



MONTREAL OFFICE  
1980 SHERBROOKE ST. WEST  
MONTREAL 25

FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

8 SPADINA ROAD  
TORONTO 4

CABLE ADDRESS "FOUNDANENG" TORONTO

VANCOUVER OFFICE  
1425 WEST PENDER ST.  
VANCOUVER 5

February 23, 1960.

Department of Highways, Ontario,  
Parliament Buildings,  
Queen's Park,  
TORONTO, Ontario.

Attention: Mr. J. Walter, P. Eng.,  
Director of Planning and Design.

Dear Sirs:

HOMER BRIDGE  
OVER  
THE WELLAND CANAL


This letter accompanies Addendum No. 1 to the Report on the Homer Bridge and will serve to place on record certain changes in the design of the structure following further detailed study.

The final soils report by Geocon dated January 29, 1960 is contained in Appendix I of this report and replaces their Interim Report dated October 29, 1959 which was contained in the original issue.

Should you require any additional information regarding the design of the bridge, please do not hesitate to call.

Yours very truly,  
FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

JTG/sb  
1903-101

  
J. T. Gregg, P. Eng.  
SUPERVISING BRIDGE ENGINEER

**FENCO**



February 23, 1960

HOMER BRIDGE  
OVER  
THE WELLAND CANAL

ADDENDUM NO. 1

Since the original report was first issued on October 29, 1959, the analysis of the foundation conditions has been completed. The existing soil conditions have proven to be more favourable to spread footings than was hitherto considered and, in consequence, piles have been omitted at all pier locations, with the exception of Piers W5 to E11 inclusive, which remain piled.

Pile tests have been carried out at the site and have confirmed the preliminary design assumption that steel H-piles, weighing 73 lbs. per foot, will support a working load of 100 tons when driven to a resistance of approximately 20 to 30 blows for the last inch in the underlying till, using a hammer with a minimum rated energy of 24,375 ft. lbs. per blow.

Sand drains under the approach embankments are no longer being considered as it is felt that the desired degree of consolidation can be achieved by means of surcharging the embankments with about fifteen feet of fill in the area of the abutments. The surcharge material will be removed after a minimum period of one year and when the desired amount of consolidation has been observed. Settlement platforms and piezometers will be installed at the abutments to enable accurate recordings of the rate and extent of consolidation to be made.

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VANCOUVER OFFICE  
1425 WEST PENDER ST.  
VANCOUVER 5

October 29, 1959.

Department of Highways, Ontario,  
Parliament Buildings,  
Toronto, Ontario.

Attention Mr. J. Walter, P. Eng.,  
Director of Planning & Design.

Dear Sirs:

HOMER BRIDGE  
OVER  
THE WELLAND CANAL

In accordance with our agreement of March 18, 1959, we are pleased to submit our report dealing with the studies for a new high level bridge on the Queen Elizabeth Way over the Welland Canal.

The report outlines the studies undertaken with regard to location of the structure, the general conditions of the site, and the requirements of the structure, together with our recommendations. Also included is the report of soil investigations undertaken to date.

**FENCO**



Department of Highways, Ontario,  
October 29, 1959,  
Page 2.

The design which we recommend, as shown by the frontispiece and on drawing 1903-T-3, is described in the report, as are several of the alternative designs considered, together with estimates of cost.

The recommended design has been developed to give an economical and functional structure of pleasing appearance involving a minimum of ornamental treatment.

The estimate of cost is \$16,800,000 and includes stabilization of the canal banks, protection for the main piers against canal shipping, and the interchange ramps at the west end of the bridge.

We have been in contact, throughout our studies, with your various design branches and have greatly appreciated the excellent co-operation received from them.

Department of Highways, Ontario,  
October 29, 1959,  
Page 3.

We are proceeding with the preparation of detailed drawings and specifications and expect that these will be sufficiently advanced to allow tenders to be called for the first construction contract before the end of November, 1959.

Yours very truly,  
FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

R. W. Crudge, P. Eng.,  
VICE PRESIDENT.

1903-101

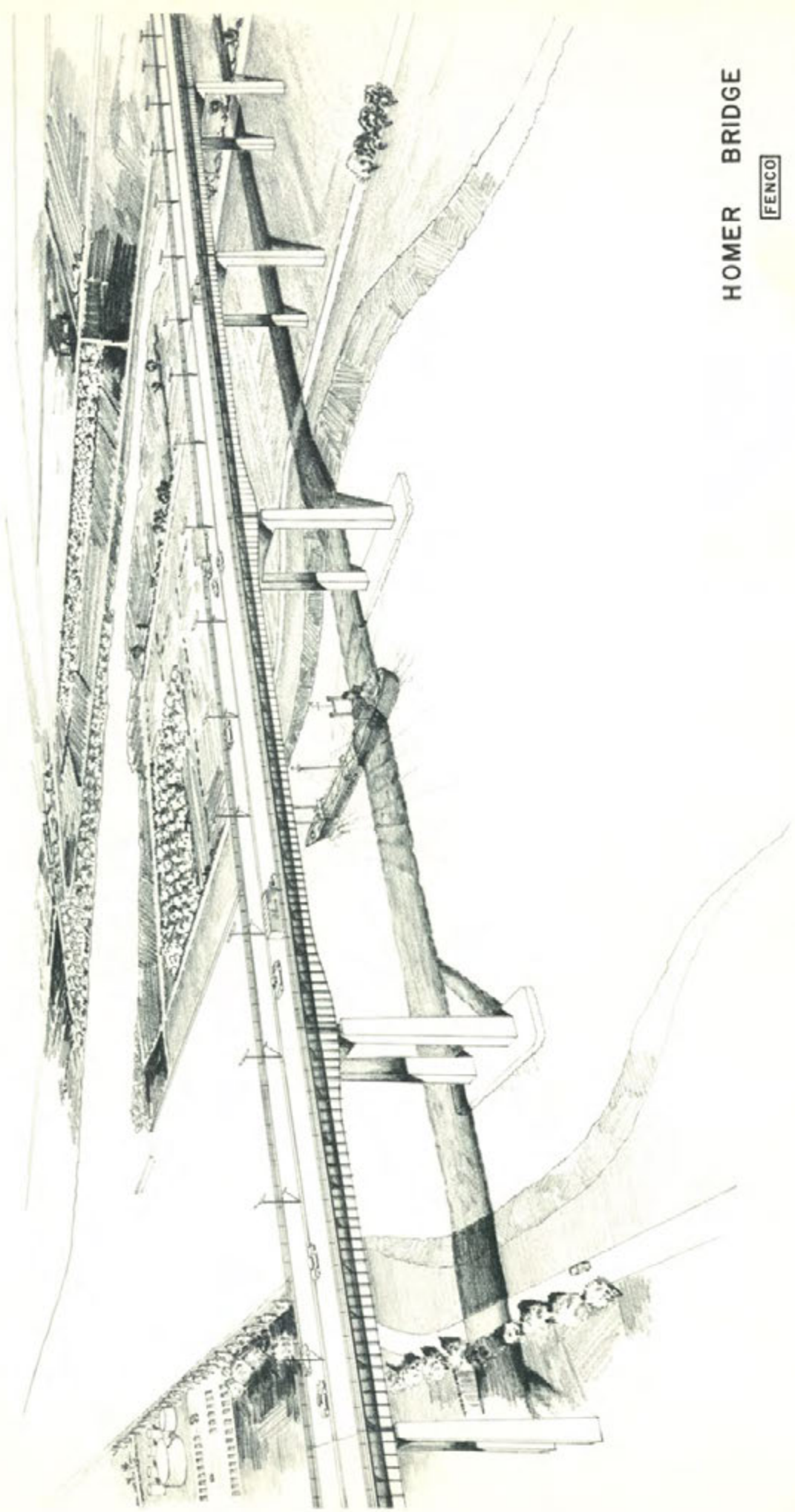
REPORT  
ON  
HOMER BRIDGE  
OVER  
THE WELLAND CANAL  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO

October, 1959

FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

**FENCO**





HOMER BRIDGE

FENCO

1959

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#### APPENDIX I - SOIL INVESTIGATION



## ACKNOWLEDGMENTS

We wish to express our appreciation for the co-operation of officials of the Department of Highways, Ontario, as well as for that received from the other agencies and bodies that have given freely of their time and effort.

Listed below are some of the organizations whose assistance we acknowledge with sincere thanks:

### DEPARTMENT OF HIGHWAYS, ONTARIO

Planning Branch  
Road Design Office  
Bridge Office  
Location Survey Branch

### DEPARTMENT OF TRANSPORT, OTTAWA

### ST. LAWRENCE SEAWAY AUTHORITY

## I. INTRODUCTION

By agreement dated March 18, 1959, the Department of Highways, Ontario, has retained Foundation of Canada Engineering Corporation Limited (Fenco) to act as their engineers for the design and supervision of construction for the Homer Bridge over the Welland Canal.

The terms of the agreement require that Fenco present the Department with a report of their recommendations for the design of the proposed structure. This document constitutes the required report.

Geocon Limited has been retained by Fenco to carry out the site investigation, and their preliminary report on soil conditions is contained in Appendix I of this report.

## II. SUMMARY

Having given full consideration to the site conditions, traffic and economic requirements, it is recommended that the new Homer Bridge over The Welland Canal be a high level, six lane structure as discussed in the following report. The estimated construction cost of such a bridge is approximately \$16,800,000.00.

It is proposed that the structure be composed of deck plate girders throughout, having main spans of deep web, box girder construction and approach spans of composite plate girder design. The supporting substructure is founded on spread footings wherever the site conditions permit. Elsewhere the foundations are supported on steel piles driven to positive bearing.

The recommended location of the canal crossing is some 450 feet north of the existing lift bridge. The future requirements of the St. Lawrence Seaway Authority have been established for a bridge at this location.

The approach embankments are limited to a height of approximately 25 feet. Consideration is being given to the use of surcharge and sand drains in an effort to accelerate consolidation of the subsoil supporting these embankments.



### III. LIST OF DRAWINGS

The following drawings accompany and form part of this report:-

Dwg. No.	1903-T-1	-	Key Plan
	1903-T-2	-	Site Plan
	1903-T-3	-	General Arrangement
	1903-T-4	-	Canal Spans - Proposed Scheme
	1903-T-5	-	Canal Spans - Alternative "A"
	1903-T-6	-	" " - Alternatives "B" & "C"
	1903-T-7	-	" " - Proposed Pier
	1903-T-8	-	" " - Pier - Alternative "A"
	1903-T-9	-	Proposed Approach Spans
	1903-T-10	-	Typical Deck Cross-Section
	1903-T-11	-	Typical Ramp Cross-Section
	1903-T-12	-	Pier Protection Works
	1903-T-13	-	Existing Bridge

#### IV. SITE & LOCATION

The Queen Elizabeth Way (Q. E. W.) crosses the Welland Canal at Homer, a hamlet on the east bank of the canal and close to the city of St. Catharines. A plan of the site is shown on drawing No. 1903-T-2.

At present the Q. E. W. crosses the canal on a rolling type, double leaf bascule bridge. A general arrangement of the existing span is shown on drawing No. 1903-T-13. Construction of this bridge was started in 1916 and not completed until 1927. Considerable difficulty was experienced during this period with embankment failures, and additional open end spans were added in an effort to reduce the surcharge on the banks.

This existing bridge was completed some twelve years before the opening of the Q. E. W. (1939) and was built to carry Highway 8 over the canal. Highway 8 is a two lane roadway which converges with the Q. E. W. at Homer.

Canal traffic has the right of way over road traffic and often vehicles are obliged to stop in order that the lift span may be raised to allow ships to pass. With the opening of the St. Lawrence Seaway, the frequency of these interruptions to highway traffic has been increased.

The narrow two lane bridge represents a serious restriction to the flow of traffic on the Q. E. W. and the priority given to the shipping aggravates this situation.

Preliminary soils information indicates that the material through which the canal is constructed is composed mainly of clay. This clay has a relatively good strength when exposed by excavation but deteriorates with age after exposure. An analysis of the existing slopes of the canal indicates that they possess a factor of safety of less than one under conditions of sudden draw down. At the close of each navigation season the canal is dewatered and at such times localized embankment sloughing may be observed. There is a record of numerous surface slides and several deep seated slope failures in the area of Homer.

The above factors and stability calculations show that a slope of 5 : 1 is the minimum required for long term stability of the canal banks.

Three bridge locations have been investigated as indicated on drawing No. 1903-T-2. Line B was untenable due to the proximity of the existing bridge and the possibility that excavations for the



main bridge piers would affect the stability of the existing structure. Line C, to the south of the existing bridge, was rejected due to circumstances as outlined below. The St. Lawrence Seaway Authority plan to widen the canal and construct a tie-up wharf between the present bridge and lock number 3 to the south of the bridge. It has not been decided on which side of the canal this wharf will be built. In order to provide for all possibilities, it would be necessary to build a long central span of approximately six hundred feet. This would add considerably to the cost of the bridge. The location also of Line C is such that it would be impossible to construct the bridge without first constructing a major detour for both the Q. E. W. and Highway 8 traffic on the west side of the canal.

The recommended location, Line D, to the north of the present bridge eliminates the necessity for major detours, and requires a shorter canal span than for Line C. In addition, the Seaway Authority have been able to establish their minimum clearance requirements for a crossing at this location. The one disadvantage of this alignment is the fairly extensive property damage at the east approach. However, this expense is more than offset by the reduced

cost of the structure and detours when compared with a bridge located south of the existing structure.

The Department of Highways is planning to build a direct connection between the Q. E. W. and the New York Thruway. Drawing No. 1903-T-1 shows that the connection will be made just east of Homer. We can expect that this will induce additional traffic on this stretch of the Q. E. W., which will then link the East-West Highway 401 to the New York Thruway.

## V. SPECIFIED REQUIREMENTS

The St. Lawrence Seaway Authority has specified that a fixed highway bridge across the Welland Canal at a location approximately 450 feet north of the existing Homer Bridge shall provide the following minimum clearances:-

### Vertical Clearance

123 feet above elevation 335.5 (Welland Ship Canal Datum) for a width of 250 feet normal to and symmetrical about the centre line of the canal.

### Horizontal Clearances

250 feet normal to the canal at a height of 123 feet above elevation 335.5 (Welland Ship Canal Datum).

300 feet normal to the canal, face to face of pier footings, less fendering.

200 feet normal to the canal during construction.

In all cases the minimum horizontal clearance shall be symmetrical about the centre line of the navigation channel.

Electrical navigation aids and warning devices shall be included for both marine and air traffic, subject to the approval of the Department of Transport.

## VI. DESIGN CRITERIA

The following design criteria have been agreed to by the Department of Highways, Ontario:-

### Geometric Requirements

- a) Maximum Grade - 3 percent
- b) Maximum Curvature on Structure - 2 degrees
- c) Maximum Superelevation on Structure - 3 percent
- d) Minimum Horizontal Sight Distance - 1000 feet

### Specifications

- a) Grading and Paving

"General Specifications - 1956" and Highway Standards of the Department of Highways, Ontario."

- b) Structure

"Specifications for Structures, D.H.O. Form No. 9, revised March 1957" and "C.S.A. Specification for Steel Highway Bridges S6-1952."

H25-S20 Live Loading and Permissible Stresses shall be applied in accordance with "A. A. S. H. O. Standard Specifications for Highway Bridges, 1957".



## VII. NUMBER OF TRAFFIC LANES

The average summer weekend daily traffic using the Q. E. W. at Homer is approximately 24,000 vehicles, and a maximum count of 30,000 vehicles per day has been recorded. Assigning a reasonable growth factor to the former figure, this could amount to 50,000 v. p. d. in 20 years.

Traffic studies indicate that approximately 50 per cent of the daily traffic is in each direction. Assuming that the peak hourly movement is approximately 10 per cent of the total daily figure, this indicates a flow of 2,500 v. p. h. in one direction. It is reasonable to assume that about ten per cent of the traffic will be commercial vehicles, which will be reduced to a speed of well under 30 m. p. h. due to the long three per cent approach grades. Thus the capacity of one lane in each direction will be reduced effectively and it will be necessary to provide two lanes to cater to the remaining 2,250 v. p. h. The practical capacity of each lane may be considered to be approximately 1,500 v. p. h. at speeds of 35-40 m. p. h. and this indicates the necessity for three lanes in each direction.

If, for any reason, it is found necessary to close the existing bridge after the new structure has been completed, Highway 8 traffic, at present using the low level crossing, will be rerouted over the new high level bridge.

It is anticipated that the future direct connection between the Q. E. W. and the New York Thruway will add to the traffic using the Q. E. W. at Homer.

Cost estimates have been prepared for building the bridge as:

- (1) six lanes initially
- (2) four lanes without provision for expansion to six
- (3) four lanes with provision for widening to six

The summary of costs for each scheme is given in Chapter IX of this report. Of these, Scheme 3 appears, at this time, to be impractical unless it could be arranged for the approaches to be laid out as three separate two-lane facilities, with the central two lanes being reversible to cope with peak flow in either direction. However, this is not feasible due to the layout of the approach highways. With the present highway arrangement it is necessary to build the complete substructure in the first instance, any future widening being confined to widening the superstructure. It is also necessary to

build the main central spans as six lanes initially, since it is uneconomical to design these to accomodate the widening from four to six lanes. In addition, it would be necessary to reduce the bridge to one lane traffic in each direction in order to effect the widening. The cost of the widening would exceed the initial saving in construction costs.

Taking the above factors into consideration, we believe that in the interests of economics and sound engineering, the structure should be constructed to provide for six lanes from the very outset.

## VIII. MAIN STRUCTURE

### a) General Arrangement

The overall length of the recommended structure is 7,082'-0" centre to centre of abutment bearings, and is of deck girder construction throughout. The general arrangement of the bridge is shown on drawing No. 1903-T-3.

The structure divides naturally into three distinct sections, namely, the east approach spans, the main spans, and the west approach spans.

The main section, 2,097'-6" in length, is composed of three continuous canal spans which are flanked on each side by three 200 ft. spans. The three west flanking spans are designed as a continuous structure, whereas the east spans are simply supported, as the bridge is on a two degree horizontal curve at this point.

The east approach section is comprised of thirteen 150 ft. simple girder spans of composite construction having a total length of 1,970'-0" from the centre of the abutment bearing to the centre of pier E5. It would possibly be more economical to reduce several of the lower spans from 150 feet to 125 feet. However, in doing so, it would be necessary to revise the location of Highway 8 where it



passes under the bridge. This would introduce a less desirable alignment for this highway, and any revision of alignment to Highway 8 would involve additional property purchase. It is not believed that the structural savings would be sufficient to offset the additional property costs.

The west approach is composed of fifteen 125 ft. spans and eleven 100 ft. spans giving a total length of 3,014'-6" from the centre line of pier W5 to the centre of bearing at the west abutment. As is the case with the east approach spans, these are also of simple composite plate girder construction.

The soil conditions dictate that the footings of all piers between pier W5 and E10, inclusive, be supported on piles driven to positive bearing. It would be possible to carry all the remaining piers on spread footings founded in the desiccated clay crust described in the soil report. On the east approach the grade separation between the Q. E. W. and Highway 8 requires that Highway 8 be constructed directly above several of the main pier footings. Because of the expected settlements associated with spread footings, it is advisable to carry these footings on piles. This being the case, it was decided to carry all the footings for the east approach on piles,

as it was deemed inadvisable to have a short intermediate section of this approach on spread footings. On the other hand, footings for the west approach section from pier W6 to the abutment are founded on spread footings with the exception of piers W16 to W20, inclusive. These latter piers are on piles so as to accommodate the access ramp from Highway 8 and the service road, which cross under the bridge and over the footings of these piers. Pile loading tests will be carried out shortly in order to verify the design assumptions.

b) Main Spans

The central three spans are comprised of three continuous spans of 259'-9", 365'-6", and 259'-9", giving a total length of 885 feet. The deck is supported by two deck type box girders spaced 52 feet apart. These vary in depth - at the centre of the main span the depth is 12'-3" back to back of angles varying to 20'-6" over the central piers and to 11'-6" at the end supports.

A general arrangement of the central three spans is shown on drawing No. 1903-T-4. The "fish belly" haunches which can be seen on the elevation are somewhat shorter than the optimum required for structural reasons, but the clearances

requested by the Seaway Authority restrict the maximum length of the haunches. Also, the end spans are somewhat deeper than the minimum required for structural considerations, but it was felt desirable to increase the depth of the end spans beyond the minimum requirements for two reasons: (1) It greatly increases the stiffness of the end spans, thereby reducing the tendency of the three spans to deflect and vibrate, and (2) It also conforms to the depth of the 200 foot approach spans.

The webs of the box girders have been designed in accordance with the theory of elastic stability applied to structural design as postulated by Moisseiff and Lienhard in their paper No. 2120 delivered to the American Society of Civil Engineers, and published in that Society's Proceedings Volume 106 (1941). In addition to the requirements of elastic stability, the webs have been designed to resist the normal shearing stresses and also the radial stresses produced by the curvature of the haunches. It is proposed that high strength steel be employed in the construction of the box girders over the central span and to the point of contraflexure in each end span. This material would be a low alloy steel conforming with A. S. T. M. - A242 specification for riveted



construction. The remaining sections of the two approach spans would be constructed in normal grade structural steel (A7) since the proportioning of these sections is dependent upon conditions other than the stresses. The floor beams and their cantilever sections would be constructed of normal grade steel and be of all shop welded construction. The simply supported stringers are composed of wide flange beams. At the location of each floor beam and at intermediate locations, there is a complete diaphragm in the main box girder which renders the sections comparatively stiff in torsion when compared with a conventional single web plate girder or a truss. Alternative designs for the central three span section are shown on drawings Nos. 1903-T-5 and 1903-T-6. Of these alternatives, the most competitive is the four plate girder design shown on drawing No. 1903-T-5. On a straight comparison with the box girder recommended, the plate girder would appear to be less expensive. However, when we include the cost of substructure, the box girder is the more economical overall scheme. In addition, the pure plate girder design has several distinct structural disadvantages. To begin with, each half of the bridge will act



independently of the other half and under live load there could be a differential deflection of 5" between each half of the bridge. To eliminate this differential deflection, each fourth floor beam would be made continuous between each half of the bridge. Another disadvantage is that it is more difficult to erect the light deep plate girder sections than the stiff box girder sections. It has been found in practice with similar type bridges that it is more difficult to control the dead load deflections of simple plate girders than the deflections of box girder sections, which have a greater inherent rigidity.

Of the other alternatives shown on drawing No. 1903-T-6, the cheaper of these is the tied arch design with simple plate girder approach spans. The St. Lawrence Seaway Authority has requested that the canal be kept open for navigation for the entire period of construction. This necessitates scheduling the erection of the tied arch to coincide with the short winter period when the canal is normally closed to shipping. This may place a severe restriction on the steel contractor, and since the cost would appear to be in excess of

either of the plate girder alternates, we do not feel it would be worth while to give further consideration to this scheme. In addition, as can be seen from the cross section on drawing No. 1903-T-6, it is necessary to construct this span in two twin arches, which produces all the disadvantages of a double through type structure.

The remaining alternative shown on drawing No. 1903-T-6 is a three span continuous truss, the central span being arched to give the required navigation clearances. This span has the disadvantage of being a through type structure, and being of truss construction, it does not have the clean lines of either of the girder alternates. It is somewhat more expensive than any of the other schemes.

The three flanking spans on each side of the canal spans form a transition between these larger spans and the shorter approach spans. They have the same structural arrangement as the canal spans and together they form a central unit, 2,097'-6" long, composed entirely of box girders. It is possible that it would be slightly more economical to use four longitudinal plate girders for this section, but it is felt that the appearance would not be in

accord with the canal spans. This latter system would introduce another change in cross-section and necessitate another transition pier and additional substructure costs.

c) Approach Spans

The approach spans are made up of simply supported all welded plate girder spans which vary in depth from 9 feet for the 150 foot spans to approximately 6'-3" for the 100 foot spans. It is proposed that stud type shear connectors be used to induce composite action. The strength of the deck concrete is 3750 p. s. i. giving a modular ratio of  $N = 8$ .

All steel in these girders 1" thick or over, shall conform to ASTM A373 steel, otherwise normal grade A7 structural steel shall be used. All field connections shall be riveted or bolted. A typical cross-section through the approach spans is shown on drawing No. 1903-T-9.

In order to accomodate possible settlement of piers founded on spread footings, it is proposed to provide jacking facilities on the superstructure at these locations. The minimum practical span length is considered to be 100 feet for piers founded on spread footings. For spans less than 100 feet there is a danger



of increased loading on the compressible clay layers due to the proximity of spread footings. The resulting overlap of the pressure bulbs in this material will lead to excessive settlement.

In the course of the investigation consideration was given to the use of pre-stressed concrete girders for the approach spans. These compare favourably with the 100 foot and 125 foot composite steel spans. However, the extra weight of the concrete girders is such that it would be necessary to provide larger footings in order to maintain the permissible spread load. When this is taken into account it is found that steel is the more economical medium. Apart from cost considerations, the whole nature of the underlying soils dictates the use of the lightest possible superstructure, as any increase in the area of the spread footings, even with the same applied unit load, will result in greater settlement of the piers.

d) Main Piers and Pier Protection

A sketch of the main piers W1 and E1 is shown on drawing No. 1903-T-7. The main piers take the form of a simple, two shafted bent. This is supported on a cellular type footing, which in turn, is supported on steel H piles driven to positive bearing.



The shafts of the pier are so positioned that they will line up with the shafts of the approach span piers. The footings of the main canal piers are encased in a sheet pile wall and are protected by rock filled timber cribs located as indicated on drawing No. 1903-T-12. In addition, a fendering system is provided on the canal face of each pier. The main piers will have sufficient strength to resist a glance from a vessel of 35,000 ton total displacement travelling at 6 knots through the canal. Past experience on the Welland Canal indicates that the possibility of such a vessel running directly into the ends of the piers at this speed is unlikely, but the timber cribs have been so positioned and designed that they will dissipate most of the ship's energy before the ship can strike the pier.

All remaining piers under the main spans are carried on solid concrete footings supported on H-piles. These piers utilize the same type of bent as E1 and W1.

e) Approach Piers

A typical approach pier is shown on drawing No. 1903-T-9. In accordance with the results of our site investigation, the maximum applied dead load for spread footings shall be limited

to 1000 pounds per square foot and the maximum load shall not exceed 2250 pounds per square foot. From pier W5 to the east abutment and from pier W16 to W20, all foundations shall be piled footings supported by steel H piles driven to required bearing.

f) Transition Piers

Piers E5 and W5 are located at the points where the box girders end and the composite approach spans begin. They accommodate the resulting change in the cross-section of the bridge. It is recommended that these piers be extended and constructed so as to provide a smooth transition from the approach spans to the main spans.

g) Ashland Avenue Ramps

A typical cross-section through a ramp is shown on drawing No. 1903-T-11.

The two ramps total approximately 700 feet in length and are composed of a series of simply supported spans on "hammer head" piers. The beams are of composite construction and are built up, welded sections. The piers are founded on spread footings.

The ramps are structurally separate from the main bridge.

h) Approach Fill & Abutments

It is physically possible to construct the approach embankment to a height of approximately 35 feet and a side slope of 2 : 1. Beyond this height there is a danger of a general failure in the underlying clay. The embankment at the east approach is restricted by the relocation of Highway 8 to a height of about 25 feet. Every reasonable effort should be made to hasten the expected settlement. This may be done by means of a temporary 10 foot surcharge, possibly aided by sand drains and a granular blanket. Investigations are currently in hand to determine the most appropriate and economical method of construction.

The approach fill at the west approach is also limited to a maximum height of approximately 25 feet, due to Ashland Avenue. This should receive similar treatment to the east approach fill.

It is desirable that both approach fills be constructed as soon as possible and well in advance of the abutments and the neighbouring piers. This would prevent excessive foundation settlements due to the consolidating effects of approach fills.



As mentioned in the accompanying soil report, an embankment of approximately 25 feet can be expected to settle about 20 inches over a period of approximately 40 to 50 years, unless artificial means are employed to accelerate the consolidation of the embankment and the underlying soil.

i) Deck Details

A typical section through the deck is shown on drawing No. 1903-T-10. The deck is divided by a six foot median and provision is made for three twelve ft. traffic lanes on each side. Two ft. wide paved shoulders are provided at the outside curbs and on each side of the median. Two ft. escape curbs are also incorporated. It would be possible to provide four temporary ten ft. lanes on either side of the bridge, as the clear distance from curb to curb is 40 feet.

The concrete deck slab is 7 inches thick throughout and supports a 3 inch wearing surface of asphalt. The concrete has a minimum strength of 3,750 p. s. i. at 28 days.

An aluminum hand rail will be provided, similar to Alcan Panel Type 336V. This has a height of 36 inches and incorporates three horizontal rails.



The bridge roadway is illuminated to 1.2 to 1.4 foot-candle level by the use of fluorescent luminaires. These are mounted in pairs and supported by aluminum alloy brackets on aluminum alloy poles installed in a single row in the centre of the median, as indicated on the drawing. The poles are spaced at intervals of approximately 100 feet. The mounting height for the luminaires is 30 feet above the road surface to the centre of the light source. The luminaires are connected in multiple on a 120/240 volt single phase secondary system composed of two sections separated at the mid point of the structure. The deck lighting will be controlled by an automatic switching device.

j) Slope Stabilization at Canal

Investigation has shown that it is necessary to trim the canal banks to a slope of 5 : 1 in order to obtain a factor of safety of 1.5. This necessitates excavation in the vicinity of the main canal piers and requires a relocation of the service road and power line belonging to the St. Lawrence Seaway Authority.

In addition, the exposed faces of the slope, up to high water level, are to be covered with a two foot graded granular blanket which is protected from erosion by hand laid rip rap. Exposed slopes above H. W. L. are to be protected by sodding.

## IX. ESTIMATE OF CONSTRUCTION COSTS

a) Six Lane Bridge

Slope Trimming, Pier Protection & Canal Works	-	\$ 750,000
Foundations	-	6,800,000
Superstructure Steelwork	-	7,700,000
Deck, Lighting, Handrail, etc.	-	1,550,000
Total	-	<u>\$16,800,000</u>

b) Four Lane Bridge

Slope Trimming, Pier Protection & Canal Works	-	\$ 750,000
Foundations	-	5,350,000
Superstructure Steelwork	-	5,600,000
Deck, Lighting, Handrail, etc.	-	1,250,000
Total	-	<u>\$12,950,000</u>

c) Four Lanes with Provision for Widening to Six

Total - \$15,550,000

If the bridge were to be built with four lanes initially and provision for future widening to six, the additional lanes must be added in the centre between the existing lanes for the following reasons:-

- A. The ramps at Ashland Avenue are on the outside of the bridge, and the highway approaches to the bridge will be built for six lanes initially.
- B. The piers for the West Approach Spans must be built to accomodate the final superstructure because of foundation conditions in this area. For aesthetic reasons, the piers on the East Approach Spans should be of a similar type. Hence, the complete approach substructure should be built initially for a six lane bridge.

If the bridge is to be designed to provide an ultimate capacity of six lanes, any economical solution for the main spans must accomodate six lanes from the start.

From the above it can be seen that the only saving would be in the structural steel, deck concrete, and asphalt for the approach spans. This has proved to be a relatively small percentage of the total cost of a six lane bridge, and the amount of saving would certainly be exceeded by the cost of adding two lanes in the future.

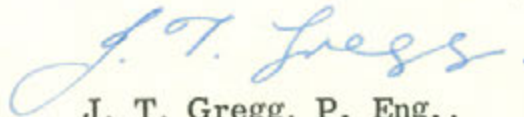
It should be noted that all the costs quoted above include a ten per cent contingency item.

Estimates have been prepared for the alternative designs discussed previously, and in each case these exceed the estimated cost for the proposed design.

X. PERSONNEL

This report has been written by J. T. Gregg,  
P. Eng., and reviewed by R. W. Crudge, P. Eng.

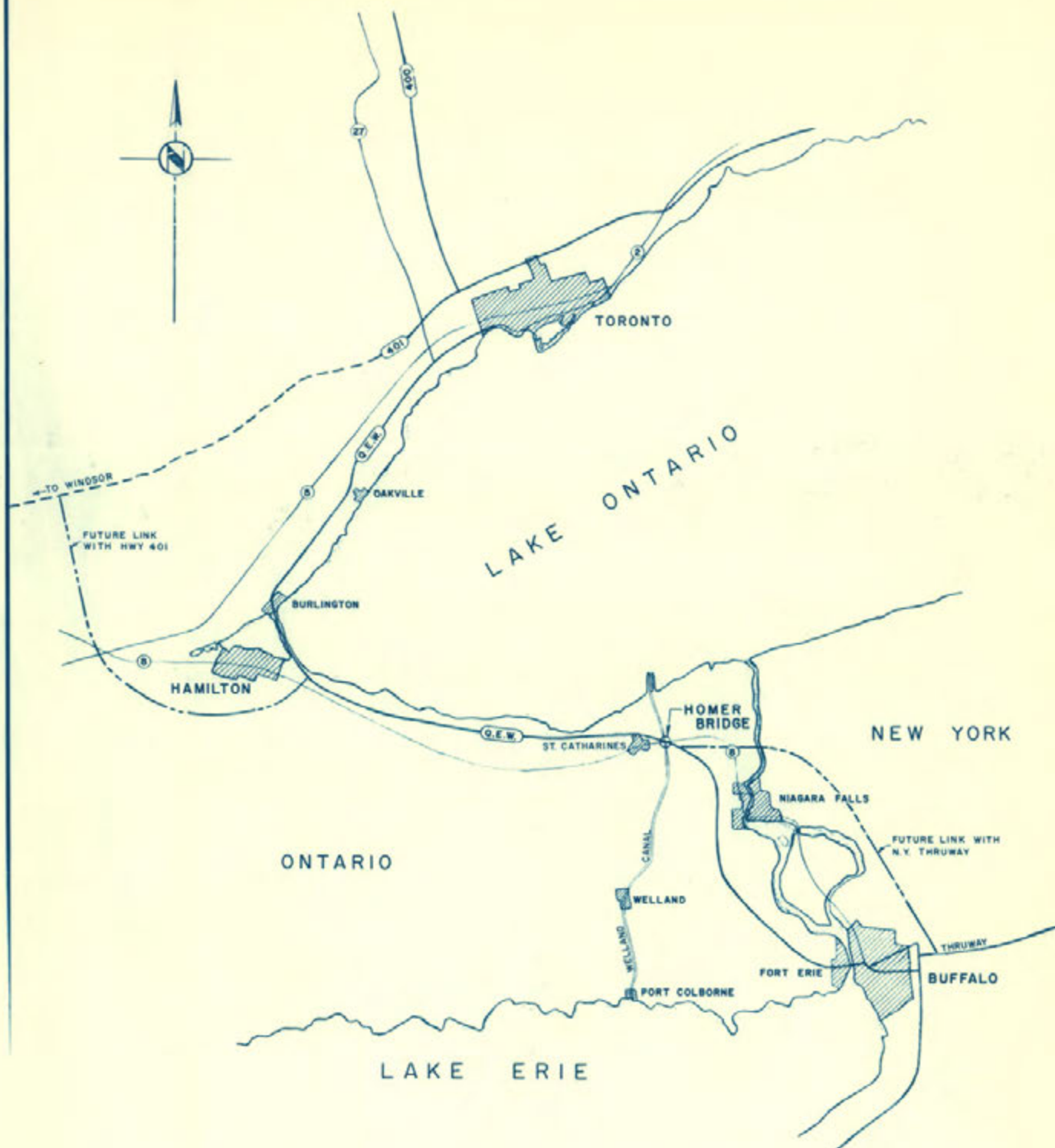
Recommendations for the design of the bridge  
have been reviewed and approved by Dr. D. B. Steinman  
and Dr. R. M. Boynton.



