

#61-F-230C

Q.E.W.

BURLINGTON

SKYWAY



ONTARIO

DEPARTMENT OF HIGHWAYS

Box 279, Burlington  
December 8, 1961

MEMORANDUM TO:

Mr. A. Rutka  
Materials and Research Engineer  
Downsview, Ontario

Re: Town of Burlington - Sanitary Sewer Crossing the  
Q.E.W. at the North Approach to the Burlington Skyway

You may recall some time ago I mentioned to you that the Town of Burlington had encountered a difficult soils problem which they felt precluded the possibility of tunnelling under the Q.E.W. for their sanitary sewer main.

I now enclose a copy of a Soils Report from H. Q. Golder & Associates Ltd. and this report outlines the problem involved.

I have arranged with Mr. J. B. Wilkes to hold a meeting in one of the Board Rooms at Downsview on December 21, 1961 at 9:00 a.m. for the purpose of discussing this problem. It is my intention to have yourself, Mr. Wilkes, representatives from James F. MacLaren Associates and Golder & Associates Ltd., the Town Engineer and myself attend this meeting so that we may arrive at a solution satisfactory both to the Town and the Department.

Will you kindly review the attached report and advise me whether or not the date is suitable where upon I will arrange to have the Town and their Consultants notified.

*J. C. Thatcher*

Attach.  
JCT:ms

J. C. Thatcher  
District Engineer

c.c. J. B. Wilkes

*ar*

JAMES F. MACLAREN ASSOCIATES  
CONSULTING ENGINEERS

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61 F 230 C

SOIL CONDITIONS AND ENGINEERING STUDY  
INSTALLATION OF WEST END SANITARY TRUNK SEWER  
BURLINGTON ONTARIO

Distribution:

- 5 copies - James F. MacLaren Associates,  
Toronto, Ontario.
- 2 copies - H. O. Golder & Associates Ltd.,  
Toronto, Ontario.

November, 1961

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## INTRODUCTION

H. Q. Golder & Associates Ltd., have been retained by James F. MacLaren Associates under the terms of a letter dated September 29th, 1961, to provide consulting services in relation to the installation of a 48 inch diameter sanitary trunk sewer in the vicinity of the Queen Elizabeth Way and Highway No. 2, in Burlington, Ontario.

Essentially, the problem is to construct the sewer approximately 10 feet below ground level in an organic silt or fine sand stratum where the water level is close to ground surface. The construction is to be in open cut wherever possible, but it has been specified that where the proposed sewer crosses under the Queen Elizabeth Way the construction is to be in tunnel. It is understood that this has been specified so that there will be no temporary restrictions to traffic flow on the Queen Elizabeth Way during construction and so that damage to the existing pavement may be avoided. Whether or not this latter reason is valid is discussed later.

This report deals with the soil conditions at the site as assumed from the results of borings carried out by Raymond Concrete Pile Company Limited and discusses the relation of the soil and water conditions at the site to certain aspects of design and construction of the proposed sewer.

INFORMATION SUPPLIED

Through discussion with Mr. D. Sexsmith and Mr. T. Low of James P. MacLaren Associates, various data concerning the proposed sewer have been supplied to us. The significant data supplied are listed below:

- i) Drawing: File No. 506-D-3926: Title, "Easements required between No. 2 Highway and Greenwood Drive".
- ii) Drawing: File No. 506-D-3938: Title, "Boring Locations at Queen Elizabeth Way and Highway No. 2".
- iii) Sketch plan showing the locations of boreholes and a cross-section along the proposed sewer between chainages 20+00 and 29+00.
- iv) Sketch plan showing the locations of boreholes and a cross-section along the proposed sewer between chainages 30+00 and 40+00.
- v) Boring logs and report by Raymond Concrete Pile Company Limited, No. Raylim. B-1203-T: Title, "Preliminary Logs and Results, West End Sanitary Trunk Sewer, Burlington, Ontario". Dated September/October, 1961.

- vi) Various sketches from Mr. J. Hodd, Boring manager, Raymond Concrete Pile Company Limited, showing the locations of piezometers numbered P1 and P2 and the water level readings taken in these piezometers between the periods October 1st, 1961 to November 6th, 1961.

#### PROCEDURE

Mr. V. Milligan, H. Q. Golder & Associates Ltd., visited the site with Mr. J. Hodd of Raymond Concrete Pile Company Limited, on September 25th, 1961, and discussed the progress of the boring work which had been going on for several weeks prior to that visit. Following this site visit recommendations were made for further work to be carried out. The recommendations were:

1. To carry out dynamic penetration tests to determine the approximate thickness of the organic silt stratum at the interchange of the proposed sewer line, Highway No. 2 and the Queen Elizabeth Way approach ramps and adjacent to borehole 5A, shown on Figure 1.
2. Once the thickness of the organic silt is known, to install two piezometers, P1 and P2, at the depths specifying  $1/3$  thickness of the organic silt stratum. The locations of the piezometers are shown on Figure 1 and further details shown

on Figure 10. The installation was to be carried out in the manner described in MRC, Division Building Research, Bulletin No. 37.

3. To take further thin wall tube samples of the organic silt at approximately the mid-thickness of the stratum in the area of Borehole 5A.
4. To put down one detailed borehole B.H. 104 adjacent to the Queen Elizabeth Way between the proposed location of manholes 11 and 12, as shown on Drawing File No. 506-D-3926 and Figure 1. The samples from this borehole were to be returned to H. Q. Golder & Associates Ltd.
5. To carry out identification tests on at least two samples from each stratum as defined from the boreholes previously put down by Raymond Concrete Pile Company Limited at the site.
6. To continue water level reading, wherever possible, in the boreholes already completed.

The locations of the boreholes together with a summary of the results of the boreholes are shown on Figure 1. The summary was compiled from the drillers' borehole logs of Raymond Concrete Pile Company Limited and from visual examination and the testing of a limited number of samples. The results of the laboratory testing are plotted on Figures 1 to 9 inclusive.

The elevations in the report are taken from the information supplied to us and are assumed to be referred to Geodetic Datum.

#### SITE TOPOGRAPHY AND GEOLOGY

The proposed sewer will run approximately parallel to the Queen Elizabeth Way in a south-easterly direction at the northern end of the spit of land known as Burlington Beach. The ground level in this area is quite flat and is generally about 5 to 15 feet above the level of Lake Ontario. The embankment for Queen Elizabeth Way was constructed in the period 1954-1956 and rises approximately 20 feet above general ground level.

The Burlington-West area is located close to the northerly shore of the water area known as Hamilton Harbour. The east side of the harbour is separated from Lake Ontario by Burlington Beach, a sand and gravel spit or bar several hundred feet in width through which Burlington Canal has been cut. The old river channel, now silted up and partially filled in, which connected the present Hamilton Harbour to Lake Ontario, was located some 2,500 feet north of the canal.

Geological deposits overlying bedrock in the Burlington area are mainly of late pleistocene glacial origin. In pre-glacial times the old Dundas valley, which underlies the present Hamilton Harbour, was carved into the Niagara escarp-



ment by the drainage system of the old Eriean Rivers, exposing red Queenston shale of upper Ordovician age below the escarpment. During the glacial period the old drainage pattern was obliterated and covered by the Halton till which overlies the shale bedrock over most of the area. As the glaciers receded Lake Iroquois was formed and flooded the district to a height of about 100 feet above the level of Lake Ontario.

During the life of glacial Lake Iroquois the area now known as the Iroquois Plain was formed below the Niagara escarpment and with the water at a high level clays which overlie the till in the harbour proper were deposited. There followed a period in which the level of Lake Iroquois dropped to below the level of present Lake Ontario as the ice in the St. Lawrence Lowland melted allowing drainage to the sea. As the water level rose to its present level in Lake Ontario, the more recent deposits which overlie the Iroquois clays were led down. The main feature of this last period is the extensive sand and gravel bar forming Burlington Beach.

In the area to be traversed by the proposed sewer it is apparent that the major deposits are of recent origin, either silts and sands of shallow thickness overlying till or, in many cases loose organic silte directly overlying Queenston shale. Part of these deposits probably fill relatively deep depressions in the rock eroded by the old drainage outlet into Lake Ontario.

## SOIL CONDITIONS

The following main soil strata are inferred from the results of the borings put down at the site.

### Fill

Below ground level, which varies from elevation 250 to about elevation 256, a layer of heterogeneous fill was encountered. The thickness of the fill varies from about 2 feet in borehole 104 to over 16 feet in thickness in borehole 5A. It was apparently not encountered in borehole 6A. Where the fill forms a part of the Queen Elizabeth Way approach embankments or berms it is a hard heavily compacted clayey material. In other areas it is a heterogeneous mixture of sand and gravel with occasional clay pockets.

