

00260  
TD-218

A. G. STERMAC  
DEPUTY MINISTER



Hwy. 401 & Keele St.,  
Downsview 488, Ontario,  
C A N A D A

Tel. 248-3282

(Area Code 416)

DEPARTMENT OF HIGHWAYS

Materials and Testing Office

July 25, 1969

Mr. W. E. Thornley, Jr.,  
Earth Sciences,  
Tetrametrics Division,  
15027 West 5th Avenue,  
Golden, Colorado 80401,  
U.S.A.

Dear Mr. Thornley:

Re: New Welland Canal Tunnel Project

Thank you for your letter of July 22, 1969, regarding the  
Welland Canal Tunnel.

Due to some unforeseeable circumstances, the commencement  
of construction of the tunnel had to be postponed until the  
spring of 1970 at the earliest.

The only instrumentation contemplated for this tunnel is  
the installation of a number of earth pressure cells of the  
Gloetzel type. However, the final decision on this has not  
yet been reached. Should a positive decision be reached, we  
will be in touch with Tetrametrics who, in turn, will be in  
touch with you. There is no doubt that we will require your  
assistance in both designing and installing these cells, should  
we decide to put them in.

This is, I believe, all the information you have requested.

Sincerely yours

*A. G. Stermac*  
A. G. Stermac

Principal Foundation Engineer

AGS/MieP

cc: Foundations Files ✓

Gen. Files



A CHALMERS COMPANY

**EARTH SCIENCES**

TERRAMETRICS DIVISION  
3000 AVENUE 55TH AVENUE  
EDMONTON, ALBERTA T6C 0B8  
CANADA (403) 462-8800

Ref: 00260  
TD-218

July 22, 1969

Mr. A. G. Stermac  
Ontario Department of Highways  
Downsville, Ontario, Canada

Dear Mr. Stermac:

Re: New Welland Canal Tunnel Project

In reviewing our key proposal file and trying to bring it up to date, we are very interested in the current status of the highway tunnels under the Welland Canal.

You are familiar with Terrametrics' instrumentation capabilities so no long explanation is required. We are interested in the current status of the project and anything we can do to assist in its planning. So we would appreciate, at your convenience, knowing the present status of the project.

Very truly yours,

TERRAMETRICS DIVISION

W. H. Thornley, Jr.

WHT:m1

cc: RocTest, Ltd.

Department of Highways Ontario

Copy for the information of

Mr. A. G. Stermac, Principal Foundation Engineer, Lab. Bldg. Downsview.

Bridge Office,  
Downsview, Ontario,  
January 24, 1969.

Mr. W. A. O'Neil, P. Eng.,  
Director of Construction,  
The St. Lawrence Seaway Authority,  
Box 592,  
St. Catharines, Ont.

Re: Your File 58-8-3-1  
Our WP 240-66, Site 34-225  
East Main St. Tunnel

Dear Sir:

We are studying the problem of cost-sharing with respect to Well Interference and hope to arrange a meeting with the Authority in the near future.

The report prepared by Hydrology Consultants does not include a table of dates of your operations which might cause interference. Is there to be a further report and another billing from the Consultant?

Yours truly,



F. I. Hewson,  
Senior Bridge Liaison Engineer.

FIH/vh

cc/ A. G. Stermac  
K. Olpinski  
E. G. Nisbit

*When can we have a (some?) proposal(s) for said meeting?*





## THE ST. LAWRENCE SEAWAY AUTHORITY

ADMINISTRATION DE LA VOIE MARITIME DU SAINT-LAURENT

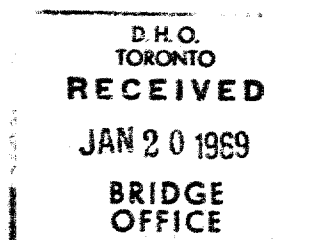
Construction Branch,  
Box 592,  
St. Catharines, Ontario.  
January 15, 1969.

Mr. F. I. Hewson,  
Senior Bridge Liaison Engineer,  
Department of Highways of Ontario,  
Downsview, Ontario.

Dear Mr. Hewson:

Further to a telephone conversation between yourself and Mr. E. C. Nisbet of The St. Lawrence Seaway Authority, Construction Branch, on January 14, 1969, we are forwarding a copy of Hydrology Consultants Limited invoice for the recent Well Interference Study conducted in the Welland area. As discussed on the telephone, we would appreciate your consideration in sharing a portion of the cost of this initial survey which was carried out under an agreement between Hydrology Consultants Ltd. and The St. Lawrence Seaway Authority, for an estimated cost of \$5,500.00. The final invoice exceeded the original estimate by \$4,798.53. The extra expense was incurred due to the inability of Hydrology Consultants Ltd. to utilize the computer as originally proposed to outline expected areas of interference. This necessitated ground survey of a much greater area to ensure that well records were obtained outside of any areas likely to be influenced by dewatering operations in the East Main Street tunnel or syphon sites.

The completed survey and developed computer program has provided us with useful data which will be used to assist us in settling water well claims. The degree of responsibility for water well interference caused by dewatering operations at the various sites could also be established by the program. The relative degree of responsibility would presumably determine the amount of money to be paid by each organization to settle claims. Cost sharing on the basis of the computer program appears attractive at first glance, however we feel there are definite drawbacks to this approach.



....2



Firstly: the quantity of water to be removed from the East Main Street tunnel is not established at present and will not be determined until dewatering operations have been underway for some time. Secondly: observation wells and private wells will require frequent monitoring to ascertain the amount of drawdown thereby establishing the relative degree of responsibility. We feel that a great deal of unnecessary administrative work will be required to establish shared costs on this basis. A simpler approach to the problem might be to share claim costs on a percentage basis which could be arrived at by mutual consultation and agreement.

We would be pleased to receive your comments in this regard along with any alternate method of cost sharing that might be agreeable to you. To this date, we have settled 34 water well claims with the total cost to the Authority being in the amount of \$7,669.78. A summary of these claims is attached. You will note that the Authority has only assumed responsibility for a portion of the well-owner's costs in re-establishing the well. Any improvements to a system are paid for by the owner.



W. A. O'Neil,  
Director of Construction.

BGN/jb  
att.

CLAIM COSTS A.S OF:  
DEC. 9 '68. T2

ST. LAWRENCE SEAWAY AUTHORITY,  
CONSTRUCTION BRANCH,  
ST. CATHARINES, ONTARIO.

# CLAIM COSTS

WELLAND CHANNEL BY-PASS  
RE: COMPLAINTS & CLAIMS  
SHEET 1 of 1; COMP BY: T.E.

WELL No.	WELL OWNER	ADDRESS	DATE OF SETTLEMENT	MODIFIED EQUIPMENT	DEPTH TO NEW INTAKE (ft)	DEPTH (ft)		WELL OWNER'S COST (\$)	SLSA COST (\$)	TOTAL COST (\$)
						DRILLED	DUG			
4003	MR. J. HAGAR	RR1, F. ROBINSON LOT 14, CONC. 1. CROWLAND TWP.	OCT. 27 '68	DEEP WELL PRESSURE PUMP		103		69.50	161.16	230.66
4008	MR. N. EGGLETON	RR1, F. ROBINSON LOT 10, CONC. 2 CROWLAND TWP.	OCT. 11 '68	DEEP WELL PRESSURE SYSTEM	80	86		58.50	171.97	230.47
4009	MR. P. GEBHAED	RR1, WELLAND CAMBRIDGE RD.	AUG. 15 '68	LABOUR & MAT'L COST		68		—	67.94	67.94
4010	MR. W. HORTON	RR1, WELLAND LOT 21, CONC. 3 CROWLAND TWP.	SEPT. 3 '68	NEW PRESSURE PUMP SYST.		87.25		73.50	153.58	227.08
4010	MR. C. SCHWANE	RR1, WELLAND LOT 22 & 23, CONC. 3 CROWLAND TWP.	NOV. 5 '68	DEEP WELL PRESSURE SYSTEM	70	82		69.50	168.29	237.79
4017	MR. M. GAREAU	RR1, WELLAND LOT 17, CONC. 5 CROWLAND TWP.	NOV. 14 '68	TAP CITY WATER				—	365.00	365.00
4023	MR. & MRS. G. DOLAN	RR3, P. COLBORNE LOT 14 & 15, CONC. 5 HUMBERSTONE	OCT. 29 '68	NEW INTAKE PIPE	+90	93		—	33.39	33.39
4024	MR. & MRS. L. PRINC	RR1, F. ROBINSON LOT 7 & 9, CONC. 1 CROWLAND TWP.	DEC. 3 '68	DEEP WELL JET SYSTEM	70	40		260.00	800.78	1060.78
4026	MR. E. A. KAZMIR	RR1, MAIN FLEET LOT 32, CONC. 4 HUMBERSTONE	NOV. 14 '68	DEEP WELL JET SYSTEM		94		112.50	179.69	292.19
4029	MR. & MRS. N. EGGLETON	RR2, WELLAND LOT 230 THOROLD TWP.	OCT. 26 '68	DEEP WELL JET SYSTEM	70	150		60.90	128.75	189.65
4030	MR. REDSHAW	RR2, WELLAND LOT 226 THOROLD TWP.	OCT. 29 '68	DEEP WELL JET SYSTEM	90	118		108.15	236.77	344.92
4031	MR. S. STINSON	RR2, WELLAND LOT 220 THOROLD TWP.	OCT. 30 '68	DEEP WELL JET SYSTEM	80	105		99.73	182.72	282.45

C. E. NISBET  
C. J. CHRISTENSEN

SUB-TOTAL COST (\$) = 912.28 2648.04 3560.32

St. LAWRENCE SEAWAY AUTHORITY,  
CONSTRUCTION BRANCH,  
St. CATHARINES, ONTARIO.

## CLAIM COSTS

WELLAND CHANNEL BY-PASS  
RE: COMPLAINTS & CLAIMS  
SHEET 2 of \_\_\_\_; COMP BY: T.B.

WELL No.	WELL OWNER	ADDRESS	DATE OF SETTLEMENT	MODIFIED EQUIPMENT	DEPTH TO NEW INTAKE (ft)	DEPTH (ft)		WELL OWNER'S COST (\$)	SLSA COST (\$)	TOTAL COST (\$)
						DRILLED	DUG			
4032	MR. M. McBRAYNE	RR2, WELLAND LOT 220 THOROLD	OCT. 29 '68	DEEPNELL JET SYSTEM		117		238.21	200.00	438.21
4033	MR. & MRS. DELLMORE	RR2, WELLAND LOT 220 THOROLD TWP.	OCT. 24 '68	DEEPNELL PRESS. SYSTEM		117		32.50	156.16	208.66
4034	MRS. MRS. C. PHILLIPS	RR2, WELLAND LOT 220 THOROLD TWP.	OCT. 29 '68	DEEPNELL JET SYSTEM		117		248.21	200.00	448.21
4035	MR. V. TOENG	RR2, WELLAND LOT 220 THOROLD TWP.	NOV. 6 '68	DEEPNELL JET SYSTEM		100		80.16	230.00	310.16
4037	MR. & MRS. H. McNEAL	RR2, WELLAND LOT 220 THOROLD TWP.	OCT. 29 '68	DEEPNELL PRESS. SYSTEM		2100		150.87	249.13	400.00
4042	MR. JOE KARMIR	RR3, MAIN FLEET LOT 31 & 32, CONC. 4 HUMBERSTONE	NOV. 25 '68	POND CLEARANCE		30' x 50' x 9'		68.30	250.00	318.30
4043	MR. M. KARMIR	RR1, MAIN FLEET LOT 31, CONC. 4 HUMBERSTONE	NOV. 14 '68	DEEPNELL JET SYSTEM		110		102.50	139.76	242.26
4044	MR. G. ROIK	RR2, WELLAND LOT 211 THOROLD TWP.	NOV. 6 '68	DEEPNELL JET SYSTEM	94	115		76.76	200.00	276.76
4052	MR. & MRS. P. PROTE	RR4, WELLAND LOT 14, CONC. 6 CROWLAND TWP.	NOV. 18 '68	DEEPNELL JET SYSTEM		70		77.40	142.29	219.69
4061	MR. F. WATERS	RR4, WELLAND LOT 16, CONC. 7 CROWLAND	DEC. 4 '68	DEEPNELL JET SYSTEM		87		105.00	196.14	301.14
4062	MR. & MRS. S. SHUMLEY	RR1, PT. ROBINSON LOT 12, CONC. 1 CROWLAND TWP.	DEC. 3 '68	DEEPNELL JET SYSTEM	90	104		73.50	165.74	239.24
4063	MR. S. STEELE	RR3, PT. COLBOURNE LOT 16, CONC. 4 HUMBERSTONE	DEC. 4 '68	POND CLEARANCE			CAP. OF POND 4,500,000 gal.	—	1009.00	1009.00

SUB-TOTAL COST (\$) = 2185.69 5784.26 7971.95

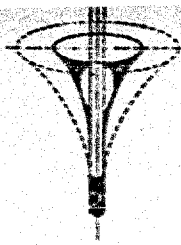
ST. LAWRENCE SEAWAY AUTHORITY,  
CONSTRUCTION BRANCH,  
ST. CATHARINES, ONTARIO.

# CLAIM COSTS

WELLAND CHANNEL BY-PASS  
RE: COMPLAINTS & CLAIMS  
SHEET 3 of 3; COMP BY: \_\_\_\_\_

WELL No.	WELL OWNER	ADDRESS	DATE OF SETTLEMENT	MODIFIED EQUIPMENT	DEPTH TO NEW INTAKE (ft)	DEPTH (ft)		WELL OWNER'S COST (\$)	SLSA COS. (\$)	TOTAL COST (\$)
						DRILLED	DUG			
4033	MR. W. H. McCORD	LOT 16, CONC. 5 HUMBERSTONE	OCT 11 '68	WATER BILL 2000 gal				—	9.00	9.00
4018	MR. T. CIEBYNSKI	TOWNLINE & YARCK HUMBERSTONE	DEC 5 '68	WATER BILL				—	85.00	85.00
4065	MR. & MRS. R. ROBINS	RR 2, WELLAND LOT 220 THOROLD	DEC. 10 '68	DEEPWELL JET SYSTEM	92	120		98.25	162.47	261.72
4066	MR. N. LEE	RR 2, WELLAND LOT 219 THOROLD	DEC 11 '68	INTAKE-JET SYSTEM	75	180		28.87	100.00	128.87
4058	MR. R. TILON	RR 2, WELLAND LOT 211 THOROLD	DEC. 12 '68	DEEPWELL JET SYSTEM	100	116		77.70	157.30	235.00
4067	MR. B. KORDYAKA	RR 3, P. COLBORNE LOT 17 1/2, CONC. 5 HUMBERSTONE	DEC 12 '68	DUG POND				—	125.00	125.00
4071	MR. S. FOSTER	RR 2, WELLAND LOT 221 THOROLD	DEC 17 '68	DEEP WELL SYSTEM	88	150		276.10 <del>276.10</del>	200.00 <del>300.50</del>	REVISED. DEC 23 476.10
4023	MR. F. OBT	RR 1, MAINFLEET LOT 33, CONC. 4 HUMBERSTONE	JAN. 13 '69	NEW WELL DRILLED		102		ALL OTHER COSTS	DRILLING 663.00	663.00 +
4076	MR. NILSON	RR 4, WELLAND, LOT B, CONC. 7 CROWLAND	JAN 15 '69	DEEPWELL JET SYSTEM	70	76		86.62	156.63	243.25
4077	MR. F. LATHERSTON	RR 2, WELLAND LOT 220 THOROLD	JAN 13 '69	DEEPWELL JET SYSTEM	77	100 +		86.62	225.12	311.74
								655.16	1883.52	2538.68

400/ Jan 15 '69 SUB-TOTAL COST (\$) = 2240.83 1663.78 10510.63  
DEFECTS IN. NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



**HYDROLOGY CONSULTANTS LIMITED**  
Suite 6, 1125 Dundas Street, East, Cooksville

Telephone:  
379-1611

January 9, 1969

Mr. W. A. O'Neill,  
St. Lawrence Seaway Authority,  
Construction Branch,  
P. O. Box 592,  
ST. CATHARINES, Ontario.  
Attention: Mr. E. Nisbet.

Invoice for work by Hydrology Consultants Limited in connection with the study of well interference resulting from dewatering on the "Well Channel Relocation" project.

The work performed included the collection, compilation and study of available hydrogeologist data, development and testing of a computer program, survey and inventory of private wells, collection and analysis of water samples, preparation of maps and tabulation of data, preparation of a "Preliminary Report", and consultation.

Period Covered: From initiation of study (Contract No. 14-222-1 dated September 17, 1968) to December 31, 1968.

A. Well Inventory Survey and Report - personnel engaged, time spent and expenses.

J. P. Nunan, Hydrogeologist: For review of hydrogeologic data, direction and supervision of project and consultation:

September	9.0 hours
October	20.5 "
November	5.0 "
December	23.5 "
Total	58.0 hours or 8.3 days

Dr. R. N. Farvolden, Hydrogeologist: For field supervision, consultation re development of computer program and report preparation:

October	7.0 hours
November	31.5 "
December	21.0 "
Total	<u>59.5</u> hours or 8.5 days

B. W. Beatty, Engineer: For field work, compilation of data and construction of maps and tables:

September	3.0 hours
October	91.0 "
November	143.5 "
December	93.0 "
Total	<u>330.5</u> hours or 47.2 days

#### CONSULTANTS' FEE:

16.8 days @ \$150/day .....	\$2,520.00
47.2 " @ \$70/day .....	<u>3,304.00</u>
Total Consultants' Fee	\$5,824.00

#### EXPENSES:

Travel, Accommodation and Meals ..	\$363.36
Telephone .....	71.30
Preparation of Base Map (does not include first nap) .....	96.59
Driller's Records .....	28.00

#### Chemical Analysis:

Chemical Supplies .....	30.00
Rental of Chemical Testing Equipment ( 2 mons. @ \$30.00/mon.) .....	60.00

#### Printing and Typing:

Typing and assembling reports (3 days @ \$40/day) .....	120.00
Zeroxing (2,580 @ 8¢/copy)	206.40

Total Consultants' Fee Brought Forward \$5,824.00

EXPENSES (Cont'd.)

Binders .....	\$ 53.02
Map Copying .....	<u>135.86</u>

Total Expenses \$1,164.53

TOTAL COST of Well Inventory Survey  
and Report \$6,988.53

B. Computer Study - personnel engaged, time spent and expenses.

J. P. Nunan, Hydrogeologist: 35 hours or 5.0 days

Dr. R. N. Farvolden, Hydrogeologist: 10.5 hours or 1.5 days

E. O. Frind, Engineer: 145 hours or 20.7 days

CONSULTANTS' FEE:

6.5 days @ \$150/day .....	\$ 975.00
20.7 " @ \$100/day .....	<u>2,070.00</u>

Total Consultants' Fee \$3,045.00

EXPENSES:

Travel .....	\$ 15.00
Computer Time .....	<u>250.00</u>

Total Expenses \$ 265.00

TOTAL COST of Computer Study \$3,310.00

TOTAL INVOICE \$10,298.53

Hwy. 401 & Keele St.,  
Downsview 464, Ontario.

Tel. 248-3282

(Area Code 416)

Materials and Testing Office

July 2, 1969

Mr. E. G. Nisbet,  
Materials Engineer,  
The St. Lawrence Seaway Authority,  
Construction Branch,  
Box 592,  
St. Catharines, Ontario.

Re: Main Street East Tunnel, Welland, Ontario  
Piezometer Details

---

Dear Sirs:

Further to your request, we are enclosing drawings and tables which indicate the locations and elevations of various piezometers within the canal prism in the vicinity of Main Street East Tunnel.

If you need any more information, please feel free to contact us.

Yours truly,



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

MD/HdeF  
Encls.





THE ST. LAWRENCE SEAWAY AUTHORITY

ADMINISTRATION DE LA VOIE MARITIME DU SAINT-LAURENT

Construction Branch,  
Box 592,  
St. Catharines, Ontario,  
June 2, 1969.

Department of Highways Ontario,  
Materials and Testing Division,  
Keelo Street S., Hwy. 401,  
Downsview, Ontario.

Attention: Mr. Devata, P. Eng.

Re: Piezometer Readings, East Main Street Tunnel.

Dear Sirs:

Attached are the results of readings taken on May 30, 1969 on piezometers located in the area of East Main Street Tunnel Site.

*Yours very truly,*

JYL/dp  
Att.

*for J. G. Esbet*  
J. G. Esbet,  
Materials Engineer.

Rwy. 401 & Beale St.,  
Downsview, Ontario.

Tel. 248-3282  
(Area Code 416)

Materials and Testing Office

December 6, 1968

Gibb, Albery, Pullerits and Dickson,  
Consulting Engineers,  
29 Cerva's Dr.,  
Don Mills, Ontario.

Attention: Mr. E. Kullerback

Gentlemen:

Re: East Main Street Tunnel -  
Final Foundation Investigation Report  
Addendum C.

---

Attached, please find Addendum C dealing with the directional shear strength properties of the irregularly layered silty clay stratum. This Addendum should be read in conjunction with Vol. I.

The findings of this investigation that are put forward as recommendations, have already been applied in the appropriate stability calculations.

Very truly yours,

*A. C. Sternec*

A. C. Sternec  
Principal Foundation Engineer

AGS/3deP  
Attach.

cc: Messrs. C. Tustin (S.I.S.A.)  
P. I. Hawson  
E. Bisbet (S.I.S.A.) - Letter only

Foundations Files  
Gen. Files

# H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS

3151 WHARTON WAY  
COOKSVILLE, ONTARIO

747 HYDE PARK ROAD  
LONDON, ONTARIO

196 BRONSON AVENUE  
OTTAWA 4, ONTARIO

Sent from:- COOKSVILLE

## TRANSMITTAL FORM

CONSIGNEE: DEPARTMENT OF HIGHWAYS, ONT.  
MATERIALS AND TESTING DIVISION  
HWY. 401 & KEELE ST.  
DOWNSVIEW, ONT.

DATE: DEC. 5, 1968

ATTENTION: MR. A. G. STERNIAK; P. ENG.

PROJECT No.: 67166

RE: FINAL FOUNDATION INVESTIGATION - ADDENDUM C,  
MAIN ST. EAST TUNNEL, WELLAND, ONTARIO

Sent by:-

☐

MESSENGER

☐

UNDER SEPARATE COVER

☒

ENCLOSED

No. Copies	Item	Description
5	REPORTS	DIRECTIONAL SHEAR STRENGTH PROPERTIES OF IRREGULARLY LAYERED SILTY CLAY STRATUM, PROPOSED CROSSING OF THE RE-ALIGNED WELLAND CANAL DATED NOV, 1968.

Remarks:

GOLDER & ASSOCIATES

Per

*[Signature]*



Hwy. 401 & Keele St.,  
Downsview, Ontario.

Tel. 248-3282

(Area Code 416)

DEPARTMENT OF HIGHWAYS  
Materials and Testing Division

September 29, 1968

Subject: PROPOSED CROSSING OF THE RE-ALIGNED  
WELLAND CANAL  
EAST MAIN STREET TUNNEL  
WELLAND, ONTARIO

Minutes of Meeting on September 29, 1968  
(Held at Department of Highways, Ontario,  
Board Room W-3)

Present:

Mr. V. Milligan	:	H. Q. Golder and Associates Ltd.
Mr. J. B. Davis	:	" " " " " "
Mr. C. Pullerits	:	Gibb, Albery, Pullerits & Dickson
Mr. K. Mullerbeck	:	" " " " "
Mr. B. S. Richardson	:	Department of Highways, Ontario
Mr. F. I. Hewson	:	" " " "
Mr. A. G. Stermac	:	" " " "
Mr. M. Devata	:	" " " "
Mr. B. T. Darch	:	" " " "

cont'd. /2 ...

## 1. INTRODUCTORY REMARKS: (9:00 A.M.)

Mr. Stermac outlined the purpose of the meeting, in which the following were discussed:

a) the results of a survey of all privately owned wells within a 4-mile radius of the Main Street East Tunnel site. The water levels in all such wells may be affected by the proposed dewatering measures at the site.

b) the results of laboratory testing on the clayey silt available on site, and a mixture of G.B.C. 'A' and bentonite (Volclay '90') and their engineering applications on the tunnel project, and

c) the dewatering requirements at the site, particularly in relation to the reduction in the piezometric pressure in the aquifer due to adjacent excavations.

Mr. Stermac mentioned that a meeting was held on September 23, 1968, between the S.L.S.A. and the D.H.O., in the St. Catharines' office of the S.L.S.A. At this meeting preliminary results of the well survey and related data were discussed.

## 2. DISCUSSION OF PRIVATE WELL SURVEY:

### 2.1) General:

A preliminary report (No. 68-F-71), containing all the factual data obtained from the well survey, the results of chemical analyses carried out on water samples, and related information, was submitted at the meeting. Messrs. Devata and Darch presented a general discussion of the preliminary data obtained and the engineering interpretations drawn.

cont'd. /3 ...

## 2. DISCUSSION OF PRIVATE WELL SURVEY: (cont'd.) ...

### 2.2) Comments on Well Survey:

Mr. Milligan pointed out that, even though there is no doubt that the water levels in the deep pumped wells will be lowered due to the excavations, it would be an extremely difficult matter to try and assess the overlapping effect of the various excavation contracts. It was agreed that it would be beneficial to have all legal aspects, associated with the water level lowering in all privately owned wells, looked after by a single organization. In this regard it was agreed that Mr. Hewson should approach the Steering Committee to gain approval for the suggestion that the S.L.S.A. be requested to take full responsibility for all such legal details.

Monitor piezometer locations have been suggested within the area of influence of the dewatering scheme to be installed at the tunnel site. Mr. Stermac requested that the proposed scheme be studied and comments made with regard to the number, location and the frequency of readings required during and following the construction phase. Mr. Davis suggested that it may be advisable to add a few additional piezometers within the 1/2 to 1 mile range, in order to more accurately determine the expected drawdown in this area.

### 2.3) Comments on Chemical Analyses Carried Out on Water Samples from Private Wells and Tunnel Borings:

It was Mr. Milligan's opinion that the relative magnitudes of the sulphate and dissolved solids content within the water obtained from the wells and tunnel borings are approximately the same. It is concluded that such minor variations will be of no significant importance in the design considerations at the tunnel. Mr. Darch stated that, in their opinion, the slight differences do exist and are not solely due to experimental scatter and the small number of water samples tested from the tunnel site. It was agreed,

## 2. DISCUSSION OF PRIVATE WELL SURVEY: (cont'd.) ...

### 2.3) Comments on Chemical Analyses Carried Out on Water Samples from Private Wells and Tunnel Borings: (cont'd.) ...

however, that as far as design is concerned, the small differences have no particular significance.

Chemical testing, carried out by H. Q. Golder and Associates Ltd., indicates that the sulphate content of the water in the bedrock increases with depth. Mr. Davis stated that it is their opinion that the sulphate content increase is due to a leaching of gypsum contained in the bedrock. This trend would tend to indicate that the aquifer is being re-charged with groundwater percolating through the overburden and down into the bedrock. In addition, tests carried out in which fresh water was passed through gypsum, disclosed that the sulphate content increased from zero concentration to something approaching complete saturation within a very short period of time. It is concluded that as far as gypsum solution at the tunnel site is concerned, the entry of fresh water at the site rather than groundwater seepage through the aquifer, will be the important factor.

## 3. HYDROLOGICAL CONSIDERATIONS AT THE TUNNEL SITE:

Mr. Milligan stated that, in their opinion, the re-charge in the aquifer is primarily due to seepage or percolation through the overburden, with groundwater seepage through the aquifer itself, of small importance. Significant lowering of the water level within the aquifer has already taken place at the site due to the excavations presently being carried out by the S.L.S.A. in Sections 2 and 3, located immediately to the north of the site. Additional lowering will no doubt occur once proposed excavations are started in the future, such as the excavation at Townline Rd. The present and expected lowering of the piezometric head in the aquifer has a beneficial effect as far as the temporary dewatering measures are concerned. The amount of future lowering will determine to what

### 3. HYDROLOGICAL CONSIDERATIONS AT THE TUNNEL SITE: (cont'd.) ...

order the temporary scheme can be reduced. It was agreed that H. Q. Golder and Associates Ltd. would submit, in writing, to the Foundation Section of the D.H.O., their recommendations regarding this matter. It was stressed, however, that the permanent dewatering scheme associated with Stage II, must be maintained as originally proposed.

Mr. Mullerbeck pointed out the importance of knowing the proposed sequence of the canal flooding. It was agreed that a slow, controlled flooding, in which the water in the canal is brought up to proposed level over a period of a few months, would be most beneficial for numerous reasons. It was agreed that Mr. Hewson would contact the S.L.S.A. and obtain their projected scheduling for the flooding.

Mr. Davis suggested that an investigation of the completed canal prism be made prior to flooding. If any holes or granular zones are noticed, they should be grouted.

### 4. RESULTS OF LABORATORY TESTING ON COMPACTED SAMPLES OF CLAYEY SILT AND G.B.C. 'A' & BENTONITE MIXTURE:

#### 4.1) General:

Mr. Stermac pointed out that locally available clayey silt is being considered for use as a compacted fill for the embankments as well as a general backfill behind the tunnel walls, etc. It is also possible that the impermeable membrane required around the tunnel could be composed of clayey silt. At some strategic locations, however, it may be necessary to form an impermeable membrane composed of a compacted mixture of G.B.C. 'A' and bentonite. The results of testing, carried out on the clayey silt by H. Q. Golder and Associates Ltd., were discussed by Mr. Milligan, while the results of the testing, performed to date by the D.H.O. on the bentonite mixture, were presented by Messrs. Devata and Darch.



4. RESULTS OF LABORATORY TESTING ON COMPACTED SAMPLES OF CLAYEY SILT AND G.B.C. 'A' & BENTONITE MIXTURE: (cont'd.) ...

4.2) Comments on Results of Testing:

Mr. Milligan stated that, in their opinion, the upper 10 to 15 feet (crust) of the clayey silt can be suitably compacted and used as embankment fill, provided the natural water content is rigidly controlled during placement and compaction. The surficial 2 to 3 feet across the site is generally in a softened and wet condition and, therefore, should not be used as fill. Everyone at the meeting was in complete agreement with these statements.

Continuing, Mr. Milligan stated that clayey silt will be used as backfill behind the tunnel wall. It would, therefore, be beneficial, as well as economical, to use it as an impermeable membrane in this area.

Mr. Fullerits commented that cracks may form along the interface between the tunnel wall and the compacted backfill. Such cracks could be due to one of the following:

1) cracking in the upper portion of the fill due to settlement of the compacted fill itself (max. settlement expected would probably be of the order of 1/2% of the overall height),  
and

11) cracks along the lower portion of tunnel wall due to shrinkage of the cohesive backfill relative to the concrete. Such cracks could provide a preferential seepage path for the canal water, allowing this water to come into communication with the confined aquifer.

Mr. Milligan commented that clayey silt, compacted on the dry side of the optimum compaction moisture content, would settle less but would have a more brittle structure than if compacted on the wet side. It may be beneficial, therefore, to compact the fill, placed immediately adjacent to the wall, on the

cont'd. /7 ...

4. RESULTS OF LABORATORY TESTING ON COMPACTED SAMPLES OF CLAYEY SILT AND G.B.C. 'A' & BENTONITE MIXTURE: (cont'd.) ...

4.2) Comments on Results of Testing: (cont'd.) ...

wet side of optimum so as to ensure a flexible material within this zone. In the extreme, a zone of "puddled" clay could be placed adjacent to the wall to provide flexibility and thus reduce the possibility of cracking in this critical area. Mr. Stermac mentioned that, as far as placement is concerned, this would be an expensive operation.

Mr. Pullerits expressed concern about the water tightness of the upper outer corners of the tunnel structure. In these areas the corners will be protected by only a 1-foot thickness of impermeable material, due to the fact that the design specifications call for the canal bottom to be underlain by a 5-foot thick protective zone composed of coarse graded stone and rip-rap. This being the case, he felt that consideration should be given to placing a bentonite panel over the top and partially down the sides of the tunnel section. It was mentioned that bentonite panels often are severely damaged during construction operations.

It was the general consensus of opinion that the impermeable membrane, required beneath the tunnel floor, should not be formed of compacted clayey silt. It is considered that the most suitable material for this purpose would be a compacted mixture of G.B.C. 'A' and bentonite because:

1) it could be readily controlled during placement and compaction, and

ii) would afford an excellent working surface.

Mr. Milligan suggested that the blanket should be approximately 2 feet in thickness. The majority of the group were in general agreement with this proposal, even though there were some feelings that this would increase the cost appreciably.

cont'd. /8 ...

4. RESULTS OF LABORATORY TESTING ON COMPACTED SAMPLES OF CLAYEY SILT AND G.B.C. 'A' & BENTONITE MIXTURE: (cont'd.) ...

Mr. Stermac pointed out that the Foundation Section will carry out additional testing on the bentonite mixture. The results will be presented at a meeting to be held within 2 to 3 weeks.

Mr. Stermac terminated the meeting at 1:20 P.M.

MINUTES OF

MEETING AT D.H.O., DOWNSVIEW

FEBRUARY 16, 1971

(9:30 A.M. TO 11:00 A.M.)

Re: Factual Information Provided  
by Additional Borings put  
Down at Relief Well Locations  
East Main Street Tunnel -  
New Welland Canal - W.P. 240-66

---

Present were Messrs.

K. Pullerits	)	Gibb, Albery, Pullerits and
K. Mullerbeck	)	Dickson, Consulting Engineers
V. Milligan	)	
J. B. Davis	)	H. Q. Golder and Associates Ltd.
A. G. Stermac	)	
M. Devata	)	Foundation Section - D.H.O.
B. T. Darch	)	

Mr. Stermac called the meeting to order, then asked Golder Associates to present the results of the additional borings put down at the relief well locations.

Mr. Davis made the presentation. The following pertinent points were made:

- 1) Borings were put down at four tentative relief well locations along either approach (total of eight).
- 2) The pervious water-bearing sand and gravel layer, which directly overlies bedrock at some locations within the tunnel excavation, was absent at the relief well locations investigated.

- 3) Pressure packer testing, carried out within the bedrock, indicated that the upper fractured zone (aquifer), along the East and West approaches, has a coefficient of permeability of about  $10^{-3}$  and  $10^{-4}$  cm./sec., respectively.

Mr. Milligan made the following comments:

- 1) Since the granular layer was not encountered above the bedrock surface in the area of the relief wells, there would be no benefit in extending well screens into the overburden.
- 2) The aquifer is relatively pervious along the East approach, therefore, no major difficulties are contemplated in developing the relief wells in this area to their required capacity (60 i.g.p.m. under a 43-foot hydrostatic water pressure head).
- 3) The aquifer along the West approach is relatively tight. It would be advantageous to put down additional borings at other relief well locations in this area, in order to confirm the range in permeability measured.

It was agreed that this be carried out under the supervision of Golder Associates Ltd.

- 4) Consideration could be given to cleaning the face of the rock at those relief wells put down along the West approach, in order to open up any joints or fissures that may be plugged.
  - Blasting should not be used.
  - Acid could be used as the cleaning agent.
- 5) Representatives of International Water Supply have expressed the opinion that relief wells should be put down using water as the drilling medium - never air. Air tends to form air locks and further plugs the joints and fissures with loose fragments.

5) (cont'd.) ...

It was agreed that this should be brought to the contractor's attention. In any event, his proposed installation procedure will have to be approved.

- Mr. Mullerbeck pointed out that the contract states that the required capacity of the wells must be proven by the contractor to the satisfaction of G.A.P.D. and D.H.O.

Mr. Fullerits recommended that a common discharge level be adopted for each of the relief wells, along either approach, in order to compensate for differential discharge heads. This could be accomplished by hooking the individual wells to a common header pipe.

Everyone agreed that this would be most beneficial and that it should be adopted.

- Mr. Fullerits stated that the pertinent drawings would be revised and re-submitted to all interested parties, such as the General Contractor, D.H.O. Bridge Section, and Engineering Audit, etc.

Mr. Sternac summarized the conclusions drawn at this meeting. Golder Associates stated that their report on the borings and related testing, carried out at the tentative relief well locations, would be submitted by February 28, 1971.

*B. T. Darch,*

ETD/MdeF

B. T. Darch  
SENIOR FOUNDATION ENGINEER  
For:  
A. G. Sternac  
PRINCIPAL FOUNDATION ENGINEER

Mr. B. H. Davis,  
Bridge Engineer,  
Bridge Division,  
Admin. Bldg.

*To: Design Notes on*  
Foundation Section, *Proposed Tunnel*  
Materials & Testing Div.,  
Room 107, Lab. Bldg. *Welland Canal.*

Attn: Mr. F. I. Newson,  
Sr. Bridge Liaison Eng.

August 15, 1967

Report No. 58134-1 by Golder & Associates --  
Design Notes on Proposed Tunnel and Bridge Schemes,  
Main Street East - Welland Canal, Welland, Ontario.  
N.P. 240-66 -- Site 34-225

Attached, please find four (4) copies of the above mentioned report for your use. It is believed that the information contained in this report will be sufficient for the present design stage. However, it should be borne in mind that some conclusions have been arrived at by making certain assumptions. These will have to be proven right during the final and detailed design stage.

Should you wish to discuss any parts of this report, please feel free to call on this Office.

JCS/mieP  
Attach.

*A. G. Sternac*  
A. G. Sternac  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. B. H. Davis (4)  
F. I. Newson

Foundations Files (2)  
Gen. Files

H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS

H. Q. GOLDER  
V. MILLIGAN  
L. G. SODERMAN  
J. L. SEYCHUK

3151 WHARTON WAY  
COOKSVILLE, ONTARIO  
G25-0094  
AREA CODE 416

August 8, 1967.

Department of Highways, Ontario,  
Materials and Testing Division,  
Hwy. 401 and Keele Street,  
DOWNSVIEW, Ontario.

Attention: Mr. A.G. Stermac, P. Eng.,  
Principal Foundation Engineer.

RE: PROPOSED CROSSING OF RE-ALIGNED  
WELLAND CANAL, MAIN STREET EAST,  
WELLAND, ONTARIO.  
W.P. 240-66, SITE 34-225.

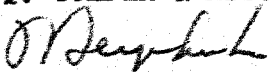
Dear Sirs:

Further to a meeting held in your office on July 21, 1967 (minutes presented in a letter prepared by F.I. Hewson, dated July 24, 1967) we submit the following notes to aid in the cost comparison studies of the proposed schemes for crossing the re-aligned Welland Canal, at Main Street East, in Welland, Ontario. The results of the preliminary foundation investigation for this project were presented in our report 66134, dated May, 1967. Two schemes are proposed, namely, either a tunnel crossing or a high level bridge crossing.

In the notes which follow the two proposed schemes are discussed separately.

Yours very truly,

H.Q. GOLDER & ASSOCIATES LTD.,

  
J.L. Seychuk, P. Eng.

