

REFERENCE TO
62-F-99

Mr. B. B. Davis,
Bridge Engineer,
Bridge Division.

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

Attention: Mr. S. McCombie

June 29, 1965

C.N.R. Overpass -- Tri-Town Bypass,
Dist. #14 (New Liskeard) W.P. 104-66

A meeting was held on June 24, 1965, to discuss the alternative schemes for the above project. The purpose of this memo is to recapitulate briefly, the history of the investigation and design of this project and to record the various schemes we proposed for this crossing, based on the results of field observation for the past two years.

The original foundation investigation for the crossing and approach fills was undertaken by Geoccon, Ltd. in 1960. The subsoil was found by the Consultant to be a thick deposit of laminated and varved clay, underlain by banded limestone bedrock at a depth of 150 - 160 ft. The consistency of the varved clay was found to be soft to firm, having values of shear strength as low as 300 p.s.f. at some locations.

The Consultant recommended an overhead crossing with approach embankments of 33 ft. high, necessitating 120 ft. long berms. The final settlement was expected to be of the order of 100 inches. In view of these shortcomings, Geoccon suggested to investigate alternative routes for the crossing and mentioned the possibility of a crossing by means of a subway.

The alternate routes were investigated by H. Q. Golder and Associates in 1961. It was found that soil conditions did not change significantly along the C.N.R. tracks within one mile north and south of the proposed line. Therefore, the present alignment was adopted.

A decision was then reached to go ahead with the construction of the overhead, utilizing stage construction for the embankments. The first stage called for a height of fill of 20 ft., followed by the full height, after sufficient strength increase in the subsoil is obtained by consolidation under the load of the first stage. In order to observe the pore pressures and settlements caused by the load, piezometers and settlement gauges were installed by this Section near the crossing under the proposed embankment.

June 29, 1965

During construction of the first stage, in June 1963, a failure took place some 500 ft. east of the crossing. The height of the fill at the failure was approximately 18 ft. A short berm was designed by this Section to increase stability, and the fill was brought back to 18 ft. in height. Further placement of the fill was called to a halt.

Results of observations for the past two years, together with additional field and laboratory tests carried out on samples from a borehole put down in October 1964, show that:

- (a) The excess pore pressure has dissipated by only 15% or less in the last two years.
- (b) No increase in strength of the subsoil was noticeable. In fact, there is a slight decrease of the undrained shear strength.
- (c) The settlement is still progressing at a rate of approximately 5 in./year, and there is no trend for the rate to decrease.

From the above conclusions, it is obvious that the stage construction method as originally contemplated, is not feasible if construction of the overpass is to be carried out in the near future. In order that construction can proceed, one of the following alternatives should be adopted:

A) Overhead Structure Supported on End-Bearing Piles Driven to Bedrock with Berms for Approach Fill:

This solution will necessitate approx. 31-ft. high embankments, which, in turn, require berms of about 100 - 120 ft. length for stability. The height of the berms are designed to be half of the heights of the embankments. Because of the required 120-ft. length of the end berms, the overhead will have a length of about 460 ft., the abutments being at around Stations 797+80 and 802+40. Spill-through type of abutments are recommended in order to maintain 2:1 side slopes for the approach fills.

Bedrock was observed around El. 475 - 467 ft., some 155 - 163 ft. below ground level. Piles should be driven to practical refusal, at around above elevations. The safe bearing pressure on the piles will be governed by the structural strength of the particular type of piles used.

Pile driving through the present fill will not be feasible due to the large amount of boulders (1 - 4 ft. diam.) in the embankments. Excavations of the existing fill down to the original ground will be necessary at the locations of pile driving. 2 horizontal to 1 vertical side slopes should be maintained for these excavations.

cont'd. /3 ...

June 29, 1925

A) (cont'd.) ...

The recommended sequence of pile driving and fill construction is as follows:

- 1.) Excavation of fill in pile driving locations.
- 2.) Pile driving and forming the pile caps.
- 3.) Refill of the excavations and construction of the berms to the designed length and height.
- 4.) Construction of the approach embankments to the final grade.

By adopting this method of crossing, a new detour will be necessary farther from the centre line of the highway; also, additional land should be expropriated to provide room for the long berms.

B) A Long Overhead Structure:

This solution calls for an overhead of approximately 1,250 ft. length in order to eliminate approach embankments higher than 19 ft. The locations of the two abutments will be at around Stations 794+30 and 806+80, respectively. The height of the approach fills at the abutments will be about 19 ft. Embankments of such heights are stable with very short berms or no berms at all, provided they are constructed with side slopes 2 horizontal to 1 vertical.

The present fill between Stations 794+30 and 806+80 will not be required any longer, so the material of this section could be removed and used for other purposes. Removal of the fill should precede pile driving. The bridge should also be supported on end-bearing piles or caissons, driven to bedrock as discussed under Section A).

Spill-through type of abutments are recommended.

C) Crossing the O.N.R. Tracks by means of a Subway:

This solution will require an approx. 160-ft. long bridge with spill-through abutments, supported on end-bearing piles on bedrock. Length of piles will be some 20 - 30 ft. shorter than those required for the overhead.

The approach cuts for the subway should be constructed with side slopes 3 horizontal to 1 vertical. It is estimated that the deepest cut at the crossing will be about 20 ft. A 1 - 2 ft. thick granular blanket or sand cushion is suggested to be used to protect the cuts.

cont'd. /4 ...

June 29, 1965

C) (cont'd.) ...

The ground water level should be lowered to below the bottom of the proposed cuts by provision of a deep drainage system to the Wabi River, some 2,000 ft. to the east.

In case of adopting the crossing by subway, the present fill must be completely removed.

It was agreed that the Bridge Office would study these schemes in further detail.

The above recommendations for the various schemes are preliminary and for estimating purposes only. Further detailed work is required on a chosen scheme for final design.

KYL/Md...


K. Y. Lo,
SUPERVISING FOUNDATION ENGINEER

cc: Mr. G. M. Sinclair

Foundations Office (2)
Gen. Files

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

July 7, 1960.

FOUNDATION INVESTIGATION REPORT
by Geocon, Limited.

Attention: Mr. S. McCombie.

Re: Proposed Tri-Town Bypass Structures -
W.P. 103-60 - W.P. 104-60, W.P. 124-60,
Highway 11, New Liskeard, District 14.

Accompanying this memo, is the above mentioned report submitted by Geocon, Ltd. We have reviewed the factual data contained in this report, and have found the computations and conclusions to be correct and sound. We believe that the recommendations given, will be adequate for your future design work.

In connection with the Ontario Northland Railway Overhead, we would like to emphasize that very serious consideration should be given to the proposal to change the Overhead to a Subway. We think that by this change, not only a more logical engineering solution would be achieved, considering the very unfavourable soil conditions, but that a substantial saving of money, would also result. It should not be forgotten that large additional areas of land will be required, and therefore, will have to be purchased, in order to enable the building of the berms for the approach fills.

Should there be any questions concerning the contents of this report that you would like to discuss, please feel free to call on our Office.

As/MdEF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
G. K. Hunter
R. S. Chapman
E. R. Saint
A. Watt

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.

Per:

for *Don de Lant (eng.)*
(A. Stermac,
FOUNDATIONS OFFICE ENGR.)

Foundations Office
Gen. Files.

BA 1087-A

File Box 100

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

November 10, 1960.

REPORT BY GEOCON, LIMITED

60-F-322 C

Attention: Mr. S. McCombie.

Re: Reduction of Grade Line,
Proposed Tri-Town Bypass Structures,
W.P. 103-60, W.P. 104-60,
Highway 11,
New Liskeard, Ontario.

Attached, is additional information
pertaining to the above projects, contained in
a letter report submitted by Geocon, Limited.

For your information and files.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.

Per:



(A. G. Stermac,
FOUNDATIONS OFFICE ENGR.)

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2) ✓
H. A. Tregaskes
D. G. Ramsay
G. K. Hunter
R. S. Chapran
E. R. Saint
A. Watt
Foundations Office
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GEOCON LTD

HEAD OFFICE

120 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 8-7632

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-0041

1425 WEST PENDER ST.
VANCOUVER 6, B.C.
TEL. MU. 1-0006

Rexdale, Ontario,
October 28th, 1960.

Department of Highways, Ontario,
Materials and Research Section,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Reduction of Grade Line,
Proposed Tri-Town Bypass Structures,
W.P.103-60, W.P.104-60,
Highway 11,
New Liskeard, Ontario.

Dear Sirs:

Further to our report dated June 17th, 1960 and to our recent discussions, it is now understood that the grade line of the proposed bypass is to be lowered to underpass the Ontario Northland Railway. With the provision of an underpass at the Ontario Northland Railway, a cut for the roadway of the order of 15 feet below general ground level will be required between about the Wabi River crossing and the railway crossing.

As discussed in our report, it is considered that side slopes of 3 horizontal to 1 vertical for a roadway cut of the order of 15 to 20 feet in depth would be stable. The cut slopes should however, be protected against sloughing due to frost action and groundwater seepage, and erosion due to surface water runoff by a clean granular blanket at least 3 feet in thickness. Provision for drainage of the cut towards the Wabi River should be made at the base of the side slopes of the roadway.

With the revised grade line, cuts of approximately 15 feet below existing ground level will also be required at the top of the Wabi River banks. The previous grade line which is given in our report was at about existing ground level at the top of the east bank and necessitated a cut about 6 feet deep at the top of the west bank of the river. The existing overall side slopes of the valley at the proposed crossing are of the order of 3.7 horizontal to 1 vertical on the west bank and 3.3 horizontal to 1 vertical on the east bank.

Department of Highways, Ontario,
October 28th, 1960,
Page 2.

As mentioned in the report, it is considered that both banks are generally stable, but for long term stability it is recommended that the east slope be trimmed as shown on Drawing S7025-6 in Appendix III of the report. This trimming would make the overall slope of the east bank approximately equal to the overall slope of the west bank.

We believe that this letter, together with our report, gives you the information that you require. If any further questions arise, we would be pleased if you would call us.

Yours very truly,

GEORGE LTD



JLS/dr
S7025

J. L. Szychuk, P. Eng.,
District Soils Engineer.

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ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Tove, **Date** February 24, 1961.
Bridge Engineer. **Subject** REVIEW OF PRELIMINARY DRAWING
From Materials & Research Section. **by:** Foundations Office.

Attention: Mr. B. Davis.

Re: Wabi River Bridge, Hwy. #11,
W.P. 103-60 - District #14.

In response to a request from your Mr. C. Grebski, we have reviewed the preliminary drawing of the above structure. Stability analyses of the river banks have been carried out, and your attention is drawn to the following comments (Ref. your Plan D 4811-P):-

1) Soil conditions at this site consist of a deep deposit of soft varved clay with a shallow desiccated (stiff) surface crust. These conditions are similar to the Big Pic River crossing.

2) During pile driving at the Pic River (East Pier), pore water pressures were measured in detail. Magnitudes of p.w.p. equal to values measured at the Pic River, have been used in an analysis of the East and West river bank slopes at the Wabi River with pier and abutment locations as shown on Plan - D 4811-P, and our results show the banks to be unstable during pile driving. (S.F. \approx 0.6).

3) Based upon the above findings, it is our recommendation that the West pier and East abutment locations be moved. Our analyses show that the safe locations (S.F. \approx 1.1) for the West and East footings are Chainages 817 + 80 and 820 + 30 (C of bearing). These chainage locations correspond to a shift of 35 feet at the West side and 15 feet at the East side.

4) Relocation of the pier and abutment, as outlined above necessitates an increase in the span of the through truss by 50 feet unless a centre pier in the river can be considered. In view of the long piling length required for support of foundations, it is our opinion that a single-span truss, 250 feet in length with no approach spans, should be considered for the river crossing.

5) Field measurements of p.w.p. induced during pile driving, should be carried out for the following reasons:-

a) The p.w.p. used in our analyses are based upon results obtained at the Pic River. It is possible that induced p.w.p. at the Wabi may be larger than those at the Pic, and should this be the case, stability of the banks can only be assured by controlled pile driving. Pile driving can be a continuous operation, but it might be necessary to drive piles at specified locations (i.e., drive alternately one pile at the South extremity of the group and then the next pile at the North extremity of the same group).

b) Recorded p.w.p. during pile driving operations in problem soils such as soft, compressible varved clays, will allow confirmation of stability analysis techniques, now used in our designs.

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The installation of piezometers and observations required during pile driving, can be carried out by a member of the Foundations Sub-section. We would require one month's notice in advance of start of construction to place the piezometers

LGS/MdeF

cc: Foundations Office
Gen. Files.

L. G. Soderman
L. G. Soderman,
PRINCIPAL FOUNDATION ENGINEER

GEOCON LTD

HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 6-7692

DISTRICT OFFICES

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

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Rexdale, Ontario,
June 17th, 1960.

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Soils and Foundation Engineer.

Re: Soil Conditions and Foundations,
Proposed Tri-Town Bypass Structures,
W.P. 103-60, W.P. 104-60, W.P. 124-60,
Highway 11,
New Liskeard, Ontario.

Dear Sirs:

This letter accompanies our detailed report covering the investigation carried out for the above sites.

We find that the area is generally covered by about 150 feet of laminated and varved silty clays of firm to stiff consistency. These clay strata are underlain by a thin layer of very dense till which overlies generally sound limestone bedrock.

As discussed in the report, it is considered that the stability of the existing Wabi River slopes is generally adequate.

It is recommended that the proposed bridge structure over the Wabi River be founded on end bearing "H" piles driven to bedrock. The overhead structure at the Ontario Northland Railway may be carried on spread footings founded in the approach berm fill, as discussed in the report.

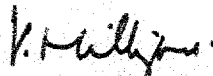
The construction of the proposed embankments at the Ontario Northland Railway will necessitate the use of berms for adequate stability. Design methods and recommended berm profiles are given in detail in the report.

Department of Highways, Ontario,
June 17th, 1960,
Page 2.

We believe that this report gives all the information for safe and economical foundation and embankment design. If we can be of any further service, however, please do not hesitate to call us.

Yours very truly,

GEOCON LTD



V. Milligan, P. Eng.,
Assistant Chief Engineer.

VM/dw
S7025

S7025
REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND FOUNDATIONS
PROPOSED TRI-TOWN BYPASS STRUCTURES
W.P. 103-60, W.P. 104-60, W.P. 124-60
HIGHWAY 11
NEW LISKEARD ONTARIO

Distribution:

- 10 copies - Department of Highways, Ontario,
Downsview, Ontario.
- 2 copies - Geocon Ltd,
Rexdale, Ontario.

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INDEX

	<u>Page</u>
Introduction	1
Summarized Soil Conditions	1
Discussion	
General	2
Wabi River Crossing	3
Ontario Northland Railway Overhead	9
Highway 65 Interchange	16
Conclusions and Recommendations	18
Personnel	20
Appendix I	
Procedure	
Site and Geology	
Soil Conditions	
Ontario Northland Railway Overhead and Wabi River Crossing Area	
Highway 65 Interchange Area	
Water Conditions	
Office Reports on Soil Exploration	
Appendix II	
Figures - Laboratory Testing	
Appendix III	
Figures - Engineering Analyses	
Drawings S7025-3 Recommended Embankment Cross Sections	
S7025-4 Recommended Approach Embankment Profile	
S7025-5 Typical Berm Section - Spread Footings	
S7025-6 Recommended Slopes - Wabi River Crossing	
Drawings in Pocket at rear of report:	
S7025-1 Boring Plan and Soil Stratigraphy	
S7025-2 Typical Procedure-Design of Proposed Embankment	

INTRODUCTION

Gecocon Ltd has been retained by the Department of Highways, Ontario (by letter of authorization dated December 16th, 1959) to investigate and report on the soil conditions at the site of three proposed Highway 11 bridge structures near New Liskeard, Ontario.

The object of the investigation was to determine and interpret the soil conditions as they affect the foundations of the three structures, the stability of the highway approach embankments and the stability of river banks at the Wabi River Crossing.

A description of the procedure, site and geology and detailed accounts of the soil and water conditions at the three sites (D.H.C., W.P. 103-60, W.P. 104-60 and W.P. 124-60), are given in Appendix I of this report. The results of the in-situ and laboratory testing are shown on the Office Reports on Soil Exploration in Appendix I and on the Figures in Appendix II. A drawing showing the borehole locations, together with the inferred soil stratigraphy for each site is located in the pocket at the rear of this report.

SUMMARIZED SOIL CONDITIONS

The site, in the area of the Ontario Northland Railway Overhead and Wabi River Crossing, is covered by a layer of topsoil up to about 1 foot in thickness. Beneath the topsoil is a stratum of laminated silty clay which extends down to about elevation 570. The laminated silty clay has been desiccated to a depth of about 8 to 15 feet, resulting in a generally very stiff to stiff upper crust. The lower portion of the stratum has a consistency of firm to stiff with depth. From the results of the strength tests, it is estimated that the stratum has been preconsolidated under a pressure of about 500 to 1000 pounds per square foot in excess of the existing overburden pressure. Underlying the laminated silty clay is an extensive stratum of varved silty clay and clayey silt, about 100 feet in thickness. The consis-

tency of the stratum is generally firm to stiff. It is estimated that the varved clay stratum has been preconsolidated by about 500 pounds per square foot in excess of the existing overburden pressure. A thin veneer of silty till, generally about 2 feet in thickness, underlies the varved silty clay and clayey silt across most of the area and this in turn is underlain by generally sound banded grey and white limestone bedrock.

The site at the Highway 65 Interchange has a surface layer of topsoil, up to 1 foot in thickness, which is underlain by a deep stratum of silty and sandy till. The upper portion of this till stratum has been desiccated and oxidized to a maximum depth of about 14 feet. The relative density of the till is generally loose to compact in the upper few feet and dense to very dense below this depth. North of about highway chainage 782+00 this stratum dips below a stratum of probably laminated silty clay which reaches a depth of 31 feet by chainage 784+00. The consistency of the silty clay stratum is estimated to be firm.

DISCUSSION

General

It is proposed to construct a new section of Highway 11 in order to bypass the Towns of Cobalt, Haileybury and New Liskeard in the County of Temiskaming, Ontario.

This so-called Tri-Town Bypass will necessitate the erection of three structures close to the Town of New Liskeard, Ontario. These are an Overhead at the Ontario Northland Railway, a bridge crossing at the Wabi River and an Interchange at Highway 65. The proposed vertical alignment will mean that approach embankments up to 33 feet in

General (continued)

height will be required at the first and last of the above mentioned sites. At the Wabi River Crossing, the vertical approach alignment necessitates shallow cuts.

The Bypass is proposed to be generally a two-lane highway. At the Highway 65 Interchange a third lane is to be provided.

For stability computation purposes the embankment top widths have been taken as 44 feet for a two-lane highway with side slopes of 2 horizontal to 1 vertical. Further, it has been assumed that all topsoil would be removed prior to construction and the embankments would be constructed of well-compacted granular fill.

Each of the three sites are discussed separately below:

WABI RIVER CROSSING

General:

The proposed Bypass will cross the Wabi River at about highway chainage 819+00. The site is shown on Drawing S7025-1, located in the pocket at the rear of this report, together with a section of the inferred soil stratigraphy and the proposed grade line obtained from Department of Highways, Ontario Profile No. F2480-9.

The subsoil across the site consists of about 50 feet of laminated silty clay underlain by about 100 feet of varved silty clay and clayey silt. The Wabi River has downcut a V-shaped river valley into these strata. This valley is about 60 feet in depth below the general ground level at about elevation 622. The existing side slopes of the valley are of the order of 3.7 horizontal to 1 vertical on the

General: (continued)

west bank and 3.3 horizontal to 1 vertical on the east bank. The upper portion of the clay strata has been desiccated to a depth of up to about 15 feet. The lower undesiccated portion of the strata has been preconsolidated to a pressure of possibly up to 1000 pounds per square foot in excess of the existing overburden pressure. The shear strength profile for the strata, determined by in-situ vane testing and laboratory strength testing is shown on Figure 7 of Appendix II. The shear strength line used for design, which is based on the in-situ vane tests, is also shown on this figure.

Approach Cuts:

The proposed grade line will necessitate a cut approximately 6 feet deep at the top of the west bank of the Wabi River. The grade line at the top of the east bank is approximately at existing ground level. No problem should exist in making shallow cuts to this depth in the desiccated crust of the laminated silty clay, though care will have to be taken to keep the cut as dry as possible during construction, in order to prevent it from being softened by the construction machinery. Provision should be made to effect drainage of the cut.

Slope Stability:

A total stress stability analysis was carried out for each bank of the Wabi River for the case when the approach cuts will be at grade. These analyses gave minimum factors of safety of about 1.3 and 1.5 against a deep circular arc type failure for the east and west banks respectively. The results of these analyses are summarized on Drawing S7025-6 in Appendix III. Based on these results it is considered that both banks will have generally adequate stability.

Slope Stability: (continued)

However, it is recommended that the east slope be trimmed as shown by the dotted line on Drawing S7025-6 in Appendix III. This trimming would increase the factor of safety to 1.4 and make the overall slope about 3.7 horizontal to 1 vertical, which would then be approximately equal to the overall slope of the west bank. This slope corresponds to an angle of about 15 degrees. Since the effective angle of shearing resistance, ϕ' , obtained from the slow undrained triaxial compression tests is about 21 degrees, it is considered that the factor of safety for long term stability of the slope is adequate if no rapid drawdown occurs. A rapid drawdown condition should not occur as the river flows into Lake Temiskaming about 1 mile downstream from the proposed crossing and the river level is thus controlled by the lake level, which is subject to only minor variations.

If the natural vegetation on the banks is removed during construction, it is recommended that the slopes be covered with a 2 foot granular blanket to prevent erosion. Rip-rap cover should be provided to about 5 feet above the existing river level which is about elevation 586, in order to prevent river scour of the banks and a consequent reduction in slope stability.

Foundations - Bridge Structure:

From computation purposes a 3-span bridge structure, with end spans of about 130 feet each and a centre span of about 180 feet, has been chosen in order to give an approximation of the load likely to be imposed on the foundations. It has been estimated that a two-lane structure of this size will impose abutment and pier loadings of about 500 and 1000 tons respectively.

Foundations - Bridge Structure: (continued)

Three types of foundations for the proposed structure have been examined in detail. These are spread footings, friction piles and end-bearing piles driven to bedrock.

Based on the measured shear strength, an allowable net bearing pressure of 1200 pounds per square foot may be used for spread or strip footings founded at least 4 feet below ground surface. Settlement computations were carried out, assuming both normally consolidated clay strata and strata which have been subject to a past pressure of at least 1000 pounds per square foot in excess of the present overburden pressure. For the pier locations these computations gave minimum and maximum total consolidation settlements of about 4 and 21 inches, while corresponding values for the abutments were 3 and 14 inches. The probable consolidation settlement, for spread footings of the size that would be required, will be between these limiting values, and it is estimated that it will be about 10 inches for the central piers and 7 inches for the abutments.

Friction type piles were next considered. Assuming an average shear strength of about 700 pounds per square foot within the probable pile depth, an adhesion between the soil and the pile of about 600 pounds per square foot will be developed along the pile shaft. Computations were carried out to determine the allowable load per pile and probable settlement of pile groups, for three types and lengths of friction piles. The results of these are shown in tabular form in Table 1.

TABLE I

7.

FRICTION PILE GROUP FOUNDATIONS

<u>Type of Pile</u>	<u>Depth of Penetra- tion</u>	<u>Computed Allowable Load per Pile</u>	<u>No. of Piles Req'd</u>		<u>Estimated Probable Total Consolidation Settlement of Pile Groups</u>	
			<u>Piers</u>	<u>Abutments</u>	<u>Piers</u>	<u>Abutments</u>
"H" Piles 12"x53 lb.	75 ft.	25 tons	40	20	12"	8"
Timber Piles 8-12 in. dia.	40 ft.	10 tons	100	50	10"	8"
Tubular Steel Piles 12 in. dia.	50 ft.	16 tons	63	32	12"	10"

NOTES:

- (i) The number of piles required at each abutment and pier location is based on the estimated loadings on these foundations of 500 and 1000 tons respectively.
- (ii) The estimated settlement of a pile group is based on a pile spacing of 3 to 4 times the individual pile diameter.
- (iii) The allowable load per pile has a factor of safety of about 3 against the ultimate computed failure load of the pile.