Standard penetration resistances or 'N' values measured in the fill ranged from about 9 blows to over 70 blows per foot with a general value of approximately 20 blows per foot. The relative density is generally high.

The average wet unit weight of the fill is given on the logs as about 130 lb. per cubic foot.

### Organic Silt

Underlying the fill, and within 2 feet of ground surface in borehole 6A, a stratum of organic sandy silt was encountered in the borings. The thickness of the stratum is quite erratic and varies from about 16 feet in borehole 1 to approximately 60 feet in borehole 3A. The stratum was not completely penetrated in borehole 2A, thus the complete thickness of the stratum is not known.

The composition of the organic silt stratum also varies extremely throughout its depth. Occasional sand lenses or layers as much as 2 feet in thickness were encountered in the borings and thin clay layers were noted in a number of the samples supplied to us. The thickness of these clay layers ranged from a fraction of an inch to over 2 inches. In borehole 2A, 18 inches of clay was noted in the borehole log. The colour of the clay layers is generally brownish-grey to red.

Typical grain size distribution curves for individual samples are given on Figures 2 to 5. It may be noted that the distribution of grain size in all the samples is relatively uniform, ranging in particle size from a coarse sand to silt. In general, the percentage of silt sizes was greater than about 50 percent in most of the samples examined. An exception to this were samples from borehole 5A taken above about elevation

250. Grain size distribution curves for these samples are shown on Figure 5. It is possible that these samples are from a hydraulic fill which was placed to form part of the approach ramp to Queen Elizabeth Way at the Highway No. 2 interchange.

Atterberg limits carried out on random samples show that the liquid limit of the material ranges between about 37 and 153, with corresponding plastic limits ranging from 23 to 82. The water content of the samples was generally at about 40 percent with the water content of some samples as high as 125 percent.

While an attempt has been made to distinguish between 'organic sandy silt' and 'organic silt with thin sand layers' in the summary logs for individual boreholes, there is little doubt that the composition of the stratum throughout the area of the investigation is quite erratic. Essentially it is comprised of silt sized particles with varying amounts of organic material present and with erratic thin sand or clay seams randomly dispersed throughout. For the purposes of this study the stratum has been broadly classified as an 'organic silt'.

Consolidation tests were carried out on the thin-walled tube samples from the borehole for piezometer P1. The results of the consolidation tests are plotted on

Figures 7, 8 and 9. Making allowance for sample disturbance, which is indicated by the shape of void ratio,  $e - \log$  pressure,  $p$  curves the silt stratum is probably normally consolidated under the existing overburden pressure. (This includes the weight of the embankment fill for the approach ramp to the Queen Elizabeth Way). The compression index,  $C_c$ , estimated from the tests has a probable range of 0.4 to 0.6 and the coefficient of consolidation,  $c_v$  ranges generally between .01 to .03 square inches per minute for the pressure enforced by the existing embankment fill. During the application of individual load cycles during the tests it was evident that the secondary compression of the samples could be quite high. However, it was not possible to assess the exact degree of secondary compression for each sample as the results were erratic.

The wet unit weight generally was between 83 lb. per cubic foot and 111 lb. per cubic foot with an average value of about 90 lb. per cubic foot.

Standard penetrations resistances of 'N' values measured in the stratum were generally between 2 to 4 blows per foot indicating that the silt is very loose throughout the depth of the stratum.

### Organic Clay

In contrast to the organic silt which was encountered elsewhere across the site, about 11 feet of an organic silty clay was penetrated in borehole 104. The clay contained occasional thin layers of fine sand but was generally clayey throughout the full thickness of the stratum. It also contained occasional organic matter in the form of roots.

The liquid limit ranged between 25 and 37.7 with a corresponding range in plastic limit from 17.3 to 25.6. The water content of the samples ranged between 15 and 30.7 percent. The average water content was generally at about 28 to 29 percent. This range in the Atterberg limits is typical of glacial lake clays and this layer or stratum encountered in borehole 104 must be considered as distinct from the 'organic silt' stratum in the other boreholes.

The laboratory compressive strength ranged between 440 lb. per square foot to 1,260 lb. per square foot with one value of 2,400 lb. per square foot for a sample taken close to the base of the stratum. Typical stress strain curves for the compression tests are shown on Figure 6. It may be noted that the strain at failure was usually in excess of 8 percent. It is probable therefore that the samples were disturbed in

sampling. The undrained shear strengths measured by two in-situ vane tests and noted on the drillers' borehole log were 1,300 lb. per square foot and 900 lb. per square foot respectively. The sensitivity measured was about 2 to 3. Taking possible sample disturbance into account it is probable that the undrained shear strength in-situ is about 500 to 1,000 lb. per square foot.

The wet unit weight of the samples tested, with one exception, fell within the narrow range of 106 to 113 lb. per cubic foot. The exception was a value of 134 lb. per cubic foot for the sample which had a compressive strength of 2,400 lb. per square foot. It is possible that this sample marks the upper limit of the basal Halton till underlying the clay.

#### Sand

In borehole 1A at the depth of approximately 24 feet and directly underlying the organic silt, a layer of coarse sand was encountered. The thickness of this sand could not be determined as it was not fully penetrated in the borehole.

The standard penetration resistances or 'N' values measured in the sand were one value of 31 blows per foot and one value in excess of 100 blows per foot. It may therefore be assumed that the sand is dense.

Queenston Shale Bedrock

In all of the borings, with the exception of boreholes 1A and 2A Queenston shale bedrock was encountered. The elevation of the upper surface of the bedrock is apparently quite erratic, varying between about elevation 243 and elevation 183. The upper few feet of bedrock are extremely weathered and in some boreholes have apparently the consistency of a hard clay. This bedrock is the parent material from which some of the clay layers defined in the borings are probably derived.

The variation in bedrock surface has not been completely defined across the site nor is it necessary to do so in all locations save where the invert of the proposed sewer is close to bedrock. It is possible that the extreme variation in bedrock surface is caused by erosion of the original drainage outlet from Hamilton Harbour into Lake Ontario. As noted previously this drainage outlet was historically some 2,500 feet north of the present Burlington canal and thus coincides very roughly with the site presently being investigated.



### WATER CONDITIONS

Water levels are noted on the logs of borings 1A, 3A, 4A, 6A and 104. The water level readings, which are noted on Figure 1, generally range between about elevations 254 and 246. The borings which are within the recent sand or organic silt deposits adjacent to Burlington Beach show water level readings close to Lake Ontario level at elevation 246.

The majority of the water level readings were apparently taken shortly after the completion of the borehole; therefore, they may not truly represent the ground water level as sufficient time may not have elapsed in order for the water levels to have stabilized. To check on this point two Geo-Nor porous point piezometers, P1 and P2, were installed adjacent to borehole 5A. The final water level readings taken approximately 30 days after installation of the piezometers are shown on Figure 10 and correspond approximately with the range of readings taken across the site.

### STATEMENT OF PROBLEM AND POSSIBLE SOLUTIONS

Partial cross-sections of the sewer are shown on references (iii) and (iv) under the heading "Information Supplied". From this and discussions with the staff of James P. MacLaren Associates, we understand that it has been proposed that the construction of the sewer will fall in to four main sections.

These are:

- Location*
- a) Chainages 0+00 to 4+00 approximately:  
Tunnel section underneath the existing Queen Elizabeth Way.
  - b) Chainages 4+00 to 20+25: Open cut section.
  - c) Chainages 20+25 to 26+13: Tunnel Section  
*Location?*  
below approach ramps from Highway No. 2 to the Queen Elizabeth Way.
  - d) Chainages 26+13 et seq: Open-cut section close to the Queen Elizabeth Way existing embankment.

To construct a sewer in very loose organic silt below the water table poses a number of problems. If excavation is carried into the water-logged silt it may flow like a liquid. The problem is therefore either to cut off the silt from the excavation so that it cannot flow or to change its properties in some way so that it will not flow.

One solution is to isolate the area for the proposed sewer by driving sheet piling around it and then to excavate inside the piling. This essentially will be the scheme as proposed for sections (b) and (d) above; however for sections (a) and (c), some form of tunnelling has been proposed. Methods to carry out tunnelling are considered below:

Grouting:

In certain relatively permeable soils it is possible to change their properties by injecting grouts, either of suspensions such as clay or cement, or of chemical solution such as sodium silicate and calcium chloride. As the coefficient of permeability,  $k$ , for the organic silt is probably of the order of  $10^{-4}$  to  $10^{-6}$  centimeters per second, these methods cannot be used for this site.

