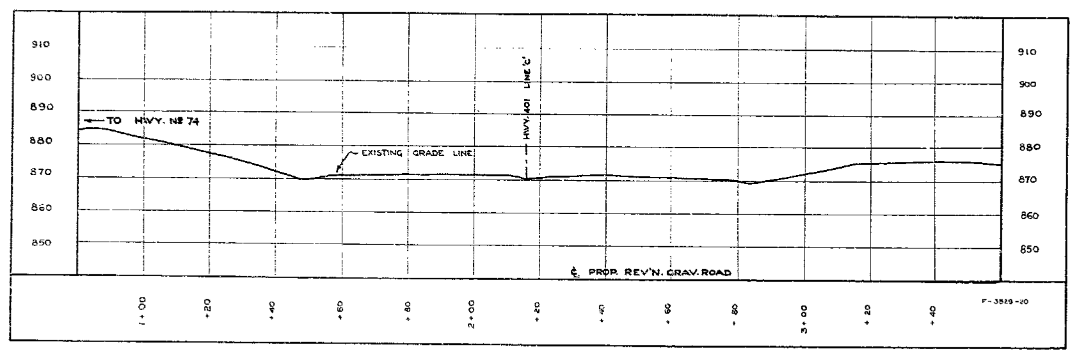
LEGEND

BORE HOLES

PENETRATION HOLES

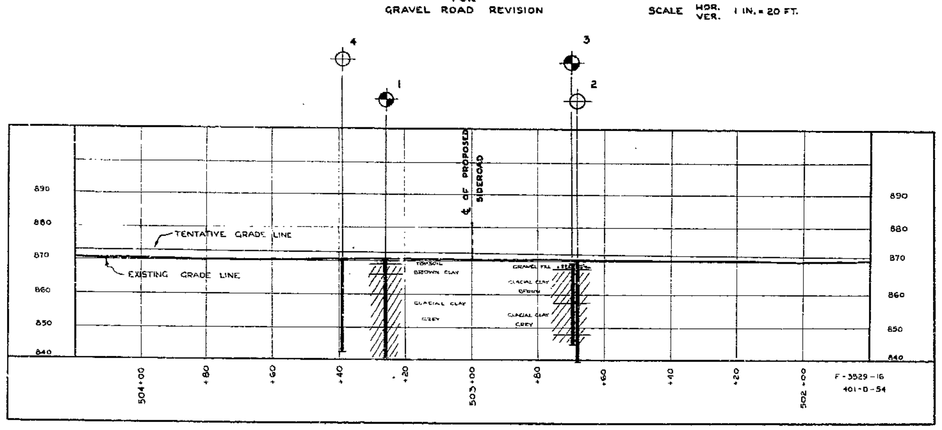
BORE & PENETRATION HOLES

HOLE NO.	ELEVATION	STATION	DISTANCE FROM EXIS. ℄
1	870.3'	503+26	54' LT.
2	870.0'	502+68.5	54' LT.
3	869.3'	502+68.5	58' RT.
4	870.1'	503+39	56' RT.



PROFILE
FOR
GRAVEL ROAD REVISION

SCALE HOR. VER. 1 IN. = 20 FT.



PROFILE
FOR
HWY. NO. 401 LINE 'C'

SCALE HOR. VER. 1 IN. = 20 FT.

40I12-??

GEOCRES No.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION - DOWNSVIEW

GRAVEL ROAD REVISION PROPOSED CROSSING 5 MILES S. OF LONDON

SHOWING POSITION & ELEVATION OF HOLES

HWY. Nº 401 LINE 'C'	W.P. 98-57	DIV. NO. 2
CO. MIDDLESEX		
TWP. WESTMINSTER	LOT. 27	CON. VI
SCALE: AS SHOWN	SUBMITTED BY	DATE JAN. 31. 1958
DRAWN BY O.M.Z.	APPROVED BY	DRAWING Nº F-57-47A

TRIM LINE

TRIM LINE

TL-129 (REV. 56) H.D.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

40I/4-085
GEOCRES No.

DRILL RIG 54-2 OPERATION BORE & PENET'N JOB F-57-47 W.P. 98-57 BORING 1 STA. 503+26 (54' LT.)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957.
SAMPLER HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY H.L. CHECKED BY A.L. DATE BORING 27 NOV. 1957.