JLS/je  
66134-1



## TABLE OF CONTENTS

	<u>PAGE</u>
<u>TUNNEL SCHEME</u>	1
A. SEQUENCE OF EXCAVATION	1
1) General	1
2) Excavation Stage I	2
3) Excavation Stage II	7
4) Overall Stability of Temporary Cut Slopes (Stages I and II)	8
5) Excavation Stage III	8
B. RETAINING WALL STRUCTURES	10
1) General Scheme	10
2) Reduction in Length of Monolithic Floor Slab	10
3) Extent of Granular Backfill Behind Retaining Walls and Under Floor Slab	11
4) Permanent Pressure Relief Well System	12
C. UPLIFT AND EARTH PRESSURES ON VARIOUS STRUCTURES	14
1) General	14
2) Tunnel Section	14
3) Retaining Wall Section with Monolithic Floor Slab (No Permanent Relief Wells)	15
4) Retaining Wall Section with Monolithic Floor Slab (Permanent Relief Wells)	16
5) Retaining Wall Structure With Paved Floor	17
D. ESTIMATED COST OF DE-WATERING SYSTEM	17
1) Approximate Cost of De-Watering Tunnel Excavation	17
2) Approximate Cost of Permanent Relief Wells	18
<u>BRIDGE SCHEME</u>	18
A. SUITABILITY OF EXCAVATED MATERIAL AS COMPACTED BACKFILL	19
1) Compaction Characteristics of Clayey Silt	19
2) Shear Strength Properties of Compacted Clayey Silt	21
B. APPROACH EMBANKMENTS	22
1) Stability Considerations	22
2) Settlement of Embankment and Foundation Subsoil	24
C. FOUNDATIONS	26
D. CONSTRUCTION PROCEDURES	27
1) Central Pier Excavations	27
2) Approximate Cost of De-Watering System	28

Cont'd.

TABLE OF CONTENTS (CONT'D.)

FIGURES - TUNNEL SCHEME

- FIGURE 1 - Sequence of Excavation and Proposed De-Watering Scheme.
- 2 - Sections Showing Proposed Excavation Sequence  
- Including De-Watering System.
- 3 - Inferred Soil Stratigraphy (West Side of Canal)
- 4 - Details of Proposed Scheme and Design Criteria
- 5 - Plan Showing Location of Permanent Relief Wells
- 6 - 9 - Summary of Earth and Uplift Pressure Computations  
(Sections A-A to D-D)
- 10 - Properties of Compacted Borrow Material

DESIGN NOTES ON PROPOSED MAIN STREET CROSSING  
RE-ALIGNED WELLAND CANAL

TUNNEL SCHEME

Based on our investigation it is considered that a tunnel is feasible from a soil mechanics point of view. With this in mind, and utilizing information provided in our report, a tunnel layout and excavation scheme has been proposed by the Department of Highways; this scheme is shown on D.H.O. Drawings D-6273-1 and D-6273-2 (dated July, 1967).

A. SEQUENCE OF EXCAVATION

1) General

Prior to commencement of the tunnel excavation the canal prism will have been excavated down to elevation 537.3. The east and west canal side slopes will be standing at  $2\frac{1}{2}$  horizontal to 1 vertical. The top of the slopes will be at about elevation 600 (existing ground level). Further, in accordance with St. Lawrence Seaway specifications, outlined on their Drawing No. 8913, dated June 1, 1967, a bench 132 feet wide will be provided at elevation 582. The conditions which will be existing at the time excavation is to commence are shown in plan and section on Figures 1 and 2, respectively.

The excavation for the tunnel, retaining walls and permanent approach cuts will be carried out in three stages. Each of these stages is discussed separately below.

2) Excavation Stage 1 (Excavation of Central Portion of Tunnel Section - 354 ft. length - See Figures 1 and 2).

a) Limits and Extent of Excavation.

The base of the tunnel is to be at elevation 504.5 and the width of the tunnel is to be about 78 feet. To reach this elevation excavation below the bottom of the canal is required. It is proposed that the temporary slopes be cut on slopes of 3 horizontal to 1 vertical. The proposed cut is shown in plan on Figure 1, while section 1-1 (east-west) and 2-2 (north-south) taken along the axes of the excavation are shown on Figure 2.

A portion of the excavated material is to be stock piled in two spoil areas located as shown on Figure 1; this soil is later to be used as backfill behind the walls of the structures. In this regard, it is recommended that only material acceptable for use as compacted backfill be placed in these stockpiles. The acceptability of the excavated material as compacted fill will be discussed in detail below. ("Bridge Scheme").

b) Upper Portion of Excavation (Elev. 582 to 550)

From elevation 582 to 550 the excavation will be carried out mainly through the clayey silt stratum. Excavation in this material can be carried out using a scraper operation, assisted by bulldozer pushing as required, however, on the west bank of the canal there is a water bearing silt deposit some 20 feet thick at about elevation 558 (See Figure 2). As discussed in our report excavation of the water bearing silt with scraper equipment may be difficult, if not impossible, due to the loosening of the silt by upward seepage. A dragline operation, will, therefore, be required.

c) Necessity of Relieving Hydrostatic Water Pressure Differential in the Lower Till and Bedrock.

Computations indicate, that to ensure basal stability as the glacial till is approached, the excavation can only be taken down to elevation 550. To permit excavation below this elevation it will be necessary to relieve the hydrostatic water pressure differential within the lower portion of the till and bedrock. It is recommended that the lowering of the piezometric level be effected by installing deep pumped wells along the east and west bank of the canal.

#### d) Temporary Deep Pumped Wells

Temporary wells should be installed at elevation 550 (before excavation below this elevation) and extend at least 5 feet into bedrock. (See Figure 2 for detail of wells in section).

The results of pumping tests carried out this summer along the proposed re-aligned Welland Canal are presented in reports prepared by hydrologists for the St. Lawrence Seaway Authority (Pumping Tests No. 1, 2, 3 and 4). The pumping tests closest to the proposed crossing indicate that deep wells, pumped at a rate of 200 gpm and placed on about 1,000 foot centres (2 rows), would effectively draw down the piezometric water level to about elevation 540 for canal construction.

Using the above as a guide, and considering that the piezometric level for tunnel excavation should be lowered to at least elevation 520, it is recommended that 5 deep wells be installed on about 100 foot centres on either side of the canal (i.e. total of 10). The proposed locations of the pumped wells are shown on Figure 1.

e) Excavation from Elevation 550 to 530.

Once continuous pumping from the deep wells has been in operation for a period of several days the excavation can be continued down to about elevation 530. It should be noted that at this stage pumping of the wells must be carried on continuously until the tunnel section is backfilled. The type of excavation equipment will be governed as discussed under section 2,b) above.

f) Installation of Temporary Relief Wells in Central Portion of Tunnel Excavation.

Once elevation 530 is reached it is recommended that temporary gravity drained relief or bleeder wells be installed around the perimeter of the proposed tunnel. These wells should be about 18 inches in diameter, gravel packed and extend at least 5 feet into bedrock. For preliminary design it is recommended that 14 such wells be installed as shown on Figure 1. The installation of these wells serves two purposes. Firstly, they will aid in reducing the unbalanced hydrostatic water pressure in the till and bedrock, particularly in the most critical central and deeper portion of the excavation. Secondly, by providing these relief wells it may be possible to discontinue the pumping of the deep wells as discussed in section h).

Provision for ditches and sumping should be made to remove the water from the relief wells collecting at excavation level.

g) Excavation from Elevation 530 to 504.5

The excavation can now be carried down to elevation 504.5, provided the unbalanced hydrostatic water pressure in the till and bedrock has been lowered sufficiently to prevent basal heave. Computations, summarized on Figure 2, indicate that in the tunnel section proper the piezometric groundwater level in the till and bedrock should be lowered to at least elevation 520 to ensure adequate stability against basal heave.

The most effective way of determining the piezometric level at different excavation stages is to provide monitor piezometers installed within the till and bedrock. Piezometers will therefore have to be installed prior to and during the various excavation phases. If the piezometric water level in the till and bedrock is higher than the level required to ensure stability it will be necessary to install additional relief wells in the central portion of the excavation.

An estimate of the costs for providing the de-watering system are presented in Section D, below.



h) Stage at which Pumping of Deep Wells can be Terminated.

Pumping of the deep temporary wells can only be stopped when the tunnel is constructed and backfilled or the piezometric level is kept down by other well installations (see Figures 4 and 6). Due to the relatively high monthly cost of pumping and maintaining deep wells it is economically desirable to terminate pumping at an early date. In this regard, it is recommended that the permanent gravity flow relief well system, to be discussed in section B, 4) be installed as soon as possible, particularly those relief wells adjacent to the tunnel section proper. These permanent relief wells, in combination with the temporary relief wells discussed under f) above, should provide the necessary groundwater control. If this is the case pumping of the deep wells can be terminated.

A close watch should be kept on the piezometer installations. If the piezometric water level in the till and/or bedrock rises appreciably it may be necessary to resume pumping again. This would, however, only be an emergency measure.

3) Excavation Stage II (End Portion of Tunnel, Including Portals and Part of Retaining Walls).

The slopes will be further trimmed. The cut at the end of this stage will extend from elevation 504.5 to

elevation 600. (see Figures 1 and 2); the cut will have a side slope of 3 horizontal to 1 vertical. It is imperative that the pressure relief well system outlined above be fully employed during this excavation stage.

#### 4) Overall Stability of Temporary Cut Slopes (Stages I and II)

The temporary cut slopes for the tunnel excavation, extending into and beyond the canal banks, will be stable at 3 horizontal to 1 vertical during the construction period, providing the water lowering system is fully operational.

The north and south cuts (along the axis of the canal) will be of the order of 33 feet in height (see section 2-2, Figure 2). This cut can be steepened to 2 horizontal to 1 vertical, provided the toe of the spoil areas are kept at least 200 feet back from the crest of the cuts. This would ensure that the spoil areas do not add appreciable surcharge to the slope.

#### 5) Excavation Stage III (Excavation for Retaining Walls and Permanent Approach Cuts)

##### a) Permanent Approach Cuts

As can be seen from Figure 3, the maximum height of the permanent approach cuts will occur adjacent to the end

of the retaining walls; these cuts are of the order of 70 to 75 feet in height. From this point on, the height of approach cuts decrease at the rate of 6 feet in 100 feet. As proposed the permanent cuts will have side slopes of 3 horizontal to 1 vertical with two 10 foot wide benches provided; the locations of these benches are shown in plan on Figure 5. These slopes will be stable over the long term, providing a granular drainage blanket some 2 to 3 feet thick is placed over the outer face of the slopes. This blanket will control the surficial drainage on the slope and thus prevent erosion gullyng; further, it will provide frost protection and thus improve the surficial stability. The granular blanketed slopes should be sodded and mulched. If provision is not made for a granular drainage blanket the slopes should be flattened to 4 horizontal to 1 vertical.

b) Temporary Cut Slopes Behind Retaining Wall

The temporary cut slopes behind the retaining walls will vary in height from zero at the portal to about 55 feet at the end. Temporary slopes of this height will be stable at a slope of 3 horizontal to 1 vertical. Following backfilling operations the permanent slopes formed, as shown on Figure 5, should be blanketed and treated as above.

## B. RETAINING WALL STRUCTURES

### 1) General Scheme (see Figure 3)

The portal for the tunnel is located some 210 feet from the centre line of the canal. The approaches to the tunnel section, on either side, are to be provided by retaining walls some 313 feet in length. The maximum height of the retaining walls is 60 feet and, as initially proposed by you, the retaining walls were to be integrally connected by a monolithic floor slab for their full length. Further, the retaining walls are to be strutted from the portal back about 120 feet.

### 2) Reduction in Length of Monolithic Floor Slab

If the length of the monolithic floor slab can be reduced the economics of the scheme would be improved. It is considered that the integral slab is required, however, to some point past the line of struts. For preliminary design it has been assumed that the monolithic floor slab is continuous for a distance of 170 feet past the portal, as shown on Figures 3 and 4.

By reducing the length of the monolithic floor slab, seepage from the canal to the transition zone between the

monolithic slab and the non-monolithic section (in this case a pavement section) would be increased due to the shortened seepage path. Seepage computations carried out, however, indicate that the increase would be quite nominal; i.e. the maximum total seepage at the transition point would not exceed 500 gpm. In the computations it was assumed that the retaining walls are backfilled for a horizontal distance of 5 feet with granular material with a coefficient of permeability,  $k$ , equal to  $10^{-1}$  cm/sec. This assumption is conservative since the granular backfill will not be continuous along the full length of the retaining wall as in section B-B, Figure 3) thus the quantity of flow given above is liberal.

It is recommended that a permanent sump pump drain be placed at the transition point between the monolithic floor slab and the pavement section, as shown on Figure 4. This sump should be designed to handle at least 500 gpm.

3) Extent of Granular Backfill Behind Retaining Walls and Under Floor Slabs.

It is recommended that granular backfill be placed behind the retaining walls and under the floor slab for a portion of the retaining wall length. The backfill should be extended

for a nominal distance beyond the permanent sump pump system; for preliminary design it is extended for a distance of 90 feet (see Figure 4). This granular backfill will draw down the surface of seepage behind the retaining wall and by so doing will reduce the exit gradient at the critical transition point between the monolithic slab and pavement section. This will eliminate the possibility of a piping failure at this critical point. The most probable phreatic surface is shown on Figure 4. Further, with full effective drainage of the granular backfill behind the retaining walls, the lateral earth pressures acting on the walls will be significantly reduced, as discussed below.

It is recommended that the granular blanket behind the retaining walls and under the monolithic slab be of the order of 4 feet and 2 feet thick, respectively. The 2 foot thick granular under-blanket should be continuous beneath the pavement section as well and would be incorporated in the sub-base of the roadway.

4) Permanent Pressure Relief Well System (see Figures 3 and 4)

As discussed previously, in section A,2), there will be a tendency for the base of excavation to heave, when

excavations are carried through the clayey silt and into the till, due to the unbalanced hydrostatic water pressure head in the lower portion of the till and bedrock. In the construction stage this pressure will be controlled primarily by the deep pumped wells. Following the construction excavation phase, however, in the sections where the weight of the retaining walls is not sufficient to resist full uplift, permanent gravity relief wells will be required. These wells should be taken 5 feet into bedrock; the specifications for and location of these wells are given on Figure 5. In all 11 permanent gravity flow relief wells are recommended on either side of the canal i.e. a total of 22 wells.

The wells should extend into trenches constructed along the sides of the granular under-drainage blanket. Properly filtered porous drainage pipe (nominally 6 inches in diameter) should be placed in the trenches to collect the water; this pipe should in turn gravity drain to the permanent sump pump drain (for details see Figures 8 and 9).

## C. UPLIFT AND EARTH PRESSURES ON VARIOUS STRUCTURES

### 1) General

The results of lateral earth and uplift pressure computations on various components of the tunnel structures are presented on Figures 6 to 9, inclusive. All this design data is summarized in tabular form on Figure 4. A brief summary of these pressures is given below.

### 2) Tunnel Section (Figure 6 - Section A-A)

#### a) Backfill

The structure can be backfilled with the upper portion of the clayey silt material available from the excavations (Stage I).

#### b) Design Earth Pressures

It is recommended that the structure be designed for the following pressures.

loading on roof - 2,800 lb/sq.ft.  
 loading at base slab level - 4,600 lb/sq.ft.  
 lateral earth pressure on walls - 4,800 lb/sq.ft. (maximum at base of wall) assuming

- i) full hydrostatic pressure acting on walls
- and ii) Coefficient of lateral earth pressure on rigid walls,  $K_o$ , equal to 0.6. (uniformly distributed)



c) Uplift Pressure on Tunnel

Computations indicate that once the canal is flooded the weight of the structure, the soil above the structure and water will be sufficient to resist maximum probable uplift. Following construction and backfilling the temporary relief wells on Figure 1 need not be operational. They could, therefore, be sealed and capped.

3) Retaining Wall Section with Monolithic Floor Slab  
(No Permanent Relief Wells) (Figure 7 - Section B-B)

a) Backfill

To be backfilled in a similar manner as the tunnel section.

b) Lateral Earth Pressure for Design

The walls should be designed for a lateral earth pressure ranging from 1,485 lb/sq.ft. at the top of the wall to 5,065 lb/sq.ft. at the bottom of the wall, assuming

- i) full hydrostatic water pressure acting
- and ii) Coefficient of lateral earth pressure on rigid walls,  $K_o$ , equal to 0.6 (uniformly distributed).

c) Uplift Pressure on Section

Computations carried out indicate, that to prevent uplift of the floor slab, the structure must provide a weight equivalent to 4,300 lb/sq.ft. at base slab level. If necessary, the required resistance to uplift could be provided by adding nibs to the structure.

4) Retaining Wall Section with Monolithic Floor Slab and Permanent Relief Wells - (Figure 8 - Section C-C)

a) Backfill

Compacted, free draining granular material to be placed behind retaining wall and beneath floor slab.

b) Lateral Earth Pressure for Design

The walls should be designed for a lateral earth pressure assuming

- i) full hydrostatic water pressure below the phreatic surface (design line shown on Figure 4)
- and ii) Coefficient of lateral earth pressure at rest on a rigid wall,  $K_0$ , equal to 0.5 (uniformly distributed)

c) Uplift Pressure on Section

The weight of the retaining wall, including the monolithic floor slab, will not be sufficient to resist the uplift pressures. Permanent relief wells will be required

as discussed in section B 4) above. Even with the provision of pressure relief wells the floor slab should be designed to resist the nominal uplift pressure shown on Figure 4.

#### 5) Retaining Wall Structure with Paved Floor (See Figure 9 - Section D-D)

##### a) Backfill

Compacted granular backfill is to be placed behind the retaining walls and beneath the pavement section.

##### b) Lateral Earth Pressure for Design.

Assuming full effective drainage of the granular backfill behind the walls, a coefficient of active earth pressure,  $K_a$ , equal to 0.4 (taking into consideration the sloping surcharge) can be used, if some movement of the top of the wall can take place.

In order to ensure overall stability of the retaining walls, 30 feet or greater in height, it would be necessary to extend the line of struts into this section (see Figure 4). If this is required, a coefficient of lateral earth pressure,  $K_o$ , equal to 0.6 should be used in the strutted section, instead of the value of 0.4 given above.

##### c) Uplift Pressure on Section

Permanent relief wells are required as discussed above.

#### D. ESTIMATED COST OF DE-WATERING SYSTEM

##### 1) Approximate Cost of De-Watering Tunnel Excavation

##### a) Deep Wells

Installation of 10 wells (\$1,500 per unit) = \$15,000.

Pumping and maintenance of 10 deep wells = \$ 3,500./mo.

## b) Temporary Relief Wells

Installation of 14 wells (\$600.per unit) = \$ 8,400.  
 Additional piezometer installations (20) = \$10,000.  
 (\$500.per unit)

2) Approximate Cost of Permanent Relief Wells (Under Retaining Wall Base Slab and Pavement Section).

22 relief well installations (\$600. per unit) = \$13,200.  
 Plus Drainage Pipe, Permanent Sump Pumps (2)  
 and Granular Blanket Material.

BRIDGE SCHEME

Preliminary information has already been provided on the arrangement and stability of approach embankments for the bridge crossing scheme (our letter dated July 17, 1967). Utilizing information provided in our report and the letter, a tentative bridge layout has been prepared; this layout is shown on D.H.O. Drawing No. D-6273-1 titled "Welland Canal Diversion Bridge - East Main", dated July, 1967.

As proposed the bridge will be a continuous structure of the order of 1,325 feet in length and will provide a minimum of 120 feet of clearance above canal level at elevation 537.3. The deck which is to be at a maximum elevation of

711, will be 40 feet wide and will carry 2 lanes of traffic. It is understood that the structure is to be supported on 4 piers and spill-through-type stub abutments. The abutments and piers are to be founded on steel H piling (12BP53 lb.) driven to refusal on or within the bedrock.

The approaches to the bridge structure are to be provided by earth fill embankments (approach grade 6 percent). The maximum height of the embankments (adjacent to the abutments) will be of the order of 85 feet above existing ground level. According to the above D.H.O. drawing the side slopes are proposed at  $2\frac{1}{2}$  horizontal to 1 vertical; in addition 4 drainage benches, about 10 feet in width, are located equidistantly along the slope. The end slope is similar with the exception that a 120 to 130 foot wide bench is provided at elevation 582 (in accordance with St. Lawrence Seaway specifications).

#### A. SUITABILITY OF EXCAVATED MATERIAL AS COMPACTED BACKFILL

##### 1) Compaction Characteristics of Clayey Silt

The principal foundation stratum on which approach embankments will be founded is a stiff clayey silt, the upper portion of which is desiccated. From excavation of the new canal this material will also be available for use as embankment fill.

The properties of the clayey silt as compacted fill are significant in relation to the stability of the embankments. It is known (report 56134) that the material in situ, in the upper portion of the stratum, has a water content varying between 15 and 21 percent. The maximum standard Proctor dry density is about 107 lb/cu.ft. at an optimum water content of 18 to 19 percent. Thus a portion of the material excavated for the canal will be relatively close to the optimum water content and, if placed in the summer, could be compacted close to maximum standard Proctor dry density.

Based on observation made during the field phase of the investigation, it is considered that the upper 20 feet (desiccated zone) of the stratum would be the most ideal for use as compacted fill. Below this upper zone, say to about a depth of 50 feet, the excavated clayey silt could be used for fill; however, due to its relatively higher liquidity index it would require strict engineering control during placement and compaction. For this reason, it would be advantageous to limit the use of clayey silt from this zone to areas where the height of the embankment is lower, say 40 feet or less. Below a depth of 50 feet the in situ water content

of the clayey silt stratum will probably be too high to allow it to be properly compacted without prior treatment such as drying. Some silt and sandy silt layers will be encountered during the canal excavation; this soil is unacceptable as embankment fill and as such should be wasted.

It is imperative that the clayey silt fill not be allowed to freeze during placement and compaction. Thus the fill should not be placed during the winter period.

## 2) Shear Strength Properties of Compacted Clayey Silt

The properties of some typical borrow materials in Eastern Canada are shown in Figure 10. It has been assumed from this data, and from the results of testing on thin walled tube samples of the clayey silt stratum, that the effective shear strength parameters of the material, when compacted close to standard Proctor maximum dry density are  $c' = 200$  lb/sq.ft.,  $\phi = 25$  degrees. In terms of undrained shear strength,  $C_u$ , would be greater than 2,000 lb/sq.ft. providing the clayey silt is compacted in lifts to at least 90 percent standard Proctor dry density.

## B. APPROACH EMBANKMENTS

### 1) Stability Considerations

#### a) Side Slopes

The stability of the earth fill embankment, composed of the compacted clayey silt, was discussed in detail in our letter dated July 17, 1967. In this letter it was concluded that an embankment up to 90 feet in height with an overall slope of 3 horizontal to 1 vertical, could be built safely at this site. It is therefore concluded that the proposed embankment, which will have a maximum height of 85 feet,  $2\frac{1}{2}$  horizontal to 1 vertical side slopes and three 10 foot wide benches will be stable, providing the side slopes are protected as discussed in d) below.

#### b) End Slopes

The end slope, i.e. the slope facing the canal, will be the critical slope as far as overall stability of the section is concerned. From existing ground surface to the crest, the proposed end slope has the same dimensions as the side slopes (slope of  $2\frac{1}{2}$ :1 with 3 benches). The toe of this slope, which is at elevation 582, is however, set back a minimum distance of 120 feet from the crest of the canal bank.



Preliminary total stress stability computations were carried on the proposed end slope section assuming

- i) The height of the slope - 147 feet (El. 684 to El. 537)
- ii) Overall Slope - slightly flatter than 3.5 horizontal to 1 vertical.
- iii) The average undrained shear strength,  $C_u$ , equal to 2,000 lb/sq.ft. (both for the compacted embankment fill and foundation subsoil)
- iv) Water level in the canal at Elev. 567.3

The results of these computations gave a factor of safety against failure of about unity. It is therefore considered that for preliminary design the end slopes between the benches, be flattened from  $2\frac{1}{2}$  horizontal to 1 vertical to 3 horizontal to 1 vertical. Depending on the results of the final sub-surface investigation, it may be possible to steepen the end slopes in the final design.

c) Other Considerations.

As discussed above the side slopes proposed should be stable at  $2\frac{1}{2}$  horizontal to 1 vertical. It is understood however, that slopes of the height contemplated are difficult to maintain because of their relative steepness. For instance power mowers cannot effectively operate on slopes greater than 40 feet in height if such slopes are much steeper than  $2\frac{1}{2}$

horizontal to 1 vertical. Based on this it is probable that the final side slopes will be flattened to at least 3 horizontal to 1 vertical between the drainage benches.

d) Protection of Permanent Slopes

The approach side slopes should be protected against surficial erosion due to runoff, as well as seasonal temperature changes, etc. For relatively flat side slopes (say 4 to 1) with benches, sod would probably be sufficient for this purpose. If slopes of the order of  $2\frac{1}{2}$  to 1 or even 3 to 1 are used, consideration should be given to providing a relatively free draining granular blanket some 2 feet thick covered by sod. The end slopes will require protection such as a rip rap cover.

2) Settlement of Embankment and Foundation Subsoil

Settlement of the embankment will occur due to i) settlement of the compacted embankment fill itself following placement and ii) settlement of the foundation subsoil due to the induced surcharge loading of the fill.

The net increase in vertical stress within the clayey silt subsoil at depth, due to an embankment loading some 85

feet high, would exceed the pre-consolidation pressure of the stratum, which ranges from 4 tons/sq.ft. in the upper desiccated zone to about  $1\frac{1}{2}$  tons/sq.ft. in the lower zone. Preliminary computations indicate that the settlement of the embankment, due to consolidation of the foundation subsoil, could be about  $1\frac{1}{2}$  to 2 feet. Assuming that the fill itself will settle something of the order of  $\frac{1}{2}$  percent of its height, the total settlement of the embankment (85 feet in height) could be about 2 to  $2\frac{1}{2}$  feet. At the point where the embankment is 45 feet high (mid height) settlement would be something less than 1 foot, with about 3 inches of this due to settlement of the embankment fill itself.

It is considered, based on the available laboratory consolidation tests, that 50 percent of the consolidation settlement of the embankment should occur within a period of about 1 year following construction, while the total consolidation settlement should be almost completed within 5 years. It would be advantageous to induce as much of the settlement as possible prior to the construction of the bridge structure so as to reduce differential settlement. In this regard the embankment should be constructed as soon as possible prior to the structure.

Differential settlement will occur between the approach embankments and the adjacent pile supported bridge abutments. It will therefore be necessary to make provision for maintaining and raising the grade of the embankment. A hinged approach slab could be utilized to provide a transition between the approach embankment and the fixed stub abutment founded on piles end bearing on bedrock.

### C. FOUNDATIONS

Since the proposed structure is to be continuous, and thus settlement sensitive, the piers and abutments should be supported on steel H piles driven to or within the bedrock. The dense sandy silt till directly overlying the bedrock is very bouldery; this may make it difficult, if not impossible, to drive the piles through the till. If the piles are founded in the till some settlement will occur. A hardened steel driving tip should be welded to the end of the piles to facilitate driving and to prevent damage to the piles. The required pile length will vary from 40 feet at the centre pier locations to about 170 feet at the stub abutment locations.

For preliminary design purposes a pile capacity of 70 tons/pile is being used for a 12BP53 lb. steel H pile. In determining the pile capacities, consideration should be given to the effect of negative skin friction which will be applied to the piles due to settlement of the foundation subsoil under the weight of the approach embankments. To allow for this, and to minimize damage during driving in the till, it is recommended that a heavier steel H section, say a 12BP74 lb. section, be allowed for in preliminary design and that the design capacity of this heavier section remain at 70 tons/pile.

For final design the capacity of the proposed piles should be checked by carrying out full scale pile loading tests.

#### D. CONSTRUCTION PRECEDURES

##### 1) Central Pier Excavations

The central pier pile caps are to be located at about elevation 519 to 520. Excavation to this level will be carried out through the clayey silt stratum and down into the basal till stratum; it is recommended that this excavation be carried out from within a sheeted cofferdam. As discussed

in section 2,c) (Tunnel Scheme) there is the danger of basal heave of the excavation unless the unbalanced piezometric groundwater level in the till and bedrock is lowered. This water lowering can be effected by deep pumped wells, installed and operated around the outer perimeter of the cofferdam. These wells should extend some 5 feet into bedrock, i.e. be about 75 to 80 feet in length below elevation 557 and conform to the specifications for deep wells outlined on Figure 2. For preliminary consideration it is recommended that 4 wells be allowed for at each of the two pier locations.

## 2) Approximate Cost of De-Watering System

Cost at each pier location	
Installation of 4 deep wells (\$1,500/unit)	\$6,000
Pumping and Maintenance of 4 wells	\$1,500/mo.

We trust that the above is sufficient for your immediate requirements. If we can be of any further assistance, please call us.

Yours very truly,

H.Q. GOLDER & ASSOCIATES LTD.

*B.T. Darch,*

B.T. Darch, P. Eng.,

BTD/je  
66134-1

**GOLDER & ASSOCIATES**



**FIGURES**

LEGEND

- TEMPORARY DEEP PILEDRILL WELLS (NOMINALLY 8" TO 12" IN DIA. WITH SCREEN AND GRAVEL PACKING)
  - WELLS SHOULD BE INSTALLED AT EL. 550 AND EXTEND ABOUT 5' INTO BEDROCK, & APPROX. 50' TO 70' IN LENGTH.
  - PUMPING FROM WELLS SHOULD BE CONTINUOUS ONCE EXCAVATION HAS REACHED EL. 550.
- TEMPORARY PRESSURE RELIEF WELLS (18" DIA. GRAVEL PACKED)
  - WELLS SHOULD BE INSTALLED FROM ABOUT EL. 550 AND EXTEND 5' INTO BEDROCK, & APPROX. 40' TO 50' IN LENGTH.
  - PROVISION SHOULD BE MADE TO COLLECT WATER IN THE EXCAVATION (BY MEANS OF SHALLOW DITCHES ETC.) AND BE PUMPED AWAY WITH SUMPS.

SEQUENCE OF EXCAVATION

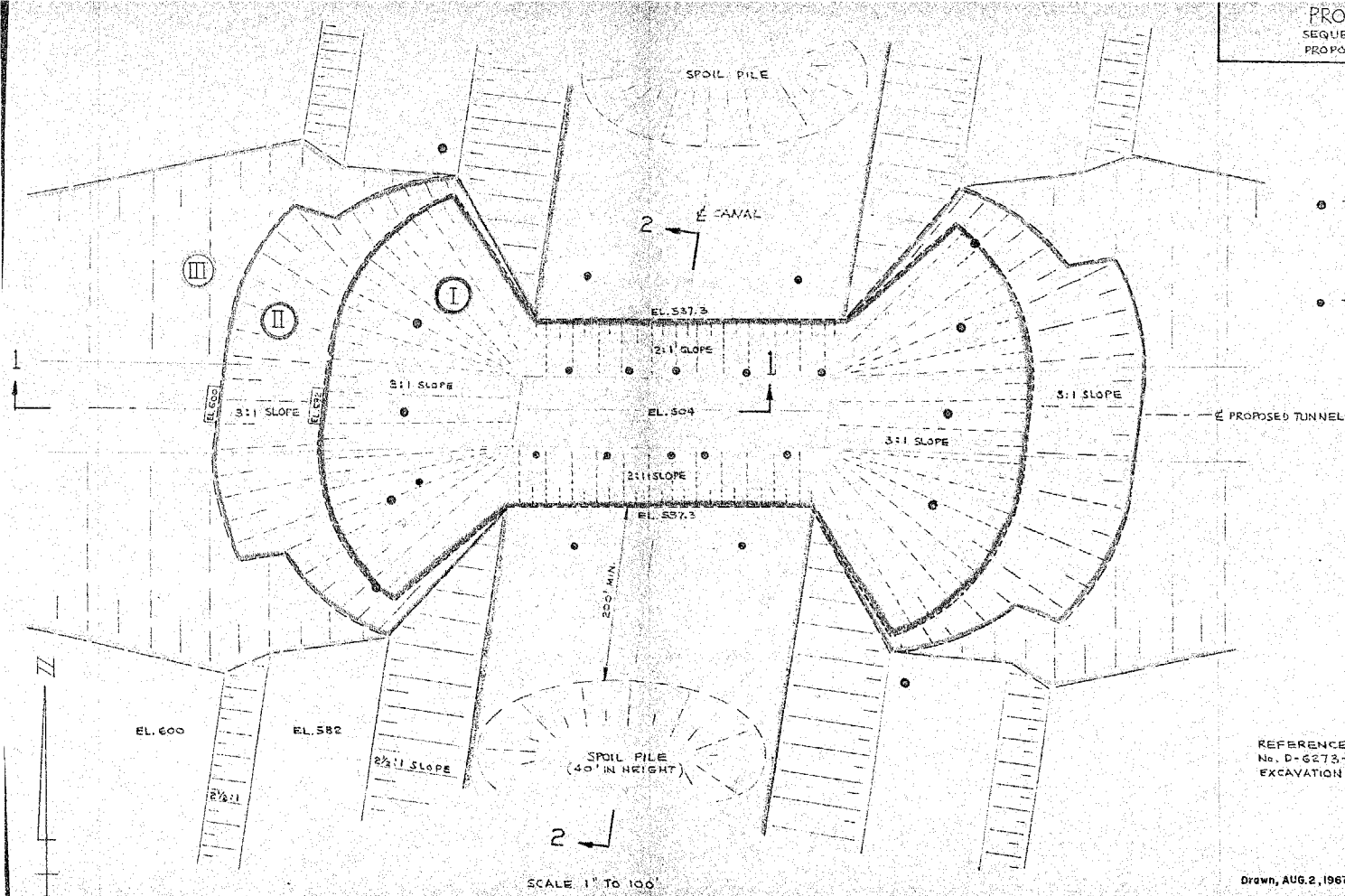
- I - STAGE NUMBER

REFERENCE: DEPARTMENT OF HIGHWAYS, ONTARIO DRAWING No. D-6273-2, WELLAND CANAL DIVERSION TUNNEL - EAST MAIN, EXCAVATION - 3 STAGES - SCHEME V, DATED: JULY '67.

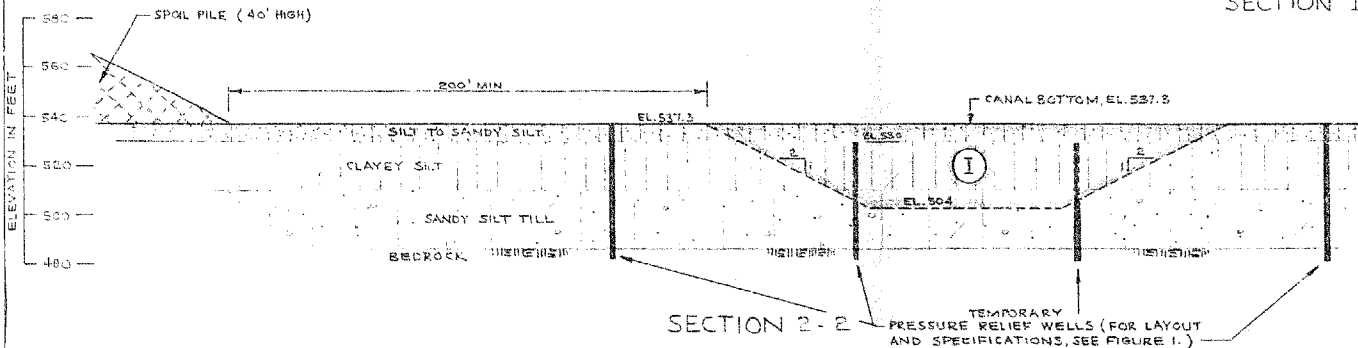
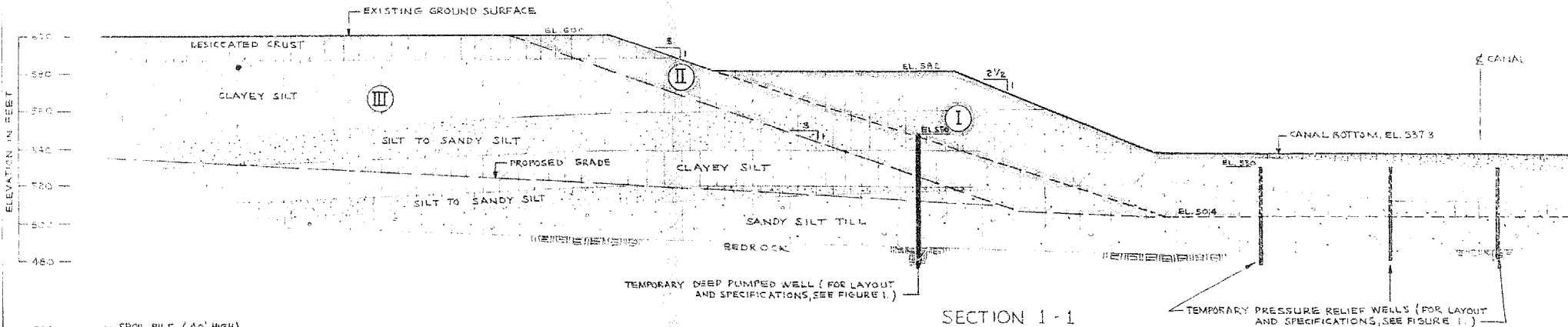
Drawn, AUG. 2, 1967.

GOLDER & ASSOCIATES

Made *Ans.*  
Chkd. *E.T.*  
Appd. *011*







NOTE: FOR LOCATIONS OF SECTIONS,  
SEE FIGURE 1.

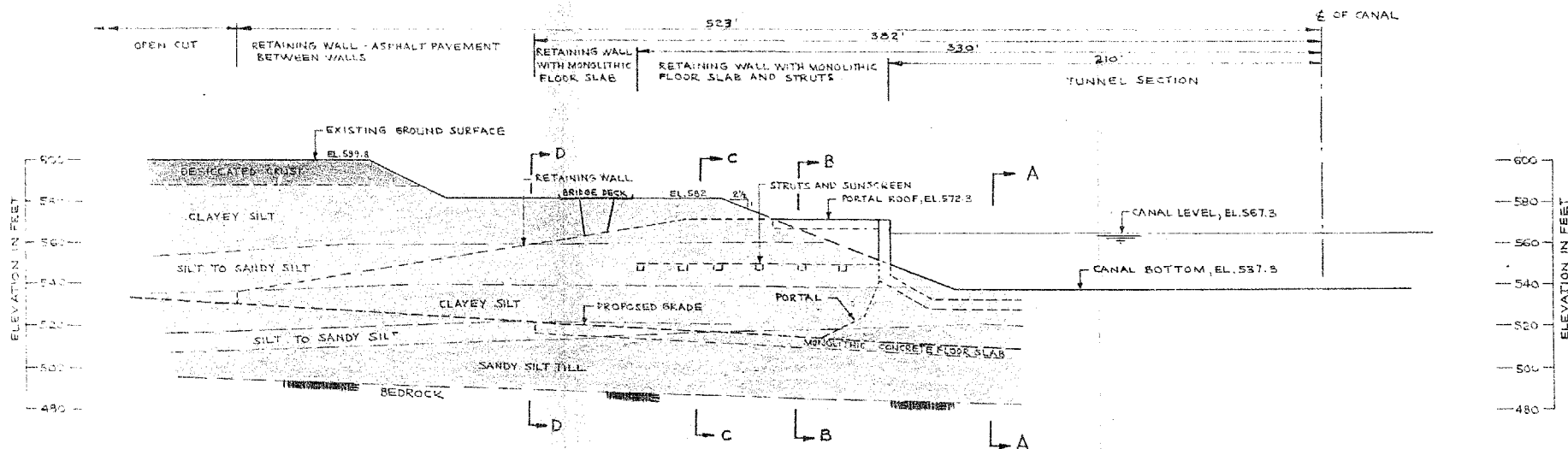
SCALE 1" TO 40'

**BASAL STABILITY CONDITIONS:**

- 1) AT PRESENT TIME PIEZOMETRIC G.W.L. IN TILL & BEDROCK IS ABOUT EL. 575.
- 2) QUESTION - TO WHAT LEVEL MUST THE DE-WATERING SYSTEM LOWER THE PIEZOMETRIC LEVEL IN THE CENTRAL PORTION OF THE CUT TO ENSURE AGAINST BASAL HEAVE?  
ASSUME BOUNDARY BETWEEN BEDROCK AND TILL TO BE ABOUT EL. 482  
MAXIMUM DEPTH OF CUT TO BE EL. 534  
LET HEAD OF WATER ABOVE EL. 482 BE X FEET.  
THEN THE BASAL STABILITY OF THE EXCAVATION CAN BE EXPRESSED BY  
$$F.S. = \frac{130(504 - 482)}{X \times 62.4}$$
  
TO PROVIDE A FACTOR OF SAFETY OF 1.2 AGAINST BASAL HEAVE OF THE EXCAVATION THE PIEZOMETRIC G.W.L. IN THE LOWER TILL AND BEDROCK MUST BE AT OR BELOW EL. 520 IN THE CENTRAL PORTION OF EXCAVATION.
- 3) THE ADEQUACY OF THE DE-WATERING SYSTEM MUST BE CHECKED BY INSTALLING AND MONITORING PIEZOMETERS PLACED IN THE TILL AND BEDROCK.