J. T. Gregg, P. Eng.,  
Supervising Bridge Engineer,

FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED





DEPARTMENT OF HIGHWAYS  
ONTARIO

HOMER BRIDGE  
OVER  
THE WELLAND CANAL

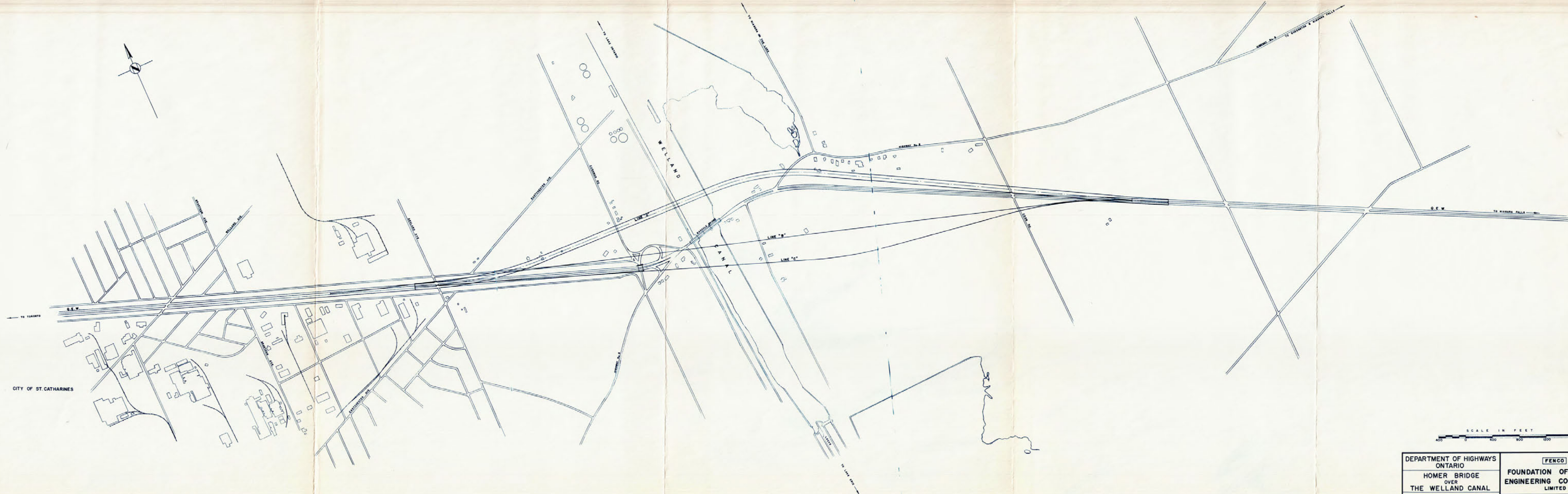
KEY PLAN

FENCO

FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED

No. 1903-T-1

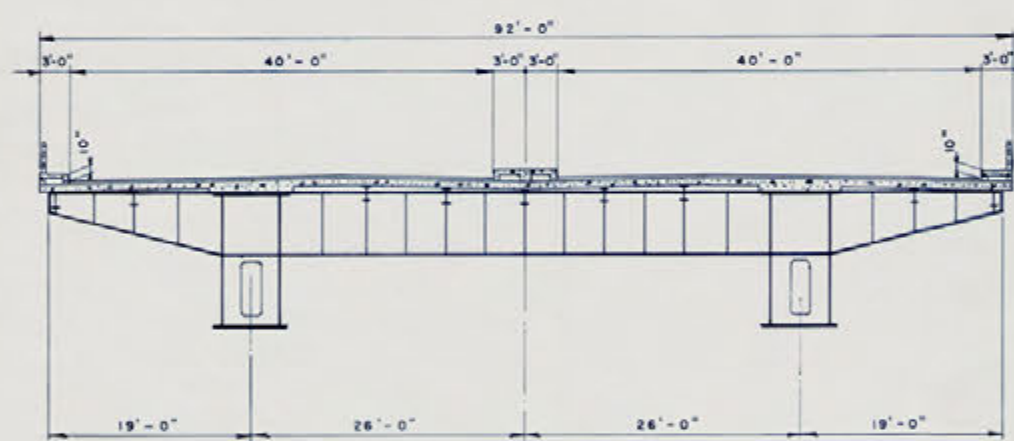
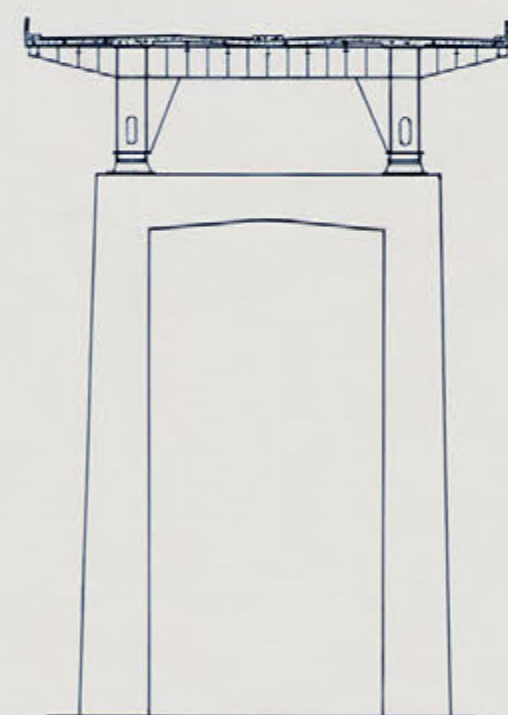
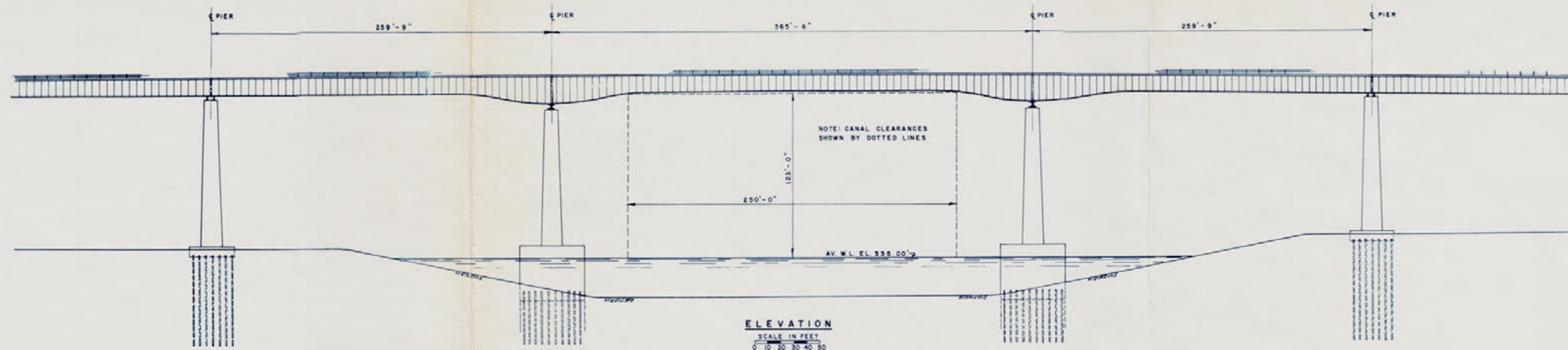


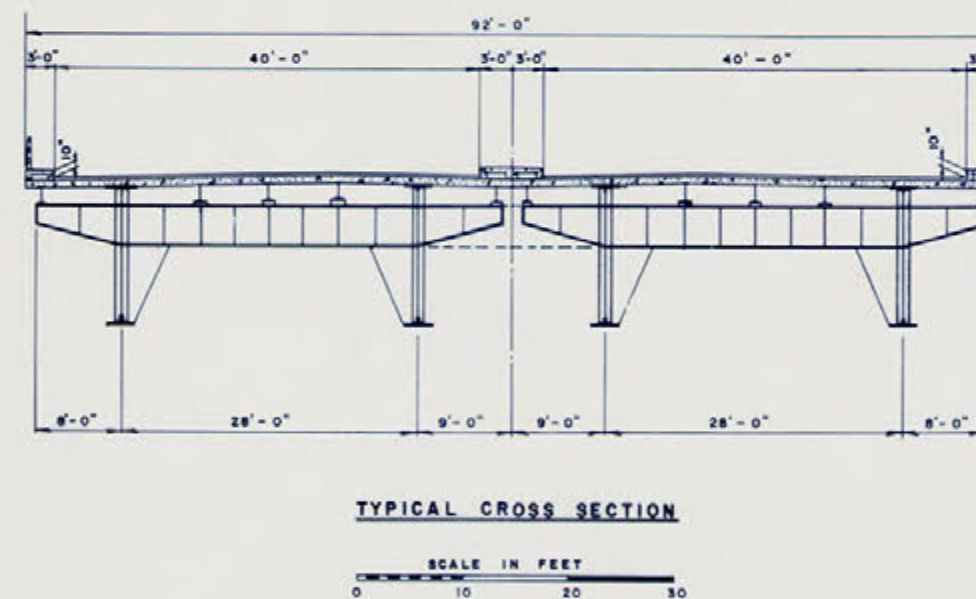
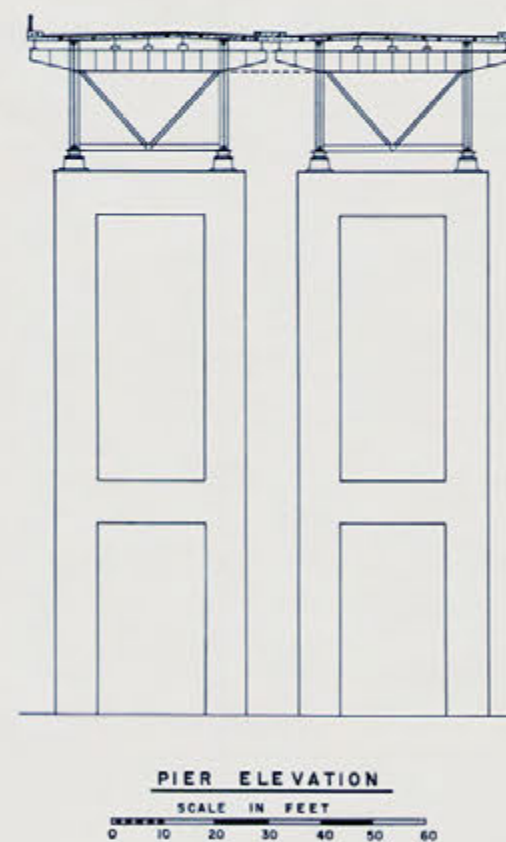
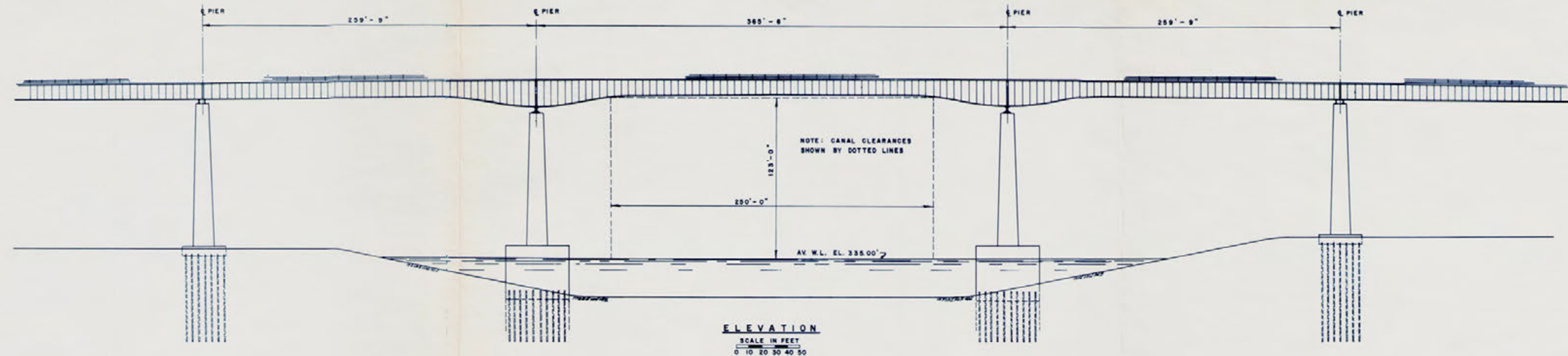


DEPARTMENT OF HIGHWAYS  
ONTARIO  
HOMER BRIDGE  
OVER  
THE WELLAND CANAL  
SITE PLAN

FENCO  
FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED  
No. 1903-T-2







DEPARTMENT OF HIGHWAYS  
ONTARIO

HOMER BRIDGE  
OVER  
THE WELLAND CANAL

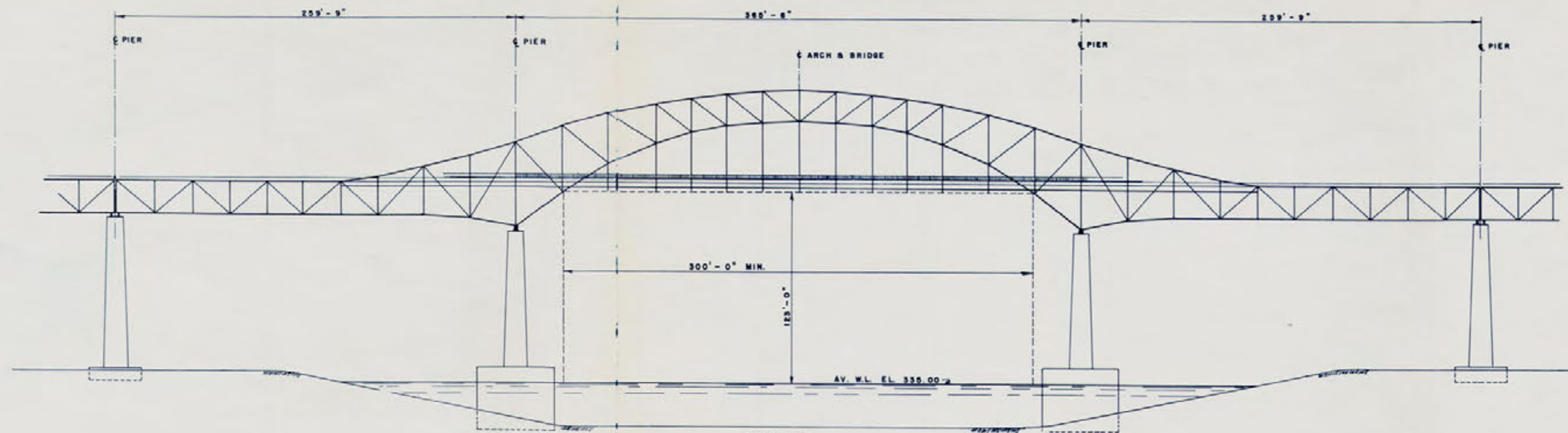
CANAL SPANS - ALTERNATIVE "A"

FENCO

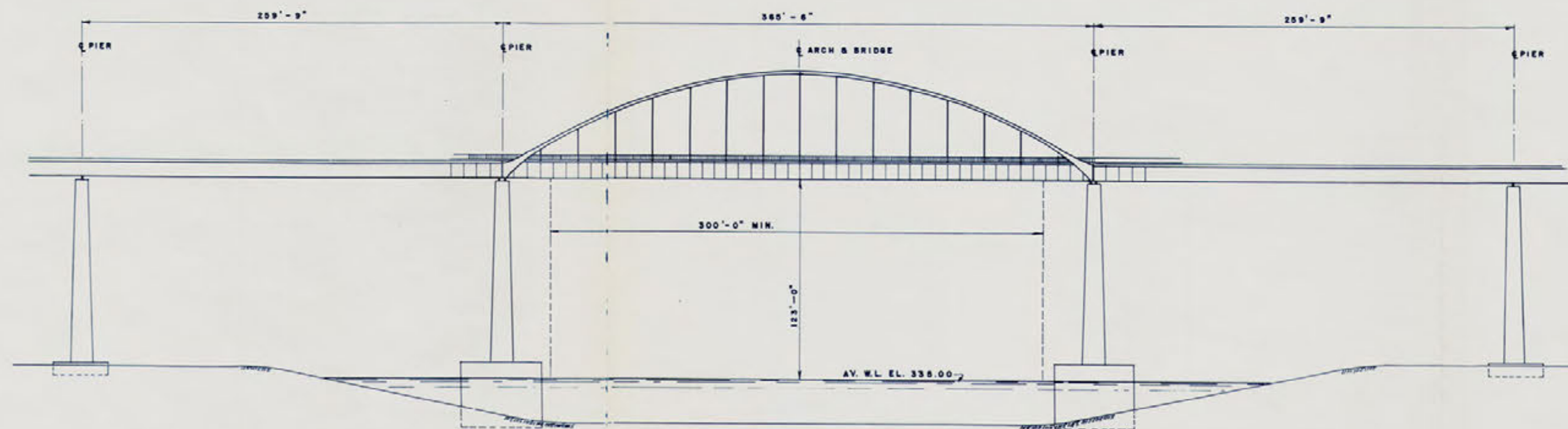
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LIMITED

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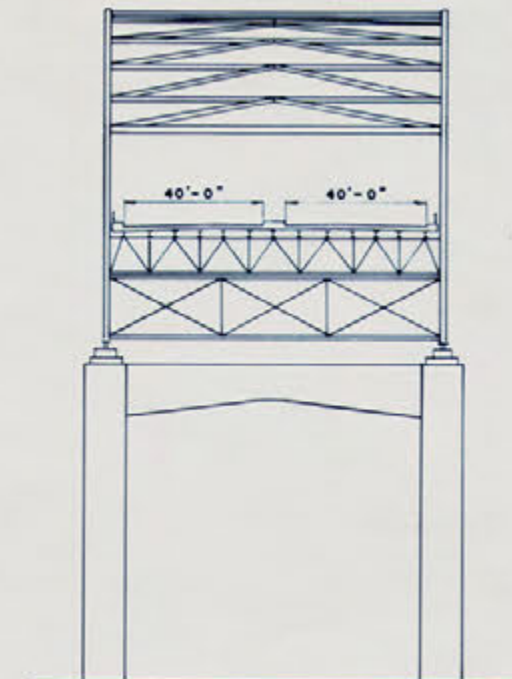




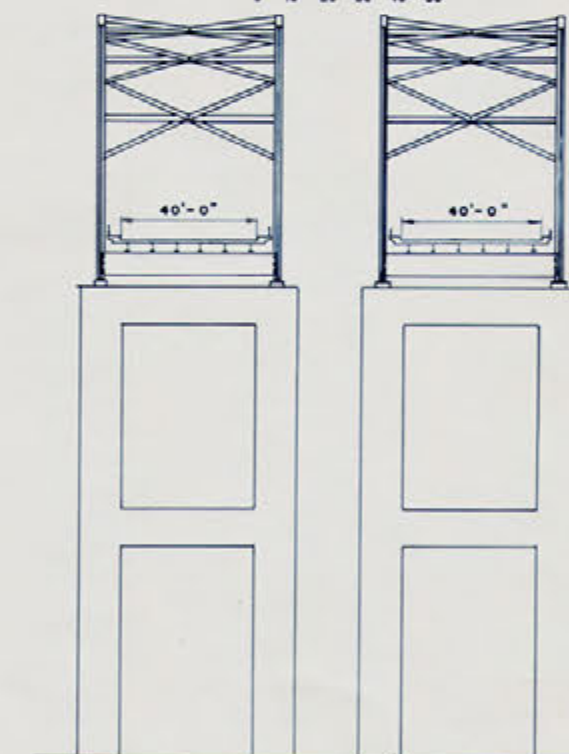
SCHEME "B"  
ELEVATION  
SCALE IN FEET  
0 10 20 30 40 50



SCHEME "C"  
ELEVATION  
SCALE IN FEET  
0 10 20 30 40 50



CROSS - SECTION  
SCALE IN FEET  
0 10 20 30 40 50

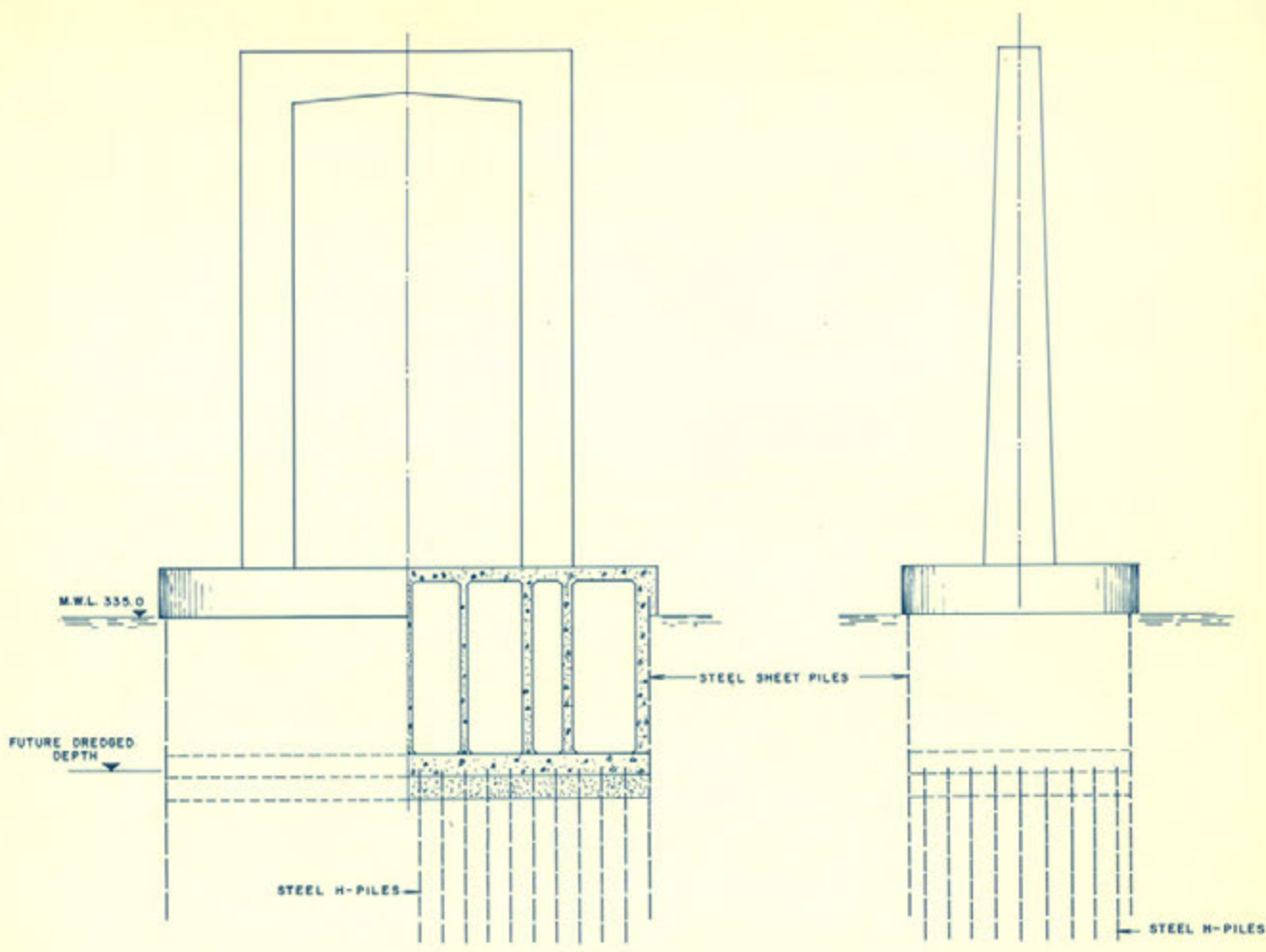


CROSS - SECTION  
SCALE IN FEET  
0 10 20 30 40 50

DEPARTMENT OF HIGHWAYS  
ONTARIO  
HOMER BRIDGE  
OVER  
THE WELLAND CANAL  
CANAL SPANS - ALTERNATIVES "B" & "C"

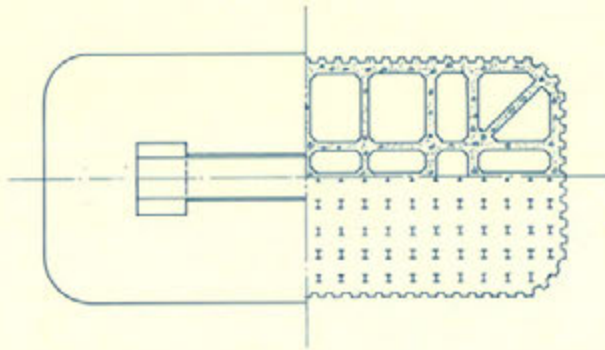
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FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED

No. 1903 - T - 6



SECTIONAL ELEVATION

END ELEVATION

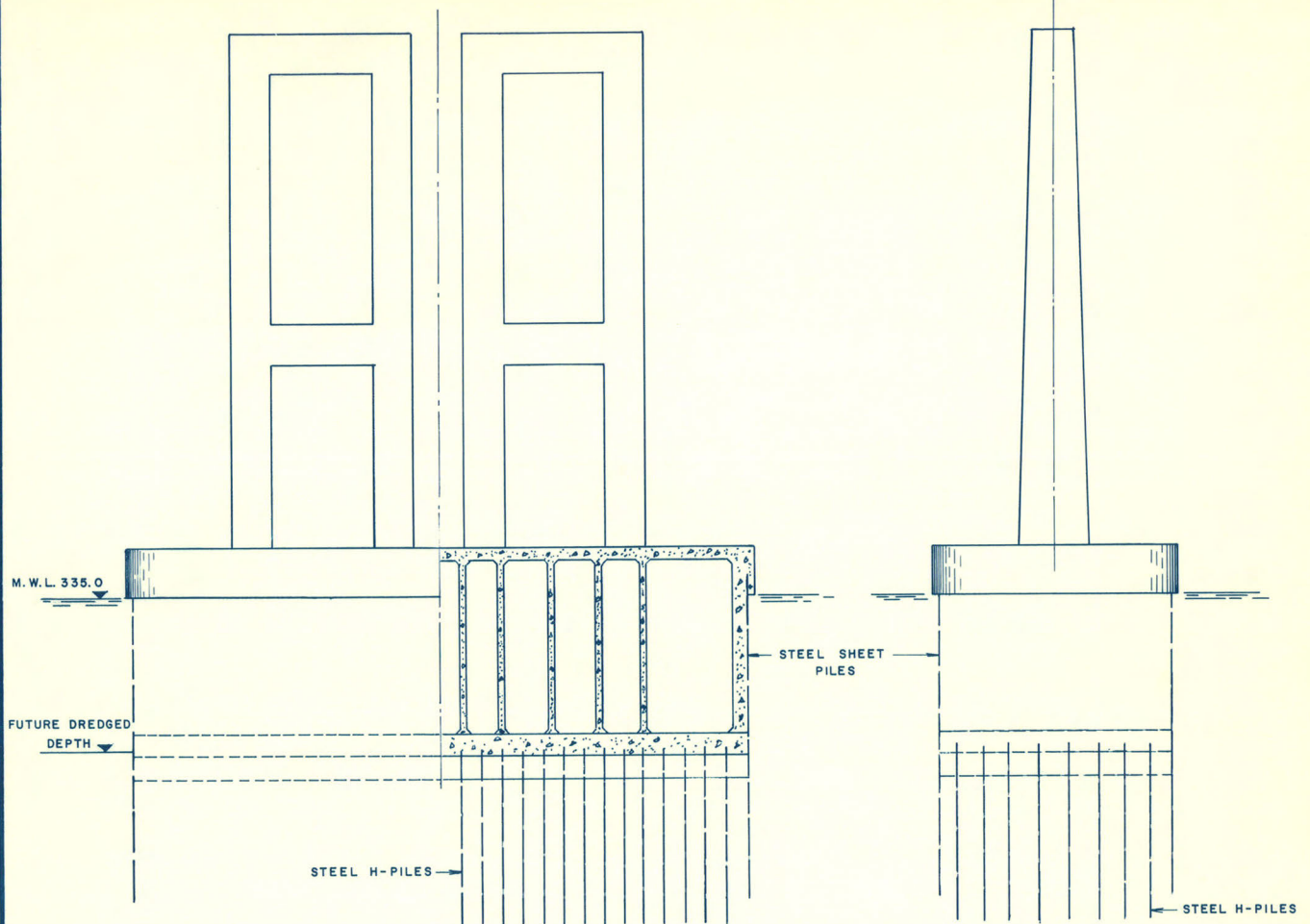


SECTIONAL PLAN



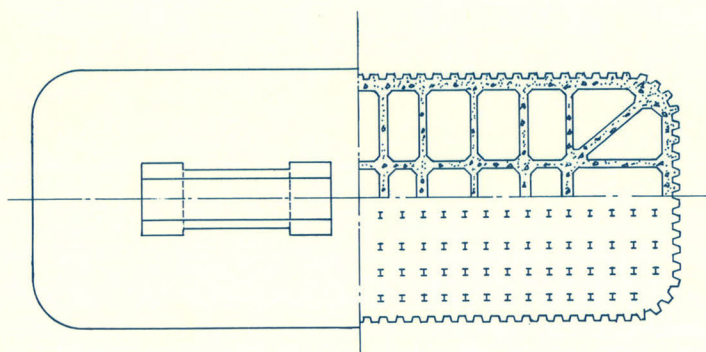
DEPARTMENT OF HIGHWAYS ONTARIO	FENCO
HOMER BRIDGE OVER THE WELLAND CANAL	FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED
CANAL SPANS - PROPOSED PIER	No. 1903 - T - 7





SECTIONAL ELEVATION

END ELEVATION



SECTIONAL PLAN



DEPARTMENT OF HIGHWAYS  
ONTARIO

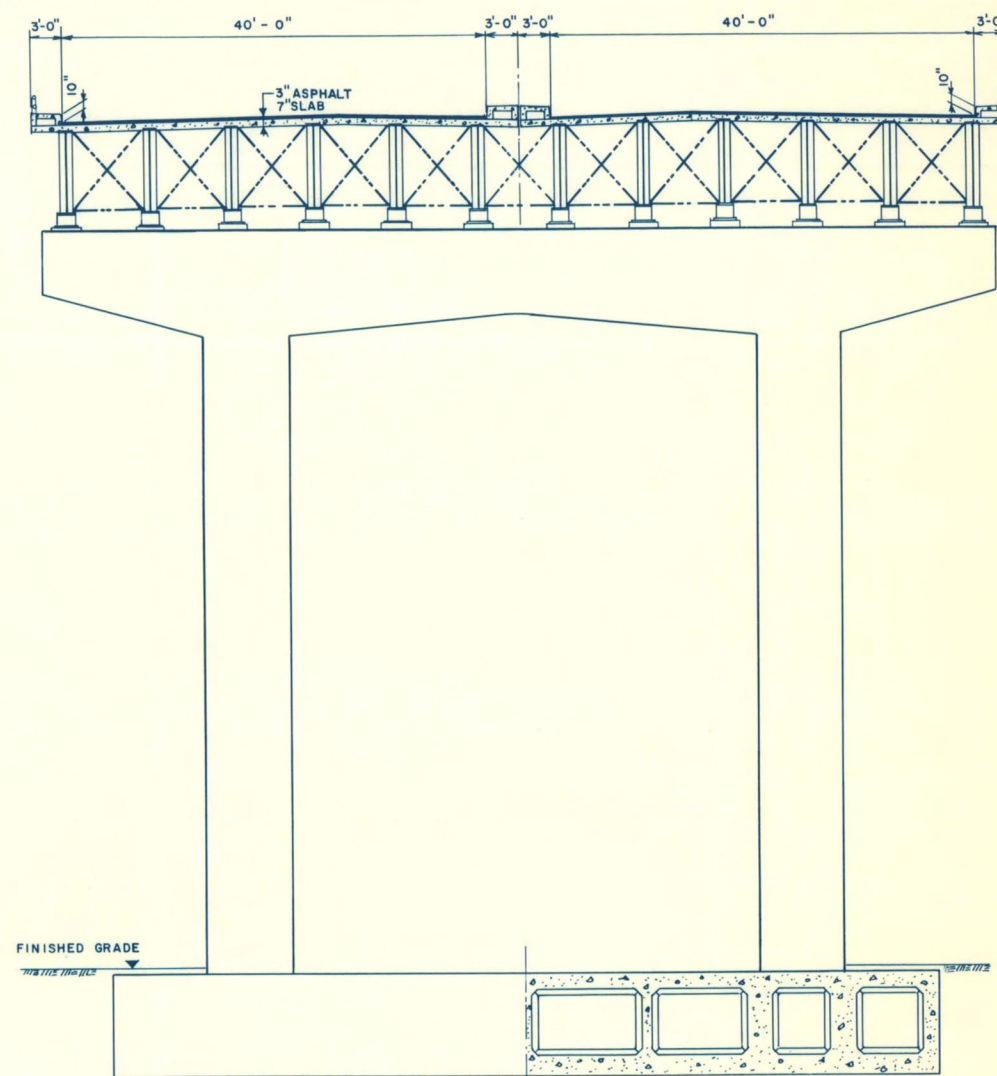
HOMER BRIDGE  
OVER  
THE WELLAND CANAL

CANAL SPANS - PIER - ALTERNATIVE "A"

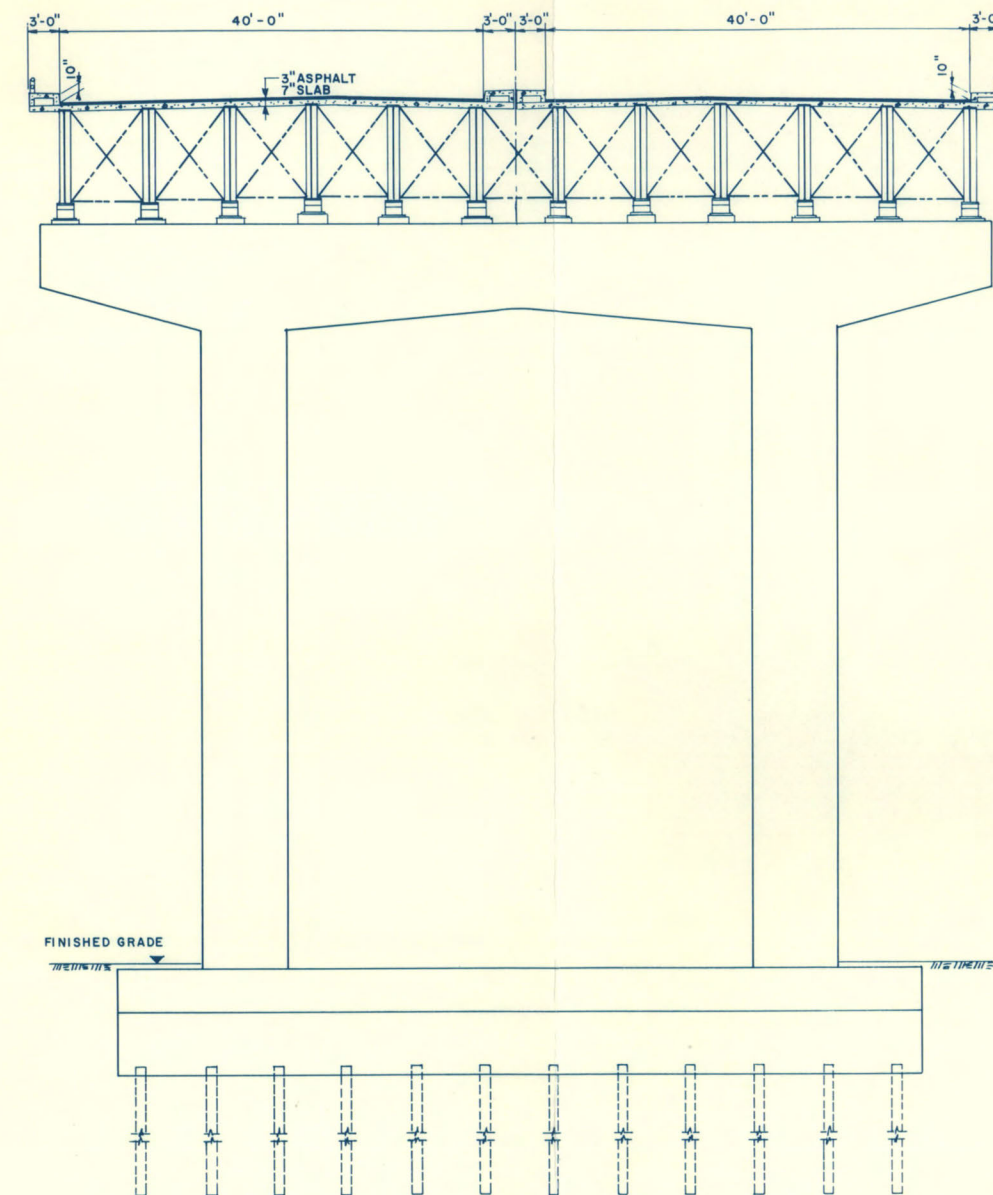
FENCO

FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED

No. 1903 - T - 8



WEST APPROACH



EAST APPROACH

SCALE IN FEET  
0 10 20 30

DEPARTMENT OF HIGHWAYS  
ONTARIO

HOMER BRIDGE  
OVER  
THE WELLAND CANAL

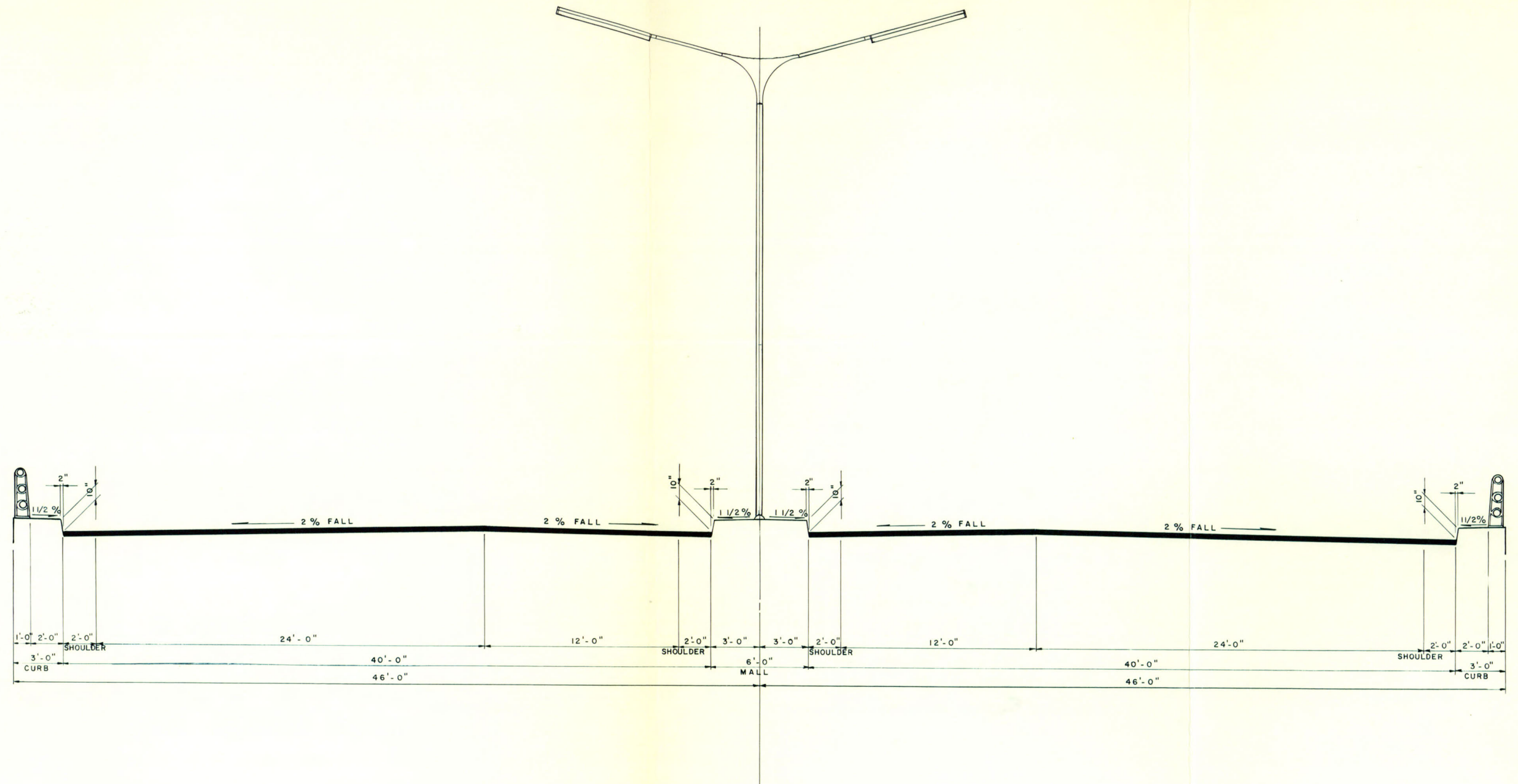
PROPOSED APPROACH SPANS

FENCO

FOUNDATION OF  
ENGINEERING CORP  
LIMITED

No. 1903 - T - 9





SCALE IN FEET  
0 1 2 3 4 5 6 7 8 9 10

DEPARTMENT OF HIGHWAYS  
ONTARIO

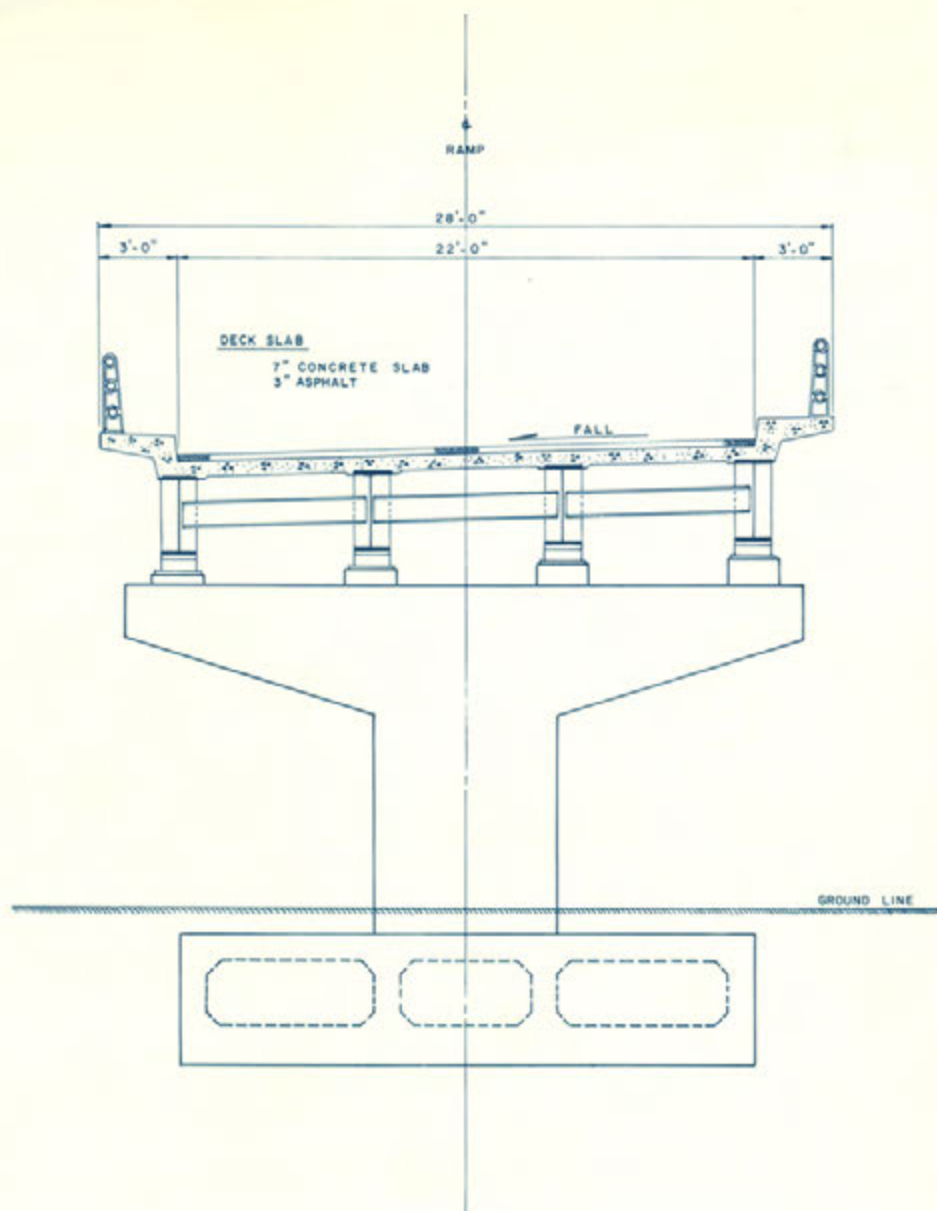
HOMER BRIDGE  
OVER  
THE WELLAND CANAL

TYPICAL DECK CROSS-SECTION

FENCO

FOUNDATION OF C.  
ENGINEERING CORP.  
LIMITED

No. 1903-T-10



SCALE IN FT.  
0 1 2 3 4 5 6 7 8 9 10

DEPARTMENT OF HIGHWAYS  
ONTARIO

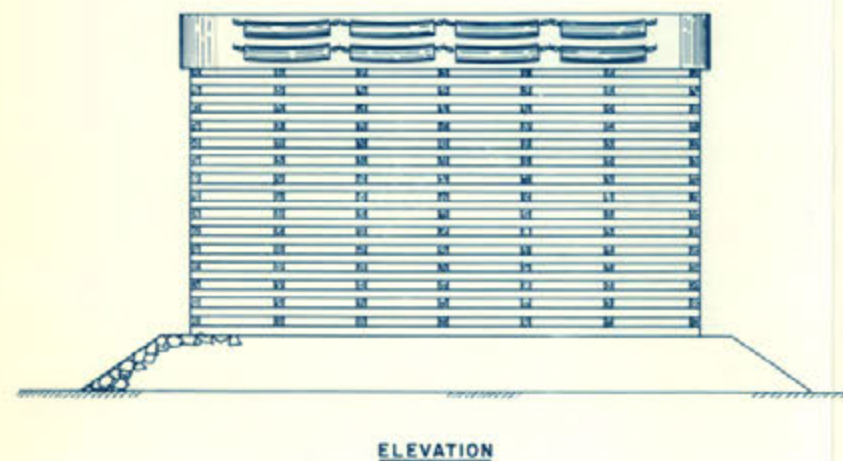
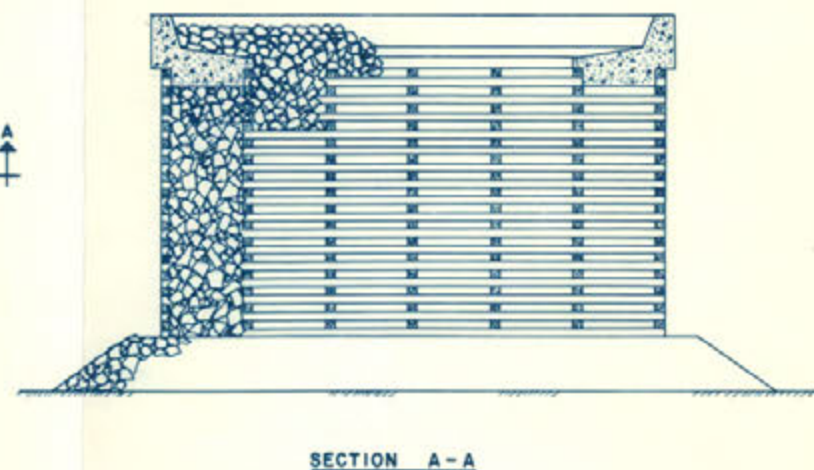
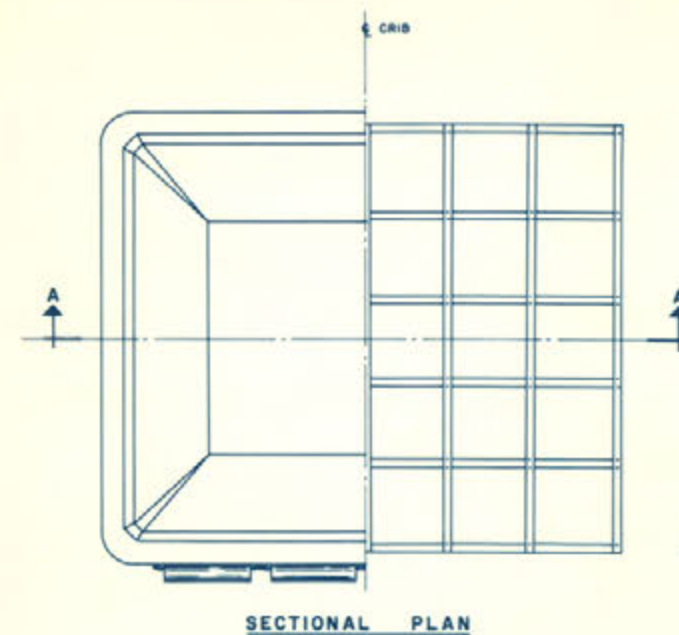
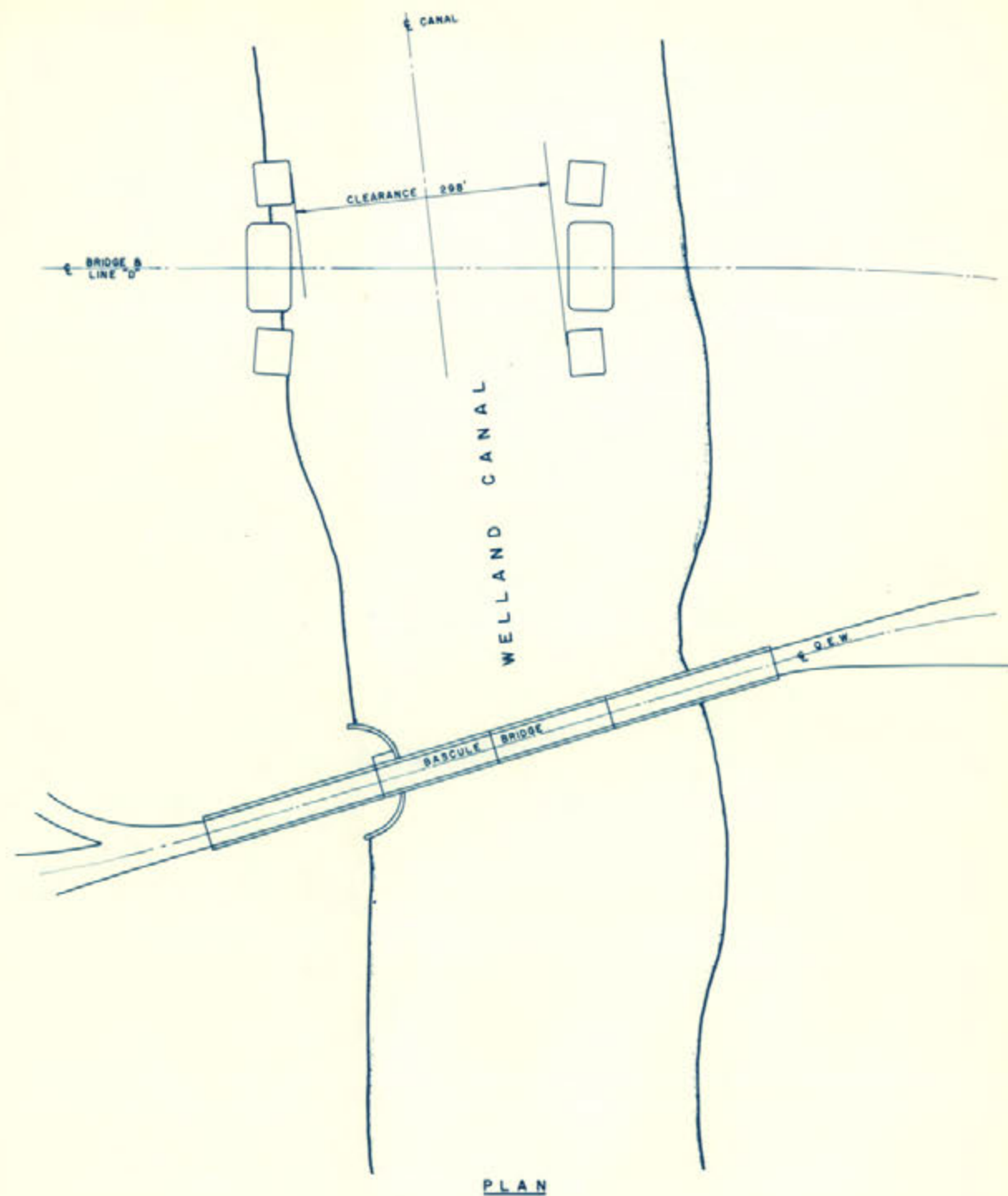
HOMER BRIDGE  
OVER  
THE WELLAND CANAL