Foundations - Bridge Structure: (continued)

The total consolidation settlements of the various friction pile groups have been computed on the assumption, firstly, that the clay is normally consolidated and secondly, that it has been subjected to a past pressure of at least 1000 pounds per square foot in excess of the existing overburden pressure. The most probable total consolidation settlement of the pile groups will be between these limits and it is this value which is given in Table I. It is recommended that, if friction piles are used, a displacement type pile be employed in order to take advantage of the high thixotropic regain in strength of the remoulded clay strata.

Due to the fact that the horizontal permeability of the varved clay is much higher than the vertical permeability, it is difficult to determine the actual drainage path and hence compute the time-rate of consolidation settlement. However, it is estimated that 90 per cent of the consolidation settlement should take place within about 50 years. Based on this, a time-rate of settlement curve has been computed and is shown on Figure 1 of Appendix III.

Because of the settlements that will be experienced with spread footings and friction type piles, it is recommended that the bridge foundations be supported on "H" piles driven to refusal on bedrock. It is considered that 12 inch x 53 lb. "H" piles would be suitable. Each pile may carry an allowable load of 60 to 70 tons. The settlement with this type of foundation will be negligible.

General:

The proposed Tri-Town Bypass will cross the Ontario Northland Railway at about highway chainage 800+00. The area under investigation is shown on Drawing S7025-1, located in a pocket at the rear of this report, together with a section of the inferred soil stratigraphy along the proposed centre line and the proposed grade line obtained from Department of Highways, Ontario, Profile No. F2480-9.

The subsoil across the site is basically similar to that encountered at the Wabi River Crossing and is described elsewhere. The shear strength line used for design purposes is shown on Figure 7 of Appendix II.

Embankment Stability:

From Drawing S7025-1 it may be seen that the grade line, as proposed by the Department of Highways, Ontario, necessitates a maximum height of fill of about 33 feet above existing ground level at chainage 801+00. Based on this requirement, studies were carried out, assuming a top embankment width of 44 feet and side slopes of 2 horizontal to 1 vertical, in order to determine the factor of safety against sliding block and circular arc type failures. It is considered that the actual mode of failure of the embankment and the underlying laminated clay stratum will be between these two limiting cases.

Considering the stress-strain characteristics of the embankment material, the upper desiccated crust and the lower portion of the laminated silty clay, it is considered that the strain required to exceed the shear strength of the desiccated and lower portion of the laminated silty clay would be insufficient to mobilize full frictional effects in the granular embankment material. Hence for computation

Embankment Stability: (continued)

purposes, it has been assumed that no frictional effects would be mobilized in the embankment material. For this reason, the lateral pressure of the embankment in the sliding block analysis has been taken as that approaching the "at rest" condition.

The stability of a 35 foot high embankment was examined using the sliding block analysis, and it was found that a wedge extending to about a 24 foot depth was the most critical. The stability was checked by a circular arc total stress analysis. A summary of typical calculations for the two methods is given on Drawing S7025-2, located in a pocket at the rear of this report. For this height of embankment, it was computed that the minimum factor of safety against failure by spreading was about 0.7 for both the sliding block and circular arc analyses. Hence, it was concluded that a berm would be required for immediate embankment stability.

From the geometry of the proposed embankment and the measured shear strength profile at the site, the criterion of zero overstress beneath the embankment cannot be completely satisfied. As the area of overstress for a factor of safety of 1.3 will be small in relation to the overall width of the embankment and berm, it is considered that the stability in this case will be adequate and any increase in the berm size would be uneconomical.

Assuming that the angle of internal friction, ϕ , for the embankment material has a value of about 30 degrees, the coefficient of active earth pressure K_a is about 0.33. In the sliding block analyses, K_a has been assumed to have a value of 0.4, which is an approximate average value between the "active" and "at rest" earth pressure conditions.

Embankment Stability: (continued)

Stability computations were carried out for embankment heights of 22, 25 and 35 feet, in order to determine the berm sections required to give a minimum factor of safety of 1.3 against both sliding block and circular arc type failures. The results of the computations are summarized on Drawing S7025-3 in Appendix III. The berm heights and lengths required are plotted versus embankment height above existing general ground level at elevation 628. Typical embankment and berm sections are also shown on this drawing. In the stability computations it was assumed that the berm, as well as the embankment, would be constructed of well-compacted granular material.

Approach Embankment Stability:

Preliminary computations for an embankment height of 35 feet indicate that non-spill through type abutments would be unstable. Hence it is recommended that a sloped and bermed approach embankment be employed.

Stability calculations were carried out based on a 35 foot embankment height and a $1\frac{1}{2}$ horizontal to 1 vertical end slope. However, these showed that a berm in excess of 20 feet in height was required for adequate stability and this is considered excessive. Hence a 2 horizontal to 1 vertical end slope with a 20 foot high berm was decided on. This is similar to the lateral berm section required for this height, except that it was found possible to reduce the berm length to 95 feet, instead of 120 feet as with the lateral berm, considering an increase of about 10 per cent in the stability due to end effects.

Approach Embankment Stability: (continued)

The recommended berm section for a 35 foot high approach embankment is shown on Drawing S7025-4 in Appendix III. In the design of the berm section, the stability computations are based on general ground level at elevation 628 and the height of the berm shown is above elevation 628. Where the existing ground surface is below elevation 628, as is generally the case in the railway ditches, the toe of the berm should be moved back from the ditch a distance equal to that indicated by the graph on Drawing S7025-4, which relates this distance to the depth of ditch below elevation 628. This distance is required to counteract the loss of a portion of the desiccated crust below elevation 628 with a consequent reduction in passive resistance against a sliding block type failure. A typical section, assuming a 3 foot deep ditch, is shown on this drawing.

Foundations - Overhead Structure:

An overhead structure of the order of 380 feet in length will be required at the Ontario Northland Railway. At the abutment locations, approach embankments about 35 feet in height above existing ground level will be constructed to obtain the necessary clearance over the railway tracks. For stability, as discussed under the above heading, the maximum allowable end slope of the approach embankments is 2 horizontal to 1 vertical. To satisfy this condition, spill-through type abutments would be required for the overhead structure.

The structure may be founded on piles driven through the approach embankment and berm section. Due to the possibility of lateral strain in the embankment and in the underlying clay strata following the placing of the embankment material, it is recommended that if this type of foundation is decided upon, the piles should be driven following completion of the approach embankment and berm

Foundations - Overhead Structure: (continued)

sections, in order to minimize any lateral movement of the foundation piles at the abutment locations. To accommodate some possible horizontal movement over and above normal lateral consolidation, the overhead should be designed in simply supported spans and provision made to accommodate this movement. As the laminated silty clay and the grey silty clay layers in the underlying varved silty clay and clayey silt are sensitive to remoulding, non-displacement type piles such as steel "H" piles should be used for the overhead foundation to avoid possible instability of the embankment during pile driving due to remoulding and consequent decrease in the shearing strength of the clay strata. It is recommended, if a friction piled foundation is decided upon, that 12 inch x 53 lb. "H" piles, 75 feet in length, be used. As may be seen from Table I above, these piles may carry an allowable load of 25 tons per pile.

An alternative scheme to founding the overhead structure on friction piles is shown on Drawing S7025-5 in Appendix III. This is to found the overhead on spread footings founded in the granular embankment and berm material. In no case should the berm section be smaller in size than that shown on Drawing S7025-4 in Appendix III; however, the berm section will probably have to be slightly larger in order to provide sufficient granular material below and above the spread footings, as indicated on Drawing S7025-5. Based on the shear strength of the upper desiccated portion of the laminated silty clay, a net allowable bearing pressure of 1.0 tons per square foot may be used for design of spread footings founded in the granular fill and having a minimum depth of fill below them equal to about half the width of the footing.

Settlement:

Computations were carried out to determine the probable total consolidation settlement that will occur at the approach embankment and berm sections due to consolidation of the underlying clay strata. The computations, which are based on the results of the laboratory consolidation tests and assume a preconsolidation load of about 1000 pounds per square foot in excess of the existing overburden pressure, indicate that the maximum consolidation settlement below a 35 foot high embankment is of the order of 100 inches. At the abutment locations and at the toe of the berms, the total consolidation settlements are computed to be of the order of 80 and 3 inches, respectively. It is considered that the probable total consolidation settlement will be of the order of 70 per cent of the values given above. The probable consolidation settlement induced below the railway embankment will be of the order of 2 inches. A graph illustrating the settlements at the approach embankments is shown on Figure 2 of Appendix III.

It is estimated that for the pier arrangement shown on Drawing S7025-5 in Appendix III, the total consolidation settlement of a friction-piled foundation will not exceed the consolidation settlement of the embankment, which is given on Figure 2 of Appendix III. For this same pier arrangement and using spread footings founded in the berm section, the consolidation settlement in the areas of the central and outer piers will be about 5 and 2 inches, respectively, in excess of the consolidation settlement induced by embankment load only.

It is recommended that to accommodate part of the approach embankment and overhead structure settlements and to maintain sufficient clearance over the railway tracks, the grade line at the Ontario Northland Railway Overhead be raised by about 2 feet above that shown on Drawing S7025-1. This would necessitate approach embankments up to about 35 feet in height above existing ground level.

Settlement: (continued)

As at the Wabi River Crossing, the time-rate of settlement is difficult to compute. However, it is considered that 90 per cent of the consolidation settlement will occur in about 50 years. The estimated time-rate of settlement curve is shown on Figure 1 of Appendix III.

Due to the large consolidation settlements that an overhead structure will have to accommodate, it is suggested that consideration be given to realigning the grade of the proposed bypass in the region of the Ontario Northland Railway to pass below the railway. In order to provide sufficient clearance under the railway the underpass would necessitate a cut about 20 feet deep below general existing ground level. It is considered that slopes of 3 horizontal to 1 vertical, protected by a granular blanket, would be stable for a cut of this depth. The railway would have to be supported on a bridge structure which could be constructed without interruption of traffic. The bridge structure may be founded on friction piles or "H" piles driven to refusal on bedrock. In the case of friction piles some settlement will take place. Drainage of the cut would be a problem and provision would have to be made for either a pumping station or a deep drainage system to the Wabi River about 2000 feet distant.

Should a decision be made to proceed with an underpass, we would be pleased to discuss details of the structure at that time.

General:

It is proposed to erect a single span bridge structure over Highway 65 at about highway bypass chainage 780+00. The structure will have a skew span of about 83 feet and a roadway opening of about 52 feet. This will necessitate approach embankments up to about 27 feet in height, with a top width of about 56 feet in order to accommodate three traffic lanes.

The site is shown on Drawing S7025-1, located in a pocket at the rear of this report, together with a section of the inferred soil stratigraphy and the proposed grade line obtained from Department of Highways, Ontario, Profile No. F2480-9.

The subsoil south of about chainage 782+00 consists of a deep deposit of loose to very dense light brown to grey silty and sandy till. The upper portion of the stratum has been desiccated and oxidized resulting in a light brown colour. North of chainage 782+00, the till stratum dips below a stratum of probably laminated silty clay.

Foundations - Overpass Structure:

The stratum of loose to very dense light brown to grey silty and sandy till, which was encountered at ground surface in the proximity of Highway 65, is considered a suitable bearing stratum for spread footings for the proposed bridge structure. It is recommended that the proposed structure be founded on spread or strip footings in this stratum at maximum elevations of 642 and 654 at the north and south abutments, respectively. Based on the estimated relative density obtained from the penetration tests and on the results of four unconfined triaxial compression tests, which gave shear strengths ranging

Foundations - Overpass Structure: (continued)

from about 1500 to 4450 pounds per square foot, a net bearing pressure of 2.0 tons per square foot may be used for design purposes.

Under the above allowable load and considering the effects of the approach embankments, it is estimated that the total consolidation settlement of the structure will be negligible and will take place largely during construction.

The excavation for the footings should be kept dry during construction, and a thin layer of lean concrete should be placed immediately the excavation is down to grade to prevent softening of the till stratum.

Embankment Stability:

The approach embankments reach a maximum height of about 27 feet above existing ground level at about chainage 782+00. To the south of approximately this chainage the loose to very dense light brown to grey silty and sandy till is encountered at ground surface. The till is a good foundation stratum and if the embankments are constructed of well-compacted granular material, with side slopes of 2 horizontal to 1 vertical, there will be no stability problem. To the north of chainage 782+00, the till is overlain by a stratum of probably laminated silty clay. The thickness of the clay increases with increasing chainage and reaches a depth of about 30 feet by chainage 784+00. At this chainage the height of fill will be about 19 feet. Based on the results discussed for the approach embankments at the Ontario Northland Railway Overhead, it is considered that side slopes of 2 horizontal to 1 vertical will give an adequate factor of safety in this region, without the use of berms.

Embankment Stability: (continued)

The total consolidation settlement of the subsoil due to the imposed embankment loading will be negligible south of about chainage 782+00. North of this chainage, there will be settlement, but it will be generally uniform due to the fact that the height of embankment decreases as the depth of compressible clayey strata increases. It is estimated that the differential settlement will be small and will have no appreciable effect on the roadway surface.

Prior to construction of the embankment, all topsoil underlying the embankment area should be stripped. The side slopes of the embankment should be protected against surface water erosion.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by about 150 feet of generally firm to stiff laminated and varved silty clay strata overlying very dense till and limestone bedrock.
2. The water level at the site during the investigation was generally within a few feet of ground surface. The Wabi River was at about elevation 586.
3. Stability analyses show that the existing slopes at the Wabi River Crossing have generally an adequate factor of safety. The slopes should be covered with a granular blanket and rip-rap cover to prevent erosion and scour.

4. Computations show that an embankment with a top width of 44 feet and side slopes of 2 horizontal to 1 vertical can be constructed to a height of 21 feet without the use of berms. Above this height berms, as specified on Drawing S7025-3 in Appendix III, will have to be employed to maintain an adequate factor of safety against ultimate failure of the embankments.

Alternate methods of founding the proposed structures have been discussed in detail in the report. The methods considered most feasible are summarized below:

5. It is recommended that the bridge structure at the Wabi River be carried on "H" piles driven to refusal on bedrock. If this type of foundation is used, the bridge may be designed as a continuous structure.

6. The overhead structure at the Ontario Northland Railway may be carried on spread footings founded in the approach fill material. The individual spans of the structure should be simply supported and be able to accommodate some longitudinal and vertical differential movement between adjacent piers. Provision should be made for vertical jacking.

7. At the Ontario Northland Railway Overhead, for the grade shown on Drawing S7025-1, a sloped and bermed approach embankment, as shown on Drawing S7025-4 in Appendix III, will be necessary to provide an adequate factor of safety against ultimate failure.

8. Settlements of the structures and the approach embankments will take place due to consolidation of the laminated and varved silty clay strata. The magnitudes of these settlements are given throughout the report under the appropriate headings. The time-rate of consolidation is given on Figure 1 of Appendix III.

9. The effect of settlement of the approach embankments at the Ontario Northland Railway Overhead on the railway track is discussed in the report.

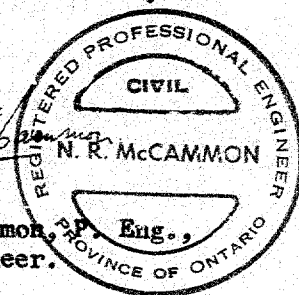
10. It is recommended that the overpass at the Highway 65 Interchange be supported on spread footings founded in the till stratum, as discussed in the report.

PERSONNEL

The field work was carried out under the supervision of Mr. N. R. McCammon. This report was written by Mr. N. R. McCammon, checked by Messrs. J. L. Seychuk and V. Milligan and reviewed by Mr. M. A. J. Matich.

NRMCC/dw
S7025

N. R. McCammon,
Soils Engineer.



APPENDIX I

Procedure

Site and Geology

Soil Conditions

Ontario Northland Railway Overhead and Wabi

River Crossing Area

Highway 65 Interchange Area

Water Conditions

Office Reports on Soil Exploration

PROCEDURE

The field work was commenced on December 14th, 1959 and completed on February 24th, 1960. A total of 12 boreholes, 2 with adjacent dynamic penetration tests, 5 additional dynamic penetration tests, and 2 wash borings were put down at the three sites. In-situ vane testing was carried out in 7 of the boreholes. The borings were put down in HX, NX and BX sizes to a maximum depth of about 160 feet, using skid-mounted standard machine drillrigs. Bedrock was proved in two boreholes by core drilling in AXT size to a maximum depth of 11.5 feet.

The locations of the boreholes together with the inferred soil stratigraphy at the three sites are shown on Drawing S7025-1 located in the pocket at the rear of this report. A detailed log of each boring put down in this investigation is given on the Office Reports on Soil Exploration in this Appendix.

The laboratory testing of soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto and the results are shown on the Office Reports on Soil Exploration in this Appendix and on the Figures in Appendix II. The samples remaining after testing will be stored until December 1st, 1960, at which time you will be contacted regarding their disposal.

Elevations are referred to Geodetic Datum and were obtained from Department of Highways, Ontario bench marks located in the area. Chainages are referred to the centre line of the proposed bypass, as located by the Department of Highways, Ontario.

SITE AND GEOLOGY

The proposed structures are located approximately one mile northwest of New Liskeard, in the County of Temiskaming, Ontario, along the route of the proposed Tri-Town Bypass.

From available geological information and previous work in this district, it is known that the area of investigation is located near the outskirts of a clay plain deposited during recent geological times. The clay is underlain by glacial till which rises above the clay deposits at the edge of the plain in this locality. Bedrock underlies the till and consists of a banded grey and white limestone.

SOIL CONDITIONS

For convenience in describing the soil conditions the overall site has been divided into two separate areas. The principal soil strata encountered in each of these areas are described separately below:

Ontario Northland Railway Overhead and Wabi River Crossing Area

Topsoil

Except for the river bed, a layer of dark brown clayey topsoil, up to 1 foot in thickness, extends across the two sites.

Very Stiff to Firm Grey Laminated Silty Clay

Below the topsoil in all boreholes, except borehole 6 in the Wabi River, is a stratum of laminated silty clay varying between about 15 and 54 feet in thickness. The lower boundary of this stratum is generally at about elevation 570 at the Wabi River Crossing and at about elevation 575 at the Ontario Northland Railway Overhead. The top portion of this stratum, to a depth of about 8 to 15 feet, is generally mottled brown and grey in colour, while below this depth the colour is mainly grey. In borehole 3 the mottled zone extends to a 22 foot depth.

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Very Stiff to Firm Grey Laminated Silty Clay (continued)

It is considered that this colour change is due to oxidation and desiccation of the upper portion of the stratum following a temporary drawdown of the water table. When in the natural state samples from this stratum generally exhibit a homogeneous structure. However, on drying a sample, the laminated structure becomes more distinct. The laminations consist of thin alternate medium grey and light grey silty clay layers of the order of 1/16 inch thick. The lighter coloured layers have a slightly higher silt content than the darker coloured layers. There are a few thin layers of light grey clayey silt in the stratum above about elevation 600.

The liquid limits obtained from the stratum generally varied between 46 and 85 with an average value of about 60. The plasticity index ranged generally from 24 to 58 with an average value of about 38. Typical values of the Atterberg limits are plotted on the plasticity chart on Figure 1 of Appendix II, together with the Casagrande "A" line. Based on this classification the stratum may be considered as an inorganic silty clay of generally high plasticity.

Wet unit weight determinations for the upper desiccated portion of the stratum gave an average value of about 116 pounds per cubic foot at a corresponding average moisture content of about 39 per cent. The lower portion gave an average wet unit weight of about 106 pounds per cubic foot at an average moisture content of about 61 per cent. For design purposes a wet unit weight of 115 pounds per cubic foot and a submerged unit weight of 52 pounds per cubic foot have been used throughout.

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Very Stiff to Firm Grey Laminated Silty Clay (continued)

The Liquidity Index, the ratio of moisture content minus plastic limit to plasticity index, has been computed to have an average value of 0.4 for the upper desiccated portion and an average value of 1.0 for the lower portion of the stratum.

One typical grain size distribution curve for the stratum is shown on Figure 2 in Appendix II. This indicates that the laminated silty clay contains about 90 per cent clay sizes.

The Activity of the laminated silty clay, which is defined as the plasticity index divided by the percentage of grain sizes less than 0.002 millimeters varies generally between 0.27 and 0.65. This is in the inactive zone for clays.

The shear strength of the stratum was determined by in-situ field vane testing and undrained triaxial compression tests carried out in the laboratory. Four 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 field vane tests are generally about 50 per cent higher than those given by the undrained triaxial compression tests on samples from the stratum. The in-situ vane test results are considered to give a better indication of the actual shear strength of the soil and have been utilized for design purposes.

In the upper desiccated portion of the stratum at the Ontario Northland Railway Overhead the in-situ vane shear strength varies between about 750 and 1200 pounds per square foot with an average value of about 900 pounds per square foot, while at the Wabi River

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Very Stiff to Firm Grey Laminated Silty Clay (continued)

Crossing, it varies generally between 850 and 3700 pounds per square foot with an average of 1800 pounds per square foot. The consistency of the crust is thus estimated to be stiff to firm at the O.N.R. Overhead and very stiff to firm at the Wabi River Crossing. Below the desiccated portion the shear strength ranges generally from 500 to 1250 pounds per square foot with an average value of about 800 pounds per square foot. From this the consistency is established to be firm to stiff.

The shear strengths obtained from in-situ vane testing and from undrained triaxial compression tests carried out on samples of this stratum, together with the shear strengths obtained for the underlying stratum, are plotted against elevation on Figure 7 in Appendix II. The shear strength line used for design is also given on this figure. It may be seen that the shear strength reaches a minimum value of about 500 pounds per square foot at about elevation 612. Below this elevation the shear strength increases with depth. The pattern of shear strength with depth suggests that the stratum has been preconsolidated.

During the period of the investigation the groundwater level was generally within 3 feet of ground surface. The mottled brown colouration, in the upper 8 to 15 feet of the stratum, resulting from oxidation indicates that desiccation in this portion of the stratum took place following a temporary lowering of the groundwater table. This has resulted in a generally stiff upper crust and preconsolidation of the lower portion of the clay due to the lowering of the groundwater level. A preconsolidation load of the order of 500 to 1000 pounds per square foot in excess of existing over-

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Very Stiff to Firm Grey Laminated Silty Clay (continued)

burden pressure may be computed for the portion of the clay below the desiccated crust, taking the present groundwater level as being within 3 feet of ground surface. By using a preconsolidation load of the order of 1000 pounds per square foot, it is possible to compute for the portion of the shear strength line, between about elevation 612 and 570, a $(c/p)_n$ value of 0.26. This value is in good agreement with the measured $(c/p)_n$ figure for clays of similar plasticity.

The sensitivity of the laminated silty clay, as determined by in-situ vane testing in the undisturbed and remoulded states ranges from about 3 to 7.

Three consolidation tests were carried out on samples obtained from different depths below ground level in the laminated silty clay. One of these samples was taken from the upper desiccated zone and the other two from the lower portion of the stratum. The resulting log pressure-void ratio curves are shown on Figures 8, 9, and 10 in Appendix II, together with a plot of the computed coefficients of consolidation, C_v , against log pressure for each test. The results from the consolidation tests are summarized in the table below. Based on the results given in this table average values of e_o , C_c and C_R of 1.71, 1.05 and 0.11 respectively have been used for computation purposes.

CONSOLIDATION TESTS - SUMMARY OF RESULTS

VII.