SEQUENCE OF EXCAVATION

① - STAGE NUMBER



NOTE: FOR SECTIONS A-A TO D-D,  
SEE FIGURES 2 TO 3 INC.

Drawn, AUG. 1, 1967

SCALE 1" TO 40'

GOLDER & ASSOCIATES

Made by  
Chkd. *[Signature]*  
Appd. *[Signature]*

$K_0$  - COEFFICIENT OF LATERAL EARTH PRESSURE AT REST (FOR RIGID WALLS)

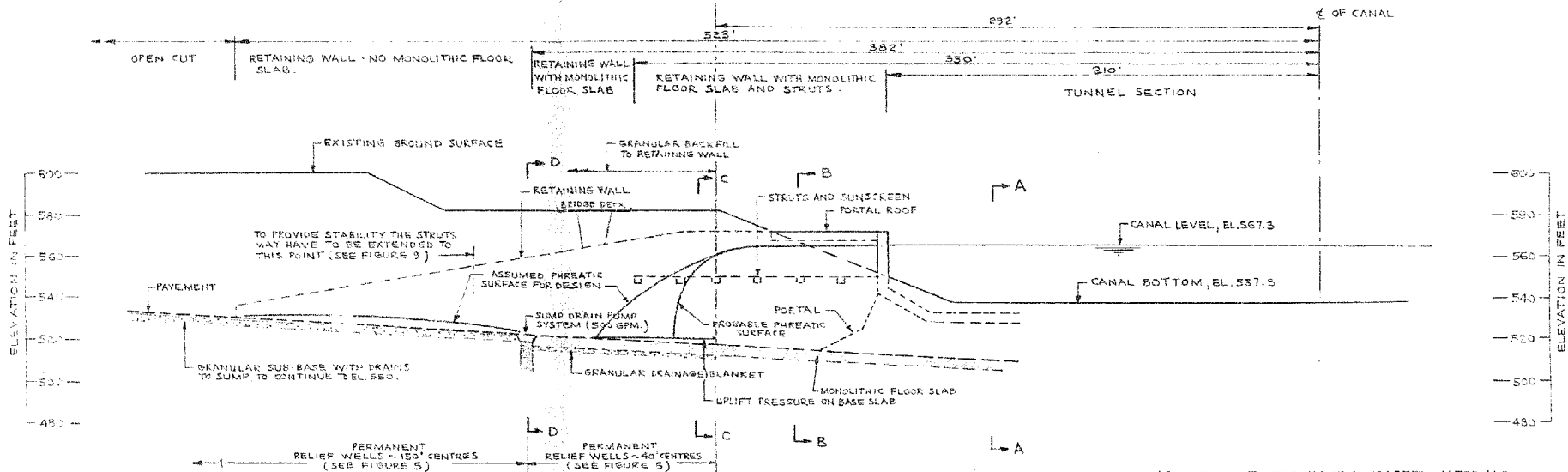
\*  $K_A$  - COEFFICIENT OF ACTIVE EARTH PRESSURE (ASSUMING THE TOP OF WALL CAN TOLERATE SOME MOVEMENT).

PROPOSED TUNNEL  
DETAILS OF PROPOSED SCHEME AND  
DESIGN CRITERIA

FIGURE 4

TABLE

<p>a) NEGLECT HYDROSTATIC WATER PRESSURE IN DESIGN (EFFECTIVE DRAINAGE PROVIDED WITHIN GRANULAR BACKFILL).</p> <p>b) BACKFILL <math>\gamma = 130</math> LB/CU.FT.  <math>K_A = 0.4</math> (SECTION WITHOUT STRUTS) *  <math>K_0 = 0.6</math> (SECTION WITH STRUTS) *</p> <p>RELIEF WELLS PROVIDED TO REDUCE UPLIFT PRESSURE ON PAVEMENT SECTION.</p> <p>FOR TYPICAL SECTION SEE FIG. 6 (SECTION D-D)</p>	<p>c) DESIGN FOR FULL HYDROSTATIC WATER PRESSURE BELOW PHREATIC SURFACE (DESIGN LINE).</p> <p>b) BACKFILL <math>\gamma = 125</math> LB/CU.FT.  <math>K_0 = 0.5</math></p> <p>RELIEF WELLS PROVIDED TO REDUCE UPLIFT PRESSURE ON BASE SLAB.</p> <p>FLOOR SLAB SHOULD BE DESIGNED FOR NOMINAL UPLIFT PRESSURE SHOWN BELOW.</p> <p>FOR TYPICAL SECTION SEE FIG. 3 (SECTION C-C)</p>	<p>a) DESIGN FOR FULL HYDROSTATIC WATER PRESSURE.</p> <p>b) BACKFILL <math>\gamma = 130</math> LB/CU.FT.  <math>K_0 = 0.6</math></p> <p>STRUCTURE MUST PROVIDE AN EQUIVALENT WEIGHT OF ABOUT 4300 LB/SQ.FT. AT BASE OF FLOOR SLAB TO RESIST HYDROSTATIC UPLIFT PRESSURES (<math>F_S = 1.2</math>) (MAY REQUIRE NIBS).</p> <p>SEE FIG. 7 (SECTION B-E)</p>	<p>a) DESIGN FOR FULL HYDROSTATIC WATER PRESSURE</p> <p>b) BACKFILL, <math>\gamma = 130</math> LB/CU.FT., <math>K_0 = 0.6</math></p> <p>WEIGHT OF STRUCTURE, BACKFILL ABOVE STRUCTURE AND WATER IS SUFFICIENT TO RESIST UNBALANCED HYDROSTATIC UPLIFT PRESSURE.</p> <p>SEE FIGURE 6 (SECTION A-A)</p>	<p>← LATERAL EARTH PRESSURES ON WALLS (RECOMMENDED DESIGN CRITERIA)</p> <p>← UPLIFT PRESSURES</p> <p>← REFERENCES</p>
--	--	---	---	---



ASSUMED UNIT WEIGHTS OF COMPACTED MATERIALS

CLAYEY SILT BACKFILL,  $\gamma = 130$  LB/CU. FT.  
GRANULAR BACKFILL,  $\gamma = 125$  LB/CU. FT.

Drawn, AUG. 1, 1967

GOLDER & ASSOCIATES

Made  
Chkd  
Appd

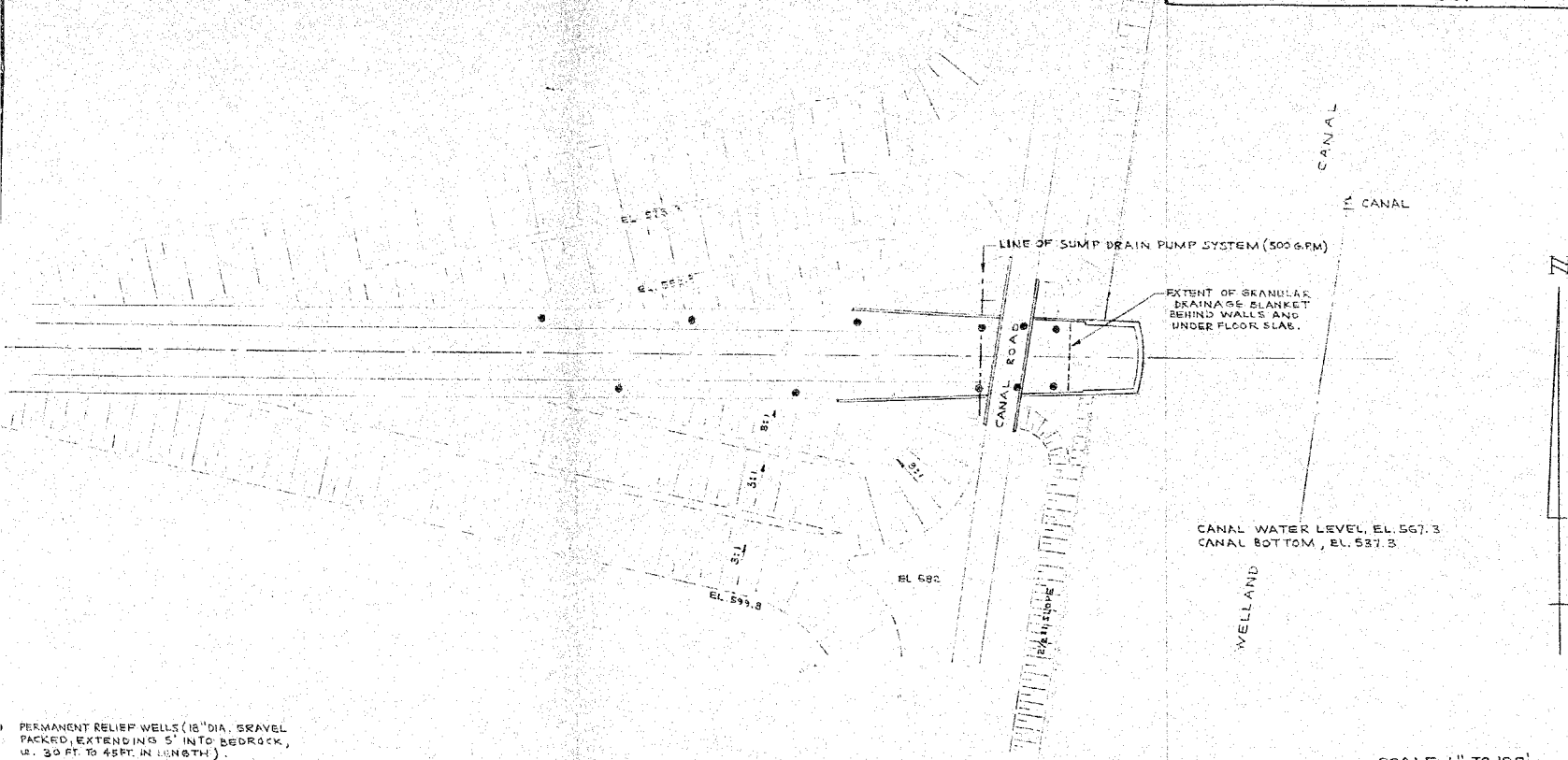
SCALE 1" TO 40'

NOTE: FOR LAYOUT OF RELIEF WELLS  
IN PLAN SEE FIGURE 5.

FOR SECTIONS A-A TO D-D,  
SEE FIGURES 5 TO 9 INC.

PROPOSED TUNNEL  
PLAN SHOWING LOCATIONS OF  
PERMANENT RELIEF WELLS.

FIGURE 5



PERMANENT RELIEF WELLS (16" DIA. GRAVEL  
PACKED, EXTENDING 5' INTO BEDROCK,  
W. 30 FT. TO 45 FT. IN LENGTH).

NOTE: FOR DETAILS OF RELIEF WELLS,  
SEE FIG. 8 (SECTION C-C) AND  
FIG. 9 (SECTION D-D).

SCALE 1" TO 100'

Drawn, AUG. 2, 1967.

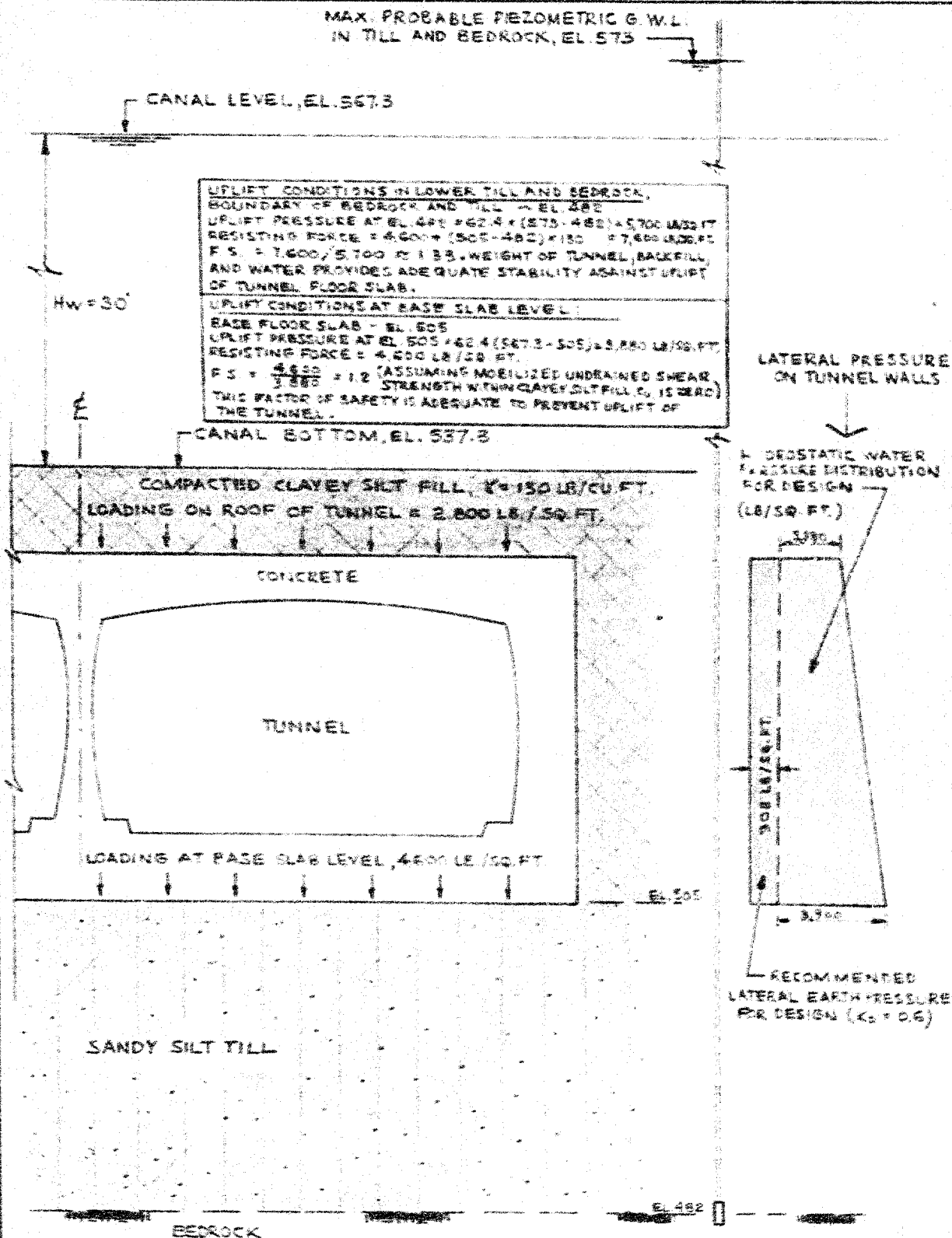
GOLDER & ASSOCIATES

Made *4/15/68*  
Chkd. *4/22/68*  
Appd. *5/2/68*

# TUNNEL SECTION A-A

## PRESSURES ON ROOF, BASE AND WALLS AND UPLIFT PRESSURE.

FIGURE 6

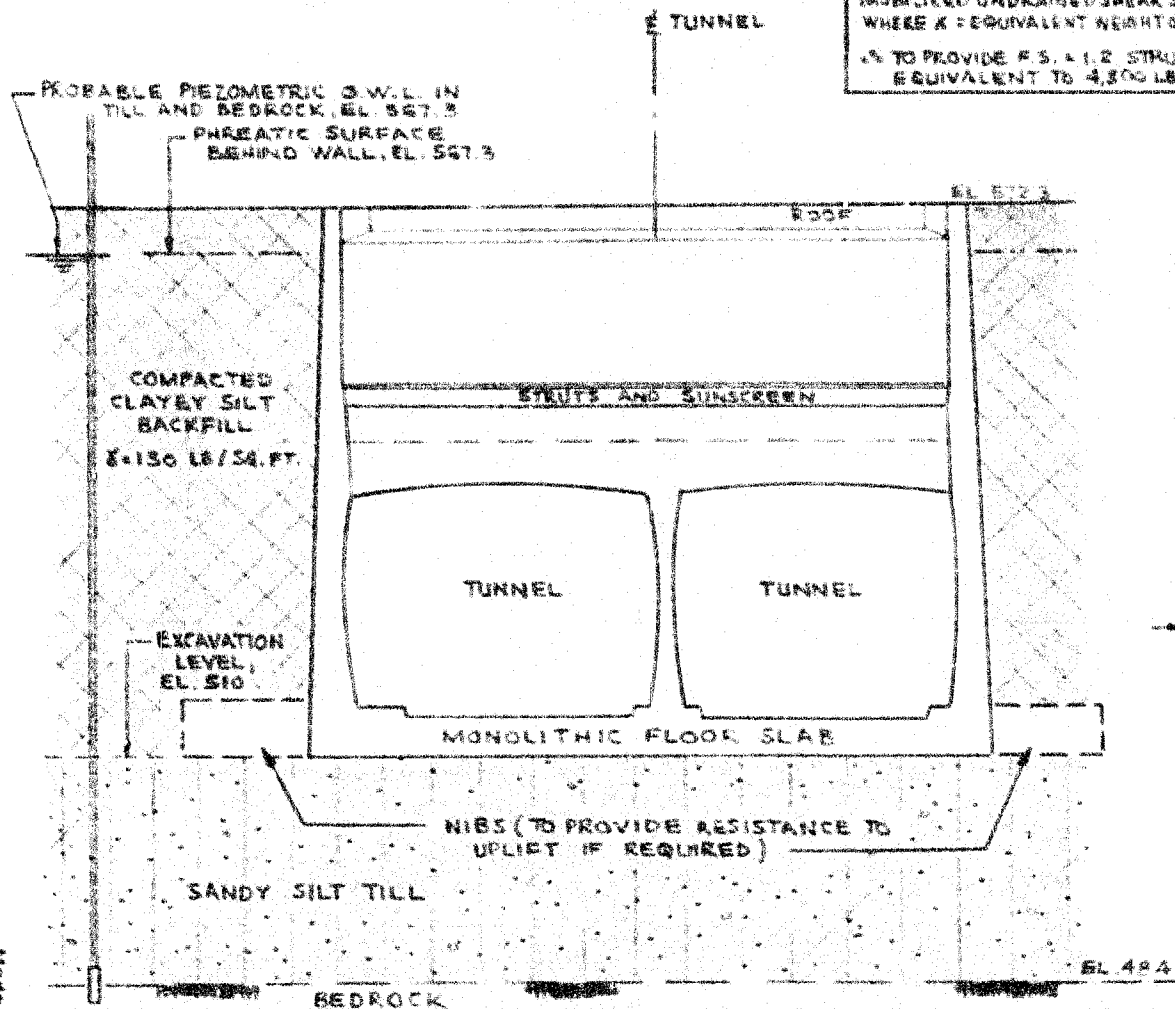


SCALE 1" TO 10'

GOLDER &amp; ASSOCIATES

Made *[Signature]*  
 Chkd *[Signature]*  
 Appd *[Signature]*

GOLDER &amp; ASSOCIATES



# UPLIFT CONDITIONS AT BASE OF FLOOR SLAB:

BASE FLOOR SLAB - EL. 510

UPLIFT PRESSURE AT EL. 510 =  $62.4 (567.3 - 510) = 3,580 \text{ LB/SQ. FT.}$

RESISTING FORCE = WEIGHT OF STRUCTURE

FACTOR OF SAFETY WITH RESPECT TO UPLIFT CAN BE REPRESENTED

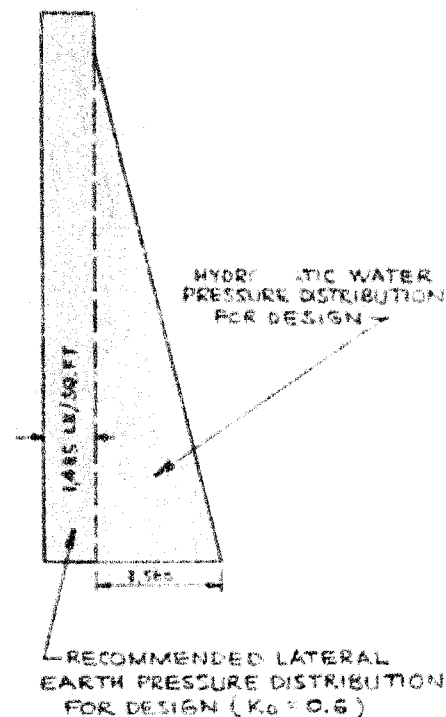
BY THE FOLLOWING EXPRESSION,  $F.S. \text{ UPLIFT} = K / 3,580$  (ASSUMING

MOBILIZED UNDRAINED SHEAR STRENGTH WITHIN CLAYEY SILT FILL,  $C_u = 0$ ,  $\phi = 0$ )

WHERE  $K$  = EQUIVALENT WEIGHT OF STRUCTURE AT BASE SLAB LEVEL, LB/SQ. FT.

IN TO PROVIDE  $F.S. \geq 1.2$  STRUCTURE MUST PROVIDE A WEIGHT EQUIVALENT TO 4,300 LB/SQ. FT. AT BASE SLAB LEVEL.

LATERAL EARTH PRESSURE ON RETAINING WALLS (LB./SQ. FT.)



SCALE 1" TO 20'

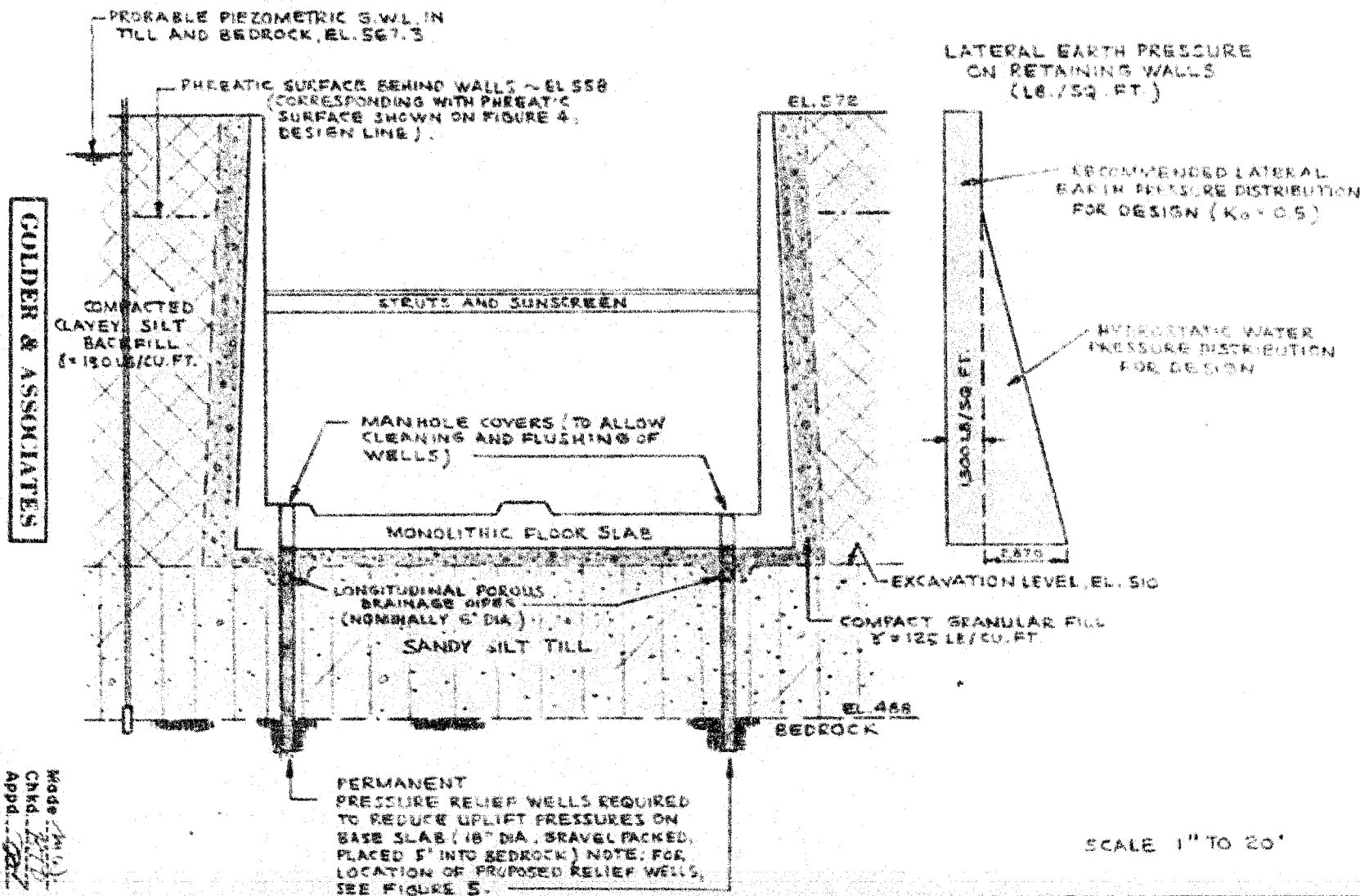
RETAINING WALL SECTION B-B  
MONOLITHIC FLOOR SLAB - NO GRANULAR BACKFILL  
(LATERAL EARTH AND UPLIFT PRESSURES)

FIGURE 7

MADE BY  
CHD  
APPD

RETAINING WALL SECTION C-C  
MONOLITHIC FLOOR SLAB - WITH GRANULAR BACKFILL  
(LATERAL EARTH PRESSURE AND RELIEF WELL SYSTEM)

FIGURE 6

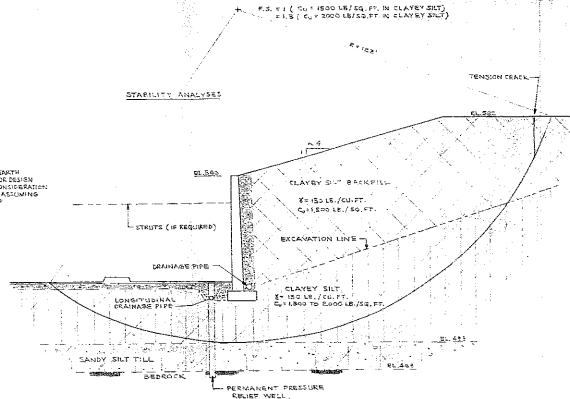
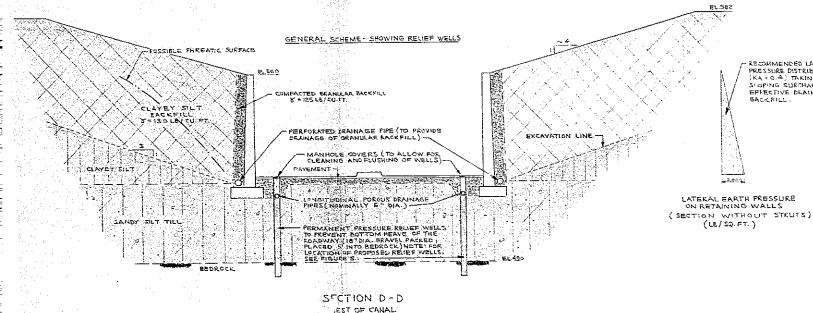


GOLDER & ASSOCIATES

Mode  
CHKD.  
Appd.

SUMMARY OF STABILITY ANALYSES

- 1) IF THE UNDRAINED SHEAR STRENGTH OF THE CLAYEY SILT,  $C_u$ , BETWEEN ELEV. 540 AND 438 IS EQUAL TO 2,000 LB./SQ. FT. OR GREATER THE RETAINING WALL SECTION WITH PAVED FLOOR WILL BE STABLE (F.S.  $\text{min.} = 1.3$ ). THE SCHEME SHOWN IN SECTION D-D WILL THEREFORE BE ACCEPTABLE.
- 2) IF HOWEVER THE UNDRAINED SHEAR STRENGTH OF THE CLAYEY SILT,  $C_u$ , BETWEEN ELEV. 540 AND 438 IS OF THE ORDER OF 1,500 LB./SQ. FT. OR LESS THE RETAINING WALL SECTION, WITH A HEIGHT OF 80 FEET OR MORE, WILL BE IN A STATE OF LIMITING EQUILIBRIUM (F.S.  $\text{min.} = \text{UNITY}$ ). IF THIS IS THE CASE THE FOLLOWING IS SUGGESTED:
  - a) THE STABILITY OF THE OVERALL SECTION CAN BE IMPROVED BY EXTENDING THE LINE OF STRUTS INTO THE SECTION WITH THE PAVED FLOOR. THE STRUTS COULD BE TERMINATED AT THE POINT WHERE THE RETAINING WALL IS 30 FT. IN HEIGHT, AS AN EXTENSION OF APPROXIMATELY 30 FT. OVER THAT PROVIDED (SEE FIGURE 4).
  - b) IN THAT PORTION OF THE RETAINING WALL SECTION REQUIRING STRUTS THE LATERAL EARTH PRESSURE ON THE WALL SHOULD BE DESIGNED USING A COEFFICIENT OF LATERAL EARTH PRESSURE AT REST,  $K_0$ , EQUAL TO 0.5 (FOR A RIGID WALL WITH A SLOPING SURCHARGE).
- 3) THE RESULTS OF IN SITU VANE TESTING AND LABORATORY TRIAXIAL TESTING (SUMMARIZED IN FIGURE 3, REPORT 65124) INDICATE THAT THE UNDRAINED SHEAR STRENGTH RANGES FROM 1,500 TO 2,000 LB./SQ. FT. THIS WILL BE CONFIRMED AT THE TIME OF THE FINAL SUBSURFACE INVESTIGATION.



SCALE 1" = 20'

Drawn, AUG. 14, 1967.

GOLDER & ASSOCIATES

Made  
CHS  
Appr

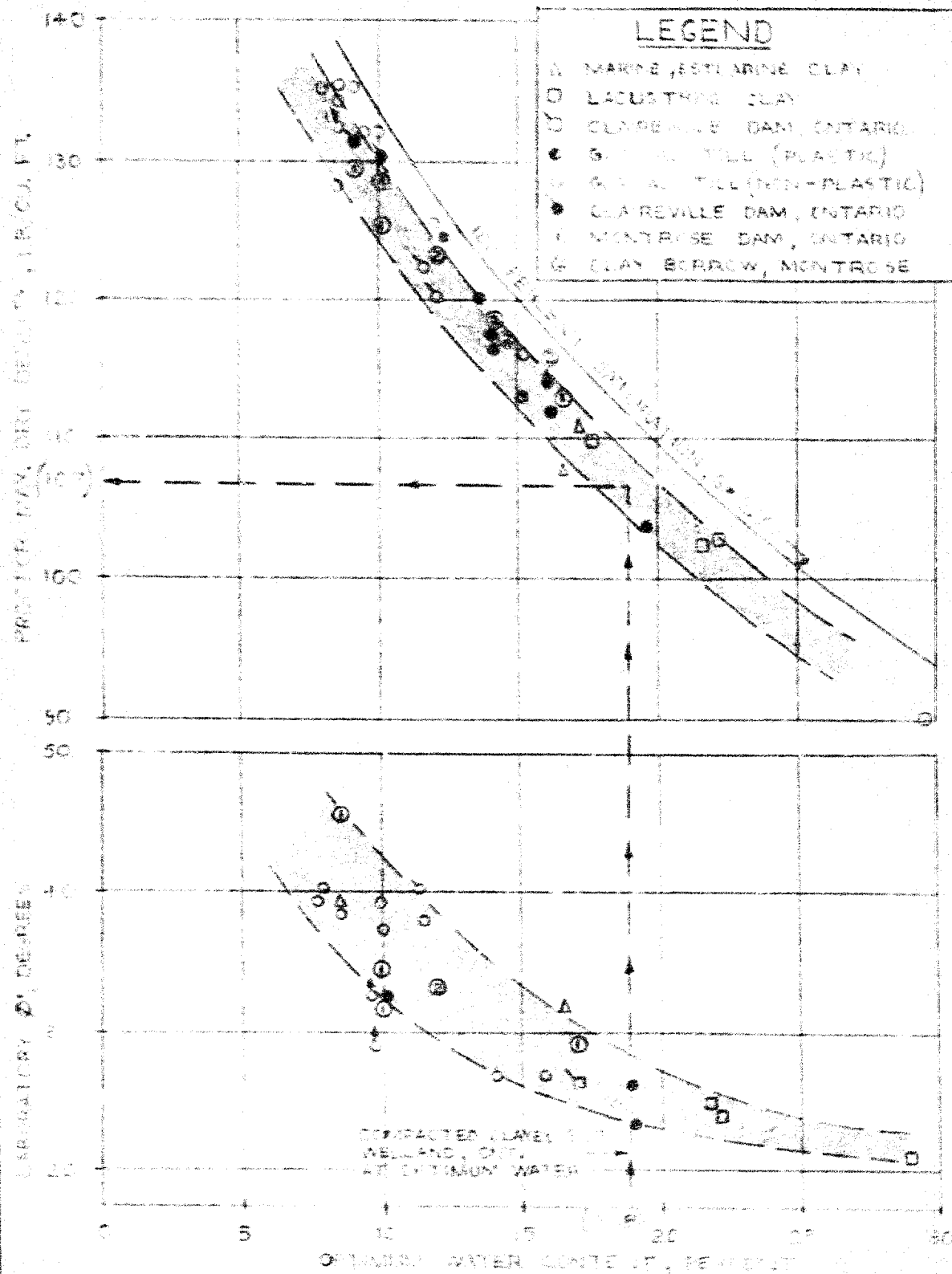


PROJECT No. 4408  
 8/26/58 130-7

# PROPERTIES OF COMPACTED BORROW MATERIAL

COMPACTION TO STANDARD PROCTOR DRY DENSITY AT  
 STANDARD PROCTOR OPTIMUM WATER CONTENT

FIGURE 10



GOLDER & ASSOCIATES

Made R.S.  
 Chkd J.S.  
 Appd J.S.

**H. Q. GOLDER & ASSOCIATES LTD.**

**CONSULTING CIVIL ENGINEERS**

**H. Q. GOLDER  
V. MILLIGAN  
I. G. SODERMAN  
J. L. SEYCHUK**

**2444 BLOOR STREET WEST  
TORONTO 9, ONTARIO  
763-4103  
767-9201**

**W.P. 240-66**

**REPORT**

**TO**

**DEPARTMENT OF HIGHWAYS, ONTARIO**

**ON**

**PRELIMINARY FOUNDATION INVESTIGATION**

**PROPOSED CROSSING OF THE**

**RE-ALIGNED WELLAND CANAL**

**MAIN STREET EAST**

**WELLAND**

**ONTARIO**

**Distribution:**

**14 copies - Department of Highways, Ontario,  
Toronto, Ontario**

**2 copies - H.Q. Golder & Associates Ltd.,  
Toronto, Ontario.**

**May, 1967**

**66134**

## TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	1
INTRODUCTION	2
PROCEDURE	2
SITE AND GEOLOGY	4
SOIL CONDITIONS	6
BEDROCK CONDITIONS	16
GROUNDWATER CONDITIONS	16
DISCUSSION	18
Tunnel Crossing (Open Cut)	19
a) General	19
b) Stability of Tunnel Excavation	20
c) Protection of Cut Slope	24
d) Basal Stability of Cut	24
e) Sequence of Excavation	28
f) Suitability of Excavated Material as Compacted Backfill	29
g) Permanent Approach Cut Slopes	30
Overpass Structure	32
a) General	32
b) Foundations	32
c) Approach Embankments	35
d) Other Considerations	37
ABBREVIATIONS	In Order Following
RECORD OF BOREHOLE SHEETS	Page 39
FIGURES	
1 - Boring Plan and Soil Stratigraphy Section	
2 - Summary Plot of Piezometric Groundwater Levels	
3 - Summary Plot Engineering Properties (Overburden)	
4 - Summary of Grading Results - (Clayey Silt Stratum)	
5 - Plasticity Chart - (Clayey Silt Stratum)	

Cont'd.....

TABLE OF CONTENTS (Cont'd)

FIGURES	6 - Results of Directional Strength Tests (Clayey Silt)
	7 - 9 - Consolidated Undrained Triaxial Test Results (Clayey Silt Stratum)
	10 - 13 - Consolidation Test Results (Clayey Silt Stratum)
	14 - Most Probable Pre-Consolidation Pressure (Clayey Silt Stratum)
	15 - Compaction Test Results (Clayey Silt Stratum)
	16 - Summary of Grading Results (Lower Granular Deposits)
	17 - Summary Results of Stability Analyses (Proposed Tunnel Excavation)
	18 - Summary Results of Basal Stability Analyses
	19 - Summary Results of Stability Analyses (Approach Cut Slopes)

APPENDIX I

FIGURES	20-24 - Grading Curves (Clayey Silt Stratum)
	25-29 - Grading Curves (Silt Deposits)
	30-31 - Grading Curves (Silty Till Deposit)

## ABSTRACT

The results of a preliminary subsurface investigation to assist in the feasibility study of a proposed tunnel crossing of the re-aligned Welland Canal, along Main Street East in the eastern outskirts of Welland, Ontario are reported. As an alternative to the tunnel scheme a bridge structure over the canal is being considered.

The site is flat lying to gently undulating in relief and is underlain by an extensive stratum of very stiff to stiff clayey silt, varying in thickness from 40 to 115 feet. In the western portion of the site a deposit of dilatant silt up to 30 feet thick is sandwiched within the clayey silt stratum. Directly underlying the clayey silt or a thin layer of lower silt is a basal deposit of very dense sandy silt till resting on dolomitic limestone bedrock.

The piezometric groundwater level within the clayey silt and silt deposit was found to be at a depth of between 3 to 8 feet below existing ground surface, while the groundwater level within the till and bedrock is some 25 feet below ground surface.