TYPICAL RAMP CROSS-SECTION

FENCO

FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED

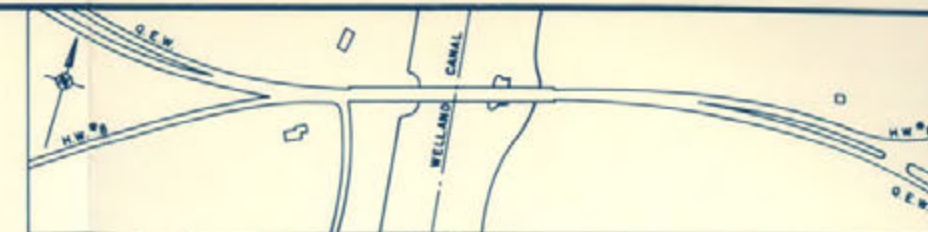
No. 1903-T-II



DEPARTMENT OF HIGHWAYS  
ONTARIO  
HOMER BRIDGE  
OVER  
THE WELLAND CANAL  
PIER PROTECTION WORKS

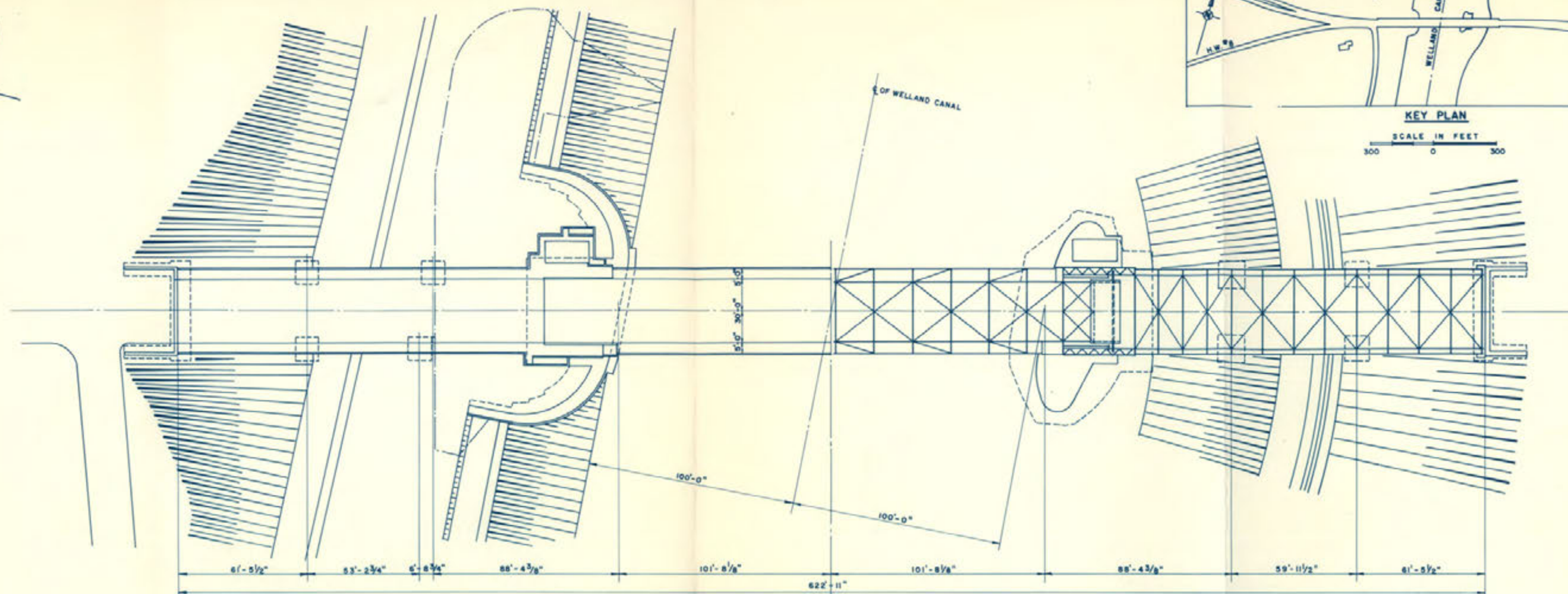
FENCO  
FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED  
No. 1903-T-12



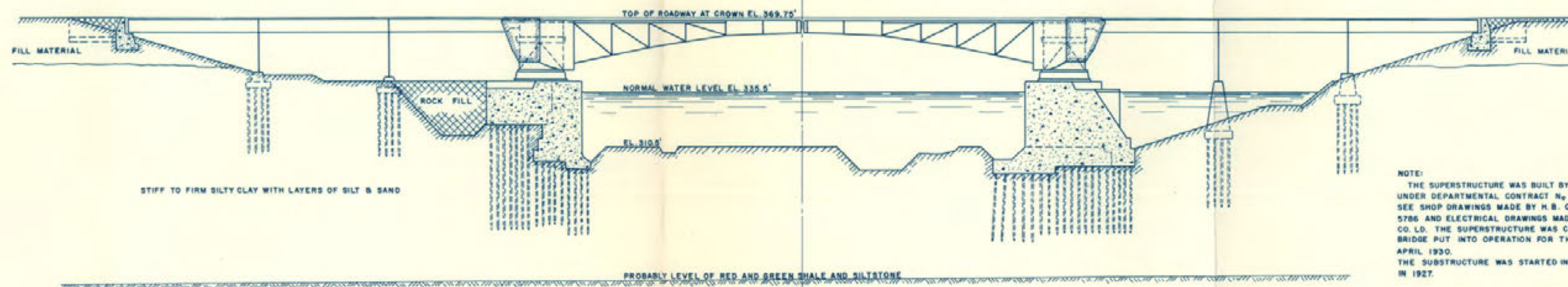


KEY PLAN

SCALE IN FEET  
300 0 300



GENERAL PLAN



SECTIONAL ELEVATION

SCALE IN FEET  
30 0 30 60 90

NOTE:  
THE SUPERSTRUCTURE WAS BUILT BY THE HAMILTON BRIDGE CO. LD. UNDER DEPARTMENTAL CONTRACT No. 26793 OF DECEMBER 31, 1926. SEE SHOP DRAWINGS MADE BY H.B. CO. LD. UNDER THEIR CONTRACT 5786 AND ELECTRICAL DRAWINGS MADE BY CANADIAN WESTINGHOUSE CO. LD. THE SUPERSTRUCTURE WAS COMPLETED IN 1928 AND THE BRIDGE PUT INTO OPERATION FOR THE PASSING OF NAVIGATION IN APRIL 1930. THE SUBSTRUCTURE WAS STARTED IN 1916 AND FINALLY COMPLETED IN 1927.

DEPARTMENT OF HIGHWAYS  
ONTARIO  
HOMER BRIDGE  
OVER  
THE WELLAND CANAL  
EXISTING BRIDGE

FENCO  
FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED  
No. 1903 - T - 13



APPENDIX I

SOIL INVESTIGATION

# GEOCON LTD

## HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC  
TELEPHONE UN. 6-7632

## DISTRICT OFFICES

14 HAAS ROAD  
REXDALE, TORONTO, ONT.  
TEL. CH. 4-8641

1425 WEST PENDER ST.  
VANCOUVER 5, B.C.  
TEL. MU. 1-6926

Rexdale, Ontario,  
January 29th, 1960.

Foundation of Canada Engineering Corporation  
Limited,  
8 Spadina Road,  
Toronto 4, Ontario.

Attention: Mr. R. W. Crudge, P. Eng.,  
Vice-President.

Re: Soil Conditions and Foundation Analyses,  
Proposed High Level Bridge,  
Homer, Ontario.

Dear Sirs:

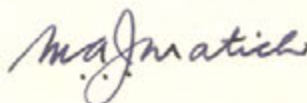
This letter accompanies our detailed report on the soil conditions at the site of the proposed high level bridge at Homer, Ontario.

The actual soil and water conditions encountered are described fully in the report and soil engineering properties are given. During the course of the work we have provided interpretation and analyses of such of the above data as was required to permit you to carry out detailed foundation design. Details of this interpretation are presented in the report.

We believe that our report gives all the required soils engineering information for the proposed project. If we can be of any further assistance, we would be pleased if you would contact us.

Yours very truly,

GEOCON LTD



M.A.J. Matich, P. Eng.,  
Chief Engineer.

MAJM/dw  
S6849

S6849  
REPORT  
TO  
FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED  
ON  
SOIL CONDITIONS AND FOUNDATION ANALYSES  
PROPOSED HIGH LEVEL BRIDGE  
HOMER ONTARIO

Distribution:

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- 10 copies - Department of Highways, Ontario,  
Toronto, Ontario.
- 4 copies - Geocon Ltd,  
Toronto, Ontario.

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## INTRODUCTION

Geocon Ltd has been retained by the Foundation of Canada Engineering Corporation Limited under the terms of our proposal dated June 8th, 1959, to investigate and report on the soil conditions along the site of the proposed Queen Elizabeth Way High Level Bridge over the Welland Canal at Homer, Ontario. The object of the investigation was to obtain and interpret all pertinent soils information relevant to the foundation design of the proposed bridge.

A description of the procedure, site and geology and detailed accounts of the soil and water conditions are given in Appendix I of this report. The results of the laboratory testing are shown on the Office Reports on Soil Exploration in Appendix I and on the Figures in Appendices II, IV and V. A drawing showing the borehole locations together with the inferred soil stratigraphy along Line "D" is given in Appendix VI.

## SUMMARIZED SOIL CONDITIONS

The site is covered by an average of 1 foot of brown clayey and sandy topsoil. Below this topsoil, in an area extending approximately between chainage 220+00 and 224+00 on the west side of the Welland Canal and chainage 231+00 and 247+00 on the east side of the canal is a stratum of brown silty sand and gravel with a maximum thickness of 11 feet. Underlying the brown silty sand and gravel, where it was encountered, and the topsoil along the remainder of the site is a stratum of silty clay varying between 13 and 65 feet in thickness. The upper portion of this stratum has been oxidized to a maximum depth of about 20 feet, resulting in a mottled grey and brown colour and generally a hard to stiff consistency. The lower part of the stratum is mainly grey in colour and has a stiff to firm consistency. The grey silty clay is generally underlain by a stratum of reddish-brown and grey layered silty clay ranging from 8 to 55 feet in thickness. The consistency of

this stratum is generally firm. The layered structure is generally indistinct and the composition of the stratum, except for a slightly higher silt fraction, is for engineering purposes similar to the overlying silty clay.

Subangular to subrounded pebble gravel sizes and pockets or lenses of grey silt or reddish-brown sandy silt are generally scattered throughout the grey silty clay and layered silty clay strata.

A stratum of grey to brown clayey silt underlies the layered silty clay. This stratum varies between 4 and 28 feet in thickness and is generally of stiff consistency. Beneath the grey to brown clayey silt is a very dense reddish-brown silty and sandy till up to 58 feet in thickness which overlies reddish-brown Queenston Shale bedrock.

There is evidence that the strata overlying the till have been preconsolidated to at least a depth of 40 to 50 feet and probably throughout their complete depth as a result of a previous lowering of the groundwater and lake level.

## DISCUSSION

### HISTORY

It is known from construction records of the present Welland Canal that considerable difficulty was encountered during the excavation and trimming of the canal slopes and also in the construction of the existing Queen Elizabeth Way Bascule Bridge at about Welland Canal chainage 297+00. The main piers of the Bascule Bridge were built before 1914. While the work was temporarily shut down a large slide occurred on the east bank



HISTORY (continued)

in July, 1917 and moved the east abutment of the bridge laterally towards the canal for about 20 feet. The foundation timber piles were inclined about 15 degrees from the vertical. The abutment was removed in 1919, the bank of the canal trimmed from the original 2 horizontal to 1 vertical slope to a 3 horizontal to 1 vertical slope and new piles driven in 1920. To meet the requirements of a heavier bridge than originally anticipated, further piles were driven behind the footings in 1926 and while this work was in progress, signs of movement developed at the west main pier. To deal with this situation, the existing bearing piles under the pier were jacked down to refusal and the pier underpinned at the toe by additional vertical and batter steel tube piles jacked to refusal and filled with concrete.

During the excavation of the canal from about 1921 to 1926 in the vicinity of the existing Bascule Bridge numerous bank slides on both sides of the canal took place. Remedial measures included flattening or benching of the slopes and the provision of rip-rap cover. Since completion of the present canal several large slides and numerous shallow surface slides have occurred along the existing banks in the vicinity of chainage 297+00. During the further dredging carried out several years ago in a programme of deepening the canal, numerous shallow slope slides took place resulting in subsidence of concrete slabs or rip-rap cover on the canal slopes. From observation of the canal banks in the vicinity of the proposed bridge at chainage 291+50, there is evidence of several major slides and evidence also to indicate that shallow surface movement of the canal slopes, which are generally about 3 to 4 horizontal to 1 vertical, is still taking place.

SLOPE STABILITY

To examine the requirements for stability of the canal slopes a series of consolidated undrained triaxial tests with pore pressure measurements were carried out on undisturbed samples of the silty clay strata. The resulting Mohr circle diagrams are shown on Figures 7, 8 and 9 of Appendix II. From the Mohr circle diagrams average shear strength parameters,  $C'$ , of about 150 pounds per square foot and  $\phi'$ , of about 20 degrees may be obtained.

It was considered that the most critical conditions of the canal slopes for stability would take place following rapid drawdown of the canal. Preliminary stability computations were, therefore, carried out for the rapid drawdown condition using total stress methods of analysis and a weighted value of  $\phi$  reduced to  $\frac{\phi'}{2}$  for the rapid drawdown case. The results of this analysis are shown on Figure 6 of Appendix III which indicates the approximate factor of safety for various slopes and shear strength parameters. Also plotted on this figure are the results of a slide which occurred on the east bank of the canal to the south of the existing Bascule Bridge at Line "C". Using a  $\phi'$  value of about 20 degrees and  $C'$  values between 100 and 200 pounds per square foot, it may be seen from the figure that, for canal banks with an average slope of 3 to 4 horizontal to 1 vertical and of the height encountered in the area of the proposed bridge crossing, the factor of safety against deep seated slides is 1.0.

Effective stress stability analyses were carried out for the rapid drawdown condition on the existing west bank of the canal at line "B" and the results are summarized on Figure 7 in Appendix III. The average overall slope of the bank at this section is about 3.2 horizontal to 1 vertical. The analyses were carried out



SLOPE STABILITY (continued)

using a  $\phi'$  value of 20 degrees and  $C'$  values of 100, 150 and 200 pounds per square foot, considering the forces between slices, and it was found that for a deep seated circular failure minimum factors of safety of 0.93, 0.99 and 1.05, respectively, were obtained. Examination of the west bank of the canal in the field indicated that a deep seated type slide had occurred about 50 feet to the south of this section. The factor of safety of about 1 obtained using  $\phi' = 20$  degrees and  $C' = 150$  pounds per square foot confirms that these parameters, chosen on the basis of the triaxial compression tests with pore pressure measurements, are of the right order.

After an overall study of the various lines proposed, the Consultants considered that Line "D" should be examined for the final bridge crossing.

Effective stress stability analyses were therefore carried out for the existing slope on the west bank at Line "D". The results of these analyses are shown on Figure 8 of Appendix III and it was found that the minimum factor of safety for a deep seated slide was 1.26 following rapid drawdown of the canal. The average overall slope of the section examined is approximately 4.4 horizontal to 1 vertical with portions of the slope as steep as 2.8 horizontal to 1 vertical. It has been observed that sections generally steeper than about 4 horizontal to 1 vertical are subject to surficial slides during drawdown conditions. Effective stress analyses of plane surfaces of sliding indicate that this type of failure may only be prevented by maintaining a slope angle of approximately  $\frac{\phi'}{2}$ . At the same time to maintain the minimum desirable factor of safety of 1.3 against

SLOPE STABILITY (continued)

deep seated slides a slope of approximately 5 horizontal to 1 vertical is required for the height of canal banks in the area. This is shown in Figure 6 of Appendix III for the total stress conditions and in Figure 9 for effective stress conditions. It is therefore recommended that for adequate long term stability the banks of the canal at the proposed bridge crossing be trimmed to a 5 horizontal to 1 vertical slope. It is further recommended that to prevent shallow surface slides, by accommodating surface piezometric variations, a 2 foot thick granular blanket be placed over the trimmed slope and protected against surface erosion by a rip-rap cover.

It is understood that the proposed fender cribs to the north of line "D" will be located as shown on Figure 10 of Appendix III. Total stress stability analyses were carried out to determine the safe and economical canal side slopes to be used during the construction of the fender cribs following a rapid drawdown of the canal. The results of the analyses and the suggested construction slopes are shown on Figure 10 of Appendix III.

Since soil sampling could not be carried out in the canal during the period of the investigation due to canal traffic, the stability analyses were based on the average strength profile shown on the above figure. It is therefore possible that minor sloughing can be expected in the steeper portions of the cut slopes, but this could be readily accommodated as construction progresses.

FOUNDATIONS

Based on preliminary foundation design data supplied by the Consultants, the allowable soil bearing pressure and probable settlement for spread footing foundations for the approach portions of the proposed high level bridge were examined. From available information, it is understood that the portion of the bridge under consideration, away from the main central spans, could be economically designed as simply supported spans. Each span would be of the order of 100 to 150 feet in length and the footings proposed would be of the rigid hollow box type, up to 95 feet in length and 45 feet in width and founded about 9 feet below existing ground level. The net total dead load would be of the order of 1000 pounds per square foot and the maximum total dead load plus full live load including wind, traction and temperature effects, would be of the order of 1750 pounds per square foot. The net total dead load plus an average sustained live load would be of the order of 1250 pounds per square foot.

Bearing capacity computations were carried out for footings at the site more than about 1000 feet away from either side of the canal and specifically outside the area between about chainages 218+00 and 247+00. Based on the shear strength profile shown on Figure 12 of Appendix II, obtained from the results of quick tri-axial compression tests, it was found that the factor of safety against ultimate failure, for the size of footings and loads discussed above, would be adequate and equal to 3 or greater. Computations were also carried out to determine if zones of elastic overstressing in the subsoil would occur at any depth beneath the footing. The factor of safety against elastic overstress was found to be at least 1.0.



FOUNDATIONS (continued)

Settlement computations were carried out for the rigid spread footing, 95 feet by 45 feet in size, using the results of the consolidation tests shown on Figures 14 to 23 inclusive, Appendix II, to predict the amount of consolidation of the underlying clay that would occur due to the effective footing load. From the range of preconsolidation effects with depth computed from the consolidation test results and shown on Figure 3 of Appendix III, it is considered that the clay strata have been preconsolidated to a considerable depth. It is further considered that at least the upper 50 feet of the clay strata have been preconsolidated to over 1250 pounds per square foot in excess of existing overburden pressure; below this depth the precise limits of overconsolidation are difficult to establish; however, the limits of consolidation settlement can be computed on the basis of overconsolidation throughout the full depth of the strata or normal consolidation below a depth of 50 feet.

As determined from the corrected pressure-void ratio relationships, average coefficients of compressibility,  $C_R$ , of 0.07 and  $C_C$ , of 0.44 were used in the settlement analyses within the limits discussed above. Using these values the possible range of maximum and minimum settlement was established to be approximately 5 and 2 inches, respectively, for the rigid spread footing, 95 feet by 45 feet in size, under a sustained loading of 1250 pounds per square foot. It is considered that the most probable total settlement will be of the order of 3 inches, of which approximately  $\frac{1}{2}$  inches will be elastic settlement occurring during construction. Due to the uniformity of the soil conditions in this area, it is considered that little tilting would take place across individual footings,

FOUNDATIONS (continued)

and that the differential settlement between adjacent footings would be small.

In view of the fact that the footings will be founded within the upper desiccated crust, the possibility of movement of the footings due to volumetric changes within the clay induced by a variation in groundwater level was investigated. A swelling test was carried out on a typical sample obtained from approximate footing elevation and the results are shown on Figure 23 of Appendix II. It was found that the maximum volume change of the soil when exposed to free water was about 3.4 percent under no vertical load. Under the stress conditions beneath a typical footing the gross vertical load will be in excess of 1 ton per square foot at a depth of 9 feet; consequently to induce any volumetric changes of the soil beneath the footing, of any significance, the water table would have to rise from an average depth of about 15 feet to a depth of 9 feet below ground level. In this unlikely event a uniform swelling movement of less than 1 inch might be expected across the whole site.

Based on an average coefficient of consolidation,  $C_v$ , value of 0.02 square inches per minute determined from the consolidation tests and a mean effective drainage path of 50 feet, approximating for three dimensional consolidation effects, it is estimated that 90 per cent of the consolidation settlement will take place in about 30 years. A plot of time versus percentage consolidation is given on Figure 4 of Appendix III.



FOUNDATIONS (continued)

Chemical analyses of the groundwater and soil obtained from approximate footing elevation were carried out and the results are summarized in Table I and Table II in Appendix V. These results indicate that the maximum soluble sulphur trioxide concentration in the groundwater is approximately 3700 p.p.m. and the maximum sulphur trioxide content in the soil is 0.17 per cent generally in an area between about chainages 250+00 and 265+00. It is known from experience that where the concentration of soluble  $\text{SO}_3$  in the groundwater generally exceeds 1000 p.p.m. or 0.5 percent solids in the soil, ordinary Portland cement concrete is attacked. 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.

It is understood that the main central span and adjacent spans of the bridge are to be designed as a continuous section. It is further understood that in order to eliminate differential settlement it is planned to found the footings for this section of the high level bridge on end bearing piles driven to refusal in the lower very dense till or to bedrock.

Along the portion of the site between about chainages 217+00 and 247+00 there is generally an absence of a thick desiccated crust on the west side of the canal; on the east side of the canal variable upper deposits consisting of fluvial sands and gravels generally cover a thin desiccated crust. Between about chainages 244+00 and 245+00 the site is covered by swamp deposits. Due to the variable upper soil conditions the use of spread footing foundations in this area would lead to possibly erratic differential settlement between footings. Consequently, it is



FOUNDATIONS (continued)

recommended that the bridge piers be founded on end bearing piles as planned, between about chainages 217+00 and 247+00, to eliminate detrimental differential settlement between adjacent pier foundations.

A variety of end bearing pile types would be suitable. In order to check the most economical pile type and length, it is suggested that prior to final foundation design, pile load tests be carried out at 3 locations, at about stations 222+00, 232+00 and 244+50; also that at each location, 3 piles be driven, one to the till stratum, one to the middle of the till stratum and one to refusal on bedrock. Each pile should be load tested independently, using dead load methods. It is further suggested that 2 specific pile types be used with at least 2 piles of a constant type at each location.

APPROACH EMBANKMENTS

It is understood that the approach embankments to the bridge will have a top width of about 100 feet and side slopes of 2 horizontal to 1 vertical and will not exceed approximately 25 feet in height.

Stability computations indicate that the proposed height of embankment will have an adequate factor of safety against lateral failure and that the critical height of the embankment section proposed is in excess of 40 feet. The settlement under various heights of embankment due to consolidation of the underlying clay strata has been computed; the results of the computations presented on Figure 5 of Appendix III. The basis of these analyses for varying degrees of overconsolidation with depth has been dis-

APPROACH EMBANKMENTS (continued)

cussed under spread footings in the Foundations section of the report. It is further computed that the time required for 90 per cent consolidation is approximately 30 years.

The most probable total settlement computed for the proposed embankment 25 feet in height is about 12 inches. This total settlement at the junction to the bridge structure, founded on spread footings, could induce significant differential settlement over the time required for complete consolidation. It is suggested therefore that measures be taken to induce as much as possible of the consolidation settlement before completion of the bridge structure. The expected consolidation settlement could be accelerated by initial overload of the proposed embankment to the maximum suggested height of 40 feet then trimming to the final height of 25 feet. Detailed analyses of the settlement to be expected under 15 feet of overload on the 25 foot high embankment show that approximately 20 percent of a total settlement of 18 inches will take place in 1 year. Consequently if spread footings are used throughout and the abutment is of the spill-through type and founded in the fill, then it may be seen that after a period of 1 year the maximum differential settlement to be expected between the abutment and adjacent pier footing would be about 6 inches. This figure may be further reduced if the fill end slope is placed to 3 horizontal to 1 vertical thus inducing some additional load on the pier footing. This measure would tend to increase the footing settlement by approximately 1 inch.

It is recommended that the procedure discussed above be followed and the time rate of settlement observed during and

APPROACH EMBANKMENTS (continued)

following overload by piezometers located within the underlying clay strata. The total degree of settlement should further be controlled by the use of settlement plates placed at approximate existing ground level.

PERSONNEL

The field work was carried out under the overall supervision of Mr. J.L. Seychuk, assisted by Mr. N. McCammon and at various stages by Messrs. A. Prior, R. Sorokoski, and E. Henye. This report was written by Messrs. Seychuk and McCammon, checked by Mr. V. Milligan and reviewed by Mr. M.A.J. Matich.

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S6849



J. L. Seychuk, P. Eng.,  
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## APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

## PROCEDURE

The field work was carried out in two successive stages. The first stage was commenced on April 14th, 1959 and completed on September 11th, 1959. A total of 6 boreholes along lines "B" and "C" and 28 boreholes along Line "D" were put down. In-situ vane testing was carried out in 16 of the boreholes. Three additional boreholes were put down along Line "D" in the second stage of the investigation which was started on October 16th, 1959 and finished on October 23rd, 1959. The borings, in both stages of the investigation, were put down in HX and BX sizes to a maximum depth of 150 feet, using skid-mounted standard machine drillrigs. Bedrock was proved in 20 boreholes by core drilling in BXT and AXT sizes to a maximum depth of 26 feet.

The locations of the boreholes along Line "D" together with the inferred soil stratigraphy are shown on Drawing S6849-1 in Appendix VI. The locations of the boreholes along Lines "B" and "C" are shown on Drawing S6849-2 in Appendix VI. A detailed log of each boring put down in this investigation is given on the Office Reports on Soil Exploration, Figures 1 to 19 inclusive, in this Appendix.

A total of 5 piezometers and 10 standpipes were installed along Line "D" using mobile power auger equipment. Water level observations were taken in these installations and in the boreholes during the course of the investigation. The locations and details of these installations together with the stabilized groundwater level are shown on Drawing S6849-2 in Appendix VI.

Four 6-inch diameter holes were put down to below the water table by the power auger in order to obtain samples of the groundwater for chemical analyses. The locations of these holes are shown on Drawing S6849-2 in Appendix VI and the results of the chemical tests are given in Table I of Appendix V.

PROCEDURE (continued)

II.

A grooved plastic casing guide for slope indicator measurements, in order to detect horizontal movement of the soil mass in the canal bank, was installed to a depth of 75 feet below ground level on the west bank of the canal to the north of the existing Bascule bridge. The location and details of this installation are shown on Drawing S6849-2 in Appendix VI.

A brief survey of the known gravel pits in the Niagara Peninsula was carried out between September 21st, 1959 and September 23rd, 1959. The results of the survey are given in Appendix IV.

The laboratory testing of soil samples was carried out in the Soil Mechanics Laboratories of Geocon Ltd in Toronto and Montreal and in the Soil Mechanics Laboratory of the Department of Highways, Ontario, at Downsview. The chemical analyses of the soil and groundwater samples were performed at the Laboratories of Warnock Hersey Company Limited in Toronto, Ontario. The results of the testing are shown on the Office Reports on Soil Exploration and on the Figures in Appendix II and the Tables in Appendix V. The samples remaining after testing will be stored until September 1st, 1960, at which time you will be contacted with regard to their disposal.

Elevations are referred to Geodetic Datum and were obtained from Geodetic and Department of Highways, Ontario, Bench Marks located in the area. Borehole chainages are with reference to the centre lines of the proposed bridge crossings as located by the Department of Highways, Ontario.

SITE AND GEOLOGY

The site of the proposed Queen Elizabeth crossing over the Welland Canal at canal chainage 291+50 near Homer, Ontario, is located on the lowland between the Niagara Escarpment to the south and Lake Ontario to the north.

**GEOCON**



Bedrock beneath this whole lowland area consists of red oxidized sandy Queenston shale of Ordovician age.

The bedrock is overlain by about 120 feet of glacial and post-glacial sediments of complex geological history. A dense reddish sandy glacial till directly overlies the Queenston shale and consists largely of material derived from it.

The till is overlain by a variable thickness of layered clay probably deposited in a moat-like lake located between the Niagara Escarpment and a large ice mass in Lake Ontario. The layered clay grades upwards into a structureless silty clay stratum containing small pebbles and tiny pockets of silt. The upper portion of this stratum has apparently been oxidized to a brown colour and preconsolidated by desiccation.

The retreat of the Lake Ontario ice lobe resulted in the formation of Lake Iroquois. The shore line and associated beach deposits of this lake generally cover the central portion of the site.

With further retreat of the ice and the opening of the St. Lawrence River, the level of Lake Ontario may have dropped nearly to sea level resulting in oxidation of the upper clay strata and possible preconsolidation effects to a considerable depth. Since this time, Lake Ontario has been slowly rising as the outlet at Kingston has slowly rebounded following the retreat of the continental ice mass.

#### SOIL CONDITIONS

The principal soil strata encountered by the borings in Line "D" are as follows:

Topsoil

A layer of brown clayey and sandy topsoil up to 2 feet in thickness extends across the site. In borehole #244+48-CL a layer of loose to compact brown sandy silt with organic matter was encountered to a depth of about 6 feet.

Loose to Compact Brown Silty Sand and Gravel

Below the topsoil between boreholes #230+26-11R and #246+80-25R on the east side of the canal, and boreholes #223+14-2L and #220+34-CL on the west side of the canal is a stratum of brown silty sand and gravel which reaches a maximum thickness of 11 feet in borehole #238+34-3L. A similar material was encountered to a depth of 25 feet in borehole #224+22-5.5L. This is a fill embankment most probably derived from this stratum. The general shape of the grain sizes is subrounded to rounded.

Standard penetration tests carried out in the stratum gave "N" values ranging from 7 to about 30 blows per foot with an average of 17 blows per foot indicating the relative density of the stratum to be loose to compact.

Three mechanical and hydrometric grain size analyses were carried out on samples from the stratum, and these are plotted on Figure 1 of Appendix II.

From these it can be seen that the stratum is generally well-graded containing from 12 to 19 per cent silt sizes, 36 to 60 per cent sand sizes, and 22 to 52 per cent gravel sizes.

Hard to Firm Grey & Brown to Grey Silty Clay

Underlying the topsoil and the brown silty sand and gravel, where it was encountered, is a stratum of silty clay varying between 13 and 65 feet in thickness. The top portion of this stratum, to a maximum depth of 20 feet, is generally mottled grey and brown in colour, and below this depth the colour of the stratum is mainly grey. It is considered that the colour change is due to desiccation of the stratum. Weathering of the silty clay, where it is not overlain by the brown silty sand and gravel, has generally taken place in the upper 5 to 6 feet. There are widely scattered subangular to subrounded pebble gravel sizes distributed throughout the stratum. These are generally  $1/8$  inch to  $1/4$  inch in size, though occasionally being as large as  $2\frac{1}{2}$  inches. The pebble gravel is composed mainly of grey limestone and reddish-brown shale fragments.

Irregularly scattered pockets or lenses of grey silt or reddish-brown sandy silt, generally  $1/8$  inch to  $\frac{1}{2}$  inch in size were also encountered. On occasions these small lenses group together to form a large heterogeneous pocket several inches in diameter.

The upper portion of the stratum, where desiccation has occurred, has a fissured and brittle structure resulting from oxidation following a drawdown of the groundwater level. The lower portion has no specific structure.

The liquid limits obtained for the stratum generally varied between 40 and 60 with an average value of 47. The plasticity index ranged generally from 18 to 35 with an average value of 25. On Figure 3 of Appendix II typical values of the Atterberg limits are plotted on the plasticity chart together with the



Hard to Firm Grey & Brown to Grey Silty Clay (continued)

Casagrande "A" line. The stratum may thus be classified as an inorganic silty clay of medium to high plasticity. On Figure II of Appendix II, values of Atterberg limits from several boreholes are plotted against depth below ground level for the silty clay and underlying clayey strata.

Three typical grain size distribution curves for the stratum are shown on Figure 1 in Appendix II. From these it can be seen that the silty clay contains from 35 to 60 per cent clay sizes and 40 to 64 per cent silt sizes.

The Activity of the silty clay, which is defined as the plasticity index divided by the percentage of grain sizes less than 0.002 millimeters varies between 0.42 and 0.66. This is in the inactive zone for clays.

The Liquidity Index, the ratio of moisture content minus plastic limit to plasticity index, has been computed to have an average value of 0.10 for the upper desiccated crust and an average value of 0.36 for the lower portion.

The shear strength was determined by in-situ vane testing and undrained triaxial compression tests. Seven typical stress-strain curves obtained from the undrained tests are shown on Figure 5 of Appendix II. The shear strengths obtained from the in-situ vane tests are generally equal to or less than those given by the undrained triaxial compression tests, and hence the latter results have been utilized for design purposes.

In the desiccated portion of the stratum the shear strength varies between about 850 and 8000 pounds per square foot with an

Hard to Firm Grey & Brown to Grey Silty Clay (continued)

average value of about 3000 pounds per square foot. The consistency of the crust is therefore estimated to be hard to stiff. There is generally an absence of a hard to stiff upper crust between about chainages 220+00 and 250+00. At borehole #203+70-37L the desiccated crust has an apparent lower average shear strength of 1020 pounds per square foot. Below the desiccated portion the shear strength ranges generally from 500 to 2000 pounds per square foot with an average of 1190 pounds per square foot. From this the consistency is established as stiff to firm. The higher shear strength values were generally obtained close to the upper desiccated portion of the stratum. On Figure 12 of Appendix II values of shear strength from several boreholes are plotted against depth below ground level for the silty clay and underlying clayey strata.

..... The sensitivity of the silty clay as determined by in-situ vane testing in the undisturbed and remoulded state was about 2 to 3.

Wet unit weight determinations for the upper desiccated portion of the stratum gave an average value of 126 pounds per cubic foot at a corresponding average moisture content of 25 per cent. The lower portion gave wet unit weights ranging from 114 to 136 pounds per cubic foot with an average of 123 pounds per cubic foot. This corresponded to a natural moisture content which varied generally between 25 and 40 per cent with an average of about 32 per cent. On Figure 10 of Appendix II values of moisture content from several boreholes are plotted against depth below ground level for the silty clay and underlying clayey strata.



Hard to Firm Grey and Brown to Grey Silty Clay (continued)

For design purposes a wet unit weight of 125 pounds per cubic foot and a submerged unit weight of 65 pounds per cubic foot have been used for the complete stratum.

Four consolidated undrained triaxial compression tests with pore pressure measurements were carried out on samples from the lower portion of the stratum and the resulting Mohr stress circles are shown on Figure 8 of Appendix II. From the envelope, an effective angle of shearing resistance of 19.5 degrees and an effective cohesion of 120 pounds per square foot were obtained.

Values of the pore pressure parameter  $A_f$ , generally the parameter relating change in deviator stress with change in pore water pressure, were determined for various degrees of overconsolidation on samples of the silty clay and underlying layered silty clay. The resulting plot of overconsolidation ratio versus  $A_f$  is shown on Figure 13 of Appendix II. Undrained triaxial compression tests with pore pressure measurements were next carried out on undisturbed soil samples consolidated under an all around pressure approximately equal to existing overburden pressure and the resulting  $A_f$  values together with values of  $A$  at 75 per cent of the failure strain are tabulated in Table I. With these  $A_f$  values, using the results of known overconsolidation ratio versus  $A_f$ , a plot of probable overconsolidation ratio versus depth was made and is also shown on Figure 13 of Appendix II. This plot indicates a probable overconsolidation ratio of over 1 above a depth of about 50 feet.

Twenty-two consolidation tests were carried out on samples obtained from different depths below ground level in the silty



Hard to Firm Grey & Brown to Grey Silty Clay (continued)

clay. The resulting log pressure-void ratio curves are shown on Figures 14 to 23 in Appendix II. A plot of the computed coefficient of consolidation,  $C_v$ , against log pressure for each test is also given in the figures. In addition, the figures also show the probable field consolidation curves obtained by the Schmertmann method. The results from the consolidation tests for the silty clay and the underlying strata are summarized in Table II.

The compression index,  $C_c$ , obtained from the field curves, varied between 0.13 and 0.83 with an average value of 0.42. The rebound compression index,  $C_R$ , applicable up to the preconsolidation load, varied between 0.03 and 0.11 with an average value of 0.06. For design purposes a  $C_c$  of 0.42 and a  $C_R$  of 0.06 have been used. A plot of  $C_R$ ,  $C_c$  and  $C_c'$ , the uncorrected slope of the log pressure-void ratio curve, against depth below ground level is given on Figure 2 of Appendix III for all the consolidation tests carried out on the silty clay and underlying clayey strata.