Compressed Air

When silt is encountered in soft ground tunnelling which is at a significant depth the method usually adopted to deal with it is to apply compressed air. The action of the air is to balance the water pressure in the silt in the face of the tunnel and thus eliminate the tendency for the silt to flow. It is the unbalanced water pressure in the silt which causes the trouble. In the present case this solution clearly cannot be used since, quite apart from the cost of using air, the overburden pressure over most of the tunnel is so small in relation to the diameter of the tunnel that there would probably be extensive loss of air during construction.

Wellpoints and/or Electro-Osmosis

A further possible solution is drainage of the silt by some means before tunnelling. This could be either by the use of a vacuum drainage well point system

and/or electro-osmosis.

Although a silt is relatively impermeable it will drain slowly under gravity. This process can be aided by applying a vacuum to the drainage points. This has the effect of adding atmospheric pressure to the gravity forces causing drainage. The phenomenon of electro-osmosis can also be used in the same way. The electrodes are placed in the ground between the drainage points and a direct current is passed in the electrodes. At these points an electro-osmotic force causes water to flow to the points. The electro-osmotic permeability is the same for all soils but in the case of silts the gravity permeability is so low that in comparison with it the electro-osmotic permeability is relatively high. It is for this reason that the process is used in silts.

It must be stressed that pre-drainage of the silt for tunnels below existing pavements would result in some settlement above the line of the tunnel due to consolidation induced by the increased overburden pressure. The composition of the silt stratum is erratic and the resulting settlement would be expected to be also erratic. The order of settlement could be several inches. This could lead to serious cracking of existing pavements and would indeed be dangerous for high speed traffic.

### Freezing

Freezing is another possible solution to the problem. By pumping a refrigerant through pipes in the silt it is possible to reduce its temperature until the water in the voids turns to ice. Excavation would then take place in a solid material. The freezing is maintained until the tunnel is completed.

This method is quite expensive. It would necessitate some disruption to traffic during installation of the refrigerant pipes. As it would not be practical to freeze the silt throughout the full depth of the stratum, capillary rise from unfrozen portions of the stratum could result in ice lensing causing heave of the existing pavement. Irregular settlement of the existing pavement would probably result once the freezing process is discontinued and the frozen soil thaws.

### Continuous Jacking

Occasionally for short lengths of small diameter tunnels it is possible to jack a pipe from an open cut excavation for the full length of the tunnel. In view of the lengths of tunnel at sections (a) and (c), namely <sup>one hundred</sup> ~~one hundred~~ feet approximately and the diameter of the tunnel, 4 feet, this method, is for this site, impractical.

From the review of possible tunnelling methods above, it is apparent that the initial reason that a tunnel section was proposed, namely to avoid endangering the existing highway pavements or disrupting traffic, cannot be fully met in tunnelling at this site. It seems logical therefore to propose that the sewer be constructed in open cut along the complete route. This would not obviate damage to the pavements but it would be a more economical construction procedure and one which should be less hazardous than tunnelling.

#### Open cut excavation

Open cuts in the loose silt will have to be adequately strutted and braced and as the ground water level is high, closed sheeting should be used.

It may be possible to effect some pre-drainage of the silt by well points and/or electro-osmosis as discussed above. This, if used, would have to be checked by wellpoint pumping tests in the field, prior to construction. Therefore it is recommended that the worst conditions be assumed; namely, that the silt is of great depth relative to the depth of excavation and that it is not possible to effect complete drainage of the silt prior to excavation.

In order to prevent boiling or heave at the base of the excavation when it is pumped dry, the sheeting should be driven to a sufficient depth to reduce the

hydraulic gradient below the excavation. The table below, has been computed on the assumptions given above together with the assumptions the silt is isotropic and of infinite extent. (This condition is obviously untrue for certain parts of the site but it is a conservative basis for computations). The table lists the required sheeting penetration, D, below the base of the excavation as a function of the total water head, H and the width of excavation, B.

<u>Width of Excavation, B</u>	<u>Penetration, D</u>	
	<u>F.S.=1</u>	<u>F.S.=2</u>
20H or more	0.25H	0.5 H
8H	0.3 H	0.65H
4H	0.35H	0.75H
2H	0.4 H	1.0 H
H	0.5 H	1.4 H
0.5H	0.7 H	-

A considerable length of the proposed sewer will be about 8 to 10 feet below ground water level. Assuming that the total width of excavation will be about 6 to 8 feet, then the length of steel sheet piling to be used at the site should be approximately 20 feet or more to give a factor of safety, F.S. of 2. This is a sensible value of F.S. to use in order to allow for variations in the soil conditions along the route.

Embankment stability, Chainages 26+13 et seq.

An open cut excavation immediately adjacent to the existing Queen Elizabeth Way embankment, as at borehole 104, could endanger the stability of the embankment. Approximate computations taking the undrained shear strength of the clay/silt equal to 500 pounds per square foot and based on a 15 foot high embankment and a 10 foot deep excavation adjacent to the toe of the embankment indicate that the following procedure should be followed:

- (i) The inner edge of the excavation should be preferably 10 feet away from the toe of the embankment and at all times no closer than 5 feet from the toe.
- (ii) The excavation should be adequately braced and strutted to resist the earth pressures due to excavation plus the embankment surcharge on one face.
- (iii) No more than 20 feet of continuous excavation should be left open at any time. Thus excavation should be staggered in units of maximum lengths of 20 feet.

Settlement, Chainages 20+25 to 26+13

The question of settlement of the sewer due to consolidation of the organic silt below the fill for embankment and berms at the interchange between Highway



No. 2 and the Queen Elizabeth Way was discussed with James F. MacLaren Associates. This point is of considerable importance whether tunnel or open cut construction is used since differential settlement could cause cracking of the completed rigid concrete sewer section.

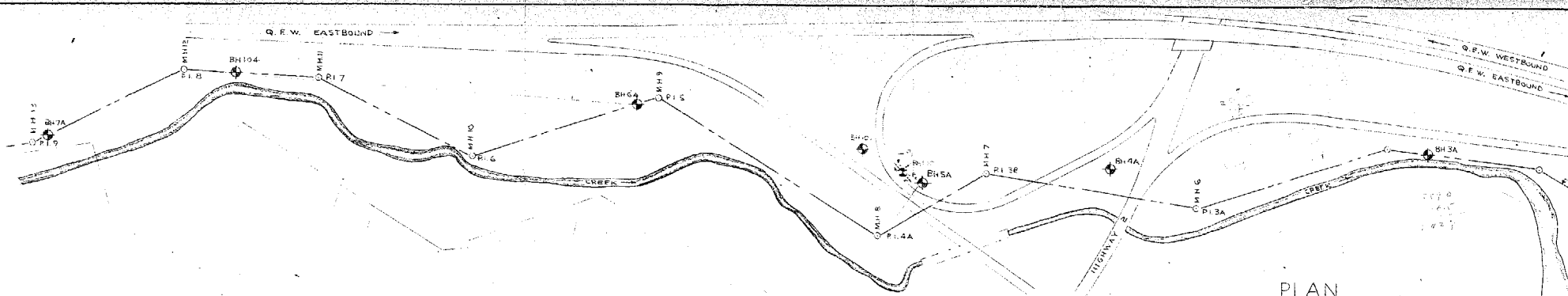
Computations, based on the measured laboratory values for the coefficient of consolidation  $c_v$  indicate that 90 percent of the consolidation of the silt (about 20 to 30 feet in thickness) below the embankment fill at chainages 20+25 to 26+13 would be complete in 2 years ( $c_v = .02$  square inches per minute) to 4 years ( $c_v = .01$  square inches per minute). It is hardly likely that  $c_v$  could have a lower value than this latter figure. As the approach embankments and berms in this area were completed about 1955 it is apparent that 90 percent consolidation should sensibly be complete. This conclusion is confirmed by the piezometric readings in P1 and P2 which fall within the normal range of ground water levels measured elsewhere at the site.

It is not possible to assess what amount of secondary compression may yet take place; however this should be relatively minor. Differential settlement of the completed sewer section should be within tolerable limits for a concrete section.