Stability analyses indicate that the tunnel excavation (maximum depth of cut approximately 105 feet) can be carried out in the dry with side slopes of 3 horizontal to 1 vertical, providing i) the upper portion of the excavation is benched to elevation 580 and ii) drainage benches are placed above and below the silt deposit as discussed in the report. Excavation of the clayey silt can be carried out with scrapers. As the silt is below the piezometric groundwater level it will be necessary, however, to excavate it with a drag line. To ensure basal stability of the excavation, relief wells carried down into the bedrock may be required.

The bridge piers can be founded on spread footings placed within the upper portion of the clayey silt stratum using an allowable bearing value of up to 2 tons/sq.ft. Piers located within or adjacent to the canal should be supported on end-bearing piles driven into the till or on bedrock. Preliminary computations indicate that embankments up to 70 feet in height can be constructed without danger of overall instability. As discussed in the report it may be necessary to reduce the height of the approach embankments so as to limit the differential settlement at the junction with the bridge structure to a reasonable value.

## INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario, letter of authorization dated December 5, 1966, to carry out a preliminary soil investigation to assist in the feasibility study of a proposed tunnel crossing of the re-aligned Welland Canal along Main Street in the eastern outskirts of Welland, Ontario. As an alternative to the tunnel scheme a bridge structure over the canal is being considered.

The purpose of this investigation was to determine the general subsoil and groundwater conditions along the proposed section and to make recommendations for the design and construction of the tunnel section, particularly with respect to the stability of the required deep cut. In addition, information is provided in this report for foundation design and related earthworks pertaining to the construction of a bridge structure.

## PROCEDURE

The field work for this investigation was carried out between December 8, 1966 and January 10, 1967. A total of 9 boreholes (numbered T-1 to T-9) were put down to depths varying from 80 feet to 132 feet. Each boring was started and carried

down to about a 65 to 70 foot depth using a Penndrill employing power auger techniques; the borings were then cased in NX size and completed by a diamond drillrig using wash boring techniques. An additional auger hole (T-7A) was put down to about a 40 foot depth; this hole was left open so as to take water samples for chemical testing. The drilling was carried out by a Penndrill and two diamond drillrigs supplied and operated by the F.E. Johnston Drilling Company Limited.

The overburden was sampled with a two inch diameter split spoon sampler supplemented with 2 and 3 inch diameter thin walled tube samplers in the cohesive portion of the overburden. In situ field vane testing was also carried out within the cohesive strata. Bedrock was proven in four of the borings by diamond core drilling in AXT or BXT size. The piezometric ground-water level was observed during and following the period of the investigation in sealed piezometers installed in all the boreholes. The field work was supervised throughout by an engineer from our staff.

A detailed log for each of the boreholes is given on the Record of Borehole sheets following the text of this report. The location of the borings, together with a section of the

inferred stratigraphy along Main Street East, are shown on Figure 1, while the piezometric groundwater level conditions are summarized on Figure 2.

Samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on the Record of Borehole sheets and on Figures 3 to 16 and on Figures 20 to 31 in Appendix 1. Chemical testing on representative samples of the groundwater was carried out by the Chemical Division of the Department of Highways, Ontario.

The locations of the boreholes and elevations given in this report were obtained by our field personnel. The elevations are referred to Geodetic datum.

#### SITE AND GEOLOGY

The proposed tunnel section, beneath the proposed new Welland Canal, is to be located along Main Street East in the eastern outskirts of the City of Welland, Ontario. The proposed route is located within a suburban residential area. The area is flat lying to gently undulating in relief with the ground surface varying from about elevation 597 to 604. The proposed tunnel section is bounded to the east by an overpass structure



which provides access over the New York Central Railway line.

From available geological information (Chapman, Putman, 1951) and inspection of the area, it is known that the overburden consists of thick deposits of silty clay and clayey silt overlying till, physiographically known as Haldimand Till. The till, which is of Wisconsin age, was covered by the silty clay, which is a lacustrine deposit laid down in glacial Lake Warren. The glacial lake phase was possibly interrupted by two major retreats of the ice front which resulted in distinctively different deposits, non-stratified relatively silty homogeneous deposits laid down with the ice front fairly close, and heavily stratified very clayey deposits laid down when the ice front had retreated some distance. All of the lacustrine deposits, except the upper 30 to 40 feet which is desiccated, are relatively soft and possibly only lightly pre-consolidated. The total thickness of the overburden generally varies from 100 to 120 feet.

The broad clay plain is bounded to the north by the Niagara Escarpment which steps down towards Lake Ontario. To the south the plain is bounded by the toe of the Onondaga cuesta.

The water shed in the area is controlled by the

Onondaga cuesta, which though quite low lying close to the shore of Lake Erie, nevertheless forces the drainage to the north and east. In general the drainage in this primarily flat heavy clay area is quite poor.

The bedrock in the area is Palaeozoic. The beds dip slightly southward under Lake Erie. The bedrock, which generally varies in elevation from 460 to 500, is a massive dolomitic limestone of the Salina formation, Devonian period. There are numerous siltstone and calcareous shale interbeds within the bedrock. In addition the bedrock contains numerous gypsum inclusions from hairline thickness to as much as 12 inches.

#### SOIL CONDITIONS

The detailed soil stratigraphy encountered by the borings is given on the Record of Borehole sheets. An inferred stratigraphic section along Main Street East is shown on Figure 1. The engineering properties of the subsoil are presented on Figures 3 to 16, inclusive. Following is a summarized account of the inferred subsoil conditions along the proposed tunnel section, together with the engineering properties of the overburden.

##### a) Clayey Silt

i) General

Directly underlying the surficial cover at the site, which is generally a topsoil, there is a stratum of reddish brown clayey silt with a trace of some sand and gravel throughout. The thickness of this stratum is quite variable ranging from 40 feet (at B.H. T-1) to 115 feet (at B.H. T-6); in general the encountered thickness is greatest at a point just east of Port Robinson Road (Station 20+00). In the western portion of the route (approximately Station 30+00 to Station 50+00) the stratum is divided by a layer composed of silt and sand; this layer will be discussed separately below.

The upper 11 to 17 feet of the clayey silt is mottled brown in colour indicating that this zone has been desiccated. In the lower portion of the stratum (generally below about elevation 540), particularly in the western half of the tunnel section where the clayey silt underlies the silt layer, there are zones present which have a higher clay content, are more plastic than the overlying soil, and can be classified as a silty clay. Occasional silt partings and seams up to  $\frac{1}{4}$  inch thick occur randomly throughout the stratum; further, occasional layers of silt and sand, up to 1 foot thick are present. In general the gravel content increases with depth.

The gradation of a) the desiccated crust material, b) the clayey silt and c) the more plastic lower zones of the cohesive stratum are summarized in the form of either typical grading curves or as an envelope on Figure 4. All the grading curves obtained for samples from the cohesive stratum are given on Figures 20 to 24 inclusive, in Appendix I.

Atterberg limit tests were carried out on samples of the clayey silt. The results are shown on the Record of Borehole sheets and are also summarized on Figures 3 and 5. The test results are summarized in tabular form below:

	Liquid Limit ( $W_L$ ) <u>Range (Average)</u>	Plasticity Index ( $I_p$ ) <u>Range (Average)</u>	Liquidity Index ( $I_L$ ) <u>Range (Average)</u>
Desiccated "Crust"	31 to 47 (37)	15 to 27 (20)	0 to 0.3 (0.2)
Clayey Silt	18 to 40 (25)	7 to 24 (12)	0.1 to 0.7 (0.4)
Lower more plastic zones within stratum	31 to 55 (40)	15 to 33 (22)	0.5 to 0.8 (0.7)

These results indicate that the "crust" and the more plastic zones of the stratum encountered at depth can be considered to be of medium plasticity, while the remainder of the stratum is generally of low plasticity. These results are typical of inorganic glacial clays and silts. The total i.e. wet

unit weight of the clayey silt was found to vary from 127 to 145 lb/cu.ft. being typically about 135 lb/cu.ft. However, the lower more plastic zones within the stratum have a total unit weight ranging from about 112 lb/cu.ft. to 125 lb/cu.ft.

#### ii) Strength and Compressibility of Clayey Silt

The undrained shear strength of the clayey silt was measured by in situ vane testing in the field and by laboratory undrained triaxial tests. The results of these tests are plotted on the Record of Borehole sheets and are summarized on Figure 3. The results indicate that the undrained shear strength in the desiccated crust is consistently greater than 3,000 lb/sq.ft. Below the desiccated zone the undrained shear strength varies from about 1,500 lb/sq.ft. to 2,500 lb/sq.ft. being about 2,000 lb/sq.ft. Samples tested from the lower more plastic zones generally gave undrained shear strengths which lie in the lower range given above. In general the triaxial tests gave lower shear strength values than the in situ field vane tests. This is as would be expected since the majority of the Shelby tube samples had to be either driven or pushed by the application of large non-uniform pressures. This would lead to some unavoidable sample disturbance resulting in lower undrained shear strengths in the laboratory due to the remoulded

condition of the clayey silt samples. An indication of the disturbance is the consistently high strain at failure (generally in excess of 10 percent). It is, therefore, concluded that the in situ vane test results provide a better indication of the in situ consistency of the stratum. The undrained triaxial testing, however, provides the lower limiting range. Based on the above it is estimated that the consistency of the clayey silt varies from very stiff to stiff with the more plastic zones being in the lower consistency range. The upper desiccated crust is estimated to be hard.

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

Standard penetration tests were also carried out within the stratum; the results of which are summarized on Figure 3. In general the pattern of "N" values with depth corroborates the shear strength profile.

The directional shear strength properties of the clayey silt were determined by carrying out undrained shear strength tests on samples obtained from 3 inch diameter thin walled tube samples. Three samples were cut from each of four

tubes; the samples were cut a) vertically, b) horizontally and c) at 45 degrees to the vertical. The samples were then failed in an undrained state by increasing the axial load parallel to the longitudinal axis. The results of the four sets of tests are plotted on Figure 6. With the exception of one of the test sets (B.H. T-1, Sa.8), all the tests indicate that the sample cut vertically from the tube gave the highest value for the undrained shear strength while the sample cut horizontally gave the lowest value. The samples cut at 45 degrees gave an undrained shear strength intermediate in value between the above.

The directional shear strength properties of the cohesive strata encountered in the Welland area are discussed in detail in our report 6375, dated July, 1964. As indicated in this report a decreasing pattern of undrained shear strength with orientation (measured from the vertical) typifies a non-stratified isotropic material. Thus the testing discussed above indicates that the deposit is basically isotropic. However, there may be locally stratified zones of limited extent, particularly in the lower more plastic zones of the deposit. Such zones would be anisotropic in nature. The pattern of undrained shear strength with orientation for anisotropic soil, as outlined in the previous report, is

similar to the pattern obtained from the testing carried out on sample 8, borehole T-1.

In order to determine the long or intermediate term stability of the cut slopes for the tunnel excavation and approaches, it is necessary to determine the effective shear strength parameters of the clayey silt stratum. It is also necessary to estimate the pore water conditions both during and following construction of the approach cuts. To determine the effective shear strength characteristics of the stratum three single stage consolidated undrained triaxial compression tests with pore water pressure measurements were carried out. The results of these tests are plotted on Figures 7 to 9, inclusive, using the method suggested by Rendulic (1937) and also using the conventional Mohr circle plot. The results obtained indicate that in the upper portion of the stratum, the effective angle of shearing resistance  $\phi'$ , can be considered to be about 28 degrees in conjunction with an effective cohesion,  $C'$ , of about 200 lb/sq.ft. An effective cohesion as high as this value is indicative of a subsoil preconsolidated with respect to existing overburden pressure. In the lower more cohesive zones of the stratum the effective angle of shearing resistance,  $\phi'$ , is about 25 degrees with the effective cohesion,  $C'$ ,



approximately equal to zero.

Four laboratory consolidation tests were carried out on samples of the stratum; the results of this testing are presented on Figures 10 to 13, inclusive, as a plot of void ratio,  $e$ , versus the logarithm of pressure. These results, which are summarized on Figure 14, indicate that the stratum is preconsolidated by up to 4 tons/sq.ft. in excess of existing overburden pressure in the upper portion of the cohesive stratum. At depth the stratum is normally consolidated to slightly pre-consolidated.

A standard Proctor compaction test, shown on Figure 15, was carried out on a composite sample of the clayey silt from the upper 20 feet of the stratum. The optimum dry density is about 107 lb/cu.ft. at an optimum compaction water content of 19 percent.

#### Silt and Fine Sand

In the western portion of the tunnel section (west of Station 28+00) there is a deposit of dilatant silt to sandy silt sandwiched within the clayey silt stratum. The silt deposit has a maximum thickness of about 32 feet to the west, generally decreasing in an easterly direction to a minimum

thickness of about 10 feet. Grading curves for typical samples of the silt deposit are shown on Figures 25 to 29 inclusive, in Appendix I; these grading results are summarized on Figure 16. The in situ water content of the deposit generally varies from 9 to 25 percent, being typically about 15 percent.

The results of standard penetration tests carried out within the silt deposit are given on Record of Borehole sheets and are summarized on Figure 3. Referring to this figure it can be seen that the "N" values range between 12 blows/ft. to greater than 100 blows/ft.; in general, however, they are greater than 50 blows/ft. The lower "N" values obtained are no doubt due to disturbance of the silt in the uncased auger holes. Based on the above it is considered that the relative density of the silt is very dense with localized dense zones.

A dense deposit of uniform reddish brown silt to sandy silt underlies the clayey silt stratum and caps the granular till at a few of the boring locations. The thickness of this deposit, where encountered, varies from 6 to 10 feet. Grading curves for two samples of the lower silt are given on Figure 29 in Appendix I. Reference to this figure shows that

the gradation of the lower silt layer is markedly similar to the silt deposit encountered at a higher elevation.

### Sandy Silt Till

Underlying the clayey silt stratum or the lower silt layer is a reddish brown generally well graded sandy silt till. The till was penetrated fully only at 4 of the boring locations; at these locations the thickness of the till varies from less than 1 foot (B.H. T-6) to as much as 40 feet (B.H. T-4). In general the till is very bouldery with the boulder content increasing with depth. Grading curves for representative samples of the till (obtained using 1½ inch I.D. sampling equipment) are given on Figures 30 and 31 in Appendix I and are summarized in envelope form on Figure 16. The in situ water content of the till deposit was found to vary from 8 to 25 percent, being typically about 12 percent.

Standard penetration tests carried out within the sandy silt till gave "N" values ranging from about 25 blows/ft. to greater than 100 blows/ft. and generally greater than 50 blows/ft. Based on these values it is considered that the till is very dense.

### BEDROCK CONDITIONS

Bedrock was proven for at least 10 feet by diamond core drilling in 4 of the boreholes put down in the tunnel section. Grey, basically massive, dolomitic limestone bedrock was encountered in these borings beneath the till sheet, i.e. below about elevations 482 to 492. The upper 3 to 5 feet of the bedrock is weathered and fractured while below this zone the rock is quite sound.

The bedrock has numerous inclusions of gypsum. Black fissile shale interbeds occur randomly throughout; such interbeds generally vary from 2 to 10 inches in thickness, however, a zone of shale some 6 feet thick was encountered at borehole T-4.

### GROUNDWATER CONDITIONS

The groundwater conditions at the site were determined during and following the period of the investigation by readings taken in sealed piezometers installed in the various strata encountered in the boreholes. The piezometric groundwater level readings taken between January and March, 1967, showed little fluctuation.

The final set of readings taken in the piezometer

installations are shown on Figure 2 and are summarized in tabular form below.

Location of Piezometer	OBSERVED PIEZOMETRIC GROUNDWATER LEVELS	
	Depth Below Ground Surface Range (Average)	Elevation Range (Average)
Upper Piezometers (located within clayey silt stratum)	1' to 5' (3')	595 to 603 (598)
Intermediate Piezometers (located within silt deposit)	3' to 22' (8')	582 to 602 (593)
Lower Piezometers (located within till or bedrock)	24' to 29' (26')	574 to 578 (576)

Based on these readings it is concluded that the piezometric groundwater level in the upper deposits are generally within 6 to 7 feet of ground surface. The piezometric groundwater level within the more pervious till and underlying bedrock, however, is some 25 feet below ground surface. This decrease in piezometric level with depth is indicative of downward drainage within the overburden.

It is recommended that further readings be taken in the piezometers to determine any seasonal fluctuation in the piezometric level.

### Water Analyses

To determine the degree of corrosivity of the groundwater with respect to normal Portland cement concrete, water samples, taken in the boreholes, were subjected to chemical testing to determine the proportion of soluble sulphates. This testing was carried out by the Department of Highways, Ontario. The results of these analyses are summarized in tabular form below.

Borehole	Depth of Water Sample (ft. Below Ground Surface)	Sulphate Content (SO <sub>4</sub> ) (P.P.M.)
T-1	20	3095
T-1	35	4715
T-1	54	1865
T-2	18	1041
T-3	10	168
T-3	34	244
T-3	35	195
T-7A	10	640
T-7A	14	565
T-7A	22	562
T-8	31	202
T-8	40	208

### DISCUSSION

The existing Welland Canal is to be re-aligned between Port Robinson on the north and Ramey's Bend on the south. In the vicinity of Welland, the re-alignment is to be located in the eastern outskirts of the city. A proposed cross-section of the re-aligned canal is shown on Figure 1.

It is understood that a crossing of the canal is proposed at Main Street East. At the present time consideration is being given to a tunnel crossing. As an alternative a bridge overpass structure is a possibility. The soil mechanics aspects pertaining to both of the above solutions are discussed separately below.

#### Tunnel Crossing (Open Cut)

##### (a) General

It is understood that the crossing is to provide 4 lanes of traffic , each lane about 12 feet wide. The width of the tunnel section is, therefore, to be about 50 to 60 feet. The proposed tunnel section, shown on Figure 1, indicates that the approach grade is about 6 percent with the lowest point of the tunnel invert at about elevation 507. With the base thickness of the tunnel being about 7 feet the deepest part of the excavation would be carried down to approximately elevation 500.

It is understood that the tunnel is to be constructed in an open excavation prior to the construction of the canal proper. Further, it is understood that up to 2 years will be required to complete the construction of the tunnel. Following construction of the tunnel and, prior to the construction of the

canal, the excavation is to be at least partially backfilled. The suitability of the excavated clayey silt as possible backfill material is considered in this report.

#### b) Stability of Tunnel Excavation

To reach subgrade level the excavation will be carried through the clayey silt stratum, including the silt layer sandwiched within the clayey silt. In the central portion of the tunnel section the invert will be within the dense till deposit while on either end at the portals the invert will be within the clayey silt or silt. The maximum depth of cut will be approximately 104 feet. One of the most important factors in determining the feasibility of the overall scheme is the stability of the cut slopes. With this in mind stability computations were carried out utilizing the computer of the Department of Highways, Ontario, supplemented by manual computations.

Most of the cut will be within the clayey silt stratum. An analysis using the total stress approach (undrained shear strength of the clayey silt, i.e.  $\phi = 0$  case) is a valid method for predicting the initial stability of the cut (end of construction case). The total stress analysis does not



consider, however, the changes in pore water pressure within the slope during and following excavation. In considering the long or intermediate term stability in cuts carried out in over consolidated cohesive deposits, such as are present on this site, large errors can be introduced if the pore pressure changes are not considered. Therefore, in order to evaluate the factor of safety for this condition, the stability has to be analysed using the effective stress approach incorporating pore pressures.

Since the construction period is likely to be of the order of 2 years, i.e. approaching a long term stability case, both types of analyses were carried out. The computations considered cuts with side slopes varying between 2 and 3 horizontal to 1 vertical; provision was also made for two drainage benches at the interface of the clayey silt and sandy silt deposit. The factor of safety obtained for cuts with slopes steeper than 3 horizontal to 1 vertical is below acceptable limits. Therefore, the discussion which follows summarizes only the analyses for cuts with side slopes of 3 horizontal to 1 vertical.

(i) Total Stress Analysis

A profile of the undrained shear strength measured

in the cohesive stratum at the site is given on Figure 3. Based on this profile an undrained shear strength varying from 2,000 lb/sq. ft. in the upper portion of the clayey silt stratum to 1,500 lb/sq. ft. at depth was used to carry out a series of total stress stability analyses; the strength values used are summarized on Figure 17a. The results of these computations, also summarized on Figure 17a, indicate that the minimum factor of safety is 1.2 and the most probable is about 1.3 for the maximum depth of cut proposed.

#### ii) Effective Stress Analyses

The effective stress parameters of the cohesive stratum, obtained from laboratory testing are summarized on Figures 7 to 9, inclusive. The parameters used in the computations, together with the assumed upper phreatic surface, are presented on Figure 17b. Based on these parameters, the minimum factor of safety for the long term stability case is computed to be unity.

The results of the effective stress analyses indicate that the factor of safety of the cut, with 3 horizontal to 1 vertical side slopes, is marginal and below acceptable limits. One method of improving the stability would be to provide a bench at the top of the cut, i.e. unload the slope.

Computations carried out indicate that by benching down to elevation 580, as shown on Figure 17b, the factor of safety could be increased to about 1.2.

In the above computations the phreatic surface was taken along the face of the cut slope for the benched case, and as shown on Figure 17b for the 3 horizontal to 1 vertical slope without a bench. This is considered to represent the most severe piezometric condition. Consequently the factor of safety given above should indicate the lower limit of stability.

Considering that there should be some drawdown of the groundwater in the excavation, the phreatic surface would be lowered below the face of the cut slope, particularly in the upper portion of the excavation. This would improve the overall stability of the excavation side slopes to a value in excess of 1.2 for the benched case. It is therefore concluded that the side slopes of the excavation for the tunnel section can be taken as 3 horizontal to 1 vertical for preliminary design purposes, together with provision of an upper bench as shown on Figure 17b and the lower benches above and below the silt deposit.

### c) Protection of Cut Slope

The stability computations discussed above cover deep seated or overall stability of the cut. Since the cut will remain open for a considerable period of time the slopes will have to be protected against surficial instability. Surficial instability could be caused by a number of factors including erosion due to run-off or groundwater seepage, or to physical changes within a surficial zone caused by seasonal freeze-thaw cycles, etc. <sup>to allow for a temporary</sup> ~~It is recommended that a drainage~~ <sup>proposition</sup> ~~blanket be placed on the slope; this blanket should be composed of free-draining granular material.~~ The thickness of the blanket can be taken as 3 feet over the silt and 2 feet over the clayey silt, for preliminary design. In addition positive drainage measures should be provided to control surface run-off. One of the best methods of accomplishing this would be to provide lateral drainage trenches, about 4 feet deep, along the benches. A perforated pipe about 6 inches in diameter, surrounded by a filter sand and gravel, should be placed in this trench. The trench then could be brought up to grade with properly compacted free-draining granular material.

### d) Basal Stability of Cut.

As discussed above, the excavation will be carried

through the clayey silt (including silt layer) and into the sandy silt till, terminating at about elevation 500. The piezometric groundwater levels encountered across the site are plotted on Figure 2. The subsoil and piezometric groundwater conditions pertinent to basal heave considerations are summarized on Figure 18.

The governing factors as far as basal stability of the excavation is concerned are:

- i) the relatively high piezometric groundwater level.
- and ii) the fact that a bouldery till deposit and weathered and fissured bedrock underlie a cohesive deposit at depth.

Reference to the grading curves for samples of the till (Figures 30 and 31) indicates that the till matrix is relatively impermeable. As mentioned previously the till contains numerous cobbles and boulders, particularly with depth. There could be more pervious zones within the till deposit. These are known to occur at other points in the vicinity. Further, the upper portion of the bedrock is known to have a higher permeability than the overlying soils. With this in mind basal heave of the excavation for the tunnel

should be taken into consideration. To this end basal stability computations were carried out, the results of which are presented on Figure 18.

The computations indicate that to ensure basal stability as the glacial till is approached, the excavation could be taken down to only elevation 550 ( a minimum acceptable factor of safety of 1.3 was used in these computations). In order to permit excavation in the dry below this elevation it would be necessary to relieve the hydrostatic pressure differential within the till stratum.

For the preliminary design stage it is suggested that provision be made to relieve the hydrostatic pressure at depth by relief wells. To be effective the relief wells would have to be carried down into bedrock. The relief wells should be about 6 inches in size and the initial spacing could be taken as about 20 feet. The relief wells should be put down using a cased hole and flushed with clean water. The backfill or filter material should be free-draining. Clean pea gravel would be suitable for this purpose.

The effectiveness of the relief well system can only be determined by the monitoring of piezometers as the

excavation proceeds. In this regard additional piezometer installations will be required. The piezometers presently in place as well as any additional piezometers installed, should be protected to ensure that they are operating properly. If the piezometers indicate that the groundwater level is not being lowered sufficiently it may be necessary to decrease the spacing of the wells by providing additional ones.

Once excavation level is reached a granular drainage blanket some 3 feet thick should be placed on the till or clayey silt subsoil as the case may be. Continuous drainage from the drainage blanket and relief wells should be maintained until the weight of the constructed tunnel section and backfill is large enough to balance the hydrostatic uplift pressure. This could be accomplished by placing perforated drainage pipes down into the relief wells and into the drainage blanket. The pipes should be carried through the base of the tunnel if necessary. Once the weight is enough to counter balance the full hydrostatic uplift pressure these pipes can be capped and sealed.

A detailed study of the permeability characteristics of the till and upper portion of the bedrock should be carried

out if the tunnel scheme is adopted to determine whether a pressure relief system will be required during the construction excavation phase.

e) Sequence of Excavation

Since the clayey silt is at or slightly above the plastic limit, particularly in the upper portion, scrapers could readily be used to excavate this stratum. The water bearing silt layer, however, will present a more difficult excavation problem. It is considered that a scraper operation will not be possible since the dilatant silt below the groundwater level will become loosened due to upward seepage and run into the open excavation. As the silt is borderline between a material that will drain only slowly under gravity or even under vacuum, lowering of the groundwater level in the silt prior to excavation would require the use of electro-osmosis in order to allow its excavation by scrapers. This is an expensive de-watering system and its use would make the tunnel scheme uneconomical.

Excavation in the silt can be carried out by a dragline operation. This would involve digging in the central portion of the section and allowing the sides of the excavation



to come to whatever slope proves stable in the field. This slope may be as flat as 7 horizontal to 1 vertical. Drainage of the central cut should be maintained from sumps and trenches. At a later stage, when the excavation is lower and below the silt, it may be possible to re-cut and gradually steepen the side slopes in the silt to 3 horizontal to 1 vertical. Lateral drains will probably be required on the bench at the bottom of the silt deposit as discussed previously.

f) Suitability of Excavated Material as Compacted Backfill

A standard Proctor compaction test carried out on a composite sample of the clayey silt (Figure 15) indicates that the optimum compaction water content is about 18 percent. The in situ water content within the upper 40 to 50 feet of the stratum varies between about 15 and 21 percent (Figure 3). The significance of this is that the clayey silt is generally within 1 or 2 percent of the optimum compaction water content and thus could most probably be satisfactorily compacted in the field. It is stressed, that due to the high silt content which does not make the clayey silt an ideal material for compaction purposes, strict water content control will be required during field placing and compaction operations if a high density is to be obtained.

Precautions should also be taken to prevent an increase in water content of the clayey silt due to precipitation or other external sources of water. Thus if the material is to be stockpiled it should be graded to allow for drainage. Because of its blocky structure, the clayey silt may have to be pulverized or disced prior to spreading and compaction to ensure uniform compaction.

g) Permanent Approach Cut Slopes

The approaches to the tunnel section will be an open cut. The maximum height of the cut slopes is to be of the order of 60 feet. Excavation for the approaches will be carried out mainly through the clayey silt stratum, however, along the western approach the cut extends into the silt layer. The long term stability of the cut slopes must be ensured. For this reason effective stress stability analyses, in addition to total stress analyses, were carried out.

A section of one of the deepest cuts is shown on Figure 19. Stability analyses were carried out for a section with side slopes of 3 horizontal to 1 vertical; two drainage benches, some 10 feet wide, are provided on the slope. The strength parameters, together with the assumed upper phreatic

surface, used in the computations are given on Figure 19.

The computations show that the factor of safety, both for the total and effective stress analyses, is about 1.3. It is, therefore, recommended that side slopes not steeper than 3 horizontal to 1 vertical be used in the preliminary design of the approach cuts.

The surficial stability of the cut slopes will have to be ensured. In this regard the measures outlined in section c) above should be applied to the permanent cut slopes. In addition the permanent slopes should be sodded and mulched to protect the granular drainage blanket.

The most westerly approach cut to the tunnel, and the proposed canal will extend into the silt deposit as shown on Figure 1. Under these conditions the silt deposit will provide direct communication for the water in the canal to enter the most westerly approach cut. Computations carried out indicate that the expected quantity of seepage through the silt should be minor and could be controlled by provision of normal drainage measures such as ditches, interceptor drains and a granular filter blanket.

### Overpass Structure

#### a) General

An alternative to the tunnel scheme is to build an overpass structure over the canal. It is understood that a clearance of about 135 feet is required above normal water level to accommodate ocean going freighters. With the normal water level in the canal at about elevation 569 (see Figure 1) the minimum elevation of the bridge deck over the canal would have to be about 705. It is further understood that for preliminary design the roadway grade is of the order of 6 percent and that the bridge structure is to be approached on either side by earth fill embankments. At the present time no additional design information is available.

The discussion which follows is of necessity of a general nature, with the main purpose being to aid in assessing the feasibility of a bridge scheme with respect to the tunnel scheme discussed above.

#### b) Foundations

The significant foundation subsoil stratum across the site is the very stiff clayey silt. As discussed above the undrained shear strength of the stratum varies from greater than

2,000 lb/sq.ft. in the crust to about 1,750 lb/sq.ft. with depth (see Figure 3). Further, the consolidation testing carried out indicates that the stratum is pre-consolidated for a depth of about 60 feet; the pre-consolidation pressure being the largest near ground surface (see Figure 14).

Based on the above it is considered that the bridge piers can be supported on spread footings founded within the clayey silt, preferably as high up in the stratum as possible so as to take advantage of the high strength of the crust. For computational purposes it is assumed that the footings would be about 100 feet long by 20 feet wide in plan and that they would be founded at a depth of about 10 feet below ground surface. This depth would satisfy frost protection requirements in this area where at least 5 feet of earth cover is required for foundations. Based on the range in undrained shear strength given above, an allowable bearing value of up to 2 tons/sq.ft. can be used in footing design, provided the foundation subsoil is not disturbed during construction. In this regard to prevent softening and remoulding of the clayey silt due to entrance of surface water and construction operations, the base of all footing excavations, once foundation grade is reached, should be immediately covered by a thin mat of lean concrete.

Settlement computations were carried out for rigid spread footings using the results of the consolidation tests shown on Figures 10 to 14, inclusive, to predict the amount of consolidation of the underlying clayey silt that would occur due to the effective footing load. The testing indicates that the effective footing load does not exceed the pre-consolidation load of the stratum. Therefore the limits of consolidation settlement can be computed on the basis of over-consolidation throughout the full depth of the strata.

As determined from the pressure-void ratio relationships, an average coefficient of compressibility,  $C_R$ , of 0.04 can be used in the settlement analyses within the limits discussed above. The settlement of a footing imposing a bearing pressure of 2 tons/sq.ft. on the foundation subsoil, would be of the order of  $1\frac{1}{2}$  to 2 inches. Approximately  $\frac{1}{2}$  inch of this settlement would be elastic in nature occurring during construction.

A total settlement of 2 inches would likely be above tolerable limits for a high, settlement sensitive structure, considering the differential movements that could take place. Therefore, it is recommended that, for preliminary

design of spread footings, a net allowable bearing value of  $1\frac{1}{2}$  tons/sq.ft. be used. Computations carried out for a footing imposing such a loading indicate that the total settlement would be of the order of 1 inch which should be within tolerable limits.

The bridge piers, within the confines of the canal and adjacent to the canal banks, should be pile supported. Because of the moderately sensitive nature of the cohesive stratum, non-displacement type piles are recommended in order to minimize disturbance and a decrease in strength of the clayey silt. Steel H-piles driven to practical refusal in the lower portion of the very dense glacial till stratum or in the underlying bedrock are recommended. For a 12 inch steel H-section an allowable load of 75 tons/pile may be used in design.

c) Approach Embankments

Preliminary stability computations were carried out to determine the safe height of embankment that can be placed on the clayey silt subsoil assuming an average undrained shear strength of 2,000 lb/sq.ft. These computations indicate that an embankment of the order of 70 feet in height would be stable with respect to overall deep-seated failure provided,

- i) the embankment is constructed of well compacted granular material.
- and ii) the side slopes are no steeper than 2 horizontal to 1 vertical.

The net increase in vertical stress within the clayey silt subsoil at depth, due to an embankment loading some 70 feet high, would exceed the pre-consolidation pressure of the stratum. Preliminary computations carried out, under these conditions, indicate that the settlement of the embankment could be between 1 to 1½ feet. To minimize differential settlement between the embankment and bridge structure the embankment height would have to be limited. For preliminary design it is suggested that the embankment height be limited to approximately 35 feet; at this height settlement of the embankment due to consolidation of the underlying clayey silt would be of the order of 3 inches. It would be advantageous to induce as much of this settlement as possible prior to the construction of the bridge structure so as to reduce the differential settlement discussed above. In this regard it is recommended that the embankment be constructed as soon as possible prior to the structure. Consideration could also be given to surcharging the embankment in order to induce



consolidation settlement within the underlying cohesive subsoil. Providing the surcharge height is controlled there will be no problem as far as the overall stability is concerned.

Consideration should be given to placing the bridge abutments on spread footings within the approach embankments. This would eliminate differential settlement between the bridge and embankment. For stub abutments placed in well compacted granular fill an allowable bearing value of about  $1\frac{1}{2}$  tons/sq.ft. could be used in design. Allowance will however have to be made in design to accommodate several inches of differential settlement between the bridge abutment and adjacent pier.

#### d) Other Considerations

In the above discussion it was assumed that the canal banks would be inherently stable. Failure of the canal banks could detrimentally affect the bridge structure. It is understood that the side slopes along the new canal are to vary from 3 to 4 horizontal to 1 vertical depending on the variation of soil conditions along the canal. At the location of the crossing the maximum depth of the canal will be of the order of 65 feet. The results of this investigation indicate

that a canal of this depth, with side slopes no steeper than 3 horizontal to 1 vertical, should have sufficient long term stability, since the lower portion of the canal banks will be submerged below about elevation 569 (see Figure 1).

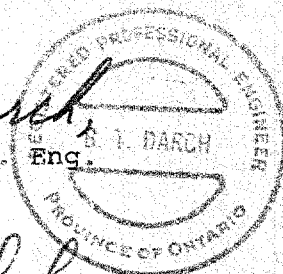
A rapid drawdown of the canal at a later date could detrimentally affect the stability of the bank. It is understood, however, that this section of the proposed canal will never be drawn down, as is the case with the existing canal in the vicinity of Welland. If an overpass structure is eventually employed it will be imperative to confirm whether this will in fact be the case.

The results of chemical analyses of the groundwater obtained from the borings indicate that the maximum soluble sulphate concentration in the groundwater is approximately 3100 p.p.m., while the minimum is 200 p.p.m. It is recommended that further sampling and testing be carried out during the final investigation phase so as to substantiate the range of concentration. If the values in any area consistently exceed 1,000 p.p.m. ordinary Portland cement concrete will be at . In such areas it is, therefore, suggested that a mud coat be provided at the base of the footings

and that the sides of the footings be lined with asphalt as a protective measure. Alternatively a sulphate resistant cement could be used.

*B.T. Darch*

B.T. Darch, P. Eng.



*J.I. Seychuk*

J.I. Seychuk, P. Eng.

BTD/je

66134

May 9, 1967

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

### I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

### II. PENETRATION RESISTANCES

**Dynamic Penetration Resistance:** The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

**Standard Penetration Resistance, *N*:** The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH	sampler advanced by static weight—weight, hammer
PH	sampler advanced by pressure—pressure, hydraulic
PM	sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Relative Density	<i>N</i> , blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils

Consistency	<i>c<sub>u</sub></i> , lb./sq. ft.
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

### IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer <sup>1</sup>
Q	undrained triaxial <sup>2</sup>
R	consolidated undrained triaxial <sup>2</sup>
S	drained triaxial
U	unconfined compression
V	field vane test

#### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>Undrained triaxial tests in which pore pressures are measured are shown as  $\bar{Q}$  or  $\bar{R}$ .

## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 3.1416
$e$	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of $a$
$\log_{10} a$ or $\log a$	logarithm of $a$ to base 10
$t$	time
$g$	acceleration due to gravity
$V$	volume
$W$	weight
$M$	moment
$F$	factor of safety

### II. STRESS AND STRAIN

$u$	pore pressure
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	unit weight of submerged soil
$G_s$	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
$e$	void ratio
$n$	porosity
$w$	water content
$S_r$	degree of saturation

#### (b) Consistency

$w_L$	liquid limit
$w_P$	plastic limit
$I_P$	plasticity index
$w_S$	shrinkage limit
$I_L$	liquidity index = $(w - w_P) / I_P$
$I_C$	consistency index = $(w_L - w) / I_P$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$D_r$	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

#### (c) Permeability

$h$	hydraulic head or potential
$q$	rate of discharge
$v$	velocity of flow
$i$	hydraulic gradient
$k$	coefficient of permeability
$j$	seepage force per unit volume

#### (d) Consolidation (one-dimensional)

$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$C_r$	coefficient of consolidation
$T_v$	time factor = $c_v t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_i$	sensitivity

\*For the case of a saturated cohesive soil,  $\phi_u = 0$  and the undrained shear strength  $\tau_f = c_u$  is taken as half the undrained compressive strength.