The minimum and maximum preconsolidation pressures have been computed by graphical methods from the log pressure-void ratio curves and the results are summarized in Table II and presented graphically against depth in Figure 3 of Appendix III for all the consolidation tests carried out on the silty clay and underlying clayey strata. These results indicate appreciable preconsolidation effects down to a depth of 40 to 50 feet. Below this depth the clay strata may be slightly overconsolidated or normally consolidated; however from the overall pattern of the results, it is estimated that the strata below 40 to 50 feet have been subjected to a past pressure of at least 1000 pounds per square foot in excess of present overburden pressure.

Hard to Firm Grey & Brown to Grey Silty Clay (continued)

A swelling test was carried out on a sample of the silty clay taken from a depth of about 8 feet below ground surface and the results are presented graphically on Figure 23 of Appendix II. It may be seen from the curves that the maximum swelling pressure exerted was about 2.8 tons per square foot and the maximum change in height, as compared to the initial height of the sample and with complete lateral restraint, was about 3.4 per cent under an almost negligible load.

The permeability of the grey silty clay, as calculated from the consolidation test results, ranged from  $0.2 \times 10^{-7}$  to  $3.2 \times 10^{-7}$  centimeters per second with an average value of about  $1 \times 10^{-7}$  centimeters per second.

Firm Reddish-Brown and Grey Layered Silty Clay

The grey silty clay is underlain in all boreholes except in borehole #256+93-CL, borehole #257+20-10L and borehole #262+32-40R by a stratum of reddish-brown and grey layered silty clay ranging between about 8 and 55 feet in thickness. The layers in this stratum are indistinct in the majority of the boreholes. In the remainder, they are indistinct at the top of the stratum, but become more distinct with depth. The separate layers are composed of clay with varying silt contents. The reddish-brown layers vary in thickness from a fraction of an inch to about 1 inch though exceptionally to  $3\frac{1}{2}$  inch size. The grey layers generally vary between  $1/16$  inch and 1 inch with a maximum size of 3 inches being encountered. The increase and decrease in the thickness of the layers appears to follow no general pattern. Except for their difference in colour, the individual layers which comprise the stratum are generally of the same composition.



Firm Reddish-Brown and Grey Layered Silty Clay (continued)

The stratum contains scattered subangular to subrounded pebble gravel sizes, generally 1/8 inch to 1/4 inch in diameter, and some pockets of grey silt and reddish-brown sandy silt with a maximum size of about 1/4 inch. The percentage of these inclusions is appreciably less than in the overlying grey silty clay stratum and in some sections they are absent altogether.

The liquid limits obtained for the layered silty clay varied generally between 40 and 55 while the plasticity index varied between about 16 and 32. The average liquid limit is about 47 and the average plasticity index about 24. Typical values of the Atterberg limit determinations for the stratum are plotted on the plasticity chart in Figure 4 of Appendix II, together with the Casagrande "A" line. The layered silty clay may thus be classified as an inorganic silty clay of medium to high plasticity.

Two typical grain size distribution curves for this stratum are plotted on Figure 1 of Appendix II. These show that the layered silty clay contains from 60 to 70 percent clay sizes, the remaining sizes being in the silt range. The activity of the stratum varies between 0.34 and 0.40, and this is in the inactive zone for clays.

The Liquidity Index was computed to have an average value of 0.54.

The shear strength was obtained from in-situ vane testing and undrained triaxial compression tests on samples of the stratum. Typical stress-strain curves from the latter tests are shown on Figure 6 of Appendix II. As in the overlying grey



Firm Reddish-Brown and Grey Layered Silty Clay (continued)

silty clay stratum, the in-situ vane tests gave generally lower values than the undrained tests and the latter results have been used for design purposes. The shear strength in this stratum varies generally between 500 and 1100 pounds per square foot with an average value of about 800 pounds per square foot. The consistency is thus estimated to be firm. The shear strength in the boreholes between about chainages 220+00 and 224+00 is slightly below the average for the stratum.

For the borings put down in the proposed embankment and spread footing areas, outside the area between about chainages 218+00 and 246+00, a plot of shear strength versus depth, for the clayey strata overlying the till, is given on Figure 12 of Appendix II. Also plotted on this figure is the shear strength line,  $(C)_n$ , for a normally consolidated clay under present effective overburden pressure and assuming  $(c/p)_n$  values of 0.20 above a 50 foot depth and 0.18 below a 50 foot depth. These values are typical for clays of this plasticity. A comparison of the average shear strength with depth determined from the triaxial compression tests and the computed normally consolidated shear strength line indicates that the strata have been possibly preconsolidated to at least a 50 foot depth.

The sensitivity as determined by in-situ vane testing varied between about 2.5 and 3.0.

The natural moisture content of samples of the stratum varied between about 30 and 46 per cent with an average value of 36 per cent while the corresponding wet unit weights ranged generally from 114 to 125 pounds per cubic foot with an average value of about 119 pounds per cubic foot. For design purposes a wet unit

Firm Reddish-Brown and Grey Layered Silty Clay (continued)

weight of 125 pounds per cubic foot and a submerged unit weight of 65 pounds per cubic foot have been used.

Four consolidated undrained triaxial compression tests with pore pressure measurements were carried out on typical samples of the layered silty clay in line "D" and five similar tests were carried out for the corresponding stratum in line "B". The resulting Mohr's stress circles are plotted on Figures 9 and 7 in Appendix II. From the envelopes effective angles of shearing resistance of 19.0 and 20.5 degrees and effective cohesions of zero and 190 pounds per square foot were obtained. Combining these results with the results obtained from the Mohr envelope from the silty clay stratum, an angle of shearing resistance of 20 degrees and a cohesion of 150 pounds per square foot were decided on as general parameters for design.

Fourteen consolidation tests were carried out on samples of the stratum and the resulting log pressure-void ratio curves are shown on Figures 14 to 23 in Appendix II together with a plot of the computed coefficient of consolidation,  $C_v$ , against log pressure for each test. The probable field consolidation curves obtained by the Schmertmann method are also shown. The compression index,  $C_c$ , as determined from the field consolidation curves, ranged from 0.24 to 0.60 with an average value of 0.42, while the rebound compression index,  $C_R$ , varied between 0.03 and 0.09 with an average of 0.06. In the settlement computations a  $C_c$  of 0.42 and a  $C_R$  of 0.06 have been used.

The permeability of the layered silty clay, as calculated from the consolidation test results, varied between about  $0.7 \times 10^{-7}$  and  $1.6 \times 10^{-7}$  centimeters per second with an average value of about  $1 \times 10^{-7}$  centimeters per second.

TABLE 1

XIV.

PORE WATER PRESSURE PARAMETER "A" RESULTS

Borehole Number	Sample Elevation	Sample Depth	Stratum	A	
				At Failure $A_f$	At 75% Failure Strain
230+26-11R	308.0	41.4	Grey silty clay	0.33	0.35
	307.3	42.1	" " "	0.38	0.31
	307.7	41.7	" " "	0.38	0.31
	307.0	42.4	" " "	0.19	0.20
238+34-3L	317.2	42.0	" " "	0.12	0.14
231+16-CL (Line "B")	304.2	36.3	Layered silty clay	0.44	0.55
	303.8	36.7	" " "	0.31	0.29
	303.5	37.0	" " "	0.45	0.42
	303.2	37.3	" " "	0.13	0.18
	308.2	32.3	" " "	0.064	0.10
225+15-39.5R	287.3	51.4	" " "	0.86	0.83
	287.0	51.7	" " "	0.80	0.76
	286.6	52.1	" " "	0.86	0.76
	286.3	52.4	" " "	-0.05	0
200+00-13L	285.8	52.0	" " "	0.39	0.43
238+34-3L	307.2	52.0	" " "	0.37	0.38

Note: All tests have been carried out under an all around pressure equal to or less than the existing overburden pressure.



TABLE II

XV.

## CONSOLIDATION TESTS - SUMMARY OF RESULTS USED FOR COMPUTATIONS

<u>Line</u>	<u>Borehole Number</u>	<u>Sample Depth</u>	<u>Sample Elev'n</u>	<u>e<sub>o</sub></u>	<u>C<sub>c</sub></u>	<u>C<sub>R</sub></u>	<u>C<sub>c</sub>'</u>	<u>P<sub>c</sub>Min.</u>	<u>P<sub>c</sub>Max.</u>
D	182+00-54L	22'	313	0.525	0.28	0.06	0.24	5.0	7.5
D	186+50-53L	21	314	0.823	0.35	0.05	0.29	2.0	5.0
		32	303	0.839	0.38	0.05	0.35	2.5	3.3
		42	293	1.130	0.59	0.06	0.54	2.5	3.5
		57	278	1.122	0.56	0.08	0.54	2.3	2.9
		71	264	0.683	0.24	0.03	0.21	1.6	2.9
D	202+00-CL	22	318	0.930	0.42	0.07	0.35	1.8	3.4
		32	308	0.880	0.51	0.07	0.44	2.2	3.5
		37	303	0.923	0.31	0.04	0.28	1.4	2.6
		47	293	1.053	0.52	0.06	0.44	1.5	2.6
		57	283	0.845	0.34	0.03	0.29	1.8	3.2
		66	274	0.854	0.43	0.03	0.37	2.2	3.2
D	203+70-37L	17	322	0.820	0.33	0.05	0.31	3.6	4.1
		32	307	0.808	0.38	0.05	0.32	1.8	2.9
		51	288	0.930	0.42	0.06	0.35	1.8	3.6
		61	278	0.795	0.25	0.04	0.19	0.6	3.3
D	214+50-CL	16	327	0.830	0.32	0.06	0.30	2.1	3.3
		26	317	0.820	0.38	0.06	0.35	2.1	2.9
D	220+34-CL	16	323	0.983	0.48	0.06	0.44	2.2	2.8
		32	307	0.950	0.52	0.06	0.40	2.2	3.2
		52	387	1.050	0.58	0.07	0.45	1.7	3.4
		72	267	0.580	0.27	0.03	0.23	2.1	4.7
D	235+46-2L	12	342	0.800	0.26	0.04	0.24	3.7	5.2
		32	322	0.920	0.43	0.05	0.39	2.5	3.5
		52	302	1.210	0.62	0.11	0.51	1.9	3.6
		71	283	0.815	0.25	0.04	0.21	1.2	2.4
D	251+97-CL	12	354	0.615	0.14	0.03	0.12	1.7	4.9
		32	334	0.845	0.83	0.07	0.42	3.6	5.2
		41	325	0.84	0.42	0.08	0.38	2.3	3.4

GEOCON

TABLE II (continued)

XVI.

CONSOLIDATION TESTS - SUMMARY OF RESULTS USED FOR COMPUTATIONS

<u>Line</u>	<u>Borehole Number</u>	<u>Sample Depth</u>	<u>Sample Elev'n</u>	<u>e<sub>o</sub></u>	<u>C<sub>c</sub></u>	<u>C<sub>R</sub></u>	<u>C<sub>c</sub>'</u>	<u>P<sub>c</sub> Min.</u>	<u>P<sub>c</sub> Max.</u>
D	251+97-CL	51	315	1.075	0.60	0.09	0.43	2.2	3.2
		59	307	0.770	0.40	0.06	0.33	2.4	3.2
D	257+20-10L	32	335	0.646	0.13	0.02	0.11	1.0	2.0
		42	325	0.853	0.53	0.06	0.46	3.0	4.4
		52	315	1.070	0.65	0.11	0.55	2.1	3.3
		61	306	0.853	0.35	0.06	0.30	1.5	2.7
D	262+32-40R	42	326	0.776	0.34	0.07	0.31	2.7	5.0
		62	306	0.904	0.39	0.06	0.35	2.2	4.5
B	227+04	46	293	0.693	0.24	0.04	0.20	1.0	2.4
B	231+16	31	310	0.840	0.42	0.06	0.41	2.5	3.5

In the above table

P<sub>c</sub> Min. = Computed Minimum possible preconsolidation pressure (tons per square foot)P<sub>c</sub> Max. = Computed Maximum preconsolidation pressure (defined as that obtained by Casagrande construction)e<sub>o</sub> = Initial void ratioC<sub>c</sub> = Probable Field Compression IndexC<sub>R</sub> = Rebound Compression IndexC<sub>c</sub>' = Laboratory Compression Index

Stiff to Firm Grey to Brown Clayey Silt

A stratum of clayey silt was found to underlie the reddish-brown and grey layered silty clay in all boreholes except boreholes #256+93-CL, #257+20-10L and #262+32-40R where it underlies the grey silty clay. It was not encountered in borehole #251+97-CL. The thickness of this stratum ranged from 4 to 28 feet. The clayey silt generally grades into a silt towards the bottom of the stratum. It contains scattered subangular pebble gravel to about 3/8 inch size and dispersed pockets or lenses to about 3/4 inch size. These pockets or lenses are generally grey or reddish-brown in colour and are composed of silt at the top of the stratum and fine sand towards the bottom. The colour of the stratum generally changes from grey to brown with increasing depth.

Liquid limit determinations for the stratum gave results ranging from about 29 to 43 with an average of 35. The plasticity index varied generally from 11 to 22 and has an average value of 16. Typical values of the Atterberg limits are plotted on the plasticity chart on Figure 4 of Appendix II together with the Casagrande "A" line. The stratum has been classified as an inorganic clayey silt of medium plasticity, because of its trend to silt with depth.

Two typical grain size distribution curves for the stratum are shown on Figure 1 of Appendix II. These show that the clayey silt is composed of from 20 to 38 per cent clay sizes, the remaining sizes being in the silt range.

The Liquidity Index for this stratum has an average value of 0.44.



Stiff to Firm Grey to Brown Clayey Silt (continued)

The shear strength of the clayey silt was determined by in-situ vane testing and undrained triaxial compression tests. As with the two overlying strata, the vane testing gave generally lower results than the undrained triaxial compression tests and the latter values have been used as a basis for design. The shear strength ranged generally from 520 to 3000 pounds per square foot with an average value of 1300 pounds per square foot. It is estimated from these values and from one standard penetration test which gave an "N" value of 19 that the consistency of the stratum is generally stiff to firm.

The wet unit weight of the clayey silt varied between 116 and 142 pounds per cubic foot with an average of 127 pounds per cubic foot. The natural moisture content ranged from 10 to 37 per cent with an average of 26 per cent.

Three consolidation tests were carried out on samples of the stratum and the resulting log pressure-void ratio curves are shown on Figures 15, 20 and 23 in Appendix II together with a plot of the computed coefficient of consolidation,  $C_v$ , against pressure for each test. The probable field consolidation curves obtained by the Schmertmann method are also shown. An average compression index,  $C_c$ , of 0.29 and an average rebound compression index,  $C_R$ , of 0.04 was obtained.

Very Dense Reddish-Brown Silty and Sandy Till

Beneath the grey to brown clayey silt a stratum of reddish-brown silty and sandy till was encountered in all boreholes. In borehole #251+97-CL the silty and sandy till was found to underlie the layered silty clay. The maximum depth penetrated was 58 feet

Very Dense Reddish-Brown Silty and Sandy Till (continued)

in borehole #185+00-61L. The stratum is essentially composed of subangular sand and gravel sizes in a reddish-brown or brown silt or sandy silt matrix. The relative proportions of sand, silt and gravel sizes vary slightly throughout the stratum. Occasional pockets or lenses of fine to medium sand up to several feet in thickness were encountered within the stratum. Three typical grain size distribution curves of the till are shown on Figure 2 of Appendix II.

The average liquid limit is about 20 and the average plasticity index about 5.

Standard penetration tests gave "N" values which varied generally between 50 and greater than 100 blows per foot with an average of greater than 100 blows per foot. The relative density is thus estimated to be very dense. The boundary between the grey to brown clayey silt and this stratum is well defined by a sudden increase in relative density.

The shear strength, as determined by undrained triaxial compression tests, ranged from about 8000 to 20000 pounds per square foot with an average value of 11000 pounds per square foot. This confirms the estimate of the relative density obtained from the "N" values.

The average wet unit weight of the stratum was found to be 145 pounds per cubic foot and the average natural moisture content 12 per cent.

Bedrock

Bedrock was found to underlie the reddish-brown silty and sandy till in the 16 boreholes where it was penetrated. The rock was core drilled in AXT and BXT sizes to a maximum depth of 25.5 feet in borehole #230+26-11R. The surface of the rock slopes from the east towards the west along the profile at an average slope of about 1 to 120. Bedrock is generally a sound reddish-brown Queenston shale with grey calcareous bands. The upper 1 to 2 feet of the bedrock is generally weathered.

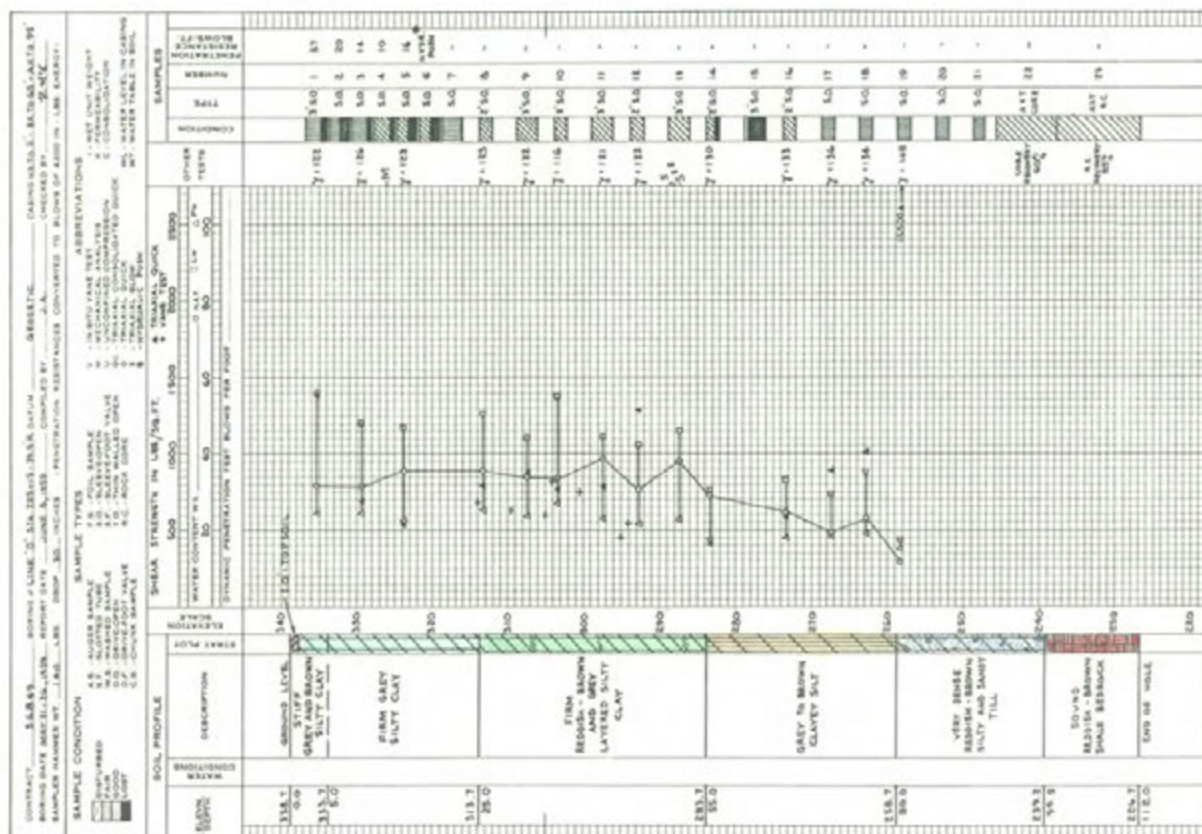
WATER CONDITIONS

Groundwater level observations were carried out in all boreholes and in 10 standpipes and 5 piezometers installed along Line "D". Generally the water level appears to be at or within 15 feet of the ground surface. Due to the very low permeability of the clay strata, and the influence of surface water, it was difficult to determine the groundwater level accurately. The stabilized water level readings in the standpipes and piezometers during September, 1959, are plotted on Drawing S6849-2 in Appendix VI.

The level of the Welland Canal has generally ranged between about elevations 335.0 and 336.5.



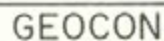
APPENDIX I  
FIGURE I  
PROJECT-S6849



GEOCON



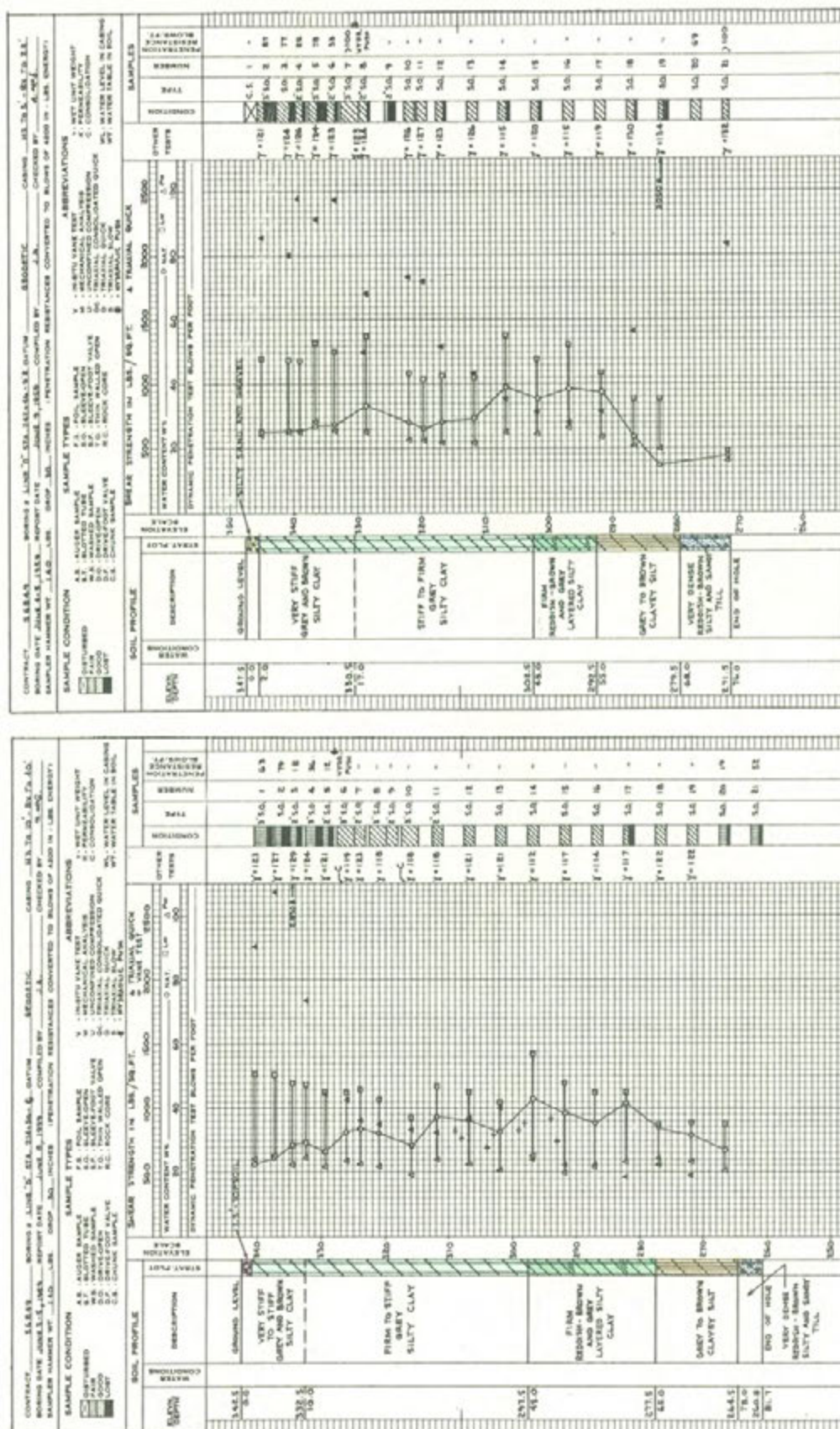
APPENDIX 1  
FIGURE 2  
PROJECT-S6849





# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 3  
PROJECT-S6849





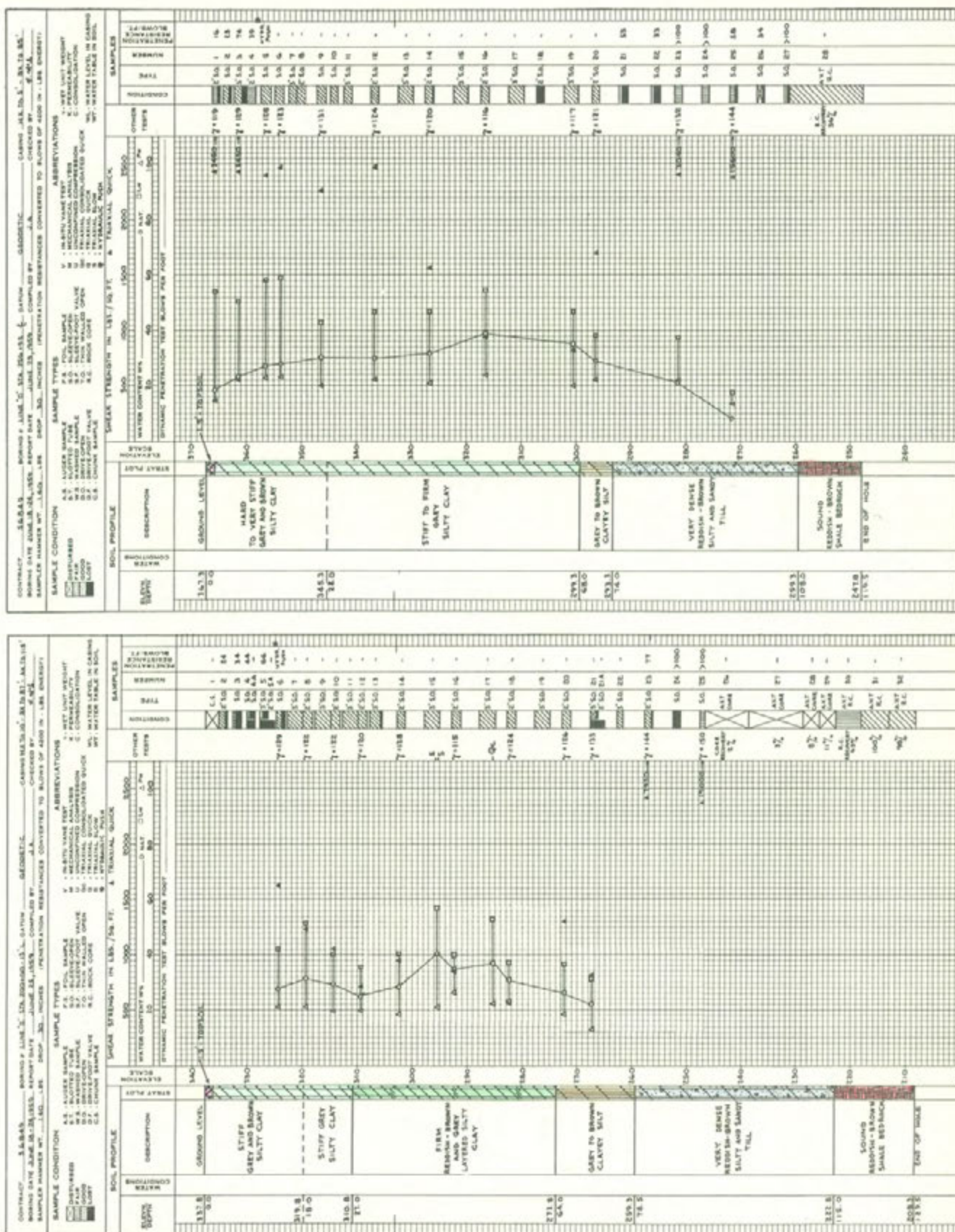








APPENDIX 1  
FIGURE 6  
PROJECT-S6849



GEOCON



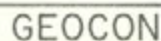








APPENDIX 1  
FIGURE 9  
PROJECT-S6849









APPENDIX I  
FIGURE II  
PROJECT S6849

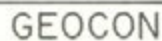








APPENDIX 1  
FIGURE 13  
PROJECT-S6849

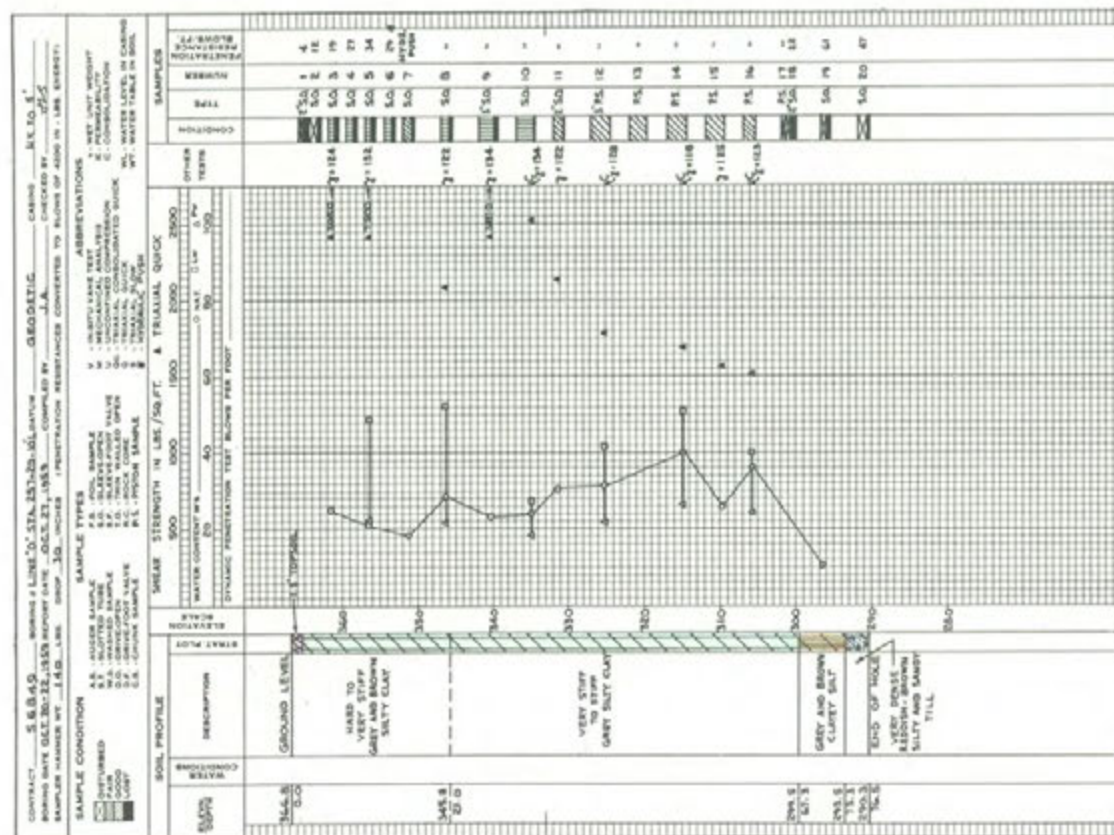


APPENDIX I  
FIGURE 14  
PROJECT-S6849





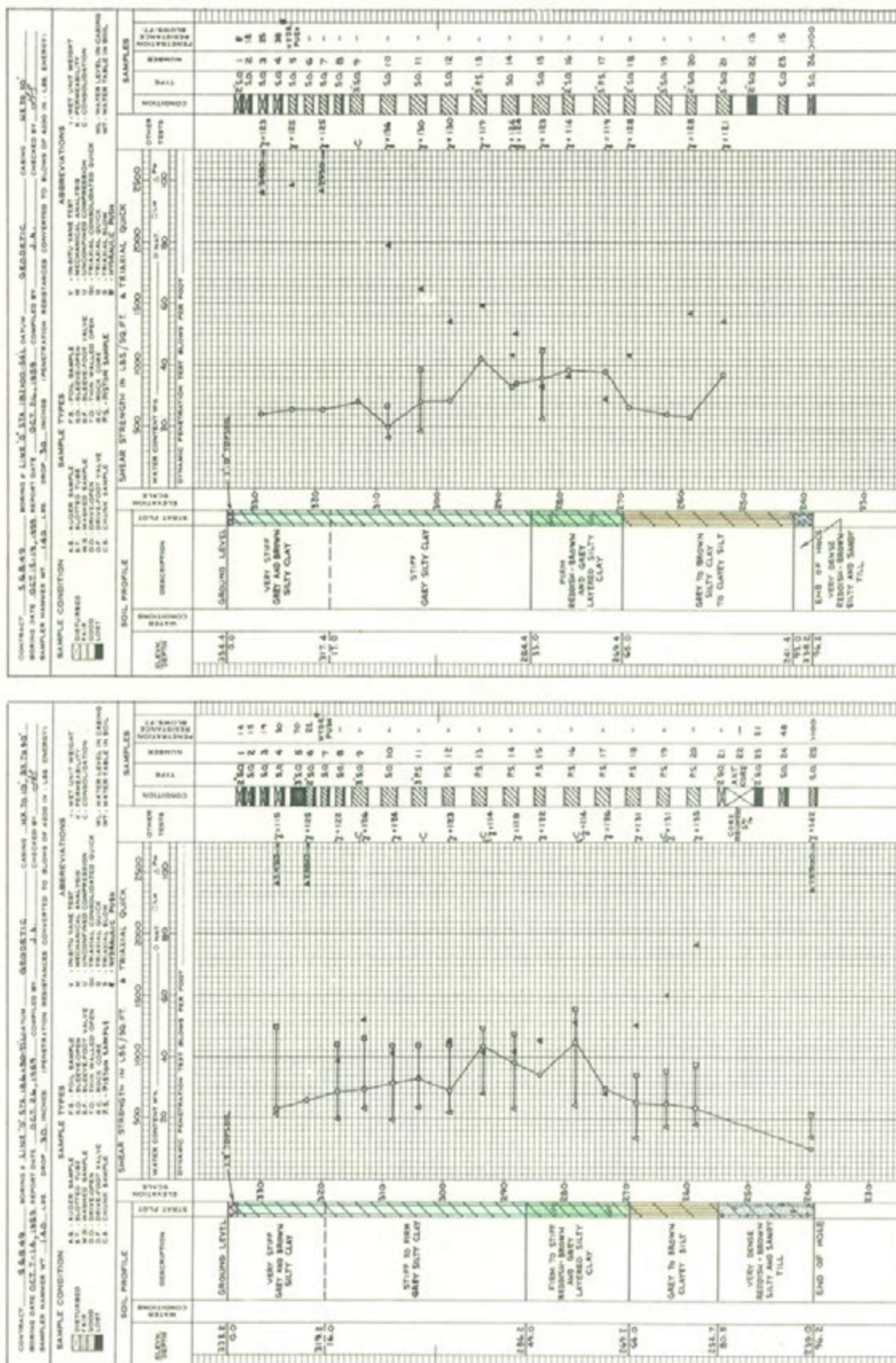
APPENDIX I  
FIGURE 15  
PROJECT-S6849



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APPENDIX 1  
FIGURE 16  
PROJECT-S6849



GEOCON

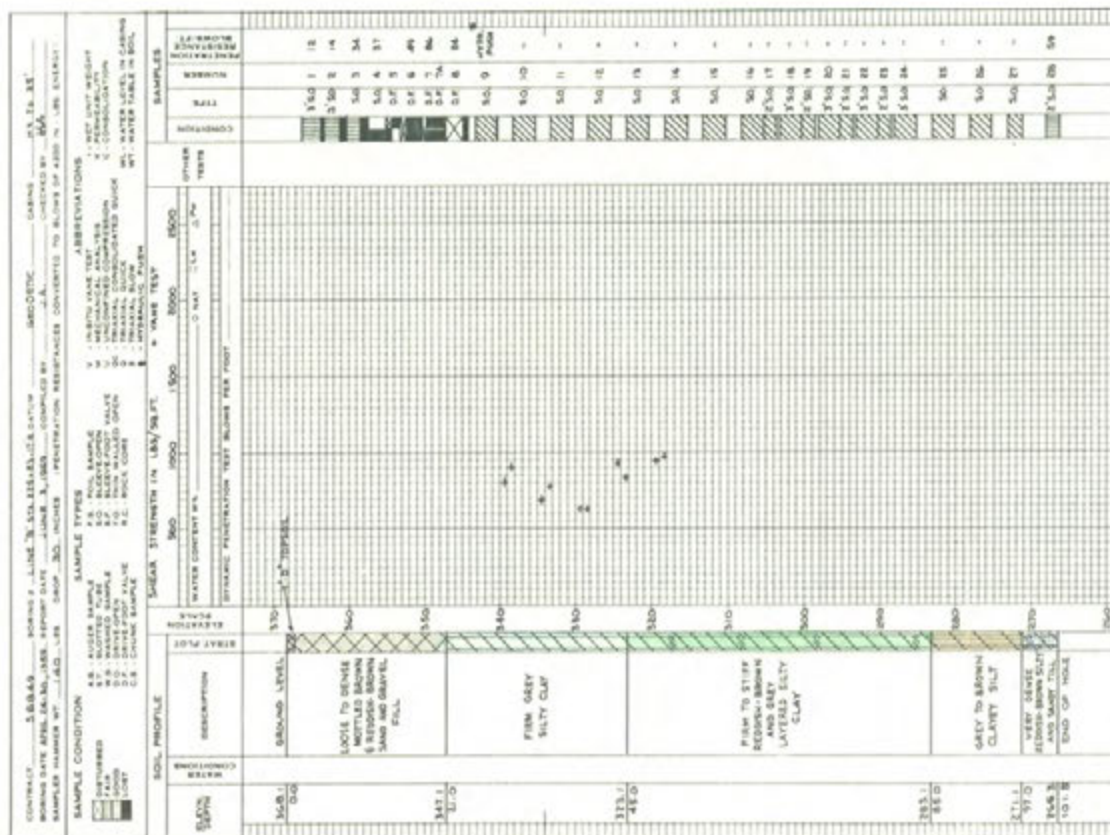


APPENDIX 1  
FIGURE 17  
PROJECT-S6849





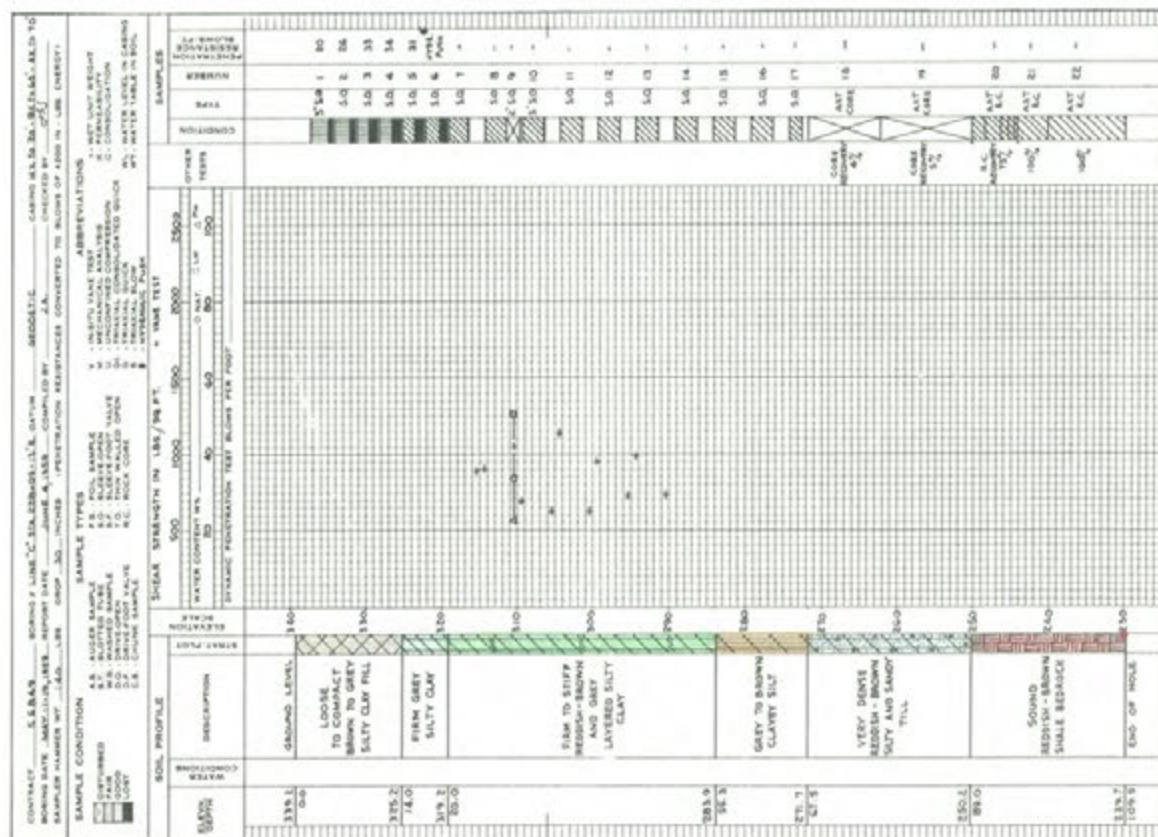
APPENDIX 1  
FIGURE 18  
PROJECT-S6849



GEOCON



APPENDIX 1  
FIGURE 19  
PROJECT-S6849



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## EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

### ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

### WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

### DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

<u>Consistency</u>	<u>U-Strength Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration Resistance. Blows/ft.</u>
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

### STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

### ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

### GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

### OTHER TESTS

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

### SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

**GEOCON**

OFFICE REPORT ON SOIL EXPLORATION INDEX

LINE "D"