<u>B.H. No.</u>	<u>Sample Depth</u>	<u>Sample Elevn.</u>	<u>Stratum Descrip.</u>	<u>e_o</u>	<u>C_c</u>	<u>C_R</u>	<u>LL</u>	<u>PI</u>	<u>MC</u>
3	15.0	573.2	Desiccated Zone Laminated Silty Clay	0.82	0.20	0.05	59	35	28%
8	30.3	598.6	Laminated Silty Clay	1.60	0.78	0.13	78	51	58%
8	45.3	583.6	Laminated Silty Clay	1.83	1.07	0.10	77	54	65%
3	30.3	557.9	Varved Silty Clay and Clayey Silt (sample from Silty Clay Layer)	1.71	1.31	0.09	66	39	64%

In the above Table: -

- e_o = Initial Void Ratio
- C_c = Laboratory Compression Index
- C_R = Rebound Compression Index
- LL = Liquid Limit
- PI = Plasticity Index
- MC = Natural Moisture Content

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Firm to Stiff Varved Dark Grey Silty Clay and Light Grey Clayey Silt

At river bottom in borehole 6 and underlying the laminated silty clay in the remainder of the borings, is an extensive stratum of varved dark grey silty clay and light grey clayey silt. The upper surface of the stratum is generally between elevations 570 and 575. The stratum varies in thickness between about 100 and 110 feet. The separate layers of the varves are composed of dark grey silty clay and light grey clayey silt. These layers are generally distinct and horizontal above about elevation 530. Below this elevation they become less distinct, have an inclination of up to 30 degrees to the horizontal and usually become contorted with increasing depth. The dark grey silty clay layers generally range in thickness from about $1/8$ to 1 inches, though occasionally to $3\frac{1}{2}$ inches size. They comprise approximately 40 to 50 per cent of the total thickness of the stratum. The light grey clayey silt layers vary generally from about $1/4$ to $1\frac{1}{2}$ inches in thickness with a maximum thickness of about $2\frac{1}{2}$ inches being encountered. There appears to be no general trend in the increase and decrease in size of the individual layers.

The liquid limits obtained for the dark grey silty clay layers varied between about 44 and 73, while the plasticity index ranged generally from 33 to 47. The average liquid limit is about 65 and the average plasticity index about 40. Liquid limits were obtained for the light grey clayey silt layers ranging from 23 to 31 with an average value of 28, while the plasticity index varied between about 2 and 13 with an average value of about 6. On Figure 1 of Appendix II, typical values of

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Firm to Stiff Varved Dark Grey Silty Clay and Light Grey Clayey Silt (continued)

the Atterberg limits are plotted on the plasticity chart together with the Casagrande "A" line. The dark grey silty clay may thus be classified as an inorganic clay of generally high plasticity while the light grey clayey silt may be classified as an inorganic clay of generally low plasticity.

Wet unit weight determinations gave an average value of 113 pounds per cubic foot at corresponding average moisture contents of 59 per cent and 29 per cent for the dark grey silty clay and light grey clayey silt layers respectively. A wet unit weight of 115 pounds per cubic foot and a submerged unit weight of 52 pounds per cubic foot have been used for design purposes.

The liquidity index was computed to have an average value of 0.85 for the dark grey silty clay and 1.00 for the light grey clayey silt.

Two typical grain size distribution curves for each of the separate layers are plotted on Figure 3 in Appendix II. These show that the dark grey silty clay contains from about 80 to 90 per cent clay sizes and the light grey clayey silt contains from 22 to 26 per cent clay sizes.

The activity of the dark grey silty clay varies between about 0.36 and 0.59, which is in the inactive zone for clays.

Typical stress-strain curves obtained from the undrained triaxial compression tests are shown on Figure 6 of Appendix II. The undrained tests gave considerably lower shear strength values

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Firm to Stiff Varved Dark Grey Silty Clay and Light Grey Clayey Silt (continued)

than the in-situ vane tests, if it is assumed that the shear strength is half the measured undrained compressive strength. An examination of the mode of failure in the triaxial compression tests showed that samples generally failed by a spreading of a dark layer of a varve in the sample, rather than by an inclined shear plane. It may be shown that, for this mode of failure, the shear strength is a function of the thickness of the squeezed layer; approximate computations based on Jurgenson's hypotheses indicate that the ratio of the sample triaxial compressive strength to the shear strength of an individual thin lamination lies closer to unity than to two. Consequently, for design purposes the shear strength has been based on the in-situ vane tests. The shear strength line used for computation purposes is shown on Figure 7 of Appendix II. The pattern of shear strength with depth indicates that the stratum has possibly been subjected to a preconsolidation pressure of up to 500 pounds per square foot in excess of the existing overburden pressure. In general the shear strength increases with decreasing elevation from about 700 pounds per square foot at elevation 570 to about 1250 pounds per square foot at elevation 510. Based on these values, the consistency of the stratum is estimated to be firm to stiff. The increase in shear strength with depth below elevation 570 gives a (c/p) value of about 0.20. This value is in good agreement with measured $(c/p)_n$ values for materials of the same plasticity as the average for the silty clay and clayey silt layers.

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Firm to Stiff Varved Dark Grey Silty Clay and Light Grey Clayey Silt (continued)

A total of 9 undrained triaxial compression tests with pore pressure measurements were carried out on samples of the varved silty clay and clayey silt. The resulting plots of the Mohr stress circles gave an effective angle of shearing resistance, ϕ' , ranging between about 19 and 22 degrees and an effective cohesion, C' , ranging between about 350 and 800 pounds per square foot. It is suggested, for computation of long term stability, that C' be reduced to zero and the average angle of shearing resistance, ϕ' , of 21 degrees be used for design. This is indicated by the envelope shown on Figure 12 of Appendix II.

A consolidation test was carried out on one of the dark grey silty clay layers in the stratum and the resulting log pressure-void ratio curve is shown on Figure 11 of Appendix II. A plot of the computed coefficient of consolidation C_v , against log pressure for the test is also given on the figure. The results of this consolidation test are summarized in Table I. The compression index, C_c , obtained from the laboratory curve, was 1.31 and the rebound compression index, C_R , was 0.09.

Since it was not practicable to carry out a consolidation test on a clayey silt layer, values of 0.2 and 0.8 for C_c and e_o , respectively, were estimated based on the average moisture content, liquid limit and plasticity index of the clayey silt. For settlement computation purposes in the varved clay stratum, the average of the consolidation characteristics for the silty clay and clayey silt layers was used. The average values were taken as 0.62 and 1.25 for C_c and e_o respectively.

Ontario Northland Railway Overhead and Wabi River Crossing Area (cont'd)Very Dense Grey Silty Till

A thin veneer of grey silty till was found to underlie the varved silty clay and clayey silt in most boreholes. In general the thickness of the stratum varied between zero and 2.5 feet, although in borehole 7 it was penetrated for a depth of 13 feet. The stratum is essentially a silt with some subangular sand and gravel sizes scattered throughout it.

Two standard penetration tests carried out in the stratum gave "N" values of 95 and greater than 100 blows per foot indicating that the relative density of the stratum is very dense.

Bedrock

Bedrock was encountered below the grey silty till where it is present and below the varved silty clay and clayey silt elsewhere. It was proved by coring in AXT size in boreholes 2 and 4 for depths of 11.5 and 7.5 feet respectively. The upper 1 foot, approximately, of the rock appears to be fractured and weathered. Bedrock is a generally sound banded grey and white limestone.

Highway 65 Interchange AreaTopsoil

A layer of dark brown silty topsoil, about 1 foot in thickness, generally covers the area.

Highway 65 Interchange Area (continued)Probably Firm Laminated Silty Clay

Beneath the topsoil at dynamic penetration tests 17 and 18 a stratum of laminated silty clay was probably encountered. As inferred from the results of dynamic penetration tests 17 and 18, the thickness of the stratum is about 4 and 31 feet respectively, at these locations.

Based on an average dynamic penetration resistance of 5 to 6 blows per foot it is estimated that the consistency of the stratum is firm.

Loose to Very Dense Light Brown to Grey Silty and Sandy Till

Underlying the topsoil in boreholes 13 and 16 and probably in dynamic penetration tests, 14, 15 and 19 is a stratum of light brown to grey silty and sandy till. The results of dynamic penetration tests 17 and 18 indicate that the till underlies the silty clay at these locations. The maximum depth penetrated in the till was 64 feet in borehole 16. The upper 9 to 14 feet of the stratum is light brown in colour, probably as a result of oxidation and desiccation following lowering of the groundwater table. Below this depth it is grey in colour. The stratum consists essentially of subangular to subrounded sand and gravel sizes in a silt matrix. There are occasional boulders scattered throughout.

Four typical grain size distribution curves are shown on Figure 4 of Appendix II. These indicate that the stratum is comprised of from 24 to 63 per cent silt sizes, 40 to 64 per cent sand sizes and 1 to 36 per cent gravel sizes.

Highway 65 Interchange Area (continued)Loose to Very Dense Light Brown to Grey Silty and Sandy Till (cont'd)

The shear strength, as determined by unconfined triaxial compression tests on samples from the stratum, ranged from about 1500 to 4500 pounds per square foot with an average value of about 2750 pounds per square foot.

Standard penetration tests carried out in the stratum gave "N" values which varied between push resistance and greater than 100 blows per foot with an average value of about 60 blows per foot. Dynamic penetration tests gave values ranging from about 3 to greater than 100 blows per foot. In both the standard and dynamic penetration tests the lower values were recorded in the upper oxidized portion of the stratum. Based on these penetration values and on the measured compressive strength of the samples it is estimated that the relative density of the stratum in about the upper 5 feet is loose to compact and dense to very dense below this depth.

The average wet unit weight of the stratum was found to be about 140 pounds per cubic foot and the average natural moisture content about 9 per cent.

WATER CONDITIONS

Groundwater level observations were carried out in 8 of the boreholes during the course of the investigation. The stabilized water levels in these boreholes at the end of the field work are shown on Drawing S7025-1. The ice level in the Wabi River was at elevation 585.7.

The site was covered by from 1 to 4 feet of snow during the investigation. The frost penetration was up to about 6 inches below ground surface at the borehole locations.

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</u> <u>Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration</u> <u>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

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 07065 BORING # 1 DATUM GEODETIC CASING BK
 BORING DATE DEC 12, 1957 REPORT DATE JAN 23, 1958 COMPILED BY A.A. CHECKED BY J.H.
 SAMPLER HAMMER WT. 140 LBS DROP 30 INCHES PENETRATION RESISTANCES CONVERTED TO BLOWS OF #250 IN. LBS. ENERGY

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ BEST

SAMPLE TYPES

AS AUGER SAMPLE FS SOIL SAMPLE
 ST SLOTTED TUBE SO SLEEVE-OPEN
 WS WASHED SAMPLE SF SLEEVE-FAST VALVE
 DO DRIVE-OPEN FO THIN WALLED GLEN
 DF DRIVE-FAST VALVE RC ROCK CORE
 CS CHUCK SAMPLE

ABBREVIATIONS

V IN-SITU VANE TEST
 M MECHANICAL ANALYSIS
 U UNCONFINED COMPRESSION
 QC TRIAXIAL CONSOLIDATED QUICK
 QK TRIAXIAL QUICK
 Q TRIAXIAL SLOW
 WU UNIT WEIGHT
 K PERMEABILITY
 C CONSOLIDATION
 WL WATER LEVEL IN CASING
 WT WATER TABLE IN SOIL

SOIL PROFILE

SHEAR STRENGTH IN LBS/50 FT + VAM TESTS

650 600 750 1000 1250

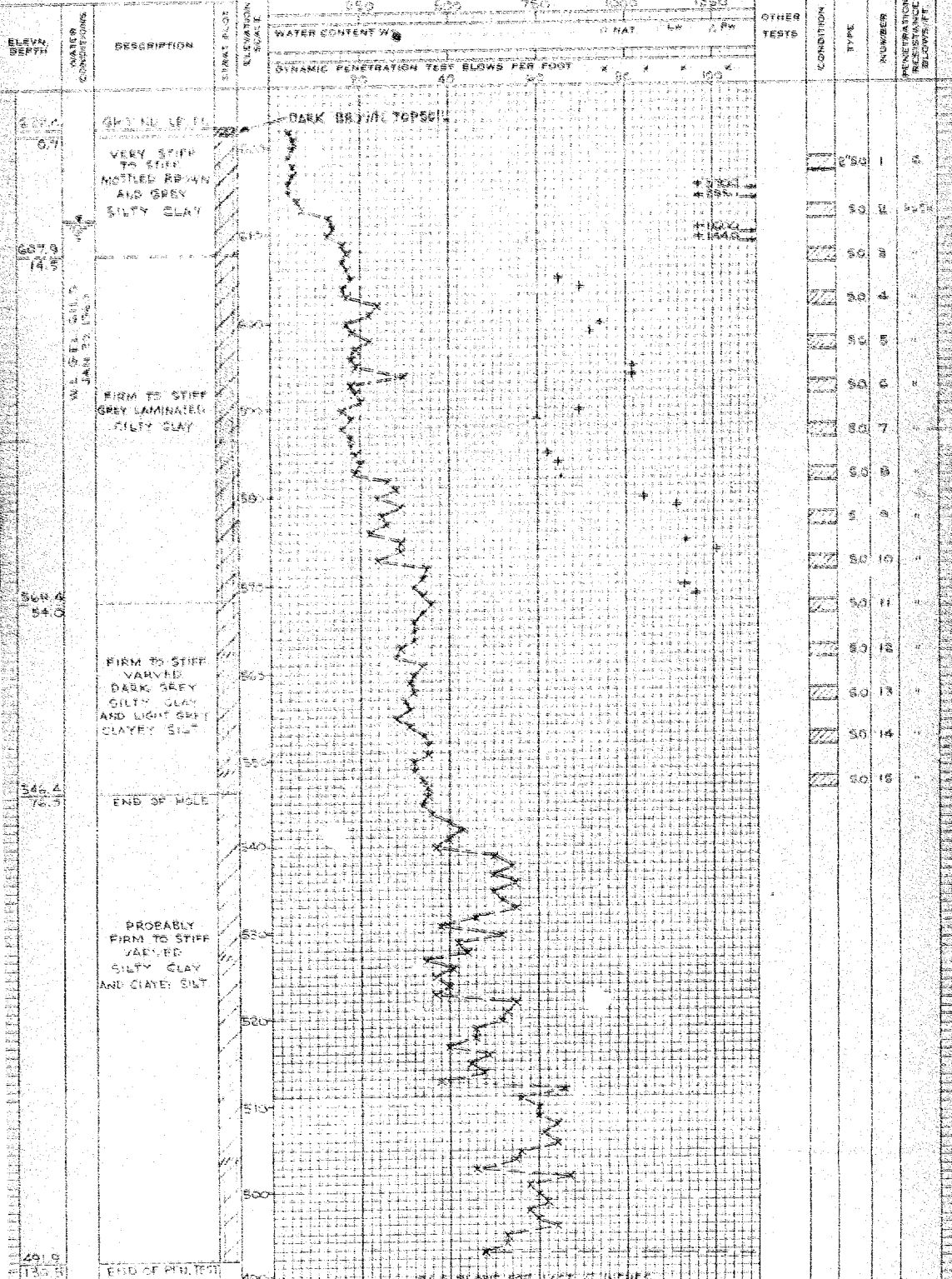
WATER CONTENT W_n NAT L_w P_w

DYNAMIC PENETRATION TEST BLOWS PER FOOT

50 100 150 200 250 300 350 400 450 500 550 600 650 700 750 800 850 900 950 1000 1050 1100 1150 1200 1250 1300 1350 1400 1450 1500 1550 1600 1650 1700 1750 1800 1850 1900 1950 2000 2050 2100 2150 2200 2250 2300 2350 2400 2450 2500 2550 2600 2650 2700 2750 2800 2850 2900 2950 3000 3050 3100 3150 3200 3250 3300 3350 3400 3450 3500 3550 3600 3650 3700 3750 3800 3850 3900 3950 4000 4050 4100 4150 4200 4250 4300 4350 4400 4450 4500 4550 4600 4650 4700 4750 4800 4850 4900 4950 5000 5050 5100 5150 5200 5250 5300 5350 5400 5450 5500 5550 5600 5650 5700 5750 5800 5850 5900 5950 6000 6050 6100 6150 6200 6250 6300 6350 6400 6450 6500 6550 6600 6650 6700 6750 6800 6850 6900 6950 7000 7050 7100 7150 7200 7250 7300 7350 7400 7450 7500 7550 7600 7650 7700 7750 7800 7850 7900 7950 8000 8050 8100 8150 8200 8250 8300 8350 8400 8450 8500 8550 8600 8650 8700 8750 8800 8850 8900 8950 9000 9050 9100 9150 9200 9250 9300 9350 9400 9450 9500 9550 9600 9650 9700 9750 9800 9850 9900 9950 10000

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION
 RESISTANCE
 BLOWS/FT



760 BLOW FOR LAST 4 INCHES

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT NO. 27000 BORING NO. 2 BATHYM. GEODETIC CASING 1/2" x 1/2"
 BORING DATE JAN. 14, 1962 REPORT DATE JAN. 27, 1962 COMPILED BY J. A. CHECKED BY M. H.
 SAMPLER HAMMER WT. 140 LBS DROP 25 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

DISBURSED
 RIG
 GOES
 LOST

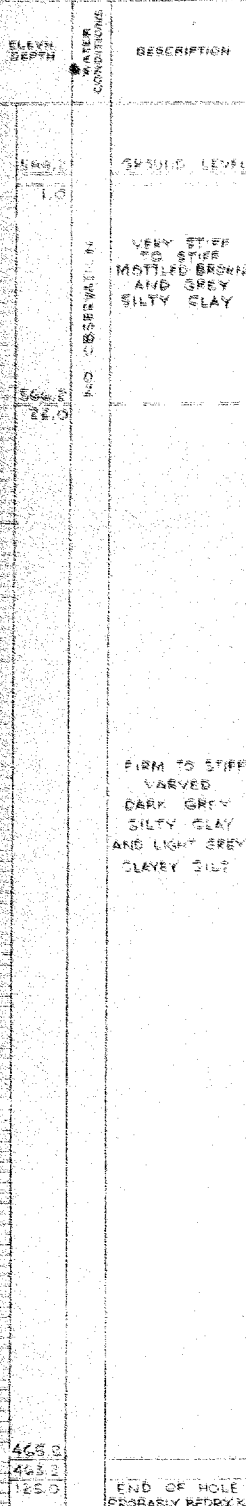
SAMPLE TYPES

1. SOIL SAMPLE
 2. SLEEVE OPEN
 3. SLEEVE FOOT VALVE
 4. THIN WALLED OPEN
 5. ROCK CORE

ABBREVIATIONS

W. IN SITU VANE TEST
 M. MECHANICAL ANALYSIS
 U. UNCONFINED COMPRESSION
 CC. TRIAXIAL CONSOLIDATED QUICK
 S. TRIAXIAL SLOW
 W. WET UNIT WEIGHT
 K. PERMEABILITY
 C. CONSOLIDATION
 WL. WATER LEVEL IN CASING
 WT. WATER TABLE IN SOIL

SOIL PROFILE



WATER CONTENT WT

CLAYEY SILT

DYNAMIC PENETRATION TEST BLOWS PER FOOT

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	WATER CONTENT	WET UNIT WEIGHT
1.0	1.0	28		
2.0	2.0	28		
3.0	3.0	28		
4.0	4.0	28		
5.0	5.0	28		
6.0	6.0	28		
7.0	7.0	28		
8.0	8.0	28		
9.0	9.0	28		
10.0	10.0	28		
11.0	11.0	28		
12.0	12.0	28		
13.0	13.0	28		
14.0	14.0	28		
15.0	15.0	28		
16.0	16.0	28		
17.0	17.0	28		
18.0	18.0	28		
19.0	19.0	28		
20.0	20.0	28		
21.0	21.0	28		
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37.0	37.0	28		
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39.0	39.0	28		
40.0	40.0	28		
41.0	41.0	28		
42.0	42.0	28		
43.0	43.0	28		
44.0	44.0	28		
45.0	45.0	28		
46.0	46.0	28		
47.0	47.0	28		
48.0	48.0	28		
49.0	49.0	28		
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92.0	92.0	28		
93.0	93.0	28		
94.0	94.0	28		
95.0	95.0	28		
96.0	96.0	28		
97.0	97.0	28		
98.0	98.0	28		
99.0	99.0	28		
100.0	100.0	28		

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57023 BORING # 4 DATUM GESSLOC CASING NO. 8X
 BORING DATE JAN 14, 1963 REPORT DATE JAN 23, 1963 COMPILED BY J.A. CHECKED BY J.M.H.
 SAMPLER HAMMER WT. 140 LBS SHSP 15 INCHES (PENETRATION) RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ BEST

SAMPLE TYPES

AS AUGER SAMPLE
 ST SLOTTED TUBE
 WS WASHED SAMPLE
 SO DRIVE OPEN
 GF DRIVE FOOT VALVE
 CS CHUCK SAMPLE
 FS SOIL SAMPLE
 SO SLEEVE OPEN
 SF SLEEVE FOOT VALVE
 FO THIN WALLED OPEN
 RS ROCK CORE

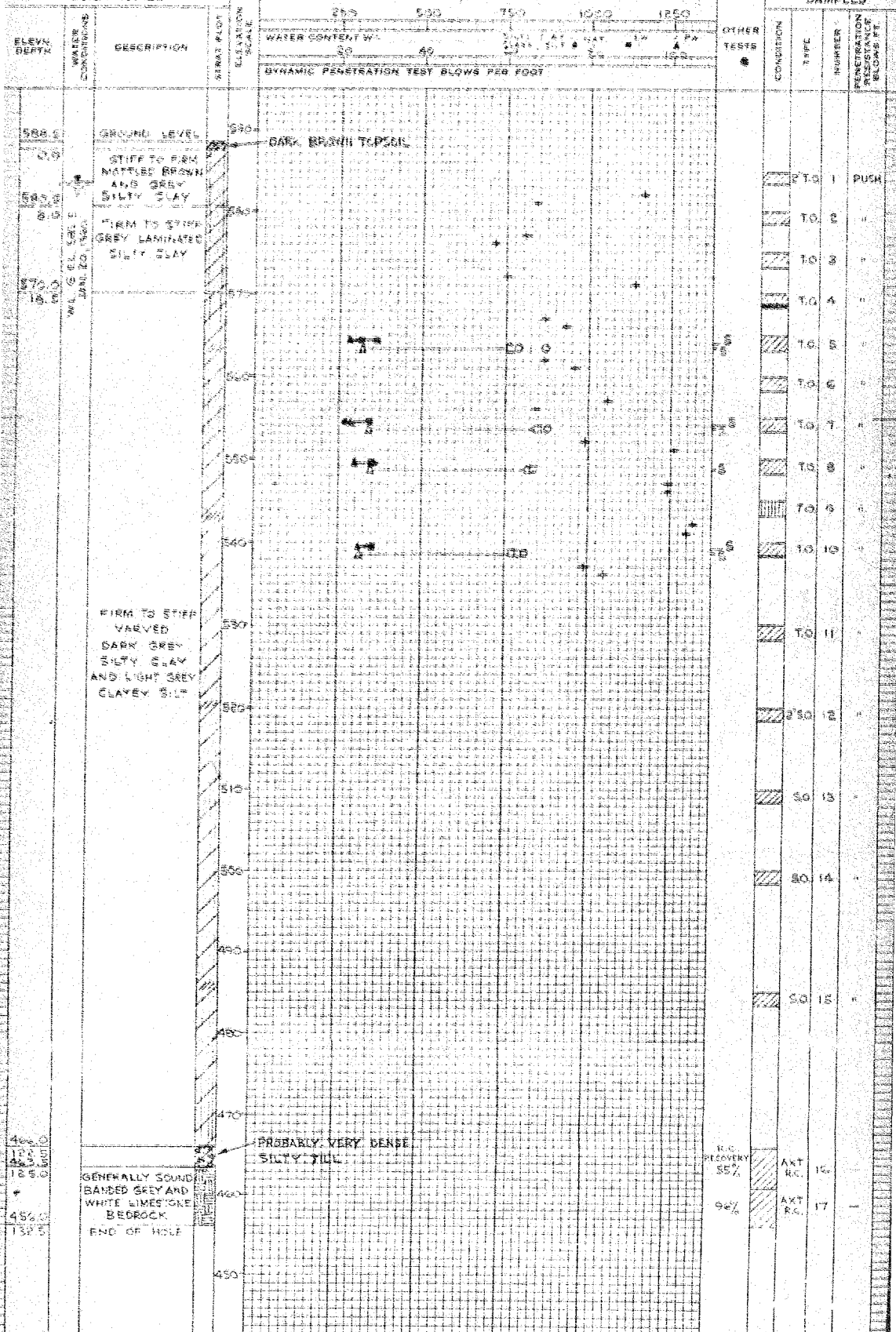
ABBREVIATIONS

V IN SITU VANE TEST
 M MECHANICAL ANALYSIS
 U UNCONFINED COMPRESSION
 C TRIAXIAL CONSOLIDATED QUICK
 Q TRIAXIAL QUICK
 S TRIAXIAL SLOW
 WET UNIT WEIGHT
 K PERMEABILITY
 C CONSOLIDATION
 WL WATER LEVEL IN CASING
 WT WATER TABLE IN SOIL

SOIL PROFILE

SHAR STRENGTH IN LBS / SQ FT + VANE TEST

SAMPLES



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

**DISTURBED
FAIR
GOOD
LOST**

SAMPLE FORM

五、六、七、八、九、十、十一、十二、十三、十四、十五、十六、十七、十八、十九、二十、二十一、二十二、二十三、二十四、二十五、二十六、二十七、二十八、二十九、三十、三十一、三十二、三十三、三十四、三十五、三十六、三十七、三十八、三十九、四十、四十一、四十二、四十三、四十四、四十五、四十六、四十七、四十八、四十九、五十、五十一、五十二、五十三、五十四、五十五、五十六、五十七、五十八、五十九、六十、六十一、六十二、六十三、六十四、六十五、六十六、六十七、六十八、六十九、七十、七十一、七十二、七十三、七十四、七十五、七十六、七十七、七十八、七十九、八十、八十一、八十二、八十三、八十四、八十五、八十六、八十七、八十八、八十九、九十、九十一、九十二、九十三、九十四、九十五、九十六、九十七、九十八、九十九、一百。

(Note: The above transcription is based on the visible characters in the image, which appear to be a sequence of numbers or small characters arranged in columns.)

V - INSIDE VANE TIP
 M - MECHANICAL AN
 U - UNCONFINED CO
 C - TRIAXIAL CONSC
 Q - TRIAXIAL QUICK
 L - TRIAXIAL SLOW

ABBREVIATIONS

D. WET UNIT WEIGHT
 E. PERMEABILITY
 F. CONSOLIDATION
 G. WATER LEVEL IN CASE
 H. WATER TABLE IN SOIL

SOIL PROFILE

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	ELEVATION SCALE	WATER CONTENT Wt. %				SHRINKAGE Wt. %				OTHER	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS FT.
				W	L	P	A	W	L	P	A					
DYNAMIC PENETRATION TEST BLOWS PER FOOT																
587.8 0.8		GROUND LEVEL	590	DARK BROWN TOPSOIL												
577.8 10.0		VERY STIFF TO B.M. MOTTLED BROWN AND GREY SILTY CLAY	580													
567.8 18.0		FIRM LAMINATED SILTY CLAY	570													
			560													
			550													
			540													
			530													
			520													
			510													
			500													
			490													
			480													
			470													
			460													
			450													
			440													
			430													
			420													
			410													
			400													
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			380													
			370													
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			200													
			190													
			180													
			170													
			160													
			150													
			140													
			130													
			120													
			110													
			100													
			90													
			80													
			70													
			60													
			50													
			40													
			30													
			20													
			10													
			0													
42.6 127.7		END OF HOLE PROBABLY BEDROCK	430	PROBABLY VERY DENSE SILTY TILL												

OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

TYPES

REVIAT

3.4.2.1.1

**DISTURBED
FAIR
GOOD
LOST**

五

62

200

Figure 1

10

0.000000

6. (b)

10

1999, 2000, 2001, 2002, 2003, 2004, 2005, 2006, 2007, 2008, 2009, 2010, 2011, 2012, 2013, 2014, 2015, 2016, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025, 2026, 2027, 2028, 2029, 2030, 2031, 2032, 2033, 2034, 2035, 2036, 2037, 2038, 2039, 2040, 2041, 2042, 2043, 2044, 2045, 2046, 2047, 2048, 2049, 2050, 2051, 2052, 2053, 2054, 2055, 2056, 2057, 2058, 2059, 2060, 2061, 2062, 2063, 2064, 2065, 2066, 2067, 2068, 2069, 2070, 2071, 2072, 2073, 2074, 2075, 2076, 2077, 2078, 2079, 2080, 2081, 2082, 2083, 2084, 2085, 2086, 2087, 2088, 2089, 2090, 2091, 2092, 2093, 2094, 2095, 2096, 2097, 2098, 2099, 2100, 2101, 2102, 2103, 2104, 2105, 2106, 2107, 2108, 2109, 2110, 2111, 2112, 2113, 2114, 2115, 2116, 2117, 2118, 2119, 2120, 2121, 2122, 2123, 2124, 2125, 2126, 2127, 2128, 2129, 2130, 2131, 2132, 2133, 2134, 2135, 2136, 2137, 2138, 2139, 2140, 2141, 2142, 2143, 2144, 2145, 2146, 2147, 2148, 2149, 2150, 2151, 2152, 2153, 2154, 2155, 2156, 2157, 2158, 2159, 2160, 2161, 2162, 2163, 2164, 2165, 2166, 2167, 2168, 2169, 2170, 2171, 2172, 2173, 2174, 2175, 2176, 2177, 2178, 2179, 2180, 2181, 2182, 2183, 2184, 2185, 2186, 2187, 2188, 2189, 2190, 2191, 2192, 2193, 2194, 2195, 2196, 2197, 2198, 2199, 2200, 2201, 2202, 2203, 2204, 2205, 2206, 2207, 2208, 2209, 2210, 2211, 2212, 2213, 2214, 2215, 2216, 2217, 2218, 2219, 2220, 2221, 2222, 2223, 2224, 2225, 2226, 2227, 2228, 2229, 2230, 2231, 2232, 2233, 2234, 2235, 2236, 2237, 2238, 2239, 2240, 2241, 2242, 2243, 2244, 2245, 2246, 2247, 2248, 2249, 2250, 2251, 2252, 2253, 2254, 2255, 2256, 2257, 2258, 2259, 2260, 2261, 2262, 2263, 2264, 2265, 2266, 2267, 2268, 2269, 2270, 2271, 2272, 2273, 2274, 2275, 2276, 2277, 2278, 2279, 2280, 2281, 2282, 2283, 2284, 2285, 2286, 2287, 2288, 2289, 2290, 2291, 2292, 2293, 2294, 2295, 2296, 2297, 2298, 2299, 2300, 2301, 2302, 2303, 2304, 2305, 2306, 2307, 2308, 2309, 2310, 2311, 2312, 2313, 2314, 2315, 2316, 2317, 2318, 2319, 2320, 2321, 2322, 2323, 2324, 2325, 2326, 2327, 2328, 2329, 2330, 2331, 2332, 2333, 2334, 2335, 2336, 2337, 2338, 2339, 2340, 2341, 2342, 2343, 2344, 2345, 2346, 2347, 2348, 2349, 2350, 2351, 2352, 2353, 2354, 2355, 2356, 2357, 2358, 2359, 2360, 2361, 2362, 2363, 2364, 2365, 2366, 2367, 2368, 2369, 2370, 2371, 2372, 2373, 2374, 2375, 2376, 2377, 2378, 2379, 2380, 2381, 2382, 2383, 2384, 2385, 2386, 2387, 2388, 2389, 2390, 2391, 2392, 2393, 2394, 2395, 2396, 2397, 2398, 2399, 2400, 2401, 2402, 2403, 2404, 2405, 2406, 2407, 2408, 2409, 2410, 2411, 2412, 2413, 2414, 2415, 2416, 2417, 2418, 2419, 2420, 2421, 2422, 2423, 2424, 2425, 2426, 2427, 2428, 2429, 2430, 2431, 2432, 2433, 2434, 2435, 2436, 2437, 2438, 2439, 2440, 2441, 2442, 2443, 2444, 2445, 2446, 2447, 2448, 2449, 2450, 2451, 2452, 2453, 2454, 2455, 2456, 2457, 2458, 2459, 2460, 2461, 2462, 2463, 2464, 2465, 2466, 2467, 2468, 2469, 2470, 2471, 2472, 2473, 2474, 2475, 2476, 2477, 2478, 2479, 2480, 2481, 2482, 2483, 2484, 2485, 2486, 2487, 2488, 2489, 2490, 2491, 2492, 2493, 2494, 2495, 2496, 2497, 2498, 2499, 2500, 2501, 2502, 2503, 2504, 2505, 2506, 2507, 2508, 2509, 2510, 2511, 2512, 2513, 2514, 2515, 2516, 2517, 2518, 2519, 2520, 2521, 2522, 2523, 2524, 2525, 2526, 2527, 2528, 2529, 2530, 2531, 2532, 2533, 2534, 2535, 2536, 2537, 2538, 2539, 2540, 2541, 2542, 2543, 2544, 2545, 2546, 2547, 2548, 2549, 2550, 2551, 2552, 2553, 2554, 2555, 2556, 2557, 2558, 2559, 2560, 2561, 2562, 2563, 2564, 2565, 2566, 2567, 2568, 2569, 2570, 2571, 2572, 2573, 2574, 2575, 2576, 2577, 2578, 2579, 2580, 2581, 2582, 2583, 2584, 2585, 2586, 2587, 2588, 2589, 2590, 2591, 2592, 2593, 2594, 2595, 2596, 2597, 2598, 2599, 2600, 2601, 2602, 2603, 2604, 2605, 2606, 2607, 2608, 2609, 2610, 2611, 2612, 2613, 2614, 2615, 2616, 2617, 2618, 2619, 2620, 2621, 2622, 2623, 2624, 2625, 2626, 2627, 2628, 2629, 2630, 2631, 2632, 2633, 2634, 2635, 2636, 2637, 2638, 2639, 2640, 2641, 2642, 2643, 2644, 2645, 2646, 2647, 2648, 2649, 2650, 2651, 2652, 2653, 2654, 2655, 2656, 2657, 2658, 2659, 2660, 2661, 2662, 2663, 2664, 2665, 2666, 2667, 2668, 2669, 2670, 2671, 2672, 2673, 2674, 2675, 2676, 2677, 2678, 2679, 2680, 26

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Figure 1

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$\frac{d}{dt} \left(\frac{\partial L}{\partial \dot{x}} \right) = \frac{\partial L}{\partial x}$

ABBREVIATIONS

VALUING

COMPRESSION
SLIP-ON-CLIP

W

1997, 1998, 1999, 2000, 2001, 2002, 2003, 2004, 2005, 2006, 2007, 2008, 2009, 2010, 2011, 2012, 2013, 2014, 2015, 2016, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025, 2026, 2027, 2028, 2029, 2030, 2031, 2032, 2033, 2034, 2035, 2036, 2037, 2038, 2039, 2040, 2041, 2042, 2043, 2044, 2045, 2046, 2047, 2048, 2049, 2050, 2051, 2052, 2053, 2054, 2055, 2056, 2057, 2058, 2059, 2060, 2061, 2062, 2063, 2064, 2065, 2066, 2067, 2068, 2069, 2070, 2071, 2072, 2073, 2074, 2075, 2076, 2077, 2078, 2079, 2080, 2081, 2082, 2083, 2084, 2085, 2086, 2087, 2088, 2089, 2090, 2091, 2092, 2093, 2094, 2095, 2096, 2097, 2098, 2099, 2100, 2101, 2102, 2103, 2104, 2105, 2106, 2107, 2108, 2109, 2110, 2111, 2112, 2113, 2114, 2115, 2116, 2117, 2118, 2119, 2120, 2121, 2122, 2123, 2124, 2125, 2126, 2127, 2128, 2129, 2130, 2131, 2132, 2133, 2134, 2135, 2136, 2137, 2138, 2139, 2140, 2141, 2142, 2143, 2144, 2145, 2146, 2147, 2148, 2149, 2150, 2151, 2152, 2153, 2154, 2155, 2156, 2157, 2158, 2159, 2160, 2161, 2162, 2163, 2164, 2165, 2166, 2167, 2168, 2169, 2170, 2171, 2172, 2173, 2174, 2175, 2176, 2177, 2178, 2179, 2180, 2181, 2182, 2183, 2184, 2185, 2186, 2187, 2188, 2189, 2190, 2191, 2192, 2193, 2194, 2195, 2196, 2197, 2198, 2199, 2200, 2201, 2202, 2203, 2204, 2205, 2206, 2207, 2208, 2209, 2210, 2211, 2212, 2213, 2214, 2215, 2216, 2217, 2218, 2219, 2220, 2221, 2222, 2223, 2224, 2225, 2226, 2227, 2228, 2229, 2230, 2231, 2232, 2233, 2234, 2235, 2236, 2237, 2238, 2239, 2240, 2241, 2242, 2243, 2244, 2245, 2246, 2247, 2248, 2249, 2250, 2251, 2252, 2253, 2254, 2255, 2256, 2257, 2258, 2259, 2260, 2261, 2262, 2263, 2264, 2265, 2266, 2267, 2268, 2269, 2270, 2271, 2272, 2273, 2274, 2275, 2276, 2277, 2278, 2279, 2280, 2281, 2282, 2283, 2284, 2285, 2286, 2287, 2288, 2289, 2290, 2291, 2292, 2293, 2294, 2295, 2296, 2297, 2298, 2299, 2300, 2301, 2302, 2303, 2304, 2305, 2306, 2307, 2308, 2309, 2310, 2311, 2312, 2313, 2314, 2315, 2316, 2317, 2318, 2319, 2320, 2321, 2322, 2323, 2324, 2325, 2326, 2327, 2328, 2329, 2330, 2331, 2332, 2333, 2334, 2335, 2336, 2337, 2338, 2339, 2340, 2341, 2342, 2343, 2344, 2345, 2346, 2347, 2348, 2349, 2350, 2351, 2352, 2353, 2354, 2355, 2356, 2357, 2358, 2359, 2360, 2361, 2362, 2363, 2364, 2365, 2366, 2367, 2368, 2369, 2370, 2371, 2372, 2373, 2374, 2375, 2376, 2377, 2378, 2379, 2380, 2381, 2382, 2383, 2384, 2385, 2386, 2387, 2388, 2389, 2390, 2391, 2392, 2393, 2394, 2395, 2396, 2397, 2398, 2399, 2400, 2401, 2402, 2403, 2404, 2405, 2406, 2407, 2408, 2409, 2410, 2411, 2412, 2413, 2414, 2415, 2416, 2417, 2418, 2419, 2420, 2421, 2422, 2423, 2424, 2425, 2426, 2427, 2428, 2429, 2430, 2431, 2432, 2433, 2434, 2435, 2436, 2437, 2438, 2439, 2440, 2441, 2442, 2443, 2444, 2445, 2446, 2447, 2448, 2449, 2450, 2451, 2452, 2453, 2454, 2455, 2456, 2457, 2458, 2459, 2460, 2461, 2462, 2463, 2464, 2465, 2466, 2467, 2468, 2469, 2470, 2471, 2472, 2473, 2474, 2475, 2476, 2477, 2478, 2479, 2480, 2481, 2482, 2483, 2484, 2485, 2486, 2487, 2488, 2489, 2490, 2491, 2492, 2493, 2494, 2495, 2496, 2497, 2498, 2499, 2500, 2501, 2502, 2503, 2504, 2505, 2506, 2507, 2508, 2509, 2510, 2511, 2512, 2513, 2514, 2515, 2516, 2517, 2518, 2519, 2520, 2521, 2522, 2523, 2524, 2525, 2526, 2527, 2528, 2529, 2530, 2531, 2532, 2533, 2534, 2535, 2536, 2537, 2538, 2539, 2540, 2541, 2542, 2543, 2544, 2545, 2546, 2547, 2548, 2549, 2550, 2551, 2552, 2553, 2554, 2555, 2556, 2557, 2558, 2559, 2560, 2561, 2562, 2563, 2564, 2565, 2566, 2567, 2568, 2569, 2570, 2571, 2572, 2573, 2574, 2575, 2576, 2577, 2578, 2579, 2580, 2581, 2582, 2583, 2584, 2585, 2586, 2587, 2588, 2589, 2590, 2591, 2592, 2593, 2594, 2595, 2596, 2597, 2598, 2599, 2600, 2601, 2602, 2603, 2604, 2605, 2606, 2607, 2608, 2609, 2610, 2611, 2612, 2613, 2614, 2615, 2616, 2617, 2618, 2619, 2620, 2621, 2622, 2623, 2624, 2625, 2626, 2627, 2628, 2629, 2630, 2631, 2632, 2633, 2634, 2635, 2636, 2637, 2638, 2639, 2640, 2641, 2642, 2643, 2644, 2645, 2646, 2647, 2648, 2649, 2650, 2651, 2652, 2653, 2654, 2655, 2656, 2657, 2658, 2659, 2660, 2661, 2662, 2663, 2664, 2665, 2666, 2667, 2668, 2669, 2670, 2671, 2672, 2673, 2674, 2675, 2676, 2677, 2678, 26

$\mu_1 = \frac{1}{2} \left(\frac{1}{\mu_2} + \frac{1}{\mu_3} \right)$

$\lambda_1 = \sqrt{2}(\cos\theta + i\sin\theta)$, $\lambda_2 = \sqrt{2}(\cos\theta - i\sin\theta)$

● 中国医药集团总公司

CONSOLIDATION

WATER LEVEL IN C

3. WATER TABLE 12.1

FAMILY

SOIL PROFILE

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

10

ABBREVIATIONS

Journal of Management Education 30(6)br/>
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 10.1177/0095687406288111
 jme.sagepub.com
 DOI: 10.1177/0095687406288111

<input type="checkbox"/>	DISTURBED
<input type="checkbox"/>	FAIR
<input type="checkbox"/>	GOOD
<input checked="" type="checkbox"/>	LOST

AS AUGER SAMPLE
ST SLOTTED TUBE
WS WASHED SAMPLE
OO DRIVE-ON
VF DRIVE-FOOT VALV
CS CHUCK SAMPLE

75 - FOM SAMPLE
90 - SLEEVE OPEN
95 - SLEEVE FOOT VALVE
100 - THIN WALLED OPEN
RE - ROCK CORE

V - IN SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATION
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW

ABBREVIATIONS

NET UNIT WEIGHT
PERMEABILITY
TENSILE STRENGTH

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE			SHEAR STRENGTH IN LBS / SQ FT - VANE TESTS					SAMPLES					
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	DIAPYLO ELEVATION SCALE	WATER CONTENT W					OTHER TESTS	CONDITION	TYPE	NUMBER	RESISTANCE BLOWS / FT
				250	500	750	1000	1250					
				WATER CONTENT W					O NAT PLW Δ Pw				
				DYNAMIC PENETRATION TEST BLOWS PER FOOT									
462.0	NO OBSERVATION	GROUND LEVEL	462.0										
461.5		FIRM MOTTLED BROWN AND GREY SILTY CLAY	461.5	DARK BROWN TOPSOIL									
461.0			461.0										
460.5			460.5										
460.0			460.0										
459.5			459.5										
459.0		FIRM TO STIFF LAMINATED GREY SILTY CLAY	459.0										
458.5			458.5										
458.0			458.0										
457.5			457.5										
457.0	457.0												
456.5	FIRM TO STIFF VARIEVED DARK GREY SILTY CLAY AND LIGHT GREEN CLAYEY SILT	456.5											
456.0		456.0											
455.5		455.5											
455.0		455.0											
454.5		454.5											
454.0		454.0											
453.5		453.5											
453.0		453.0											
452.5		452.5											
452.0		452.0											
451.5	PROBABLY VERY DENSE SILTY TILL	451.5											
451.0		451.0											
450.5		450.5											
450.0		450.0											
449.5		449.5											
449.0		449.0											
448.5		448.5											
448.0		448.0											
447.5		447.5											
447.0		447.0											
460.0		END OF HOLE	460.0										

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57035 BORING # S DATUM GEODETIC CASING HX, BX
 BORING DATE JAN. 24, 1964 REPORT DATE FEB. 1, 1964 COMPILED BY J.A. CHECKED BY M.H.E.
 SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

AS AUGER SAMPLE
 ST SLOTTED TUBE
 WS WASHED SAMPLE
 DO DRIVE-PIPE
 DV DRIVE-FOOT VALVE
 CS CHURN SAMPLE

FS FOOT SAMPLE
 CO SLEEVE-OPEN
 SF SLEEVE-FOOT VALVE
 TO THIN WALLED OPEN
 RC ROCK CORE

V IN-SITU WALL TEST
 M MECHANICAL ANALYSIS
 U UNCONFINED COMPRESSION
 CC TRIAXIAL CONSOLIDATED QUICK
 G TRIAXIAL QUICK
 S TRIAXIAL SLOW

ABBREVIATIONS

WU WET UNIT WEIGHT
 K PERMEABILITY
 C CONSOLIDATION
 WL WATER LEVEL IN CASING
 WT WATER TABLE IN SOIL

SOIL PROFILE

SHEAR STRENGTH IN LBS./SQ. FT. & VANE TESTS

250 500 750 1000 1250

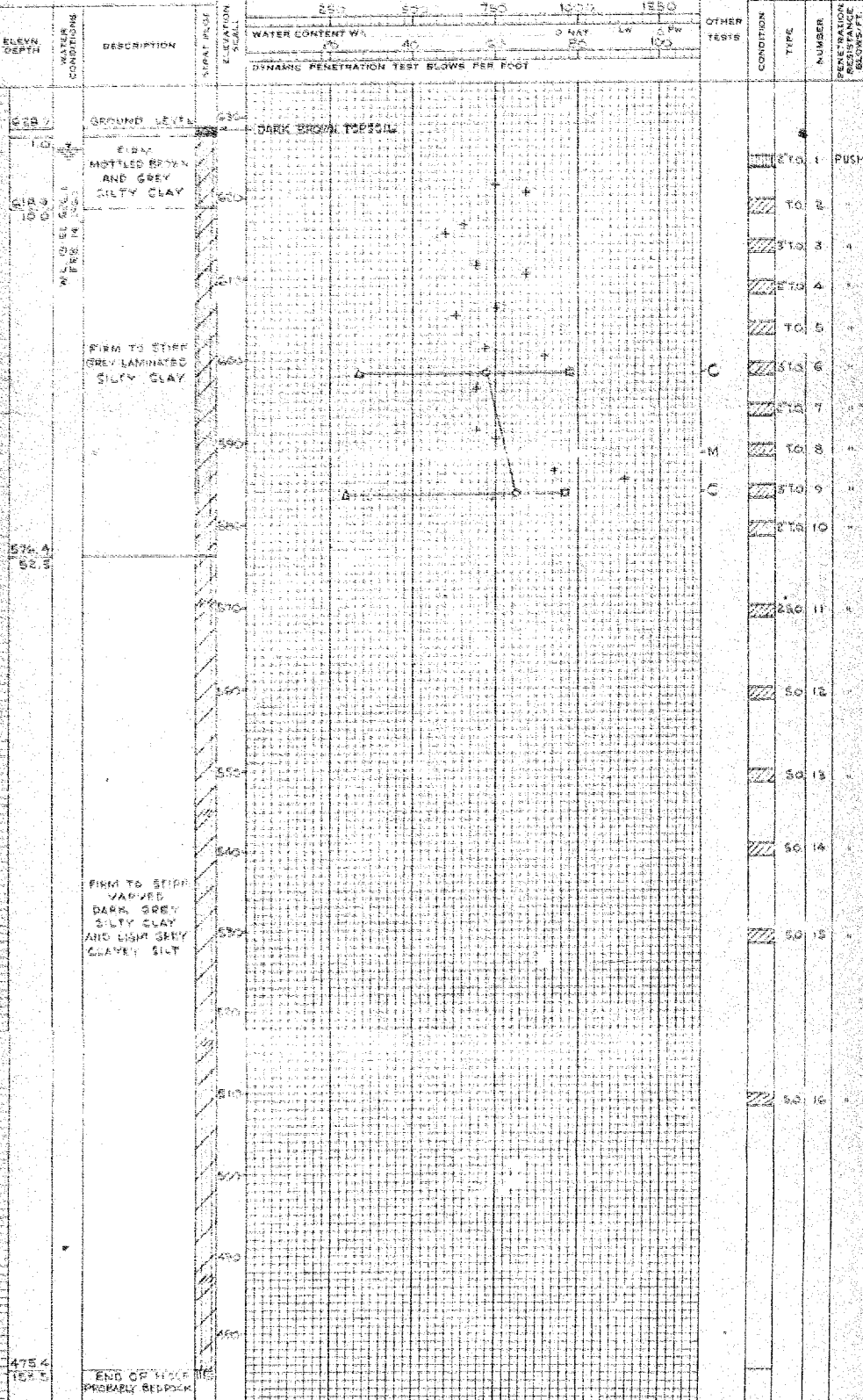
WATER CONTENT W_L 15 40 50 60 70

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

OTHER TESTS

CONDITION TYPE NUMBER PENETRATION RESISTANCE BLOWS/FT.



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57025 BORING # 11 DATUM GEODETIC CASING HX, BX
 BORING DATE JAN. 27, 1963 REPORT DATE JAN. 27, 1963 COMPILED BY J.A. CHECKED BY M.F.
 SAMPLER HAMMER WT. 140 LBS DROP 32 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

A.S. AUGER SAMPLE
 ST. SLOTTED TUBE
 WS. WASHED SAMPLE
 DCO. DRIVE-CROWN
 DVF. DRIVE-FOOT VALVE
 CS. CHUCK SAMPLE

SAMPLE TYPES

ES. SOIL SAMPLE
 SO. SLEEVE-OPEN
 SF. SLEEVE-FOOT VALVE
 TO. THIN WALLED OPEN
 RC. ROCK CORE

V. IN-SITU VANE TEST
 M. MECHANICAL ANALYSIS
 U. UNCONFINED COMPRESSION
 SC. TRIAXIAL CONSOLIDATED QUICK
 S. TRIAXIAL SLOW

ABBREVIATIONS

W. WET UNIT WEIGHT
 X. PERMEABILITY
 C. CONSOLIDATION
 WL. WATER LEVEL IN CASING
 WT. WATER TABLE IN SOIL

SOIL PROFILE

SHEAR STRENGTH IN LBS./SQ. FT. & TRIAXIAL UNDRAINED + VANE TESTS

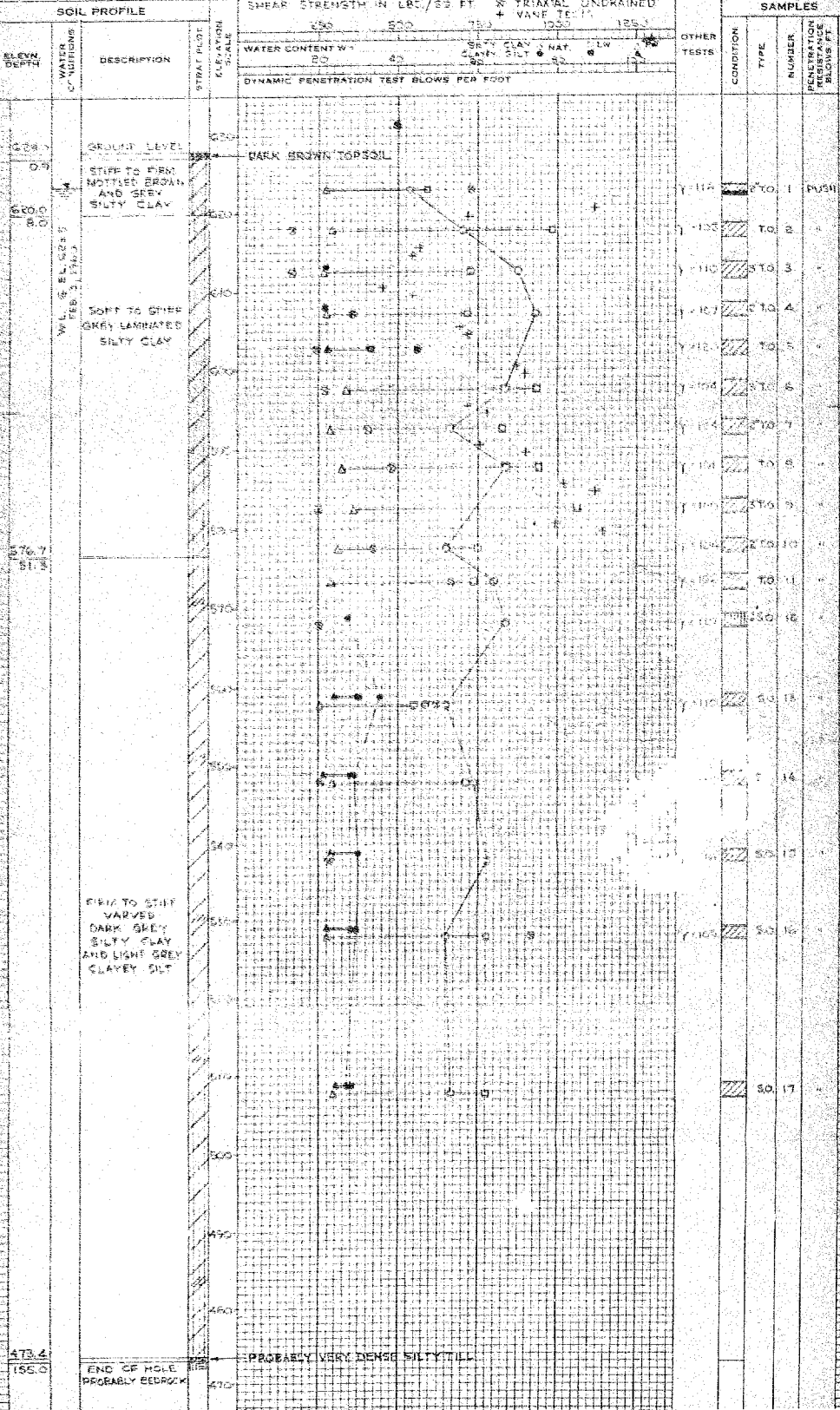
WATER CONTENT W % 20 40 60 80 100 120 140 160 180 200

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

OTHER TESTS

CONDITION TYPE NUMBER PENETRATION RESISTANCE BLOWS/FT.



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 5 T025 SPRING 12 DATUM GEODETIC CASING HX, BX
 BORING DATE JAN. 29, 1960 REPORT DATE FEB. 2, 1960 COMPILED BY J.A. CHECKED BY J.M.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY

SAMPLE CONDITION

☐ DISTURBED
 ☐ FAIR
 ☐ GOOD
 ☐ LOST

AS - AUGER SAMPLE
 ST - SLOTTED TUBE
 WS - WASHED SAMPLE
 DO - DRIVE OPEN
 DF - DRIVE FOOT VALVE
 CS - CHUCK SAMPLE

SAMPLE TYPES

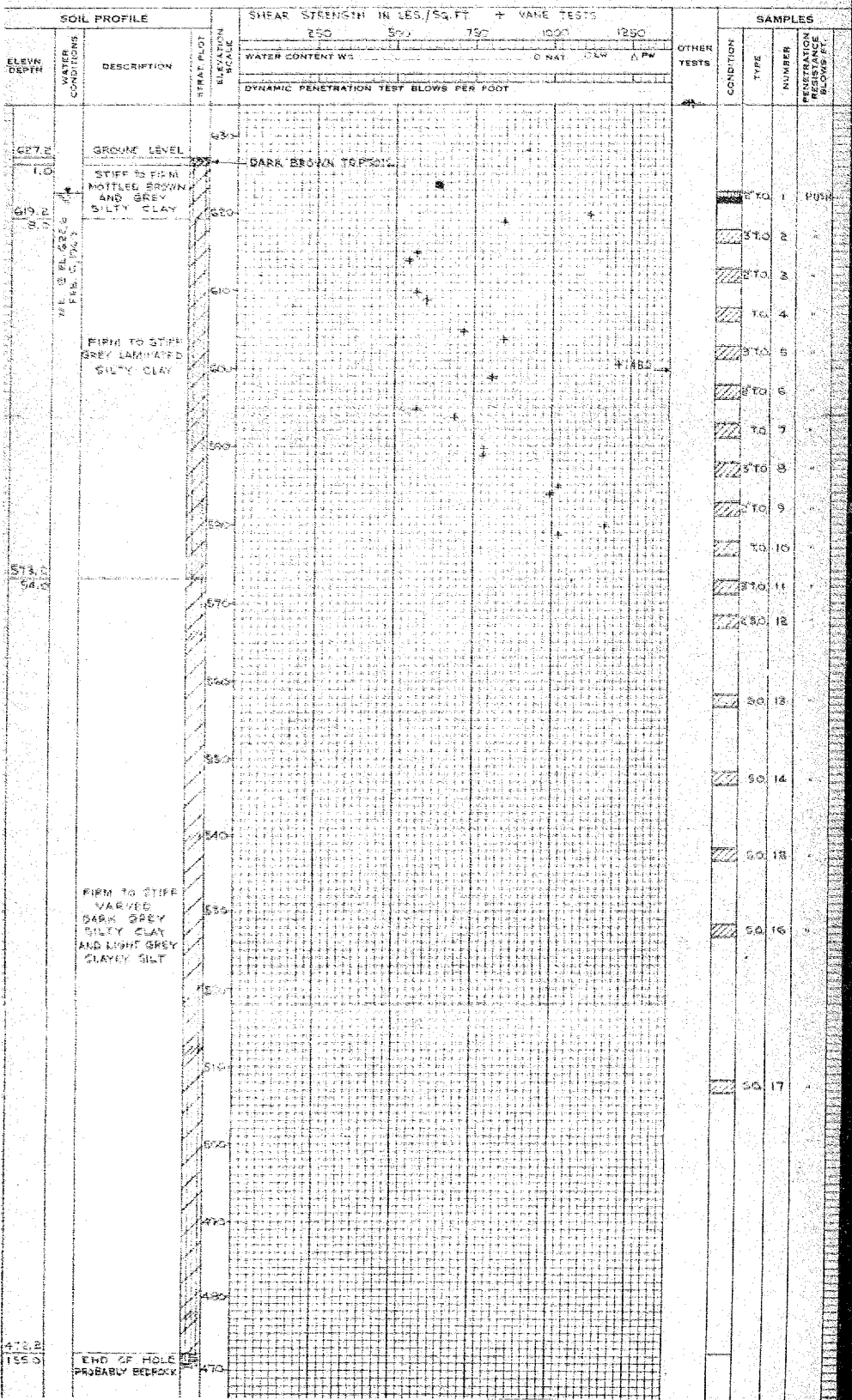
FS - FOIL SAMPLE
 SO - SLEEVE OPEN
 SF - SLEEVE FOOT VALVE
 TO - THIN WALLED OPEN
 RC - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 CO - TRIAXIAL CONSOLIDATED QUICK
 CU - TRIAXIAL QUICK
 S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
 X - PERMEABILITY
 C - CONSOLIDATION

WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57022 BORING # 12 AND PENTESTS 14 & 15 DATUM GEODESIC CASING BX
 BORING DATE FEB 20, 1960 REPORT DATE MARCH 22, 1960 COMPILED BY J.A. CHECKED BY J.M.E.
 SAMPLER HAMMER WT. 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

DISBURRED
 FAIR
 GOOD
 BEST

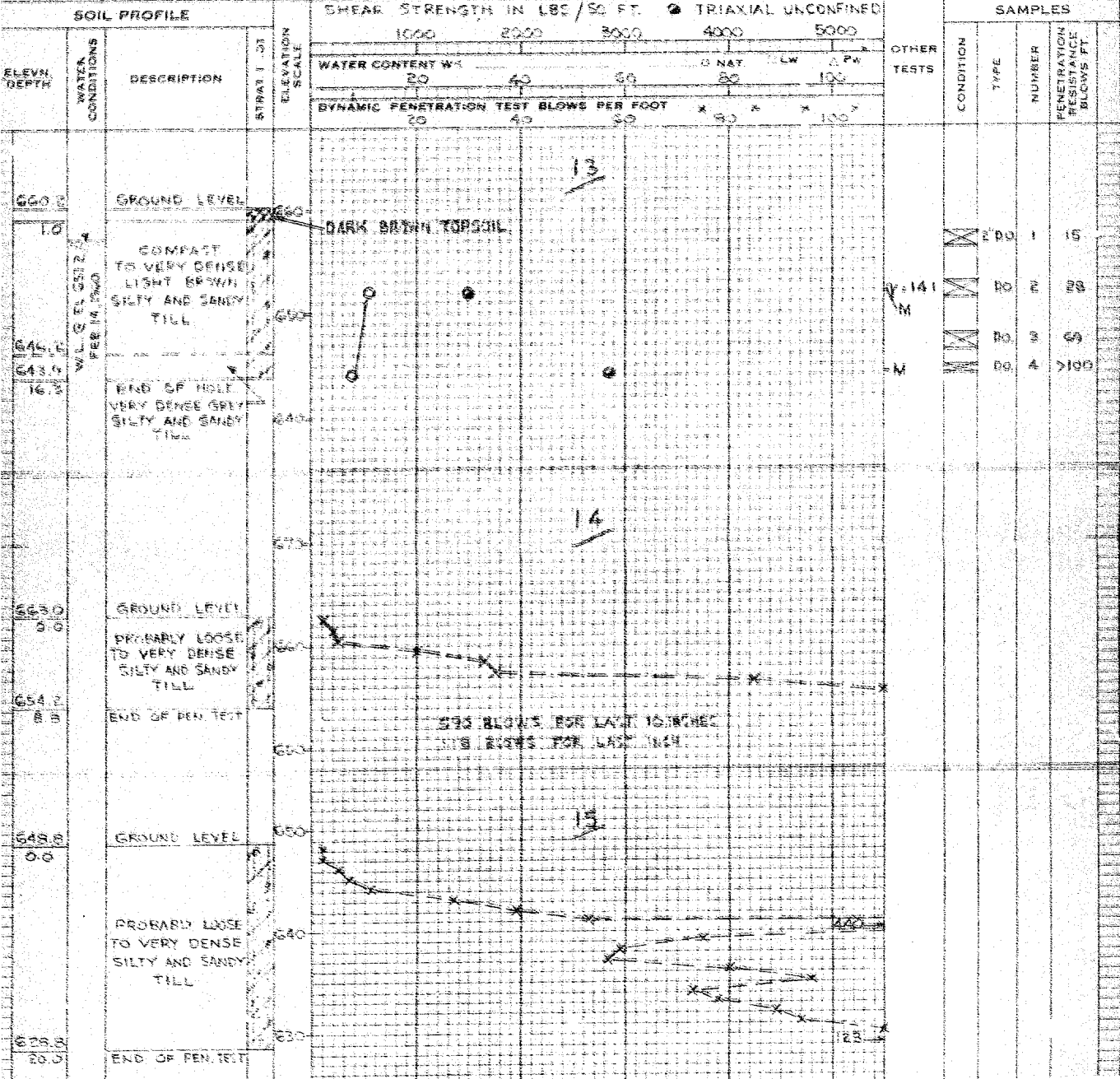
SAMPLE TYPES

A5 - AUGER SAMPLE
 ST - SLOTTED TUBE
 WS - WASHED SAMPLE
 DO - DRIVE-OPEN
 BF - DRIVE-FOOT VALVE
 CS - CHUNK SAMPLE
 FS - FOIL SAMPLE
 SO - SLEEVE-OPEN
 SF - SLEEVE-FOOT VALVE
 TO - THIN WALLED OPEN
 RC - ROCK CORE

ABBREVIATIONS

V - IN SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 Y - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57025 PEN TESTS 18 AND 19 DATUM GEODETIC CASING
 BORING DATE FEB. 21, 1960 REPORT DATE MARCH 22, 1960 COMPILED BY J.A. CHECKED BY M.H.6
 SAMPLER HAMMER WT. LBS. DROP INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

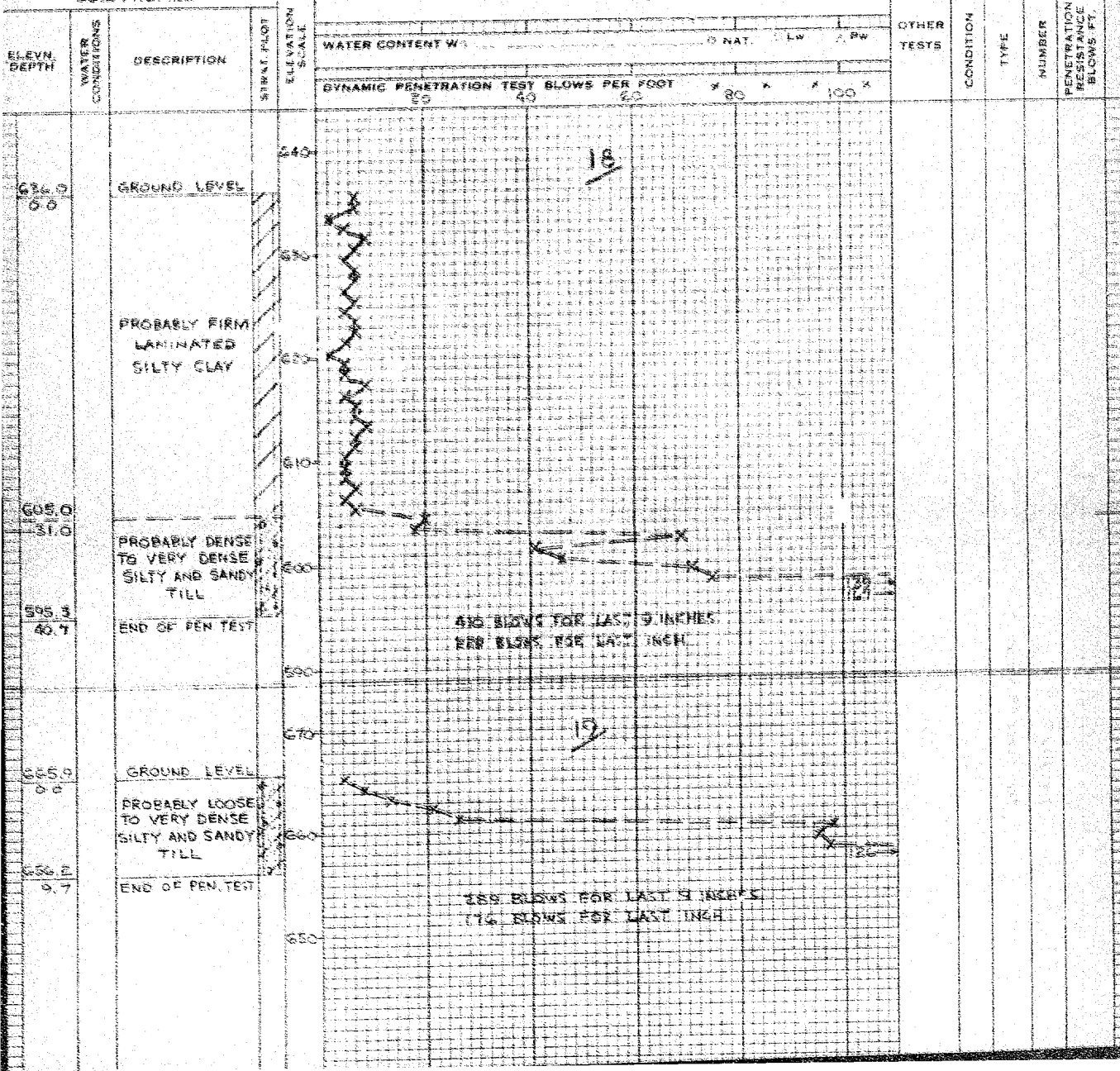
SAMPLE TYPES

A.S. AUGER SAMPLE
 S.T. SLOTTED TUBE
 W.S. WASHED SAMPLE
 D.O. DRIVE-OPEN
 D.F. DRIVE-FOOT VALVE
 C.S. CHUNK SAMPLE
 F.S. FOIL SAMPLE
 S.O. SLEEVE-OPEN
 S.F. SLEEVE-FOOT VALVE
 T.O. THIN WALL, OPEN
 R.C. ROCK COR.

ABBREVIATIONS

V - IN-SITU VANE TT
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 Q - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL SLOW
 S - TRIAXIAL SLOW
 1 - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

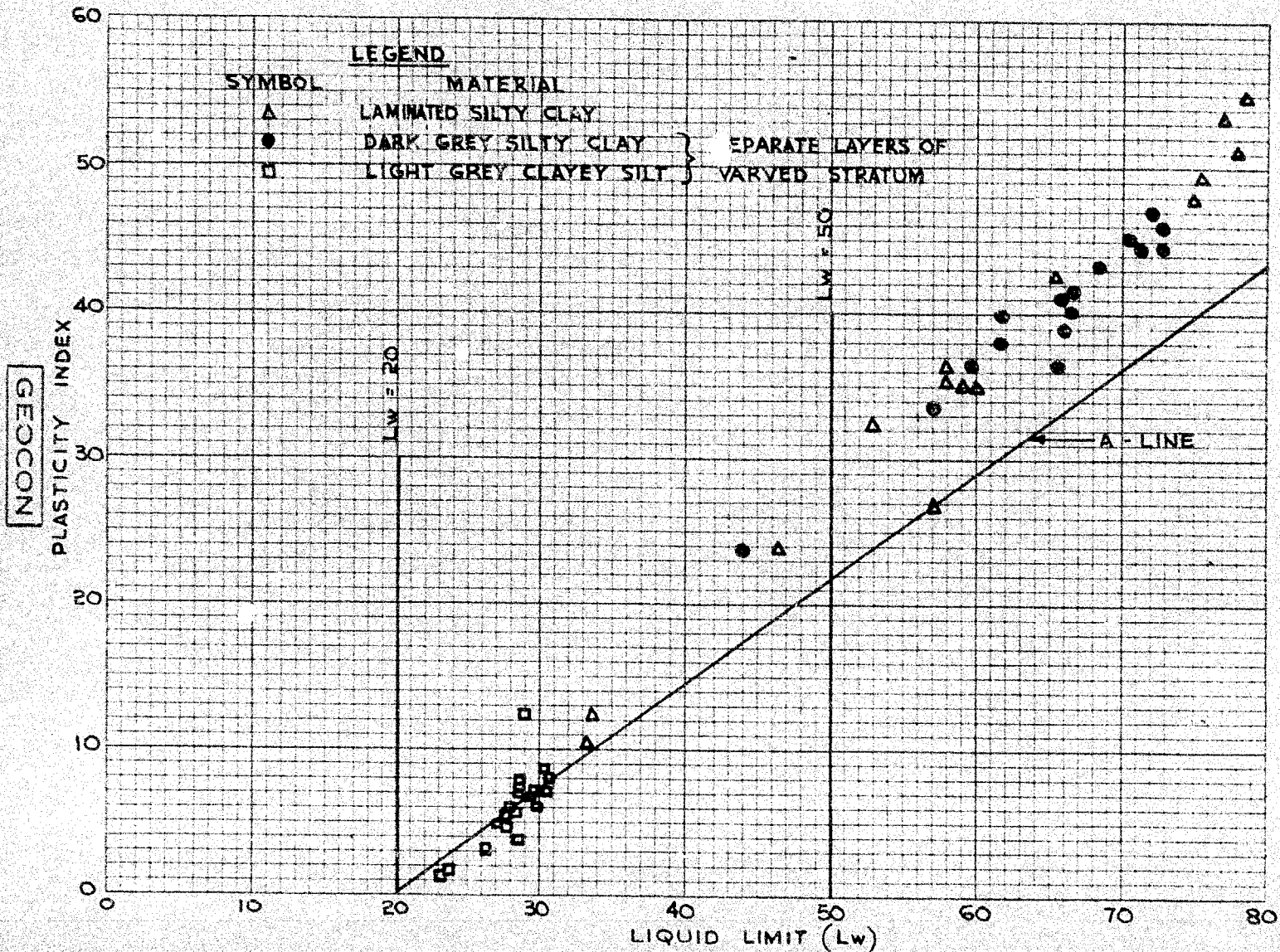


APPENDIX II

Figures - Laboratory Testing

PLASTICITY CHART

APPENDIX II
FIGURE 1
PROJECT S 7025



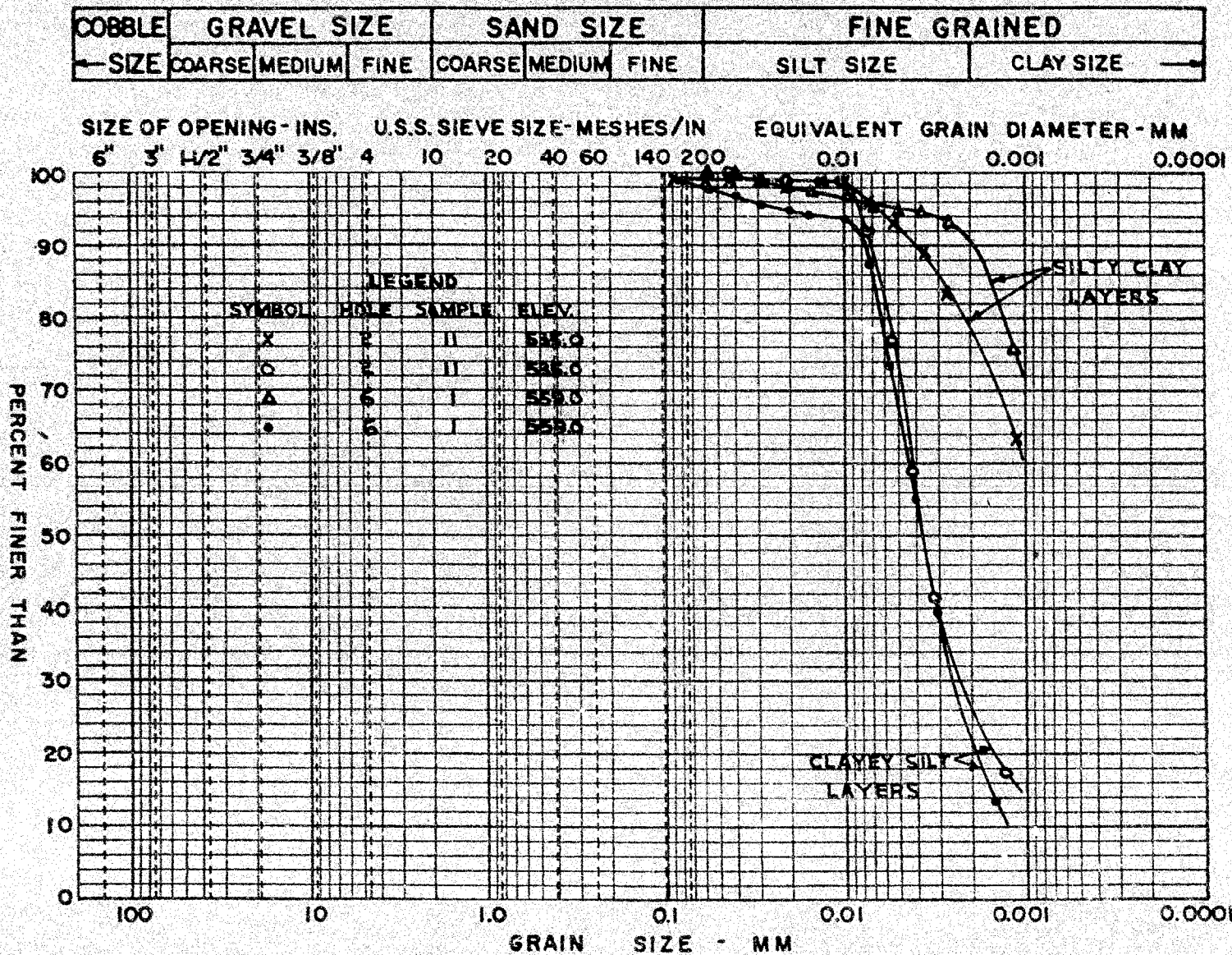
GRAIN SIZE DISTRIBUTION

VARVED SILTY CLAY AND CLAYEY SILT

APPENDIX II

FIGURE 3

PROJECT S 7025



M.I.T. GRAIN SIZE SCALE

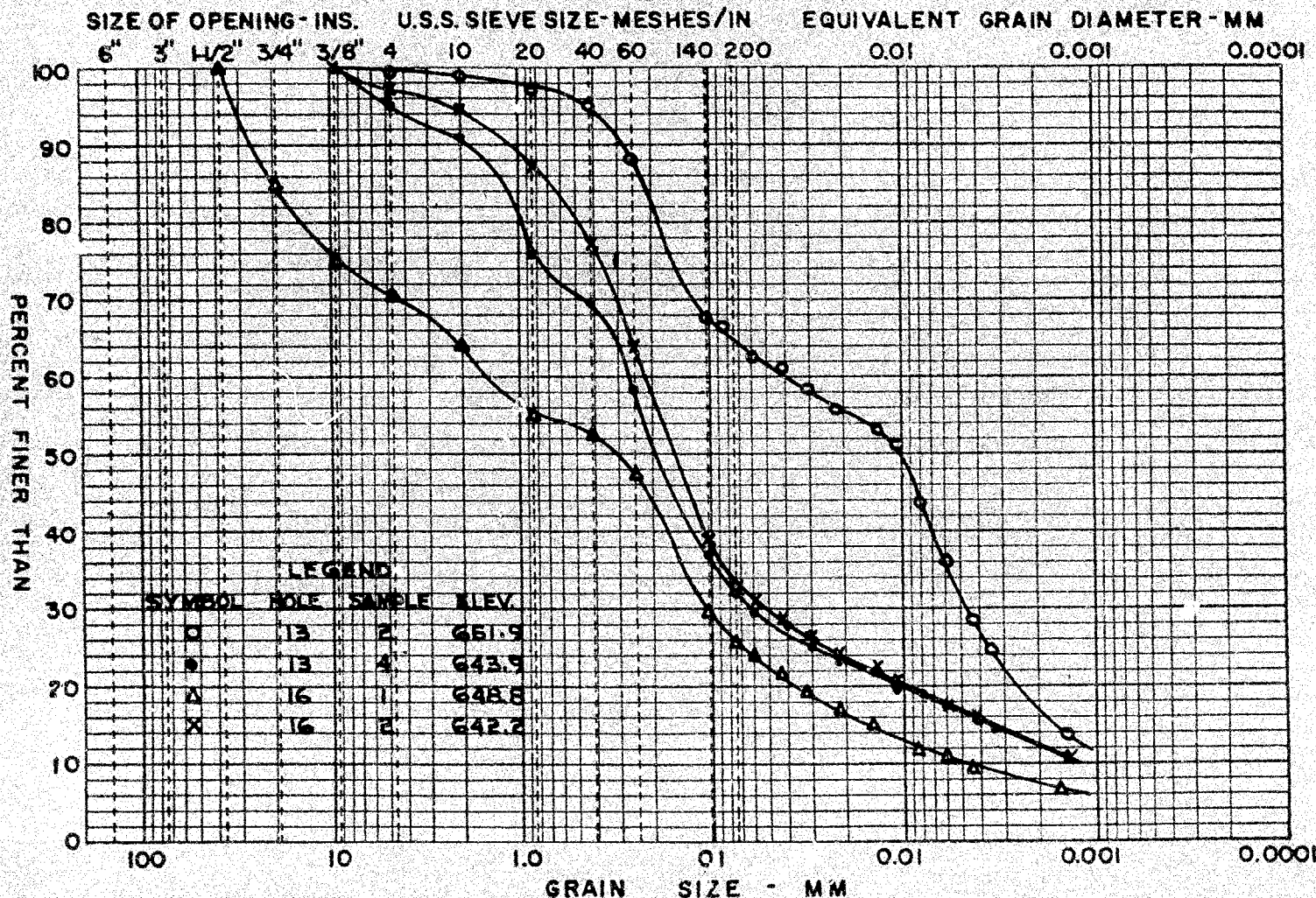
GEOCON

GRAIN SIZE DISTRIBUTION

SILTY AND SANDY TILL

APPENDIX II
FIGURE 4
PROJECT S7025

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

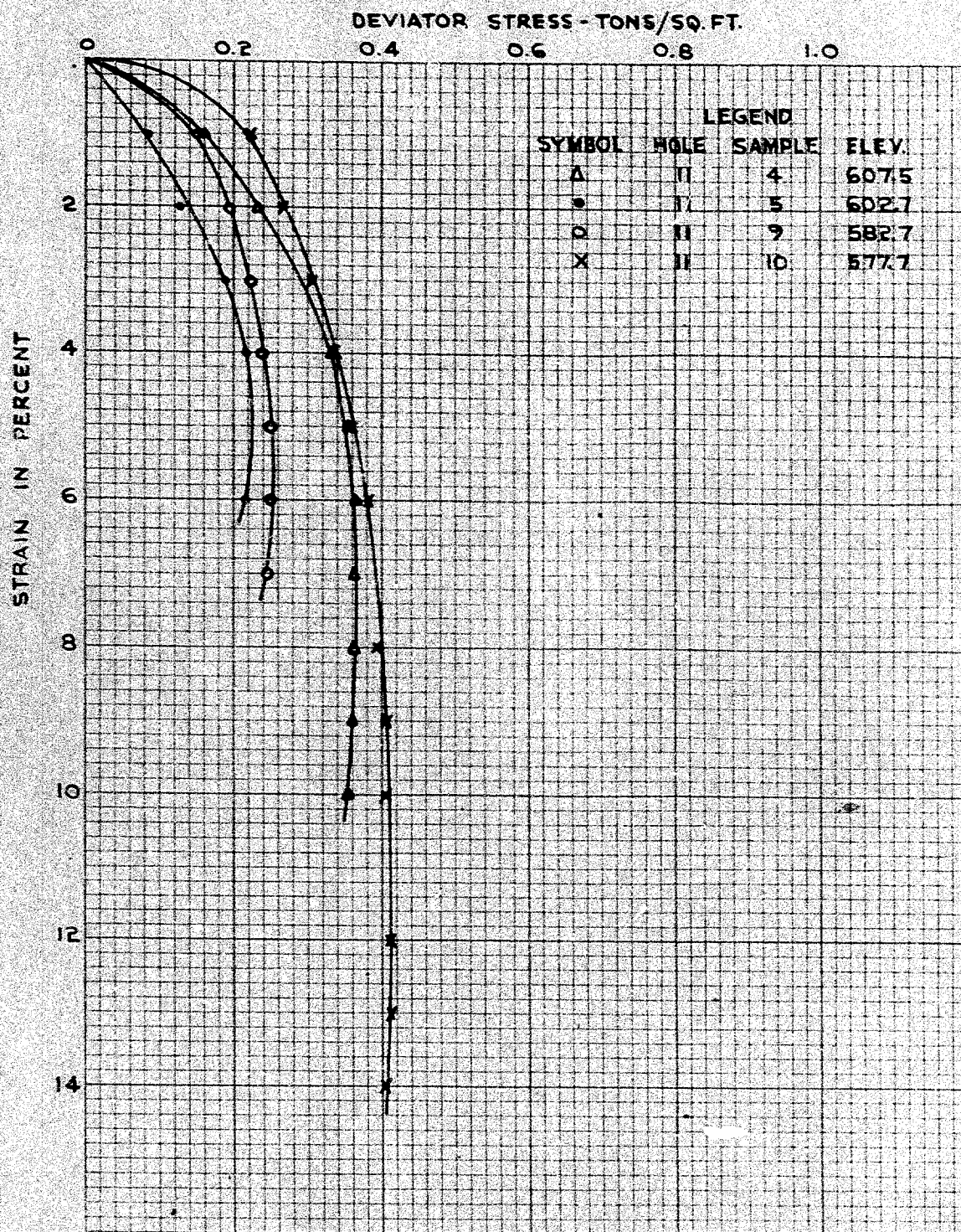


M.I.T. GRAIN SIZE SCALE

GEOCON

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

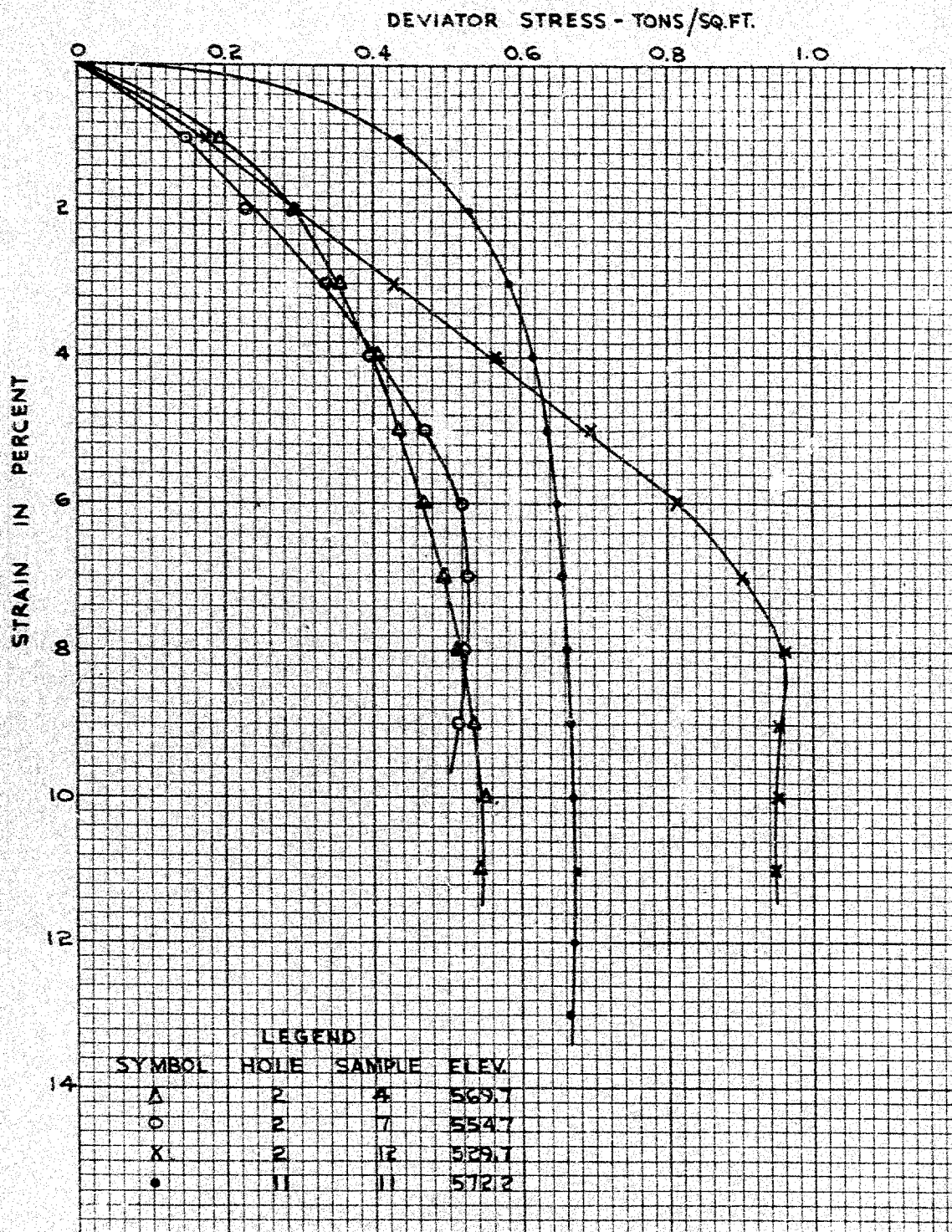
APPENDIX II
FIGURE 5
PROJECT S 7025



UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES

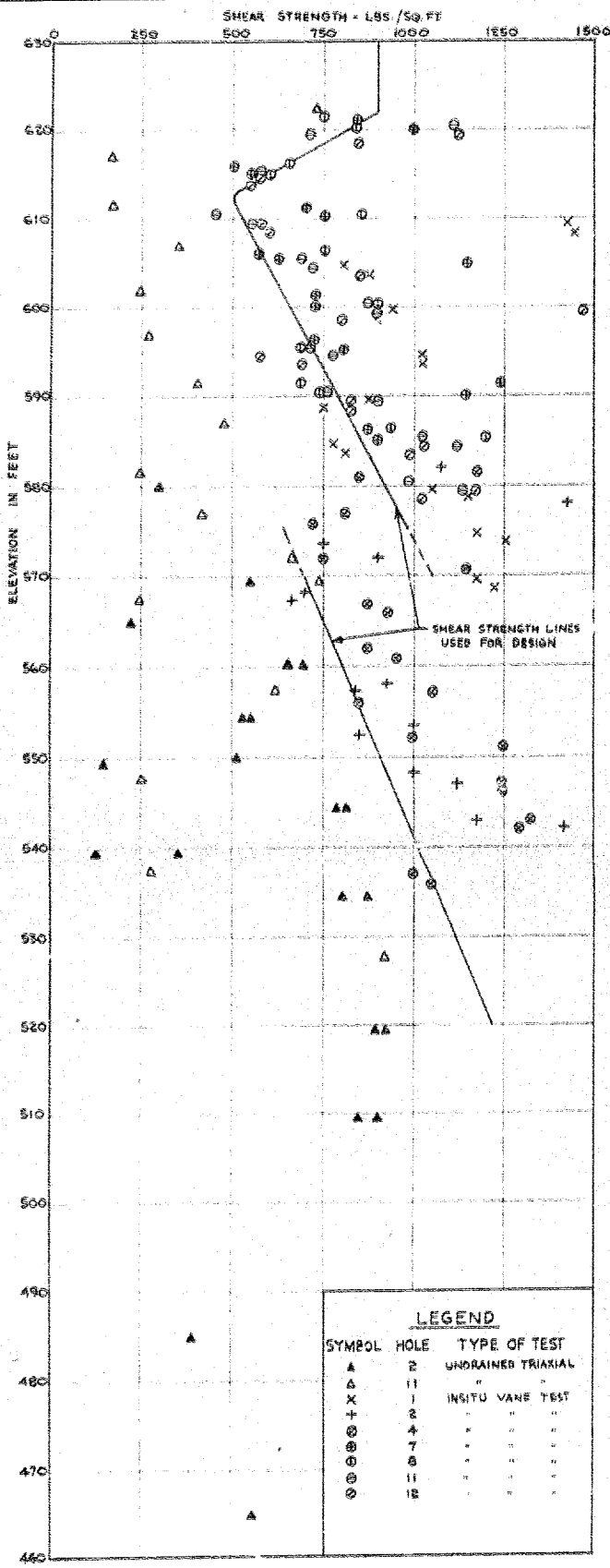
VARVED SILTY CLAY & CLAYEY SILT

APPENDIX II
FIGURE 6
PROJECT S 7025



SHEAR STRENGTH VS ELEVATION

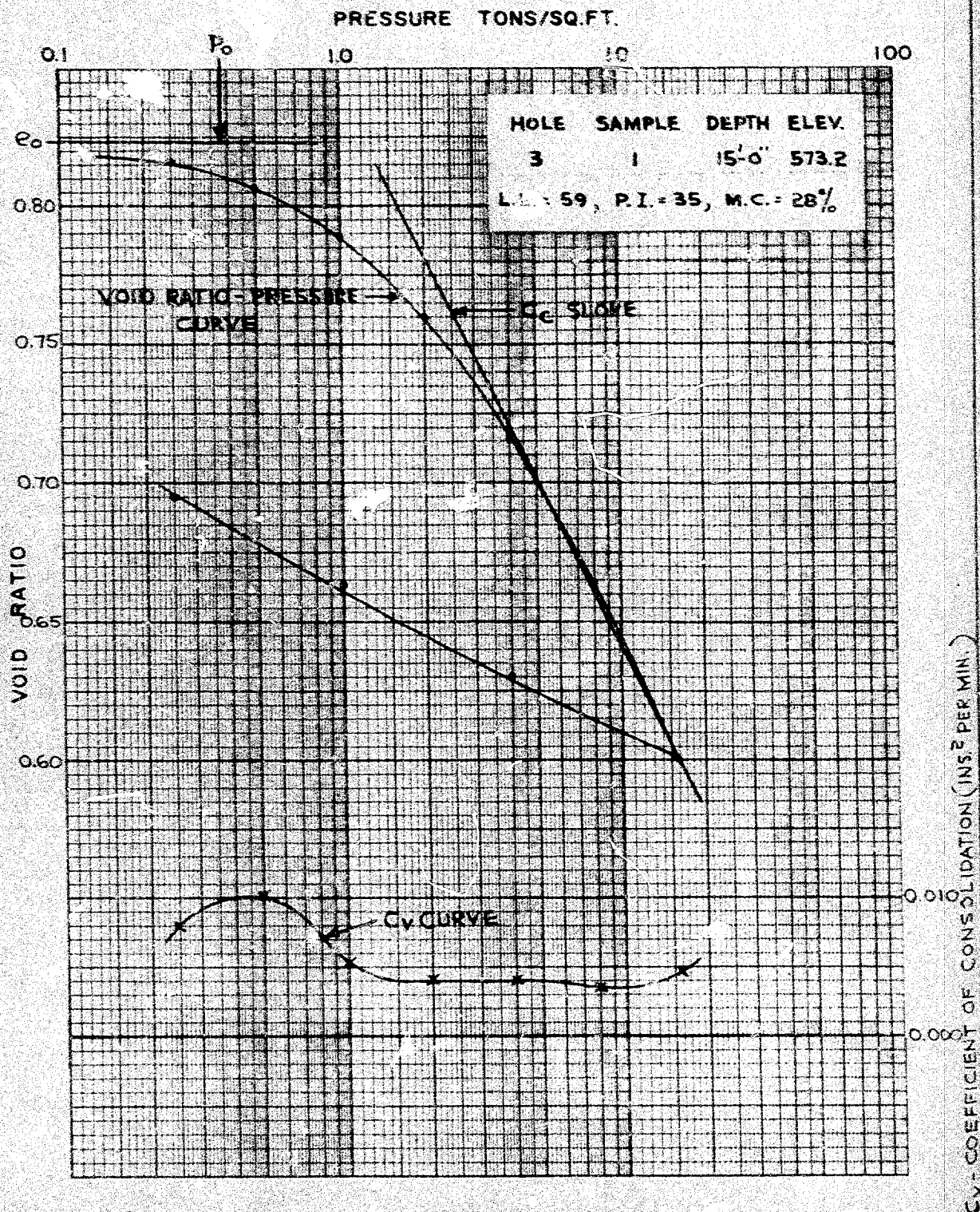
APPENDIX 7
FIGURE 7
PROJECT - S 7021



LEGEND		
SYMBOL	HOLE	TYPE OF TEST
▲	2	UNDRAINED TRIAXIAL
△	11	" "
×	1	INSITU VANE TEST
⊙	2	" " "
⊗	7	" " "
⊖	8	" " "
⊕	11	" " "
⊗	12	" " "

VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

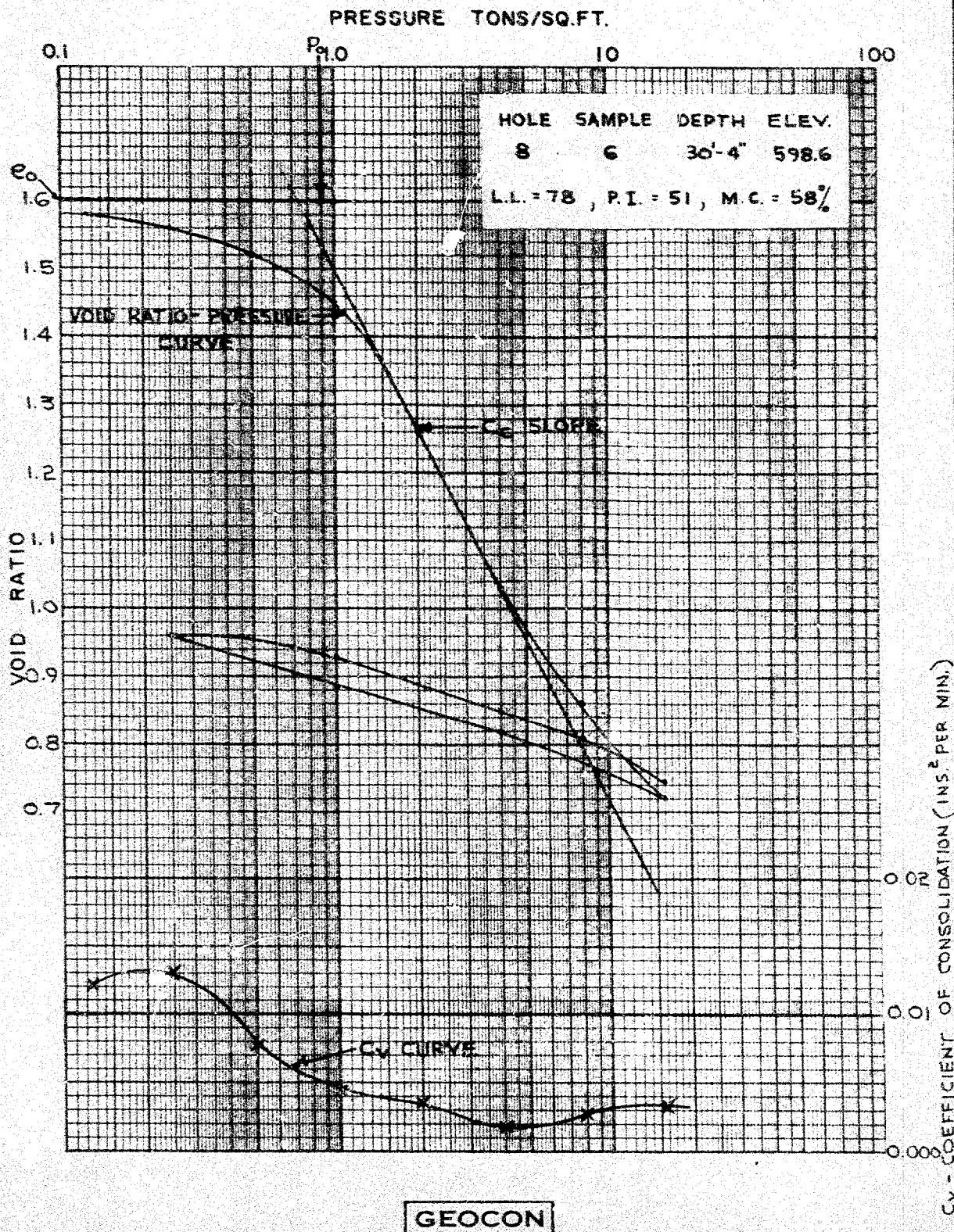
APPENDIX II
FIGURE 8
PROJECT S7025



GEOCON

VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II
FIGURE 9
PROJECT S 7025



VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

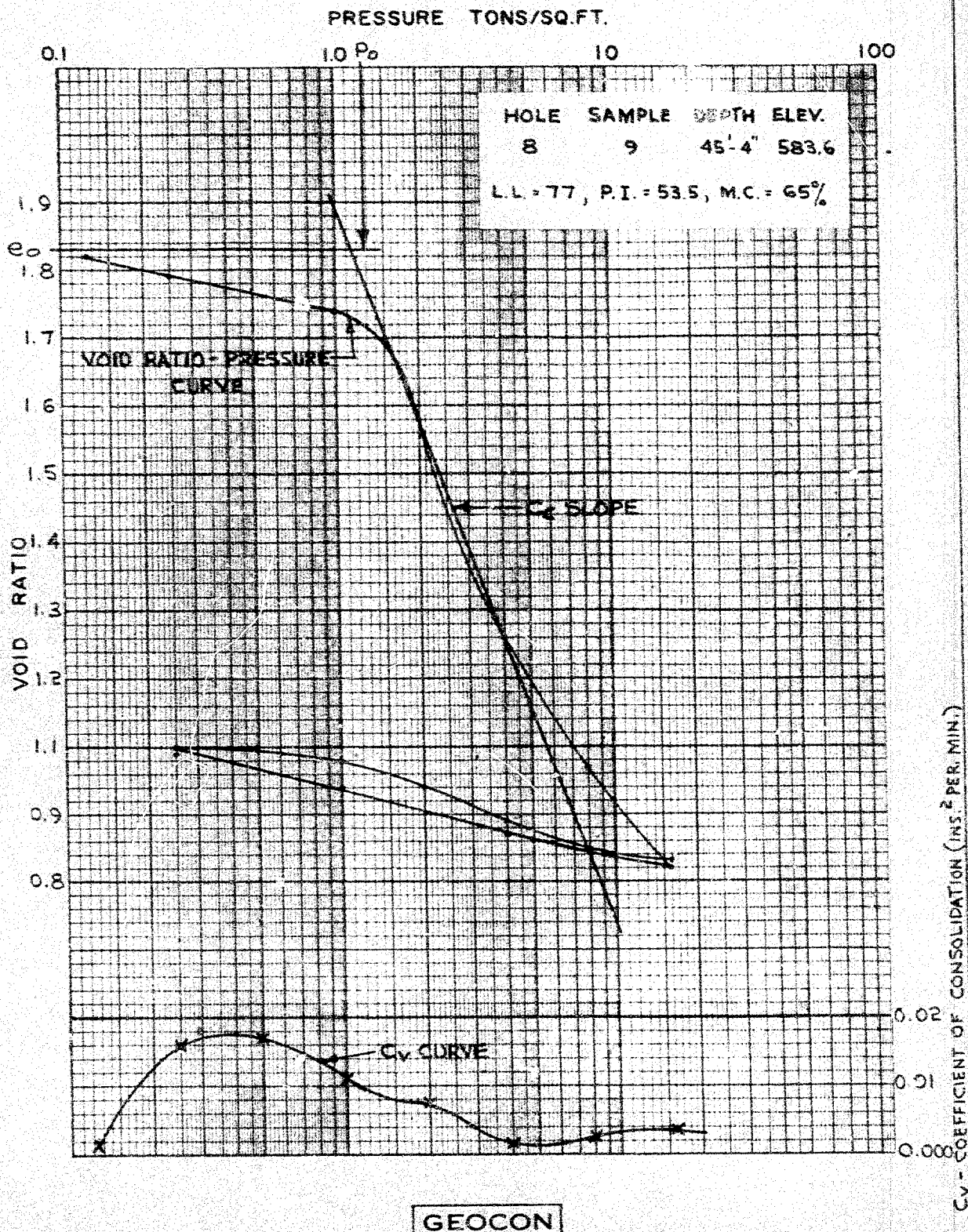
APPENDIX

II

FIGURE

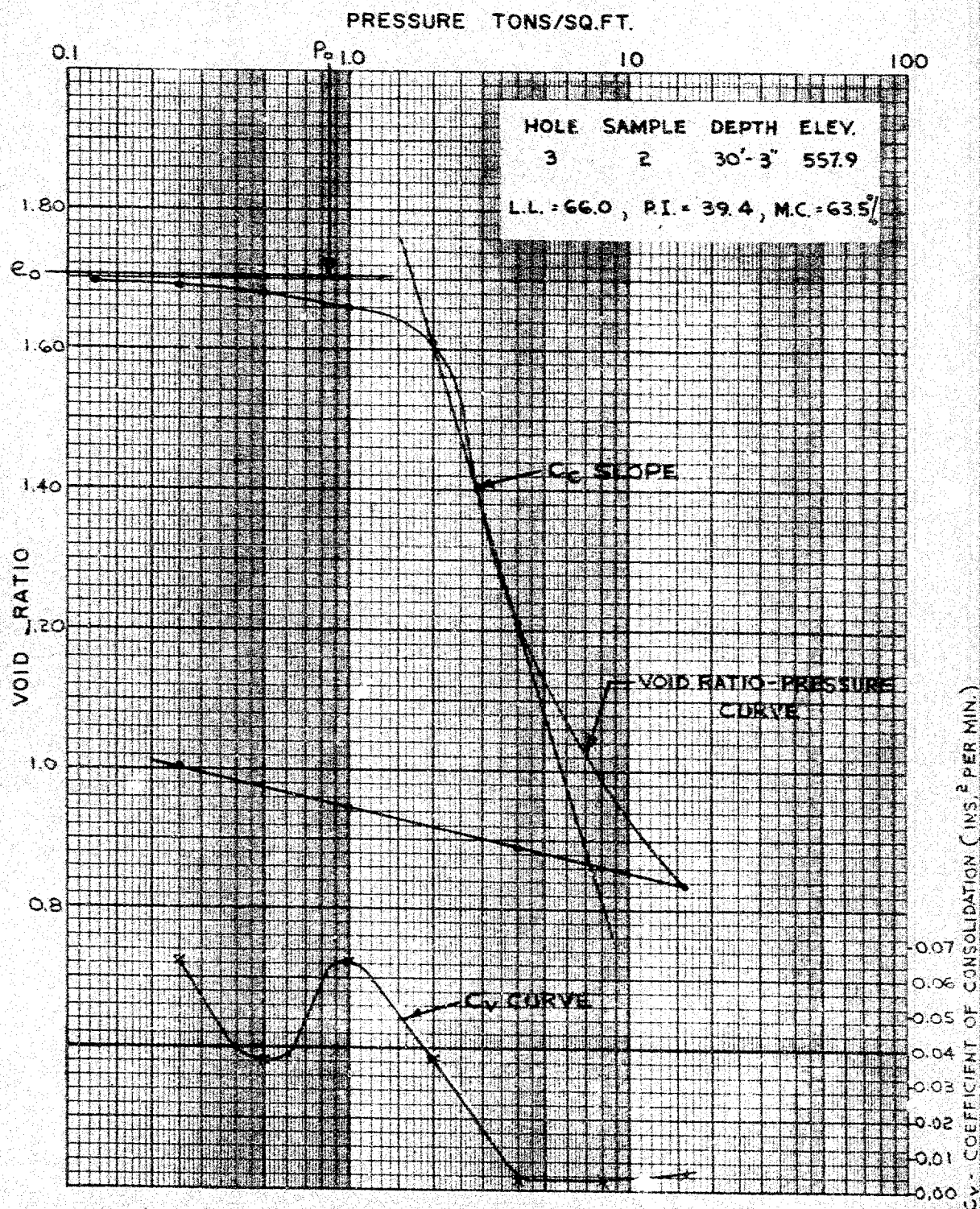
10

PROJECT S 7025



VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II
FIGURE 11
PROJECT S 7025



GEOCON

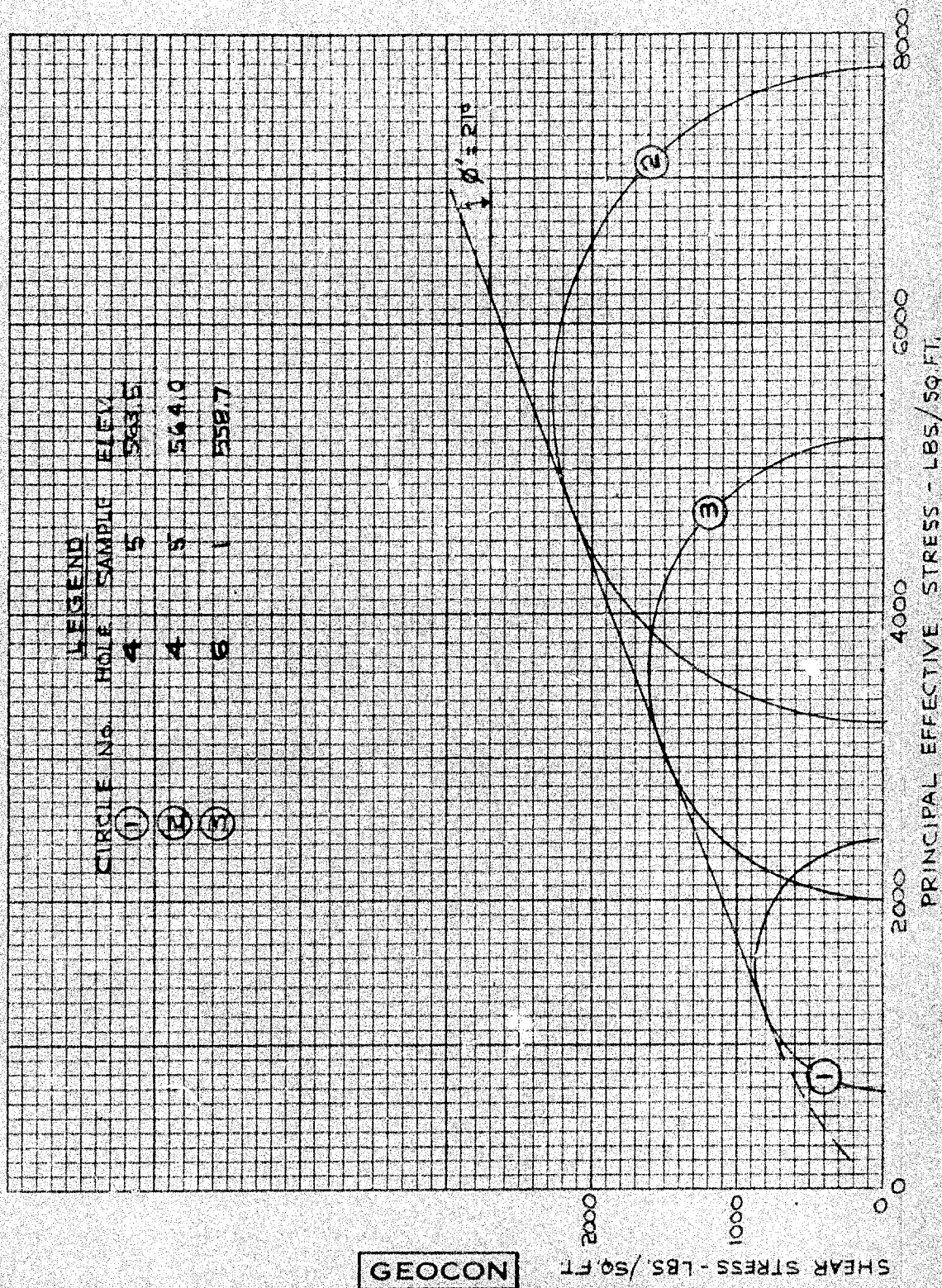
CONSOLIDATED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS MOHR'S CIRCLES

APPENDIX

FIGURE

12

PROJECT S 7025



APPENDIX III

Figures - Engineering Analyses

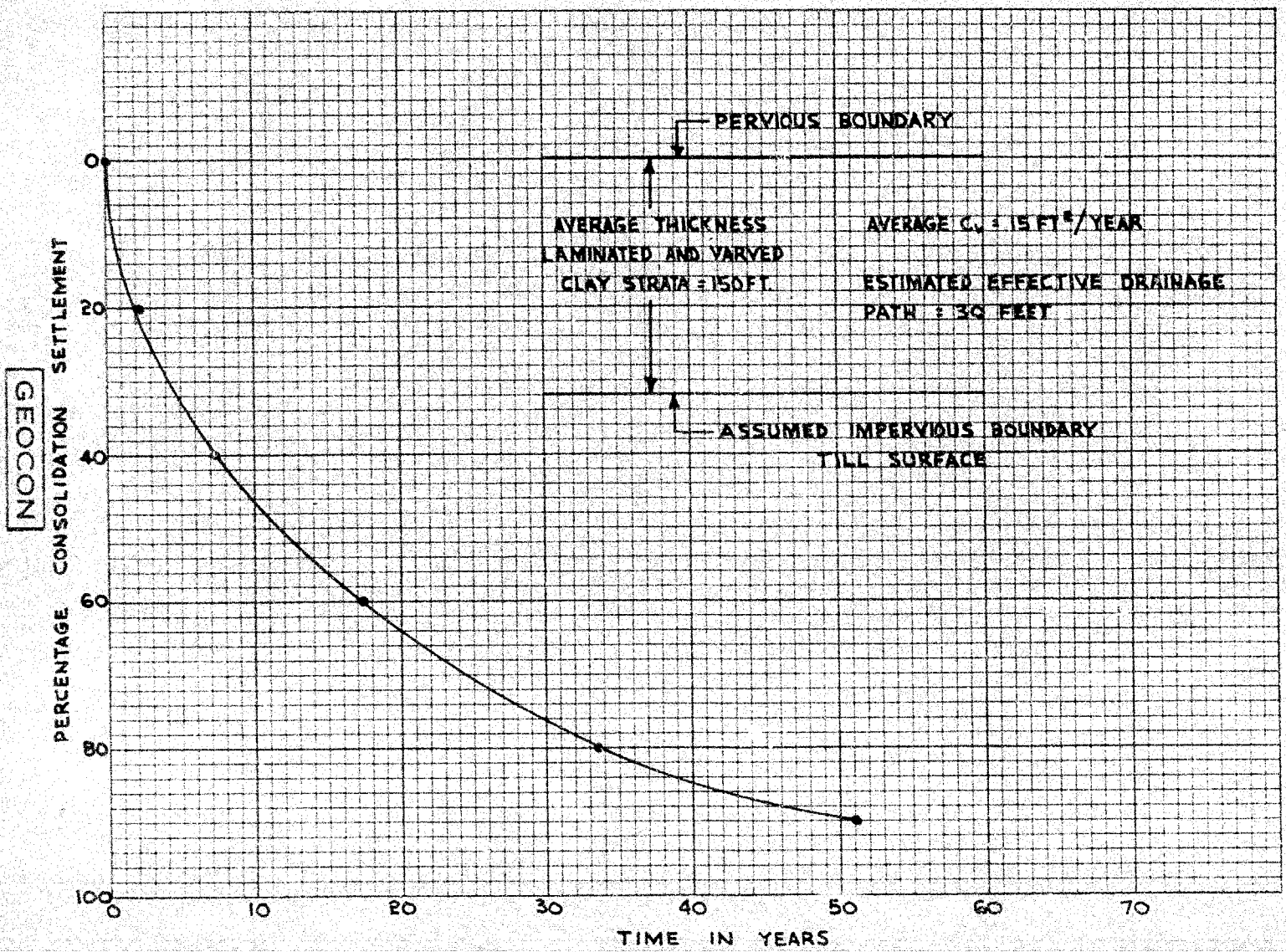
- Drawings S7025-3 Recommended Embankment Cross Sections
S7025-4 Recommended Approach Embankment Profile
S7025-5 Typical Berm Section - Spread Footings
S7025-6 Recommended Slopes - Wabi River Crossing

Drawings in Pocket at rear of report:

- S7025-1 Boring Plan and Soil Stratigraphy
S7025-2 Typical Procedure-Design of Proposed
Embankment

ESTIMATED
TIME-RATE CONSOLIDATION SETTLEMENT
CLAY STRATA

APPENDIX III
FIGURE 1
PROJECT S7025



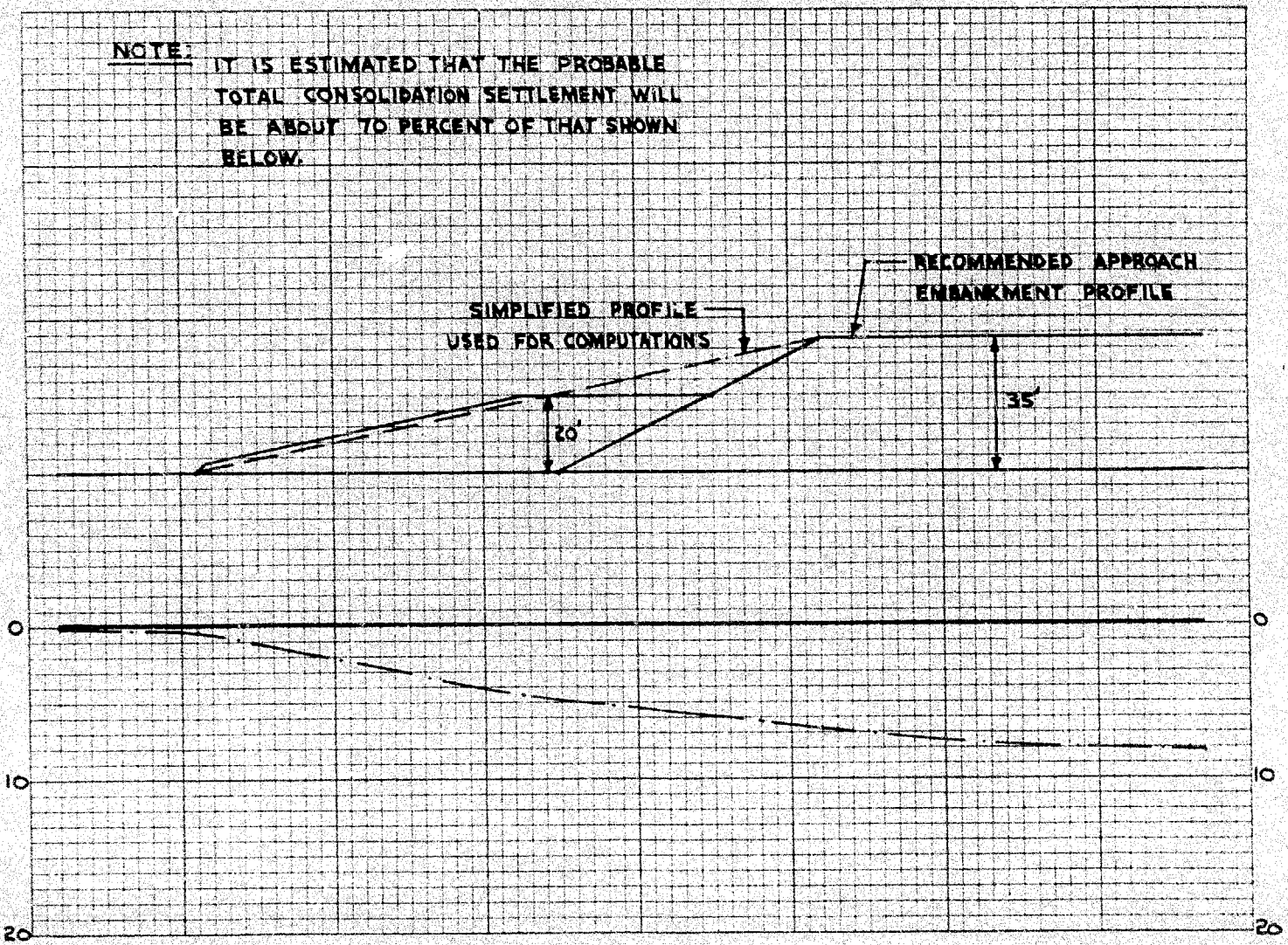
COMPUTED TOTAL CONSOLIDATION SETTLEMENT UNDER APPROACH EMBANKMENTS

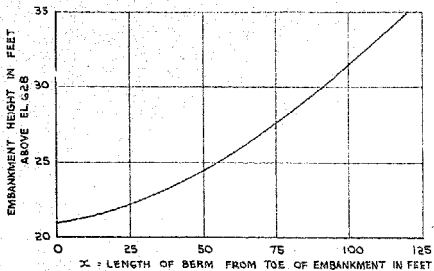
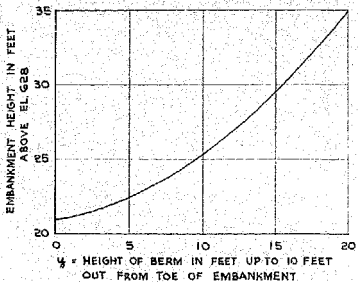
APPENDIX III
FIGURE 2
PROJECT S 7025

NOTE: IT IS ESTIMATED THAT THE PROBABLE
TOTAL CONSOLIDATION SETTLEMENT WILL
BE ABOUT 70 PERCENT OF THAT SHOWN
BELOW.

GEOCON

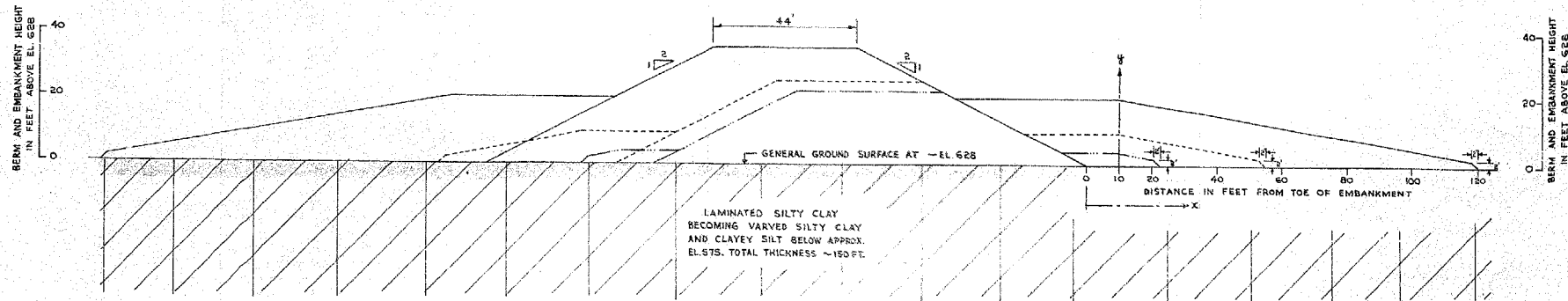
TOTAL CONSOLIDATION SETTLEMENT - FEET





DESIGN VALUES USED FOR EMBANKMENT AND BERMS

GRANULAR MATERIAL: $\gamma = 125 \text{ LBS./CU. FT.}$
 $\phi = 30^\circ$



TYPICAL EMBANKMENT-BERM SECTIONS

ONTARIO NORTHLAND RAILWAY OVERHEAD

SCALE: 1" = 20'-0"

LEGEND

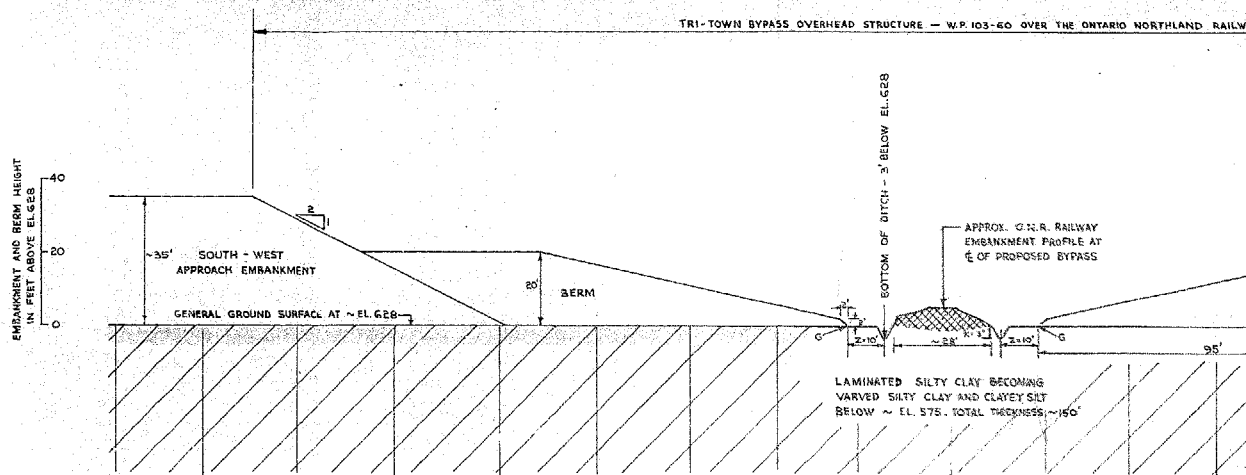
EMBANKMENT HEIGHT	EMBANKMENT AND BERM PROFILE
35 FEET	—————
23 FEET	- - - - -
22 FEET	—————

DEPARTMENT OF HIGHWAYS, ONTARIO		
TORONTO		ONTARIO
PROPOSED TRI-TOWN BYPASS STRUCTURES		
W.P. 103-60	W.P. 104-60	W.P. 124-60
HIGHWAY No. 11		
NEW LISKEARD		ONTARIO
RECOMMENDED EMBANKMENT CROSS-SECTIONS		

GEOCON LTD.

DATE: MAY 11, 1960 SCALE: AS SHOWN

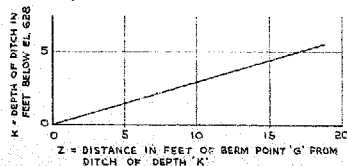
MADE BY J.A. CHKD. APPD. No. 57025-3

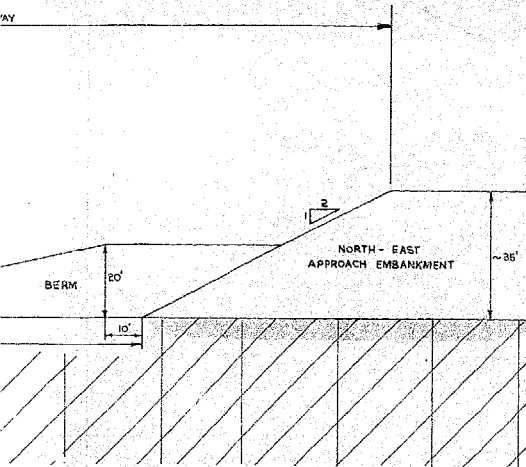


TYPICAL APPROACH EMBANKMENT PROFILE

ONTARIO NORTHLAND RAILWAY OVERHEAD

SCALE: 1" = 20'-0"





EMBANKMENT AND BERM HEIGHT
IN FEET ABOVE EL. 628

NOTES

- (1) GENERAL GROUND ELEVATION 628 USED FOR COMPUTATION PURPOSES.
- (2) COMPUTATIONS ARE BASED ON 150 FOOT THICKNESS OF CLAY.
- (3) DESIGN VALUES USED FOR GRANULAR EMBANKMENT AND BERM SECTIONS ARE:-

$$\gamma = 125 \text{ LBS./CU. FT.}$$

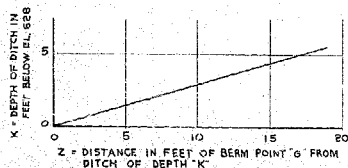
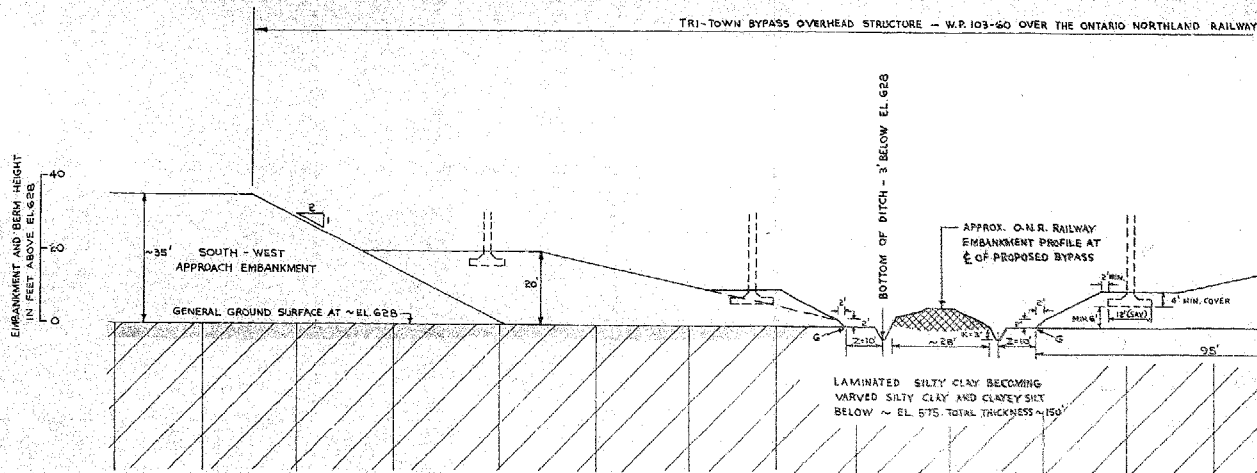
$$\phi = 30^\circ$$

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED TRI-TOWN BYPASS STRUCTURES
W.P. 103 - 60 W.P. 104 - 60 W.P. 124 - 60
HIGHWAY No 11
NEW LISHEARD ONTARIO
RECOMMENDED APPROACH EMBANKMENT PROFILE

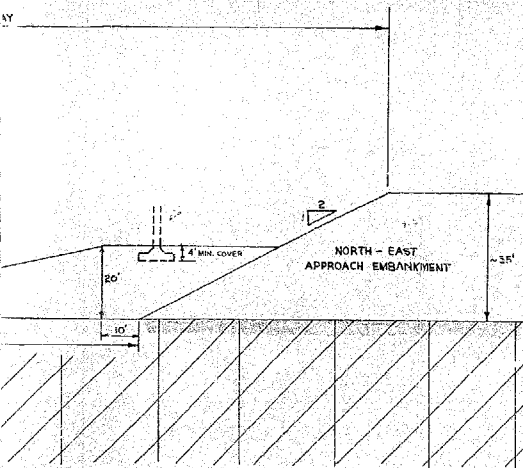
GEOCON LTD.

DATE: MAY 11, 1960 SCALE: AS SHOWN

MADE CHKD APPD
J.A. H.S. 11-1 No. S7025 - 4



SUGGESTED APPROACH EMBANKMENT PROFILE
AND TYPICAL SPREAD FOOTING ARRANGEMENT
ONTARIO NORTHLAND RAILWAY OVERHEAD
SCALE: 1" = 20'-0"



NOTES

- (1) GENERAL GROUND ELEVATION 628 USED FOR COMPUTATION PURPOSES.
- (2) COMPUTATIONS ARE BASED ON 150 FOOT THICKNESS OF CLAY.
- (3) DESIGN VALUES USED FOR GRANULAR EMBANKMENT AND BERM SECTIONS ARE:-

$$\gamma = 125 \text{ LBS./CU. FT.}$$

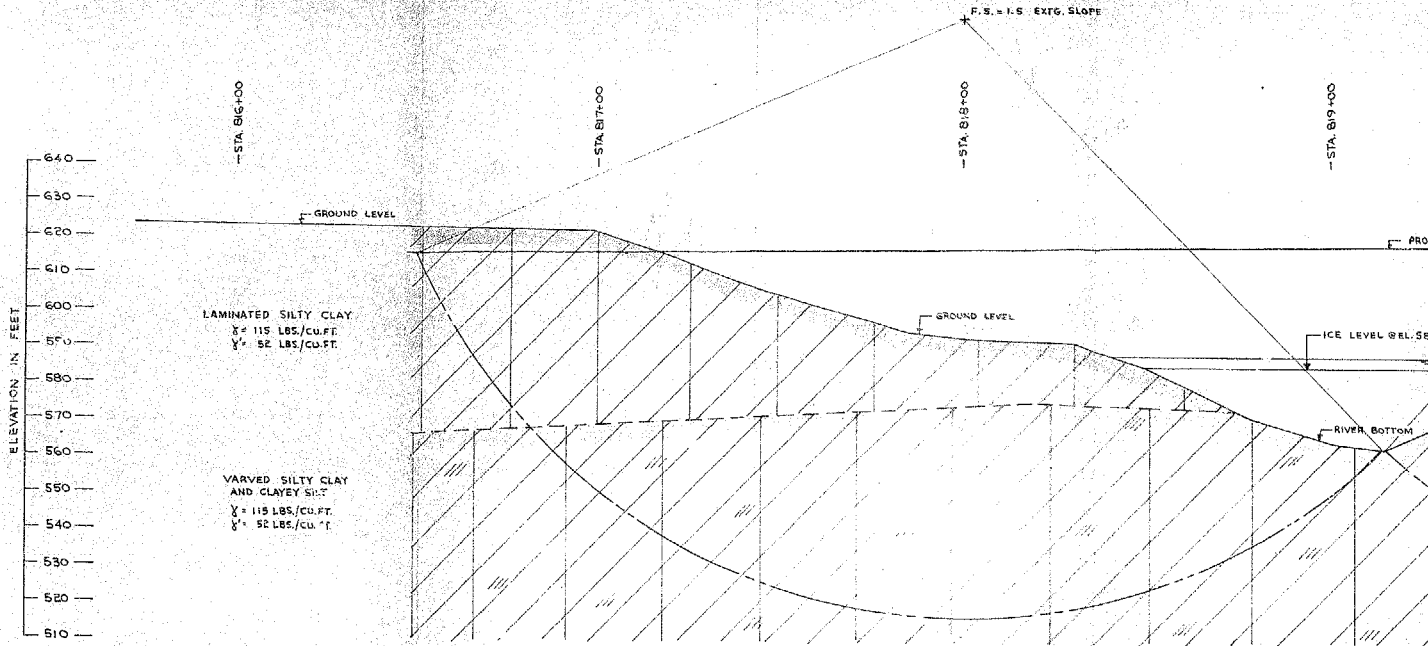
$$\phi = 30^\circ$$

DEPARTMENT OF HIGHWAYS, ONTARIO
 TORONTO
 PROPOSED TRI-TOWN BYPASS STRUCTURES
 W.P. 103 - 60 W.P. 104 - 60 W.P. 124 - 60
 HIGHWAY No 11
 NEW LISKEARD
 TYPICAL BERM SECTION - SPREAD FOOTINGS

GEOCON LTD

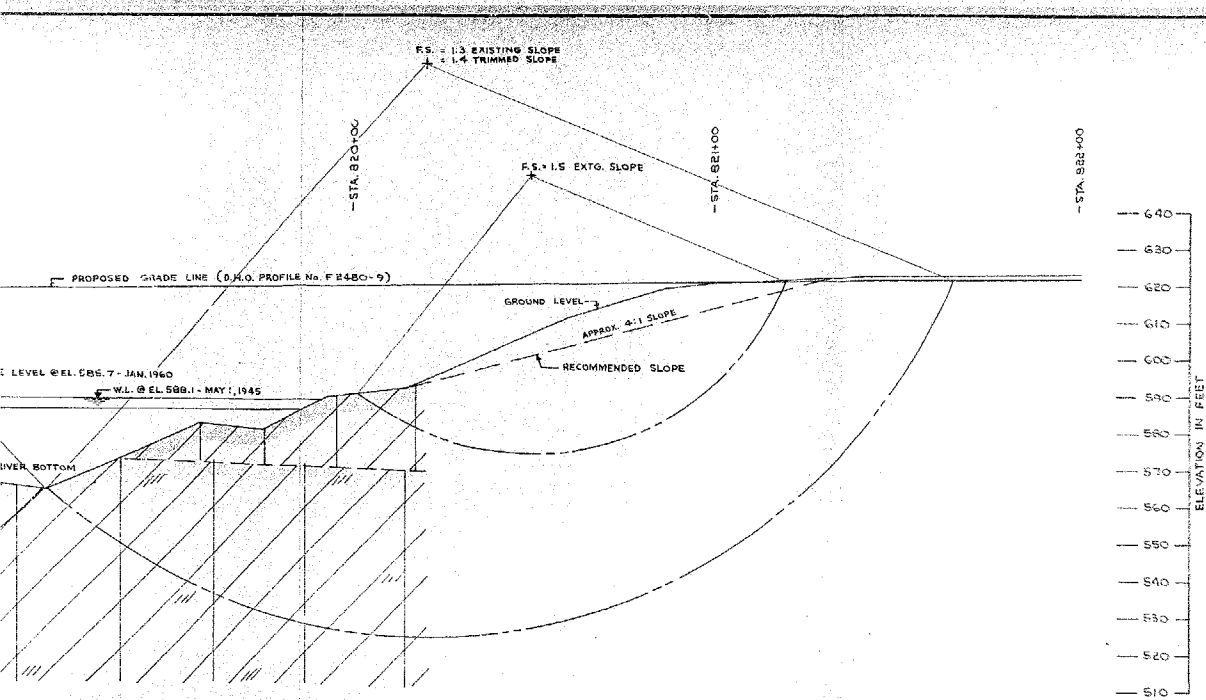
DATE: MAY 12, 1960 SCALE: AS SHOWN

MADE CHKD. APPD.
 J.A. [initials] [initials]
 No. S 7025 - 5



STABILITY ANALYSES — WABI RIVER CROSSING

SCALE: 1" = 20'-0"



NOTE

RIVER BOTTOM PROFILE DETERMINED
BY GEOCON LTD - JAN. 1960

REFERENCE

DEPARTMENT OF HIGHWAYS, ONTARIO - PROFILE No. F2480-9

CROSSING

DEPARTMENT OF HIGHWAYS, ONTARIO		GEOCON LTD	
TORONTO	ONTARIO	DATE: JUNE 6, 1960 SCALE: AS SHOWN	
PROPOSED TRI-TOWN BYPASS STRUCTURES	ONTARIO	MADE CHKD. APPD. J.A. [initials] [initials]	
NEW LISKEARD	ONTARIO		
TOTAL STRESS STABILITY ANALYSES		No. S7025 - 6	
WABI RIVER CROSSING			

60-F-322-C

W.P. # 103-60

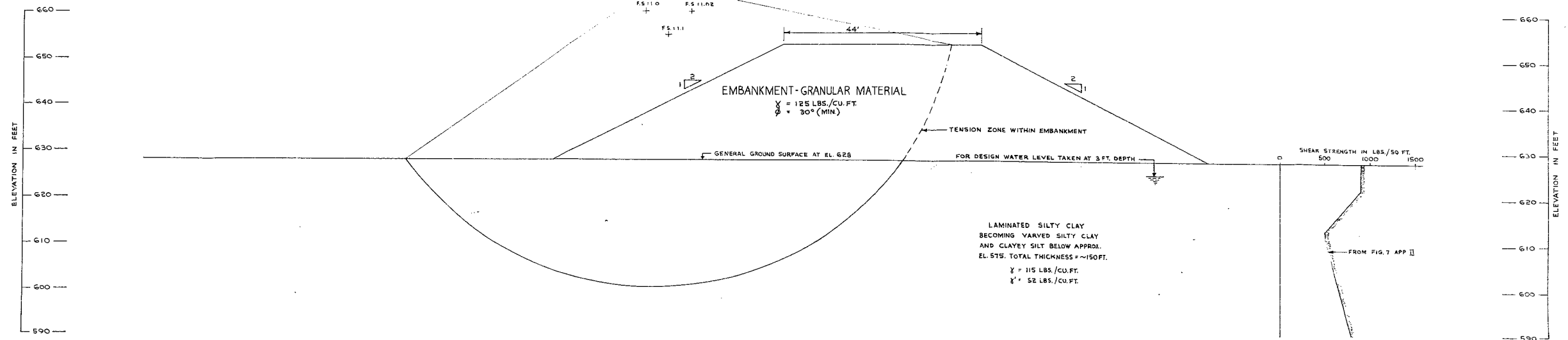
W.P. # 104-60

W.P. # 124-60

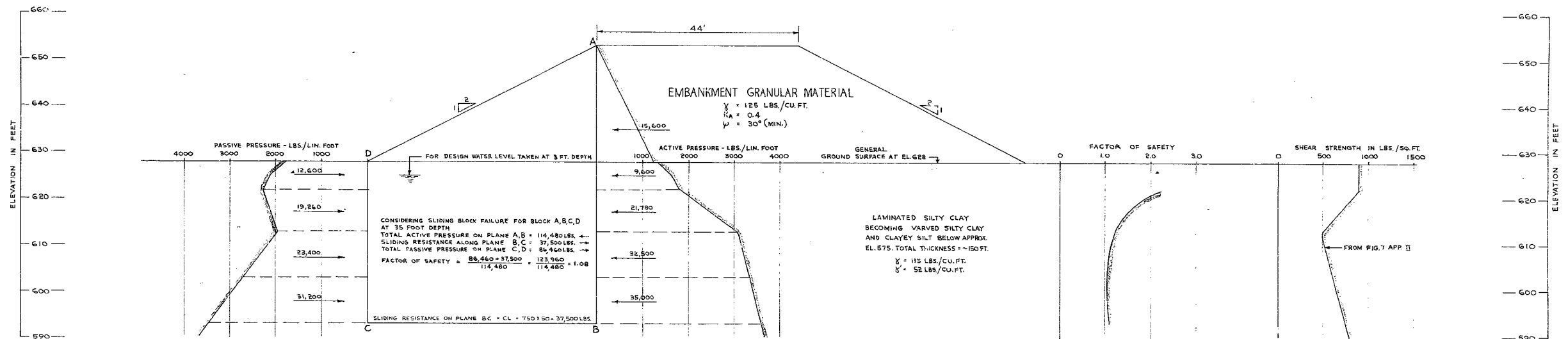
Hwy. # 11 E

TRI-TOWN

By-Pass

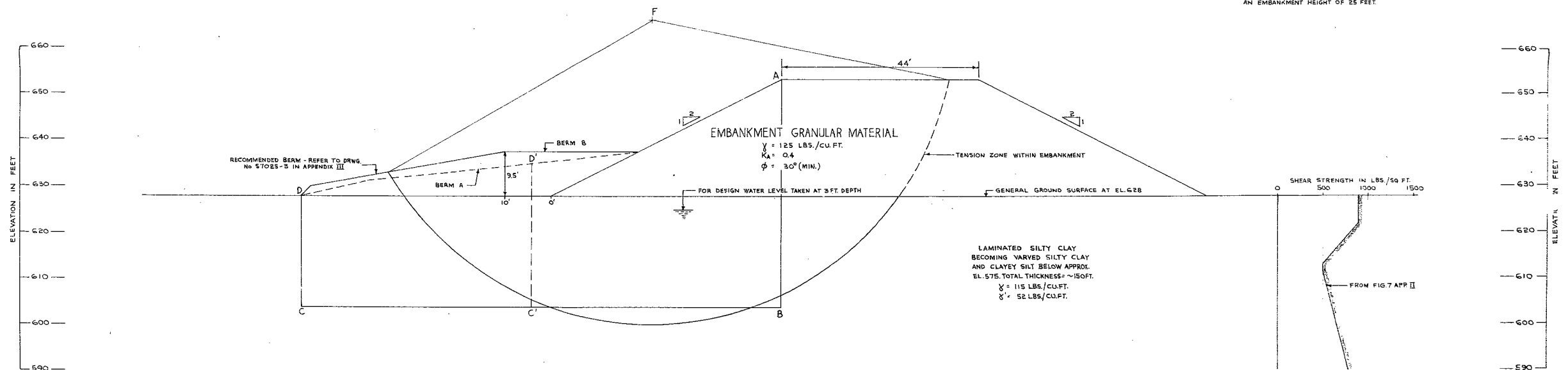


EXAMPLE - CIRCULAR ARC ANALYSIS



EXAMPLE - SLIDING BLOCK ANALYSIS

NOTE
 ALL COMPUTATIONS SHOWN ARE BASED ON AN EMBANKMENT HEIGHT OF 25 FEET.



LEGEND
 BERM 'A' - SIZE OF BERM REQUIRED FOR MINIMUM FACTOR OF SAFETY = 1.3 - SLIDING BLOCK ANALYSIS FOR PLANES ABCD OR ABC'D
 BERM 'B' - SIZE OF BERM REQUIRED FOR MINIMUM FACTOR OF SAFETY = 1.3 - DEEP CIRCULAR ARC ANALYSIS - CIRCLE F

EXAMPLE - DESIGN OF BERM PROFILE REQUIRED

REVISIONS	REVISIONS	REFERENCE	REFERENCE	DEPARTMENT OF HIGHWAYS, ONTARIO	GEOCON LTD
DATE	DATE	DESCRIPTION	DESCRIPTION	TORONTO	DATE MAY 20, 1960 SCALE 1" = 10'-0"
				PROPOSED TRI-TOWN BYPASS STRUCTURES	
				W.P. 103-60	
				HIGHWAY No. 11	
				NEW LISKEARD	
				ONTARIO	
				TYPICAL PROCEDURE	
				DESIGN OF PROPOSED EMBANKMENT	
					NO. S7025-2