## RECORD OF BOREHOLE T-1

LOCATION

See Figure

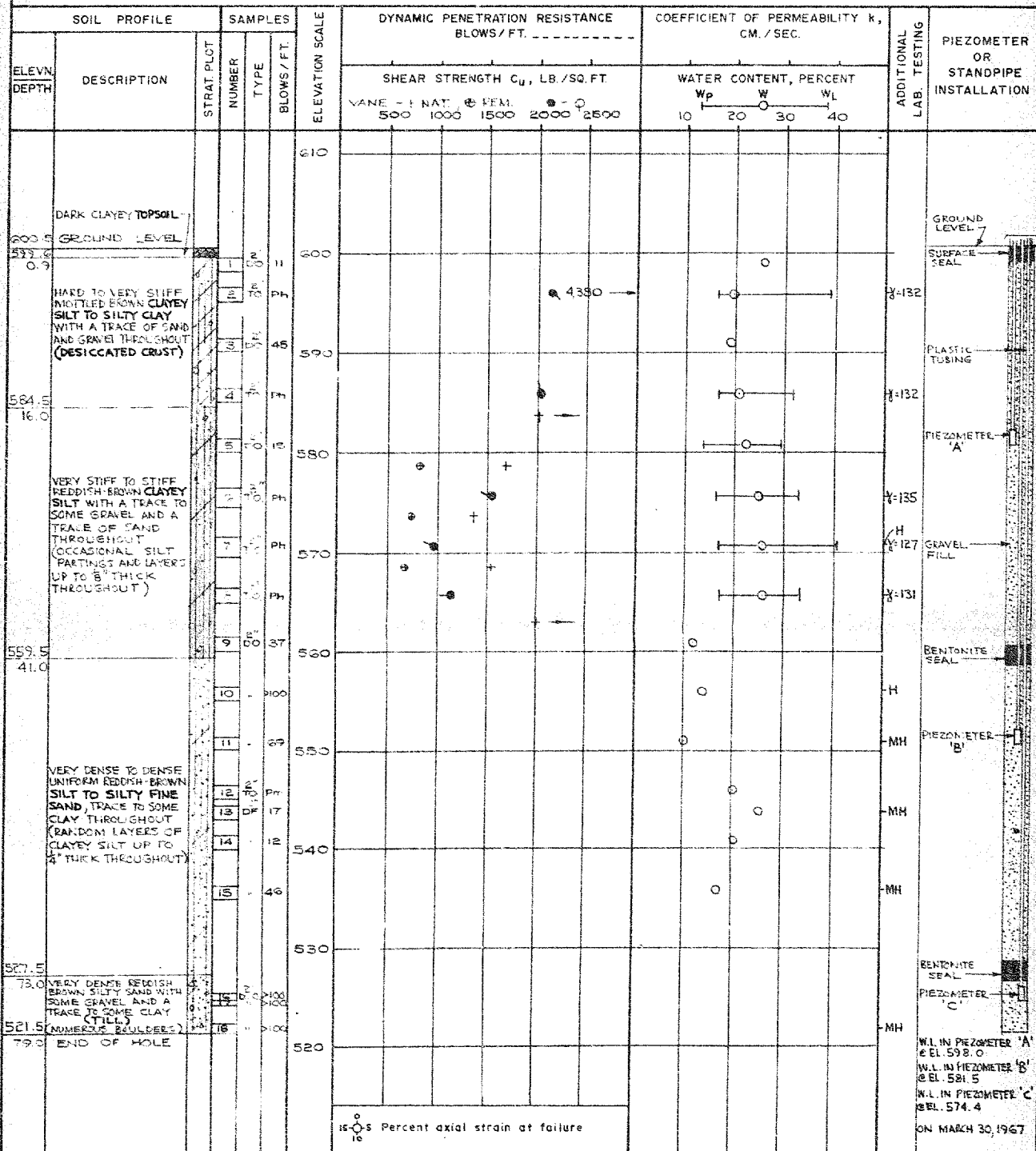
BORING DATE DEC. 29, 1966 - JAN. 5, 1967 DATUM GEODETTIC

BOREHOLE TYPE

POWER AUGER &amp; WASH BORING BOREHOLE DIAMETER 4.5" &amp; NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES

VERTICAL SCALE  
1 INCH TO 10'-0"

GOLDER &amp; ASSOCIATES

DRAWN R.H. JA  
CHECKED E.D.

## RECORD OF BOREHOLE T-2

LOCATION

See Figure 1

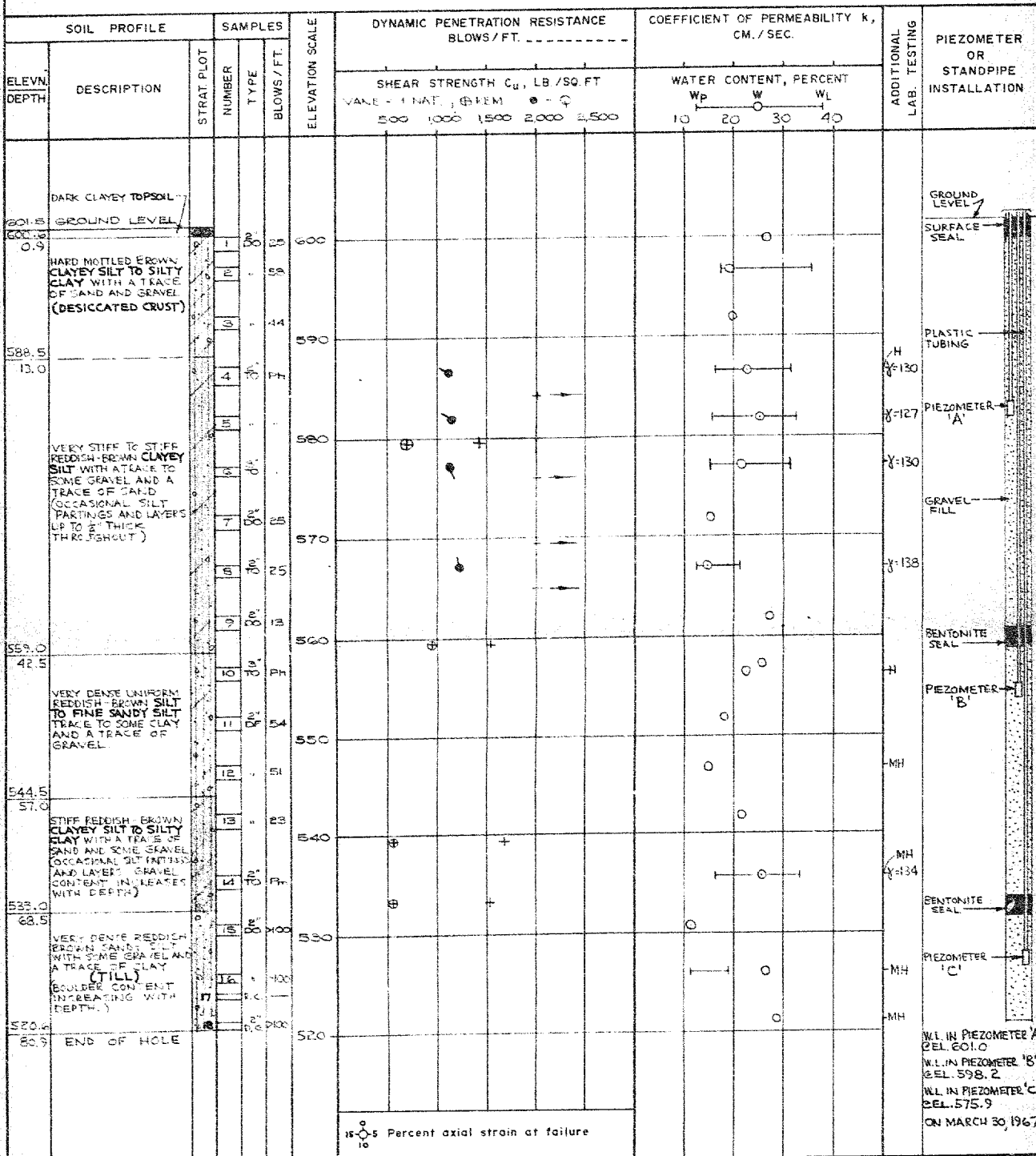
BORING DATE DEC. 30, 1966 - JAN. 10, 1967 DATUM GEODETIC

BOREHOLE TYPE POWER AUGER

BOREHOLE DIAMETER 4.5"

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES

VERTICAL SCALE  
1" INCH TO 10'-0"

GOLDER &amp; ASSOCIATES

DRAWN *[Signature]*  
CHECKED *[Signature]*

## RECORD OF BOREHOLE T-3

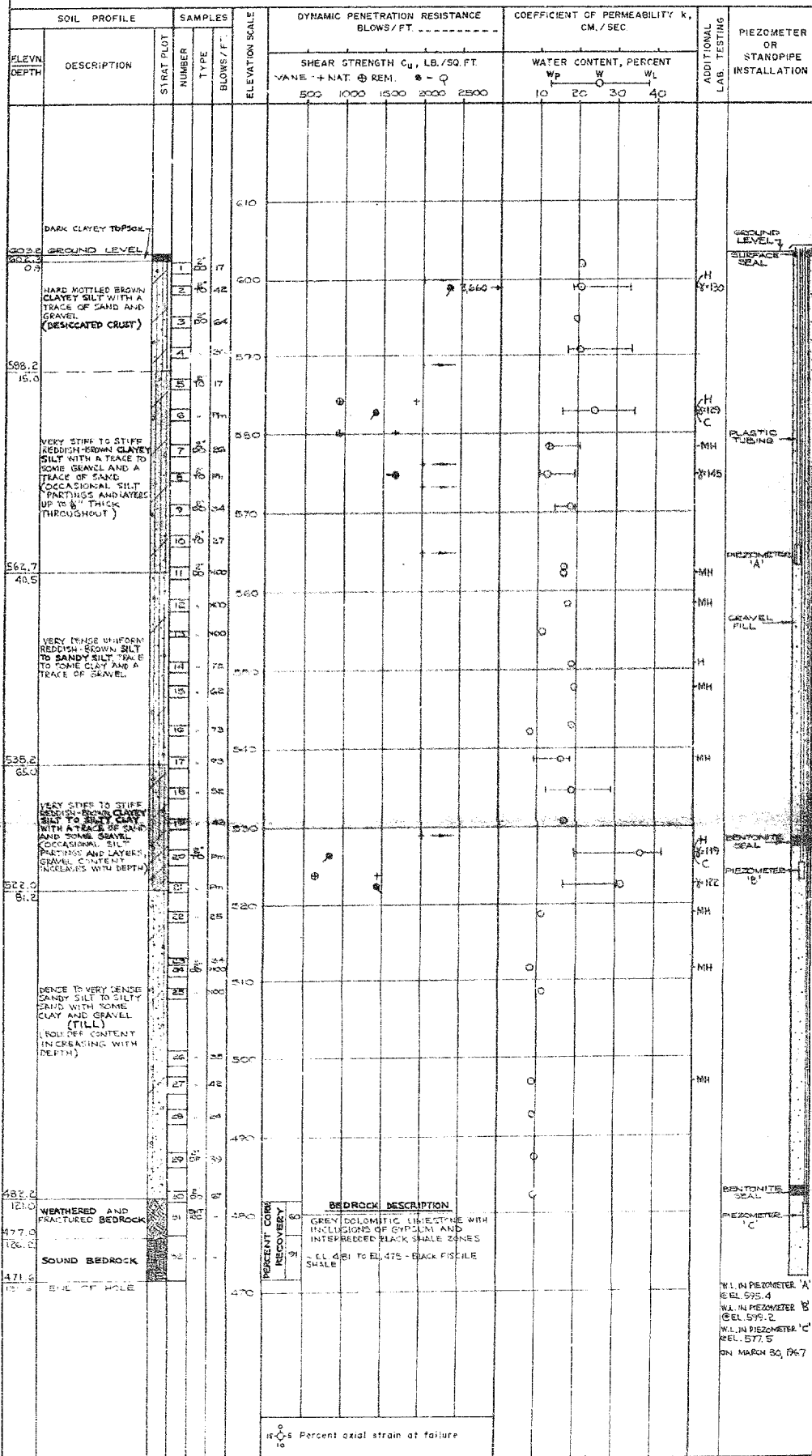
LOCATION	See Figure 1	BORING DATE	DEC. 16 - 27, 1968	DATUM	GEODETIC
	BOREHOLE TYPE	POWER AUGER & WASH BORING	BOREHOLE DIAMETER	4.5" NX BX CASING	
	SAMPLER HAMMER WEIGHT 140 LB.	DROP 30 INCHES	PEN. TEST HAMMER WEIGHT - LB.	DROP - INCHES	

[illegible]



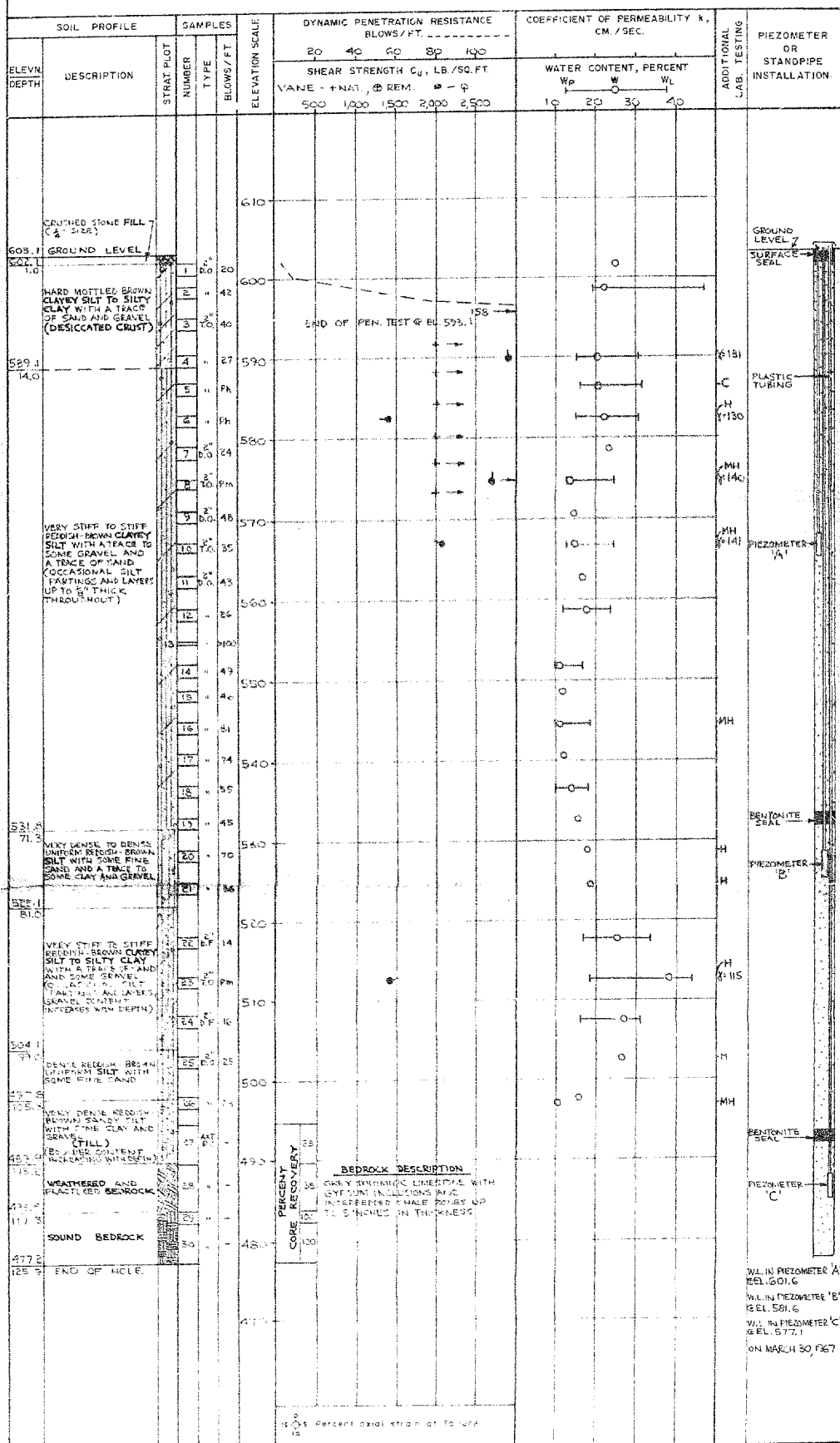
# RECORD OF BOREHOLE T-4

LOCATION See Figure 1 BORING DATE DEC 12 - 13, 1966 DATUM GEODETIC  
BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" NX BX CASING  
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES



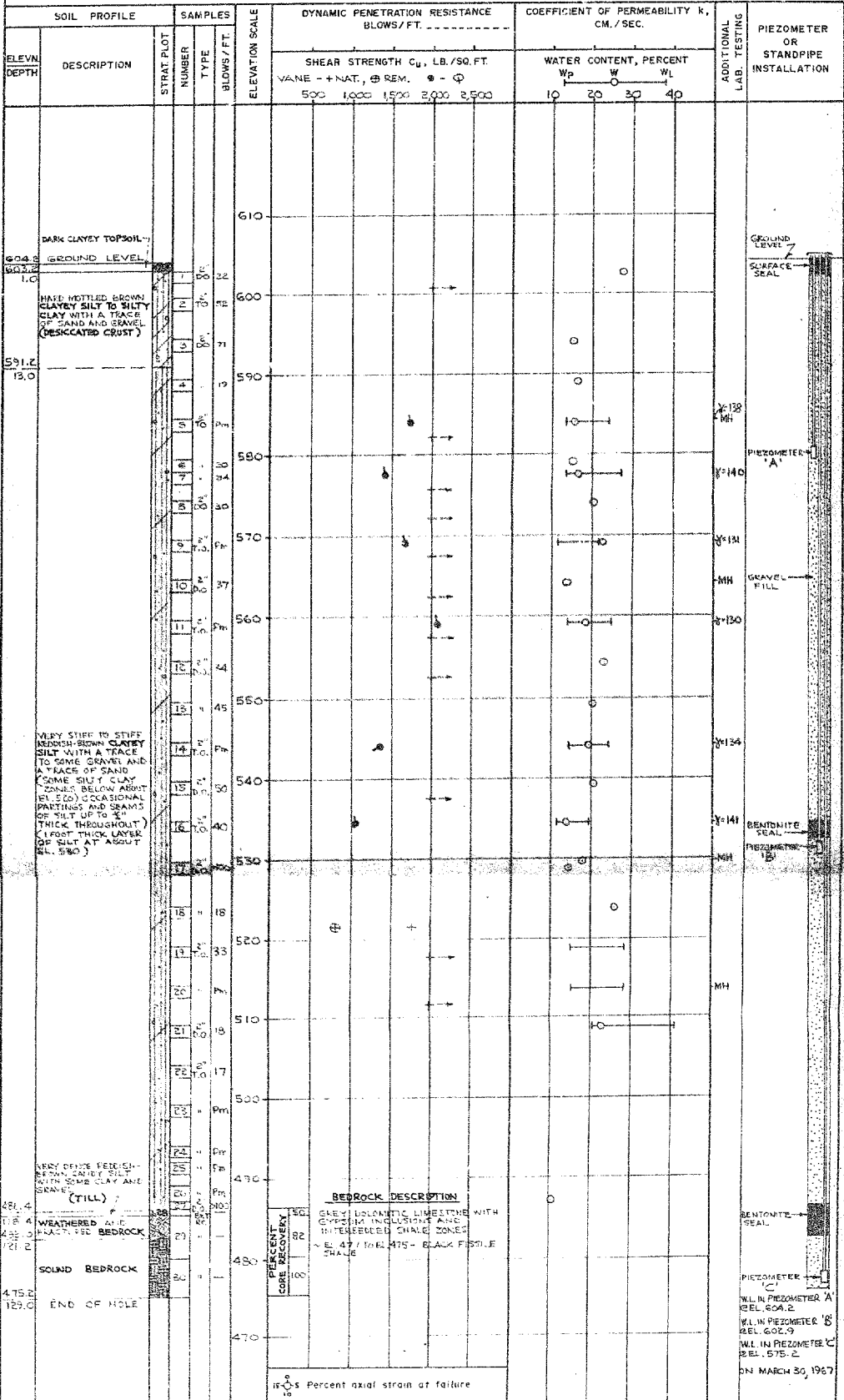
## RECORD OF BOREHOLE T-5

LOCATION See Figure 1 BORING DATE DEC 8-16, 1966 DATUM GEODETIC  
 BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" A x, B x CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



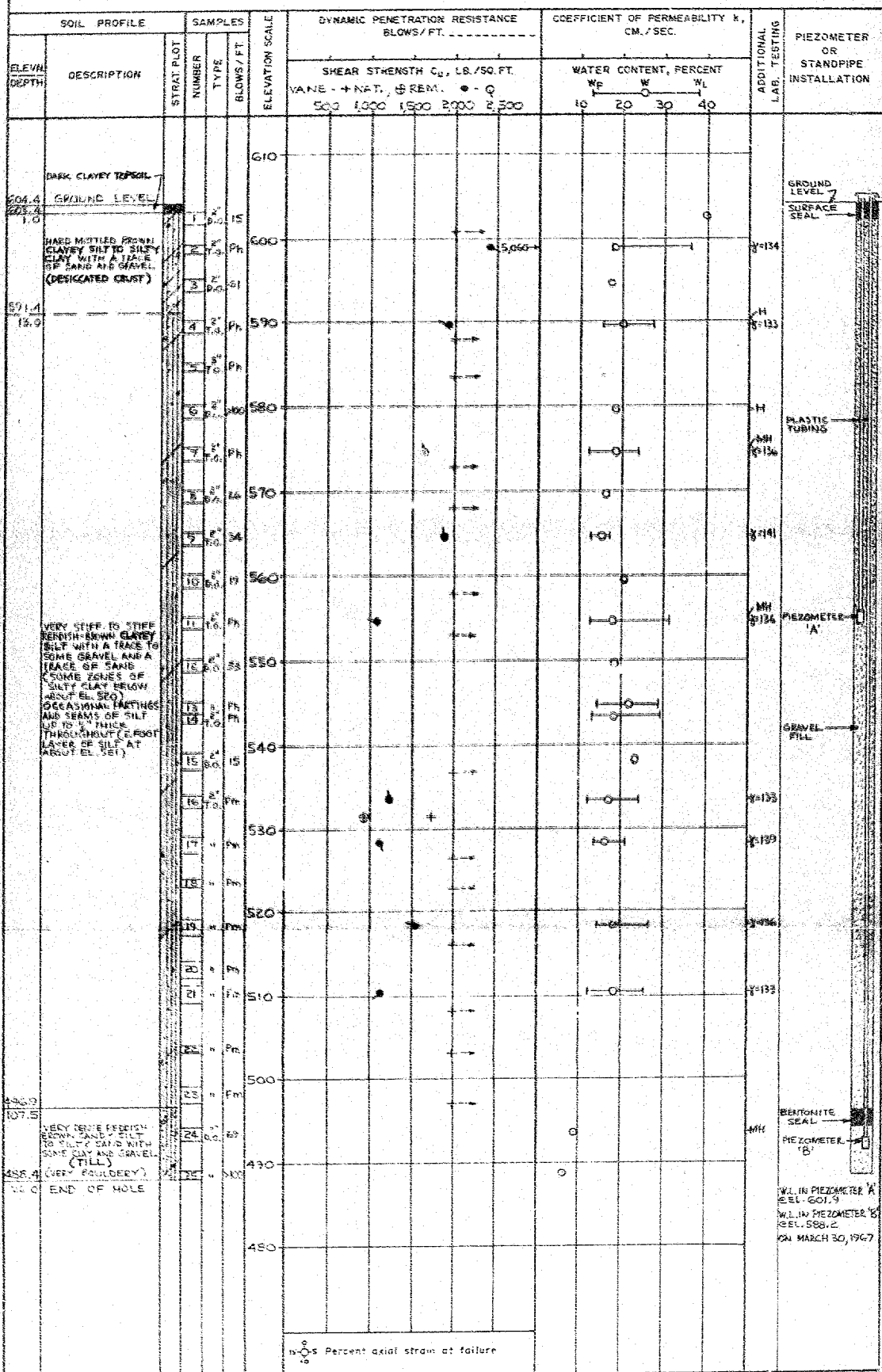
# RECORD OF BOREHOLE T-6

LOCATION See Figure 1 BORING DATE DEC 14-31, 1966 DATUM GEODETIC  
 BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" NX, 8X CASING  
 SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



## RECORD OF BOREHOLE T-7

LOCATION See Figure 1 BORING DATE DEC. 20, 1966 - JAN. 5, 1967 DATUM GEODETIC  
 BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" NX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED S.P.

## RECORD OF BOREHOLE T-8

LOCATION See Figure 1

BORING DATE DEC. 22, 1966 - JAN. 5, 1967 DATUM

GEODETIC

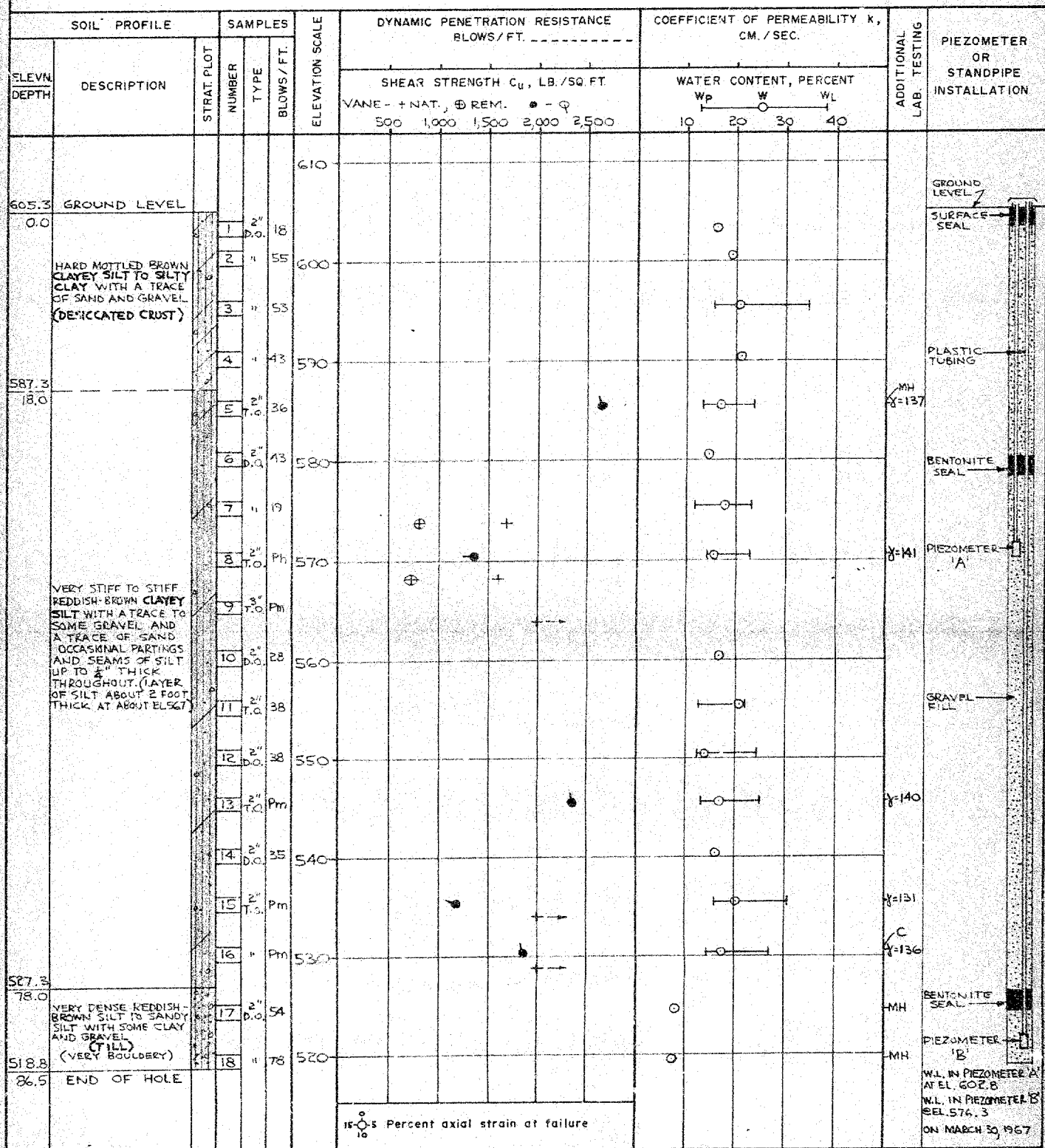
BOREHOLE TYPE POWER AUGER,  $\frac{1}{2}$  WASH BORING

BOREHOLE DIAMETER

4.5"  $\frac{1}{4}$  NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

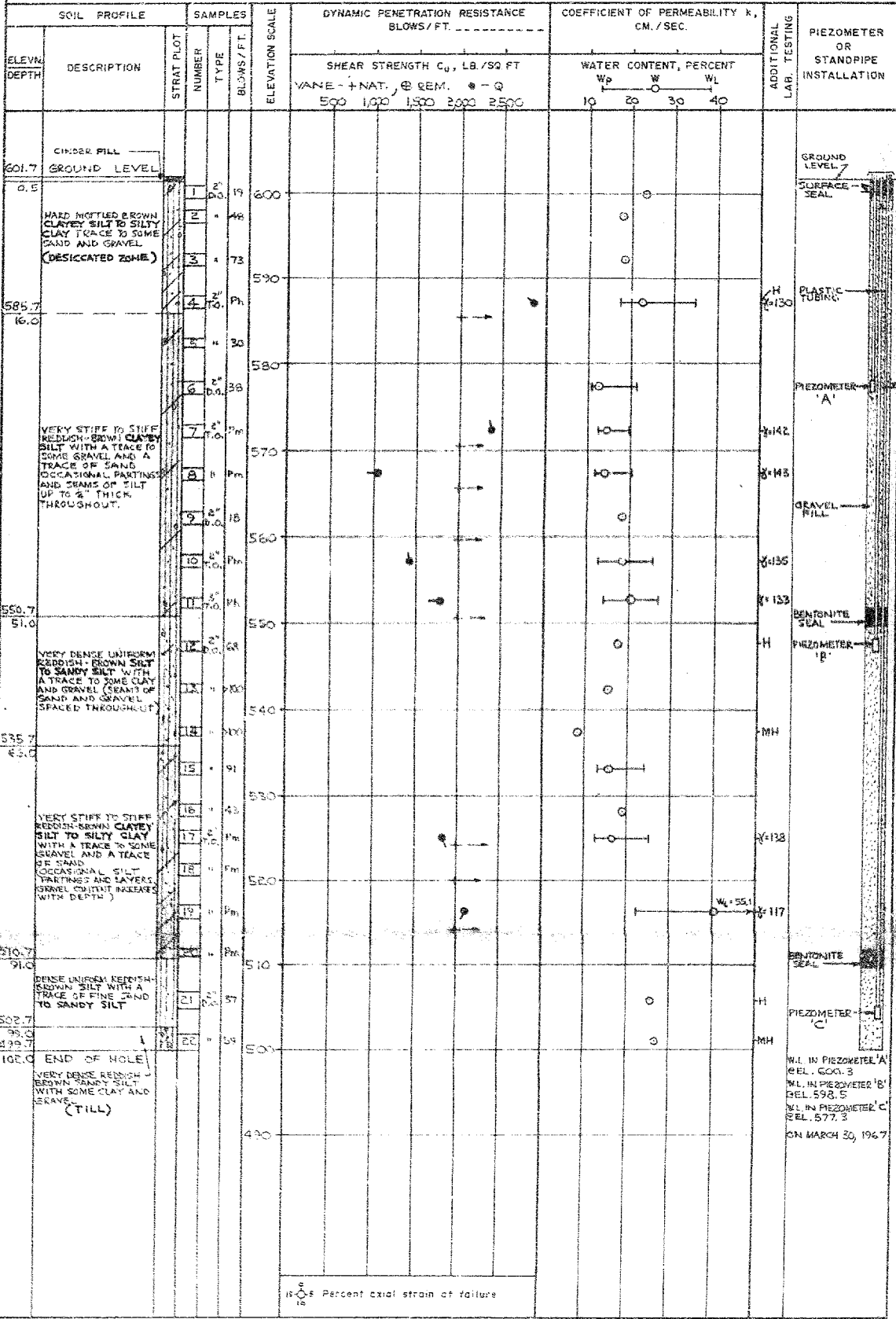
VERTICAL SCALE  
1 INCH TO 10'-0"

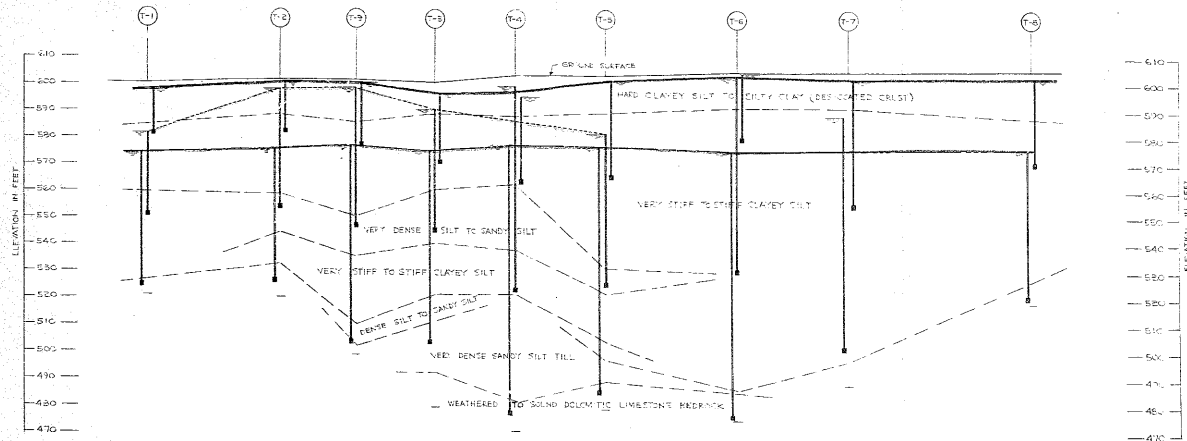
GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED E.T.D.

RECORD OF BOREHOLE T-9

LOCATION See Figure 1 BORING DATE JAN. 4-10, 1967 DATUM GEODETIC  
BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" ± NX CASING  
SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES PEN. TEST HAMMER WEIGHT --- LB. DROP --- INCHES





SCHEMATIC SECTION ALONG CENTRELINE OF MAIN STREET

SCALE: HORIZ. 1" TO 200'  
VERT. 1" TO 20'

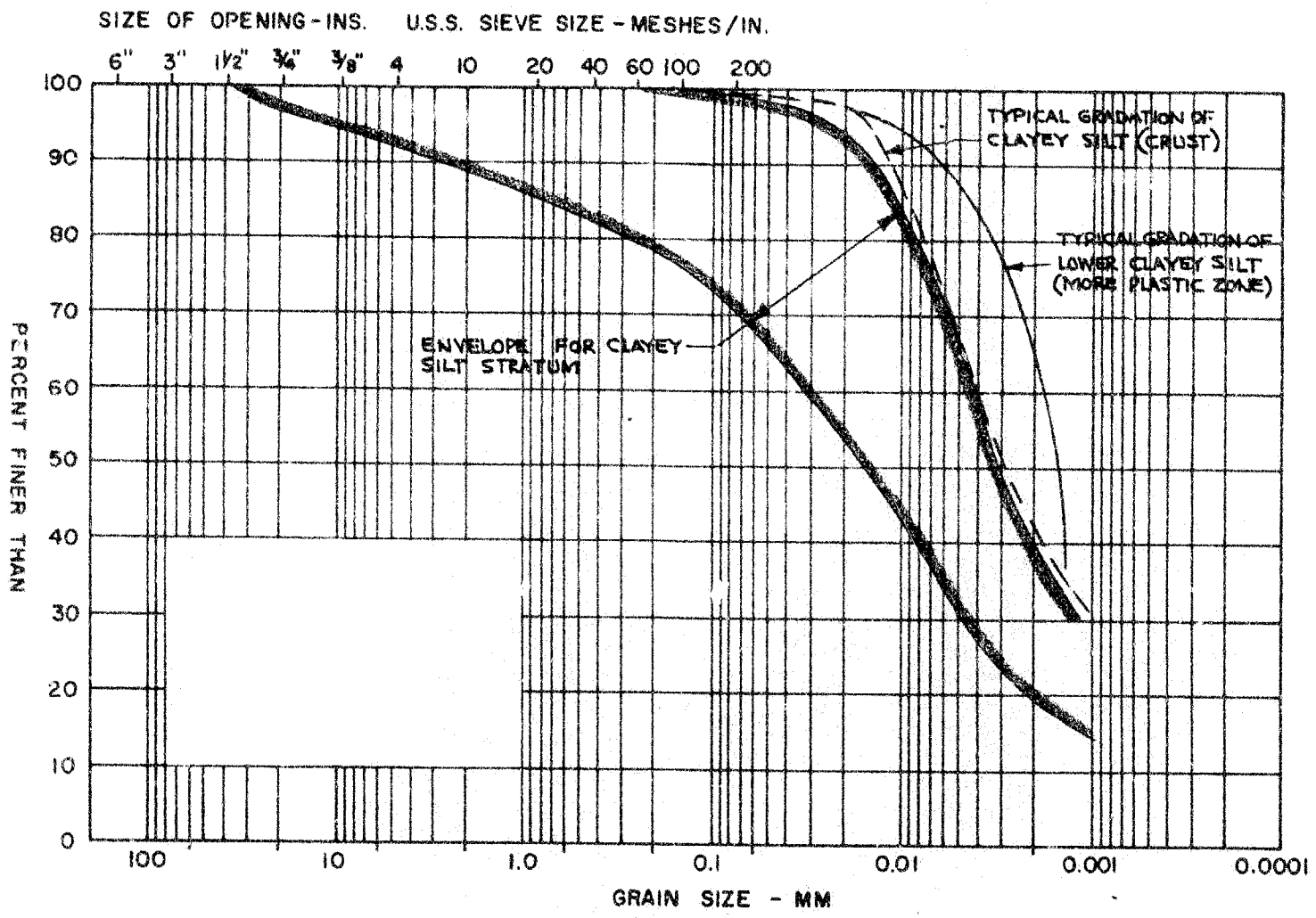
REFERENCE: FIGURE 1

Drawn: MAY 6, 1967

GOLDER & ASSOCIATES

Mod. J.A.  
Chad. J.A.

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT STRATUM

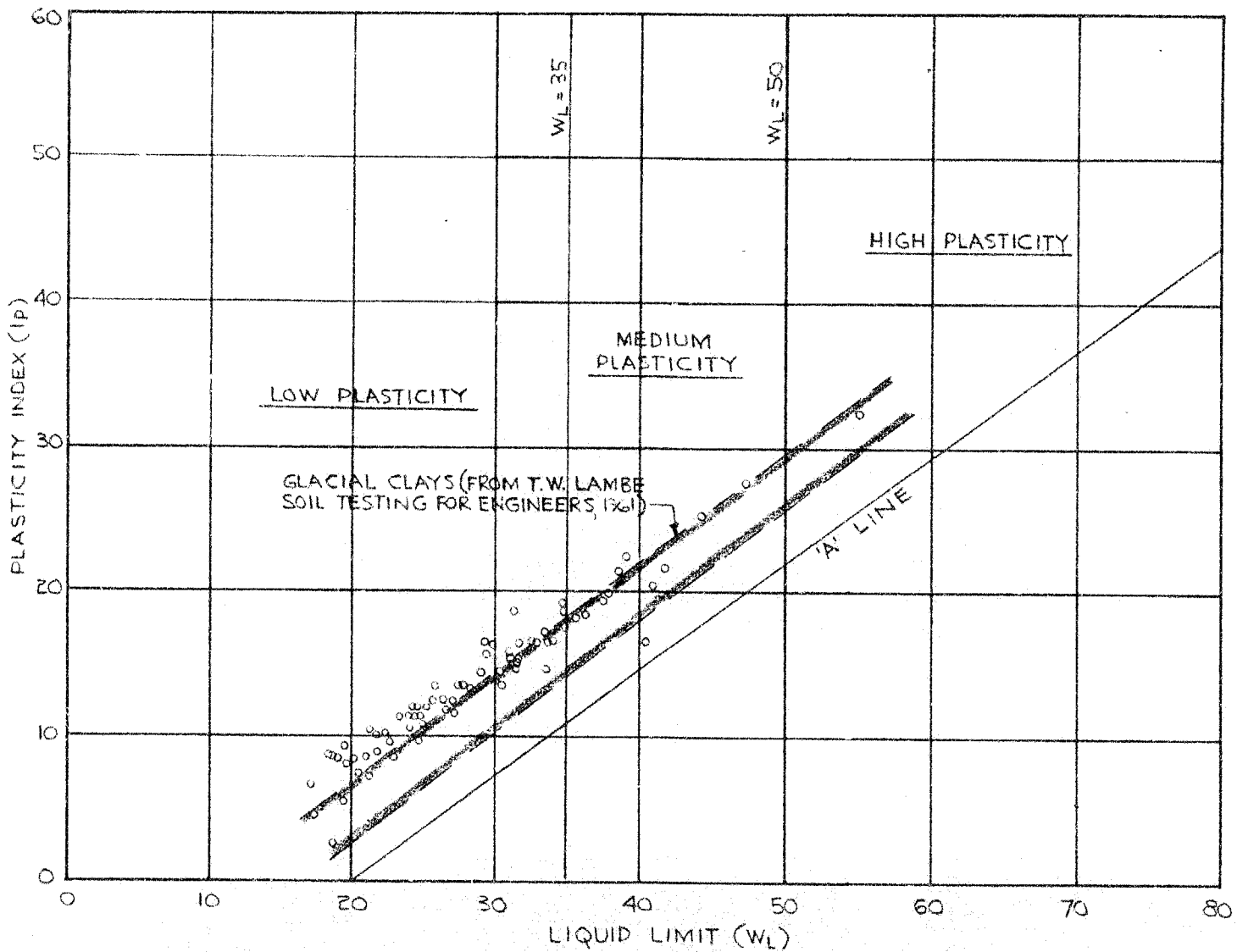
FIGURE 4

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	



PLASTICITY CHART  
CLAYEY SILT STRATUM

FIGURE 5



GOLDER & ASSOCIATES

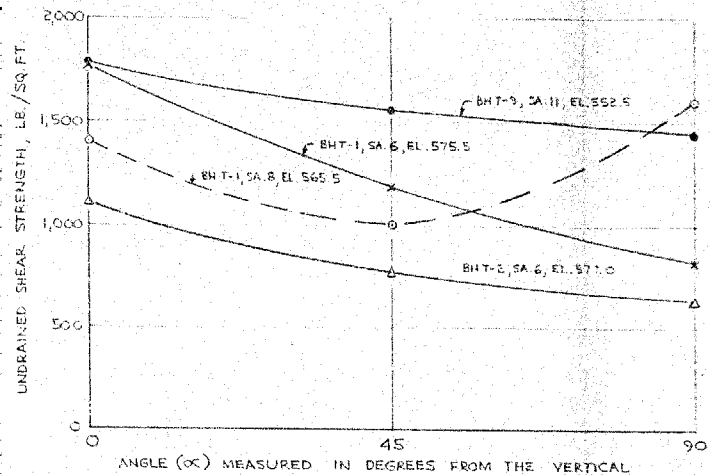
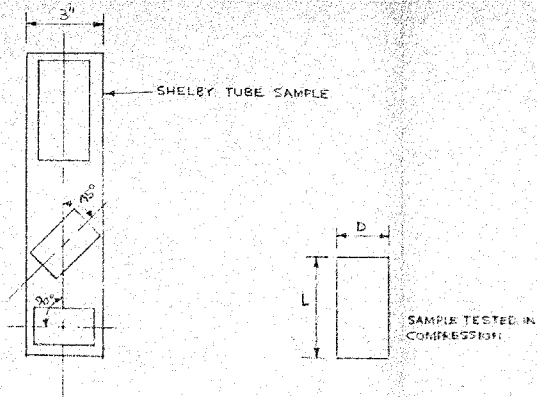
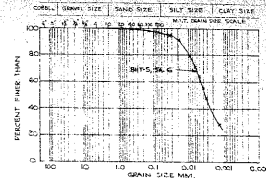
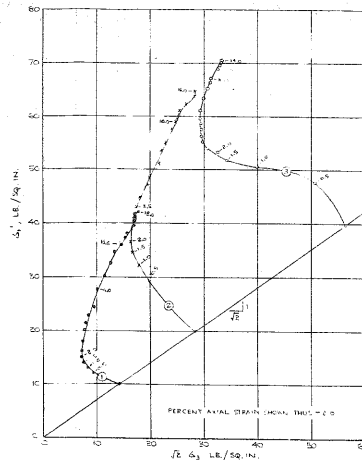
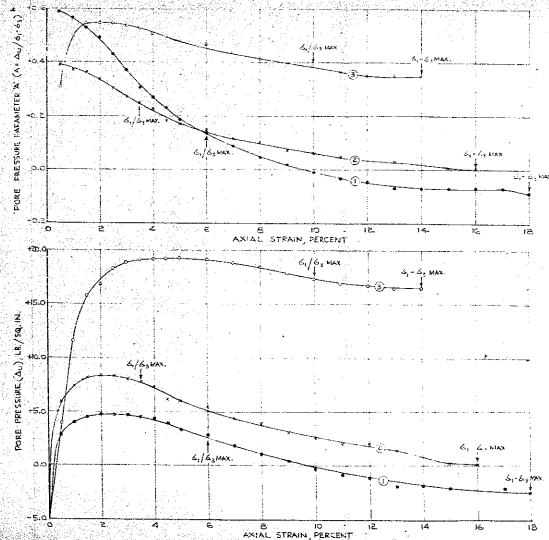
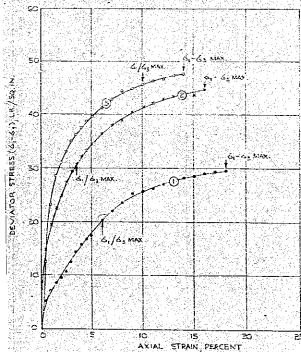
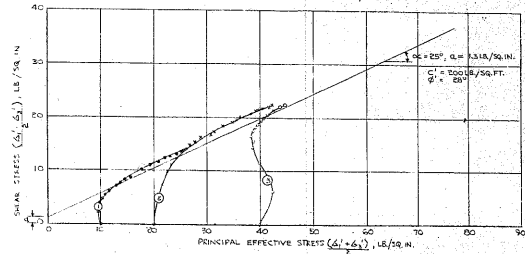


TABLE OF RESULTS

ANGLE OF SAMPLE TO VERTICAL IN DEGREES	LENGTH AND DIAMETER L AND D IN INCHES	RATIO OF L/D	UNDRAINED SHEAR STRENGTH $C_u$ LB/SQ FT.	FAILURE STRAIN PERCENT	TOTAL UNIT WEIGHT LB/CU FT.	NATURAL WATER CONTENT PERCENT	LIQUID LIMIT WL	PLASTIC LIMIT Wp
BHT-1 SA 6 EL 575.5								
0	4 AND 2	2:1	1,770	7.0	132	23.4	22.1	16.8
45	2.45 AND 1.58	1.7:1	1,190	3.5	137	28.9	—	—
90	2.25 AND 1.45	1.5:1	810	8.8	—	—	23.1	17.5
BHT-1 SA 8 EL 565.5								
0	2.48 AND 1.56	1.57:1	1,410	11.7	132	24.6	34.3	17.4
45	1.49 AND 2.75	1.91:1	1,000	12.0	127	25.0	—	—
90	1.45 AND 2.3	1.6:1	1,600	11.0	133	25.9	32.4	16.9
BHT-2 SA 6 EL 577.0								
0	1.58 AND 2.02	2:1	1,110	8.0	130	21.8	31.7	15.7
45	1.46 AND 2.24	1.5:1	770	10.0	131	23.1	—	—
90	1.54 AND 2.33	1.5:1	620	8.0	124	24.2	30.2	15.5
BHT-3 SA 11 EL 552.5								
0	1.59 AND 3.0	1.9:1	1,780	15.0	133	20.5	24.8	14.0
45	1.46 AND 2.44	1.7:1	1,560	14.0	133	27.5	—	—
90	1.5 AND 2.2	1.5:1	1,440	12.5	140	17.4	—	—



TEST STAGE ① BH-T-5, SA 5, EL 582.6 }  $w_L = 32.9$ ,  $w_p = 16.4$   
 TEST STAGE ② BH-T-5, SA 5, EL 582.1 }  
 TEST STAGE ③ BH-T-5, SA 5, EL 582.1 }  $w_L = 30.9$ ,  $w_p = 15.1$



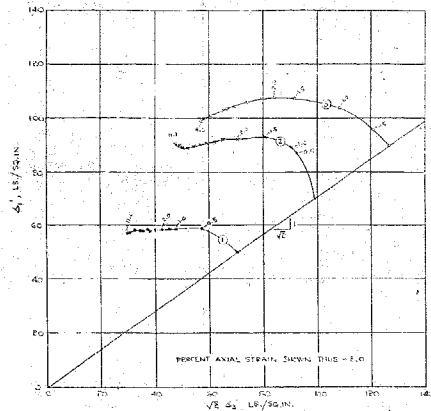
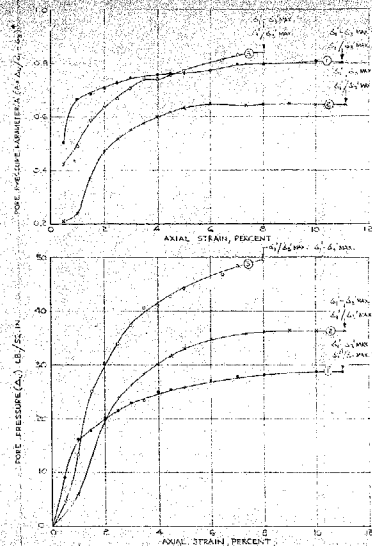
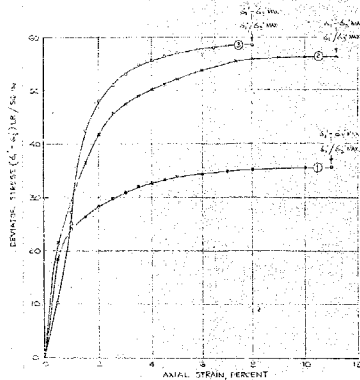
# CONSOLIDATED UNDRAINED TRIAXIAL TESTS WITH PORE PRESSURE MEASUREMENTS

FIGURE 7

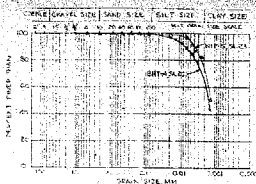
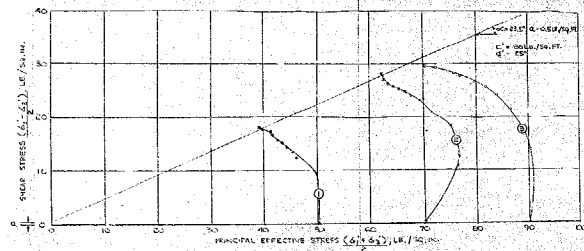
TABLE

TEST STAGE	①	②	③
TOTAL CELL PRESSURE $\sigma_3$ , LB/SQ. IN.	49.0	69.0	89.0
BACK PRESSURE $u$ , LB/SQ. IN.	39.0	49.0	49.0
EFFECTIVE CELL PRESSURE $\sigma_3'$ , LB/SQ. IN.	10.0	20.0	40.0
PARAMETER 'B'	0.97	0.99	0.99
AVERAGE RATE OF STRAIN, %/HR.	1.83	1.83	1.83
VOL. CHANGE DURING CONSOLIDATION $\Delta V$ , C.C.	0.0370	0.0374	0.0600
DEVIATOR STRESS $\sigma_1 - \sigma_3$ , LB/SQ. IN. (FAILURE)	29.6	44.8	47.8
STRAIN AT FAILURE, PERCENT	18.0	16.0	14.0
MAXIMUM STRESS RATIO $\sigma_1/\sigma_3$	3.86	3.53	3.03
STRAIN AT MAXIMUM STRESS RATIO	4.0	3.5	10.0
INITIAL WATER CONTENT, PERCENT	32.0	21.9	22.6
FINAL WATER CONTENT, PERCENT	21.9	21.4	20.8
DRY DENSITY, LB/CU. FT.	109.2	109.4	108.5

\* POPE PRESSURE PARAMETERS; REFERENCE: SKEMPTON A.W. (1954)  
 "THE POPE PRESSURE COEFFICIENTS A AND B," GEOTECHNIQUE,  
 VOL. IV, 1954.



- TEST STAGE ① BHT-4, SA 20, EL 526.7 -  $W_L = 41.8$ ,  $W_P = 19.9$   
 TEST STAGE ② BHT-5, SA 25, EL 513.1 }  $W_L = 44.2$ ,  $W_P = 18.7$   
 TEST STAGE ③ BHT-6, SA 25, EL 512.1



# LEGEND

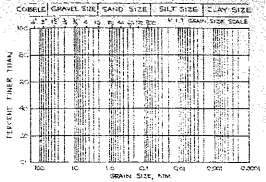
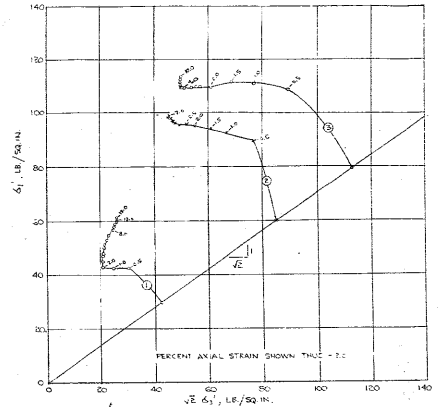
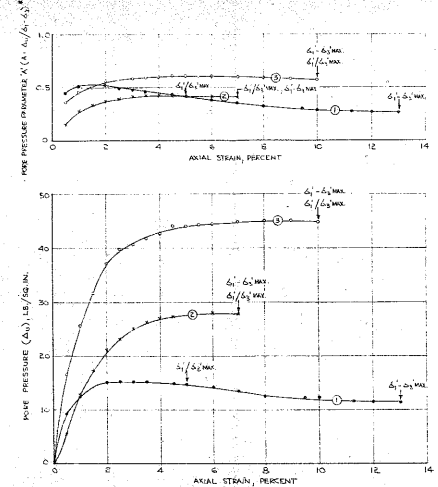
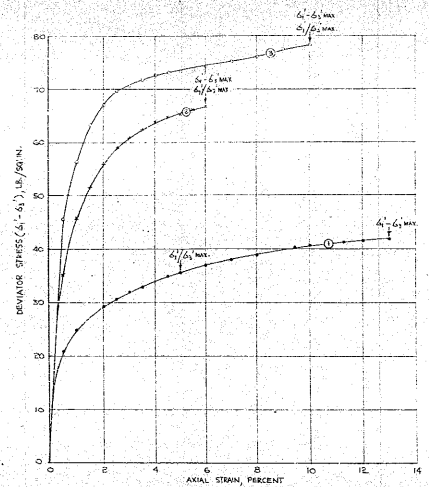
CONSOLIDATED UNDRAINED TRIAXIAL TESTS WITH PORE PRESSURE MEASUREMENTS				FIGURE 8	
TABLE					
TEST STAGE				①	②
TOTAL CELL PRESSURE $\Delta_3$ , LB/SG.IN.				79.0	117.0
BACK PRESSURE $u$ , LB/SG.IN.				43.0	47.0
EFFECTIVE CELL PRESSURE $\Delta_1$ , LB/SG.IN.				36.0	70.0
PARAMETER $k^*$				0.59	0.47
AVERAGE RATE OF STRAIN, %/HOUR				1.85	1.85
VOLUME CHANGE DURING CONSOLIDATION, %				-0.045	-0.045
DEVIATOR STRESS $\Delta_1 - \Delta_2$ , LB/SG.IN. (FAILURE)				35.8	56.8
STRAIN AT FAILURE, PERCENT				11.0	11.1
MAXIMUM STRESS RATIO $\Delta_1/\Delta_2$				2.27	2.66
STRAIN AT MAXIMUM STRESS RATIO				11.2	11.1
INITIAL WATER CONTENT, PERCENT				44.2	44.2
FINAL WATER CONTENT, PERCENT				45.5	45.5
DRY DENSITY, LB/CG.FT.				87.1	87.5

\* PORE PRESSURE PARAMETERS; REFERENCE, SKEMPTON A.W. (1954)  
 "THE PORE PRESSURE COEFFICIENTS A AND B," GEOTECHNIQUE  
 VOL. IV, 1954.

Drawn: MARCH 22, 1967

GOLDER & ASSOCIATES

Model 100  
 Class 100  
 Size 100



**LEGEND**

TEST STAGE ① BHT-7, SA. 16, EL. 535.5 -  $W_L = 24.4$ ,  $W_p = 12.4$

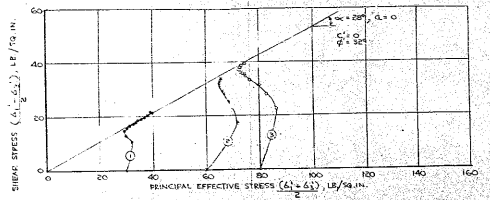
TEST STAGE ② BHT-7, SA. 17, EL. 528.5 -  $W_L = 21.1$ ,  $W_p = 14.0$

TEST STAGE ③ BHT-9, SA. 17, EL. 525.0 -  $W_L = 25.4$ ,  $W_p = 12.7$

**TABLE**

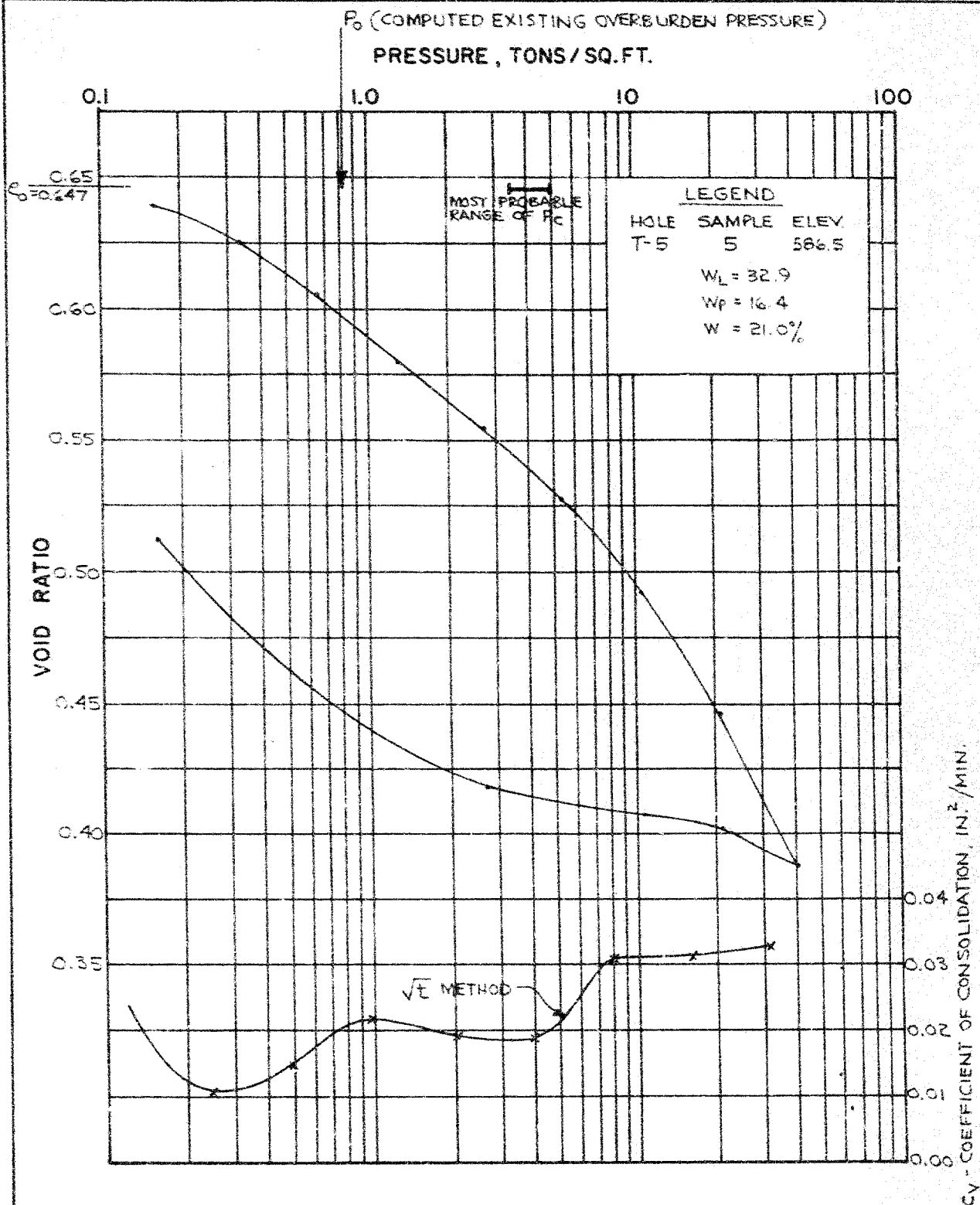
	①	②	③
TEST STAGE	①	②	③
TOTAL CELL PRESSURE $s_1$ , LB./SQ. IN.	95.0	119.0	129.0
BACK PRESSURE $u$ , LB./SQ. IN.	69.0	57.0	49.0
EFFECTIVE CELL PRESSURE $s_2$ , LB./SQ. IN.	30.0	60.0	80.0
PARAMETER $A'$	0.33	0.97	0.98
AVERAGE RATE OF STRAIN, %/HOUR	1.84	1.85	1.86
VOLUME CHANGE DURING CONSOLIDATION $\Delta V$ , C.C.	-0.0427	-0.0450	-0.0665
DEVIATOR STRESS $s_1 - s_2$ , LB./SQ. IN. (FAILURE)	42.0	66.8	78.3
STRAIN AT FAILURE, PERCENT	13.0	7.0	10.1
MAXIMUM STRESS RATIO $s_1/s_2$	3.32	3.07	3.23
STRAIN AT MAXIMUM STRESS RATIO	5.0	7.0	10.1
INITIAL WATER CONTENT, PERCENT	17.2	17.7	18.2
FINAL WATER CONTENT, PERCENT	15.3	16.2	15.3
DRY DENSITY, LB./CU. FT.	117.1	116.0	117.0

\* PORE PRESSURE PARAMETERS; REFERENCE: SKEMPTON A.W. (1954). "THE PORE PRESSURE COEFFICIENT  $A'$  AND  $B'$ ". GEOTECHNIQUE, VOL. IV, 1954.



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 10

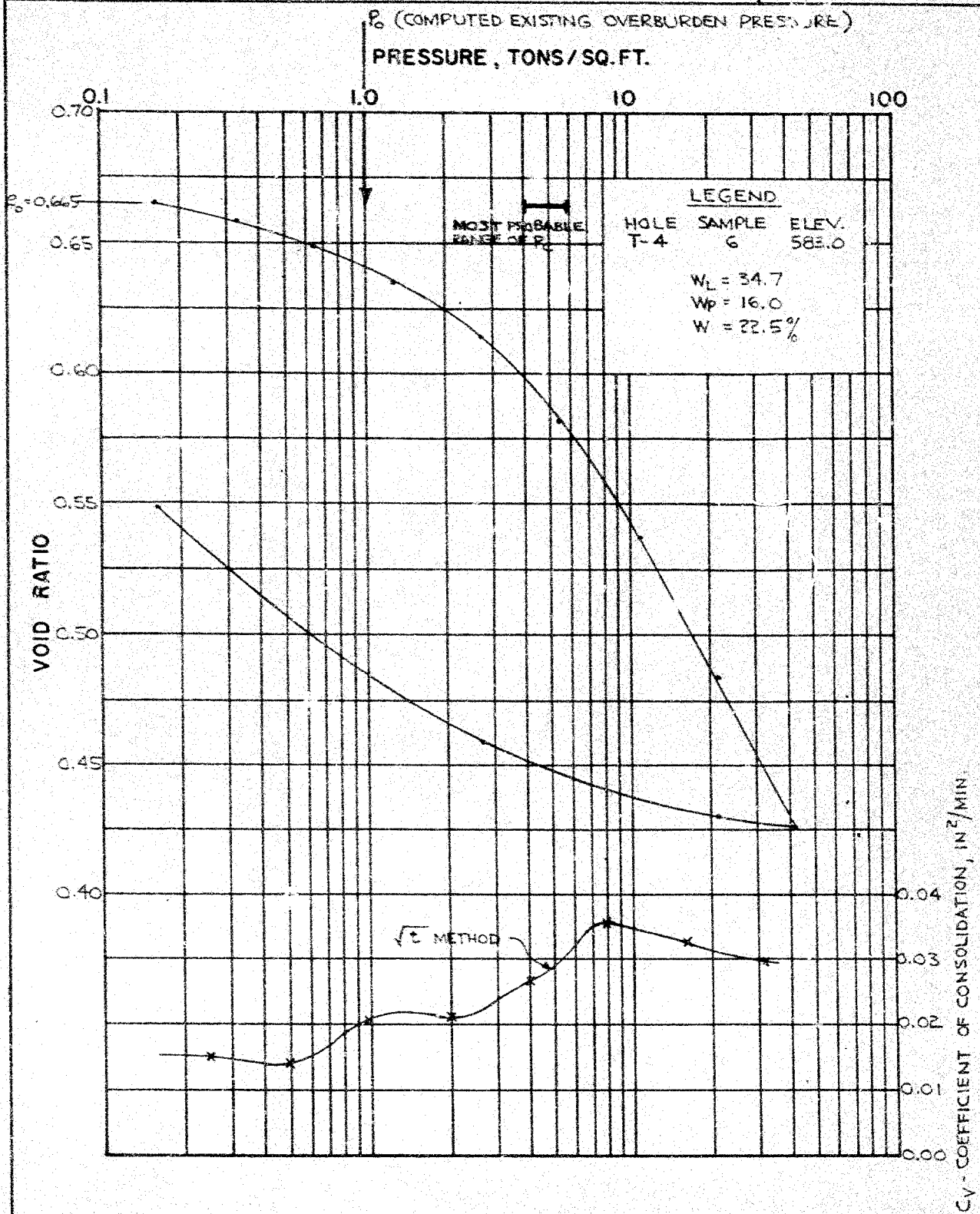


GOLDER &amp; ASSOCIATES

PROJECT NO. ... 96134

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

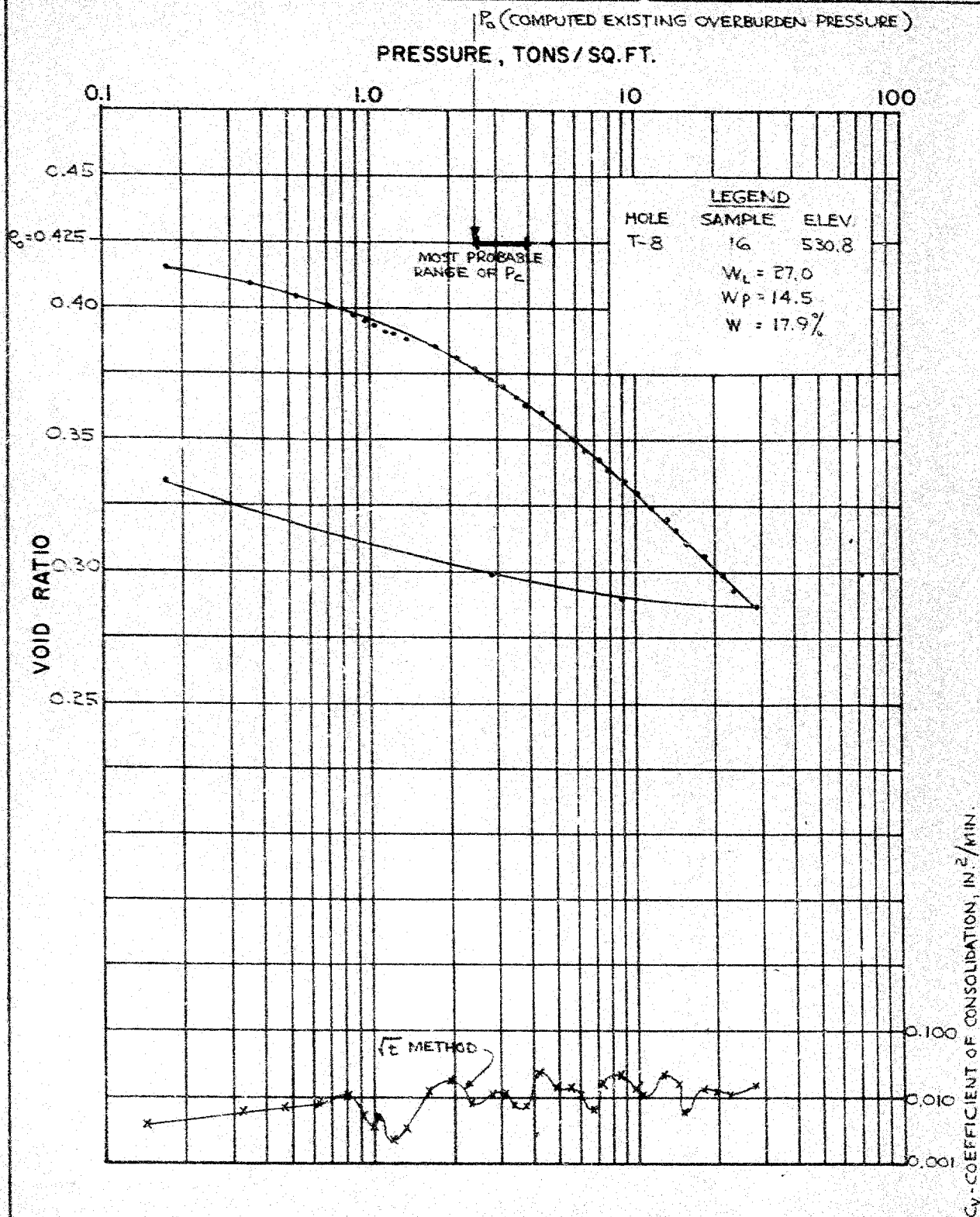
FIGURE 11



PROJECT NO. 13-001139

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 12

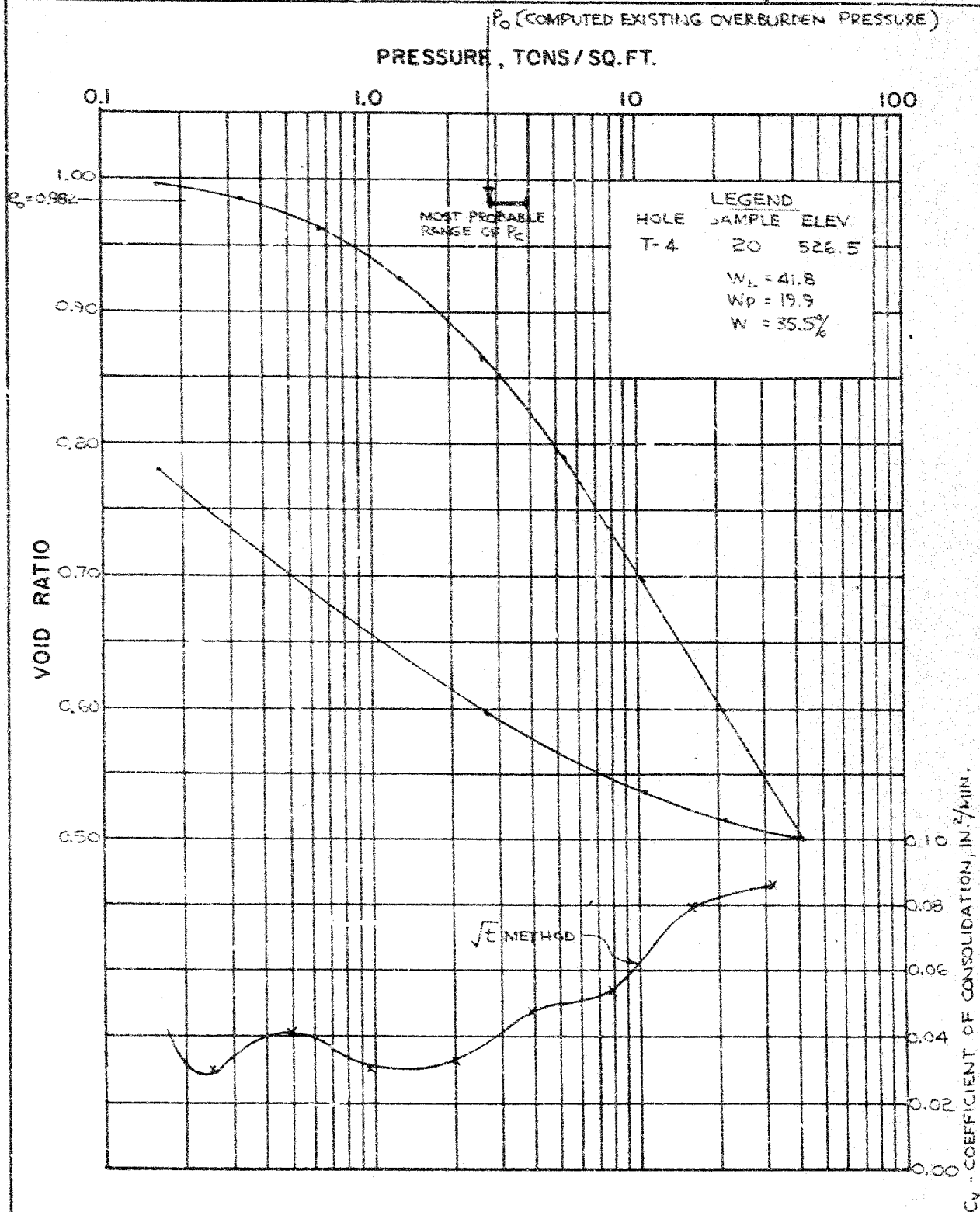


GOLDER & ASSOCIATES



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

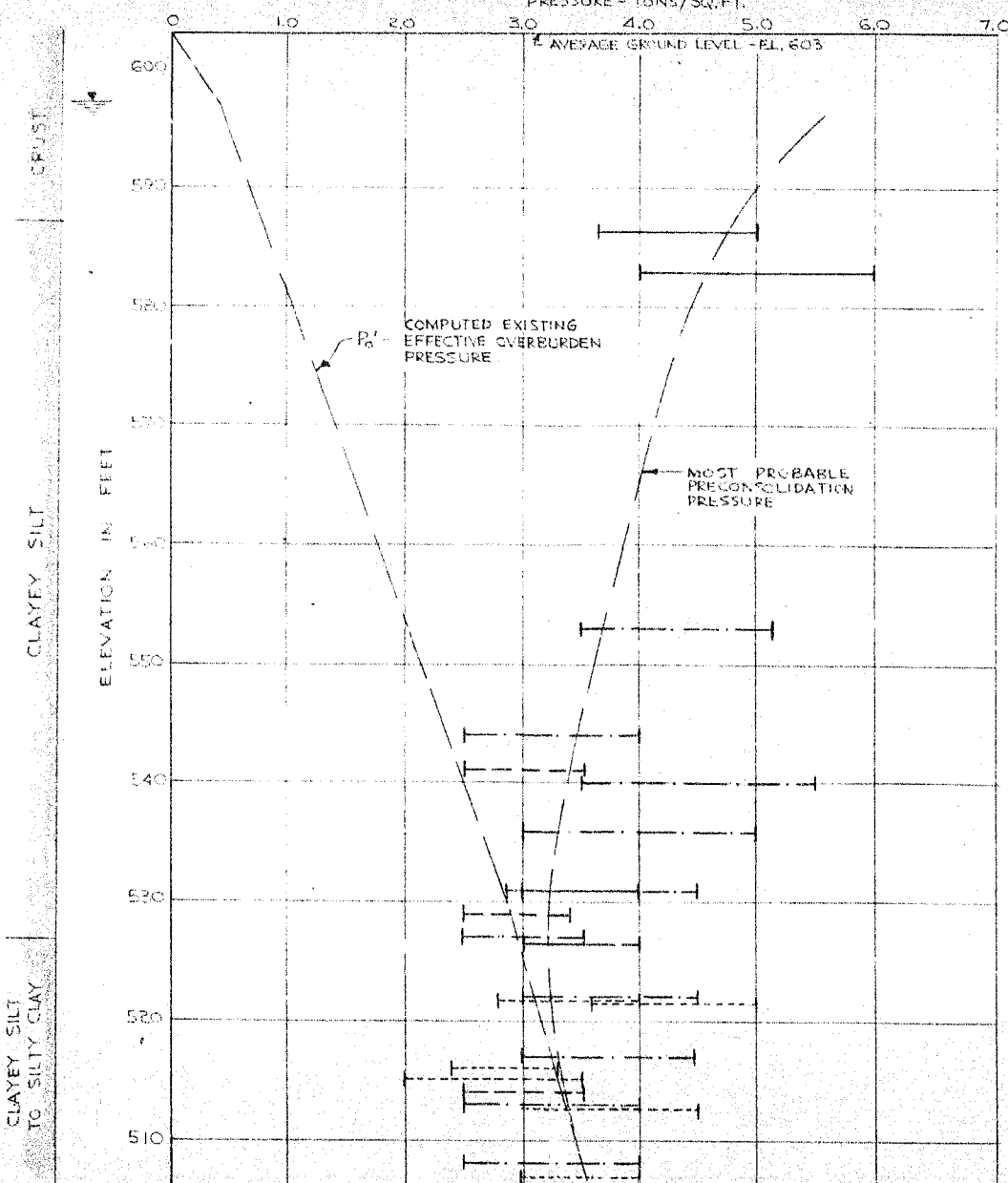
FIGURE 13



GOLDER & ASSOCIATES

# PROFILE OF MOST PROBABLE PRECONSOLIDATION PRESSURE (CLAYEY SILT STRATUM)

FIGURE 14



RANGE OF MOST  
PROBABLE  
PRECONSOLIDATION  
PRESSURE

## LEGEND

- PRESENT INVESTIGATION
  - - - REPORT No 6375
  - - - REPORT No 64005
  - - - REPORT No 6108
- REPORTS SUBMITTED ON  
PREVIOUS INVESTIGATIONS  
CARRIED OUT IN THE IMMEDIATE  
VICINITY (PREPARED BY H.Q. GOLDER  
& ASSOCIATES LTD. FOR THE  
DEPARTMENT OF HIGHWAYS, ONTARIO)

Drawn: APRIL 19, 1967

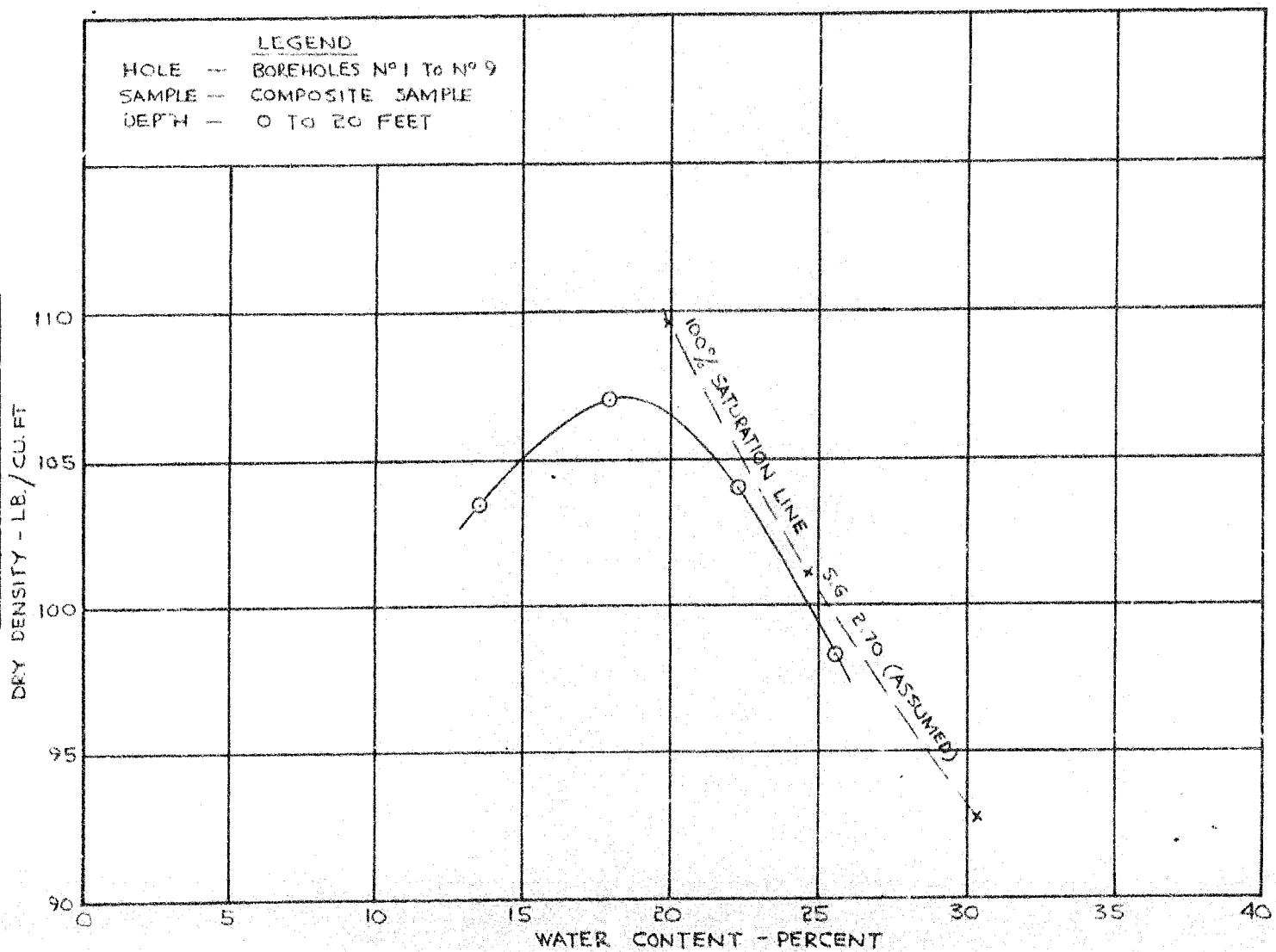
GOLDER & ASSOCIATES

Made by: J.A.  
Chkd. by: J.L.  
Appd. by: J.L.