<u>BOREHOLE</u>	<u>FIGURE NO.</u>
182+00 54' Left	16
185+00 61' Left	13
186+50 53' Left	16
190+00 65' Left	8
195+00 32' Left	7
200+00 13' Left	6
202+00 Centre Line	15
203+70 37' Left	5
209+68 24.5' Right	4
214+50 Centre Line	3
216+10 Centre Line	12
218+45 75' Left	11
220+34 Centre Line	10
222+00 Centre Line	14
223+14 2' Left	9
224+22 5.5' Left	2
225+15 39.5' Right	1
230+26 11' Right	1
231+01 8' Left	2
231+96 Centre Line	14
232+11 11' Right	8
235+46 2' Left	9
238+34 3' Left	10
242+46 9' Right	3
244+48 Centre Line	11
246+80 25' Right	4
251+97 Centre Line	5
256+93 Centre Line	6
257+20 10' Left	15
262+32 40' Right	7
267+48 23' Right	12

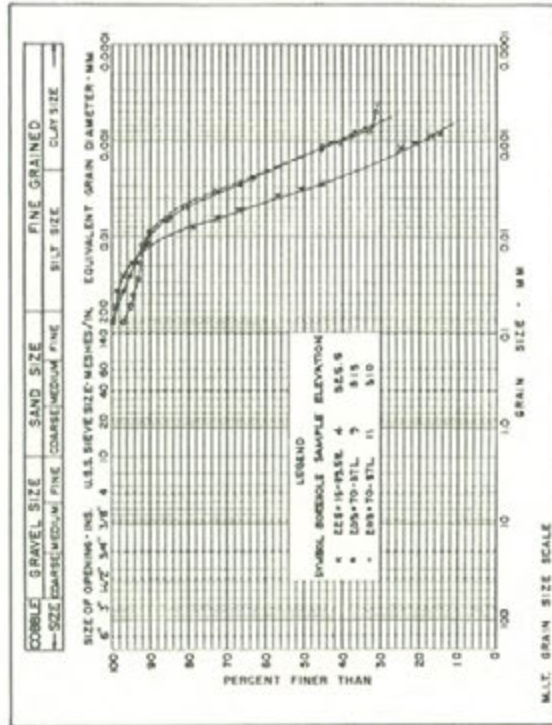


APPENDIX II

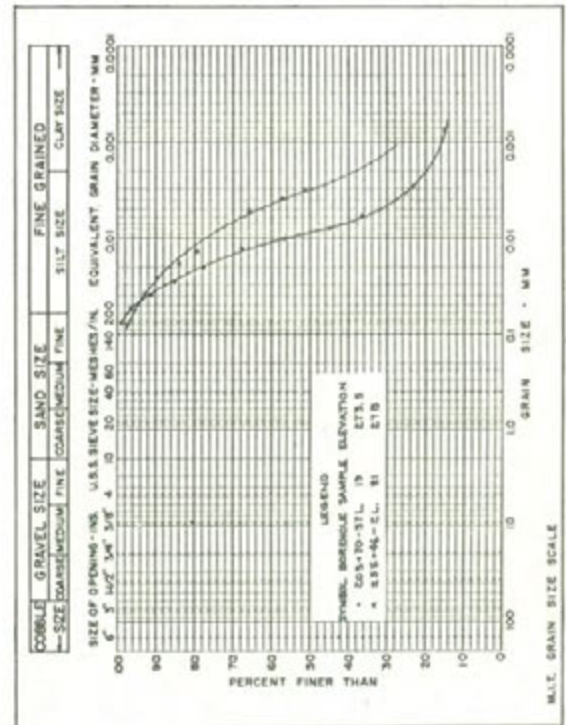
FIGURES ON LABORATORY TESTING

# GRAIN SIZE DISTRIBUTION

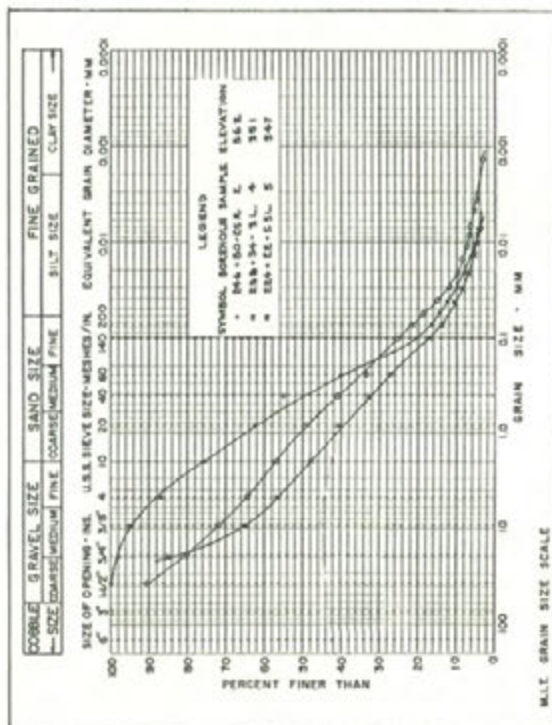
APPENDIX II  
FIGURE I  
PROJECT S6849



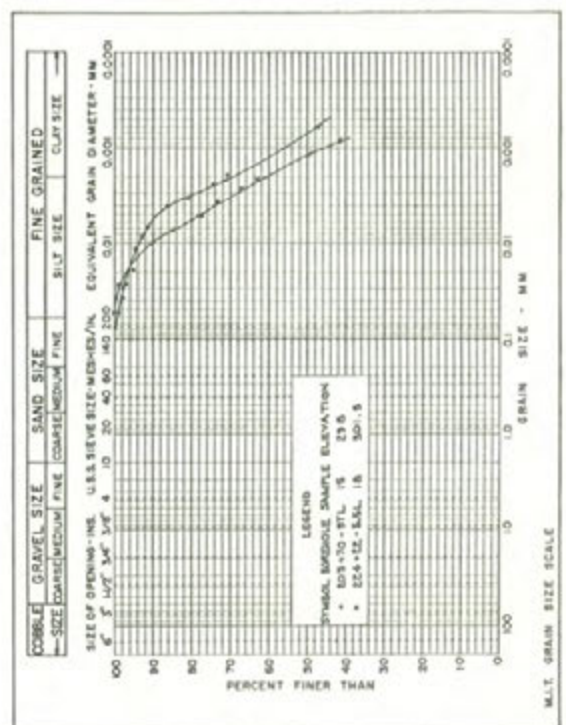
GREY SILTY CLAY



GREY TO BROWN CLAYEY SILT



BROWN SILTY SAND & GRAVEL

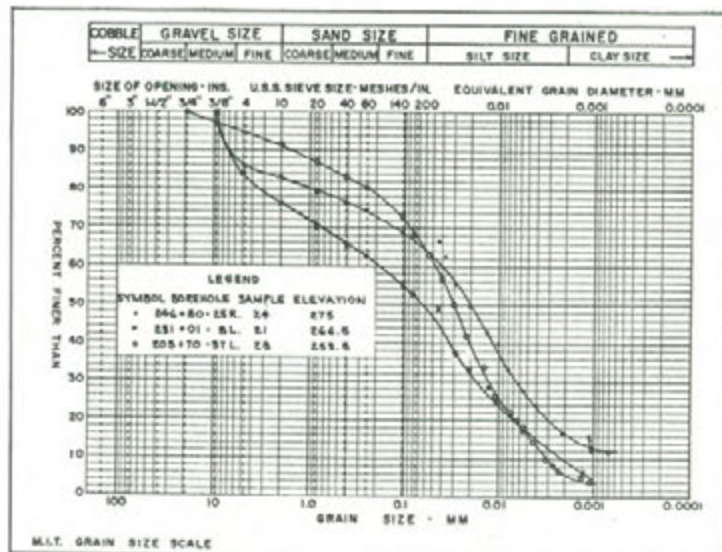


LAYERED SILTY CLAY



# GRAIN SIZE DISTRIBUTION

APPENDIX II  
FIGURE 2  
PROJECT S6849

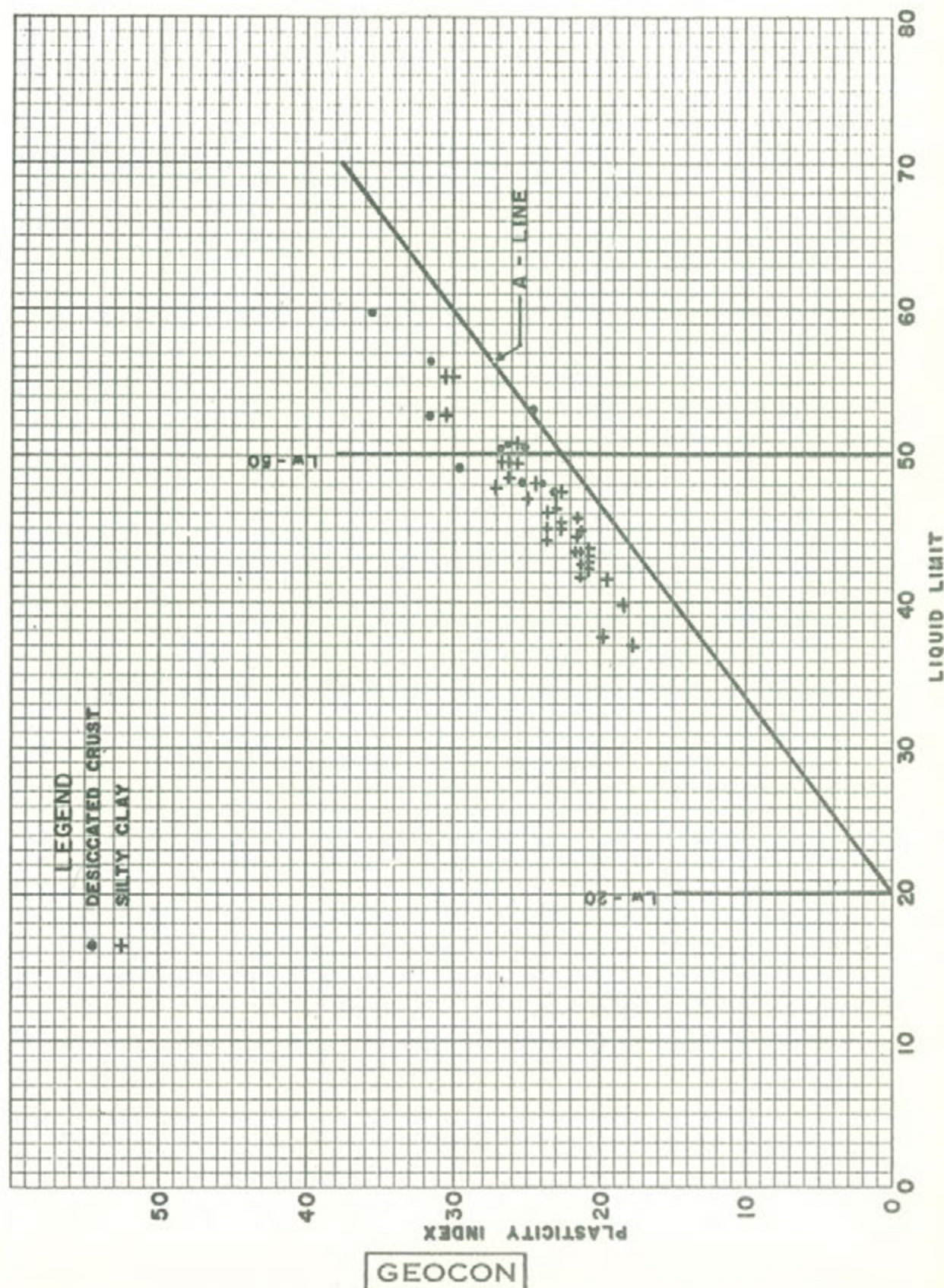


VERY DENSE SILTY & SANDY TILL

# PLASTICITY CHART

## TYPICAL RESULTS

APPENDIX II  
FIGURE 3  
PROJECT - S6849

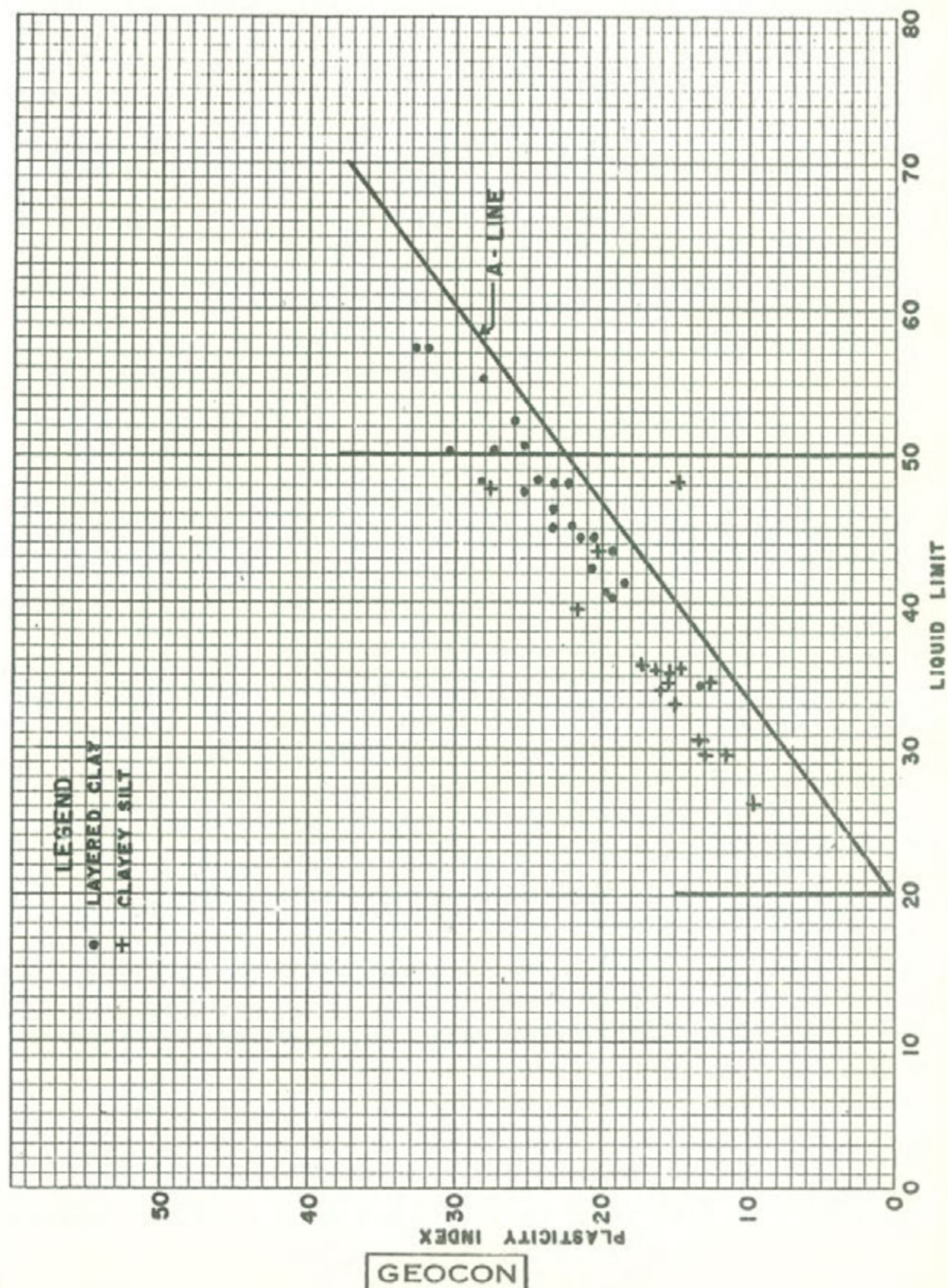




# PLASTICITY CHART

## TYPICAL RESULTS

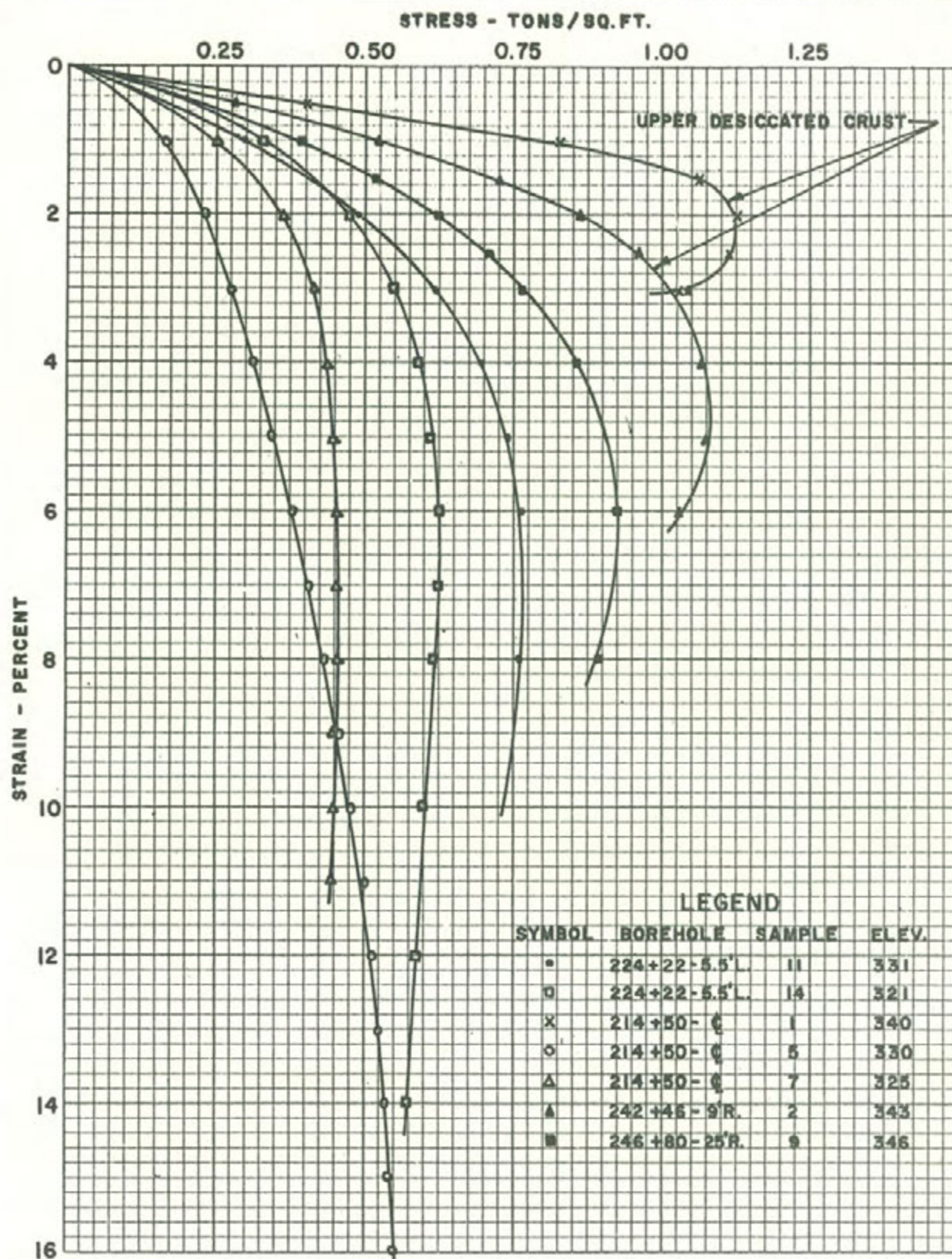
APPENDIX II  
FIGURE 4  
PROJECT - S6849





UNDRAINED TRIAXIAL COMPRESSION TESTS  
TYPICAL STRESS-STRAIN CURVES  
GREY SILTY CLAY

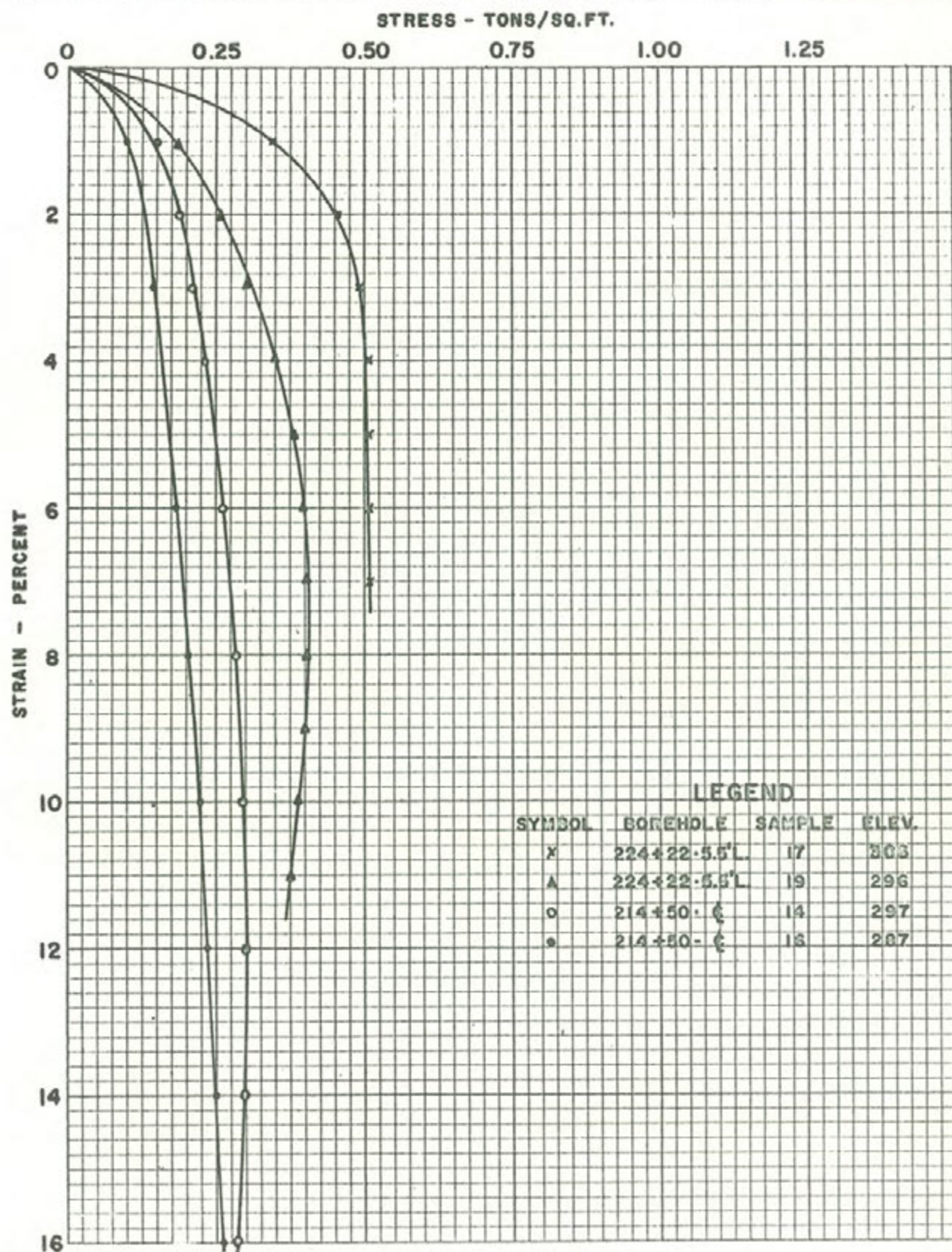
APPENDIX II  
FIGURE 5  
PROJECT - S6849





UNDRAINED TRIAXIAL COMPRESSION TESTS  
TYPICAL STRESS-STRAIN CURVES  
LAYERED SILTY CLAY

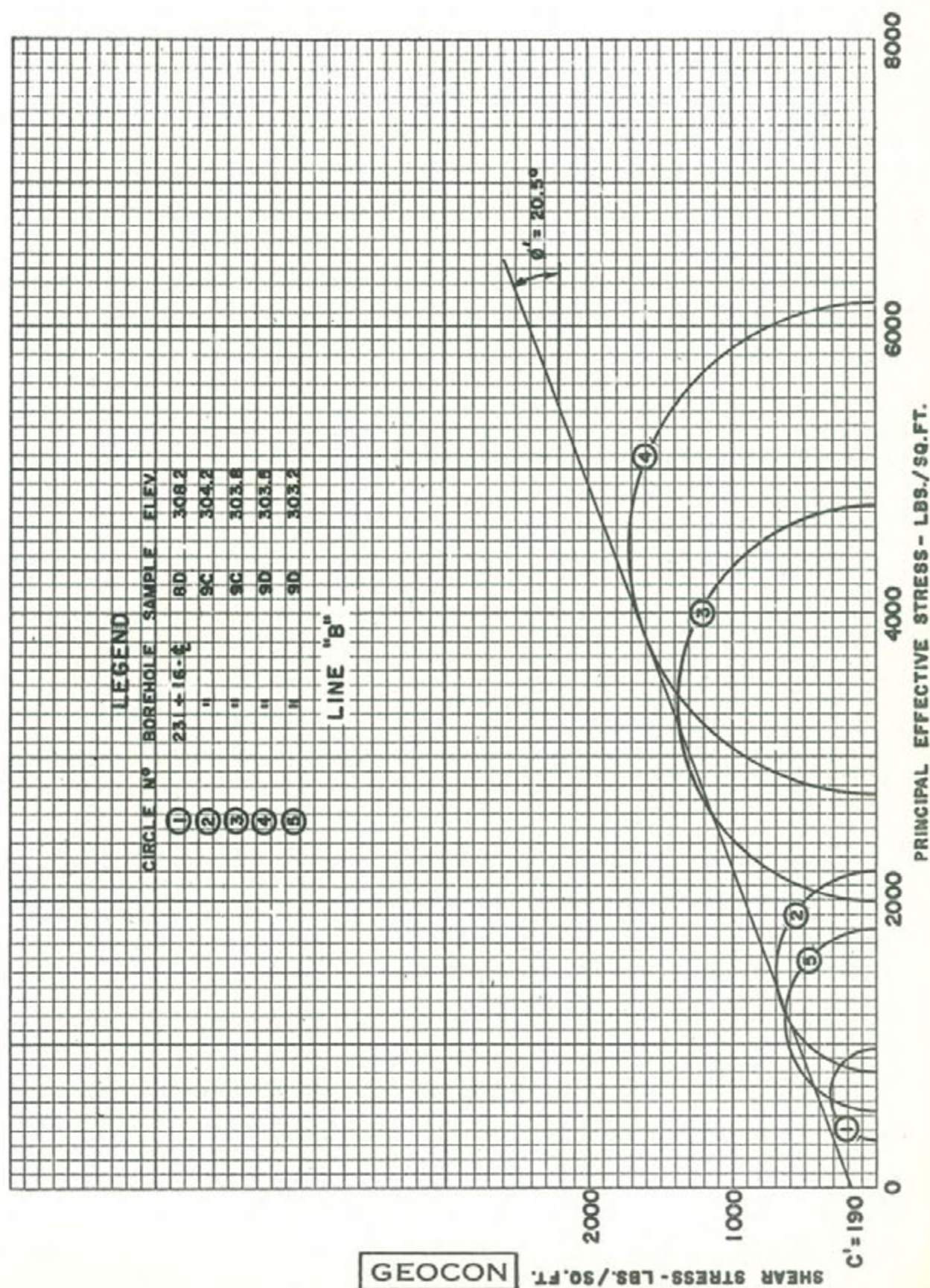
APPENDIX II  
FIGURE 6  
PROJECT - S6849





CONSOLIDATED TRIAXIAL COMPRESSION TESTS  
WITH PORE PRESSURE MEASUREMENTS  
MOHR'S CIRCLES  
LAYERED SILTY CLAY

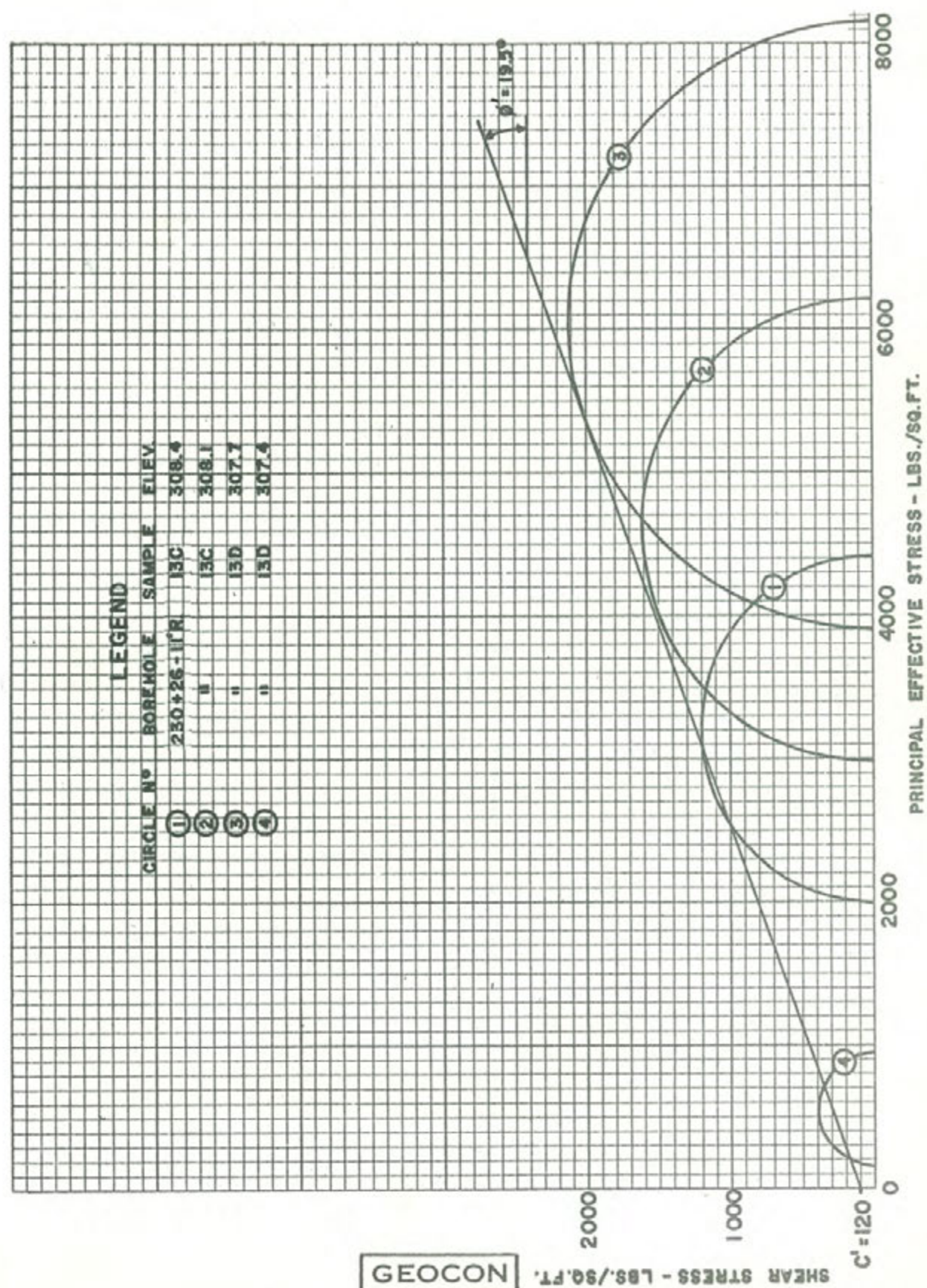
APPENDIX II  
FIGURE 7  
PROJECT - S6849





CONSOLIDATED TRIAXIAL COMPRESSION TESTS  
WITH PORE PRESSURE MEASUREMENTS  
MOHR'S CIRCLES  
GREY SILTY CLAY

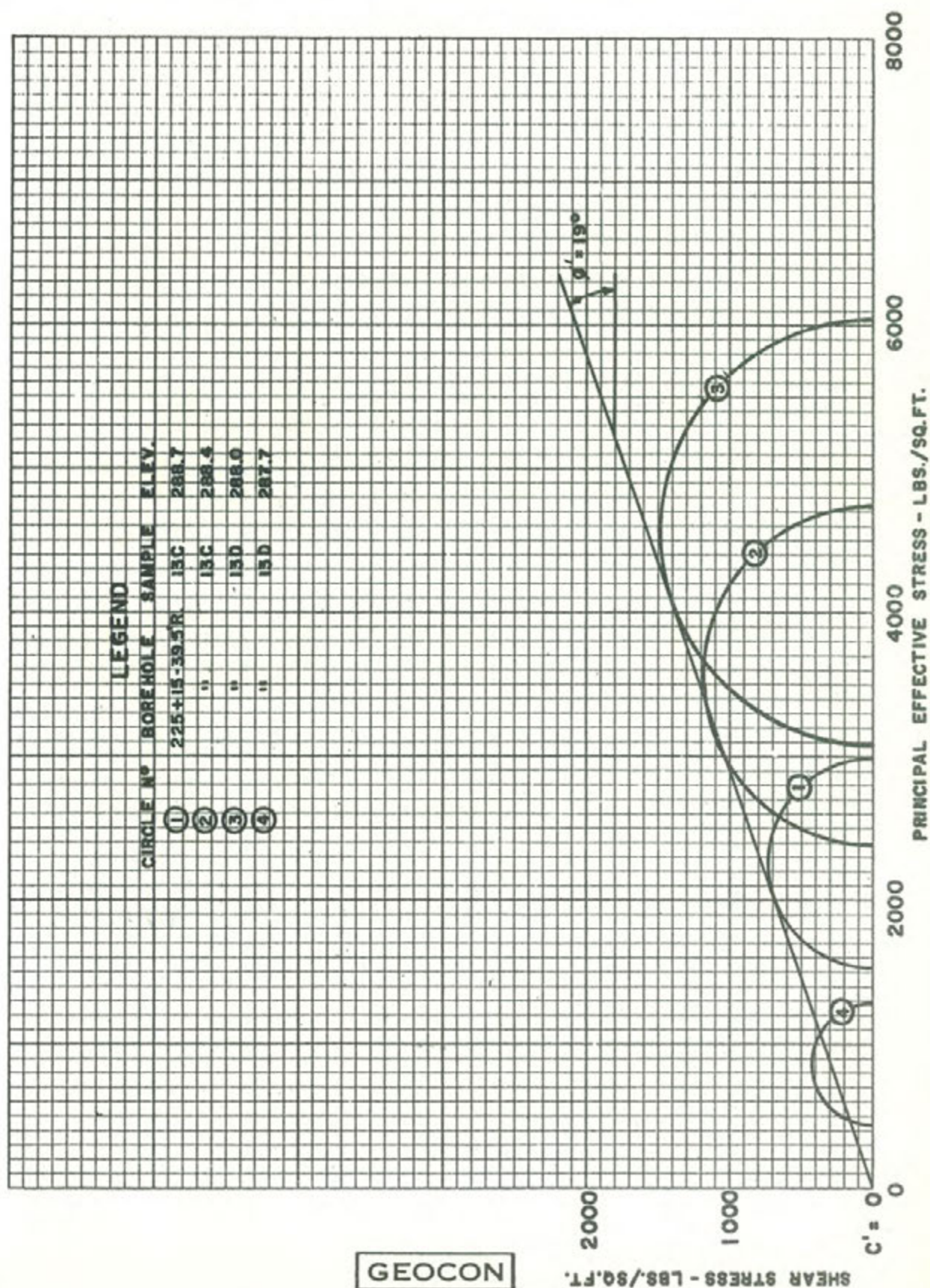
APPENDIX II  
FIGURE 8  
PROJECT - S6849





CONSOLIDATED TRIAXIAL COMPRESSION TESTS  
WITH PORE PRESSURE MEASUREMENTS  
MOHR'S CIRCLES  
LAYERED SILTY CLAY

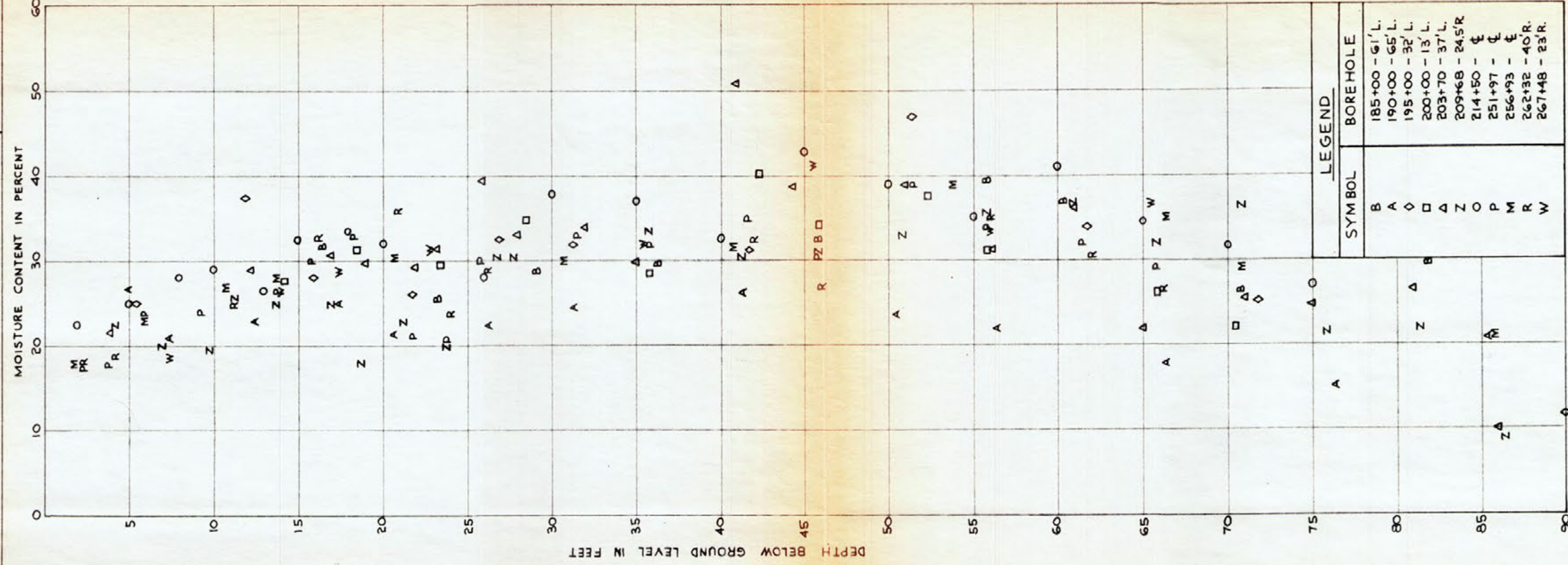
APPENDIX II  
FIGURE 9  
PROJECT - S6849





# MOISTURE CONTENT VS DEPTH EMBANKMENT AND SPREAD FOOTING AREAS

APPENDIX II  
FIGURE 10  
PROJECT - S6849

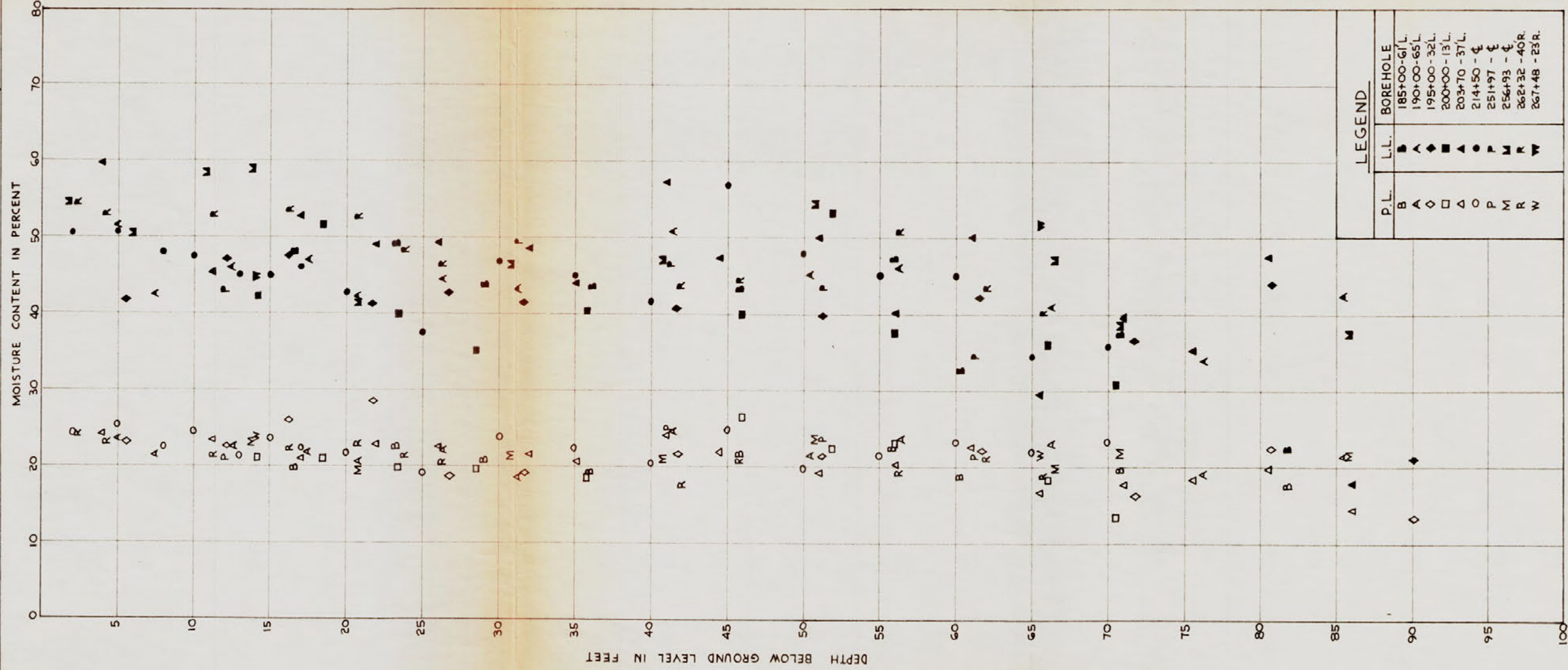


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LIQUID AND PLASTIC LIMITS VS DEPTH  
EMBANKMENT AND SPREAD FOOTING AREAS