ABBREVIATIONS
V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
W.L. - WATER LEVEL IN CASING
W.T. - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA. - CASING
γ - UNIT WEIGHT

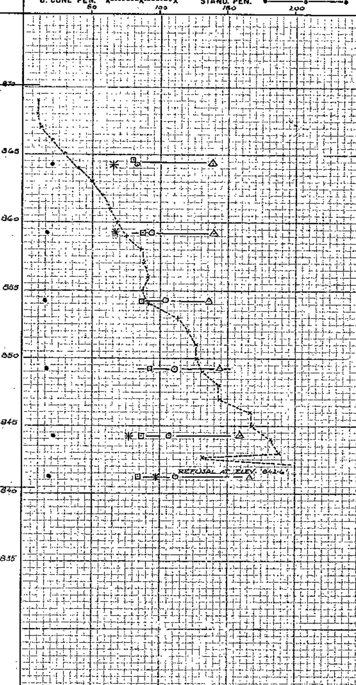
SAMPLE TYPES
C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

ELEVATION / DEPTH	WATER CONDITIONS	DESCRIPTION	ELEVATION SCALE
870.3'		GROUND LEVEL	870
869.3' / 1.0'	W.L.	TOPSOIL	
	W.L. AT ELEV. 869.2' ON DEC. 2/57.	BROWN CLAY	865
		GLACIAL CLAY GREY	860, 855, 850, 845
840.8' / 29.5'		END of BOREHOLE	840
			835

SHEAR STRENGTH IN LBS. PER SQ. FT. ✳ 1000 2000 3000 4000
WATER CONTENT W% O - NAT. □ - P.W. △ - L.W. 20 40 60
PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW)
D. CONE PEN. X-----X-----X STAND. PEN. ●——●——● 50 100 150 200



CASING BLOWS (ACTUAL): BX CASING SHOE; END OF CASING AT ELEV. 842.3'

SAMPLES

OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
					870.3'
γ = 140 p.c.f.		S.S.	1	20	865.3' 67
γ = 139 p.c.f.		S.S.	2	16	860.3' 67
γ = 136 p.c.f.		S.S.	3	15	855.3' 100
γ = 137 p.c.f.		S.S.	4	16	850.3' 56
γ = 134.2 p.c.f.		S.S.	5	22	845.3'
γ = 134.2 p.c.f.		S.S.	6	19	842.3' 78

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS

TRIM LINE

TL-129 (REV. 56) H.D.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

40 I 14-085

GEOCRES No.

DRILL RIG 54-2 OPERATION PENETRATION. JOB F-57-47 W.P. 98-57 BORING 2 STA. 602+685 (54' LT.)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957.

SAMPLER HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY H.L. CHECKED BY A.L. DATE BORING NOV. 26. 1957.

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL. - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA. - CASING
γ - UNIT WEIGHT

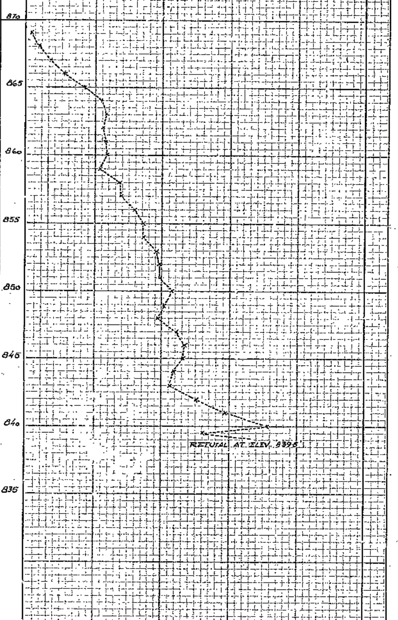
SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE							SAMPLES					
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W % — O - NAT. □ - P.W. △ - L.W. / PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. •—•—• 50 100 150 200	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
870		GROUND LEVEL		870								
5				865								
10				860								
15				855								
20				850								
25				845								
30				840								
35				835								



SOME DEFECTS IN NEGATIVE DUE

TO CONDITION OF ORIGINAL DOCUMENTS

TRIM LINE

TL- 129 (REV. 56) H.D.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

40I14-085
GEOCRES No.