# STANDARD PROCTOR COMPACTION TEST

## CLAYEY SILT STRATUM

FIGURE 15

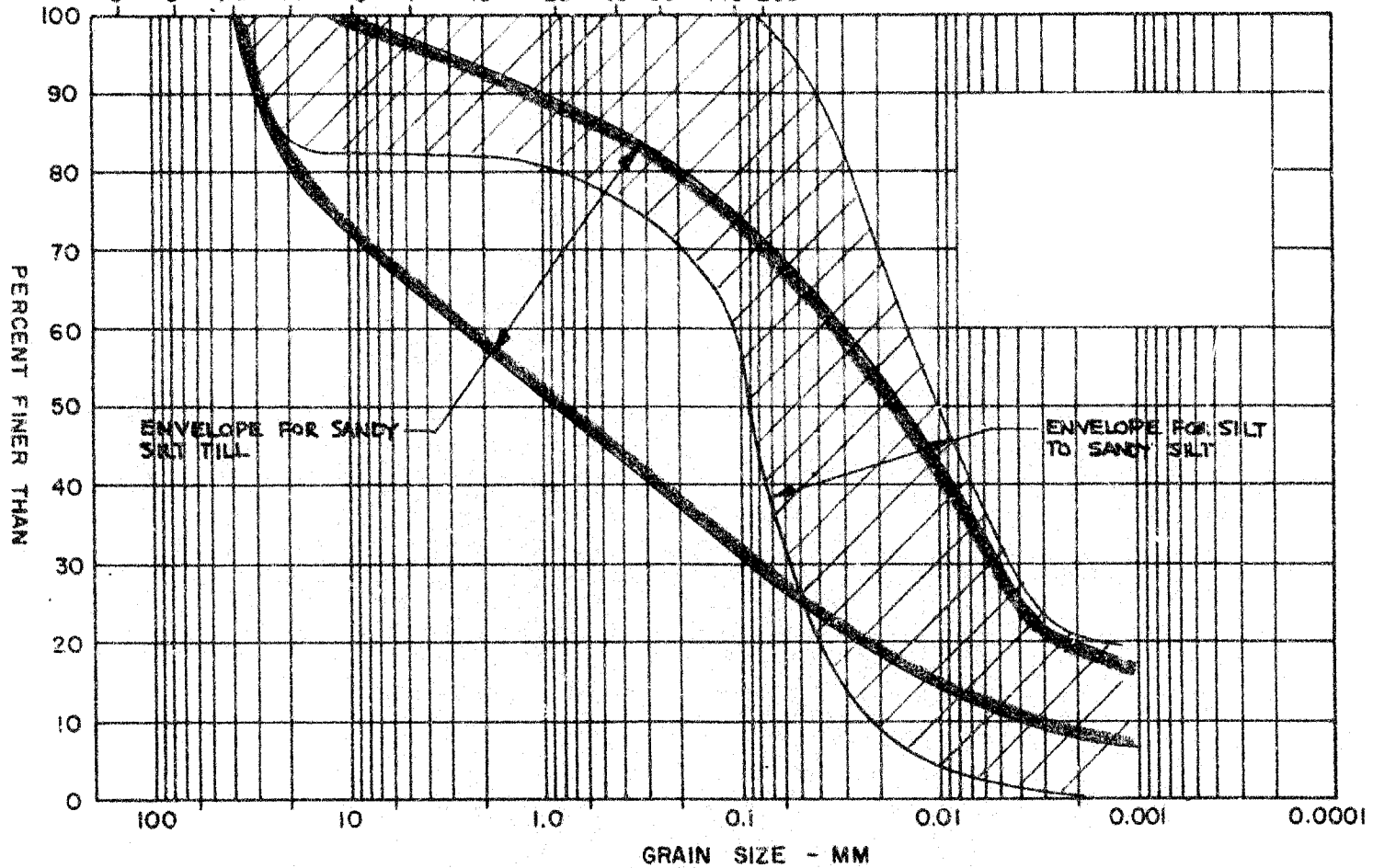


GOLDER & ASSOCIATES

**M.I.T. GRAIN SIZE SCALE**

SIZE OF OPENING-INS.	U.S.S. SIEVE SIZE - MESHES/IN.
10	20
15	10
20	10
25	10
30	10
35	10
40	10
45	10
50	10
55	10
60	10
65	10
70	10
75	10
80	10
85	10
90	10
95	10
100	10
105	10
110	10
115	10
120	10
125	10
130	10
135	10
140	10
145	10
150	10
155	10
160	10
165	10
170	10
175	10
180	10
185	10
190	10
195	10
200	10
205	10
210	10
215	10
220	10
225	10
230	10
235	10
240	10
245	10
250	10
255	10
260	10
265	10
270	10
275	10
280	10
285	10
290	10
295	10
300	10
305	10
310	10
315	10
320	10
325	10
330	10
335	10
340	10
345	10
350	10
355	10
360	10
365	10
370	10
375	10
380	10
385	10
390	10
395	10
400	10
405	10
410	10
415	10
420	10
425	10
430	10
435	10
440	10
445	10
450	10
455	10
460	10
465	10
470	10
475	10
480	10
485	10
490	10
495	10
500	10
505	10
510	10
515	10
520	10
525	10
530	10
535	10
540	10
545	10
550	10
555	10
560	10
565	10
570	10
575	10
580	10
585	10
590	10
595	10
600	10
605	10
610	10
615	10
620	10
625	10
630	10
635	10
640	10
645	10
650	10
655	10
660	10
665	10
670	10
675	10
680	10
685	10
690	10
695	10
700	10
705	10
710	10
715	10
720	10
725	10
730	10
735	10
740	10
745	10
750	10
755	10
760	10
765	10
770	10
775	10
780	10
785	10
790	10
795	10
800	10
805	10
810	10
815	10
820	10
825	10
830	10
835	10
840	10
845	10
850	10

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 140 200

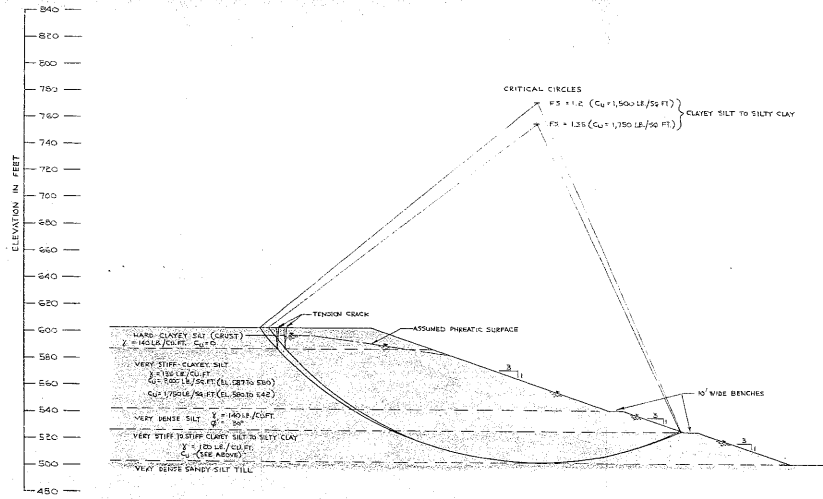


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
ENVELOPES - LOWER GRANULAR DEPOSITS

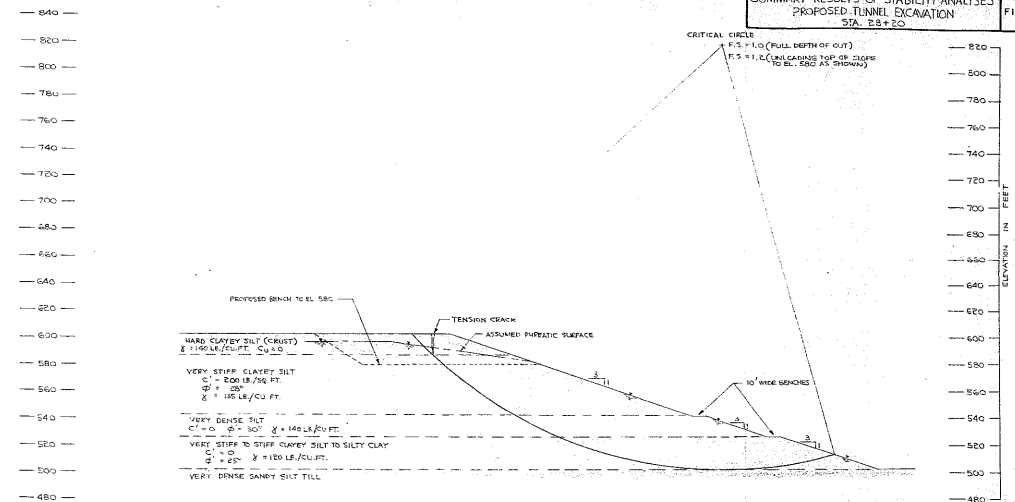
FIGURE 16

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	



(a) TOTAL STRESS ANALYSES

SCALE: 1" TO 40'



(b) EFFECTIVE STRESS ANALYSES

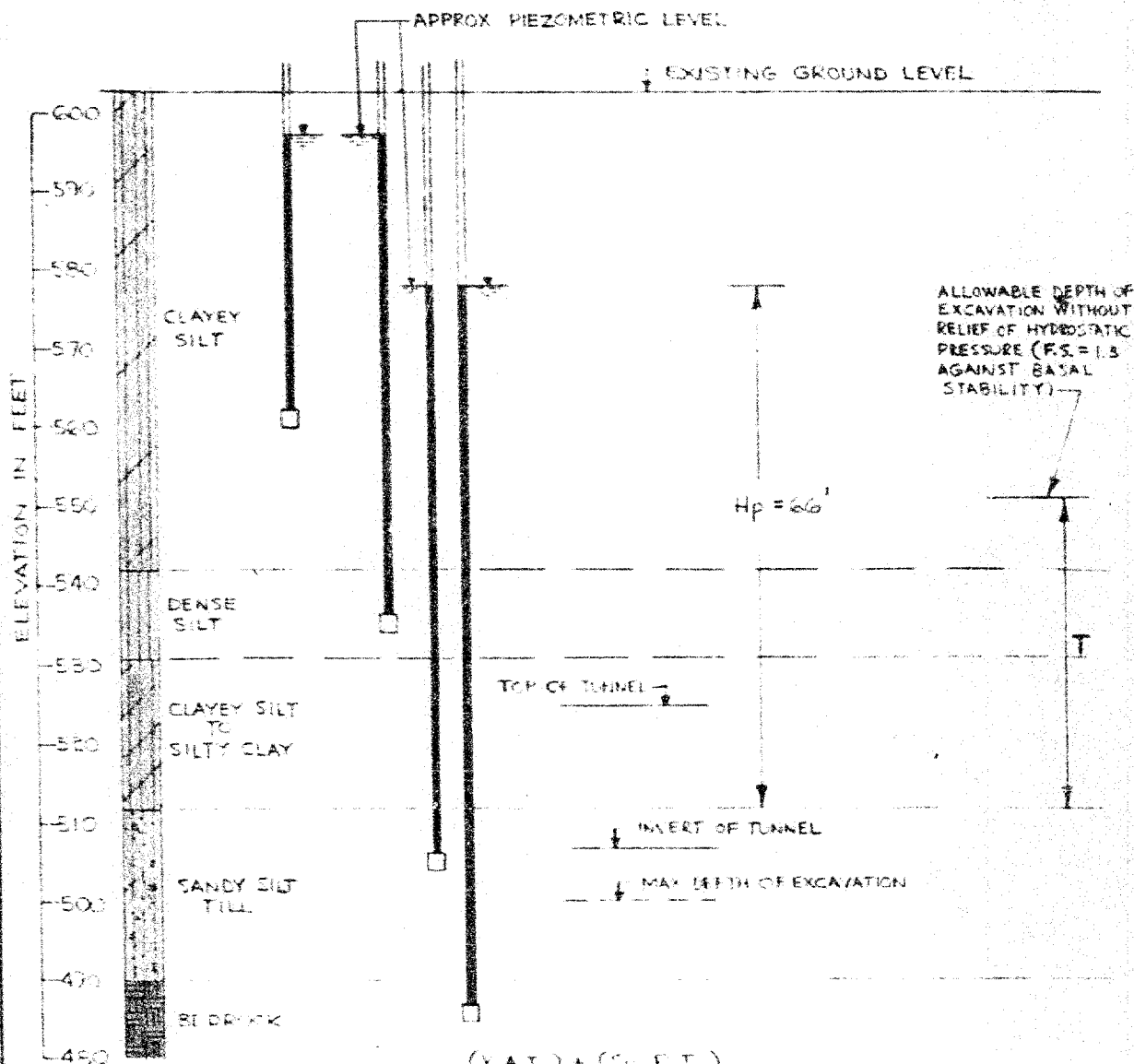
Drawn: MAY 4, 1967

GOLDER & ASSOCIATES

Made J.A.  
Checked B.D.  
Approved

# BASAL STABILITY OF EXCAVATION STA. 30+50

FIGURE 18



$$F = \frac{(Y_w A) + (S_u P)}{H_p A Y_w}$$

WHERE

- F = FACTOR OF SAFETY AGAINST BASAL HEAVE
- X = TOTAL UNIT WEIGHT OF PLUG (ASSUMED = 135 LB./CU. FT.)
- $Y_w$  = UNIT WEIGHT OF WATER (62.4 LB./CU. FT.)
- A = BASE AREA OF EXCAVATION (SQ. FT.)
- T = THICKNESS OF PLUG ABOVE CLAYEY SILT - SILT TILL INTERFACE (FT.)
- P = PERIMETER OF EXCAVATION (FT.)
- $S_u$  = MOBILIZED SHEAR STRENGTH (LB./SQ. FT.)
- $H_p$  = PIEZOMETRIC GROUNDWATER HEAD (FT.)

FOR LONG, WIDE EXCAVATION  $S_u$  P.T. IS NEGLIGIBLE COMPARED TO  $Y_w A$ .

$$\therefore T = \frac{F \cdot H_p \cdot Y_w}{X}$$

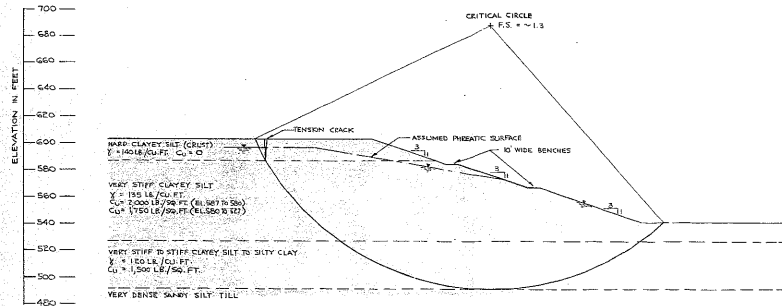
$$\text{FOR } F = 1.3$$

$$H_p = 66'$$

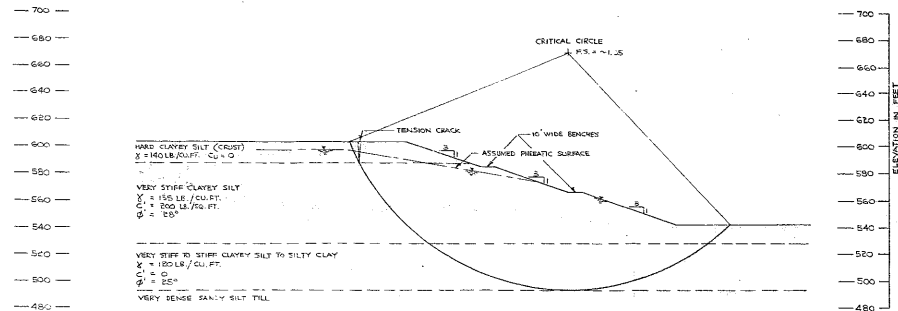
$$T \text{ REQUIRED} = 39'$$

GOLDER & ASSOCIATES

Made by J.A.  
Chkd. C.T.D.  
Appd. C.T.D.



(a) TOTAL STRESS ANALYSES



(b) EFFECTIVE STRESS ANALYSES

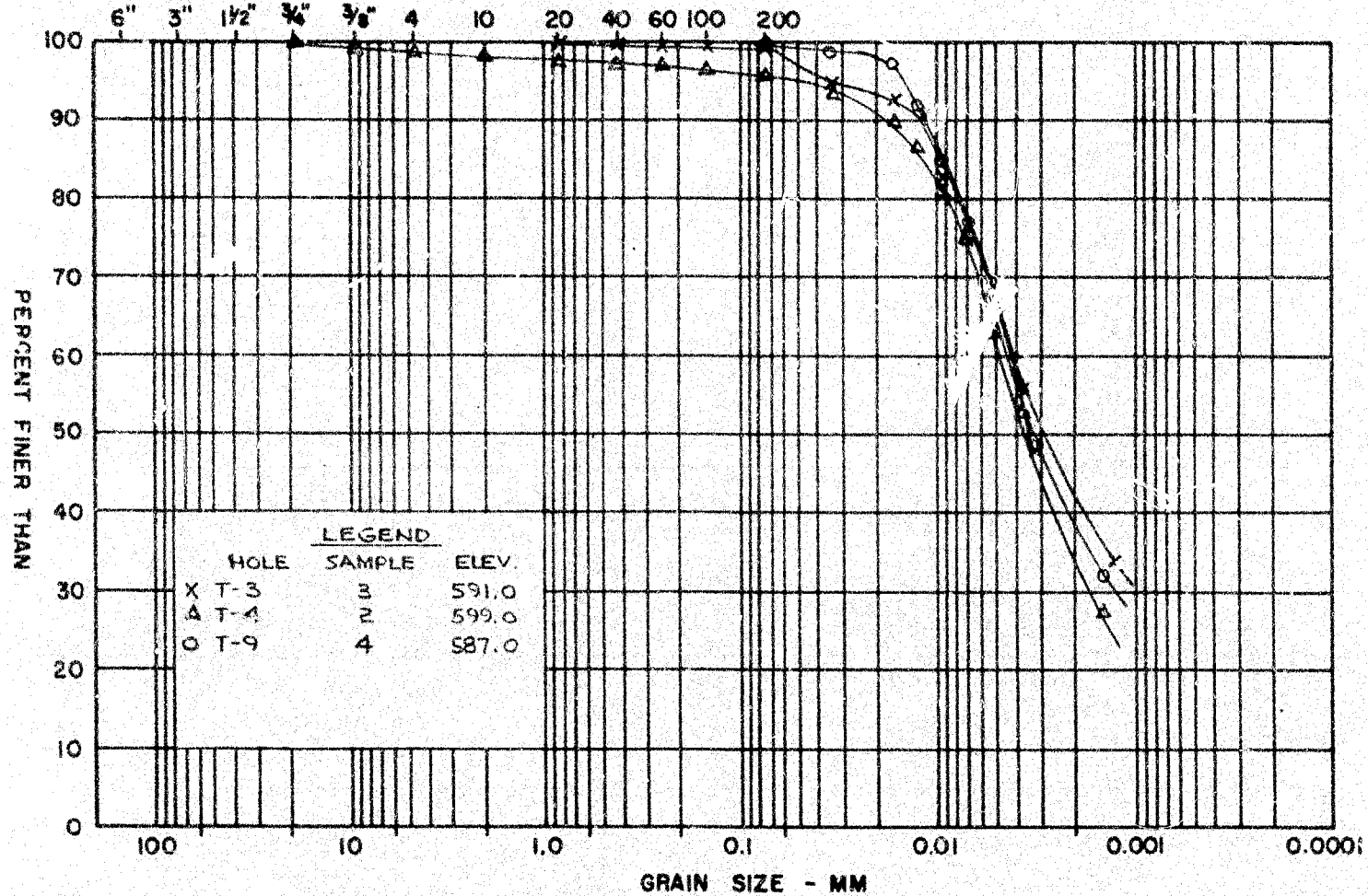
APPENDIX I

GOLDER & ASSOCIATES



M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.



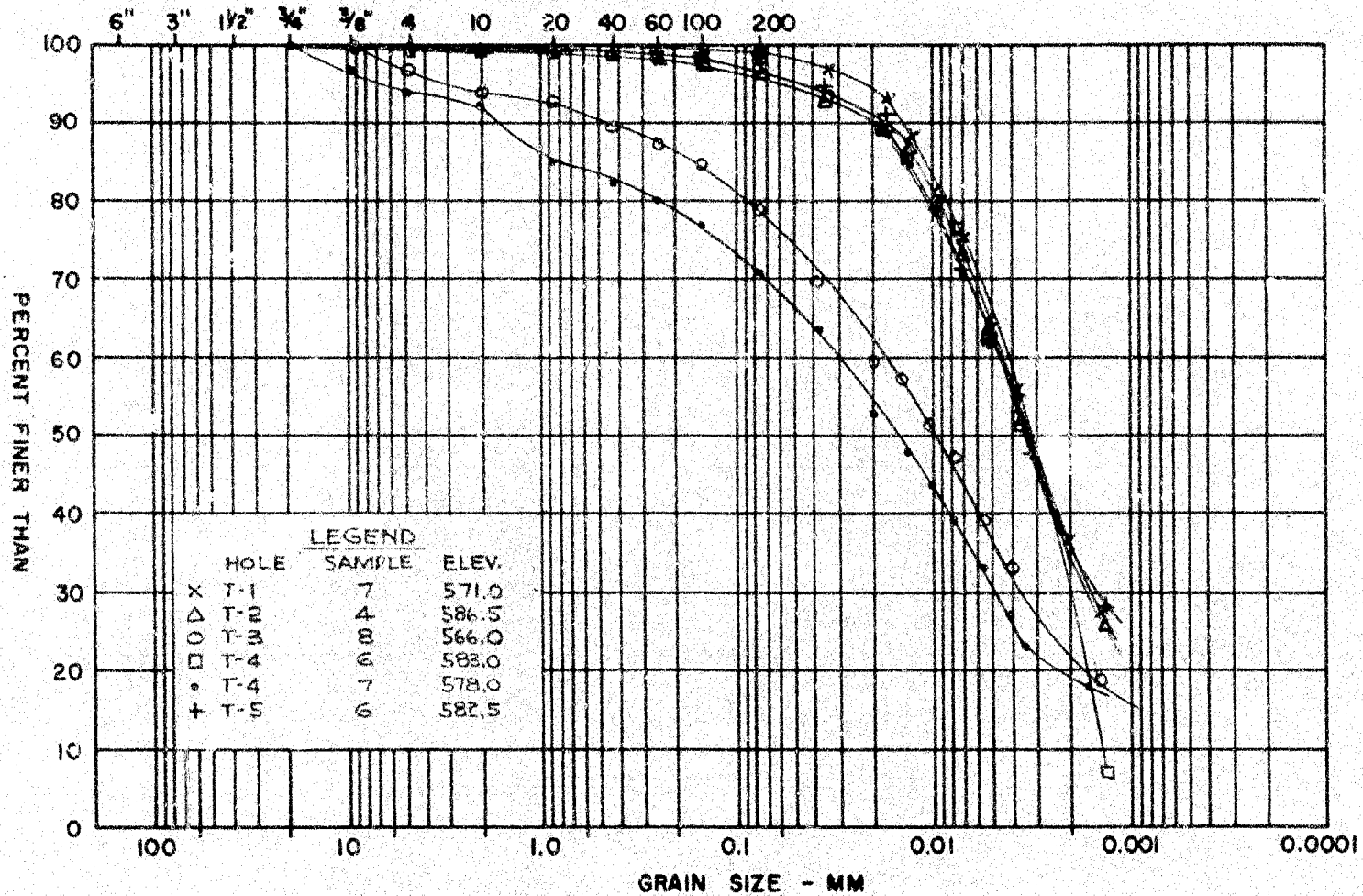
GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT (CRUST)

FIGURE 20

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS.    U.S.S. SIEVE SIZE - MESHES/IN.



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

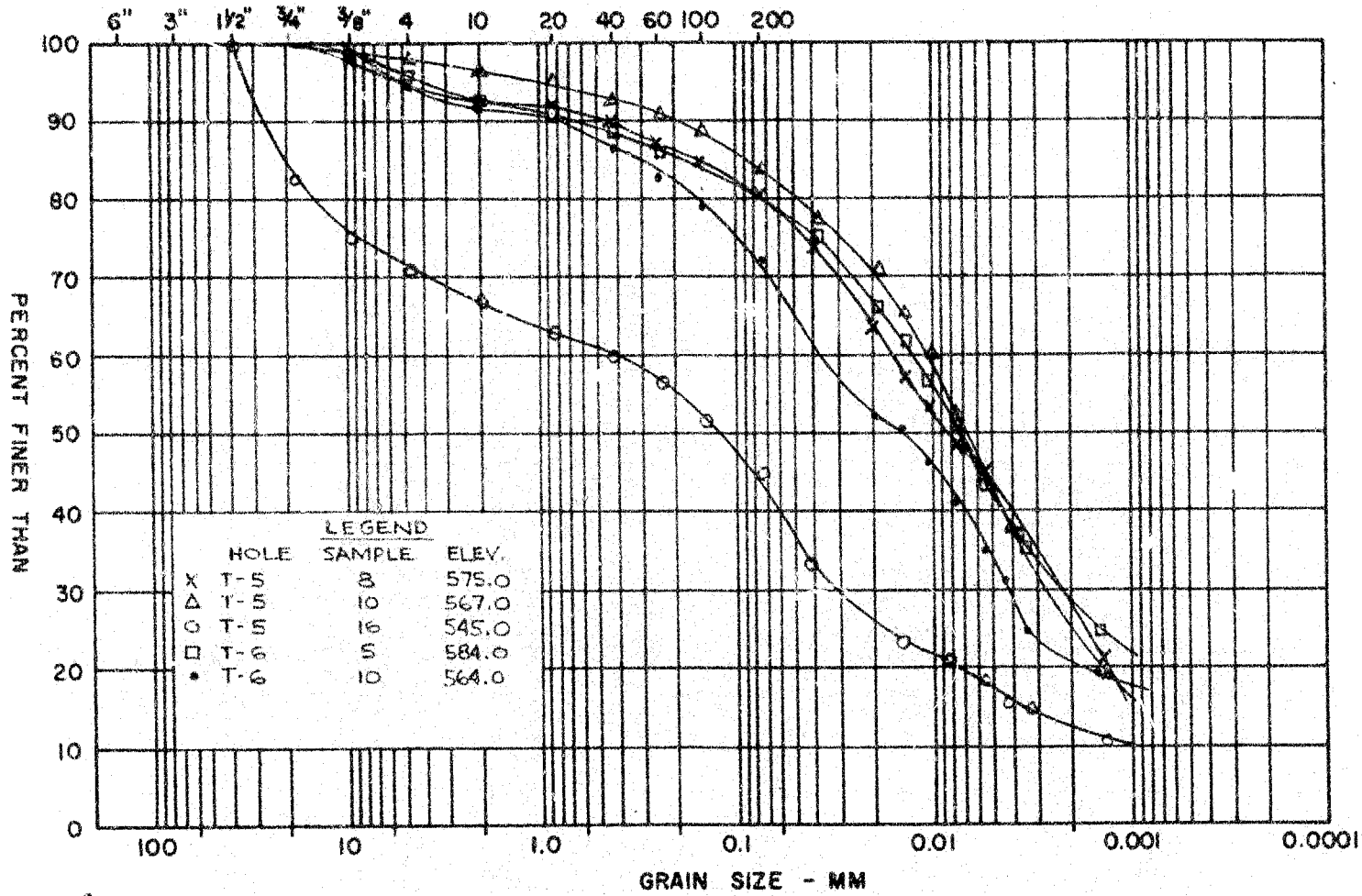
GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

FIGURE 21

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING-INS. U.S.S. SIEVE SIZE - MESHES/IN.



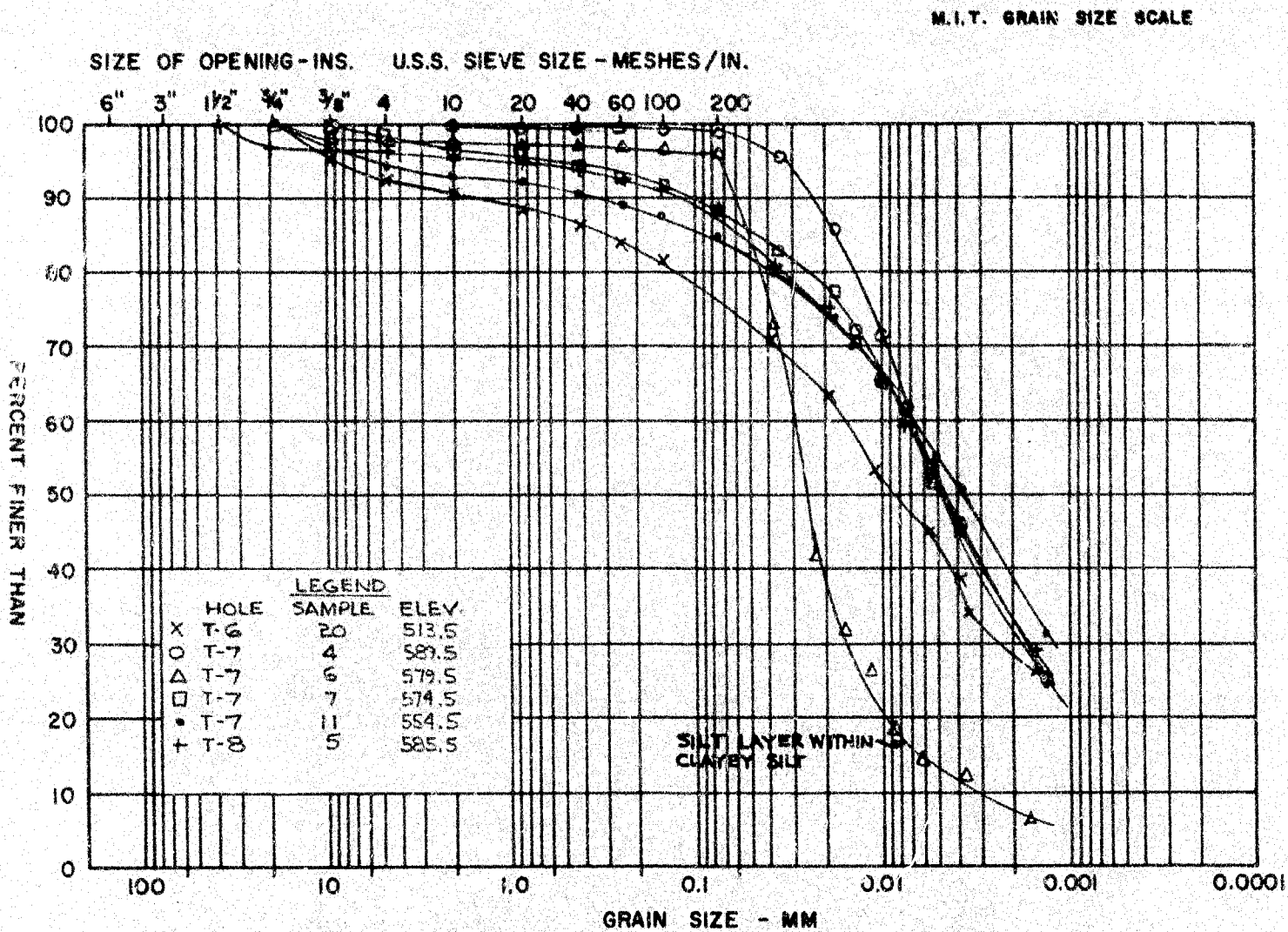
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

FIGURE 22

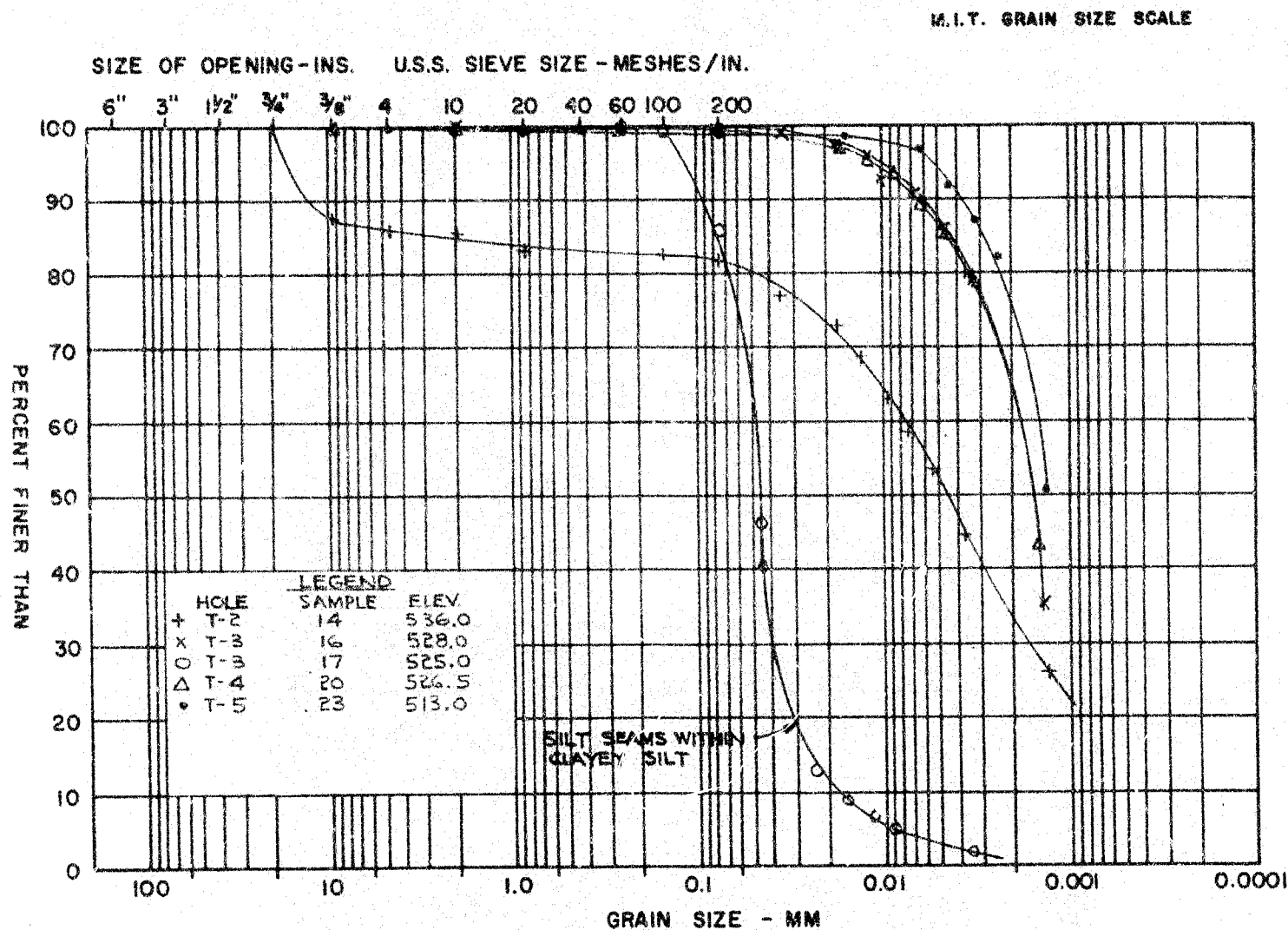
GOLDER & ASSOCIATES



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

FIGURE 23



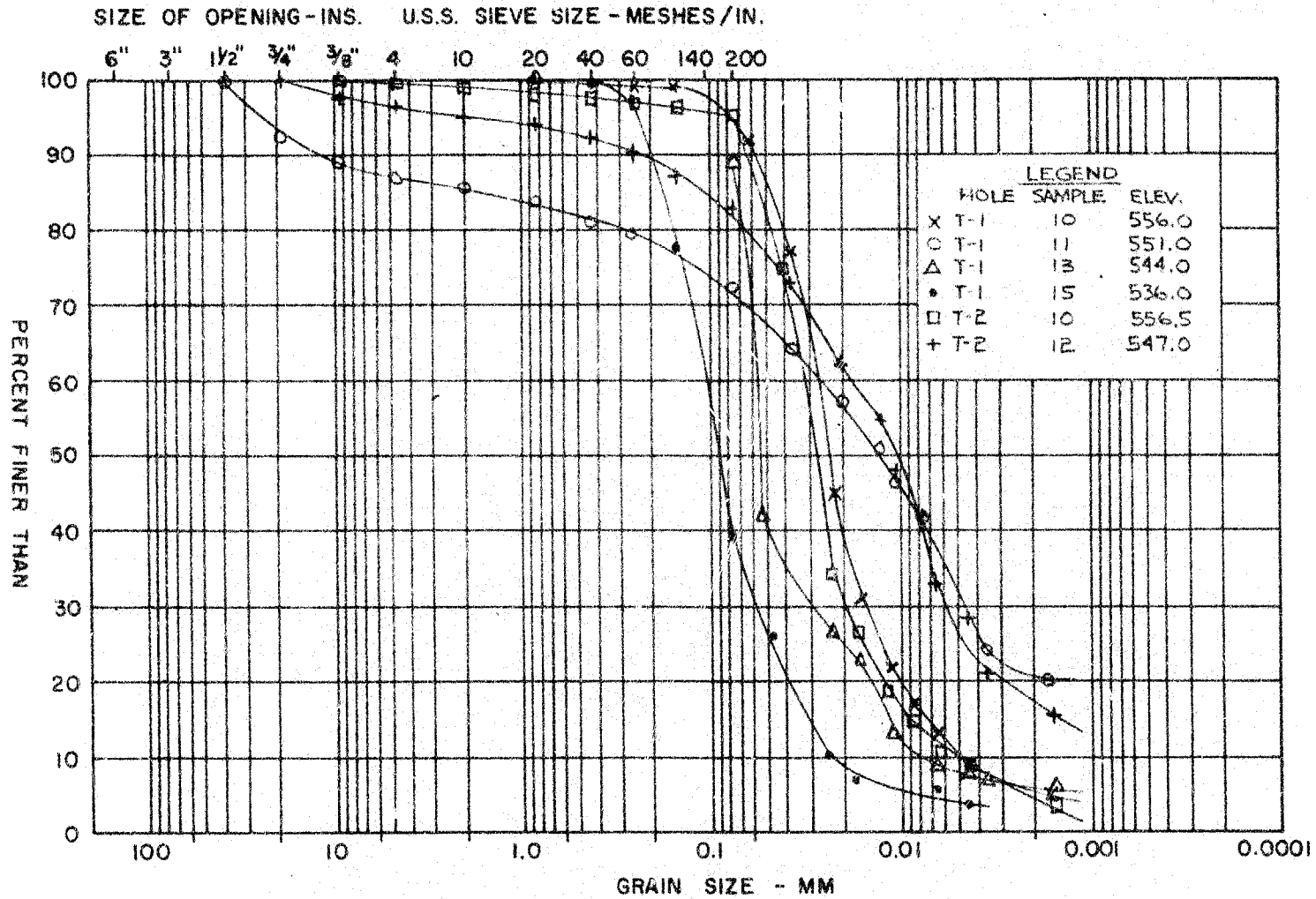
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

**GOLDER & ASSOCIATES**

**GRAIN SIZE DISTRIBUTION**  
CLAYEY SILT TO SILTY CLAY (LOWER ZONE)

FIGURE 24

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

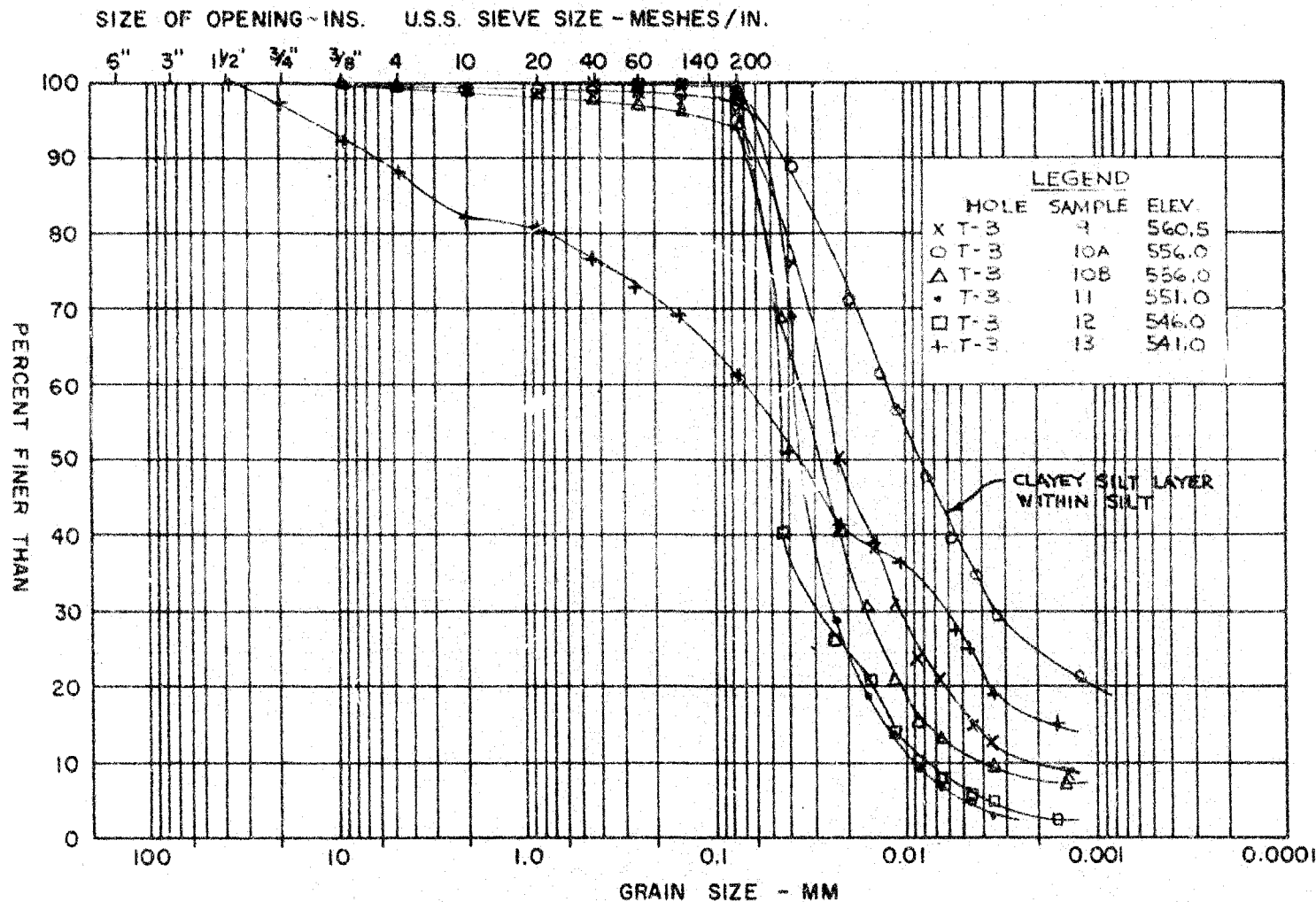
GRAIN SIZE DISTRIBUTION  
SILT TO SANDY SILT (UPPER)

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

FIGURE 25



M.I.T. GRAIN SIZE SCALE

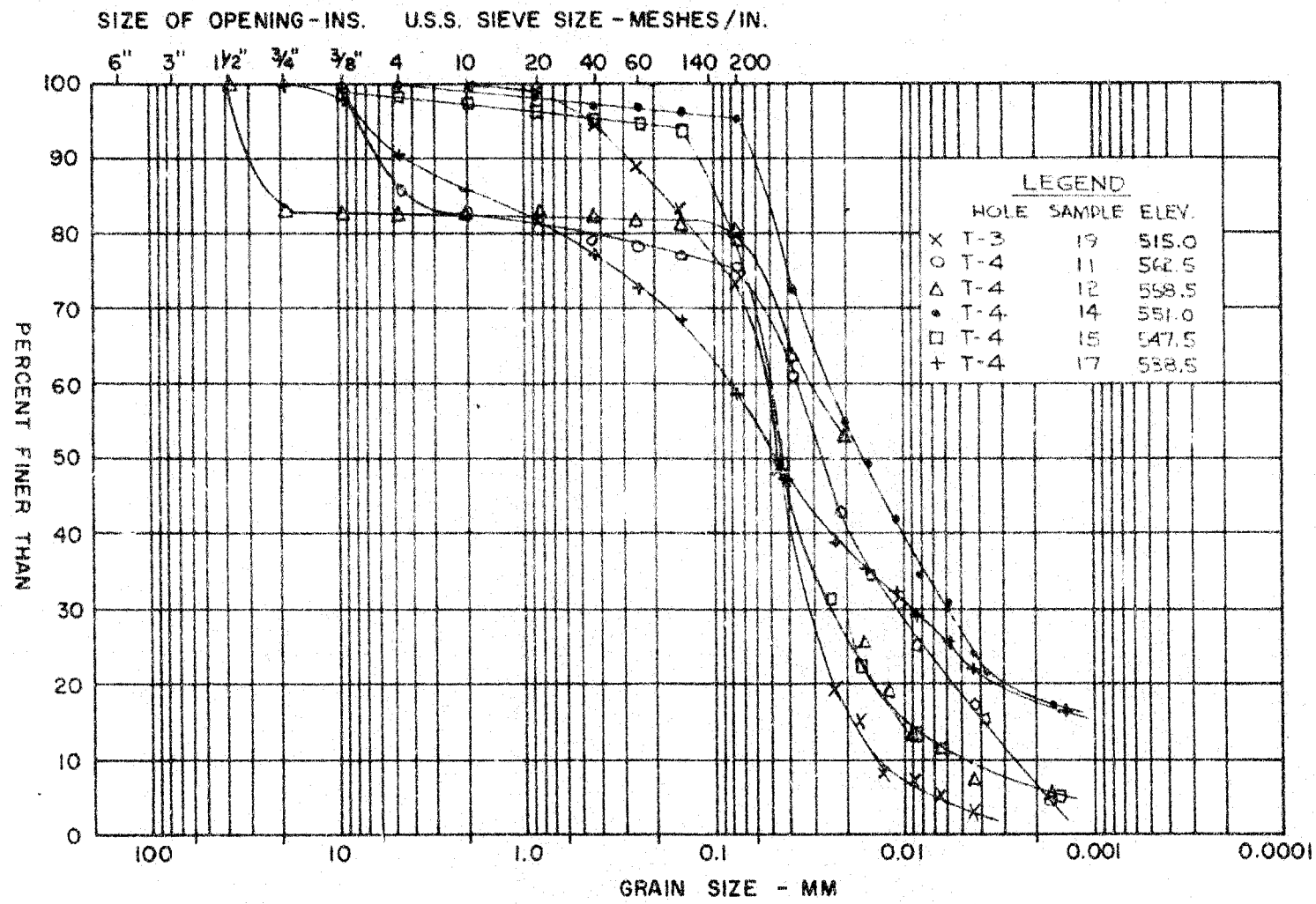


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
SILT TO SANDY SILT (UPPER)

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY-SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

M.I.T. GRAIN SIZE SCALE



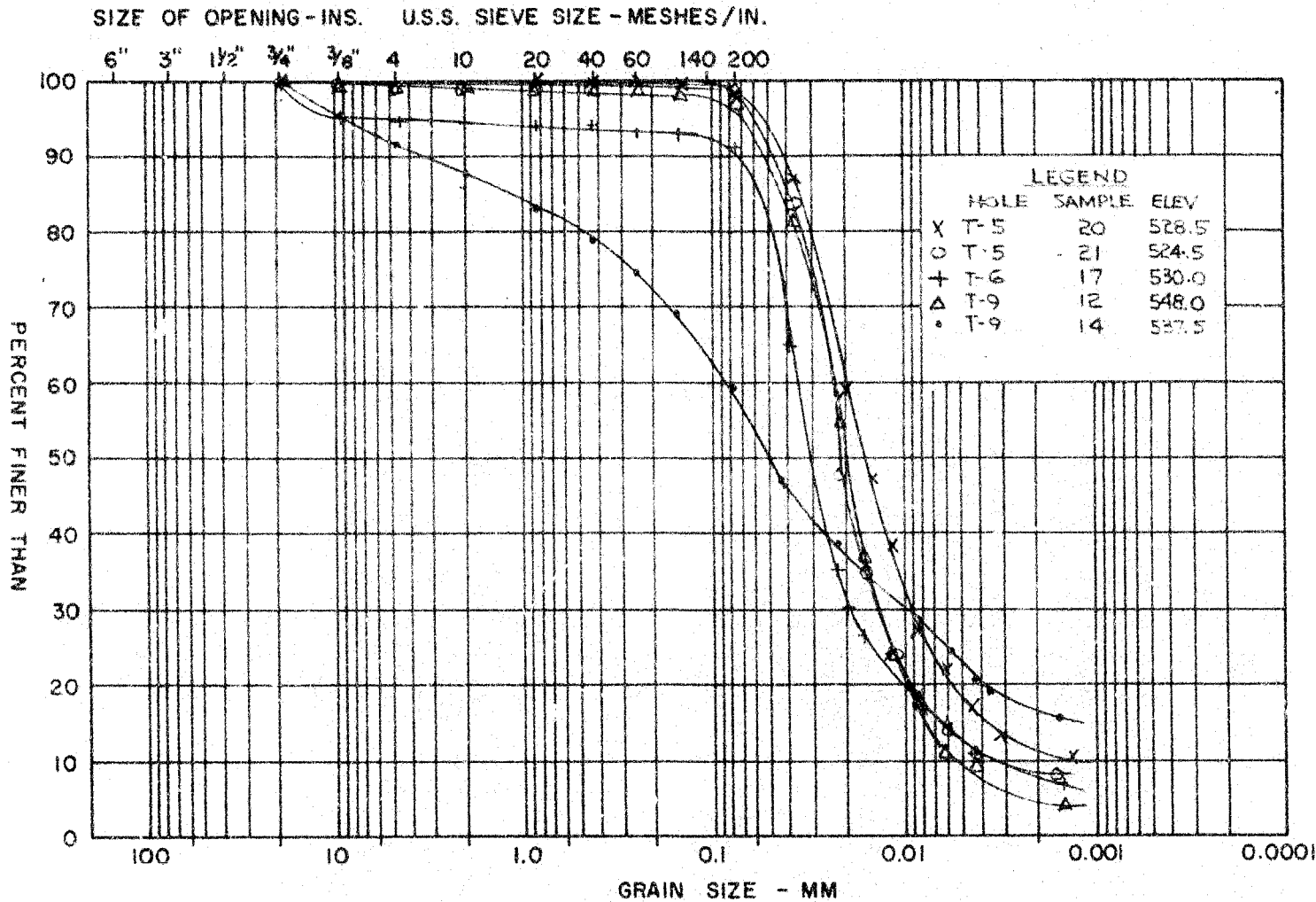
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
SILT TO SANDY SILT (UPPER)

COBBLE SIZE	COARSE GRAVEL SIZE	MEDIUM GRAVEL SIZE	FINE GRAVEL SIZE	COARSE SAND SIZE	MEDIUM SAND SIZE	FINE SAND SIZE	SILT SIZE	CLAY SIZE



## M.I.T. GRAIN SIZE SCALE



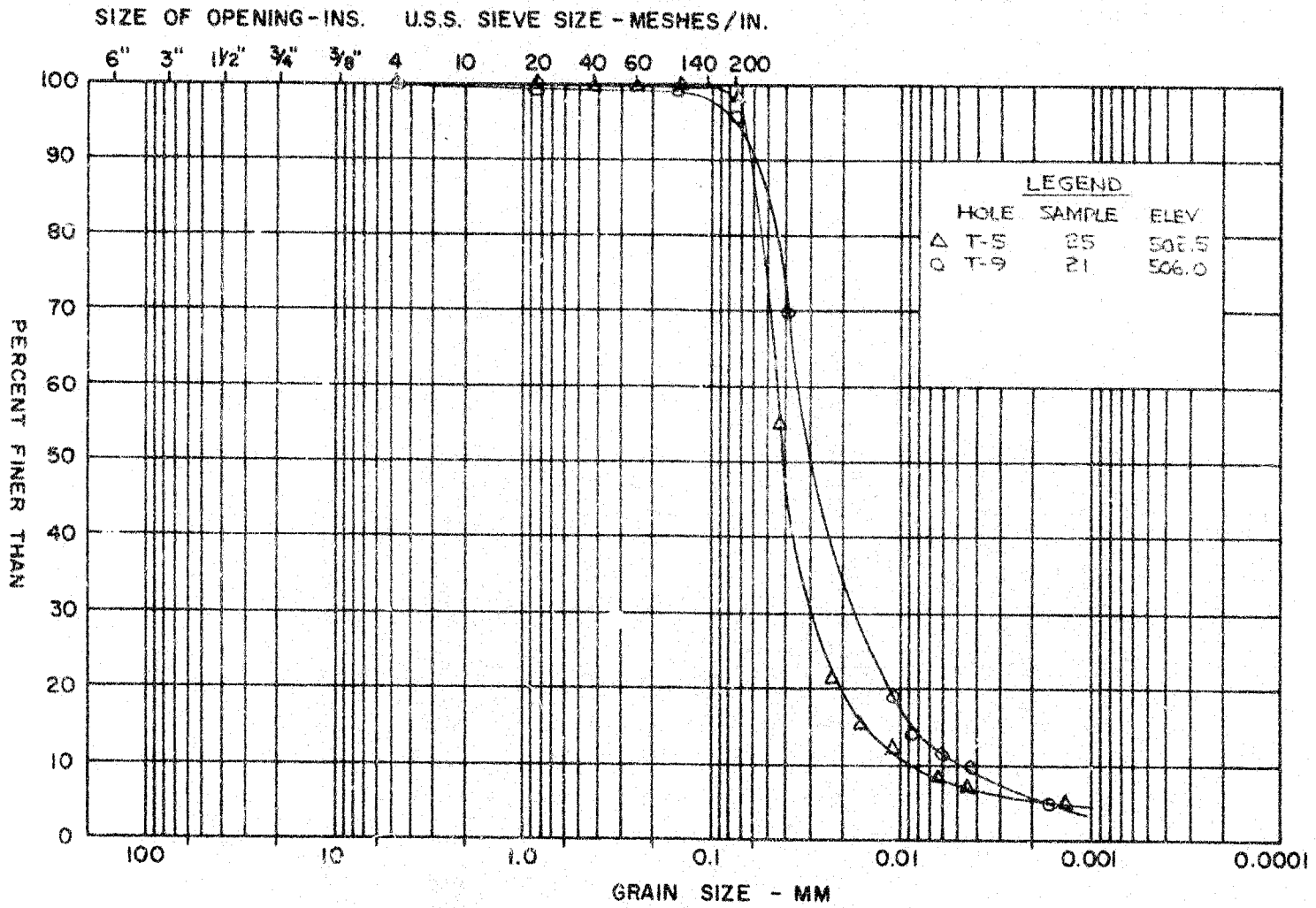
GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION  
SILT TO SANDY SILT (UPPER)

FIGURE 28

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

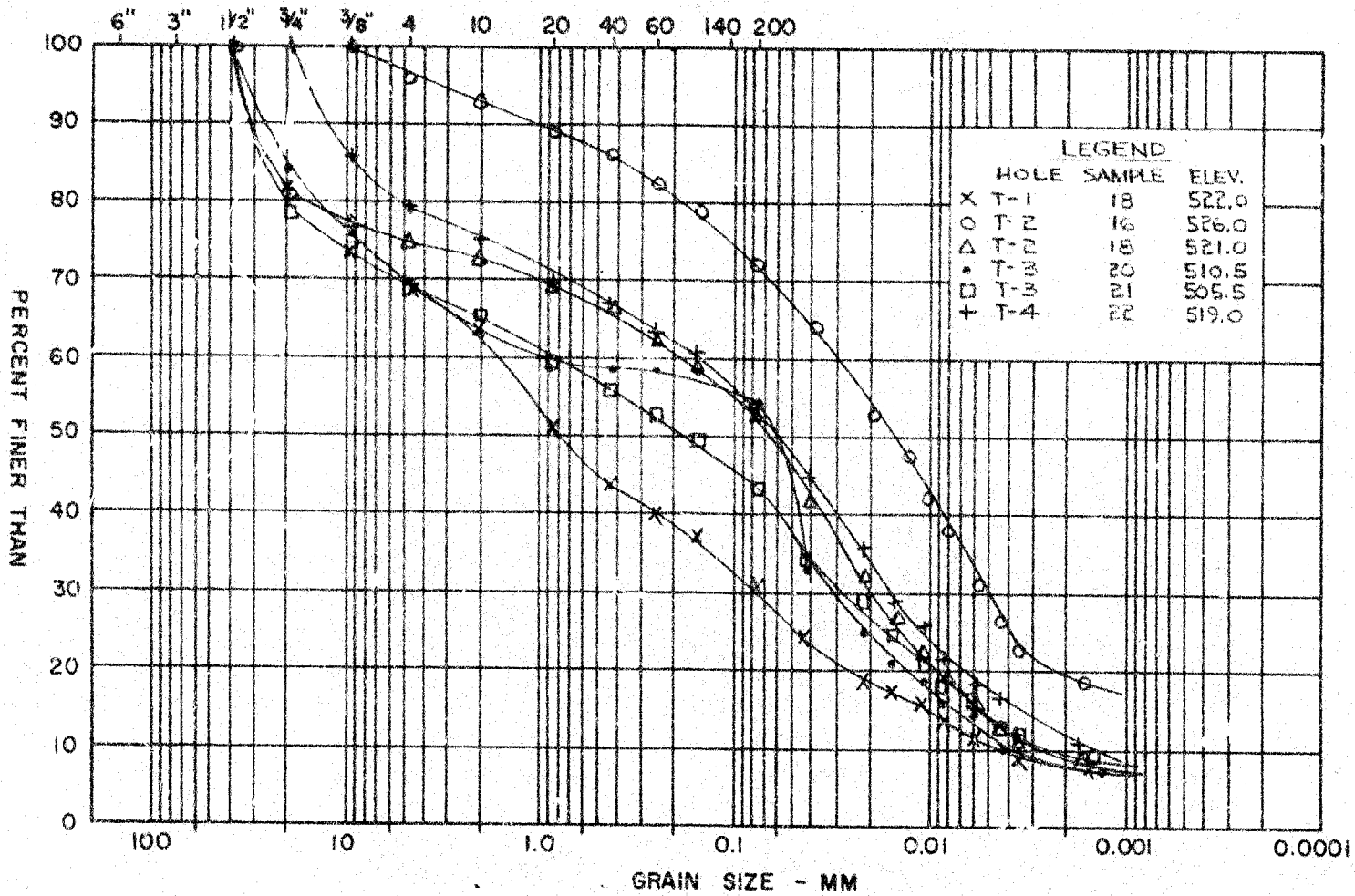
GRAIN SIZE DISTRIBUTION  
SILT (LOWER)

FIGURE 29

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.



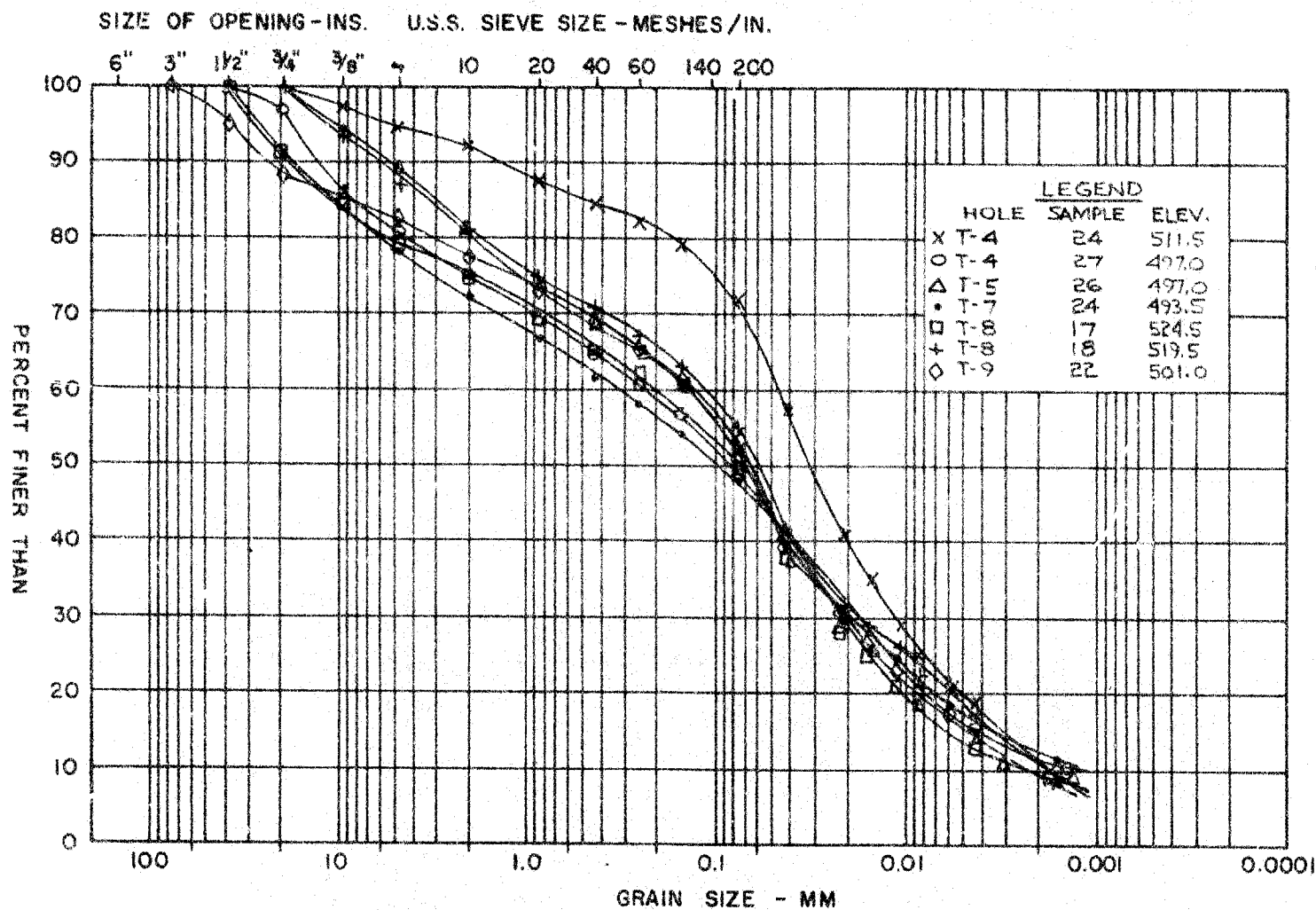
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
TILL

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

FIGURE 30

M.I.T. GRAIN SIZE SCALE



GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION

TILL

FIGURE 31

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

W.P. 240-6's Re - Foundation Survey

Mr. B. R. Davis  
Bridge Engineer  
Bridge Division

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

Attn: Mr. S. McComb

May 15, 1967

MAY 15 1967

FOUNDATION INVESTIGATION REPORT FOR D.H.C.  
BY: H. C. Golder and Associates, Limited -  
Proposed Crossing of the Re-Aligned Welland  
Canada - Main Street East - Welland, Ontario  
District #4 --- W.P. 240-66

Attached please find the foundation investigation report for the above-mentioned structure. The report was prepared and submitted by the consultant H. C. Golder and Associates Ltd.

The purpose of this report is to provide sufficient pertinent information for the preliminary design of this crossing. We feel that the report fulfills this intent. There are a number of questions that are not definitely answered and very likely some more work will have to be carried out.

The only aspect of the report that we would like to comment on at this time is the 3 ft. thick sand blanket recommended for the temporary slope protection. We feel that, although certainly beneficial, this measure would be too expensive to implement. The problems created by dispensing with this blanket could be taken care of by relatively inexpensive maintenance.

There will be, no doubt, a great number of problems that will have to be resolved in the course of the design stage, and we would suggest that you call on our office as these problems arise.

AGB:mt  
attach.

cc: Messrs.: B.R.Davis (2)

H.A. Tregeakes

D.W. Farren

G.H. Hunter (2)

H. Greenland

W.B. Melnyshyn

T.J. Kovich

B.A. Singh

St. Lawrence Seaway Authority

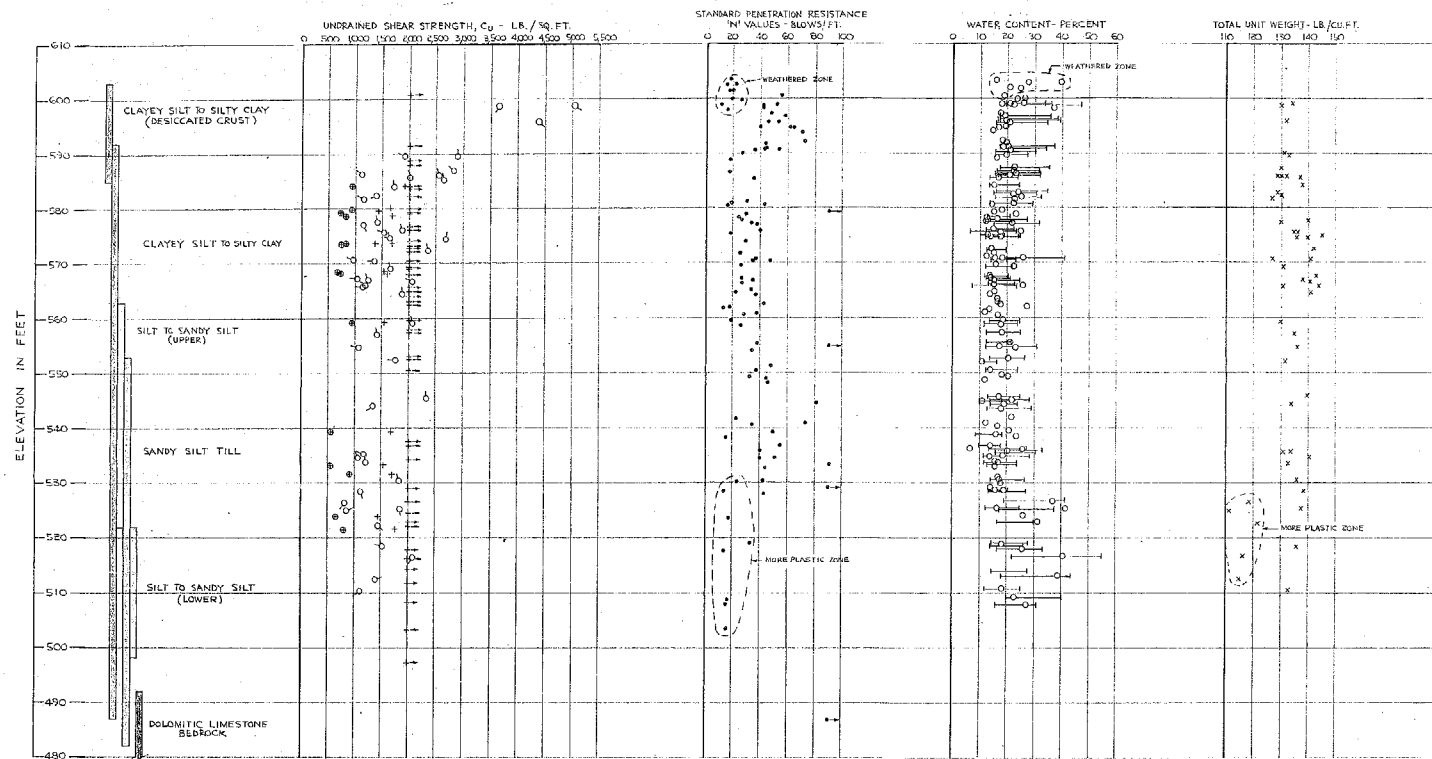
*A. C. Sternes*  
A. C. Sternes  
PRINCIPAL FOUNDATION ENGINEER

Foundation Files

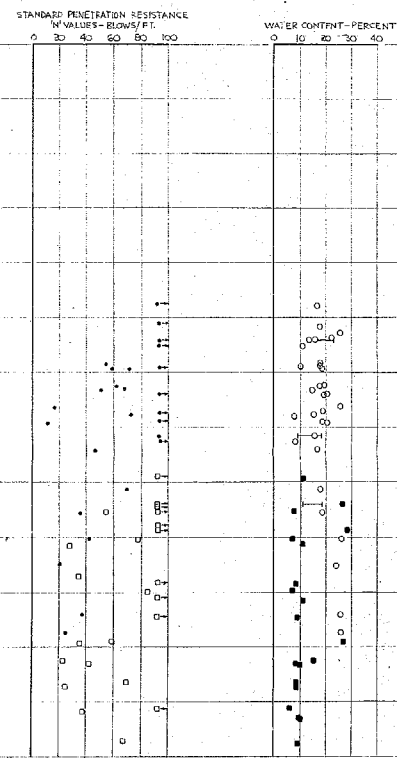
General Files



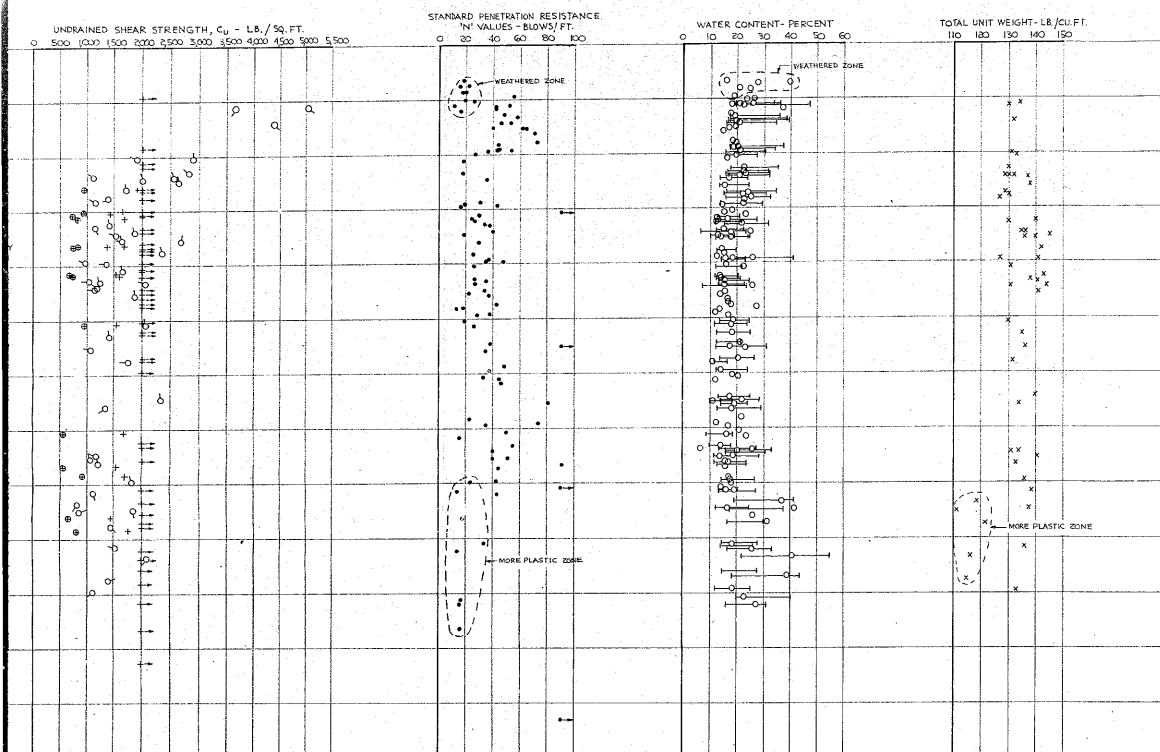
# CLAYEY SILT TO SILTY CLAY STRATUM



# SILT AND TILL STRATA



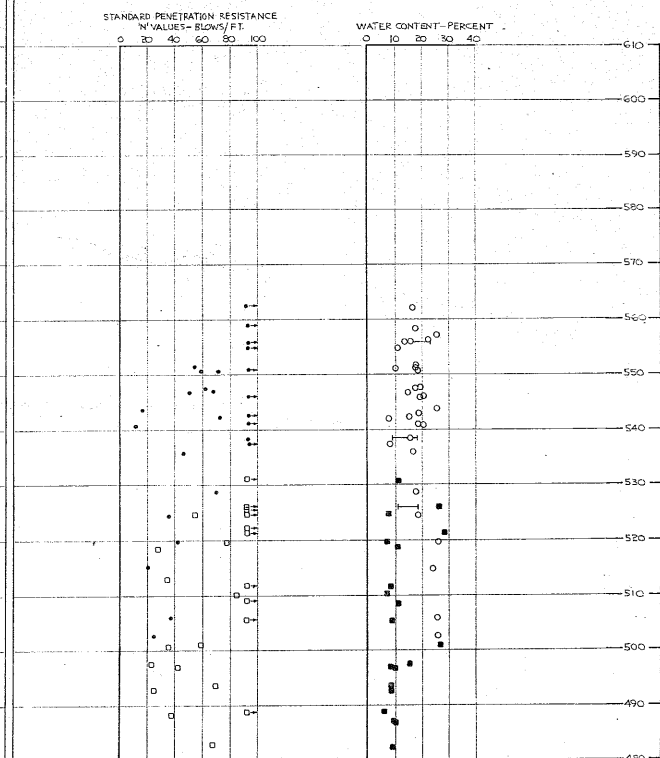
CLAYEY SILT TO SILTY CLAY STRATUM



○ LABORATORY UNDRAINED TRIAXIAL TEST  
+ IN SITU FIELD VANE TEST  
■ REMOULDED FIELD VANE TEST  
15-20 PERCENT AXIAL STRAIN AT FAILURE

Wp W WL

SILT AND TILL STRATA



• SILT TO SANDY SILT  
○ SILT TO SANDY SILT  
□ SANDY SILT TILL  
■ SANDY SILT TILL

Drawn: MAY 4, 1967

GOLDER & ASSOCIATES

Made: J.A.  
Chkd: J.A.  
Appd: J.A.



LEGEND

- ① TEMPORARY DEEP FILTERPITS (NOMINALLY 5 TO 10' IN DIA. WITH EXPOSED AND GRAVEL BACK FILL)
  - WELLS SHOULD BE INSTALLED AT EL. 500 AND EXTEND ABOUT 2' INTO BEDROCK TO AVOID RISK OF TONNAGE.
  - FILTERING DRAIN WELLS SHOULD BE INSTALLED ONLY EXCAVATION HAS REACHED EL. 500.
- ② TEMPORARY DEEP FILTERPITS (DEEP PLANT TANKS)
  - WELLS SHOULD BE INSTALLED AT EL. 500 AND EXTEND ABOUT 2' INTO BEDROCK TO AVOID RISK OF TONNAGE.
  - PROVISION SHOULD BE MADE TO COLLECT AFTER THE EXCAVATION (BY MEANS OF SHALLOW DEEP ETC.) AND BE REMOVED AWAY WITH DUMP.

SEQUENCE OF EXCAVATION

① - STAGE NUMBER

REFERENCE: DEPARTMENT OF HIGHWAYS, ONTARIO DRAWING NO. D-5173-5, WELLAND CANAL DIVERSION TUNNEL - EAST MAIN, EXCAVATION - 3 STAGES - SCHEME 'X', DATED JULY '67.