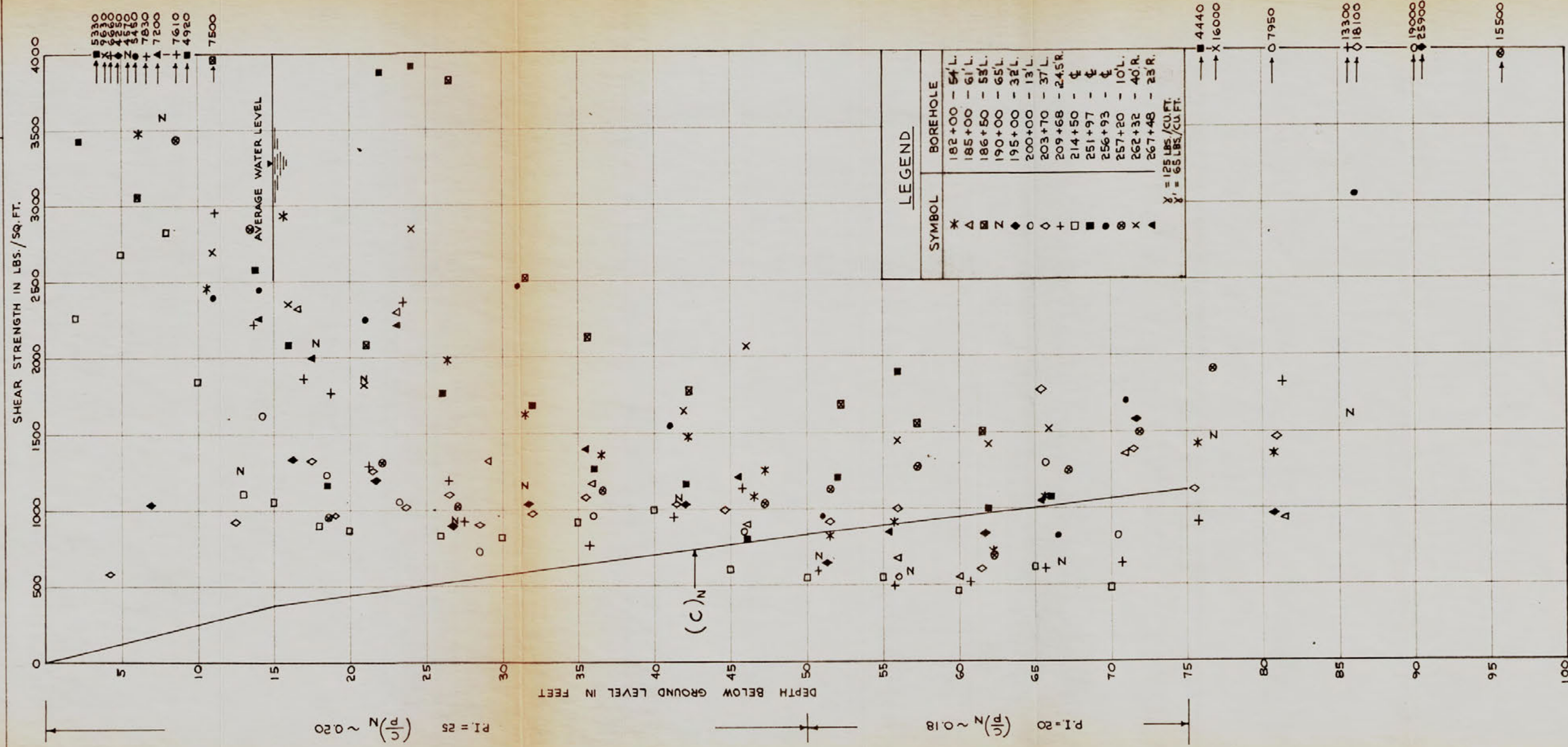
APPENDIX II  
FIGURE II  
PROJECT - S6849





# SHEAR STRENGTH VS DEPTH EMBANKMENT AND SPREAD FOOTING AREAS

APPENDIX II  
FIGURE 12  
PROJECT - S6849









# CONSOLIDATION TESTS

## INDEX

<u>Line</u>	<u>Borehole</u>	<u>Sample Elevation</u>	<u>Stratum</u>	<u>Figure No.</u>
D	182+00-54L	313	Silty Clay	14
D	186+50-53L	314	Silty Clay	14
		303	Silty Clay	14
		293	Silty Clay	14
		278	Layered Silty Clay	15
		264	Clayey Silt	15
D	202+00-CL	318	Silty Clay	15
		308	Silty Clay	15
		303	Layered Silty Clay	16
		293	Layered Silty Clay	16
		283	Layered Silty Clay	16
		274	Layered Silty Clay	16
D	203+70-37L	322	Silty Clay	17
		307	Silty Clay	17
		288	Layered Silty Clay	17
		278	Layered Silty Clay	17
D	214+50-CL	327	Silty Clay	18
		317	Silty Clay	18
D	220+34-CL	323	Silty Clay	18
		307	Layered Silty Clay	18
		287	Layered Silty Clay	19
		267	Layered Silty Clay	19
D	235+46-2L	342	Silty Clay	19
		322	Silty Clay	19
		302	Silty Clay	20
		283	Clayey Silt	20
D	251+97-CL	354	Silty Clay	20
		334	Silty Clay	20

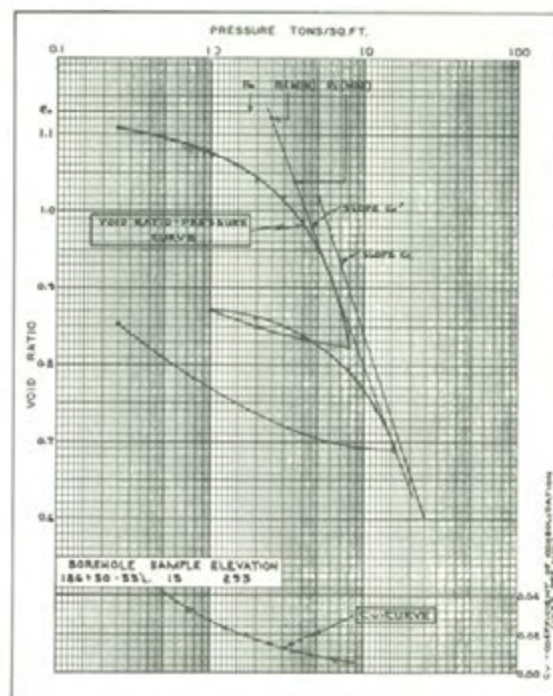
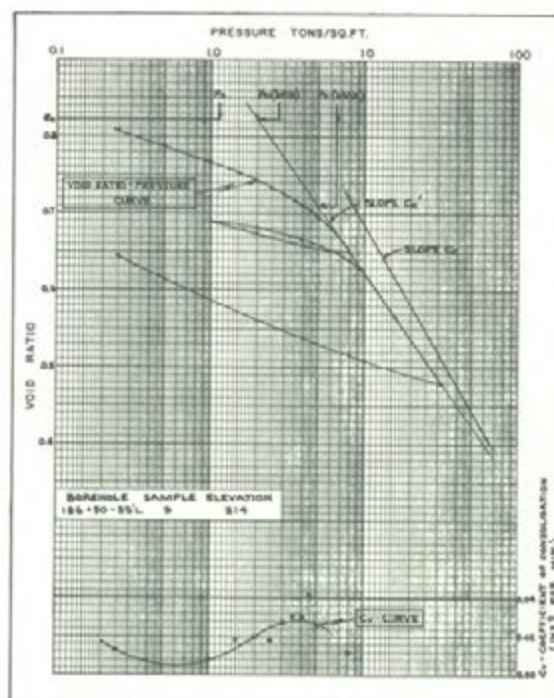
## CONSOLIDATION TESTS

### LEGEND

- $P_o$  - Computed Existing Overburden Pressure
- $P_c(\text{Min.})$  - Computed Minimum Preconsolidation Pressure
- $P_c(\text{Max.})$  - Computed Maximum Preconsolidation Pressure  
(defined as that obtained from the Casagrande construction)
- $C_c$  - Probable Field Compression Index
- $C_c'$  - Laboratory Compression Index
- $C_v$  - Computed Coefficient of Consolidation



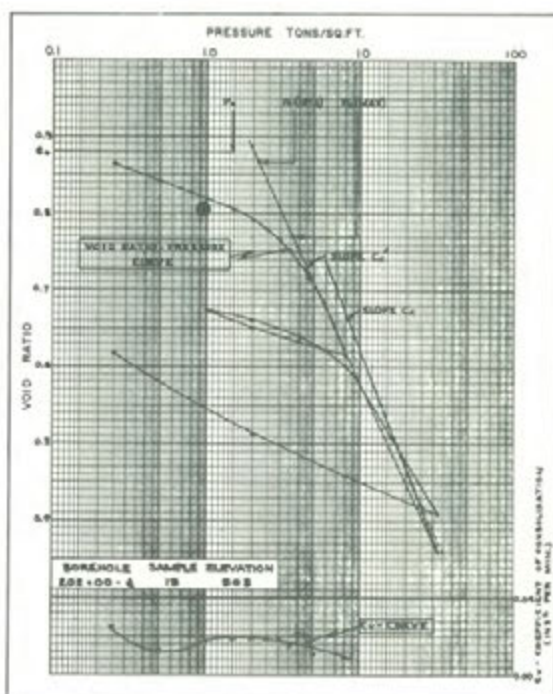
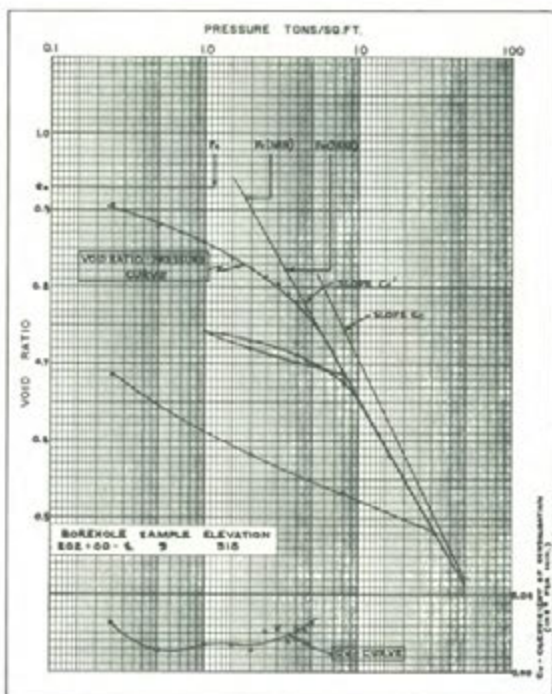
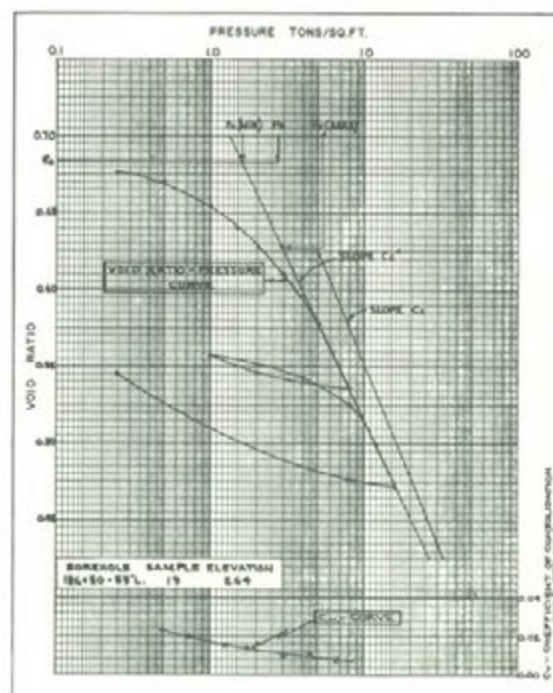
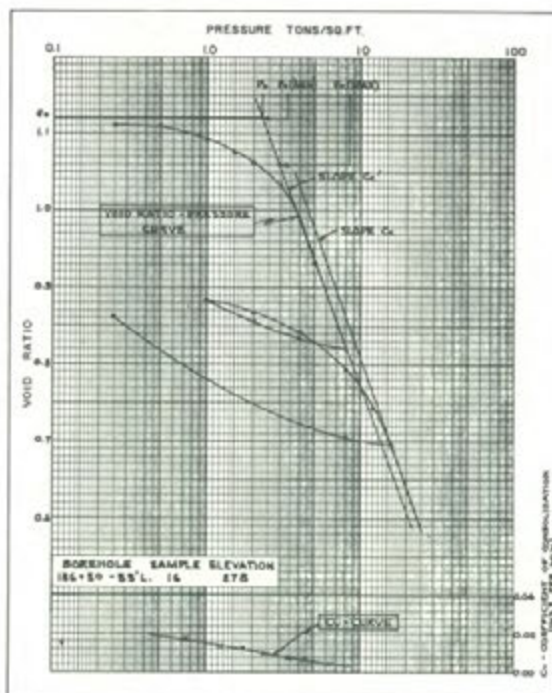
APPENDIX II  
FIGURE 14  
PROJECT S6849



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# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

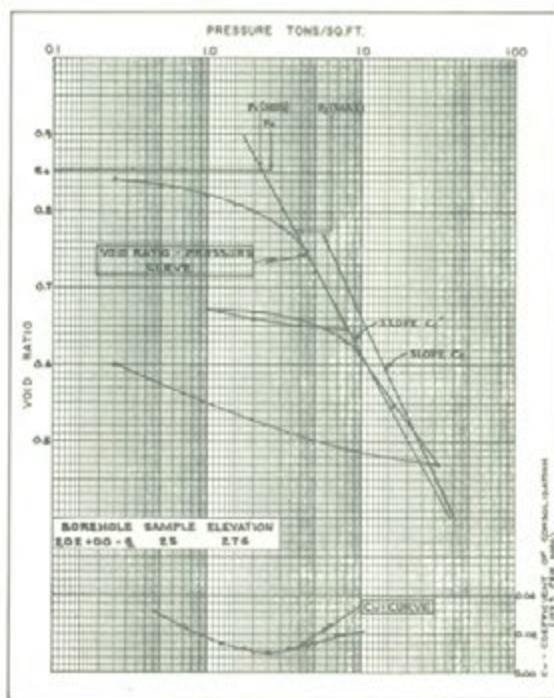
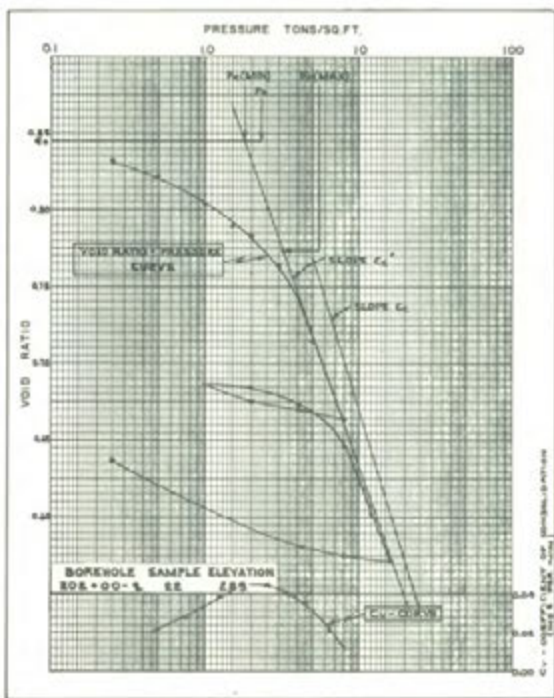
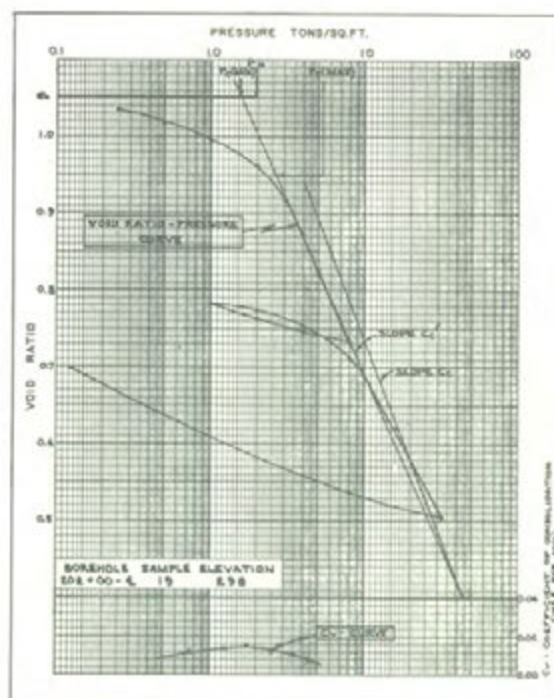
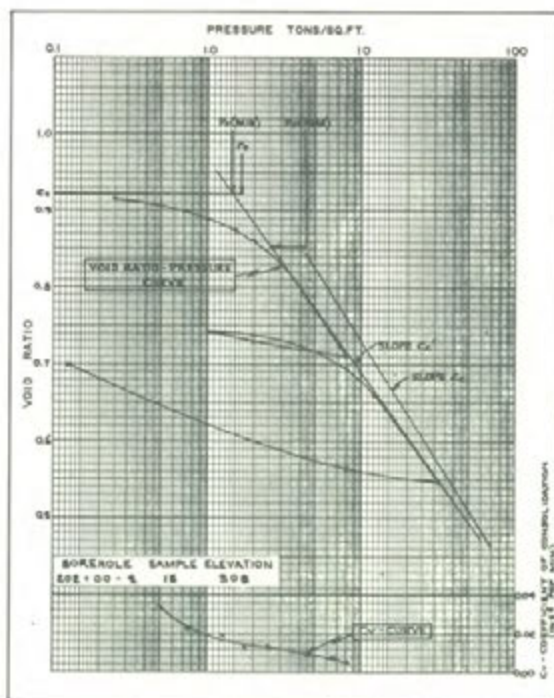
APPENDIX II  
FIGURE 15  
PROJECT S6849





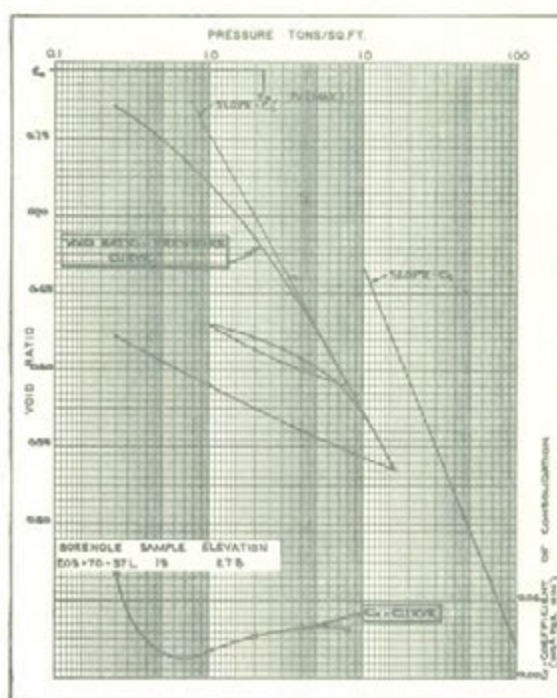
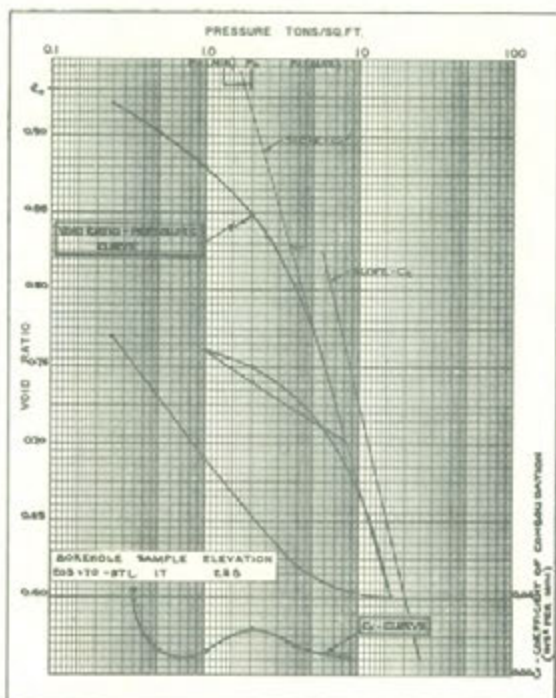
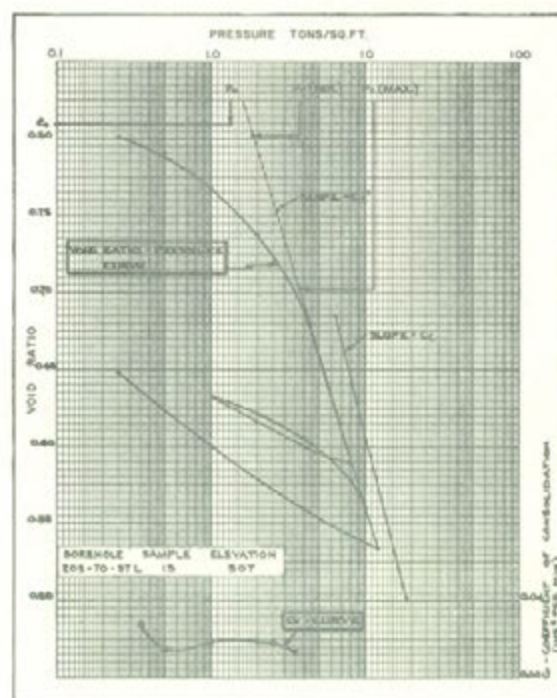
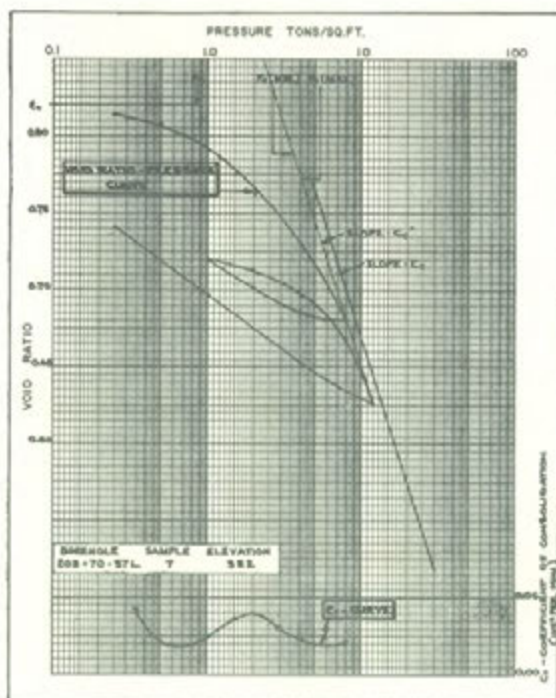
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 16  
PROJECT S6849



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

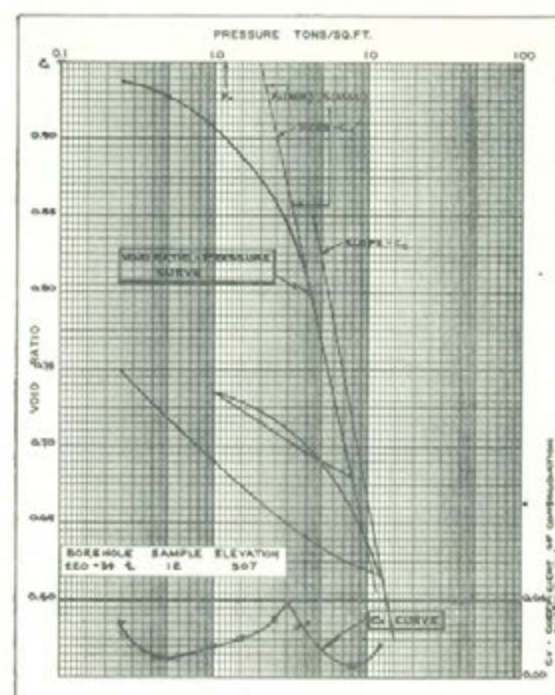
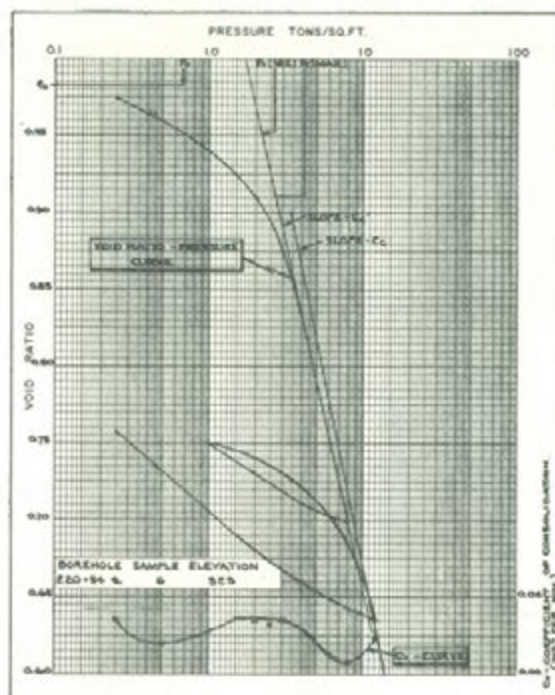
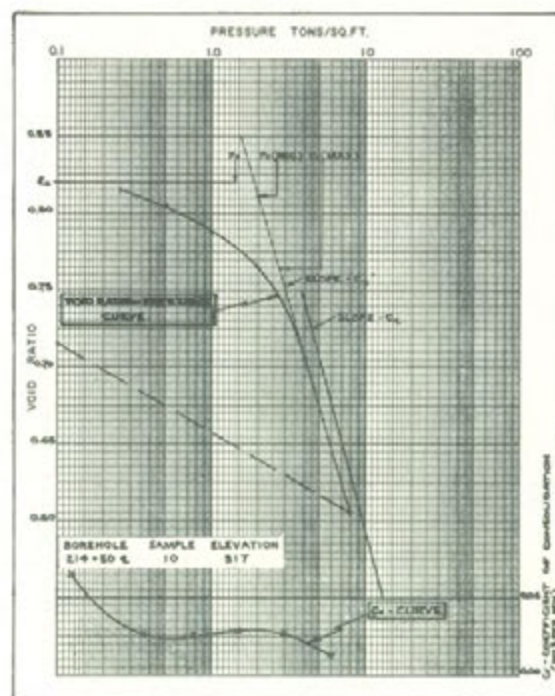
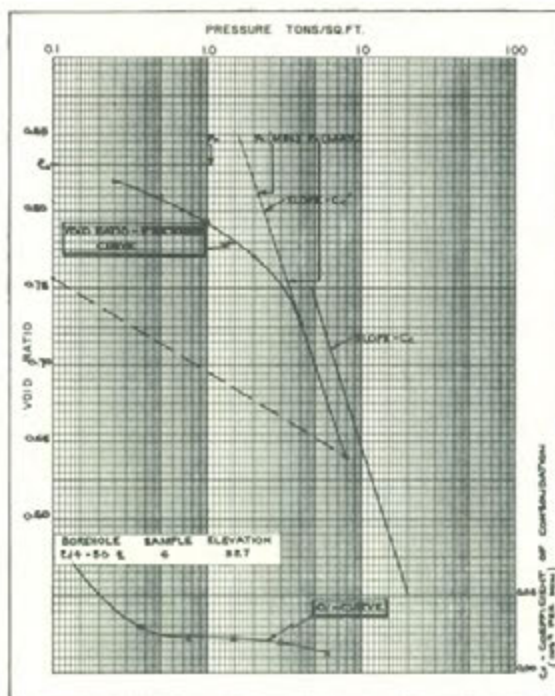
APPENDIX II  
FIGURE 17  
PROJECT S6849





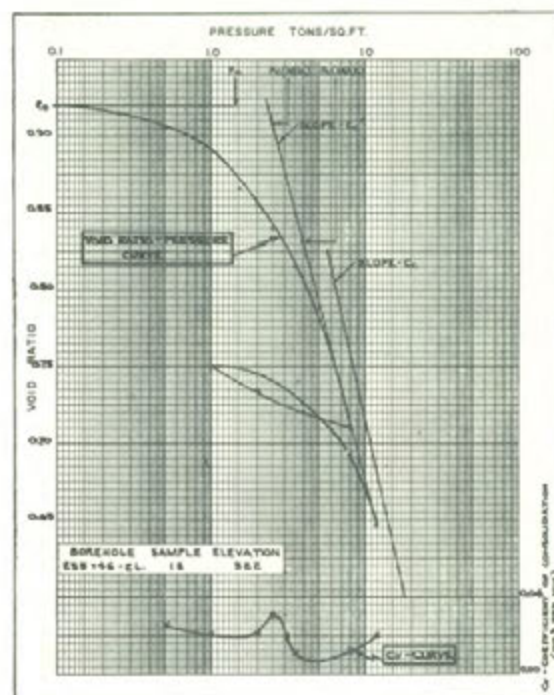
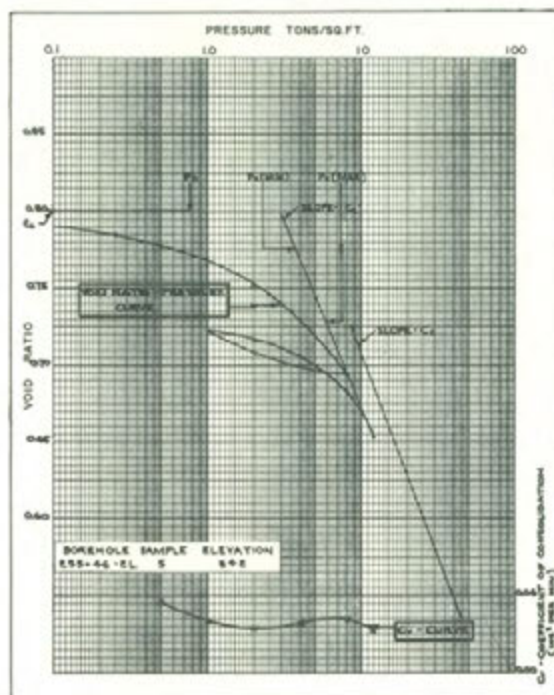
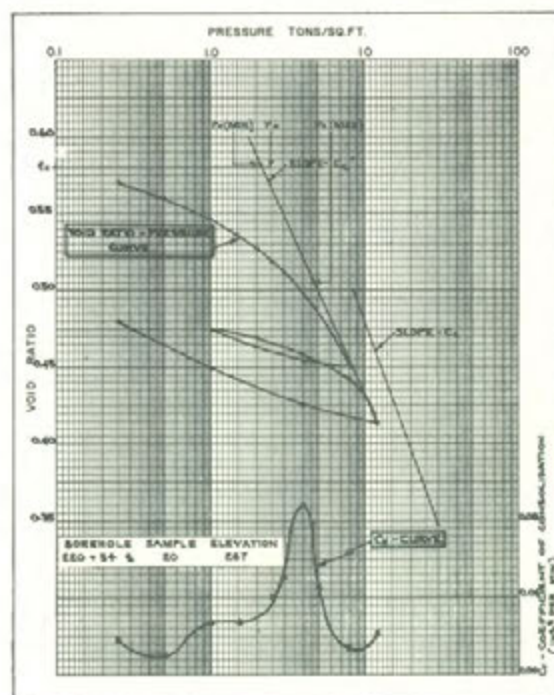
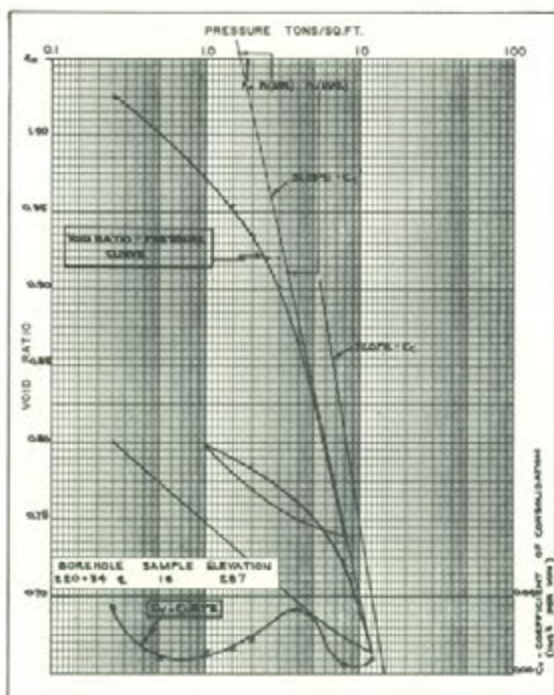
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 18  
PROJECT S6849



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

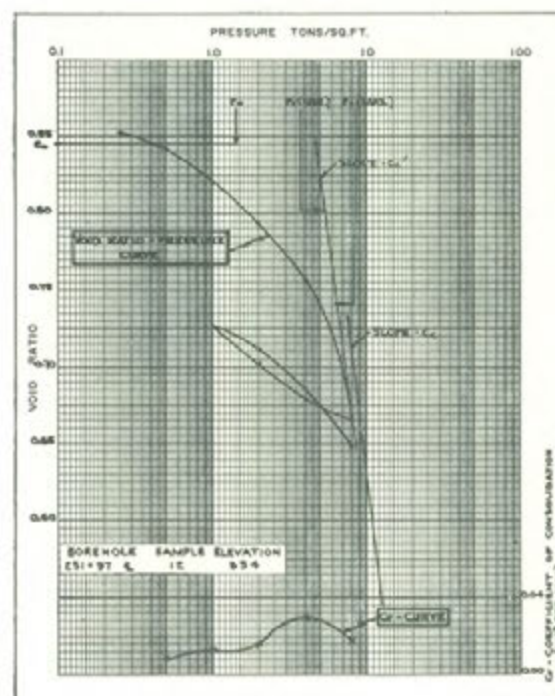
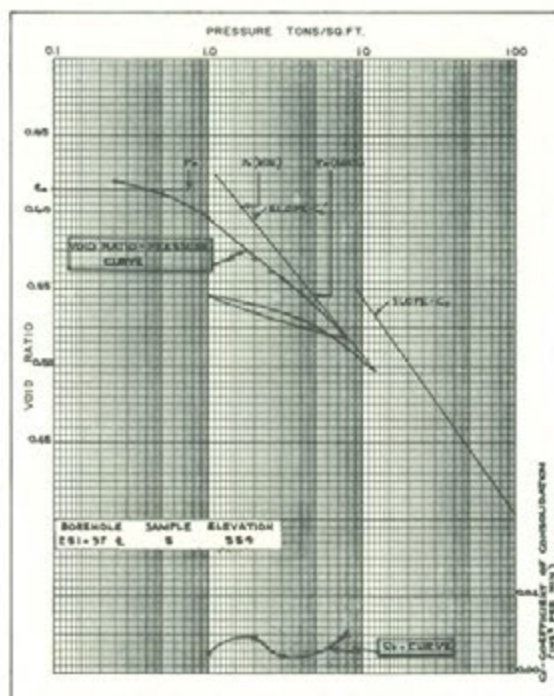
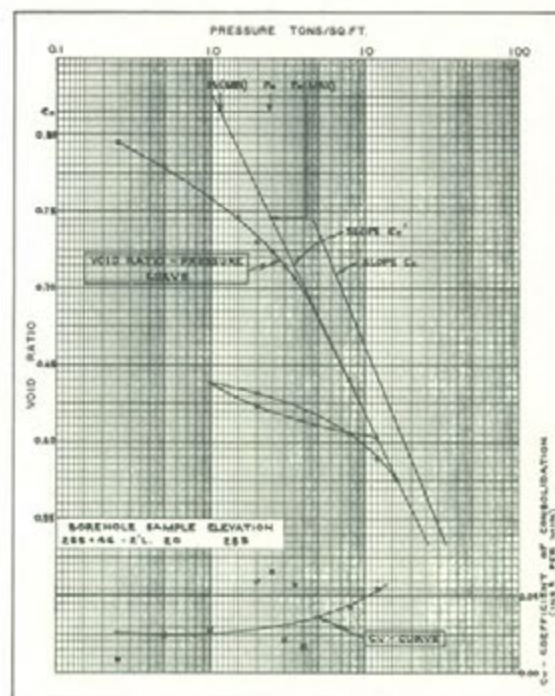
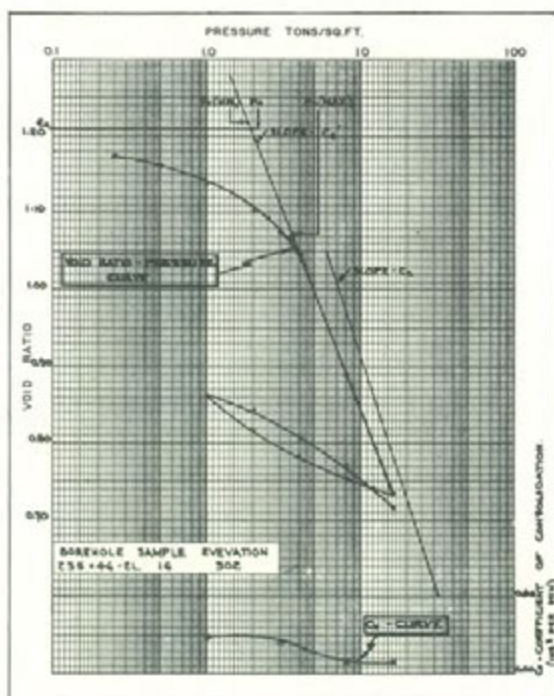
APPENDIX II  
FIGURE 19  
PROJECT S6849



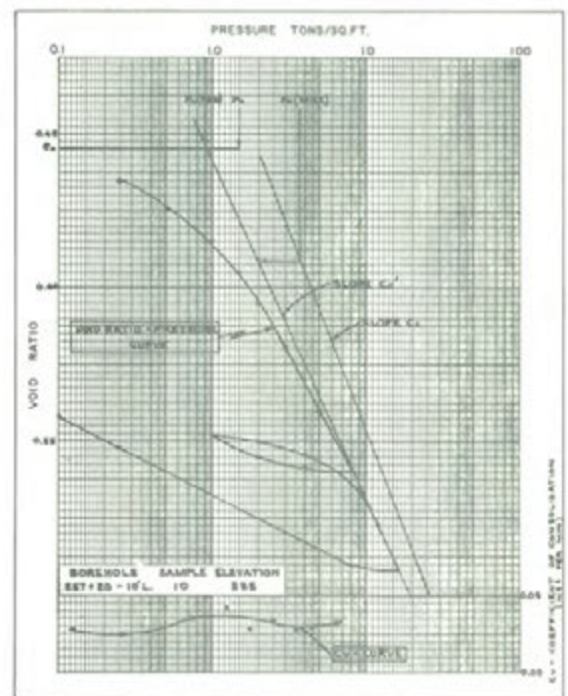
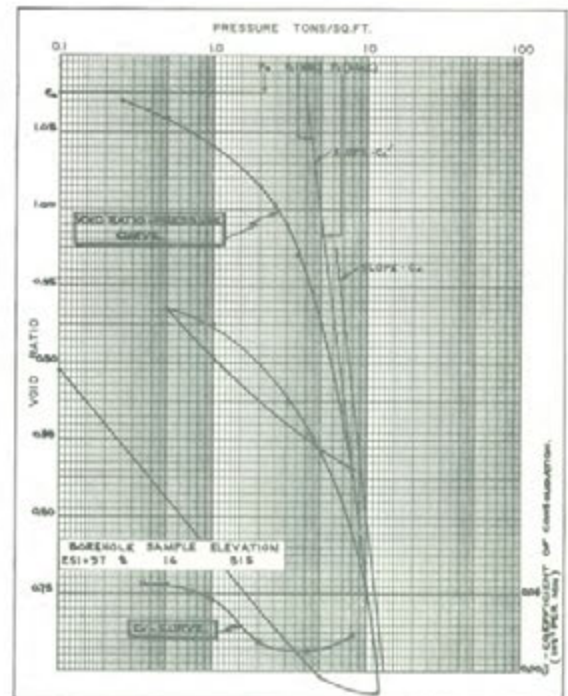


# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 20  
PROJECT S6849



APPENDIX II  
FIGURE 21  
PROJECT S6849

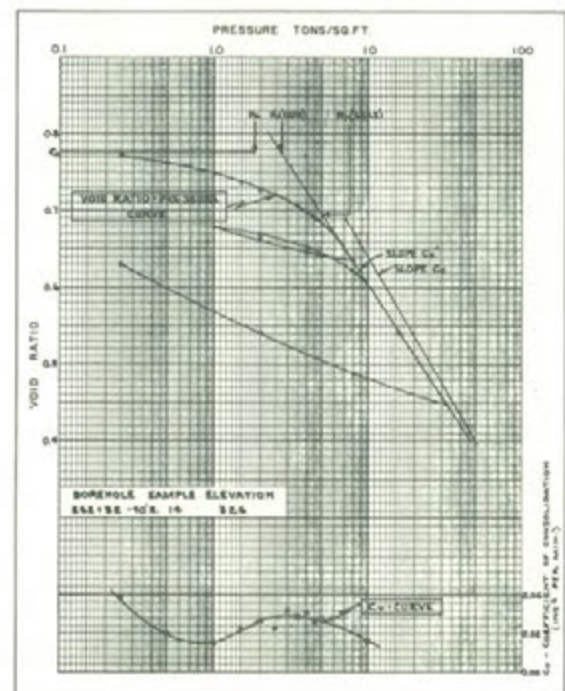
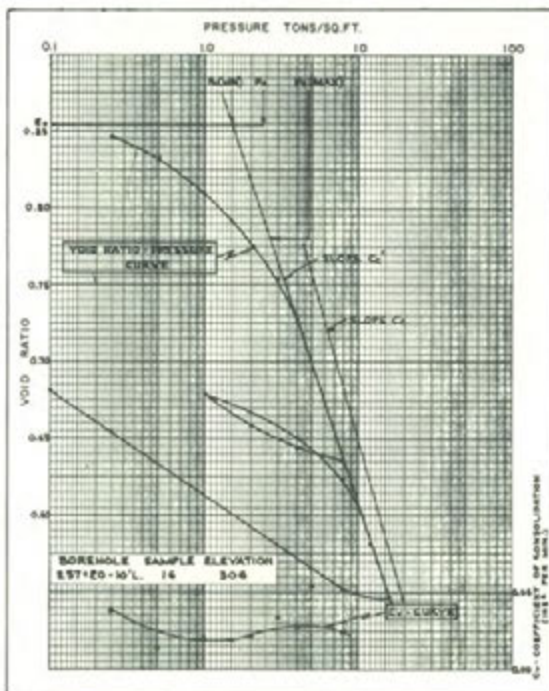
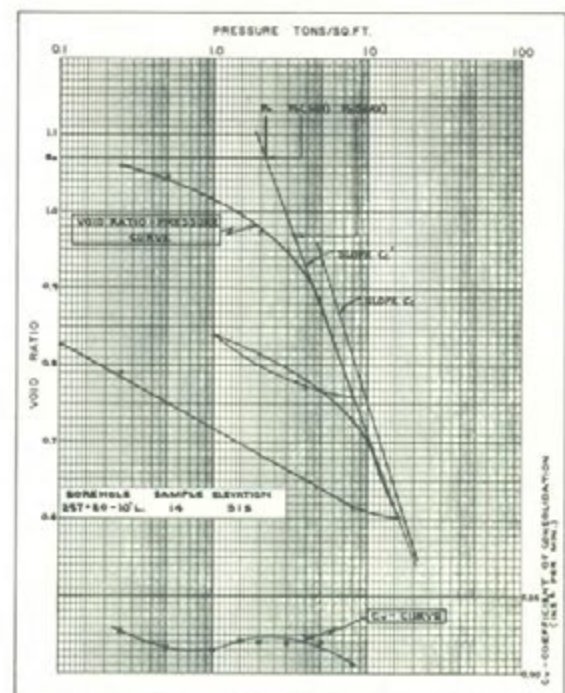
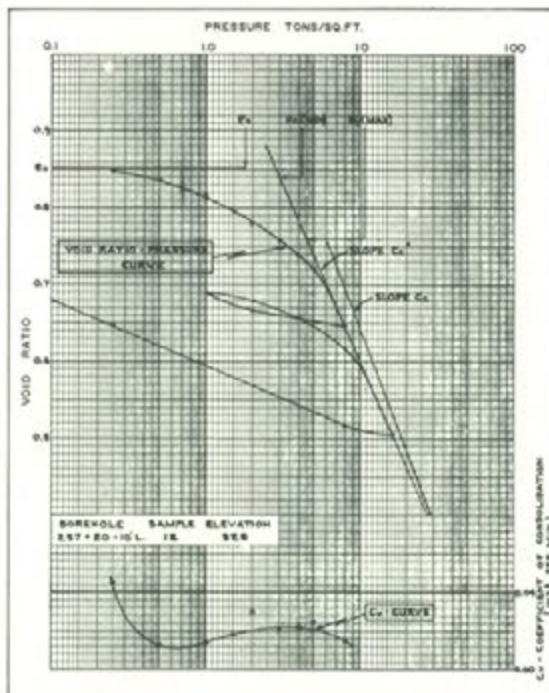


GEOCON



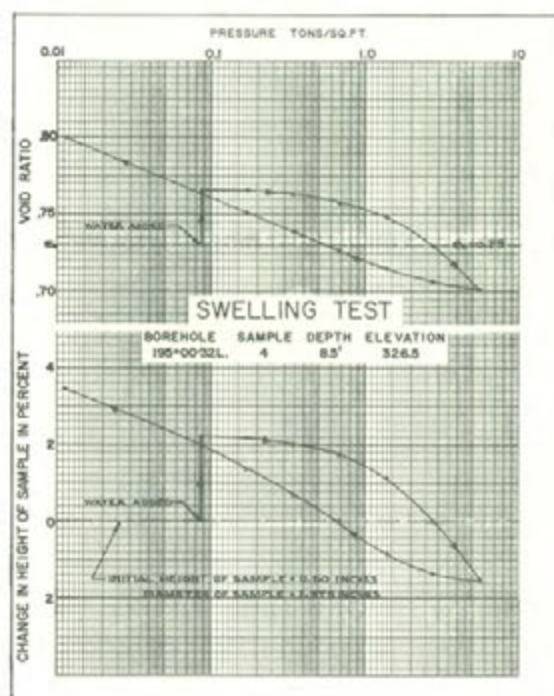
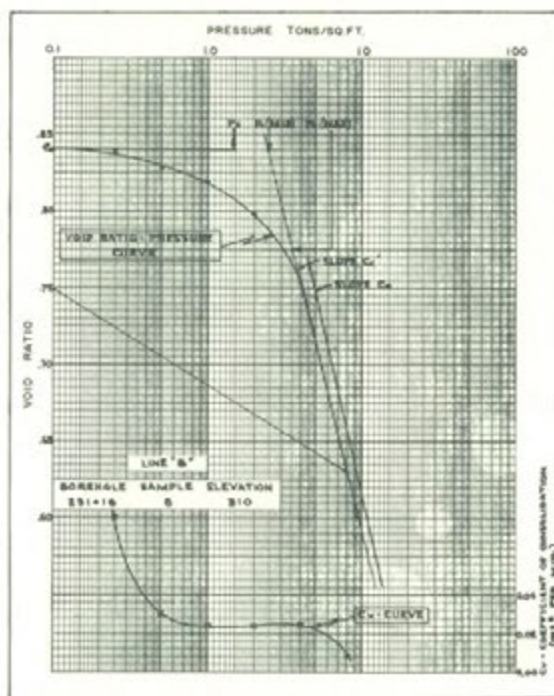
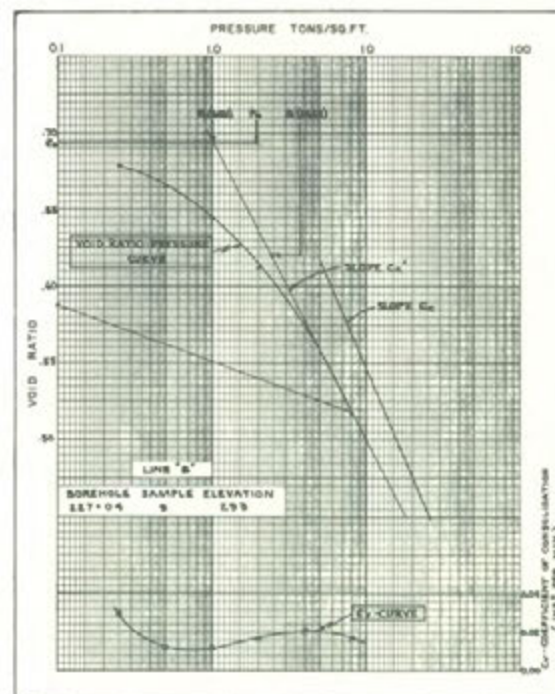
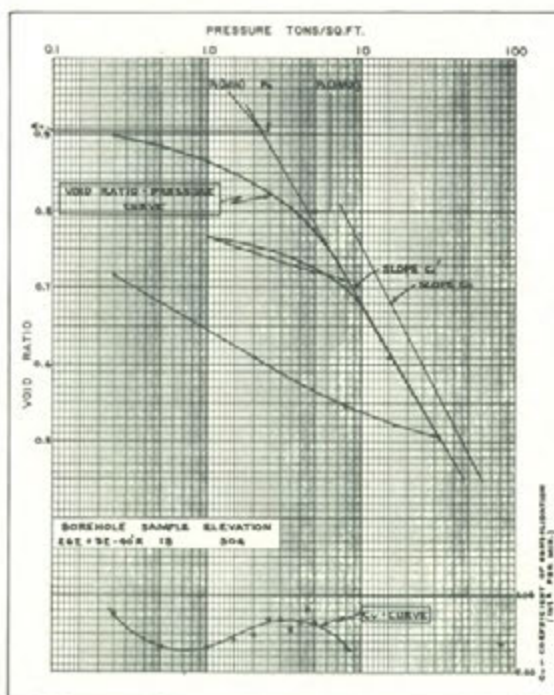
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 22  
PROJECT S6849



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 23  
PROJECT S6849



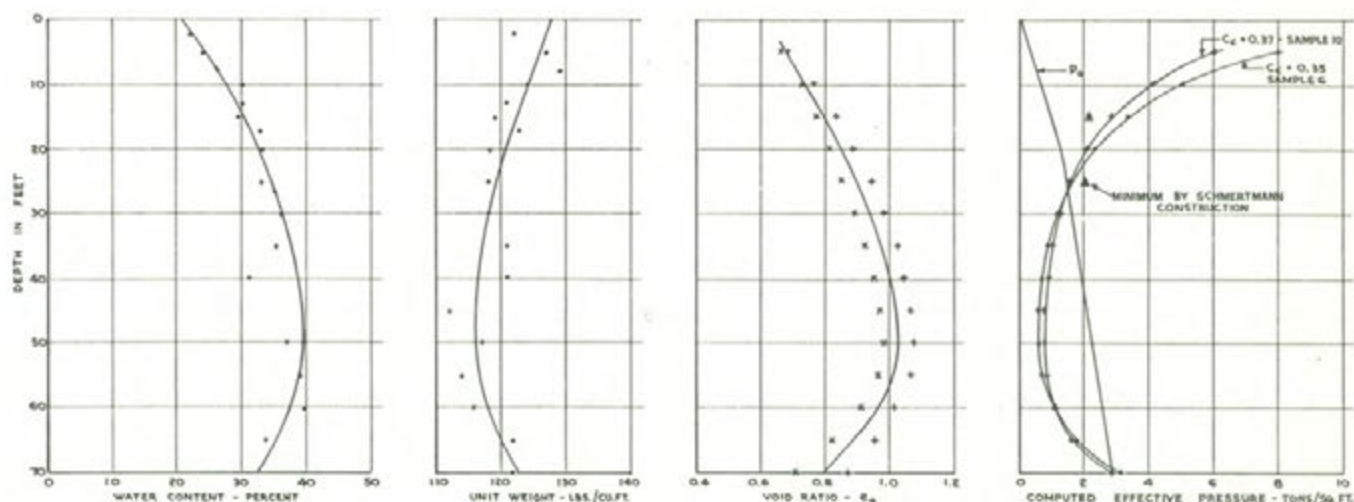


APPENDIX III

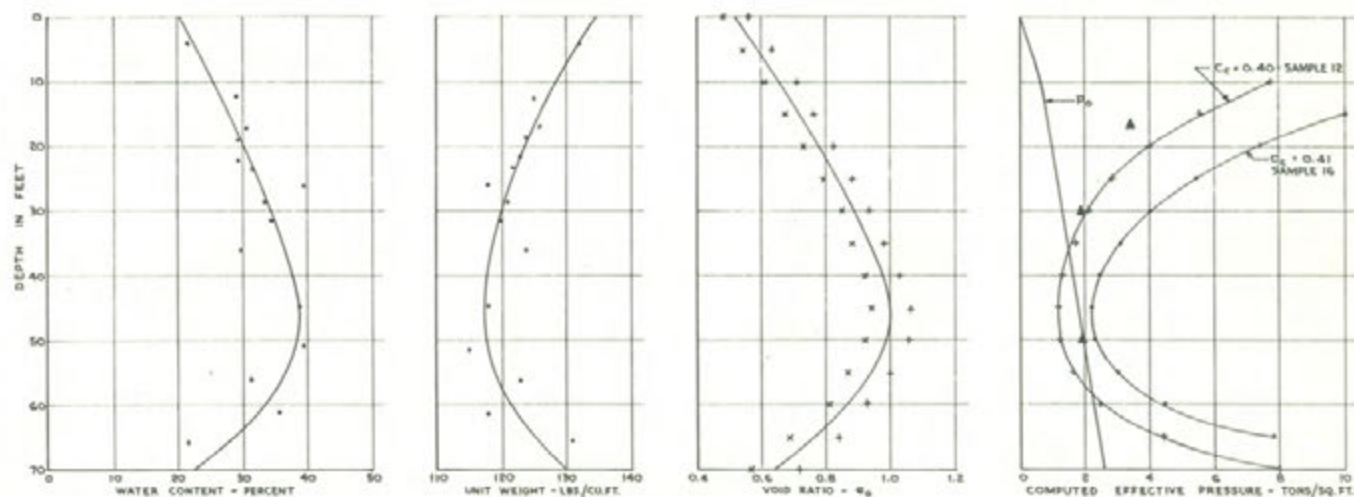
FIGURES - ENGINEERING ANALYSES

# TYPICAL COMPUTED OVERBURDEN PRESSURES

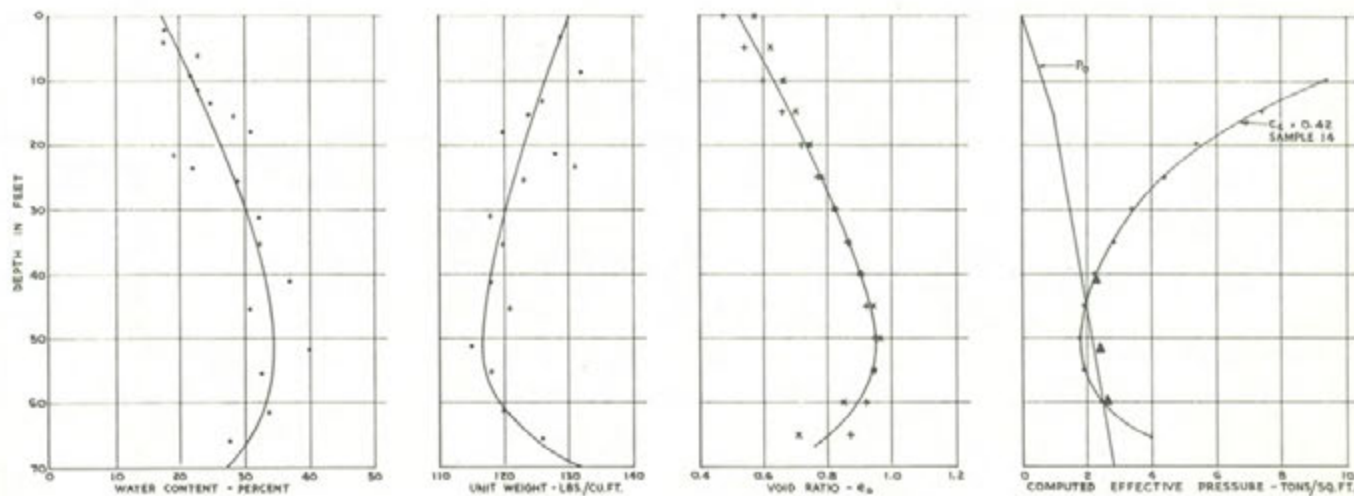
APPENDIX III  
FIGURE I  
PROJECT - S6849



BOREHOLE 214+50 -  $\phi$



BOREHOLE 203+70 - 37' LEFT



BOREHOLE 251+97 -  $\phi$

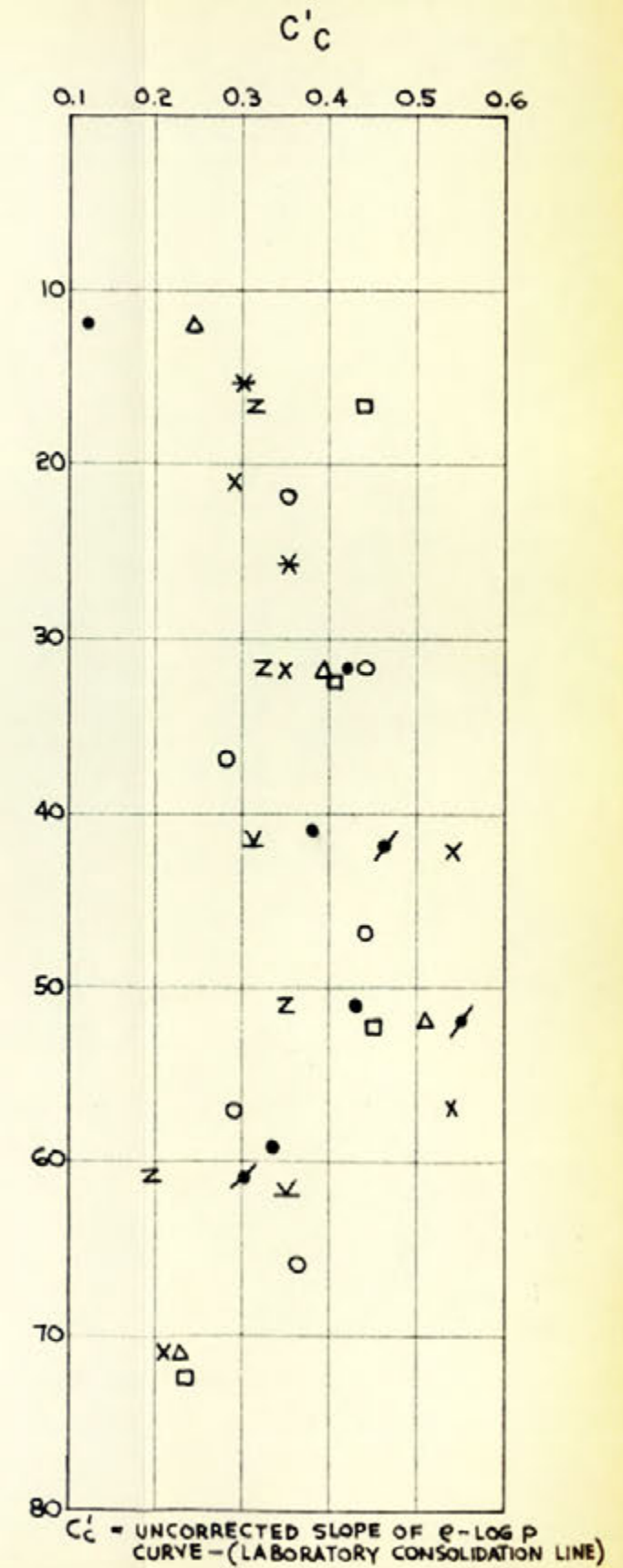
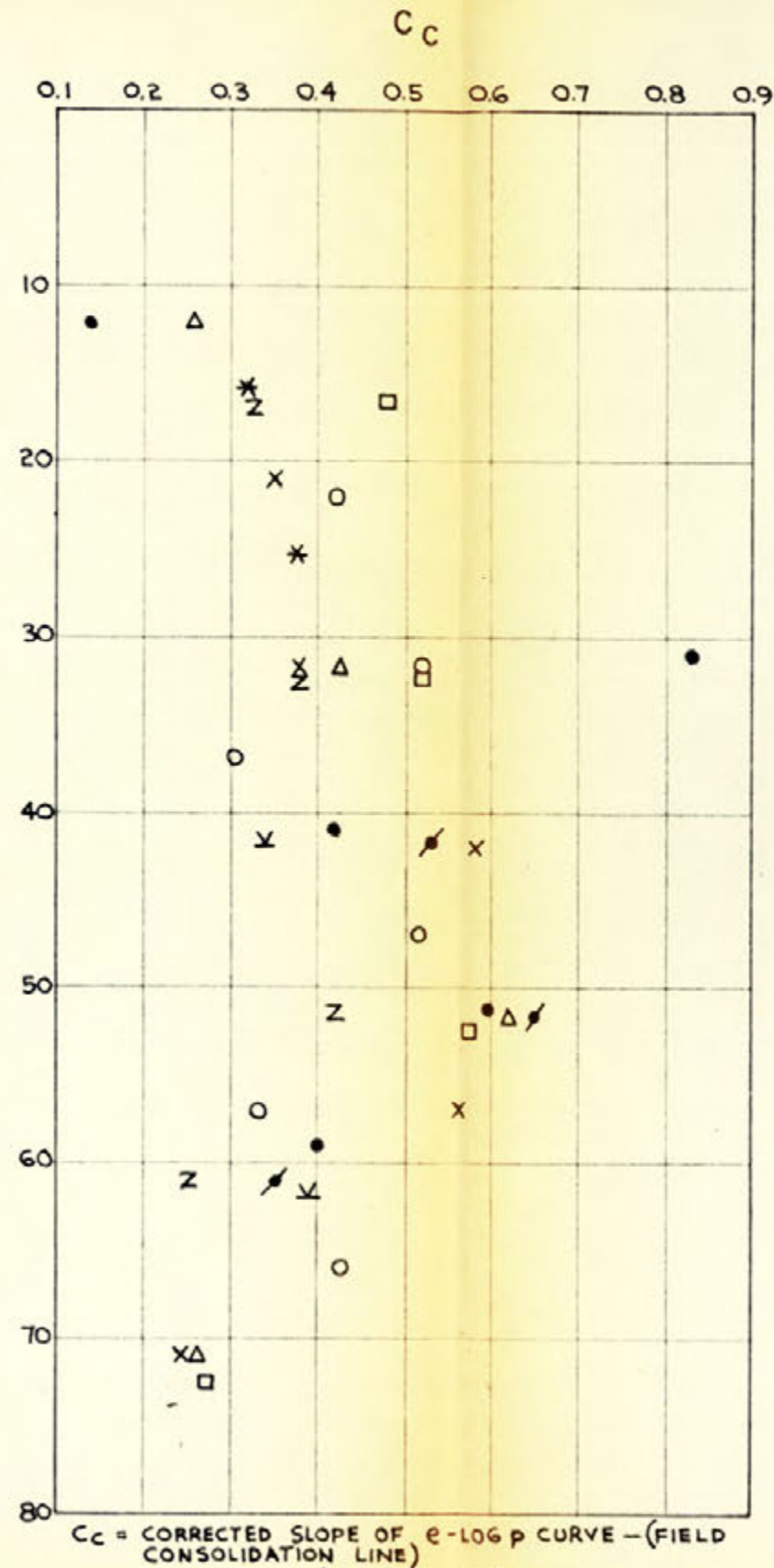
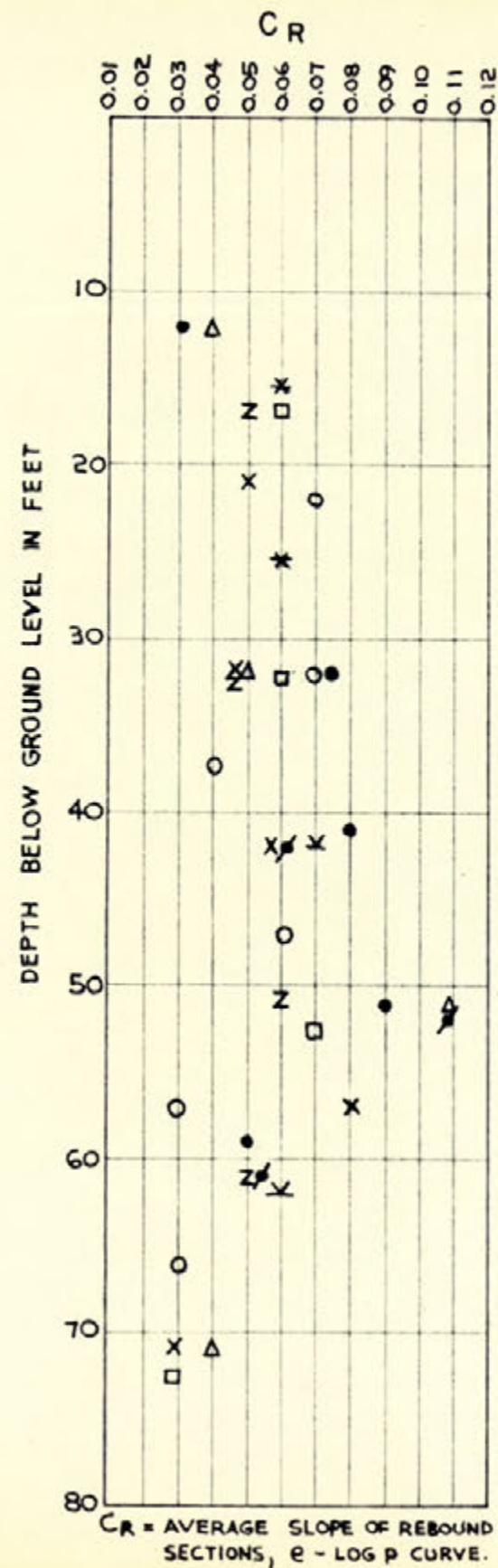
GEOCON



X - 186+50 53'L.    \* - 214+50    • - 251+97    •  
 O - 202+00    □ - 220+34    / - 257+20 10'L.  
 Z - 203+70 37L.    Δ - 235+46 2'L.    \ - 262+32 40'R.

# VARIATIONS OF $C_R$ , $C_C$ AND $C'_C$ WITH DEPTH COMPUTED FROM LABORATORY CONSOLIDATION TESTS

APPENDIX III  
 FIGURE 2  
 PROJECT - S6849

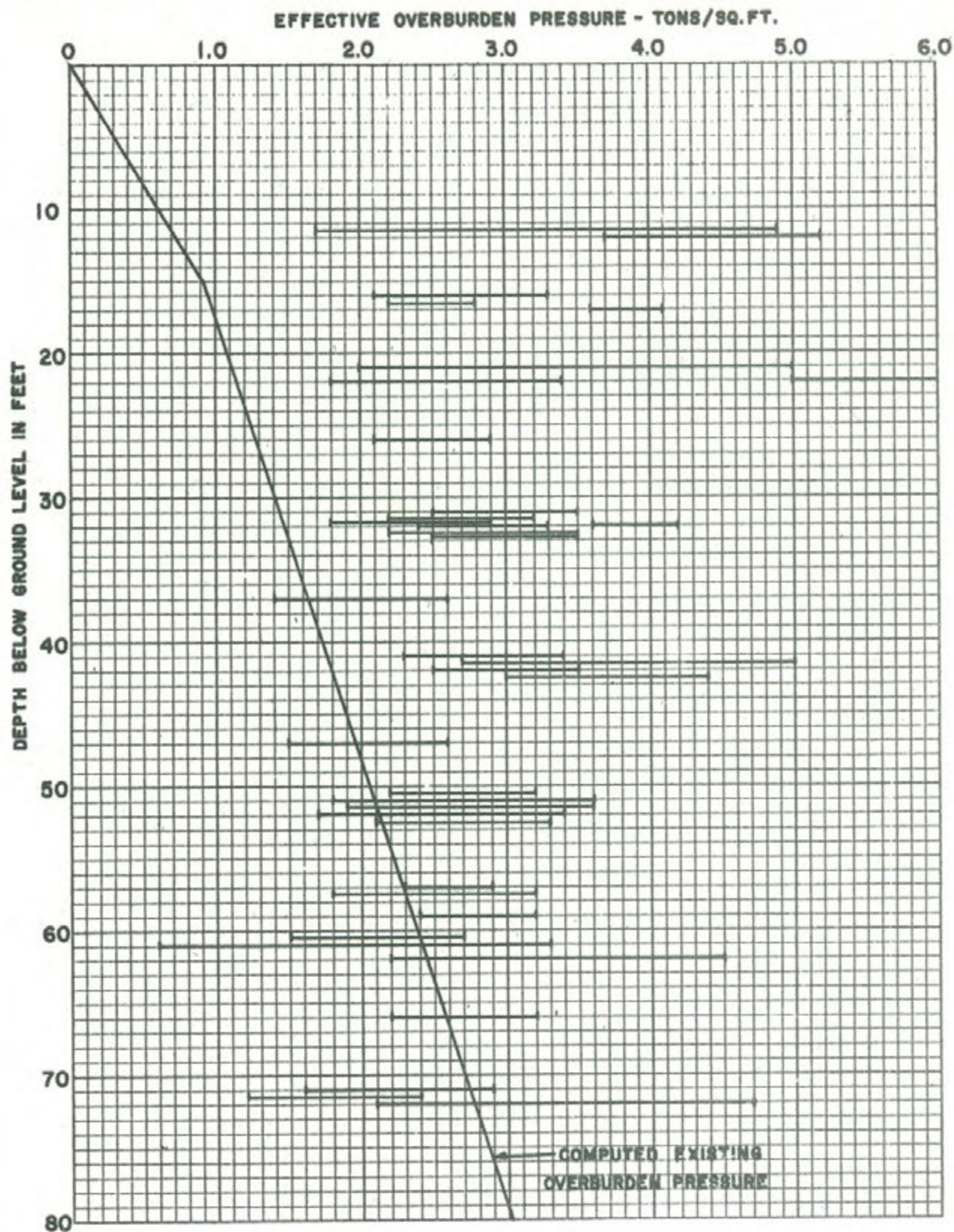


GEOCON



RANGE OF PRECONSOLIDATION EFFECTS  
SILTY CLAY STRATA  
BASED ON CONSOLIDATION TEST RESULTS

APPENDIX III  
FIGURE 3  
PROJECT - S6849

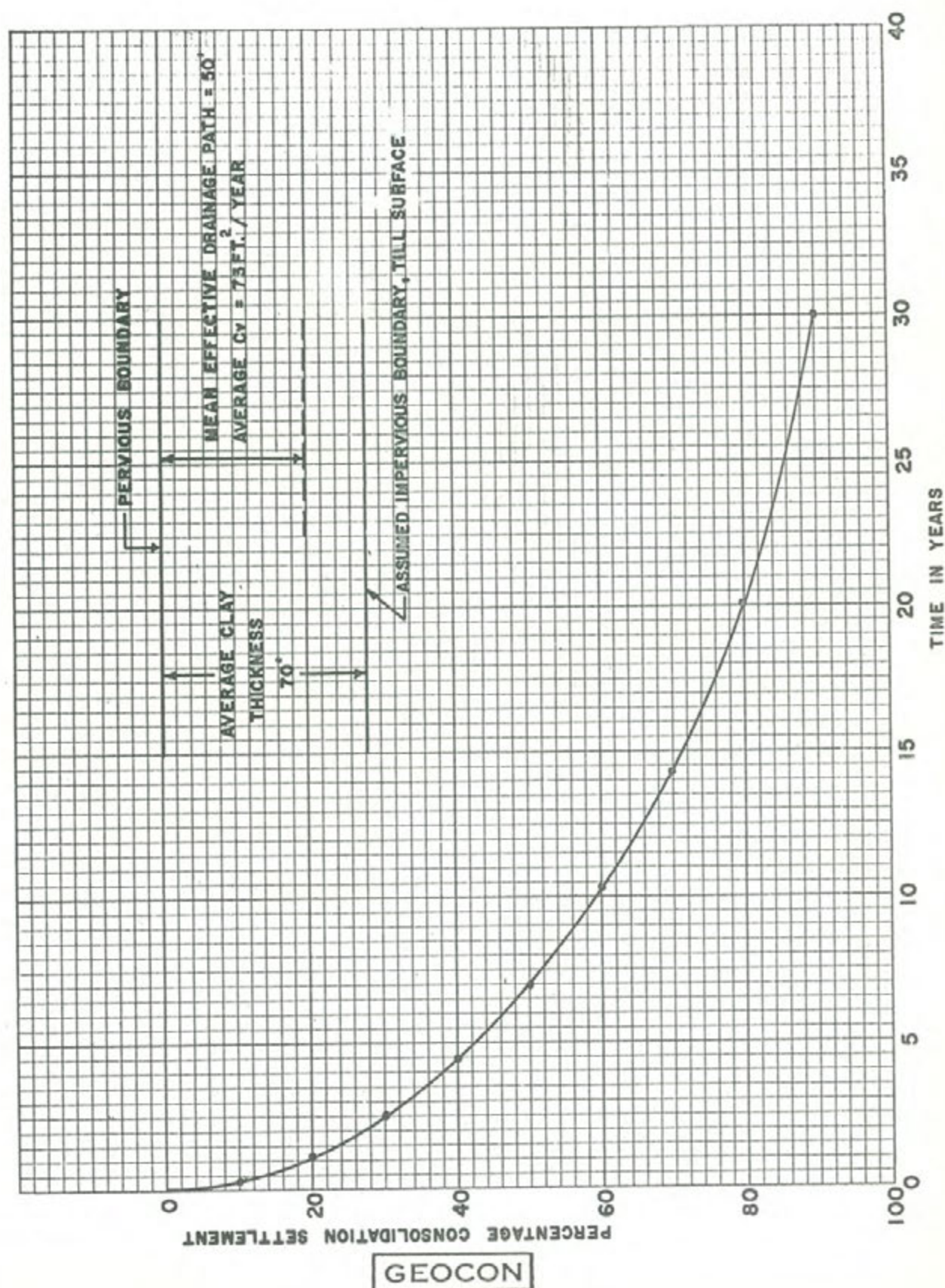


GEOCON



AVERAGE COMPUTED  
TIME-RATE CONSOLIDATION SETTLEMENT  
CLAY STRATA

APPENDIX III  
FIGURE 4  
PROJECT - S6849

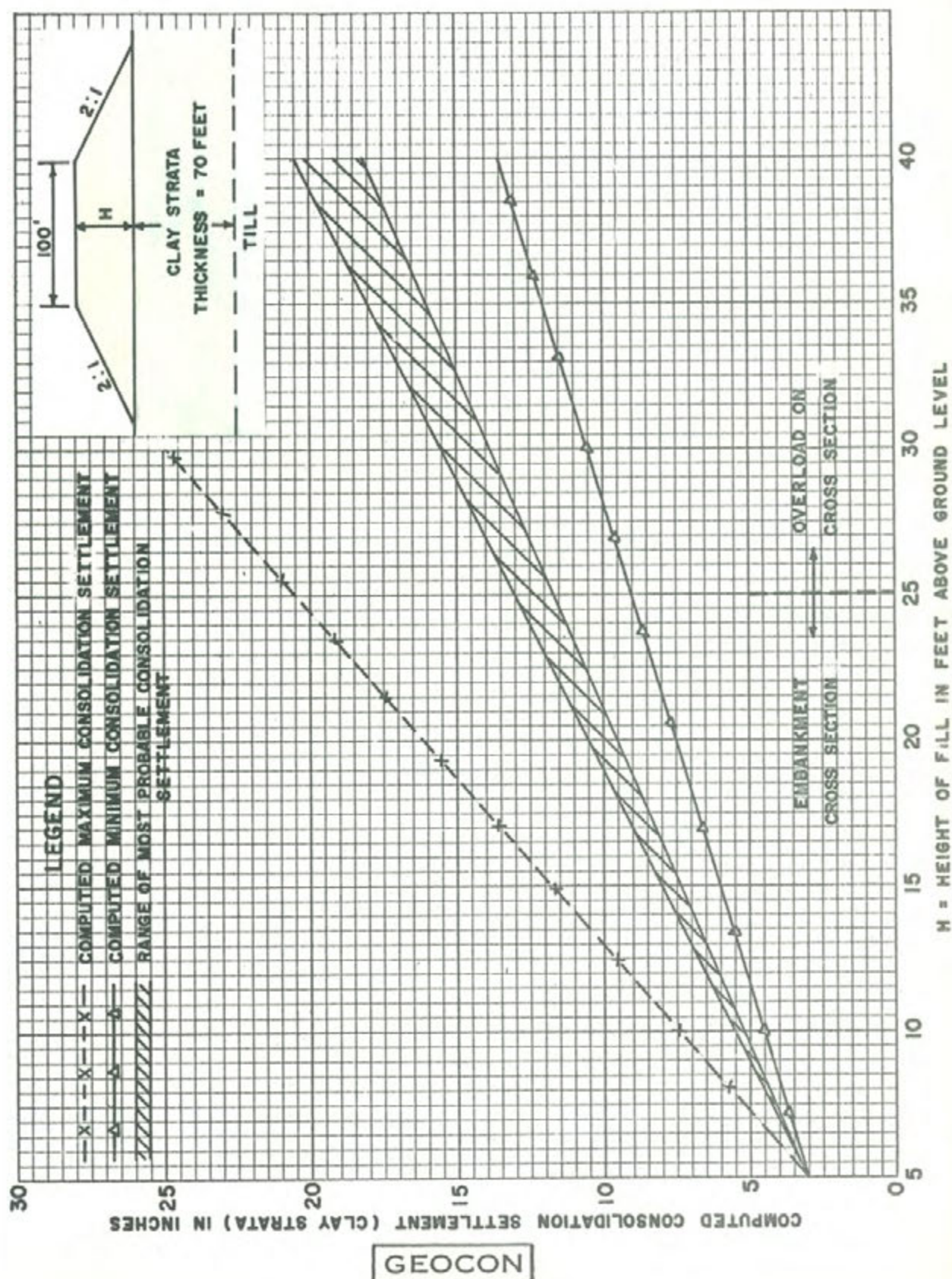




# COMPUTED SETTLEMENT

## APPROACH EMBANKMENTS

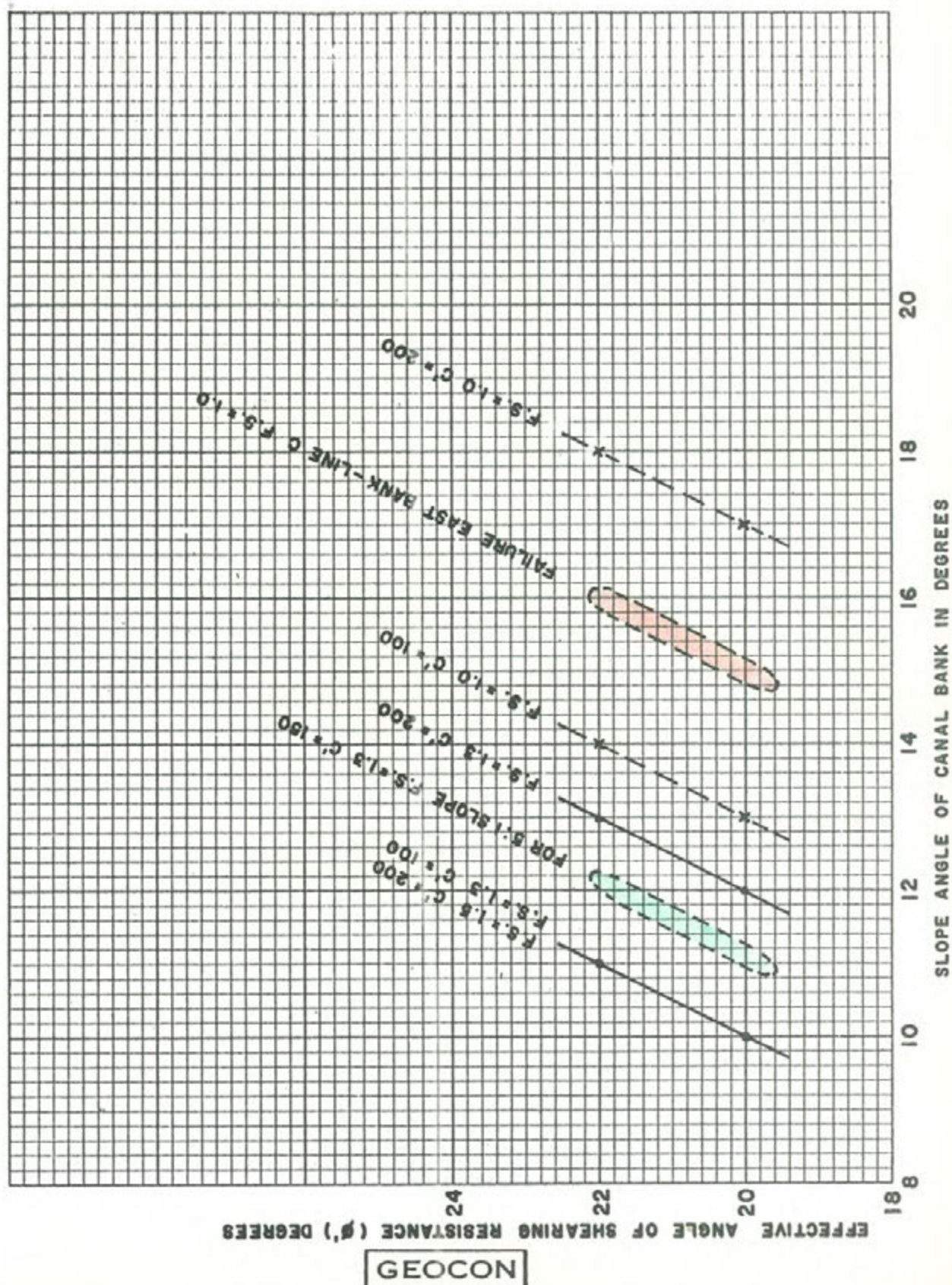
APPENDIX III  
FIGURE 5  
PROJECT - S6849





SUMMARY  
TOTAL STRESS STABILITY COMPUTATIONS  
EXISTING CANAL BANKS

APPENDIX III  
FIGURE 6  
PROJECT - S6849

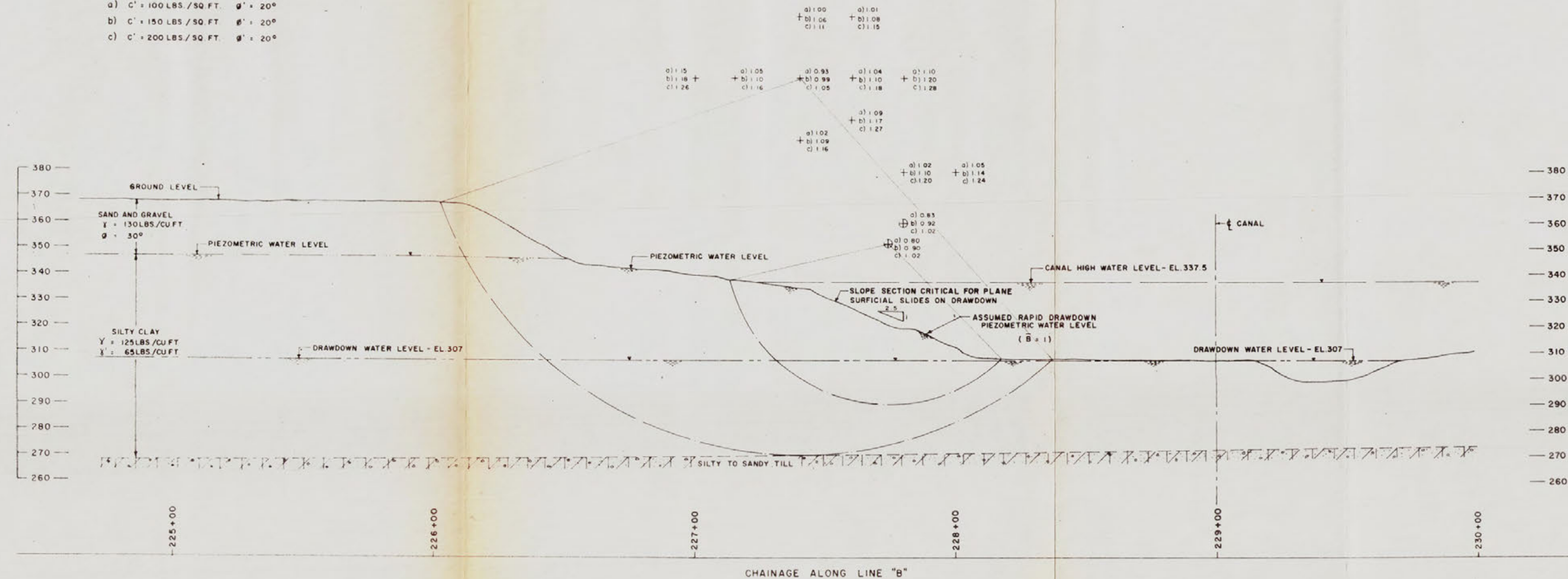




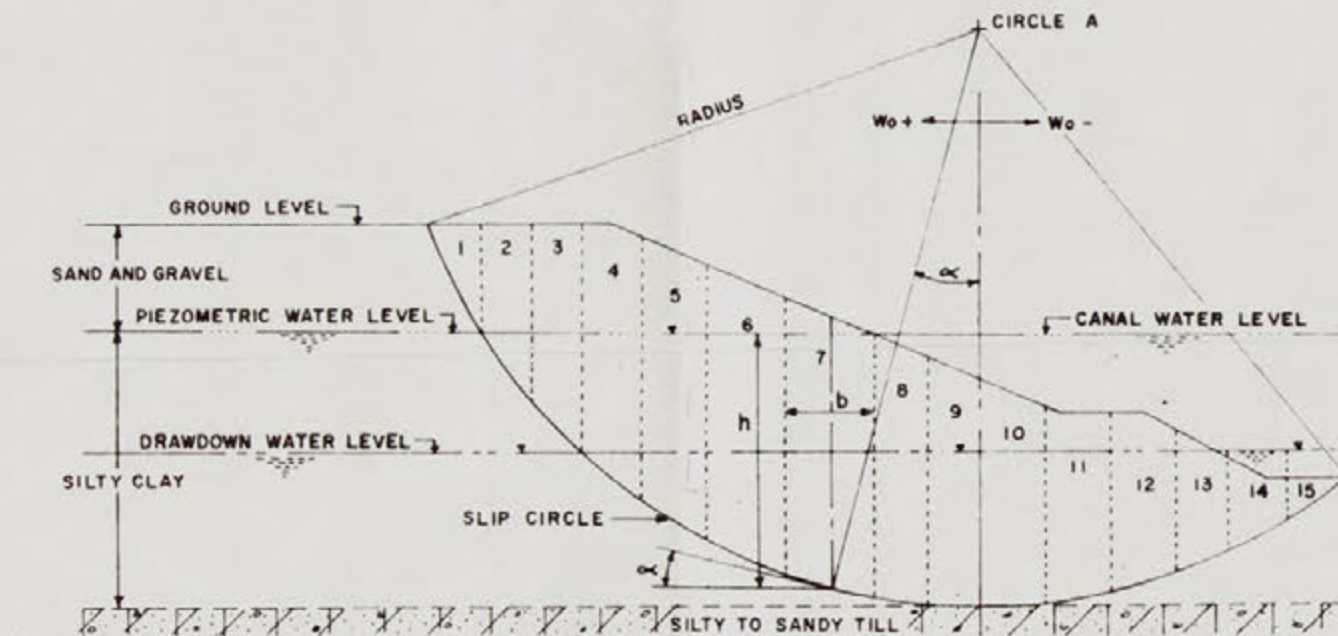
# LEGEND

- + FACTORS OF SAFETY FOR DEEP SEATED FAILURE SURFACES
- ⊕ FACTORS OF SAFETY FOR SHALLOW FAILURE SURFACES
- EFFECTIVE STRESS PARAMETERS

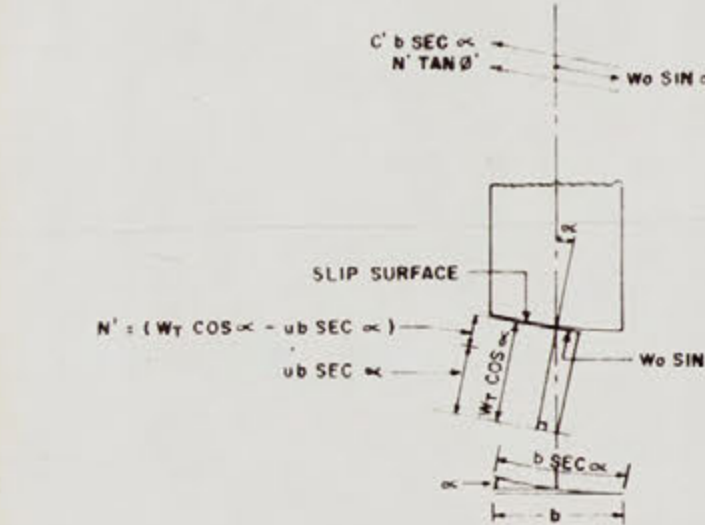
- a)  $C' = 100 \text{ LBS./SQ. FT.}$   $\phi' = 20^\circ$
- b)  $C' = 150 \text{ LBS./SQ. FT.}$   $\phi' = 20^\circ$
- c)  $C' = 200 \text{ LBS./SQ. FT.}$   $\phi' = 20^\circ$



## SCHEMATIC DIAGRAM SHOWING FORCES ACTING ON SLICE AT SLIP SURFACE



- $b$  = MAXIMUM WIDTH OF SLICE MEASURED HORIZONTALLY
- $\alpha$  = ANGLE BETWEEN HORIZONTAL AND TANGENT TO SLIP CIRCLE AT THE VERTICAL LINE PASSING THROUGH CENTRE OF GRAVITY OF SLICE
- $h$  = HEIGHT TO PIEZOMETRIC WATER LEVEL FROM SLIP CIRCLE MEASURED ALONG VERTICAL LINE PASSING THROUGH CENTRE OF GRAVITY OF SLICE



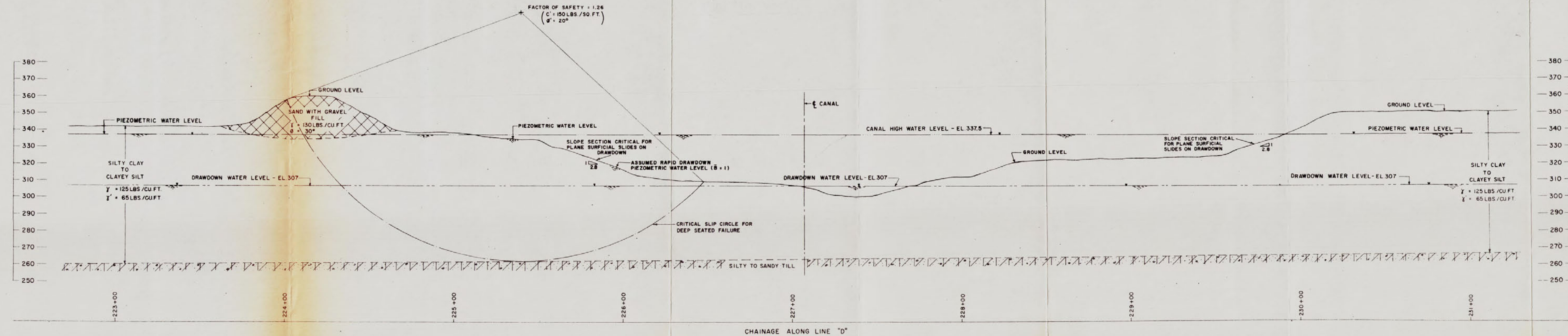
- $u$  = PORE PRESSURE AT CENTRE OF GRAVITY OF SLICE ON SLIP CIRCLE  $= \gamma_w h$
- $C'$  = EFFECTIVE SHEARING STRESS PARAMETER OBTAINED FROM UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENT
- $\phi'$  = EFFECTIVE ANGLE OF SHEARING RESISTANCE PARAMETER OBTAINED FROM UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENT
- $W_o$  = WEIGHT OF SOIL MASS IN SLICE FOR COMPUTATION OF OVERTURNING FORCE:  $\gamma$  (SATURATED) FOR PORTION ABOVE DRAWDOWN WATER LEVEL  $\gamma'$  (SUBMERGED) FOR PORTION BELOW DRAWDOWN LEVEL AND ANY AREAS CONTAINING WATER ONLY IGNORED
- $W_T$  = WEIGHT OF SOIL MASS IN SLICE FOR COMPUTATION OF FRICTIONAL FORCE:  $\gamma$  (SATURATED) THROUGHOUT AND AREAS CONTAINING WATER, WHERE  $\gamma = 62.4$ , TAKEN INTO ACCOUNT

$$\text{FACTOR OF SAFETY} = \frac{\sum \text{FORCES ACTING ON SLICES 1 TO 15}}{\sum W_o \sin \alpha} = \frac{\sum C' b \sec \alpha + \sum N' \tan \phi'}{\sum W_o \sin \alpha}$$

EFFECTIVE STRESS STABILITY ANALYSES  
RAPID DRAWDOWN CONDITION  
EXISTING SLOPE - WEST BANK WELAND CANAL  
LINE "B"  
CANAL CHAINAGE 229+00

GEOCON  
APPENDIX III  
FIGURE 7  
PROJECT - S6849



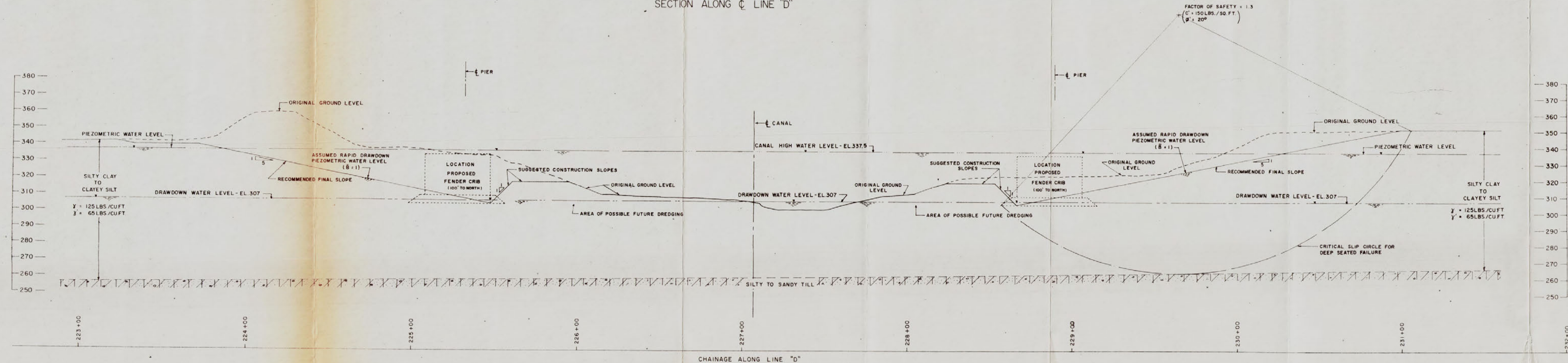


EFFECTIVE STRESS STABILITY ANALYSES  
 RAPID DRAWDOWN CONDITION  
 EXISTING SLOPE - WEST BANK WELLAND CANAL  
 LINE "D"  
 CANAL CHAINAGE 291+50

GEOCON  
 APPENDIX III  
 FIGURE 8  
 PROJECT - S6849



# SECTION ALONG CL LINE "D"

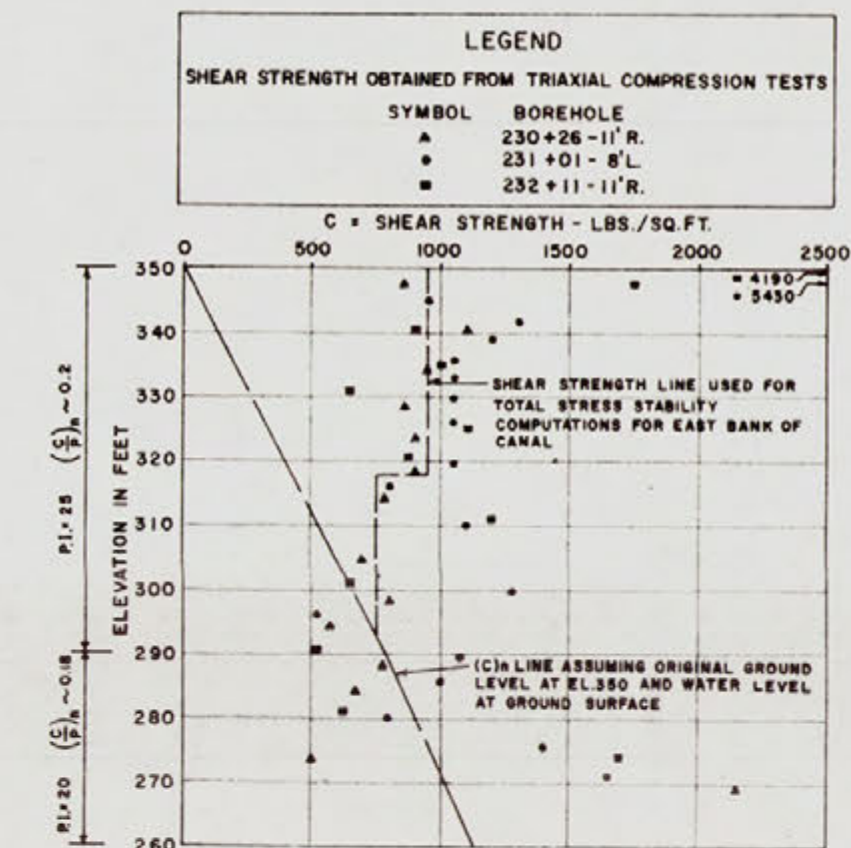
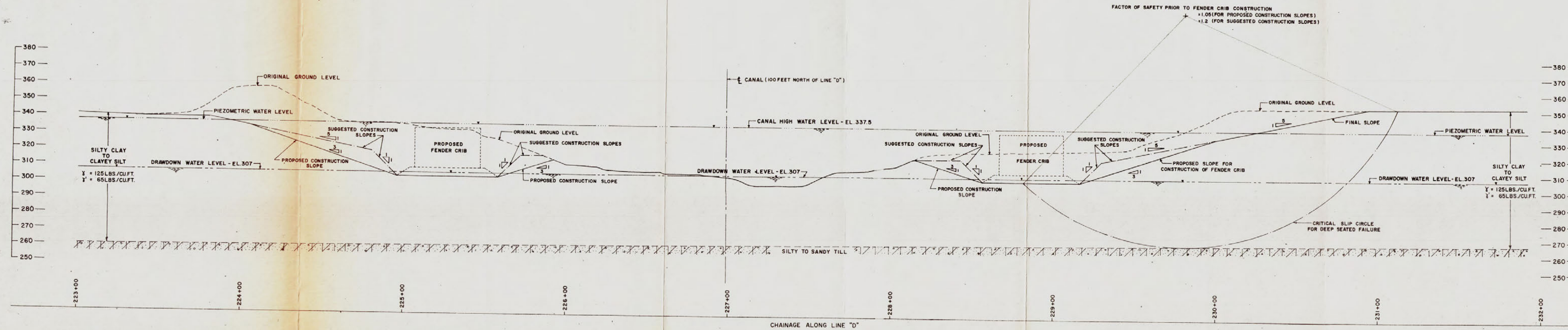


EFFECTIVE STRESS STABILITY ANALYSES  
RAPID DRAWDOWN CONDITION  
RECOMMENDED WELAND CANAL SIDE SLOPES  
- LINE "D" BRIDGE CROSSING

GEOCON  
APPENDIX III  
FIGURE 9  
PROJECT - S 6849



# SECTION 100' NORTH OF LINE "D"



TOTAL STRESS STABILITY ANALYSES  
 RAPID DRAWDOWN CONDITION  
 CONSTRUCTION SIDE SLOPES CANAL  
 FENDER CRIB CONSTRUCTION STAGE  
 LINE "D" BRIDGE CROSSING

GEOCON  
 APPENDIX III  
 FIGURE 10  
 PROJECT - S6849



APPENDIX IV

BORROW PIT SURVEY

Summary Borrow Pit Survey

Table I

Site Plan

Figure 1

Typical Grain Size Distribution Curves

Figure 2



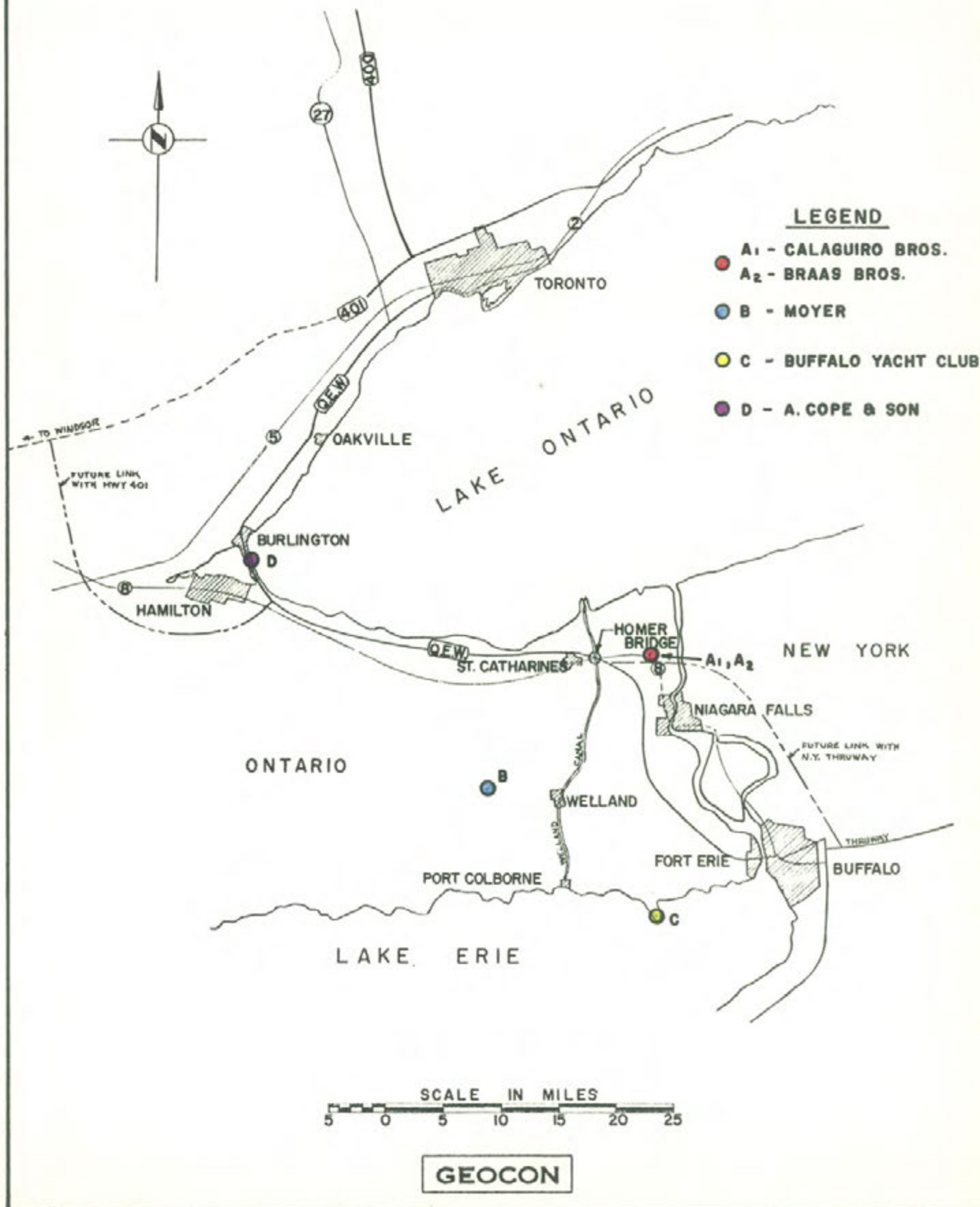
TABLE I

SUMMARY BORROW PIT SURVEY

Owner	Calaguire Bros. Ltd.		Brass Bros. Ltd.	Moyer (Ontario Custom Crushing Ltd.)	Buffalo Yacht Club	A. Cope & Son
Location	Between St. David and Stamford		Between St. David and Stamford	Near Fonthill	Point Abino	Hamilton Harbour
Location on Drawing Figure 1 Appendix IV	A <sub>1</sub>		A <sub>2</sub>	B	C	D
Type of Pit Run Granular Material	medium to coarse sand and gravel	fine to medium sand	heterogeneous fine to medium sand with coarse sand and gravel pockets	heterogeneous fine to medium sand with sandy gravel pockets	fine to medium sand	medium to coarse sand and fine gravel
Approximate Quantity	60,000 cu. yds.	virtually unlimited	virtually unlimited	virtually unlimited	virtually unlimited	indefinite
Typical Grainsize Distribution Curves Shown on	Figure 2(A) Appendix IV		Figure 2(B) Appendix IV	Figure 2(C) Appendix IV	Figure 2(D) Appendix IV	Figure 2(D) Appendix IV
Approximate Distance From Homer	5 to 6 miles		5 to 6 miles	13 miles	31 miles	34 miles
Screening Plant Available	No		No	Yes Capacity 45-50 tons/hour	Not Known	Not Known

# BORROW PIT SURVEY LOCATION PLAN

APPENDIX IV  
FIGURE I  
PROJECT - S6849





APPENDIX IV  
FIGURE 2  
PROJECT S6849



APPENDIX V

RESULTS CHEMICAL ANALYSES

Typical Groundwater Samples

Table I

Typical Soil Samples

Table II



TABLE I

RESULTS CHEMICAL ANALYSESTYPICAL GROUNDWATER SAMPLES

Location Line "D"	Date Hole Augured	Depth of Hole	Date Sample Obtained	Depth to Water Level	SO <sub>3</sub> (p.p.m.)	Calcium as Carbonates (p.p.m.)	Bicarbonates HCO <sub>3</sub> (p.p.m.)	Free CO <sub>2</sub> (p.p.m.)	pH
220+26-20L	June 24/59	25'	Aug. 13/59	4'	153	460		96	7.60
			Sept. 11/59	4'	150		366	132	
230+77.5-CL	June 17/59	35'	Sept. 11/59	4'	59		139	50	7.94
251+26-5R	June 24/59	30'	Aug. 14/59	17'	3740	515		275	7.35
			Sept. 11/59	13'	1502		771	278	

TABLE IIRESULTS CHEMICAL ANALYSESTYPICAL SOIL SAMPLES

Borehole No. Line "D"	Soil Sample Number	Depth Below Ground Level	SO <sub>3</sub> Content (percent)	Calcium as Carbonate (percent)
186+50-53L	4	9'	0.007	11.25
195+00-32L	5	11'	0.04	
203+70-37L	5	11'	Trace	
209+68-24R	5	11'	Trace	
214+50-CL	4	11'	0.05	
220+34-CL	3	9'	0.03	
225+15-39R	3	9'	0.03	
231+01-8L	4	9'	0.02	
235+46-2L	4	9'	Trace	
238+34-3L	5	11'	Nil	
242+46-9R	3	8'	0.09	
244+48-CL	5	13'	Nil	
246+80-25R	4	9'	Trace	
251+97-CL	4	10'	0.10	
256+93-CL	4	9'	0.15	
257+20-10L	4	9'	0.15	15.65
262+32-40R	3	6'	0.17	

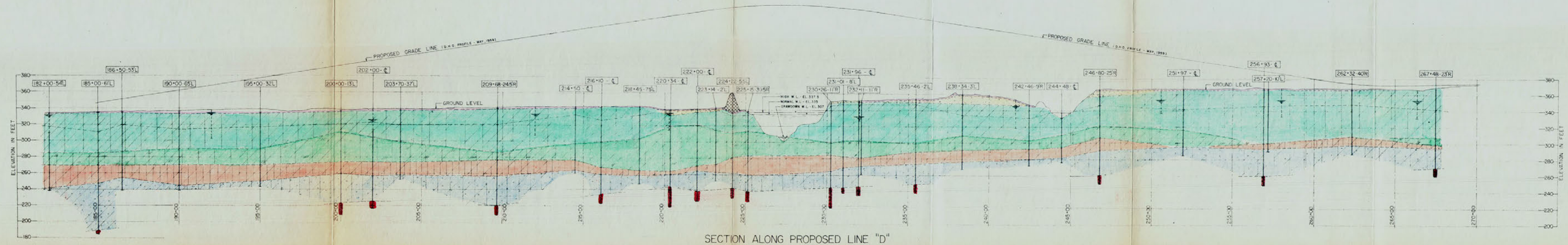
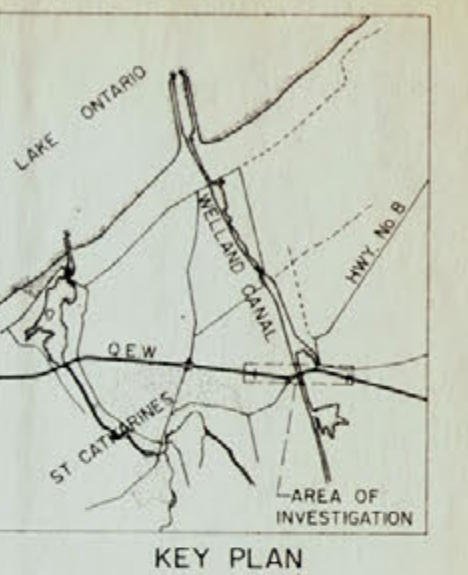
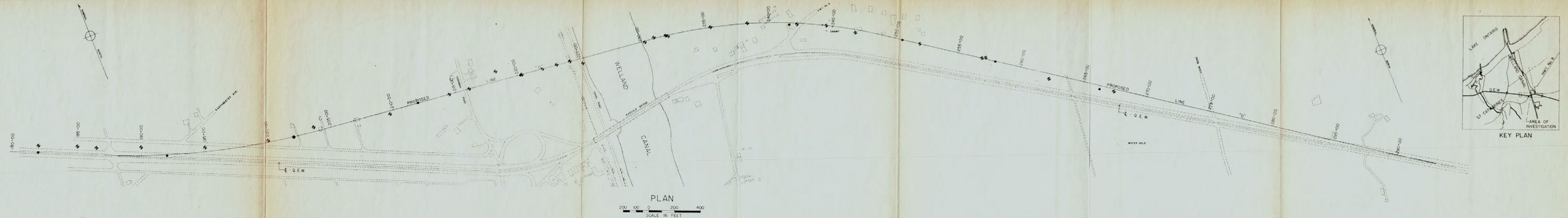


APPENDIX VI

Drawing S6849-1 Boring Plan and Soil Stratigraphy - Line "D"

Drawing S6849-2 General Site Plan and Piezometric Observations





# STRATIGRAPHY

- TOPSOIL
- LOOSE TO COMPACT BROWN SILTY SAND AND GRAVEL
- HARD TO FIRM GREY AND BROWN TO GREY SILTY CLAY
- FIRM REDDISH-BROWN AND GREY LAYERED SILTY CLAY
- FIRM TO STIFF GREY TO BROWN CLAYEY SILT
- VERY DENSE REDDISH-BROWN SILTY AND SANDY TILL
- REDDISH-BROWN SHALE BEDROCK

# LEGEND

- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- STANDPIPE IN PLAN
- STANDPIPE IN ELEVATION
- WATER LEVEL IN STANDPIPE - SEPTEMBER 11, 1957

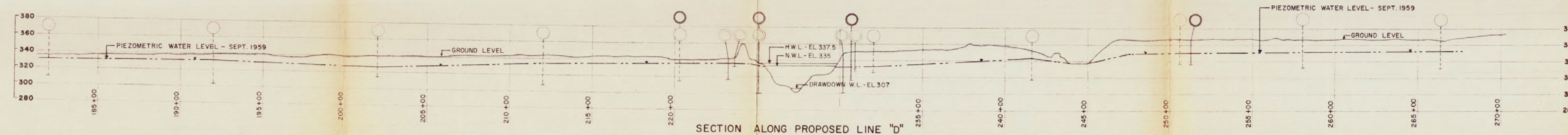
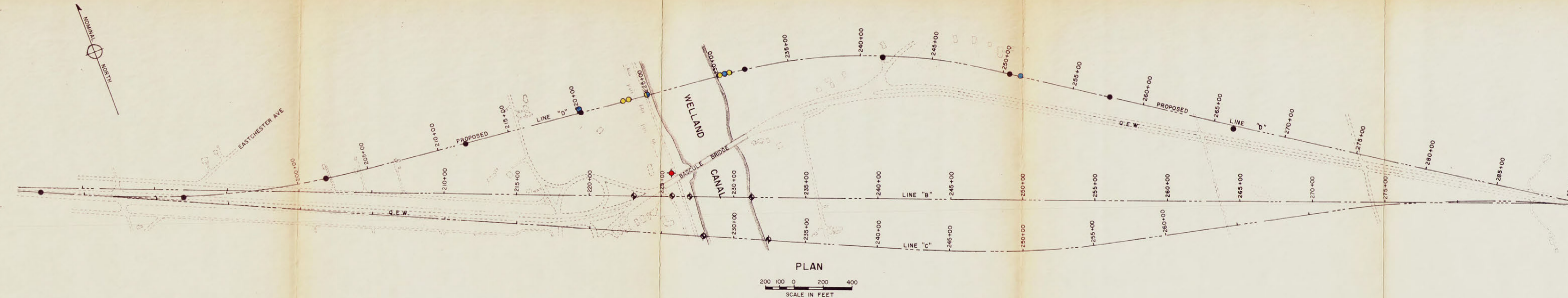
SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

DATE	DESCRIPTION	REFERENCE
1953-1-10	FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED - HOMER BRIDGE - PLAN, LINES B, C AND D	FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED - HOMER BRIDGE - PLAN, LINES B, C AND D
1953-1-11	FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED - HOMER BRIDGE - PROFILES, LINES B, C AND D	FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED - HOMER BRIDGE - PROFILES, LINES B, C AND D

FOUNDATION OF CANADA  
ENGINEERING CORPORATION LIMITED  
HOMER HIGH LEVEL BRIDGE  
HOMER  
BORING PLAN AND SOIL STRATIGRAPHY

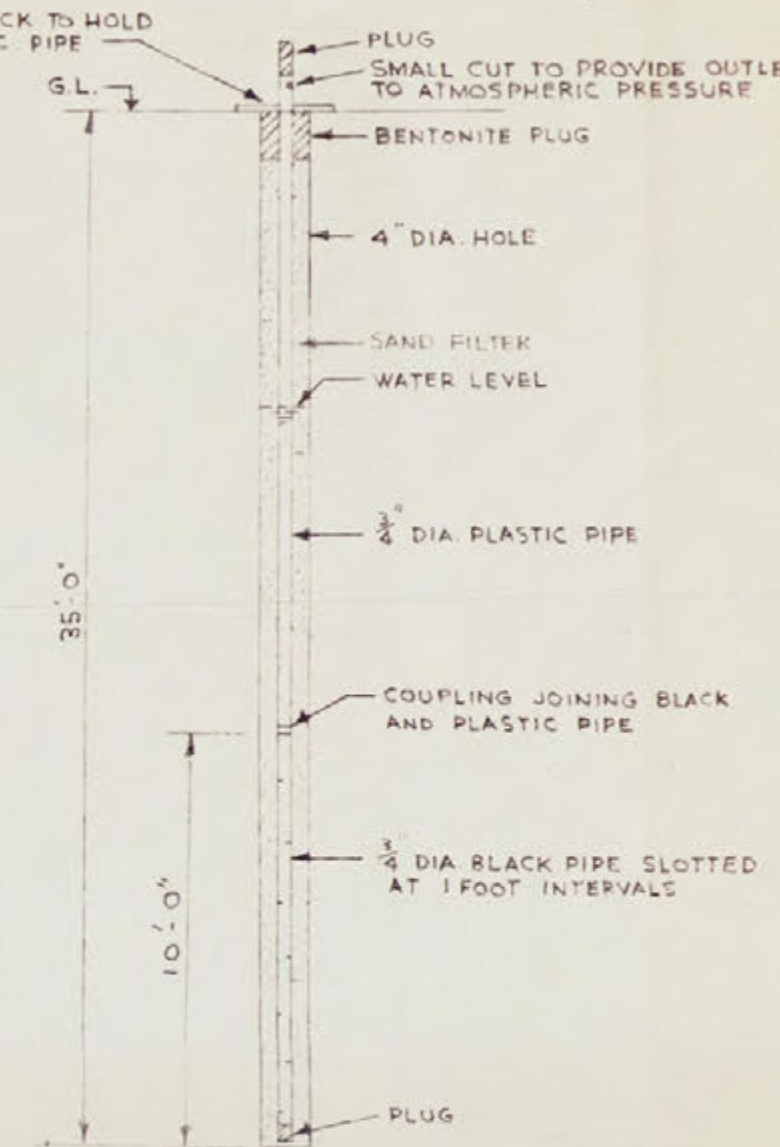
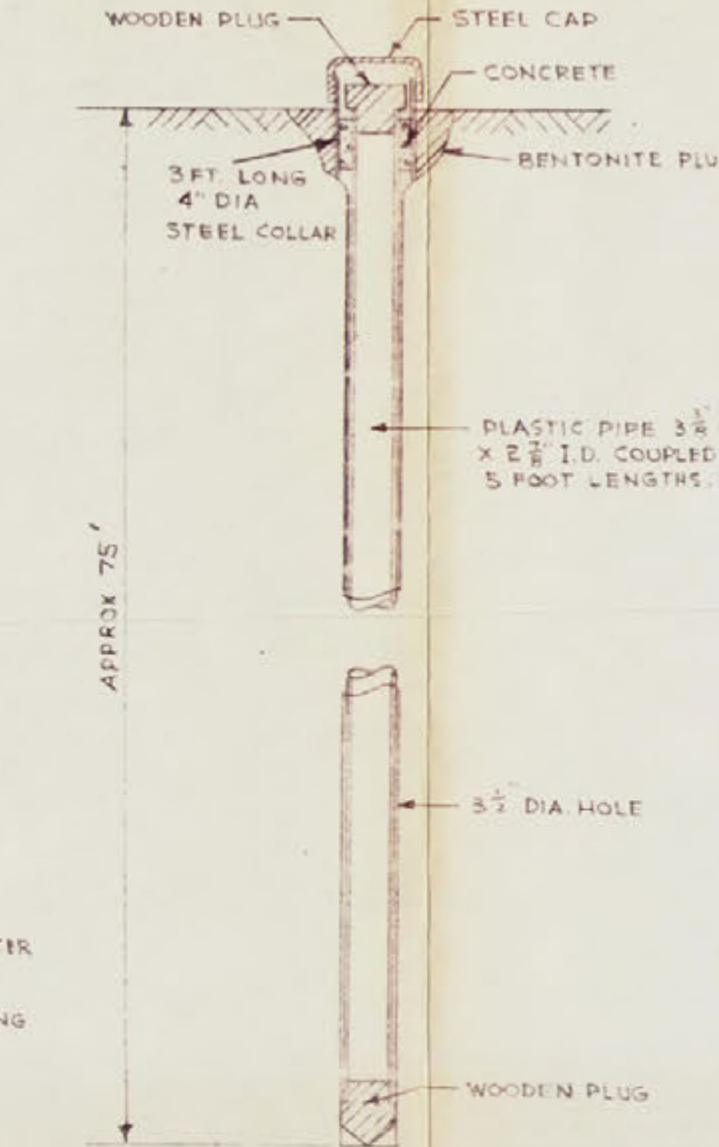
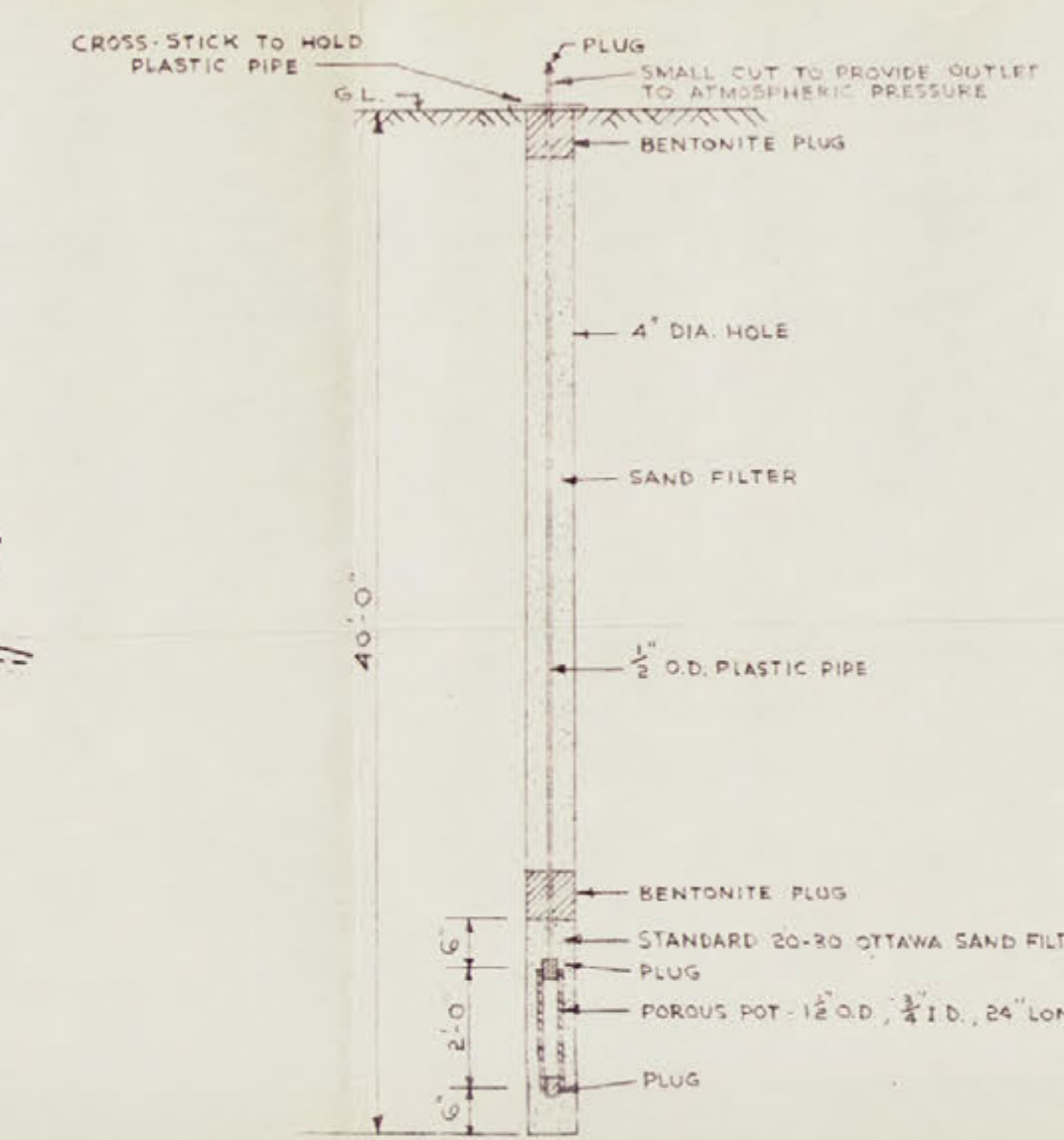
DATE: AUGUST 24, 1953 SCALE: AS SHOWN  
MADE: CHD: J.A. R. VEE  
No S6849-1





#### LEGEND

- ◆ BOREHOLE IN PLAN - LINES "B" AND "C"
- ◆ SLOPE INDICATOR CASING LOCATION IN PLAN
- STANDPIPE LOCATION IN PLAN ○ IN ELEVATION
- PIEZOMETER LOCATION IN PLAN ○ IN ELEVATION
- WATER TEST HOLE LOCATION IN PLAN ○ IN ELEVATION



FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED		GEOCON LTD	
HOMER HIGH LEVEL BRIDGE		DATE DEC. 9, 1959 SCALE AS SHOWN	
GENERAL SITE PLAN AND PIEZOMETRIC OBSERVATIONS		MADE BY J.A. JLS	NO. S6849-2