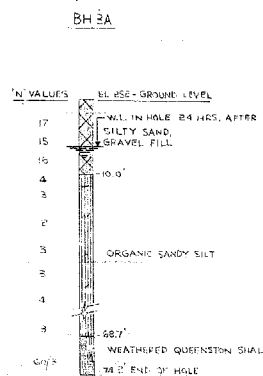
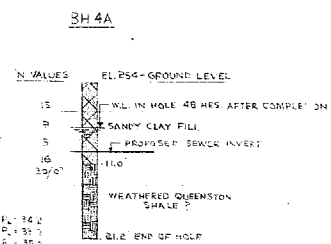
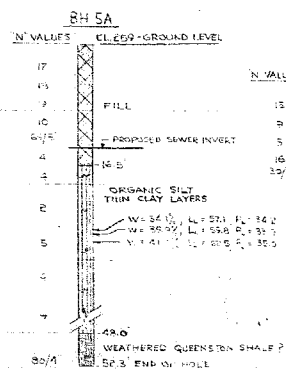
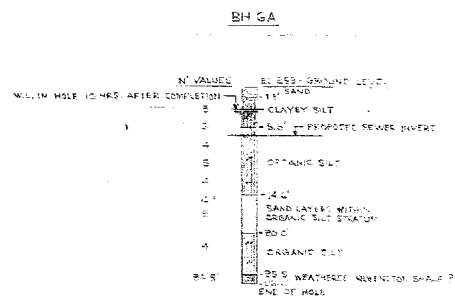
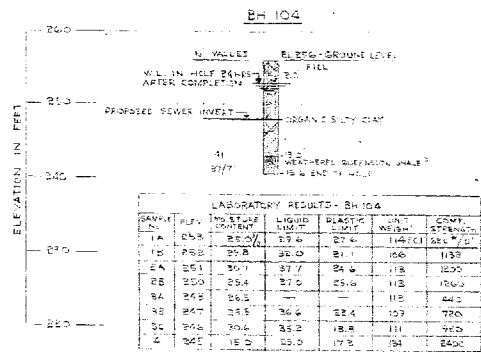
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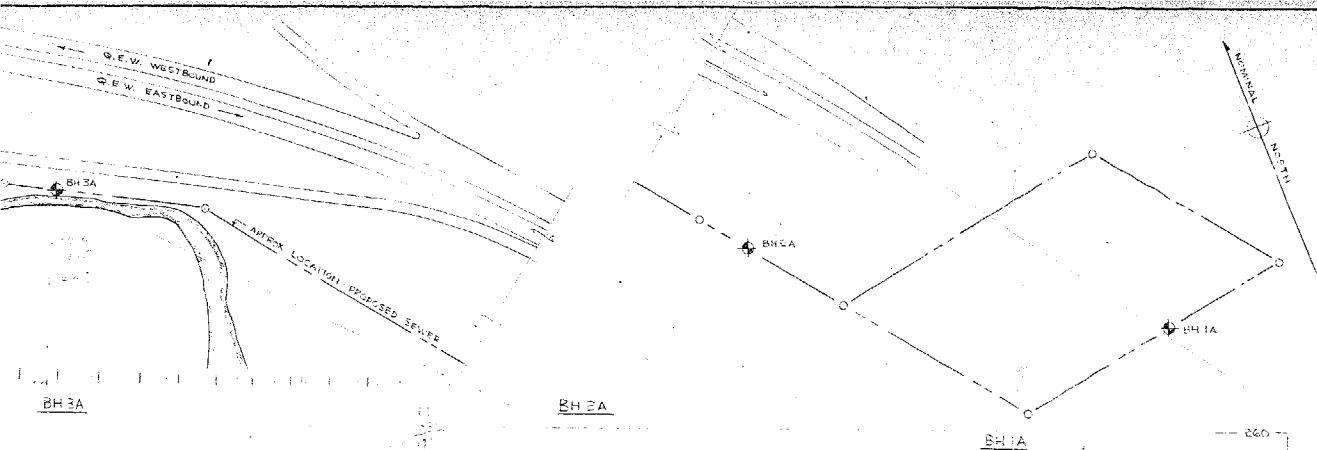
GOLDER & ASSOCIATES



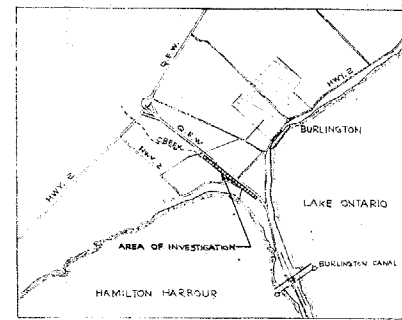


PLAN  
SCALE: 1" TO 100'

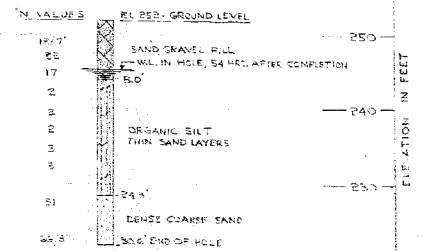
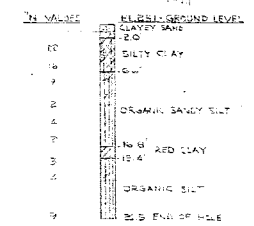
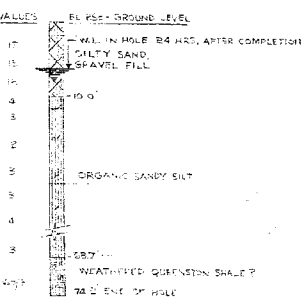




251.0  
24.4  
24.4  
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KEY PLAN  
SCALE: 1" TO 0.8 MILES



ELEVATION IN FEET

REFERENCE

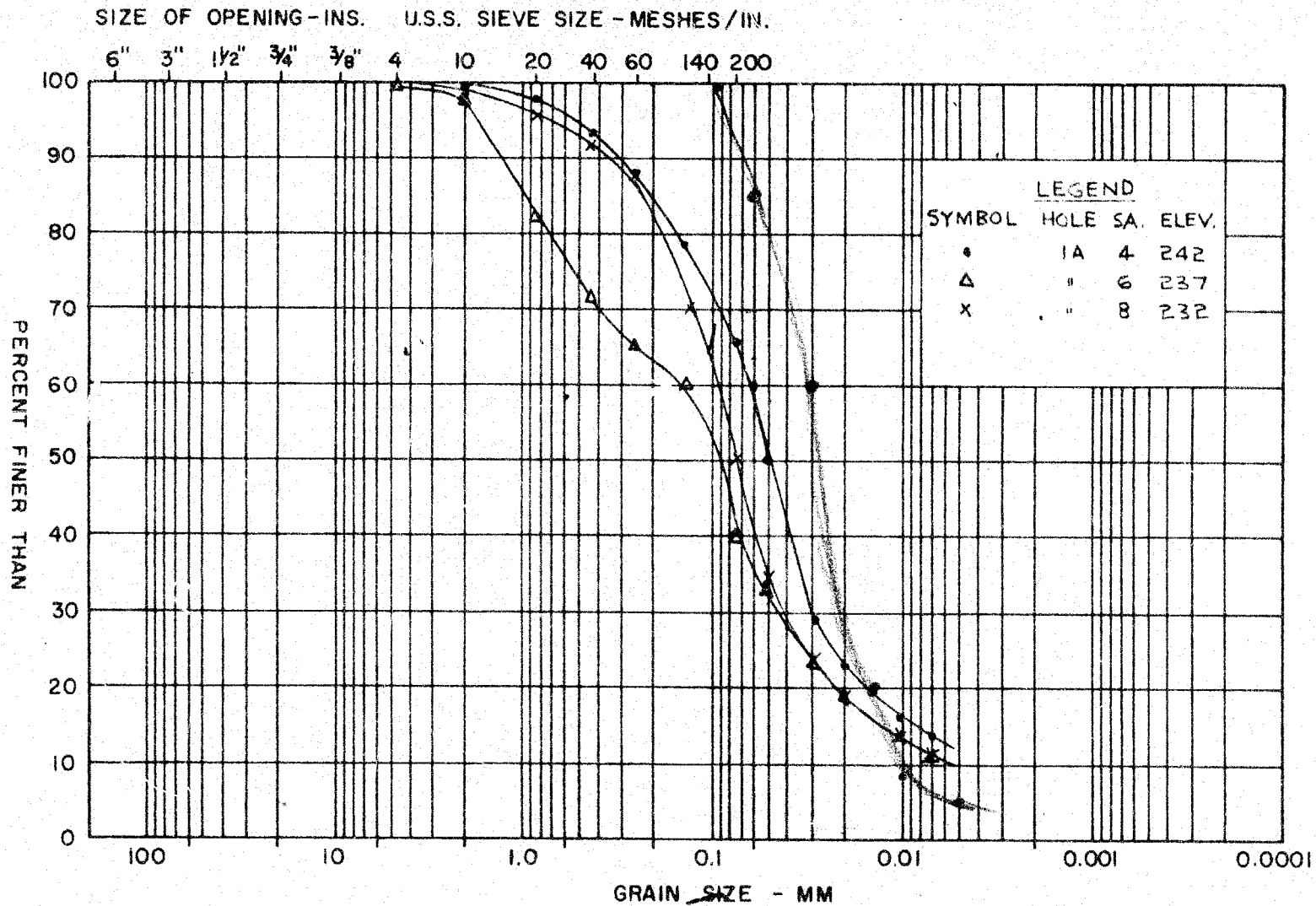
- 1. JAMES F. MACLAREN ASSOCIATES DRWG. NO. 1 OF 1 - EASEMENT REQUIRED BETWEEN NO. 2 HIGHWAY AND GREENWOOD DRIVE, DRWG. NO. 1 OF 1 - BORING LOCATIONS AT QUEEN ELIZABETH WAY AND HIGHWAY NO. 2, DATED JULY 21 & SEPT. 11, 1961 RESPECTIVELY.
- 2. JAMES F. MACLAREN ASSOCIATES - PRELIMINARY PLANS & PROFILES, NO. 2 OF 11.
- 3. RAYMOND CONCRETE PILE COMPANY LTD. - REPORT TO GOLDER & ASSOCIATES, PRELIMINARY LOGS AND RESULTS, DATED SEPT./OCT. 1961.

NOTE

LABORATORY TESTING BY GOLDER & ASSOCIATES CARRIED OUT ON SAMPLES SUPPLIED BY RAYMOND CONCRETE PILE COMPANY LTD.

JAMES F. MACLAREN ASSOCIATES TORONTO, ONTARIO		GOLDER & ASSOCIATES CONSULTING CIVIL ENGINEERS	
PROPOSED BURLINGTON WEST END SEWER BURLINGTON, ONTARIO		DATE: NOV. 23, 1961 SCALE: HORIZ. 1" TO 10' VERT. 1" TO 20'	
BORING PLAN AND SOIL STRATIGRAPHY		MADE: J.A. CHD. APPD. APPD. FIGURE 1	

## M.I.T. GRAIN SIZE SCALE



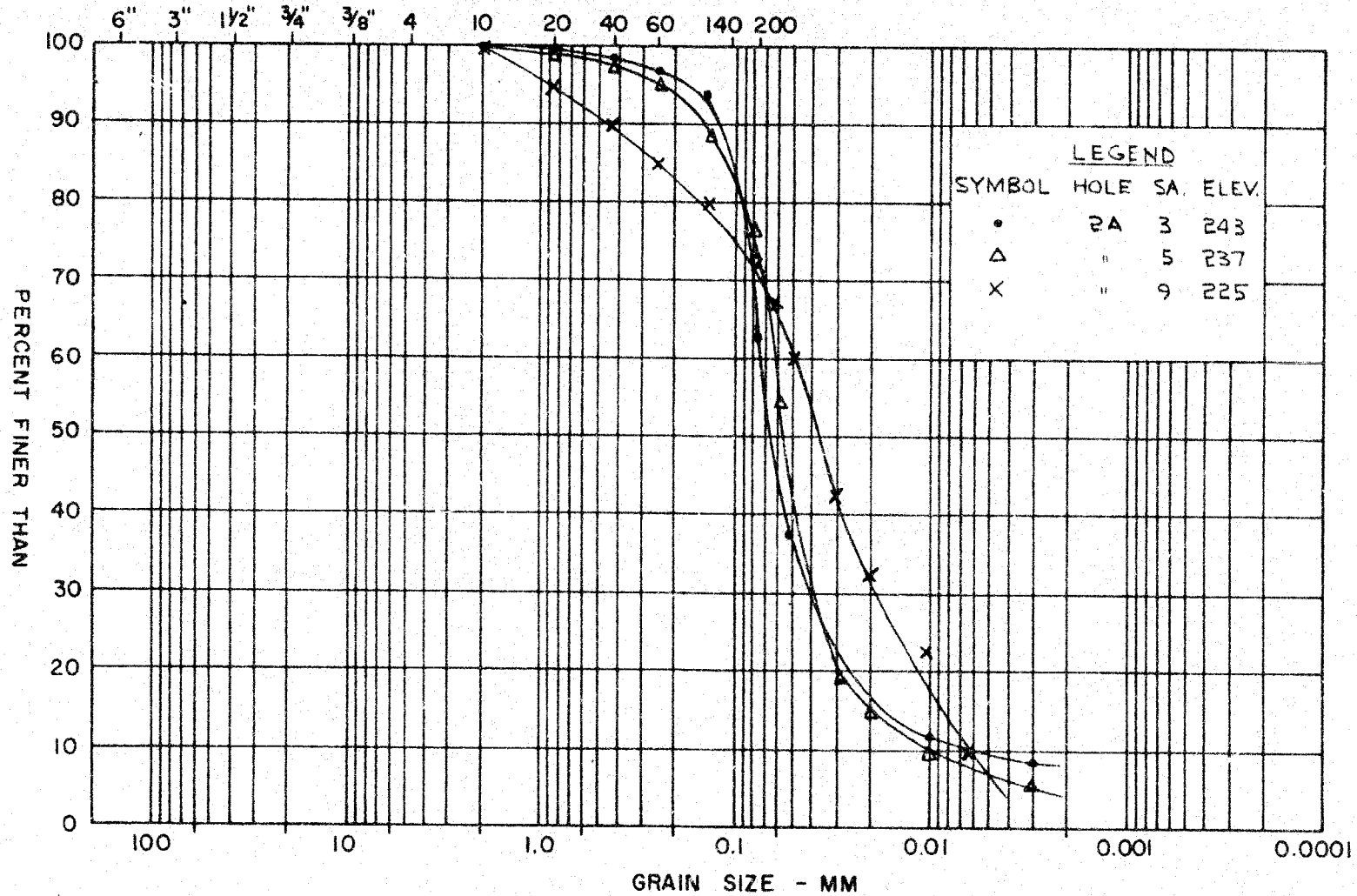
GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION  
ORGANIC SILT, THIN SAND LAYERS

FIGURE 2

M.I.V. GRAIN SIZE SCALE

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



GOLDER & ASSOCIATES

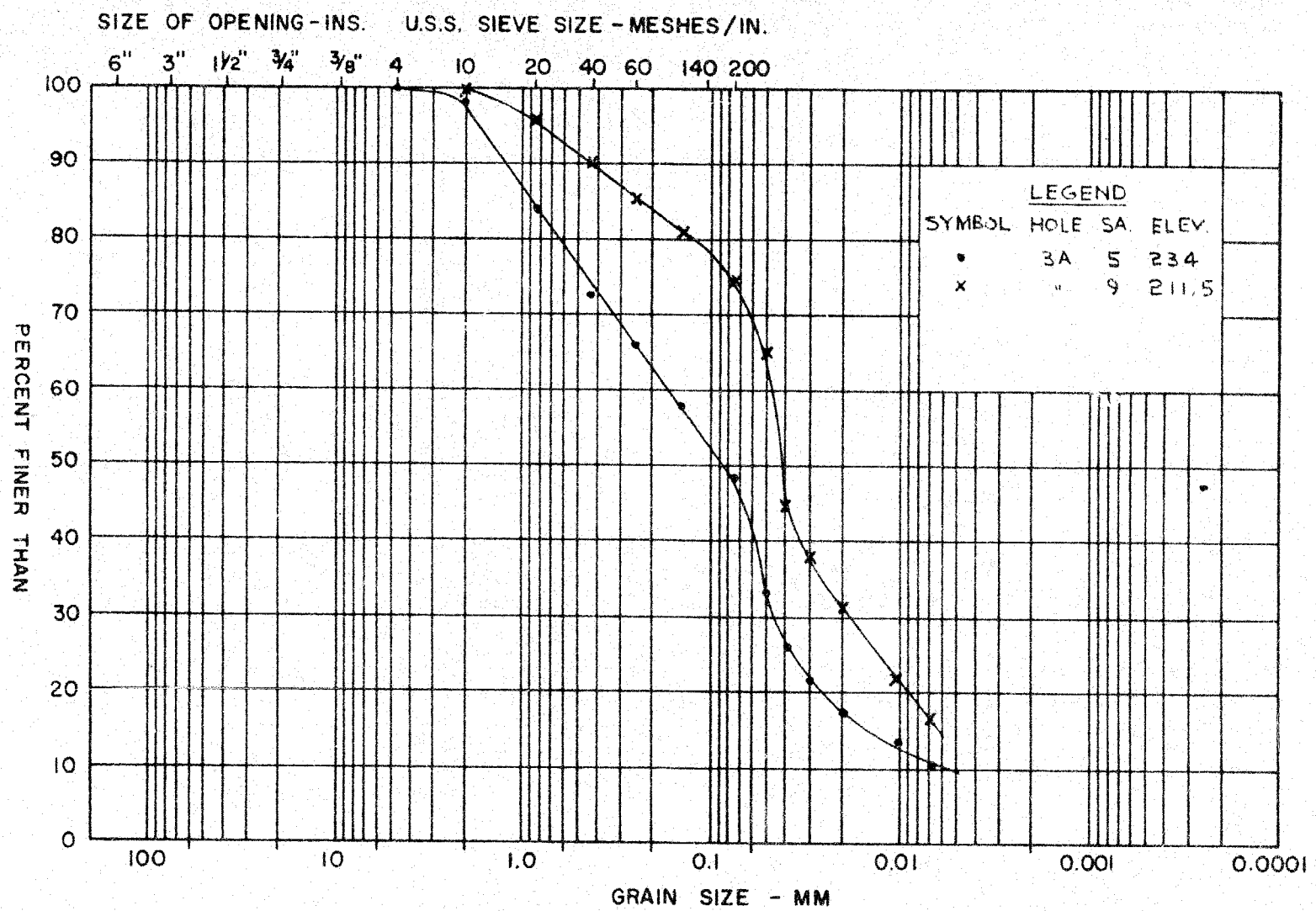
GRAIN SIZE DISTRIBUTION  
ORGANIC SANDY SILT

FIGURE 3

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

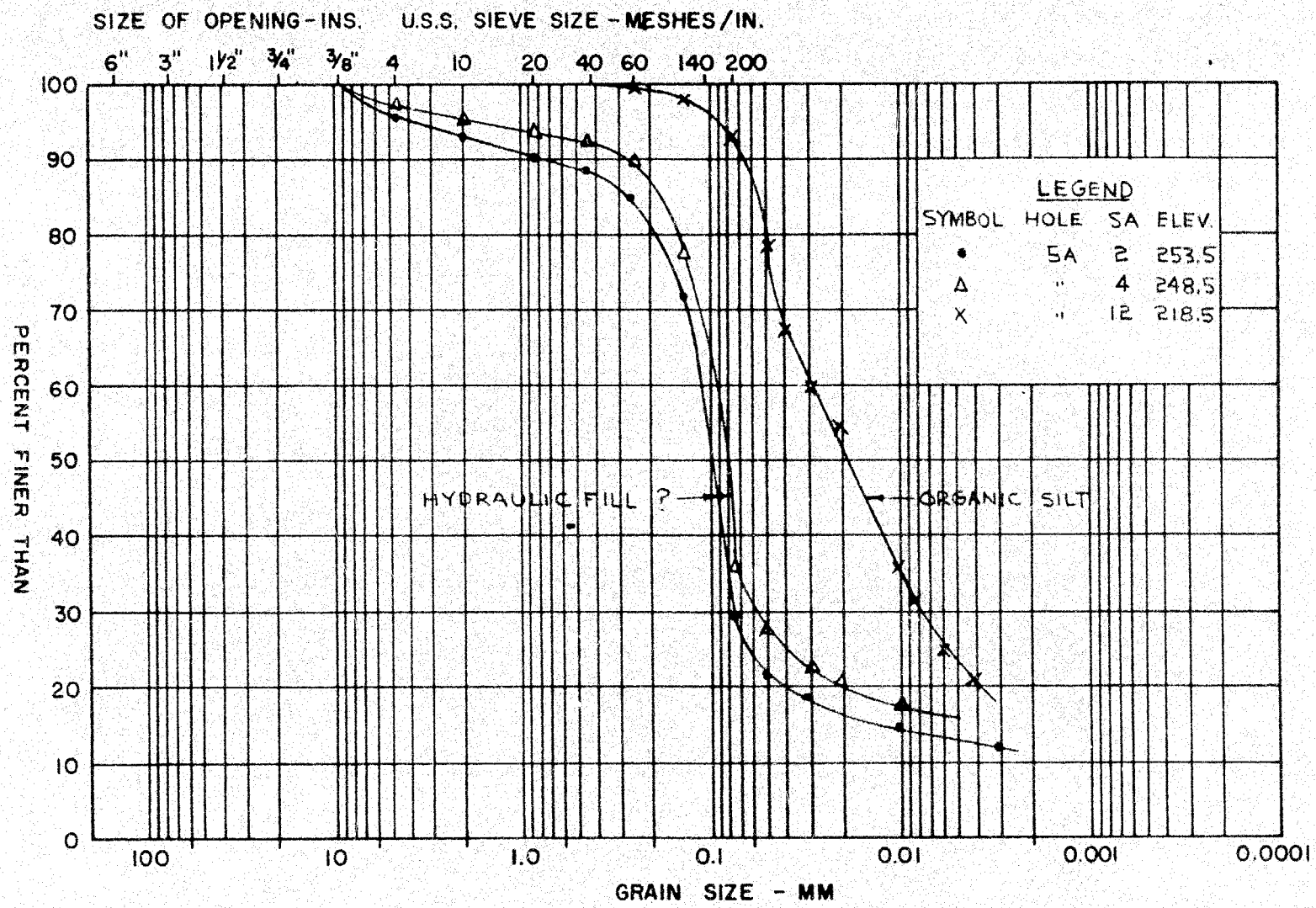


COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION  
ORGANIC SANDY SILT

FIGURE 4

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

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

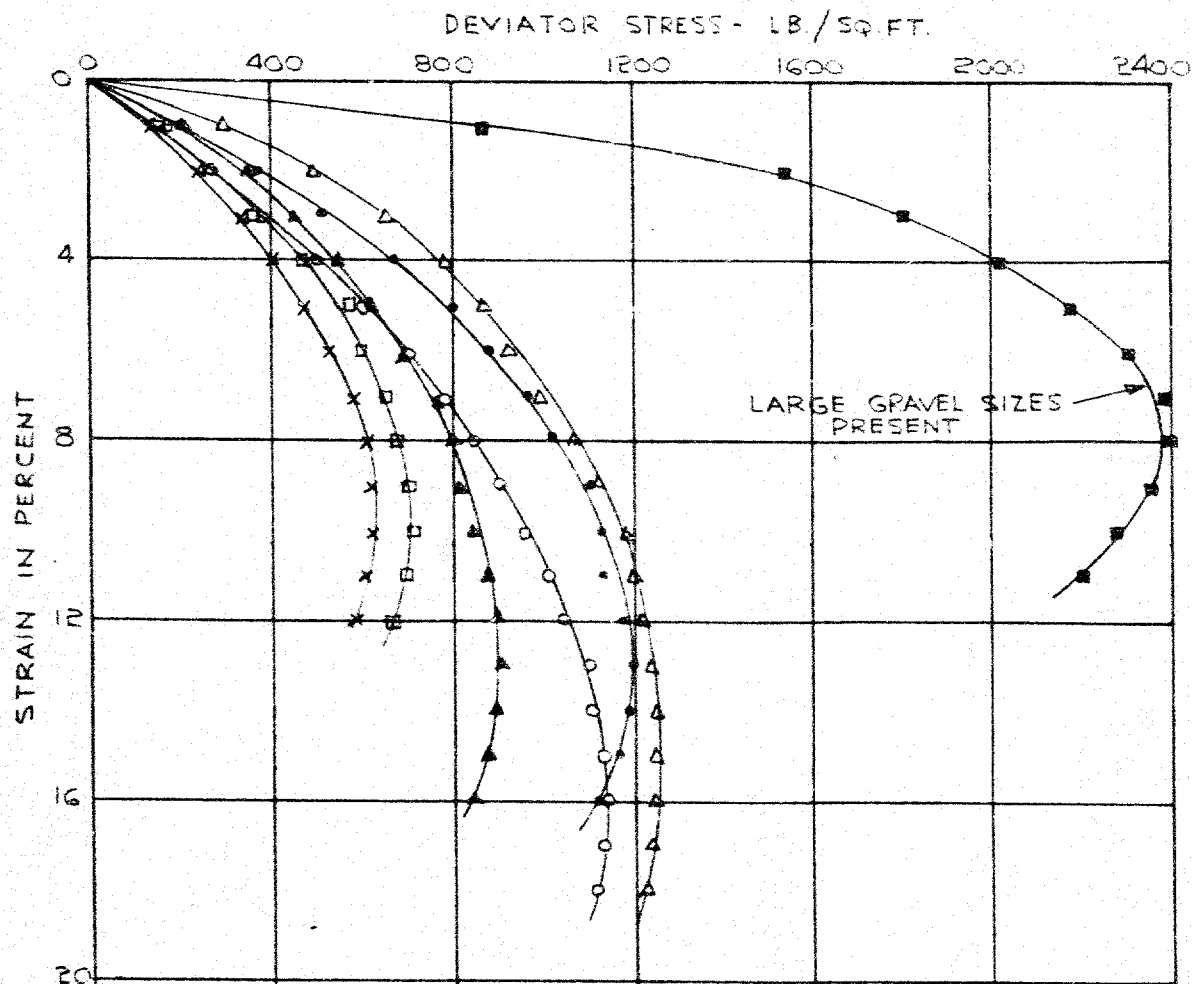
GRAIN SIZE DISTRIBUTION

FIGURE 5

# UNDRAINED TRIAXIAL COMPRESSION TESTS

## STRESS-STRAIN CURVES

FIGURE 6



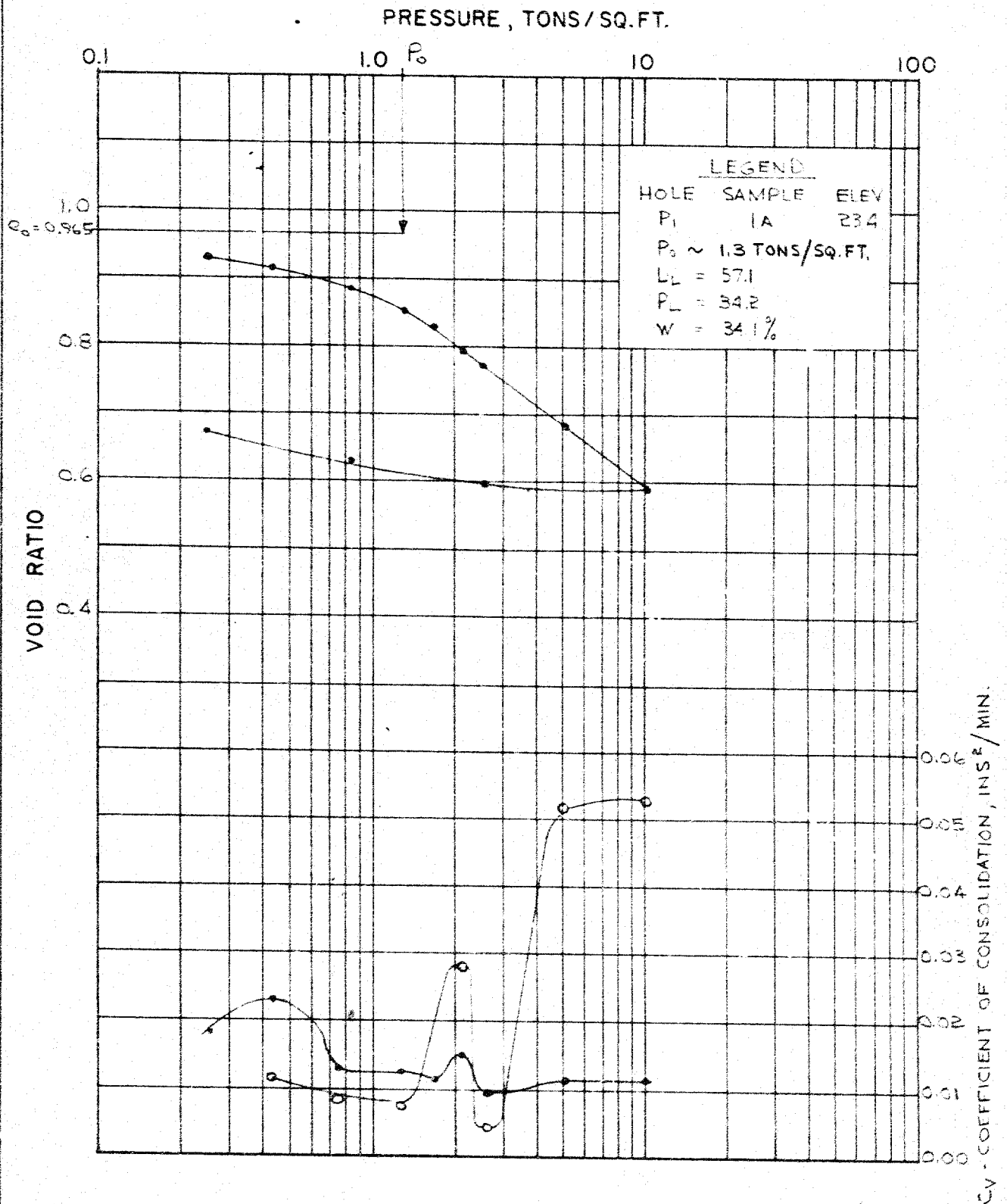
### LEGEND

SYMBOL	HOLE	SAMPLE	APPROX. ELEV.
x	104	1A	253
o	"	1B	252
•	"	2A	251
Δ	"	2B	250
□	"	3B	247
▲	"	3C	246
■	"	4	245



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 7

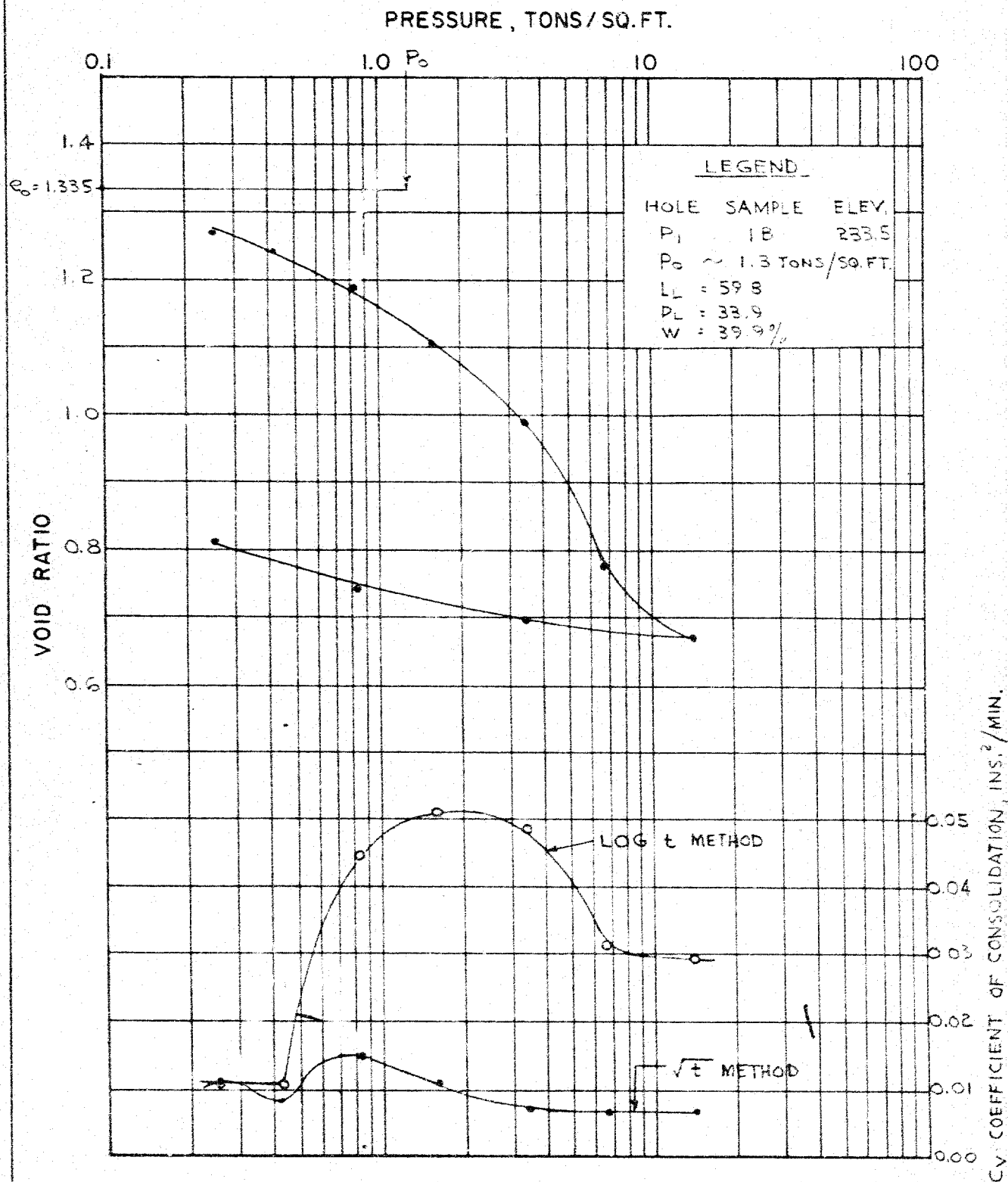


GOLDER & ASSOCIATES

PROJECT No. 6142

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

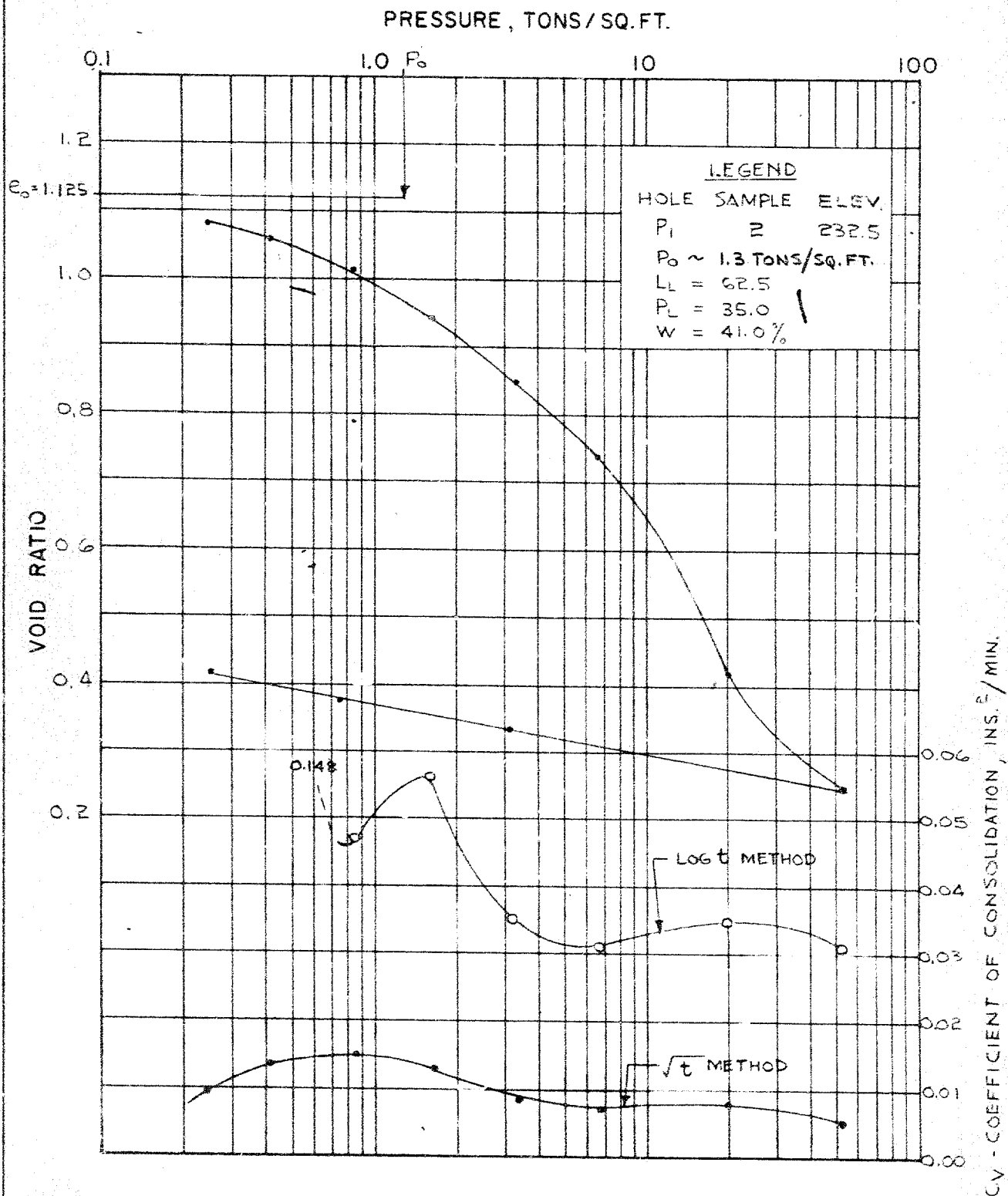
FIGURE 8



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

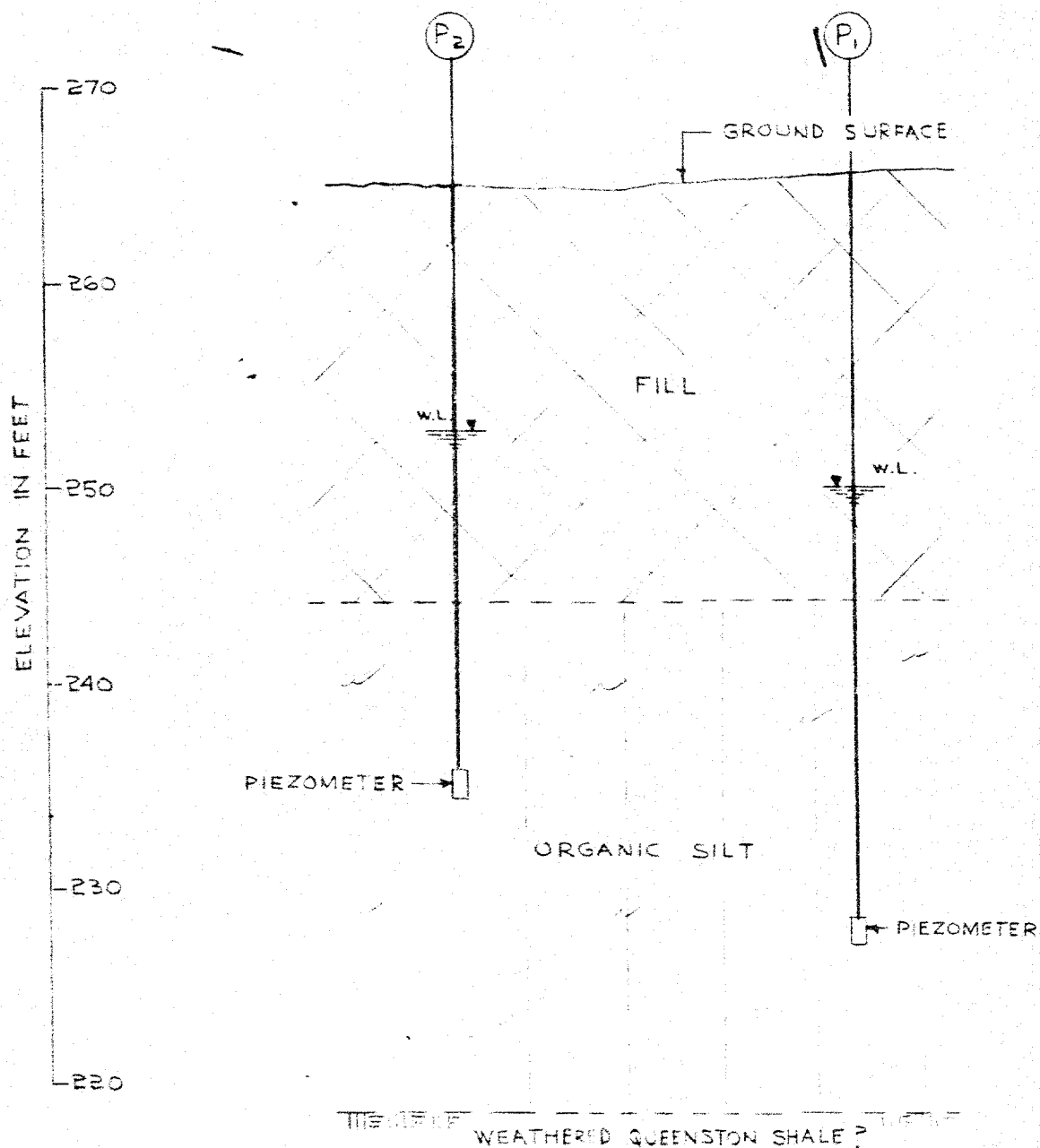
FIGURE 9



GOLDER & ASSOCIATES

# PIEZOMETER INSTALLATION

FIGURE 10



WATER LEVEL READINGS IN PIEZOMETERS TAKEN ~ 30 DAYS AFTER INSTALLATION.

GOLDER & ASSOCIATES