DRILL RIG 54-2 OPERATION BORE & PENET'N JOB F-57-47 W.P. 98-57 BORING 3 STA. 502+68.5 (59' RT)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957
SAMPLER HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 5 DEC. 1957

ABBREVIATIONS
V - INSITU VANE SHEAR TEST; Q - TRIAXIAL QUICK; K - PERMIABILITY
M - MECHANICAL ANALYSIS; S - TRIAXIAL SLOW; C - CONSOLIDATION
U - UNCONFINED COMPRESSION; WL. - WATER LEVEL IN CASING; CA. - CASING
Q_C - TRIAXIAL CONSOLIDATED QUICK; W.T. - WATER TABLE IN SOIL; γ - UNIT WEIGHT

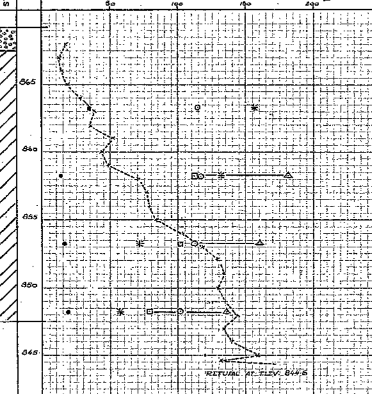
SAMPLE TYPES
C.S. - CHUNK; S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN; P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE; W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN; R.C. - ROCK CORE

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT
869.3'		GROUND LEVEL	
867.8' 1.5'		GRAVEL (FILL)	
		GLACIAL CLAY BROWN	
		GLACIAL CLAY GREY	
847.8' 21.5'		END of BOREHOLE	

SHEAR STRENGTH IN LBS. PER SQ. FT. * (1000, 2000, 3000, 4000)
WATER CONTENT W% (10, 20, 30, 40) O - NAT. □ - P.W. Δ - L.W.
PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW)
D. CONE PEN. X-----X-----X STAND. PEN. •——•——• (50, 100, 150, 200)



CASING BLOWS (ACTUAL): BX CASING JUICE; END of CASING AT ELEV. 849.3'

SAMPLES

OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
					869.3'
γ=130.6 p.c.f.	GOOD	S.S.	1	35	864.3' 100
γ=134.3 p.c.f.	GOOD	S.S.	2	14	859.3' 78
γ=136.2 p.c.f.	GOOD	S.S.	3	16	854.3' 50
γ=138.0 p.c.f.	GOOD	S.S.	4	19	849.3' 61

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS

TRIM LINE

TL-129 (REV. 56) H.D.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

40I14-085
GEOCRES No.

DRILL RIG 54-2 OPERATION PENETRATION JOB F-57-47 W.P. 98-57 BORING 4 STA. 503+39 (56'RT.)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957.

SAMPLER HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY H.L. CHECKED BY A.L. DATE BORING 4 DEC. 1957.

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
N - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
W.L. - WATER LEVEL IN CASING
W.T. - WATER TABLE IN SOIL
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SAMPLE TYPES

C.S. - CHUNK
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T.O. - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE CONDITION

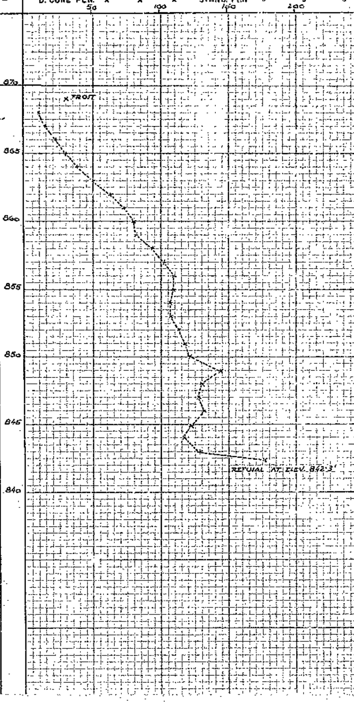
- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE						SAMPLES					
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
870.1'		GROUND LEVEL		870							

WATER CONTENT W% — O - NAT. □ - P.W. △ - L.W.

PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW)

D. CONE PEN. X-----X-----X STAND. PEN o——o——o

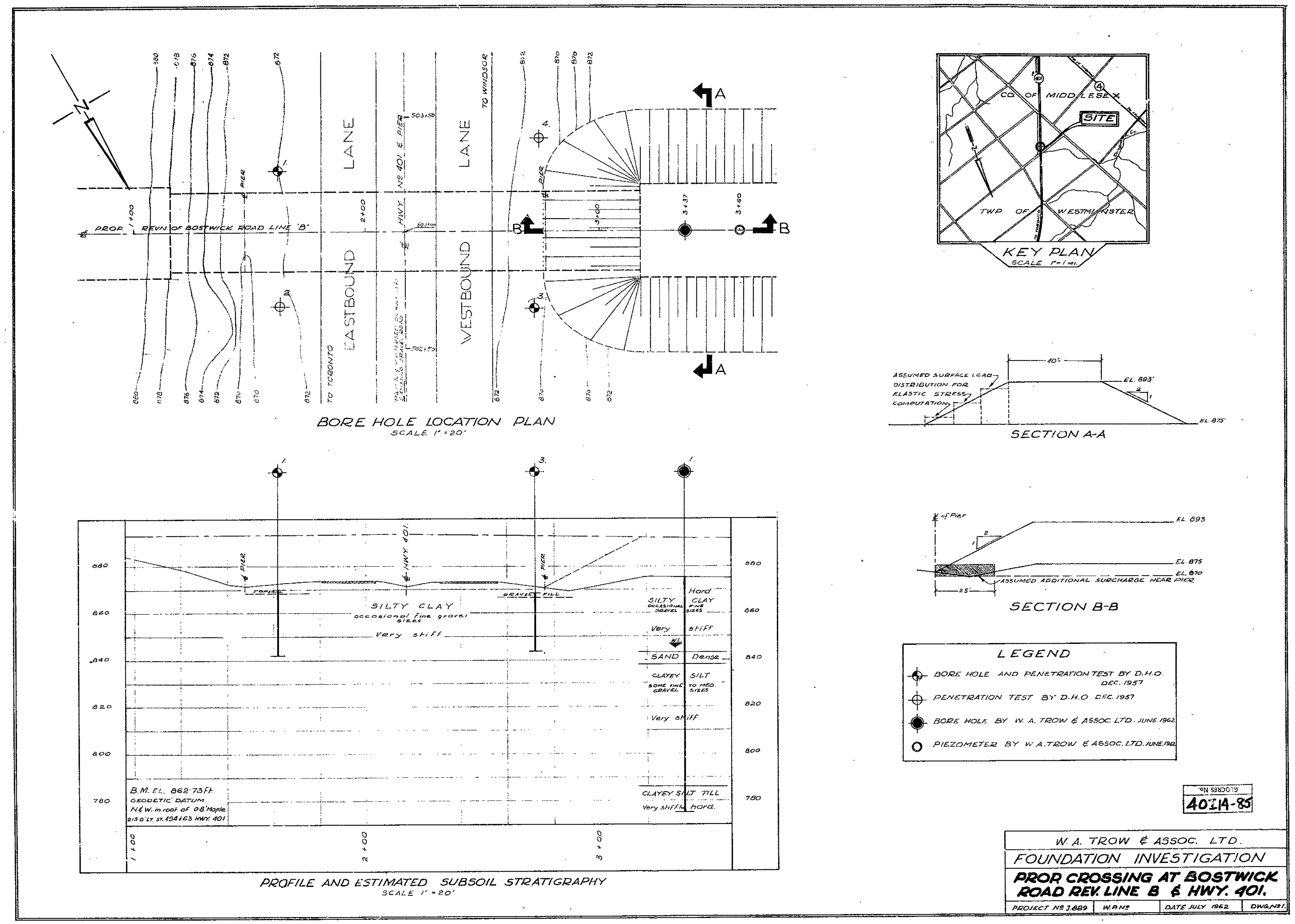


Elevation scale: 870, 865, 860, 855, 850, 845, 840

Depth: 5, 10, 15, 20, 25, 30

SOME DEFECTS IN NEGATIVE DUE

TO CONDITION OF ORIGINAL DOCUMENTS



SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS