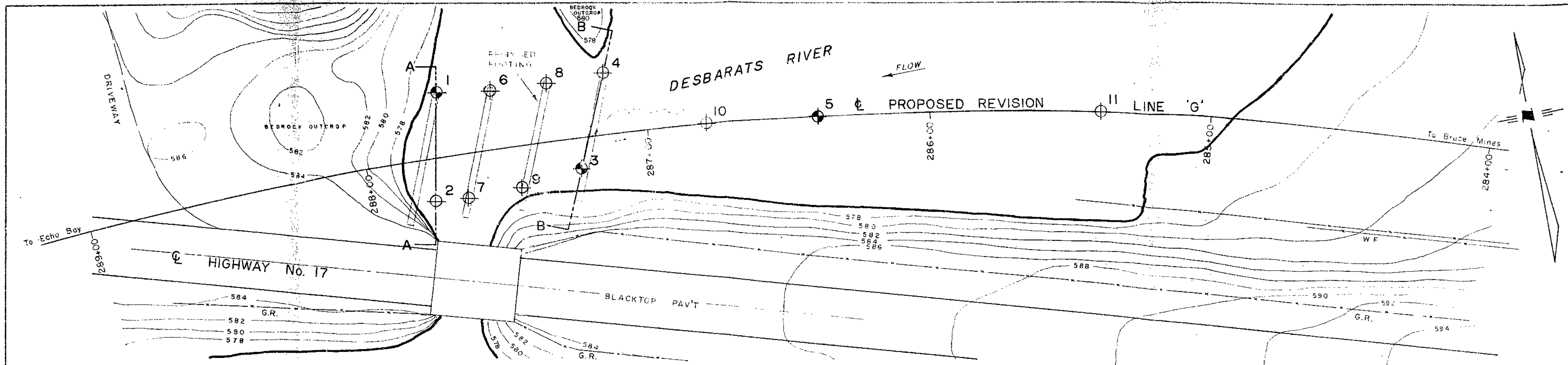
#65-F-222
W.P. #904-64
HWY #17
DESBARATS
RIVER

275100E
5136250N
17
41 J 5W



PLAN

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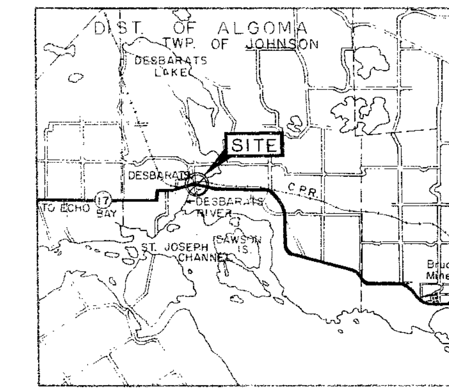
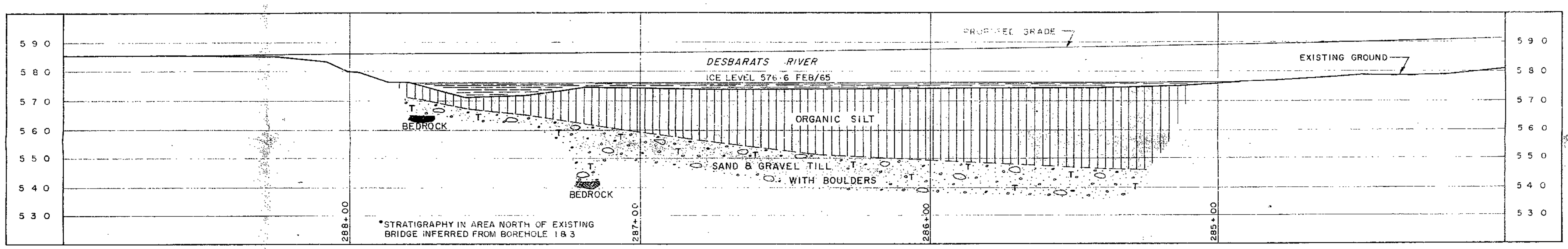


KEY PLAN

2 1 0 2



℄ PROFILE LINE 'G'

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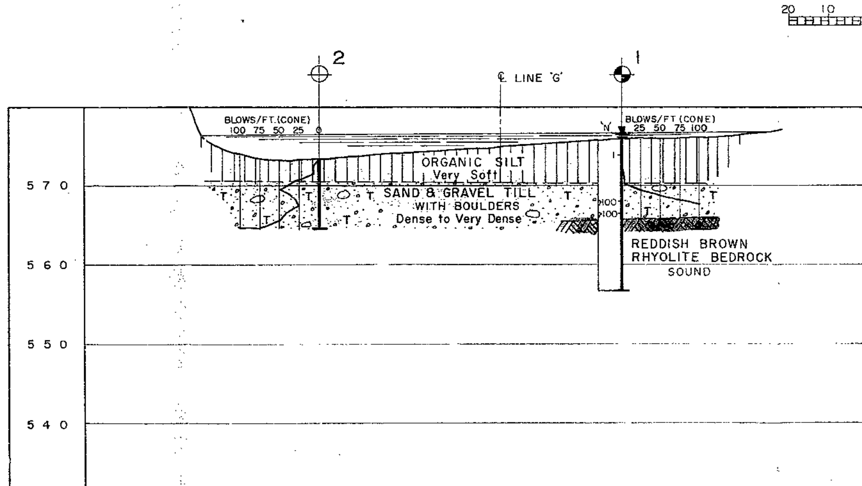
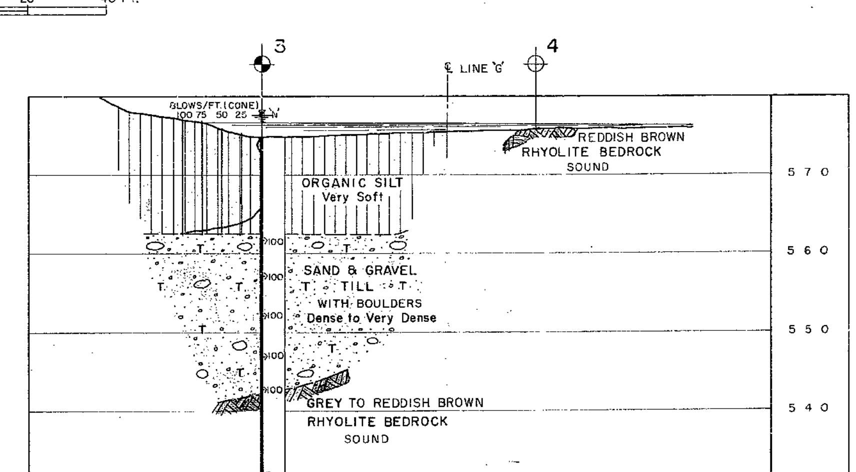


A – A



B – B

SECTIONS

SCALE 10 5 0 10 20 FT.

LEGEND

Artesian Head (Feb. 1965)

(Feb. 1965)

NO.	ELEVATION	STATION	OFFSET
1	576·9	287+72	23' RT
2	576·7	287+79	15' LT
3	576·8	287+25	11' LT
4	576·6	287+13	22' RT
5	576·4	286+40	℄
6	576·6	287+53	21' RT
7	576·6	287+66	16' LT
8	576·6	287+33	21' RT
9	576·6	287+47	15' LT
10	576·5	286+80	℄
11	576·8	285+40	℄

BM. ELEV. 565·05 GEODETIC DATUM
Cut Cross on Conc. Abutment of Bridge
54' Lt. of Sta. 289 37

– NOTE –

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

PRINT RECORD

NO.	FOR	DATE

REVISIONS

DATE	BY	DESCRIPTION

GEOCON LTD

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION – FOUNDATION SECTION

DESBARATS RIVER

KING'S HIGHWAY NO. 17 PROPOSED REV'N LINE 'G' DIST. NO. 18
DISTRICT OF ALGOMA
TWP. JOHNSON LOT 6 & 7 CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D B.T.D.	CHECKED D.B.O.	W.P. NO. 904-64	DRAWING NO. T7723-1
DRAWN A.E.L.	CHECKED B.T.D.	JOB NO.	
DATE MAR. 4, 1965		SITE NO.	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	

REF. No. E-4535-1

GEOCON LTD

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420 MICHEL JASMIN, DORVAL, QUEBEC

TELEPHONE 631-9827

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TEL. 244-6476

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

Rexdale, Ontario,
March 9th, 1965.

Department of Highways, Ontario,
Materials and Testing Division,
Downsview, Ontario.

W.P. 904-64

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

Re: Soil Conditions and Foundations
Proposed Desbarats River Bridge
Desbarats, Ontario

Dear Sirs:

This letter accompanies our detailed report on the above investigation.

We find that the overburden at the site consists of variable thicknesses of very soft organic silt and dense sand and gravel till in this order of occurrence with depth. Bedrock is an igneous extrusive rhyolite. The soil and water conditions encountered are described in detail herein.

The organic soil is not a suitable foundation stratum and should be removed from bridge foundation locations and beneath the approach embankments. As discussed, the most suitable techniques for removal beneath the embankments would probably be the use of displacement of the silt after prior remoulding by blasting. The bridge may be carried on spread footings either on the till or the bedrock, as discussed. A discussion of soil mechanics factors pertinent to construction of bridge foundations at the proposed locations is given in the report.

We believe that this report contains the subsurface information required from this investigation. Kindly call us, however, should you have any questions relative to this report, or if we can be of further assistance otherwise.

Yours very truly,

GEOCON LTD

M. A. J. Matich

M. A. J. Matich, P. Eng.,
President.

MAJM/reb
T7723

T7723

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED DESBARATS RIVER BRIDGE

W.P. No. 904-64

DISTRICT OF ALGOMA ONTARIO

Distribution:

10 copies - Department of Highways, Ontario
Downsview, Ontario

3 copies - Geocon Ltd
Rexdale, Ontario

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INDEX

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INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter dated January 25th, 1965, work permit no. 904-64, to investigate and report on the soil conditions for the bridge to be located on proposed Revised Line "G" over the Desbarats River, approximately 30 feet north of an existing 16.5 foot clear span bridge located along Highway 17, at Desbarats, Ontario. The site is located on Lots 6 and 7, Township of Johnson, District of Algoma, Ontario.

The object of the investigation was to determine and interpret the subsurface conditions as they affect the design of foundations for the proposed bridge.

SUMMARIZED SOIL CONDITIONS

The Desbarats River meanders across the site with no well defined channel; instead it covers a large area with shallow depths of water. The river flows in a south-westerly direction, with the maximum depth of the river as determined from past records being about 6 feet. The surficial stratum is composed of a very soft, compressible dark brown organic silt. The thickness of this stratum in the vicinity of the bridge structures ranged from about 2 to 12 feet with an average thickness of about 6 feet. Moving in an easterly direction across the site the stratum increases in thickness until the depth of the stratum is about 30 feet. Directly underlying the organic clay stratum is a stratum of dense reddish-brown sand and gravel till with boulders; the thickness of this stratum as proven at the borehole locations, varies from 5 to 22 feet. Underlying the till stratum is sound grey to reddish-brown rhyolite bedrock; the bedrock occurred at depths varying from 11 to 35 feet below the ice level at the borehole locations. Artesian water pressure was encountered within the dense till stratum. The readings taken indicated that the artesian water pressure varied from about 1.0 to 2.0 feet above the existing surface ice level.

DISCUSSION

It is understood that a bridge crossing the Desbarats River is planned along proposed Revised Line "G". The bridge will form part of Highway No. 17 between Bruce Mines and Echo Bay Ontario and will be located approximately 30 feet north of the existing 16-1/2 foot span timber crib bridge which it will replace. The location of the structure, as proposed by The Department of Highways, Ontario, is shown on Drawing T7723-1 at the rear of this report.

As presently planned, the bridge deck will have an overall width of 40 feet and elevation 586 approximately, and be supported on abutments and two centre piers. The grade of the approach embankment is specified as 0.50 percent increasing to the east, with an associated maximum elevation of 592.

The discussion which follows deals with the soil mechanics aspects of foundation and earthworks involved in the proposed design.

Approach Embankments

(i) History of Existing Highway No. 17 Approach Embankment in the Vicinity of the Site

The existing approach embankment in the vicinity of the proposed bridge site at Desbarats, Ontario has a maximum height of about 12 feet and side slopes of 1.5 horizontal to 1 vertical (1.5:1). From visual inspection and conversations with local Highway maintenance crews, it is understood that the embankment is composed of rock fill obtained from nearby local sources. It is further understood that the existing embankments were constructed by the displacement method, assisted by blasting where the depth of organic silt was such that displacement could not be effected under the weight of fill only. The existing rockfilled timber cribs forming the bridge abutments are founded directly on the rock fill. The above construction method appears logical in view of the depth

Approach Embankments (continued)

and low shear strength of the organic silt stratum, as discussed later.

(ii) Proposed Approach Embankments

As discussed, the approach embankments will range between elevations 586 and 592. This will involve a maximum height of fill above present ground level of about 18 feet. The history of the existing embankment, together with the measured shear strengths indicate that the organic silt is not capable of supporting an embankment of the height proposed if placed in a single lift. There is precedent for construction of Highway fills on highly organic soils, but these have called for special measures such as the use of light weight materials and stage construction to avoid shear failure of the foundation material. In addition, because of the very compressible nature of such soils,the successful construction of fills on them generally requires the use of surcharge to minimize settlement of the embankment after the Highway is put into operation. Such "on surface" construction must be carefully instrumented and supervised and may require several years to complete for the heights of embankment involved in this case. In view of the limited length of embankment involved

Approach Embankments (continued)

in this instance, and the above time requirements it is assumed for purposes of this discussion that it would be preferable to complete construction of the approach embankments in a single operation. The possibility of "on surface" construction is therefore, not considered further herein.

For completion of the embankment in one operation it would be necessary to completely remove the organic silt from beneath the embankment, either be excavation in advance or by displacement during construction. In view of the depth of organic silt involved the most economical removal method, as indicated by the experience of others, is by displacement during construction. In some areas the depth of organic silt is shallow and displacement to the underlying till could probably be achieved under the self-weight of the fill alone, if placed by end-dumping to full height. In areas where the depth of organic silt is appreciable, that is, of the order of 30 feet in the easterly portion of the site, it is possible that complete sinkage of the rock fill to the base of the underlying till may not be achieved under the weight of the fill alone. In these areas some floating of the fill may occur with resultant irregular future settlements of the roadway surface due to consolidation and progressive dis-

Approach Embankments (continued)

placement of the trapped soil.

It is recommended therefore that steps be taken to ensure that complete displacement takes place during construction. A number of methods have been used in the past to effect displacement of soft soils, including surcharging of the embankment, jetting to weaken the material to be displaced, and blasting immediately in advance of fill placement. Previous experience on the Rainy Lake Causeway project indicated that to achieve positive displacement with depths of greater than 8 feet of varved clay, it was necessary to lower the shear strength of the clay to at least 100 pounds per square foot. It is considered therefore, that by remoulding the silt to a shear strength of 100 pounds per square foot or less prior to fill placement, satisfactory displacement would be achieved with end-dumping of fill to full height. The most positive method of reducing the shear strength to the required value necessary for displacement, is in this instance, considered to be the use of blasting techniques.

Approach Embankments (continued)

Experience on the Rainy Lake Causeway project indicated that maximum remoulding was effected by the combined effect of a group of charges placed near the base of the stratum concerned. The results also indicated that a powder factor of about one pound of dynamite for each cubic yard of clay was necessary to effect the desired lowering of the shear strength below 100 pounds per square foot. Further, by using 50 percent forcite ditching dynamite it was possible to effect explosion by propogation. Using discharge by a single "primer" it was found that 50 pound pockets of dynamite would explode by propogation if spaced at 18 feet intervals or less. Successful displacement was achieved using 50 pound pockets spaced at 10 foot intervals, at a depth equal to two-thirds of the clay thickness with each blast containing at least 3000 pounds of dynamite.

Obviously the experience of Rainy Lake cannot be applied directly to this site because of the difference in soil conditions, and water depths involved. It is probable that the organic silt will require a lower powder factor than the clay at Rainy Lake because of its already very low in-situ shear strength. The actual powder factor required, however,

Approach Embankments (continued)

cannot be reliably established theoretically and it is recommended that the most suitable blasting pattern be established experimentally either before or during construction. It is believed however, that the general approach with regard to pattern of explosive charges as used at Rainy Lake would still apply, and this could form the basis of initial trials in the field.

It is recommended that the embankment be constructed of hard durable rock fill which is known to be available in the vicinity. Such rhyolite type rock fill was used in the construction of the Canadian Pacific Railway Embankment, the existing Highway 17 embankment, and the rock fill timber crib bridge structure all located in the Desbarats area. As indicated by the existing embankment, the side slopes of fill end-dumped after remoulding of the silt would be approximately 1.5 horizontal to 1 vertical.

It would be desirable to construct the embankment in advance of the bridge structure because of the factors involved with the displacement method. It may be desirable also to provide a short section of sand and gravel in the

Approach Embankments (continued)

embankment at the juncture with the bridge.

Foundations

The soil conditions are suitable for the use of spread footings for the proposed abutments and piers founded, on or within, the granular till stratum or on bedrock. In some instances mainly in the northern extremity of the foundation locations, bedrock is encountered under a thin cover of till; at these locations the spread footings would be carried on the bedrock.

Considering the size of footing that would probably be involved, it is recommended that an allowable bearing value of 3.0 tons per square foot be used for the till stratum. For spread footings carried entirely on bedrock a maximum allowable bearing value of 20 tons per square foot could be used for design. For abutments or piers founded partly on bedrock and partly on the till the maximum total settlements will also be the differential settlements. Under the above design values the total settlement of the abutments founded as described should not be more than one half inch. Settlement will, for practical purposes, take place concurrently with application of load and will therefore be largely completed at the end of construction.

Foundations (continued)

Consideration might be given to founding the bridge on piles end-bearing in the till or on bedrock, and a number of piles would be suitable for this purpose. Because of the shallow depth to bedrock, however, such as at the location of test pit 4, the use of piles would still involve carrying at least part of one or more of the foundations on bedrock directly. The comparatively short depth to bearing stratum, elsewhere, may preclude the use of piles from a practical viewpoint.

In view of the fact that bedrock outcrops a short distance from the proposed abutment locations, construction would be simplified if alignment and bridge design considerations permitted shifting the bridge location a short distance northwards and using a slightly longer and single span structure. In such a case foundations could be more conveniently constructed in the dry directly on bedrock.

Footings, or pile caps if involved, subject to frost action should be provided with a minimum of 5 feet of protective earth cover.

In design of abutments founded on spread footings in the till, it is recommended that a coefficient of lateral earth pres-

Foundations (continued)

sure of 0.4 be used for the backfill which should be select well compacted clean granular material. For abutment footings founded on bedrock, a value of 0.5 is recommended.

The abutments and piers should be designed for a factor of safety of at least 1.5 against sliding based on a coefficient of friction of concrete to granular till of 0.45. For footings on bedrock a coefficient of 0.35 should be used. In the latter case, the resistance to lateral movement could, if desired, be increased by dowelling into bedrock.

It is recommended that consideration be given to provision of rip-rap or other suitable measures for protection of foundations carried on the till against possible undermining by scour. The actual requirements will depend on the bridge design selected, on the maximum fluctuation in stream flow, and on the constriction to the present stream valley caused by the approach embankments. They are thus dependent on design and hydraulic considerations beyond the scope of this report.

Construction of spread foundations will involve excavation of as much as 12 feet of organic material at a depth of as much as 14 feet below present water level. A higher water head differential

Foundations (continued)

may have to be contended with if construction was carried out during high stages of the river. Construction could probably be best effected in the dry by the use of braced steel sheet pile cofferdams where the sheeting is driven a suitable distance into the till, or to bedrock as discussed later. The distance which sheeting should penetrate into the till, to provide safety against piping by upward action of seeping water, would depend on the maximum hydrostatic head differential allowing for artesian effects, and on the width of the cofferdam. As a preliminary guide, sheeting should be carried below base of excavation a depth at least equal to the maximum head difference after dewatering. In effect, therefore, penetration to bedrock throughout would probably be required. However, because of the very dense nature of the till and its boulder content, it is unlikely that an intact continuous cut-off to bedrock could be achieved by driving sheeting through the till without special provisions to facilitate penetration. In view of this, and the presence of artesian pressures in the till, consideration could be given to driving the sheeting a sufficient distance into the till to obtain the necessary toe-hold and to effect dewatering by a system of filter equipped screened pumped wells penetrating the till. The installation of such wells, or wellpoints, would have to contend with boulders in the till and its dense nature otherwise.

1. The site is covered by a stratum of organic silt varying in encountered thickness from 2 to 30 feet. Underlying this stratum is a dense sand and gravel till with boulders stratum which in turn is underlain by a sound rhyolite bedrock.

2. The site is partially covered by the Desbarats River which flows in a south-west direction. At the time of the investigation the maximum depth of the river was 6 feet. Artesian pressure was encountered within the till stratum as discussed in this report.

3. The approach embankments may be constructed of locally available rock fill. This rock fill should be founded on the till stratum, with the organic silt overlying this till being positively removed in advance by one of several techniques outlined in this report.

4. The bridge structure as proposed, may be carried on spread footings, founded on the till or directly on bedrock at allowable bearing values as discussed in the report. The use of end-bearing piles might also be considered, as discussed.

5. Soil mechanics considerations pertinent to construction of abutments and piers are discussed in the report.

PERSONNEL

The field work was carried out under the supervision of Mr. B. T. Darch. This report was written by Mr. Darch, checked by Mr. D. B. Oates, P. Eng. and reviewed by Mr. M. A. J. Matich, P. Eng.

B. T. Darch

BTD/reb

B. T. Darch, P. Eng.,
Senior Soils Engineer.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SURFACE ICE AND WATER CONDITIONS

SOIL AND WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

PROCEDURE

The field work was carried out between February 1st, 1965 and February 9th, 1965. A total of 3 boreholes, each with an accompanying dynamic uncased cone penetration test, and 8 additional dynamic uncased cone penetration tests were put down in BX size. For the work a Penndrill using power auger techniques was used.

The soil strata were sampled at intervals not exceeding 5 feet. Two and three inch Osterberg thin walled tube samples were taken in the organic stratum of the overburden; the three inch tube samples were taken at the top of the stratum where the boreholes stayed open without the need of casing. Three inch diameter in-situ vane shear strength tests were carried out in the organic stratum in two of the boreholes. The granular strata were sampled using a 2 inch split spoon sampler adapted where necessary with a foot valve to aid in the recovery of the material. The bedrock was proven in boreholes 1 and 3 for about 10 feet, by rock core drilling in BXL size. Piezometers were installed in boreholes 3 and 5.

Detailed logs of the boreholes and uncased dynamic cone penetration tests are presented on the Office Reports on Soil Exploration in this Appendix. The locations of the boreholes and dynamic penetration tests, together with the inferred soil stratigraphy are shown on Drawing T7723-1, at the rear of this report.

The laboratory testing of selected soil samples was carried out in the Toronto Soil Mechanics Laboratory of Geocon Ltd. The results are plotted on the Office Reports on Soil Exploration in Appendix I and the Figures in Appendix II. The soil samples remaining after testing will be stored until February 27th, 1965 at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic datum. The bench mark referred to is the Department of Highways, Ontario Geodetic Bench Mark which is a Cut Cross on the concrete abutment of the Bridge, 54 feet left of Station 289+37 along Proposed Revision Line "G". The Geodetic elevation of this bench mark is 585.05. The location of the bench mark is shown on the Department of Highways, Ontatio (D. H. O.) Drawing E-45345-1.

SITE AND GEOLOGY

The proposed D. H. O. bridge site is to be located on Revised Line "G" over the Desbarats River, approximately 30 feet north of a 16.5 feet clear span bridge located along existing Highway 17 at Desbarats, Ontario. The site is located on Lot 6 and Lot 7, in the Township of Johnson, District of Algoma, Ontario. The area is low lying in a natural valley with higher ground surrounding it on all sides. To the west of the bridge site the ground rises sharply with bedrock outcrops in evidence. To the east, north and south the terrain is flat lying supporting light marsh vegetation of grass and stunted brush. Directly to the south of Revised Line "G" is the approach embankment of existing Highway No. 17 having a maximum height of about 15 feet.

The Desbarats River meanders across the site, flowing from the north-east to the south-west. The River has no permanent channel, it inundates a large area to the north-east with shallow depths of water. From past river surveys on the site carried out by the D. H. O., it is estimated that the maximum depth of the Desbarats River is about 6 feet.

From available geological information and an inspection of the area, it is known that the overburden is an organic soil deposited during recent times, underlain by a sandy till of glacial origin, which in turn overlies rhyolite bedrock of the early Proterozoic Era of the Precambrian Period. The rhyolite which is grey to reddish-brown in colour, and high in silica content, is a well cemented variety.

SURFACE ICE AND WATER CONDITIONS

The proposed locations of the abutments and piers are within the river bed of the Desbarats River. At the time of the investigation the ice on the site varied from 1 to 2 feet in thickness, with the thicker sections occuring in the river channel and the thinner sections on the banks of the river. During this period the total depth of ice and water encountered at the boreholes and at the locations of the dynamic penetration tests varied from 2 to 4 feet across the site. From past surveys it is estimated that the maximum depth of the Desbarats River would be about 6 feet, as already mentioned.

SOIL AND WATER CONDITIONS

The principal soil strata encountered in boreholes 1, 3 and 5 were as follows:

Very Soft Dark Brown Highly Organic Silt

The surficial overburden stratum across the site is a dark brown highly organic silt containing wood fiber, roots and decayed matter; it is inferred that this material covers the entire site for the proposed bridge and associated approach embankments. This material is very plastic, peat like, and highly compressible. The surface elevation of the stratum varies from 573 to 576 across the site with a corresponding variation in thickness from 5.5 feet at borehole 1 to 23 feet at borehole 5; however, from the results of the dynamic uncased cone penetration tests the thickness is inferred to be as much as 30 feet. Within the area bounded by the abutments the maximum thickness of this stratum encountered was about 12 feet, however, moving in an easterly direction the stratum thickness probably increases to about 30 feet. The top 2 feet of the stratum is highly organic with a large percentage of wood fiber and can be considered to be a peat.

Seven natural moisture content determinations were made on samples from the stratum; the values ranged from 97 to 152 percent being typically about 130 percent. The corresponding wet unit weights of three samples tested were 76, 77 and 81 pounds per cubic foot. These results are shown on the individual Office Reports on Soil Exploration in this Appendix.

Seven Atterberg limit determinations were carried out on typical samples from the stratum. The tests indicated that the liquid limit ranged from 137 to 170 , being typically about 150, and plastic limits ranging from 49 to 59 with an average of about 53. The Unified Soil Classification System classifies the material as organic silt of high plasticity. The Atterberg limit results are shown on the Plasticity Chart, Figure 2 of Appendix II and also on the Office Reports on Soil Exploration in this Appendix.

Three unconfined compression strength tests were performed on typical samples from the stratum and gave shear strengths (taken as one half of the compressive strength) of 70, 120 and 140 pounds per square foot, respectively. It is believed that unavoidable disturbance during sampling explains the low values obtained from laboratory unconfined strength testing as compared to the results of the field and laboratory vane tests given below. Seven 3-inch in-situ vane shear strength tests were carried out in the stratum t boreholes 3 and 5. The results of these tests indicated that the undisturbed shear strength ranged from 120 to 210 pounds per square foot with an average of about 150 pounds per square foot. Remoulded vane shear strength tests were carried out in conjunction with the undisturbed tests and gave remoulded shear strength values ranging from 40 to 70 pounds per square foot. Three-quarter inch laboratory vane shear strength tests were carried out on six Osterberg tube samples; two determinations were made on each tube tested. These tests indicated that the undisturbed shear strength of the material varied from 160 to 200 pounds per square foot, and average about 170 pounds per square foot. Remoulded vane shear strength tests were carried out in conjunction with the undisturbed tests and gave remoulded shear strength values ranging from 20 to 80 pounds per square foot with an average of about 60 pounds per square foot.

Based on the above results, the consistency of the organic silt is estimated to be very soft. The strength test results are plotted on Figure 4 of Appendix II and on the Office Reports on Soil Exploration in this Appendix. The sensitivity of the organic silt, established by means of the remoulded characteristics of the field and laboratory vane tests and visual and tactile examination is typically of the order of 3 to 4 indicating that the deposit is moderately sensitive to disturbance.

A consolidation test was performed on a typical sample from the stratum and the results are shown on the "Pressure vs. Void Ratio" curve, Figure 3 in Appendix II. The results of the tests indicate that the deposit is normally consolidated. The compression index "Cc" obtained from the curve is 1.15.

The uncased dynamic cone penetration tests carried out met little resistance while passing through the stratum, the "A" rods penetrating the stratum under the weight of the hammer.

Dense to Very Dense Reddish-Brown Sand and Gravel Till with Boulders

Directly underlying the organic clay is a stratum of reddish-brown sand and gravel till with a high boulder content. The surface elevation of the stratum across the site varies from about

550 to 570. The thickness of the stratum was proven to be 4 5 feet at borehole 1 and 20. 5 feet at borehole 3; these two boreholes were the only holes in which this stratum was penetrated completely. The stratum has a matrix of sand with some silt, binding gravel sizes and boulders; the granular component of the stratum is sub-angular in shape. Boulders up to 8 inches in size were encountered while penetrating the stratum, although it is considered possible that larger boulders are present at other than borehole locations. The boulders were random in mineralogical composition being very often granitic. A sand and gravel layer was encountered within this stratum at boreholes 3 and 5, and from the results of the dynamic cone penetration tests the stratum is inferred to exist at other locations. The layer was found at elevations varying from 540 to 550 across the site and was generally about 3 feet thick.

Three mechanical analysis tests were carried out on typical samples from the stratum and the results are shown on Figure 1 of Appendix II. The grain size curves indicated that the material ranges from 12 to 19 percent silt sizes, 38 to 46 percent sand sizes, and 40 to 45 percent gravel sizes. One mechanical analysis was carried out on a sample from the sand and gravel layer; the grain size curve indicated that the material consisted of 2 percent silt sizes, 53 percent sand sizes and 45 percent gravel sizes. From observing the wash water it is believed that parts of the layer are higher in sand content than indicated from this test. It is probable that the gravel component of the tested sample is exaggerated by cave from higher levels in the open borehole.

Nine standard penetration tests were carried out in the stratum and gave "N" values ranging from 51 to greater than 100 blows per foot with an average of about 80 blows per foot. It should be noted that some of these "N" values may have been increased by the high gravel content of the stratum. One additional standard penetration test was carried out in the sand layer described above and gave an "N" value of 21 blows per foot. This result was confirmed by the drop in resistance of the adjacent dynamic penetration test which was put down in advance of the borehole. The uncased dynamic cone penetration tests met practical refusal within this stratum. From these results it is estimated that the relative density of the stratum varies from dense to very dense and is generally very dense. This is confirmed by the fact that normal augering could not penetrate any significant distance into the stratum.

For design purposes, the following parameters may be used, where appropriate:

Wet unit weight	140 pounds per cubic foot
Submerged unit weight	78 pounds per cubic foot
Angle of shearing resistance,	40 degrees.

Sound Grey to Reddish-Brown Rhyolite Bedrock

Directly underlying the sandy till stratum in boreholes 1 and 3 is grey to reddish-brown rhyolite bedrock; bedrock was not proven in borehole 5. The bedrock encountered in both boreholes was proven by diamond core drilling in BXL size for a depth of about 10 feet. The surface elevation of the bedrock was 566 in borehole 1 and 542 in borehole 3. The core recovered was high in silica content, and sound, as evidenced by the high core recovery, and the appearance of the core.

The bedrock is well cemented and massive with an estimated hardness of about 6 on the Moh's hardness scale. The dominant component minerals are quartz and feldspar. The fracture patterns are uneven and occur along prescribed planes of weakness developed during formation of the bedrock.

Water Conditions

The water level was observed during the investigation in piezometers installed in boreholes 3 and 5 and the open hole of borehole 1. Artesian water pressures were encountered within the till stratum at boreholes 3 and 5. The artesian water levels at boreholes 3 and 5, measured on February 9th, 1965, were 10 inches and 22 inches above ice level, respectively. The water level observed at borehole 1 corresponded with the river water level at the time of the investigation.

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:

Consistency	U-Strength Tons/sq. ft.
Very soft	0.03 to 0.25
Soft	0.25 to 0.5
Firm	0.5 to 1.0
Stiff	1.0 to 2.0
Very stiff	2.0 to 4.0
Hard	over 4.0

Relative Density	Standard Penetration Resistance. Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

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

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 BORING # 1 And PT. 2 DATUM GEODETIC CASING BX.
BORING DATE FEB 1-3/65 REPORT DATE FEB. 22, 1964 COMPILED BY A.E.L. CHECKED BY B.T.D.
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
- 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 WALLED OPEN
- R.C. - ROCK CORE

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

SOIL PROFILE: ELEVN. / DEPTH | WATER CONDITIONS | DESCRIPTION | STRAT. PLOT | ELEVATION SCALE

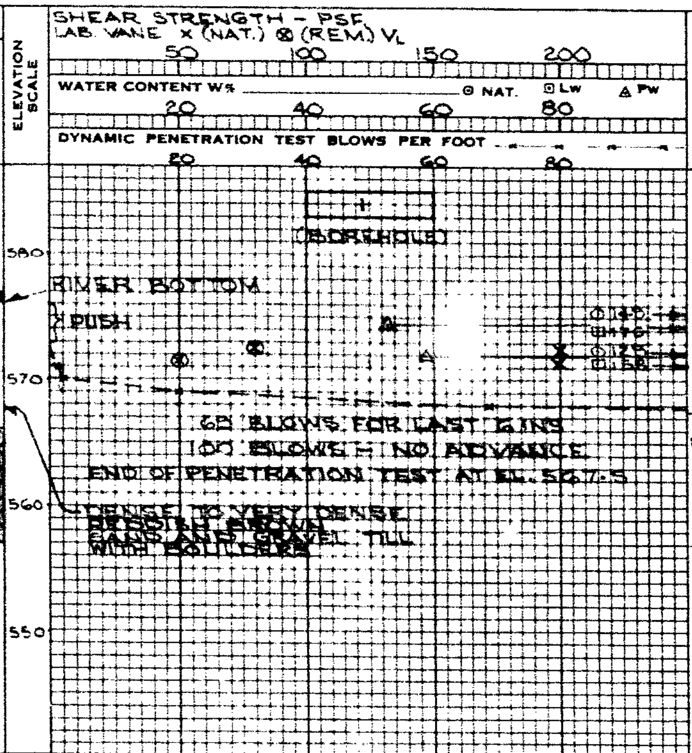
SHEAR STRENGTH - PSF: LAB. VANE x (NAT.) ⊗ (REM.) V_L — 50, 100, 150, 200
WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw — 20, 40, 60, 80
DYNAMIC PENETRATION TEST BLOWS PER FOOT — 20, 40, 60, 80

OTHER TESTS | SAMPLES: CONDITION | TYPE | NUMBER | PENETRATION RESISTANCE BLOWS/FT.

1 (BOREHOLE)

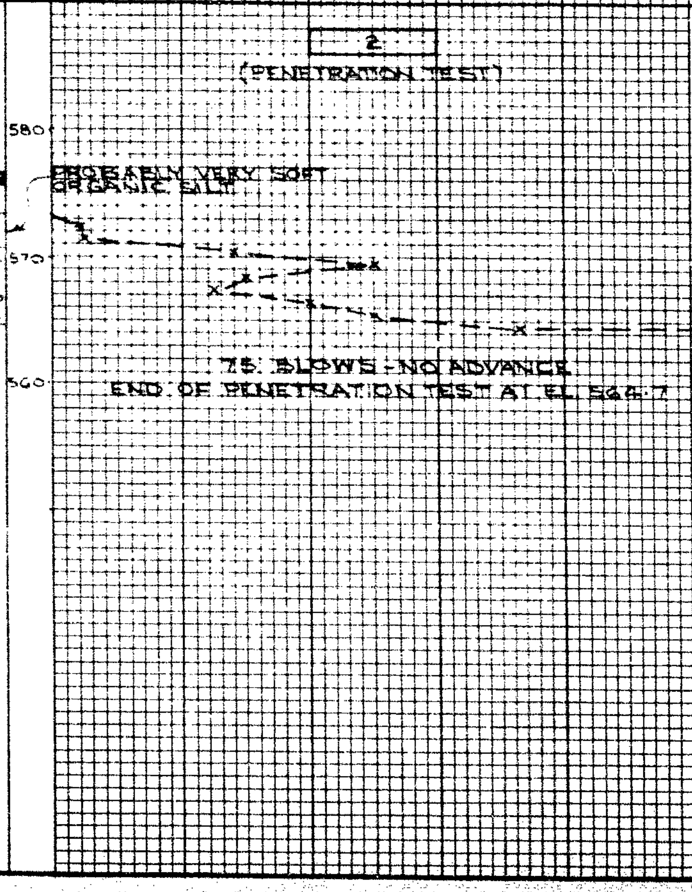
ELEVN. / DEPTH	WATER	DESCRIPTION
576.9	▼	RIVER LEVEL
575.0 / 0.0		0.9' ICE
570.4 / 5.6		VERY SOFT DARK BROWN ORGANIC SILT
565.9 / 10.1		
556.9 / 19.1		SOUND REDDISH BROWN RHYOLITE BEDROCK
		END OF HOLE

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		2" DO	1	1
V		3" TO	2	PUSH
		2" DO	3	>100
M		" DO	4	>100
R.C. RECOV. 100%		BXL. RC	5	–
58%		"	6	–



2 (PENETRATION TEST)

ELEVN. / DEPTH	WATER	DESCRIPTION
576.7	▼	RIVER LEVEL
		3.3' ICE
573.4 / 0.0		RIVER BOTTOM
570.7 / 2.7		
564.7 / 8.7		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL
		END OF PEN TEST



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 BORING # 3 And TP 4 DATUM GEODETIC CASING BX.
BORING DATE FEB. 4-5/65 REPORT DATE FEB. 22, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.
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

- 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 WALLED OPEN
- R.C. - ROCK CORE

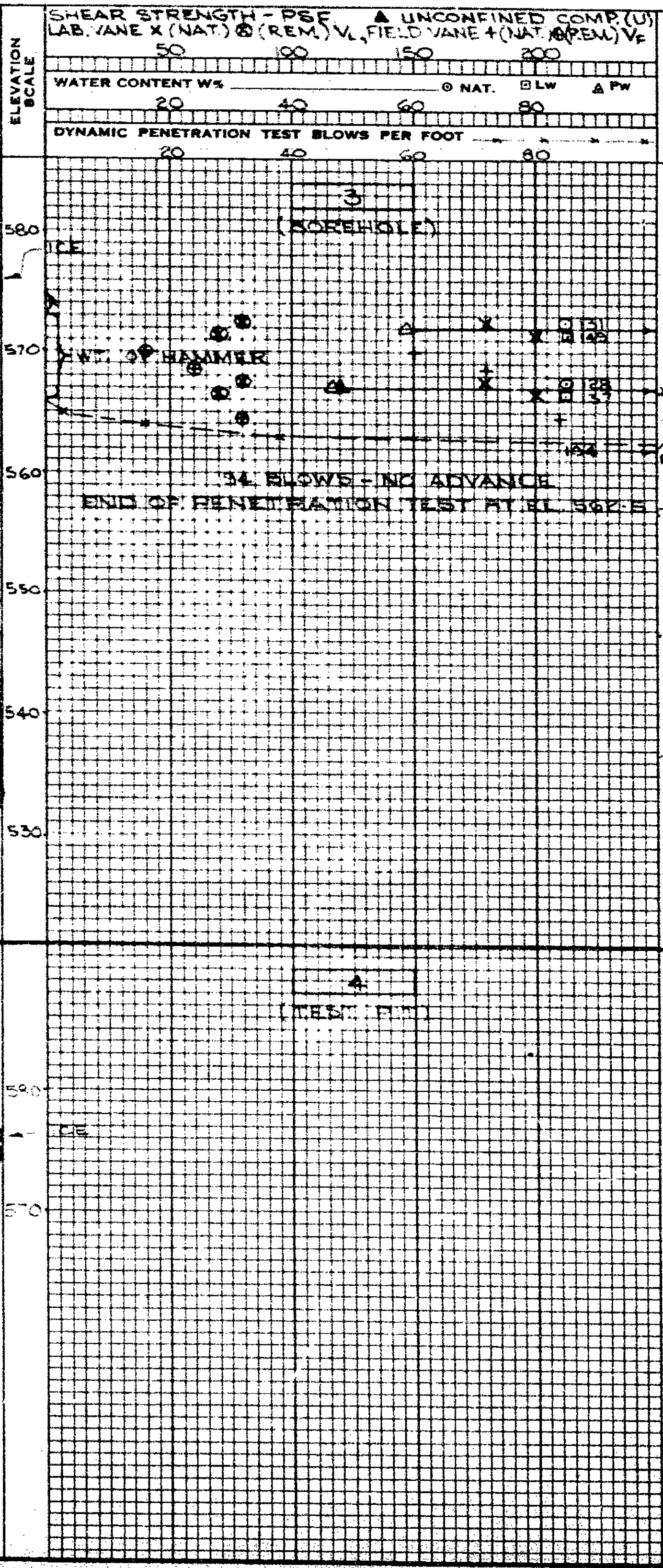
ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED UNDRAINED
- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT P.C.F.
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

3 (BOREHOLE)

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
576.8	▼	RIVER LEVEL					
574.9 / 0.0		RIVER BOTTOM					
	BENTONITE PLUG	VERY SOFT DARK BROWN ORGANIC SILT	V	DISTURBED	2" DO	1	PUSH
			V	FAIR	3" TO	2	"
			γ=77 V	FAIR	TO	3	"
562.5 / 14.3	SAND	DENSE TO VERY DENSE REDDISH BROWN SAND & GRAVEL TILL WITH BOULDERS	M	DISTURBED	2" DO	4	>100
	PIEZOMETER		RC. RECOV. 31%	DISTURBED	BXL RC	5	—
			M		2" DO	6	>100
					DO	7	>100
					DF	8	>100
541.8 / 33.1					DF	9	>100
	WL. EL. FEB.	SOUND REDDISH BROWN RHYOLITE BEDROCK	88%	GOOD	BXL RC	10	—
532.1 / 42.8			100%	GOOD	"	11	—
		END OF HOLE					



4 (TEST PIT)

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
576.6	▼	RIVER LEVEL
575.[illegible] / 0.0		RIVER BOTTOM
		REDDISH BROWN RHYOLITE BEDROCK

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 BORING # 5 And PT. 6 DATUM GEODETIC CASING BX.
BORING DATE FEB. 6-8/65 REPORT DATE FEB. 23, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLF 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 WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc. - TRIAXIAL CONSOLIDATED UNDRAINED
- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT PCF.
- K - PERMEABILITY
- C - CONSOLIDATION
- WL. - WATER LEVEL IN CASING
- WT. - WATER TABLE IN SOIL

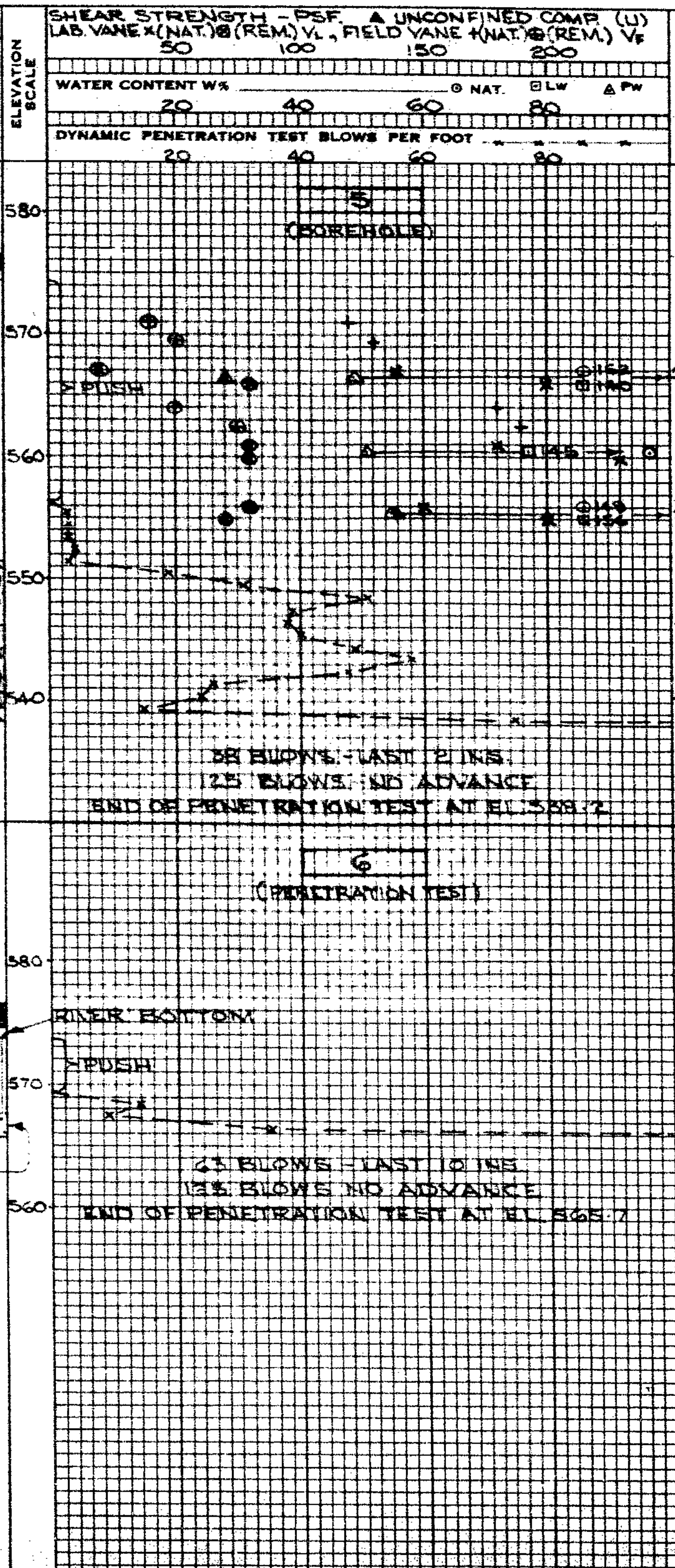
SOIL PROFILE

5 (BOREHOLE)

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
576.4		RIVER LEVEL
574.9		1.5 ICE
0.0	SAND / BENTONITE PLUG	RIVER BOTTOM
	WL. @ EL. 578.2 FEB. 8, 1965.	VERY SOFT DARK BROWN ORGANIC SILT
551.4 / 22.9	PIEZOMETER	DENSE TO VERY DENSE REDDISH BROWN SAND & GRAVEL TILL WITH BOULDERS
539.4 / 34.9		END OF HOLE
36.1		END OF PEN. TEST

6 (PENETRATION TEST)

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
576.6	▼	RIVER LEVEL
573.8		2.0 ICE
0.0		PROBABLY VERY SOFT ORGANIC SILT
567.6		
565.7 / 8.1		END OF PEN. TEST PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL



OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
V_F, V_F		2" TO - DF	1	PUSH
C, γ=76, V_L, V_L		3" TO	2	"
V_F, V_F, V_L, V_L		2" TO	3	"
γ=81, V_L, V_L		- TO	4	"
		- DO	5	62
		- DO	6	73
M		- DO	7	21

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 PEN. TEST [redacted] # 7, 8 And 9 DATUM GEODETIC CASING —
BORING DATE FEB 8-9/65 REPORT DATE MAR. 1, 1965 COMPILED BY A.E.L. CHECKED BY S.T.D.
[redacted] 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

- 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 WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

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- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE							SAMPLES			
ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% — ⊙ NAT. ⊡ Lw △ Pw / DYNAMIC PENETRATION TEST BLOWS PER FOOT (20, 40, 60, 80)	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
				580	7 (PENETRATION TEST)					
576.6	▼	RIVER LEVEL								
		1.5 ICE								
572.7 / 0.0		RIVER BOTTOM								
		PROBABLY VERY SOFT ORGANIC SILT		570	PUSH					
567.6 / 5.1		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL								
562.1 / 10.6		END OF PEN. TEST		560	125 BLOWS - LAST 6 INS. 125 BLOWS NO ADVANCE END OF PENETRATION TEST AT EL. 562.1					
				580	8 (PENETRATION TEST)					
576.6	▼	RIVER LEVEL								
		1.9 ICE								
572.1 / 0.0		RIVER BOTTOM								
		PROBABLY VERY SOFT ORGANIC SILT		570	PUSH					
566.1 / 6.0		END OF PEN. TEST			6 BLOWS - LAST 6 INS. 105 BLOWS NO ADVANCE					
				560	END OF PENETRATION TEST AT EL. 566.1					
				580	9 (PENETRATION TEST)					
576.6	▼	RIVER LEVEL								
		1.3 ICE								
573.7 / 0.0		RIVER BOTTOM								
		PROBABLY VERY SOFT ORGANIC SILT		570	PUSH					
564.6 / 9.1										
561.3 / 12.4		END OF PEN. TEST		560	128					
		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL			88 BLOWS - LAST 3 INS. END OF PENETRATION TEST AT EL. 561.3					

APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 PEN. TEST [redacted] # 10 And 11 DATUM GEODETIC CASING —

BORING DATE FEB 9/65 REPORT DATE MAR. 1, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.

[redacted] 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
- 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 WALLED OPEN
- R.C. - ROCK CORE

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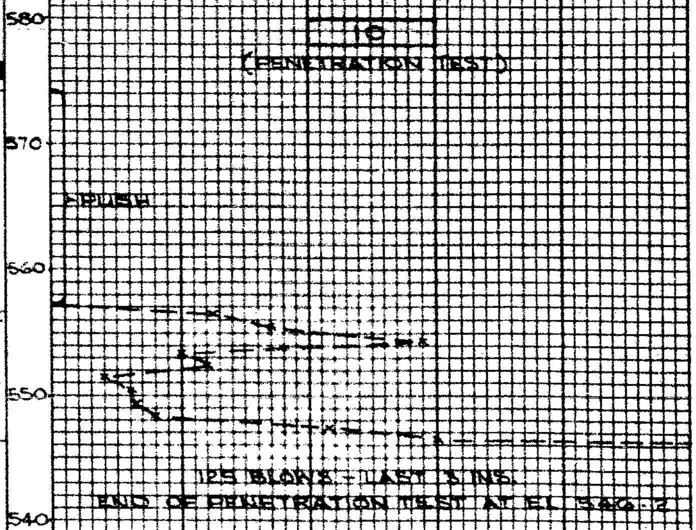
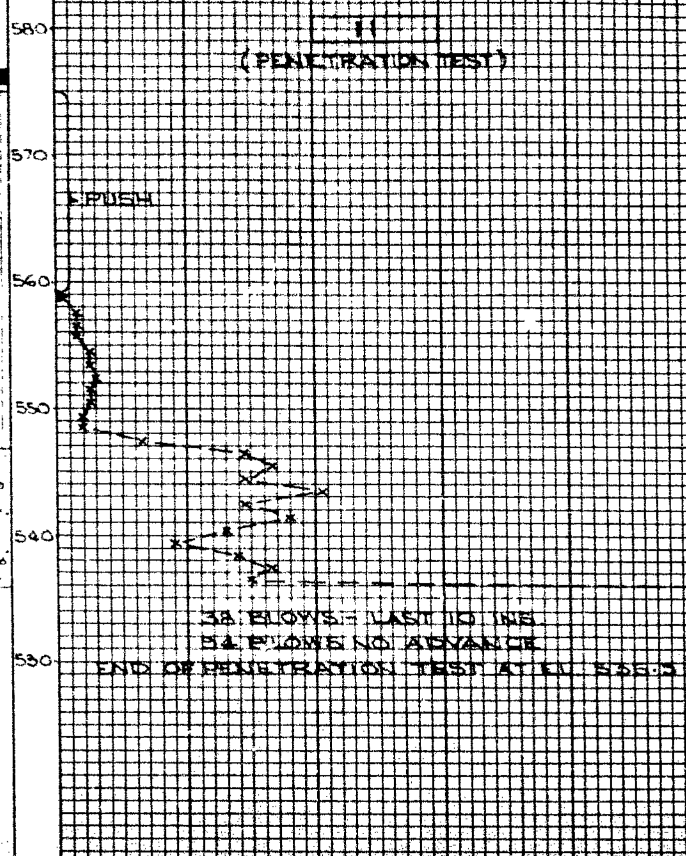
- Qc - TRIAXIAL CONSOLIDATED UNDRAINED
- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT
576.5	▼	RIVER LEVEL	
574.2		1.3' ICE	
0.0		RIVER BOTTOM	
		PROBABLY VERY SOFT ORGANIC SILT	
558.5 / 17.7		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL	
546.2 / 28.0		END OF PEN. TEST	
576.8	▼	RIVER LEVEL	
575.0		1.8' ICE	
0.0		RIVER BOTTOM	
		PROBABLY VERY SOFT ORGANIC SILT	
546.8 / 28.2		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL	
535.9 / 39.1		END OF PEN. TEST	

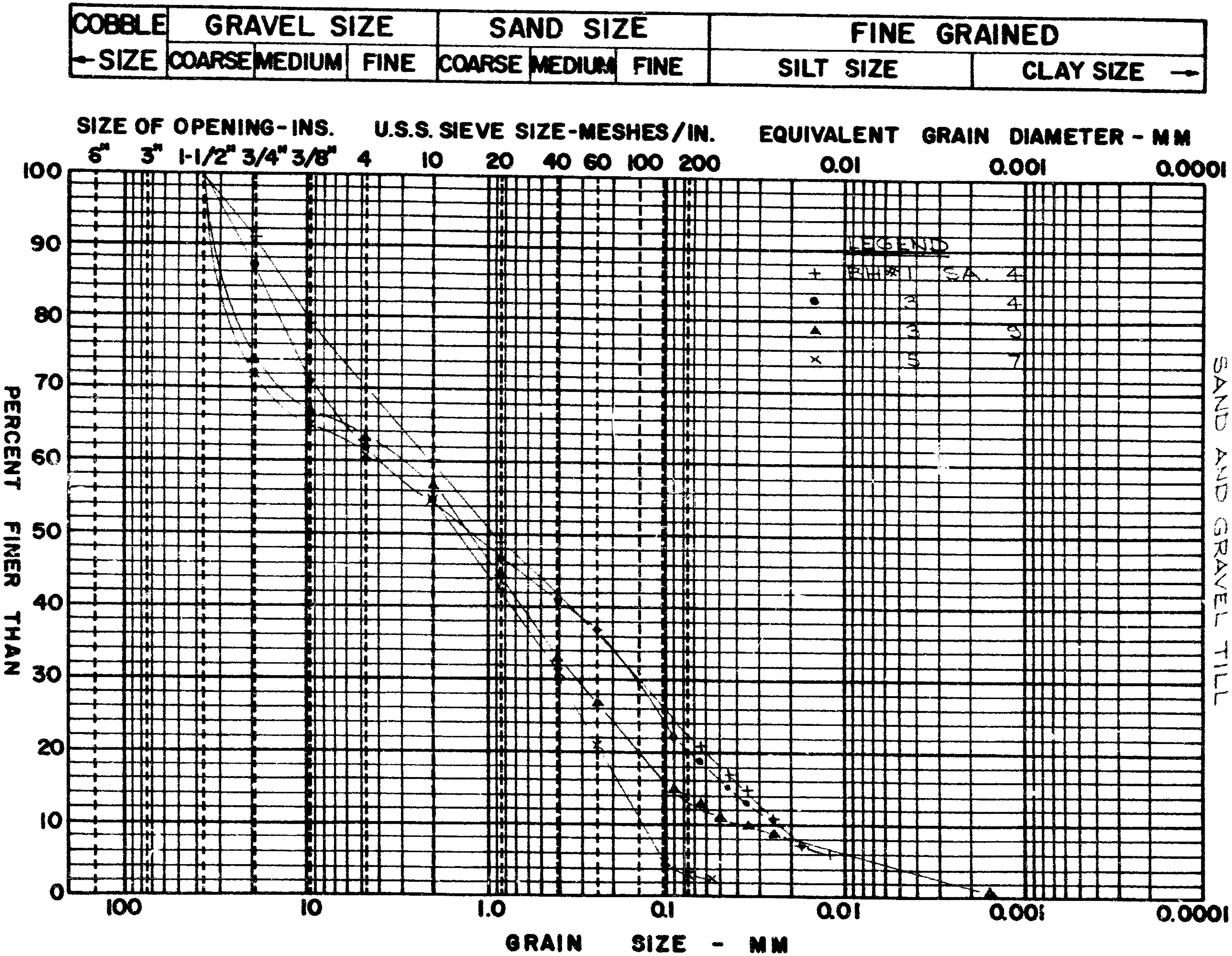
ELEVATION SCALE | WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw | DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 | OTHER TESTS | SAMPLES: CONDITION, TYPE, NUMBER, PENETRATION RESISTANCE BLOWS/FT.





APPENDIX II

FIGURES - LABORATORY TESTING



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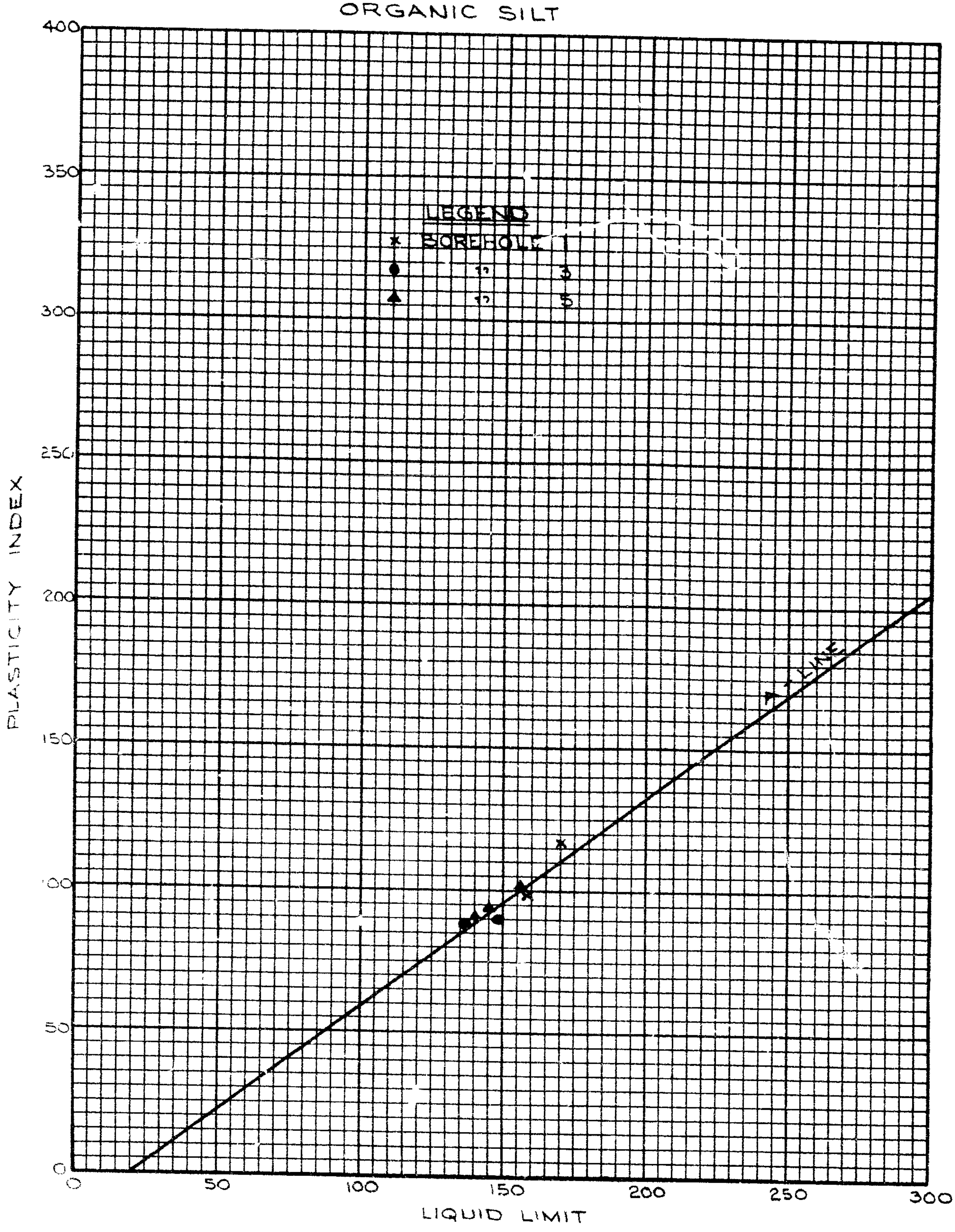
M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 1
PROJECT T7723

PLASTICITY CHART

APPENDIX II
FIGURE 2
PROJECT T7723



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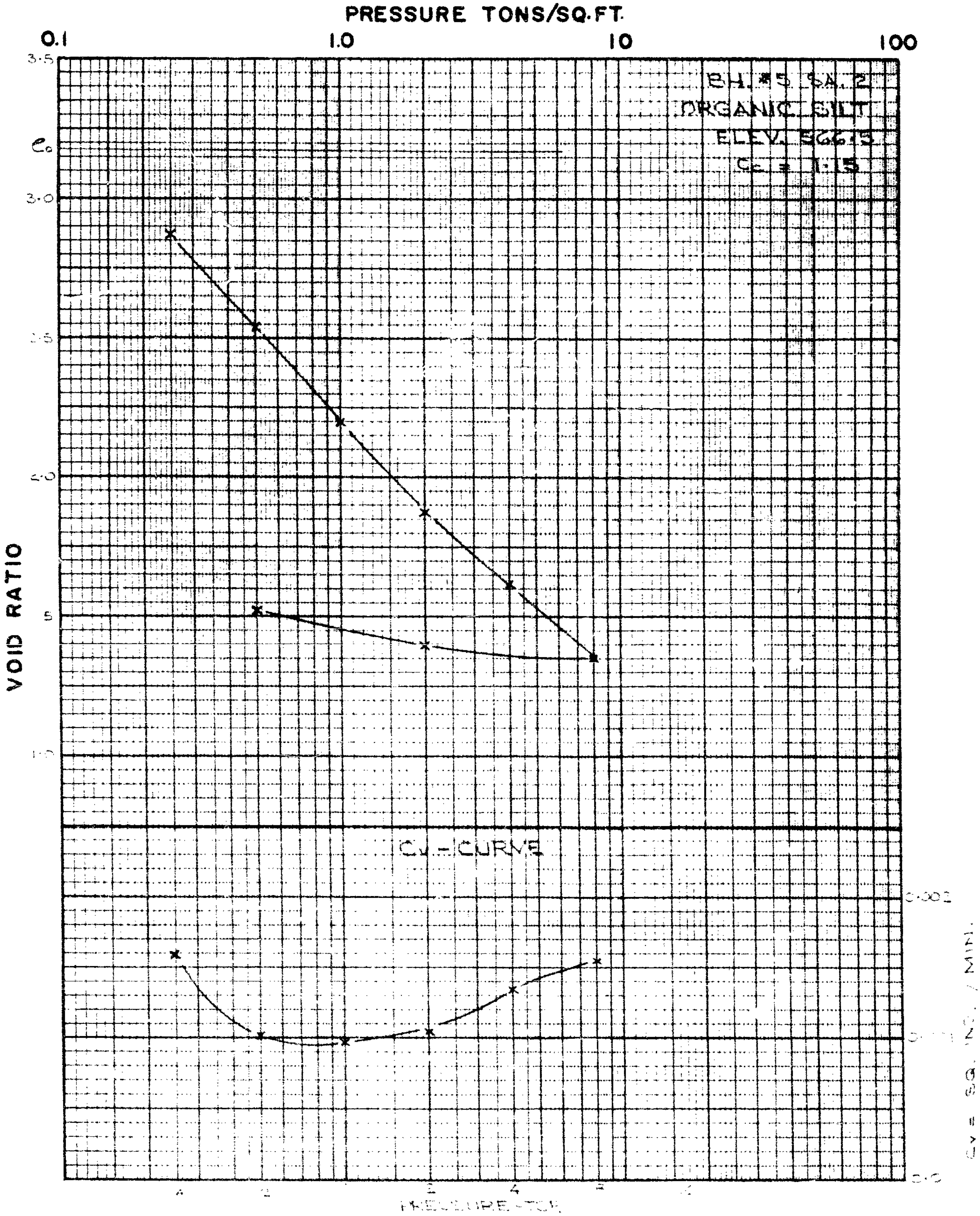
VOID RATIO-PRESSURE DIAGRAM
CONSOLIDATION TEST

APPENDIX II
FIGURE 3
PROJECT T7723



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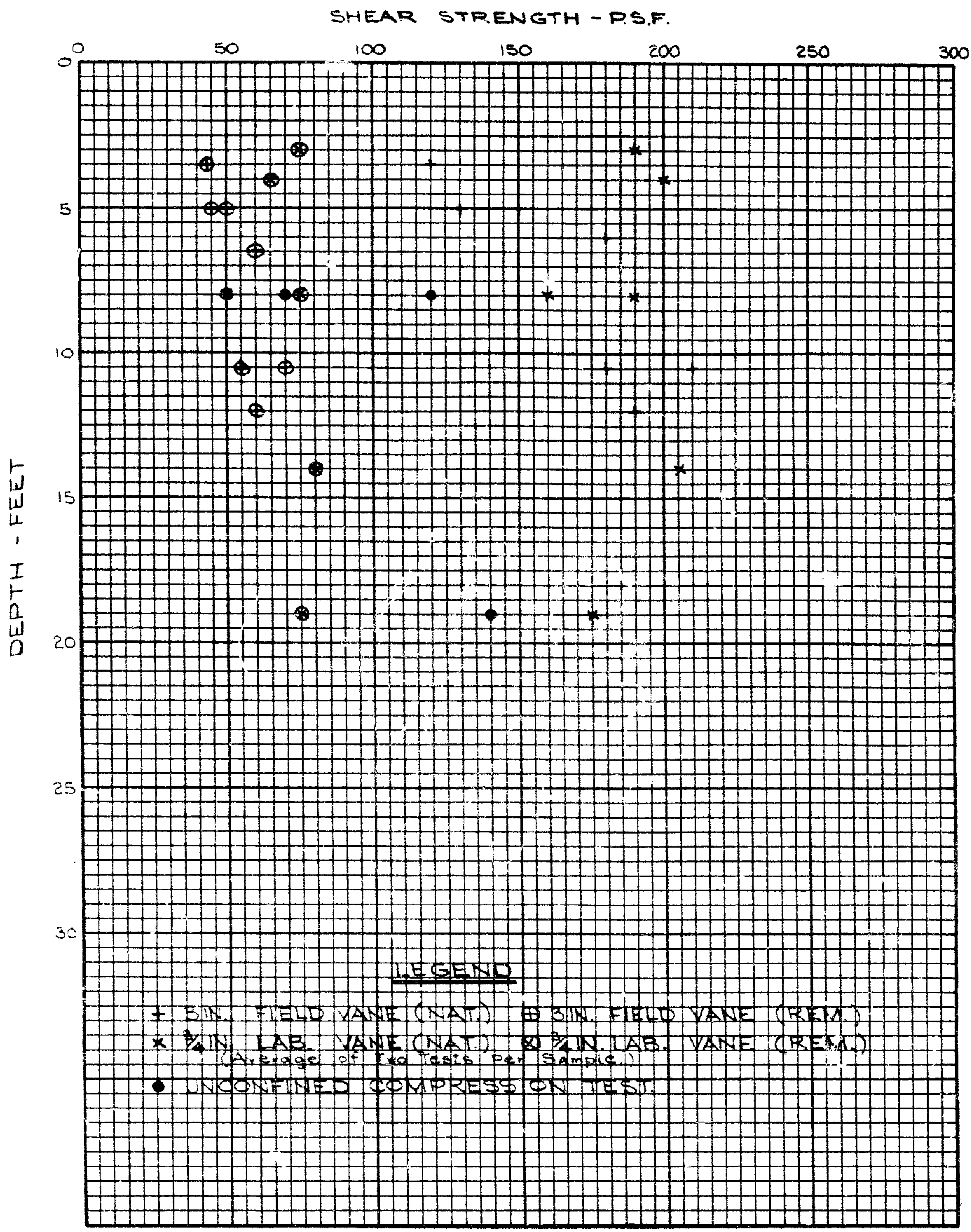
SHEAR STRENGTH vs. DEPTH

APPENDIX II
FIGURE 4
PROJECT T7723



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cc: Foundations Office (Rm. 110)

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

Materials and Testing Division

January 25, 1965

Geocon, Limited,
14 Haas Road,
Rexdale, Ontario.

Attention: Mr. [illegible]

DESBARATS RIVER

Re: W.P. 904-64, Hwy. 17, ~~[illegible]~~ River Bridge in
Desbarats, Site 385-179.
Hwy. 129, Nebskwashi River Bridge in Chapleau,
Site 46-236. -- Dist. 18, Sault Ste. Marie --

Dear Sir:

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on January 19, 1965.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Eleven copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to February 17, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the D.H.O. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide the D.H.O. with Cronaflex copies of the drawings.

cont'd. /2 ...

Charges for the work performed will be in accordance with your schedule of Rates, dated March 4, 1960, and invoice to be addressed to the attention of the undersigned.

Yours very truly,

AR.

A. Rutka,
MATERIALS & TESTING ENGINEER

RDC/McaF

cc: Messrs. G. McCombie
F. De Visser
H. McArthur
A. A. Ward
E. R. Saint
N. D. Smith (2)
Mrs. T. Tate

Foundations Office
Gen. Files (2)

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: 208 Simpson Street,
FORT WILLIAM, Ontario.

Date: January 15, 1965.

Our File Ref.

In reply to

Subject: W.P. 904-64 - Site 385-179
Desbarats ~~Walker~~ River Bridge in Desbarats
Hwy. 17 - Dist. 18

Enclosed please find one print of plan E-4535-1 for the subject structure, on which is marked the location of the footings for a new structure.

Would you please have a foundation investigation carried out, in order to determine the type of foundation required for the structure, and the stability of the fill to the east of the proposed bridge.

S McCombie

FDeV/sp

for F. DeVisser,
Regional Bridge Location Engineer.

cc. R. Fitzgibbon
N. D. Smith
S. McCombie

JOB GIVEN
TO BEGIN
JAN 19 —

GEOCON LTD

HEAD OFFICE

420 MICHEL JASMIN, DORVAL, QUEBEC

TELEPHONE 631-9827

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

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

Rexdale, Ontario,
March 9th, 1965.

Department of Highways, Ontario,
Materials and Testing Division,
Downsview, Ontario.

W.P. 904-64

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

Re: Soil Conditions and Foundations
Proposed Desbarats River Bridge
Desbarats, Ontario

Dear Sirs:

This letter accompanies our detailed report on the above investigation.

We find that the overburden at the site consists of variable thicknesses of very soft organic silt and dense sand and gravel till in this order of occurrence with depth. Bedrock is an igneous extrusive rhyolite. The soil and water conditions encountered are described in detail herein.

The organic soil is not a suitable foundation stratum and should be removed from bridge foundation locations and beneath the approach embankments. As discussed, the most suitable techniques for removal beneath the embankments would probably be the use of displacement of the silt after prior remoulding by blasting. The bridge may be carried on spread footings either on the till or the bedrock, as discussed. A discussion of soil mechanics factors pertinent to construction of bridge foundations at the proposed locations is given in the report.

We believe that this report contains the subsurface information required from this investigation. Kindly call us, however, should you have any questions relative to this report, or if we can be of further assistance otherwise.

Yours very truly,

GEOCON LTD

M. A. J. Matich

M. A. J. Matich, P. Eng.,
President.

MAJM/reb
T7723

T7723

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED DESBARATS RIVER BRIDGE

W.P. No. 904-64

DISTRICT OF ALGOMA ONTARIO

Distribution:

10 copies - Department of Highways, Ontario
Downsview, Ontario

3 copies - Geocon Ltd
Rexdale, Ontario

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INDEX

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter dated January 25th, 1965, work permit no. 904-64, to investigate and report on the soil conditions for the bridge to be located on proposed Revised Line "G" over the Desbarats River, approximately 30 feet north of an existing 16.5 foot clear span bridge located along Highway 17, at Desbarats, Ontario. The site is located on Lots 6 and 7, Township of Johnson, District of Algoma, Ontario.

The object of the investigation was to determine and interpret the subsurface conditions as they affect the design of foundations for the proposed bridge.

SUMMARIZED SOIL CONDITIONS

The Desbarats River meanders across the site with no well defined channel; instead it covers a large area with shallow depths of water. The river flows in a south-westerly direction, with the maximum depth of the river as determined from past records being about 6 feet. The surficial stratum is composed of a very soft, compressible dark brown organic silt. The thickness of this stratum in the vicinity of the bridge structures ranged from about 2 to 12 feet with an average thickness of about 6 feet. Moving in an easterly direction across the site the stratum increases in thickness until the depth of the stratum is about 30 feet. Directly underlying the organic clay stratum is a stratum of dense reddish-brown sand and gravel till with boulders; the thickness of this stratum as proven at the borehole locations, varies from 5 to 22 feet. Underlying the till stratum is sound grey to reddish-brown rhyolite bedrock; the bedrock occurred at depths varying from 11 to 35 feet below the ice level at the borehole locations. Artesian water pressure was encountered within the dense till stratum. The readings taken indicated that the artesian water pressure varied from about 1.0 to 2.0 feet above the existing surface ice level.

DISCUSSION

It is understood that a bridge crossing the Desbarats River is planned along proposed Revised Line "G". The bridge will form part of Highway No. 17 between Bruce Mines and Echo Bay Ontario and will be located approximately 30 feet north of the existing 16-1/2 foot span timber crib bridge which it will replace. The location of the structure, as proposed by The Department of Highways, Ontario, is shown on Drawing T7723-1 at the rear of this report.

As presently planned, the bridge deck will have an overall width of 40 feet and elevation 586 approximately, and be supported on abutments and two centre piers. The grade of the approach embankment is specified as 0.50 percent increasing to the east, with an associated maximum elevation of 592.

The discussion which follows deals with the soil mechanics aspects of foundation and earthworks involved in the proposed design.

Approach Embankments

(i) History of Existing Highway No. 17 Approach Embankment in the Vicinity of the Site

The existing approach embankment in the vicinity of the proposed bridge site at Desbarats, Ontario has a maximum height of about 12 feet and side slopes of 1.5 horizontal to 1 vertical (1.5:1). From visual inspection and conversations with local Highway maintenance crews, it is understood that the embankment is composed of rock fill obtained from nearby local sources. It is further understood that the existing embankments were constructed by the displacement method, assisted by blasting where the depth of organic silt was such that displacement could not be effected under the weight of fill only. The existing rockfilled timber cribs forming the bridge abutments are founded directly on the rock fill. The above construction method appears logical in view of the depth

Approach Embankments (continued)

and low shear strength of the organic silt stratum, as discussed later.

(ii) Proposed Approach Embankments

As discussed, the approach embankments will range between elevations 586 and 592. This will involve a maximum height of fill above present ground level of about 18 feet. The history of the existing embankment, together with the measured shear strengths indicate that the organic silt is not capable of supporting an embankment of the height proposed if placed in a single lift. There is precedent for construction of Highway fills on highly organic soils, but these have called for special measures such as the use of light weight materials and stage construction to avoid shear failure of the foundation material. In addition, because of the very compressible nature of such soils,the successful construction of fills on them generally requires the use of surcharge to minimize settlement of the embankment after the Highway is put into operation. Such "on surface" construction must be carefully instrumented and supervised and may require several years to complete for the heights of embankment involved in this case. In view of the limited length of embankment involved

Approach Embankments (continued)

in this instance, and the above time requirements it is assumed for purposes of this discussion that it would be preferable to complete construction of the approach embankments in a single operation. The possibility of "on surface" construction is therefore, not considered further herein.

For completion of the embankment in one operation it would be necessary to completely remove the organic silt from beneath the embankment, either be excavation in advance or by displacement during construction. In view of the depth of organic silt involved the most economical removal method, as indicated by the experience of others, is by displacement during construction. In some areas the depth of organic silt is shallow and displacement to the underlying till could probably be achieved under the self-weight of the fill alone, if placed by end-dumping to full height. In areas where the depth of organic silt is appreciable, that is, of the order of 30 feet in the easterly portion of the site, it is possible that complete sinkage of the rock fill to the base of the underlying till may not be achieved under the weight of the fill alone. In these areas some floating of the fill may occur with resultant irregular future settlements of the roadway surface due to consolidation and progressive dis-

Approach Embankments (continued)

placement of the trapped soil.

It is recommended therefore that steps be taken to ensure that complete displacement takes place during construction. A number of methods have been used in the past to effect displacement of soft soils, including surcharging of the embankment, jetting to weaken the material to be displaced, and blasting immediately in advance of fill placement. Previous experience on the Rainy Lake Causeway project indicated that to achieve positive displacement with depths of greater than 8 feet of varved clay, it was necessary to lower the shear strength of the clay to at least 100 pounds per square foot. It is considered therefore, that by remoulding the silt to a shear strength of 100 pounds per square foot or less prior to fill placement, satisfactory displacement would be achieved with end-dumping of fill to full height. The most positive method of reducing the shear strength to the required value necessary for displacement, is in this instance, considered to be the use of blasting techniques.

Approach Embankments (continued)

Experience on the Rainy Lake Causeway project indicated that maximum remoulding was effected by the combined effect of a group of charges placed near the base of the stratum concerned. The results also indicated that a powder factor of about one pound of dynamite for each cubic yard of clay was necessary to effect the desired lowering of the shear strength below 100 pounds per square foot. Further, by using 50 percent forcite ditching dynamite it was possible to effect explosion by propogation. Using discharge by a single "primer" it was found that 50 pound pockets of dynamite would explode by propogation if spaced at 18 feet intervals or less. Successful displacement was achieved using 50 pound pockets spaced at 10 foot intervals, at a depth equal to two-thirds of the clay thickness with each blast containing at least 3000 pounds of dynamite.

Obviously the experience of Rainy Lake cannot be applied directly to this site because of the difference in soil conditions, and water depths involved. It is probable that the organic silt will require a lower powder factor than the clay at Rainy Lake because of its already very low in-situ shear strength. The actual powder factor required, however,

Approach Embankments (continued)

cannot be reliably established theoretically and it is recommended that the most suitable blasting pattern be established experimentally either before or during construction. It is believed however, that the general approach with regard to pattern of explosive charges as used at Rainy Lake would still apply, and this could form the basis of initial trials in the field.

It is recommended that the embankment be constructed of hard durable rock fill which is known to be available in the vicinity. Such rhyolite type rock fill was used in the construction of the Canadian Pacific Railway Embankment, the existing Highway 17 embankment, and the rock fill timber crib bridge structure all located in the Desbarats area. As indicated by the existing embankment, the side slopes of fill end-dumped after remoulding of the silt would be approximately 1.5 horizontal to 1 vertical.

It would be desirable to construct the embankment in advance of the bridge structure because of the factors involved with the displacement method. It may be desirable also to provide a short section of sand and gravel in the

Approach Embankments (continued)

embankment at the juncture with the bridge.

Foundations

The soil conditions are suitable for the use of spread footings for the proposed abutments and piers founded, on or within, the granular till stratum or on bedrock. In some instances mainly in the northern extremity of the foundation locations, bedrock is encountered under a thin cover of till; at these locations the spread footings would be carried on the bedrock.

Considering the size of footing that would probably be involved, it is recommended that an allowable bearing value of 3.0 tons per square foot be used for the till stratum. For spread footings carried entirely on bedrock a maximum allowable bearing value of 20 tons per square foot could be used for design. For abutments or piers founded partly on bedrock and partly on the till the maximum total settlements will also be the differential settlements. Under the above design values the total settlement of the abutments founded as described should not be more than one half inch. Settlement will, for practical purposes, take place concurrently with application of load and will therefore be largely completed at the end of construction.

Foundations (continued)

Consideration might be given to founding the bridge on piles end-bearing in the till or on bedrock, and a number of piles would be suitable for this purpose. Because of the shallow depth to bedrock, however, such as at the location of test pit 4, the use of piles would still involve carrying at least part of one or more of the foundations on bedrock directly. The comparatively short depth to bearing stratum, elsewhere, may preclude the use of piles from a practical viewpoint.

In view of the fact that bedrock outcrops a short distance from the proposed abutment locations, construction would be simplified if alignment and bridge design considerations permitted shifting the bridge location a short distance northwards and using a slightly longer and single span structure. In such a case foundations could be more conveniently constructed in the dry directly on bedrock.

Footings, or pile caps if involved, subject to frost action should be provided with a minimum of 5 feet of protective earth cover.

In design of abutments founded on spread footings in the till, it is recommended that a coefficient of lateral earth pres-

Foundations (continued)

sure of 0.4 be used for the backfill which should be select well compacted clean granular material. For abutment footings founded on bedrock, a value of 0.5 is recommended.

The abutments and piers should be designed for a factor of safety of at least 1.5 against sliding based on a coefficient of friction of concrete to granular till of 0.45. For footings on bedrock a coefficient of 0.35 should be used. In the latter case, the resistance to lateral movement could, if desired, be increased by dowelling into bedrock.

It is recommended that consideration be given to provision of rip-rap or other suitable measures for protection of foundations carried on the till against possible undermining by scour. The actual requirements will depend on the bridge design selected, on the maximum fluctuation in stream flow, and on the constriction to the present stream valley caused by the approach embankments. They are thus dependent on design and hydraulic considerations beyond the scope of this report.

Construction of spread foundations will involve excavation of as much as 12 feet of organic material at a depth of as much as 14 feet below present water level. A higher water head differential

Foundations (continued)

may have to be contended with if construction was carried out during high stages of the river. Construction could probably be best effected in the dry by the use of braced steel sheet pile cofferdams where the sheeting is driven a suitable distance into the till, or to bedrock as discussed later. The distance which sheeting should penetrate into the till, to provide safety against piping by upward action of seeping water, would depend on the maximum hydrostatic head differential allowing for artesian effects, and on the width of the cofferdam. As a preliminary guide, sheeting should be carried below base of excavation a depth at least equal to the maximum head difference after dewatering. In effect, therefore, penetration to bedrock throughout would probably be required. However, because of the very dense nature of the till and its boulder content, it is unlikely that an intact continuous cut-off to bedrock could be achieved by driving sheeting through the till without special provisions to facilitate penetration. In view of this, and the presence of artesian pressures in the till, consideration could be given to driving the sheeting a sufficient distance into the till to obtain the necessary toe-hold and to effect dewatering by a system of filter equipped screened pumped wells penetrating the till. The installation of such wells, or wellpoints, would have to contend with boulders in the till and its dense nature otherwise.

1. The site is covered by a stratum of organic silt varying in encountered thickness from 2 to 30 feet. Underlying this stratum is a dense sand and gravel till with boulders stratum which in turn is underlain by a sound rhyolite bedrock.

2. The site is partially covered by the Desbarats River which flows in a south-west direction. At the time of the investigation the maximum depth of the river was 6 feet. Artesian pressure was encountered within the till stratum as discussed in this report.

3. The approach embankments may be constructed of locally available rock fill. This rock fill should be founded on the till stratum, with the organic silt overlying this till being positively removed in advance by one of several techniques outlined in this report.

4. The bridge structure as proposed, may be carried on spread footings, founded on the till or directly on bedrock at allowable bearing values as discussed in the report. The use of end-bearing piles might also be considered, as discussed.

5. Soil mechanics considerations pertinent to construction of abutments and piers are discussed in the report.

PERSONNEL

The field work was carried out under the supervision of Mr. B. T. Darch. This report was written by Mr. Darch, checked by Mr. D. B. Oates, P. Eng. and reviewed by Mr. M. A. J. Matich, P. Eng.

B. T. Darch

BTD/reb

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REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SURFACE ICE AND WATER CONDITIONS

SOIL AND WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

PROCEDURE

The field work was carried out between February 1st, 1965 and February 9th, 1965. A total of 3 boreholes, each with an accompanying dynamic uncased cone penetration test, and 8 additional dynamic uncased cone penetration tests were put down in BX size. For the work a Penndrill using power auger techniques was used.

The soil strata were sampled at intervals not exceeding 5 feet. Two and three inch Osterberg thin walled tube samples were taken in the organic stratum of the overburden; the three inch tube samples were taken at the top of the stratum where the boreholes stayed open without the need of casing. Three inch diameter in-situ vane shear strength tests were carried out in the organic stratum in two of the boreholes. The granular strata were sampled using a 2 inch split spoon sampler adapted where necessary with a foot valve to aid in the recovery of the material. The bedrock was proven in boreholes 1 and 3 for about 10 feet, by rock core drilling in BXL size. Piezometers were installed in boreholes 3 and 5.

Detailed logs of the boreholes and uncased dynamic cone penetration tests are presented on the Office Reports on Soil Exploration in this Appendix. The locations of the boreholes and dynamic penetration tests, together with the inferred soil stratigraphy are shown on Drawing T7723-1, at the rear of this report.

The laboratory testing of selected soil samples was carried out in the Toronto Soil Mechanics Laboratory of Geocon Ltd. The results are plotted on the Office Reports on Soil Exploration in Appendix I and the Figures in Appendix II. The soil samples remaining after testing will be stored until February 27th, 1965 at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic datum. The bench mark referred to is the Department of Highways, Ontario Geodetic Bench Mark which is a Cut Cross on the concrete abutment of the Bridge, 54 feet left of Station 289+37 along Proposed Revision Line "G". The Geodetic elevation of this bench mark is 585.05. The location of the bench mark is shown on the Department of Highways, Ontatio (D. H. O.) Drawing E-45345-1.

SITE AND GEOLOGY

The proposed D. H. O. bridge site is to be located on Revised Line "G" over the Desbarats River, approximately 30 feet north of a 16. 5 feet clear span bridge located along existing Highway 17 at Desbarats, Ontario. The site is located on Lot 6 and Lot 7, in the Township of Johnson, District of Algoma, Ontario. The area is low lying in a natural valley with higher ground surrounding it on all sides. To the west of the bridge site the ground rises sharply with bedrock outcrops in evidence. To the east, north and south the terrain is flat lying supporting light marsh vegetation of grass and stunted brush. Directly to the south of Revised Line "G" is the approach embankment of existing Highway No. 17 having a maximum height of about 15 feet.

The Desbarats River meanders across the site, flowing from the north-east to the south-west. The River has no permanent channel, it inundates a large area to the north-east with shallow depths of water. From past river surveys on the site carried out by the D.H.O., it is estimated that the maximum depth of the Desbarats River is about 6 feet.

From available geological information and an inspection of the area, it is known that the overburden is an organic soil deposited during recent times, underlain by a sandy till of glacial origin, which in turn overlies rhyolite bedrock of the early Proterozoic Era of the Precambrian Period. The rhyolite which is grey to reddish-brown in colour, and high in silica content, is a well cemented variety.

SURFACE ICE AND WATER CONDITIONS

The proposed locations of the abutments and piers are within the river bed of the Desbarats River. At the time of the investigation the ice on the site varied from 1 to 2 feet in thickness, with the thicker sections occuring in the river channel and the thinner sections on the banks of the river. During this period the total depth of ice and water encountered at the boreholes and at the locations of the dynamic penetration tests varied from 2 to 4 feet across the site. From past surveys it is estimated that the maximum depth of the Desbarats River would be about 6 feet, as already mentioned.

SOIL AND WATER CONDITIONS

The principal soil strata encountered in boreholes 1, 3 and 5 were as follows:

Very Soft Dark Brown Highly Organic Silt

The surficial overburden stratum across the site is a dark brown highly organic silt containing wood fiber, roots and decayed matter; it is inferred that this material covers the entire site for the proposed bridge and associated approach embankments. This material is very plastic, peat like, and highly compressible. The surface elevation of the stratum varies from 573 to 576 across the site with a corresponding variation in thickness from 5.5 feet at borehole 1 to 23 feet at borehole 5; however, from the results of the dynamic uncased cone penetration tests the thickness is inferred to be as much as 30 feet. Within the area bounded by the abutments the maximum thickness of this stratum encountered was about 12 feet, however, moving in an easterly direction the stratum thickness probably increases to about 30 feet. The top 2 feet of the stratum is highly organic with a large percentage of wood fiber and can be considered to be a peat.

Seven natural moisture content determinations were made on samples from the stratum; the values ranged from 97 to 152 percent being typically about 130 percent. The corresponding wet unit weights of three samples tested were 76, 77 and 81 pounds per cubic foot. These results are shown on the individual Office Reports on Soil Exploration in this Appendix.

Seven Atterberg limit determinations were carried out on typical samples from the stratum. The tests indicated that the liquid limit ranged from 137 to 170 , being typically about 150, and plastic limits ranging from 49 to 59 with an average of about 53. The Unified Soil Classification System classifies the material as organic silt of high plasticity. The Atterberg limit results are shown on the Plasticity Chart, Figure 2 of Appendix II and also on the Office Reports on Soil Exploration in this Appendix.

Three unconfined compression strength tests were performed on typical samples from the stratum and gave shear strengths (taken as one half of the compressive strength) of 70, 120 and 140 pounds per square foot, respectively. It is believed that unavoidable disturbance during sampling explains the low values obtained from laboratory unconfined strength testing as compared to the results of the field and laboratory vane tests given below. Seven 3-inch in-situ vane shear strength tests were carried out in the stratum t boreholes 3 and 5. The results of these tests indicated that the undisturbed shear strength ranged from 120 to 210 pounds per square foot with an average of about 150 pounds per square foot. Remoulded vane shear strength tests were carried out in conjunction with the undisturbed tests and gave remoulded shear strength values ranging from 40 to 70 pounds per square foot. Three-quarter inch laboratory vane shear strength tests were carried out on six Osterberg tube samples; two determinations were made on each tube tested. These tests indicated that the undisturbed shear strength of the material varied from 160 to 200 pounds per square foot, and average about 170 pounds per square foot. Remoulded vane shear strength tests were carried out in conjunction with the undisturbed tests and gave remoulded shear strength values ranging from 20 to 80 pounds per square foot with an average of about 60 pounds per square foot.

Based on the above results, the consistency of the organic silt is estimated to be very soft. The strength test results are plotted on Figure 4 of Appendix II and on the Office Reports on Soil Exploration in this Appendix. The sensitivity of the organic silt, established by means of the remoulded characteristics of the field and laboratory vane tests and visual and tactile examination is typically of the order of 3 to 4 indicating that the deposit is moderately sensitive to disturbance.

A consolidation test was performed on a typical sample from the stratum and the results are shown on the "Pressure vs. Void Ratio" curve , Figure 3 in Appendix II. The results of the tests indicate that the deposit is normally consolidated. The compression index "Cc" obtained from the curve is 1.15.

The uncased dynamic cone penetration tests carried out met little resistance while passing through the stratum, the "A" rods penetrating the stratum under the weight of the hammer.

Dense to Very Dense Reddish-Brown Sand and Gravel Till with Boulders

Directly underlying the organic clay is a stratum of reddish-brown sand and gravel till with a high boulder content. The surface elevation of the stratum across the site varies from about

550 to 570. The thickness of the stratum was proven to be 4.5 feet at borehole 1 and 20.5 feet at borehole 3; these two boreholes were the only holes in which this stratum was penetrated completely. The stratum has a matrix of sand with some silt, binding gravel sizes and boulders; the granular component of the stratum is sub-angular in shape. Boulders up to 8 inches in size were encountered while penetrating the stratum, although it is considered possible that larger boulders are present at other than borehole locations. The boulders were random in mineralogical composition being very often granitic. A sand and gravel layer was encountered within this stratum at boreholes 3 and 5, and from the results of the dynamic cone penetration tests the stratum is inferred to exist at other locations. The layer was found at elevations varying from 540 to 550 across the site and was generally about 3 feet thick.

Three mechanical analysis tests were carried out on typical samples from the stratum and the results are shown on Figure 1 of Appendix II. The grain size curves indicated that the material ranges from 12 to 19 percent silt sizes, 38 to 46 percent sand sizes, and 40 to 45 percent gravel sizes. One mechanical analysis was carried out on a sample from the sand and gravel layer; the grain size curve indicated that the material consisted of 2 percent silt sizes, 53 percent sand sizes and 45 percent gravel sizes. From observing the wash water it is believed that parts of the layer are higher in sand content than indicated from this test. It is probable that the gravel component of the tested sample is exaggerated by cave from higher levels in the open borehole.

Nine standard penetration tests were carried out in the stratum and gave "N" values ranging from 51 to greater than 100 blows per foot with an average of about 80 blows per foot. It should be noted that some of these "N" values may have been increased by the high gravel content of the stratum. One additional standard penetration test was carried out in the sand layer described above and gave an "N" value of 21 blows per foot. This result was confirmed by the drop in resistance of the adjacent dynamic penetration test which was put down in advance of the borehole. The uncased dynamic cone penetration tests met practical refusal within this stratum. From these results it is estimated that the relative density of the stratum varies from dense to very dense and is generally very dense. This is confirmed by the fact that normal augering could not penetrate any significant distance into the stratum.

For design purposes, the following parameters may be used, where appropriate:

Wet unit weight	140 pounds per cubic foot
Submerged unit weight	78 pounds per cubic foot
Angle of shearing resistance,	40 degrees.

Sound Grey to Reddish-Brown Rhyolite Bedrock

Directly underlying the sandy till stratum in boreholes 1 and 3 is grey to reddish-brown rhyolite bedrock; bedrock was not proven in borehole 5. The bedrock encountered in both boreholes was proven by diamond core drilling in BXL size for a depth of about 10 feet. The surface elevation of the bedrock was 566 in borehole 1 and 542 in borehole 3. The core recovered was high in silica content, and sound, as evidenced by the high core recovery, and the appearance of the core.

The bedrock is well cemented and massive with an estimated hardness of about 6 on the Moh's hardness scale. The dominant component minerals are quartz and feldspar. The fracture patterns are uneven and occur along prescribed planes of weakness developed during formation of the bedrock.

Water Conditions

The water level was observed during the investigation in piezometers installed in boreholes 3 and 5 and the open hole of borehole 1. Artesian water pressures were encountered within the till stratum at boreholes 3 and 5. The artesian water levels at boreholes 3 and 5, measured on February 9th, 1965, were 10 inches and 22 inches above ice level, respectively. The water level observed at borehole 1 corresponded with the river water level at the time of the investigation.

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:

Consistency	U-Strength Tons/sq. ft.
Very soft	0.03 to 0.25
Soft	0.25 to 0.5
Firm	0.5 to 1.0
Stiff	1.0 to 2.0
Very stiff	2.0 to 4.0
Hard	over 4.0

Relative Density	Standard Penetration Resistance. Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

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 T 7723 BORING # 1 And PT. 2 DATUM GEODETIC CASING BX.
BORING DATE FEB 1-3/65 REPORT DATE FEB. 22, 1964 COMPILED BY A.E.L. CHECKED BY B.T.D.
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
- 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 WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc. - TRIAXIAL CONSOLIDATED UNDRAINED
- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE: ELEVN. / DEPTH | WATER CONDITIONS | DESCRIPTION | STRAT. PLOT | ELEVATION SCALE

SHEAR STRENGTH - PSF: LAB. VANE × (NAT.) ⊗ (REM.) V_L — 50, 100, 150, 200
WATER CONTENT W%: ⊙ NAT. ⊡ Lw △ Pw — 20, 40, 60, 80
DYNAMIC PENETRATION TEST BLOWS PER FOOT — 20, 40, 60, 80

OTHER TESTS | SAMPLES: CONDITION, TYPE, NUMBER, PENETRATION RESISTANCE BLOWS/FT.

1 (BOREHOLE)

ELEVN. / DEPTH	DESCRIPTION
576.9	RIVER LEVEL
576.0	0.9' ICE
576.0 / 0.0	RIVER BOTTOM
570.4 / 5.6	VERY SOFT DARK BROWN ORGANIC SILT
565.9 / 10.1	DENSE TO VERY DENSE REDDISH BROWN SAND AND GRAVEL TILL WITH BOULDERS
556.9 / 19.1	SOUND REDDISH BROWN RHYOLITE BEDROCK
	END OF HOLE

Chart annotations: PUSH; 60 BLOWS FOR LAST 6 INS; 100 BLOWS - NO ADVANCE; END OF PENETRATION TEST AT EL. 567.5

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	DISTURBED	2" DO	1	1
V	FAIR	3" TO	2	PUSH
	DISTURBED	2" DO	3	>100
M	DISTURBED	" DO	4	>100
R.C. RECOV. 100%	GOOD	BXL. RC	5	–
98%	GOOD	"	6	–

2 (PENETRATION TEST)

ELEVN. / DEPTH	DESCRIPTION
576.7	RIVER LEVEL
	0.9' ICE
573.4 / 0.0	RIVER BOTTOM
570.7 / 2.7	PROBABLY VERY SOFT ORGANIC SILT
564.7 / 8.7	PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL
	END OF PEN TEST

Chart annotations: 75 BLOWS - NO ADVANCE; END OF PENETRATION TEST AT EL. 564.7

Elevation scale: 580, 570, 560, 550 (borehole 1); 580, 570, 560 (penetration test 2)

APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 BORING # 3 And TP 4 DATUM GEODETIC CASING BX.
BORING DATE FEB. 4-5/65 REPORT DATE FEB. 22, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.
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

- 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 WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED UNDRAINED
- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT P.C.F.
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SHEAR STRENGTH - PSF ▲ UNCONFINED COMP.(U) LAB. VANE X (NAT.) ⊗ (REM.) V_L, FIELD VANE + (NAT.) ⊕ (REM.) V_F — 50, 100, 150, 200
WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw — 20, 40, 60, 80
DYNAMIC PENETRATION TEST BLOWS PER FOOT — 20, 40, 60, 80

3 (BOREHOLE)

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	ELEVATION SCALE	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
576.8	▼	RIVER LEVEL	580					
574.9 / 0.0		RIVER BOTTOM	ICE					
	BENTONITE PLUG	VERY SOFT DARK BROWN ORGANIC SILT	570	V_L		2" DO	1	PUSH
				V_F		3" TO	2	"
				γ=77 V_L		TO	3	"
562.5 / 14.3	SAND			M		2" DO	4	>100
	PIEZOMETER	DENSE TO VERY DENSE REDDISH BROWN SAND & GRAVEL TILL WITH BOULDERS	560	RC. RECOV. 31%		BXL RC	5	–
				M		2" DO	6	>100
			550			DO	7	>100
						DF	8	>100
541.8 / 33.1						DF	9	>100
	WL FEB.	SOUND REDDISH BROWN RHYOLITE BEDROCK	540	88%		BXL RC	10	–
				100%		"	11	–
532.1 / 42.8		END OF HOLE	530					

WT. 3" HAMMER

31, 45, 28, 37, 194

34 BLOWS - NO ADVANCE

END OF PENETRATION TEST AT EL. 562.5

4 (TEST PIT)

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	ELEVATION SCALE
576.6	▼	RIVER LEVEL	580
575.8 / 0.0		RIVER BOTTOM	ICE
		REDDISH BROWN RHYOLITE BEDROCK	570

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 77 23 BORING # 5 And PT. 6 DATUM GEODETIC CASING BX.
BORING DATE FEB. 6-8/65 REPORT DATE FEB. 23, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.
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

- 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 WALLED OPEN
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- S - TRIAXIAL DRAINED
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- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

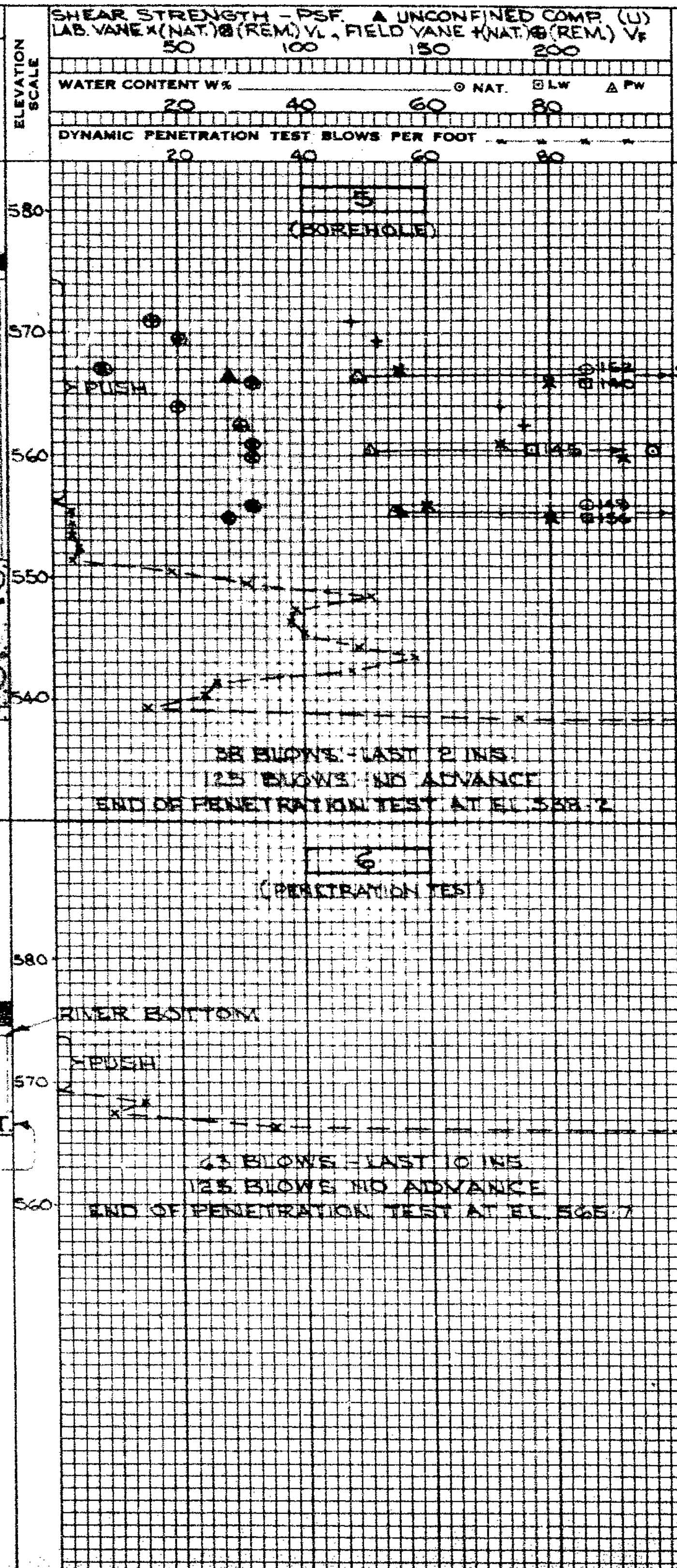
SOIL PROFILE

Boring 5 (Borehole)

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
576.4		RIVER LEVEL
574.9		1.5 ICE
0.0	SAND	RIVER BOTTOM
	BENTONITE PLUG; W.L. @ EL. 578.2 FEB. 8, 1965	VERY SOFT DARK BROWN ORGANIC SILT
551.4 / 22.9	PIEZOMETER	DENSE TO VERY DENSE REDDISH BROWN SAND & GRAVEL TILL WITH BOULDERS
539.4 / 34.5 / 36.1		END OF HOLE / END OF PEN. TEST

Point 6 (Penetration Test)

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
576.6		RIVER LEVEL
573.8 / 0.0		2.0 ICE
567.6		PROBABLY VERY SOFT ORGANIC SILT
565.7 / 8.1		END OF PEN. TEST PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
V		2" TO " DF	1	PUSH
C, γ=76, V		3" TO	2	"
V		2" TO	3	"
γ=81, V		" TO	4	"
	DISTURBED	" DO	5	62
	DISTURBED	" DO	6	73
M	DISTURBED	" DO	7	21

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 — PEN. TEST [redacted] # 7, 8 And 9 — DATUM GEODETIC — CASING —

BORING DATE FEB 8-9/65 — REPORT DATE MAR 1, 1965 — COMPILED BY A.E.L. — CHECKED BY B.T.D.

[redacted] 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

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 WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS

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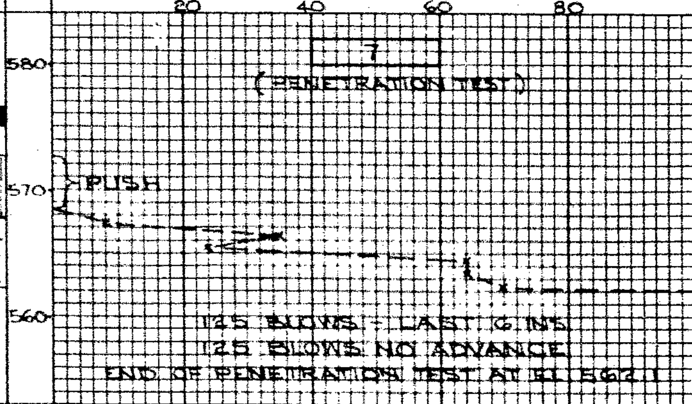
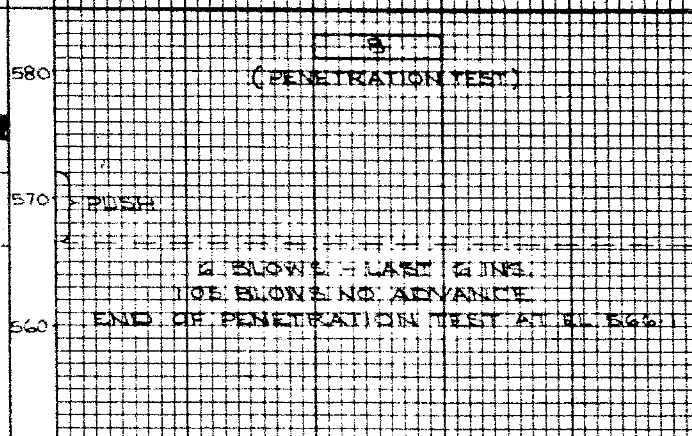
S - TRIAXIAL DRAINED
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

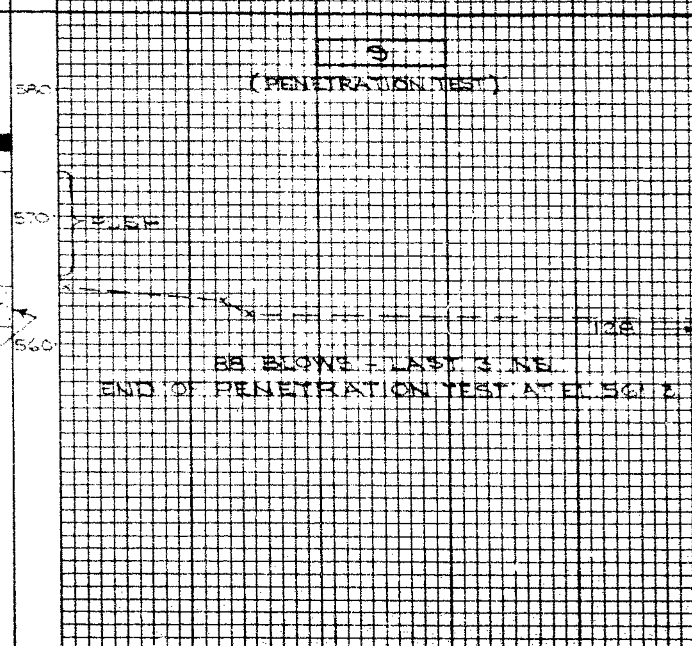
SOIL PROFILE				ELEVATION SCALE	WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw / DYNAMIC PENETRATION TEST BLOWS PER FOOT (20, 40, 60, 80)	OTHER TESTS	SAMPLES			
ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT				CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
576.6	▼	RIVER LEVEL								
		1.5 ICE								
572.7 / 0.0		RIVER BOTTOM								
		PROBABLY VERY SOFT ORGANIC SILT								
567.6 / 5.1		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL								
562.1 / 10.6		END OF PEN. TEST								



ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
576.6	▼	RIVER LEVEL
		1.9 ICE
572.1 / 0.0		RIVER BOTTOM
		PROBABLY VERY SOFT ORGANIC SILT
566.1 / 6.0		END OF PEN. TEST



ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
576.6	▼	RIVER LEVEL
		1.3 ICE
573.7 / 0.0		RIVER BOTTOM
		PROBABLY VERY SOFT ORGANIC SILT
564.6 / 9.1		
561.3 / 12.4		END OF PEN. TEST
		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7723 PEN. TEST [redacted] # 10 And 11 DATUM GEODETIC CASING —
BORING DATE FEB 9/65 REPORT DATE MAR 1, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.
[redacted] HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN.-LL. ENERGY)

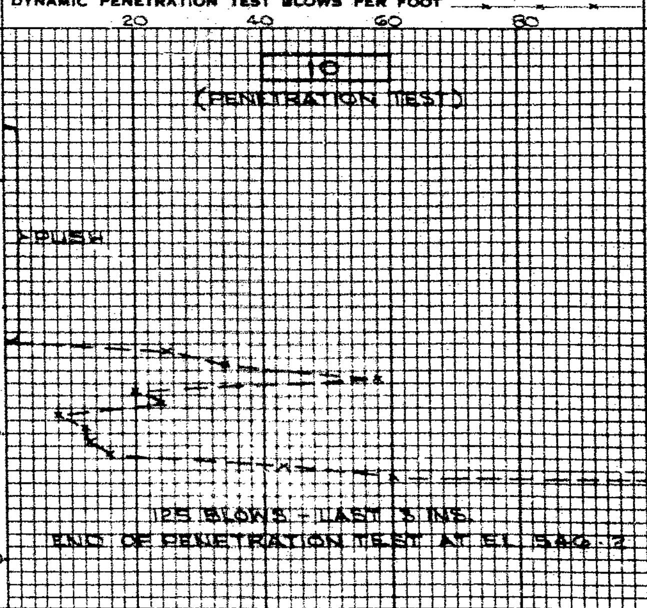
SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

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 WALLED OPEN
- R.C. - ROCK CORE

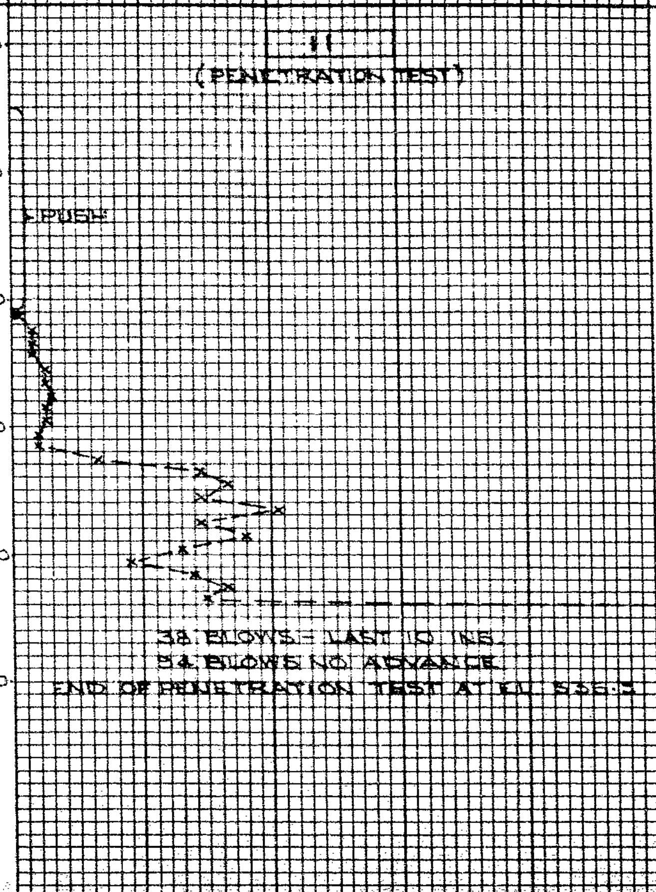
ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- QC. - TRIAXIAL CONSOLIDATED UNDRAINED
- Q - TRIAXIAL UNDRAINED
- S - TRIAXIAL DRAINED
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE | ELEVATION SCALE | WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw / DYNAMIC PENETRATION TEST BLOWS PER FOOT | OTHER TESTS | SAMPLES (CONDITION, TYPE, NUMBER, PENETRATION RESISTANCE BLOWS/FT.)

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT
576.5	▼	RIVER LEVEL	
574.2		1.3' ICE	
0.0		RIVER BOTTOM	
		PROBABLY VERY SOFT ORGANIC SILT	
556.5 / 17.7		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL	
546.2 / 28.0		END OF PEN. TEST	



ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT
576.8	▼	RIVER LEVEL	
575.0		1.8' ICE	
0.0		RIVER BOTTOM	
		PROBABLY VERY SOFT ORGANIC SILT	
546.8 / 28.2		PROBABLY DENSE TO VERY DENSE SAND & GRAVEL TILL	
535.9 / 39.1		END OF PEN. TEST	



APPENDIX II

FIGURES - LABORATORY TESTING

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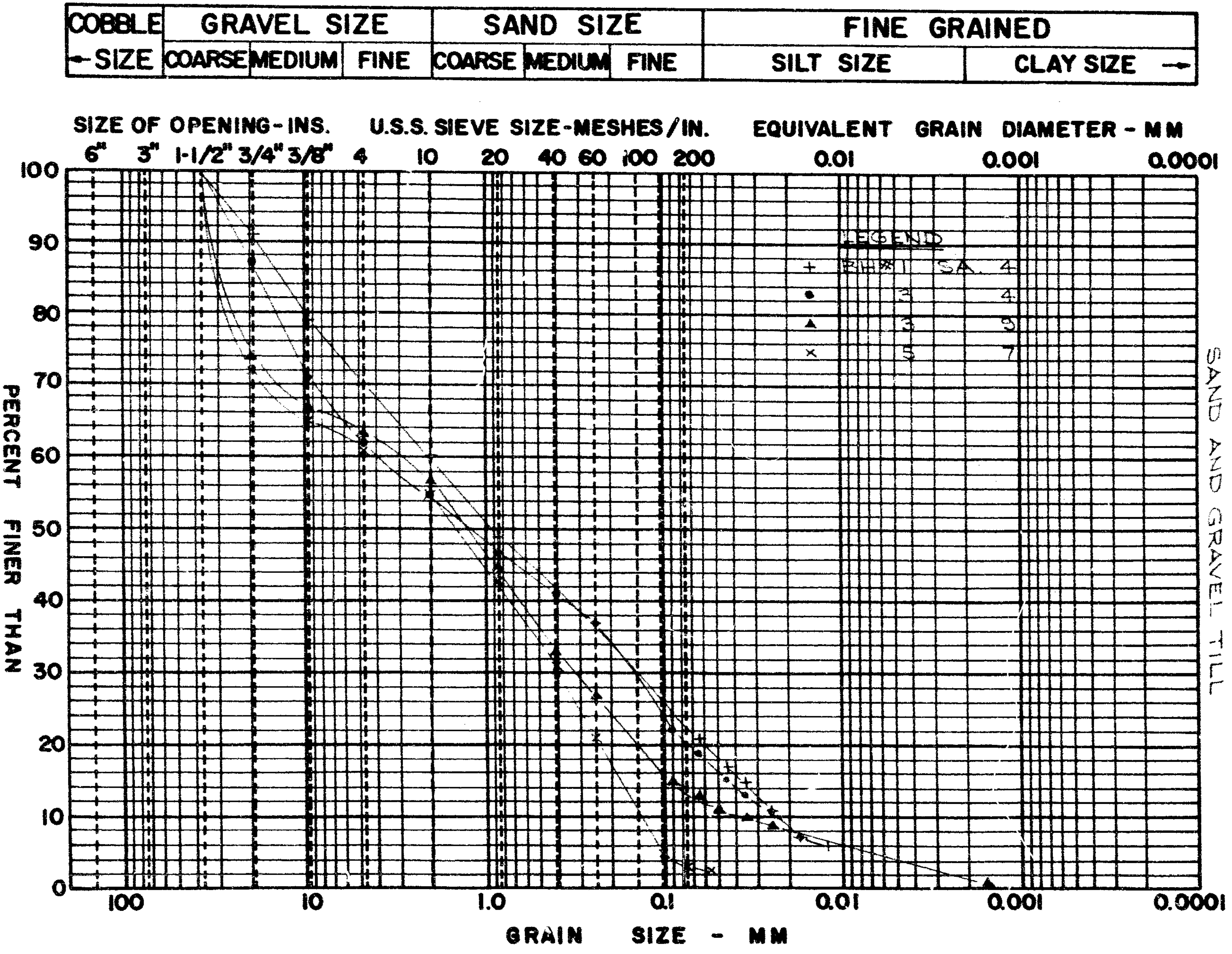


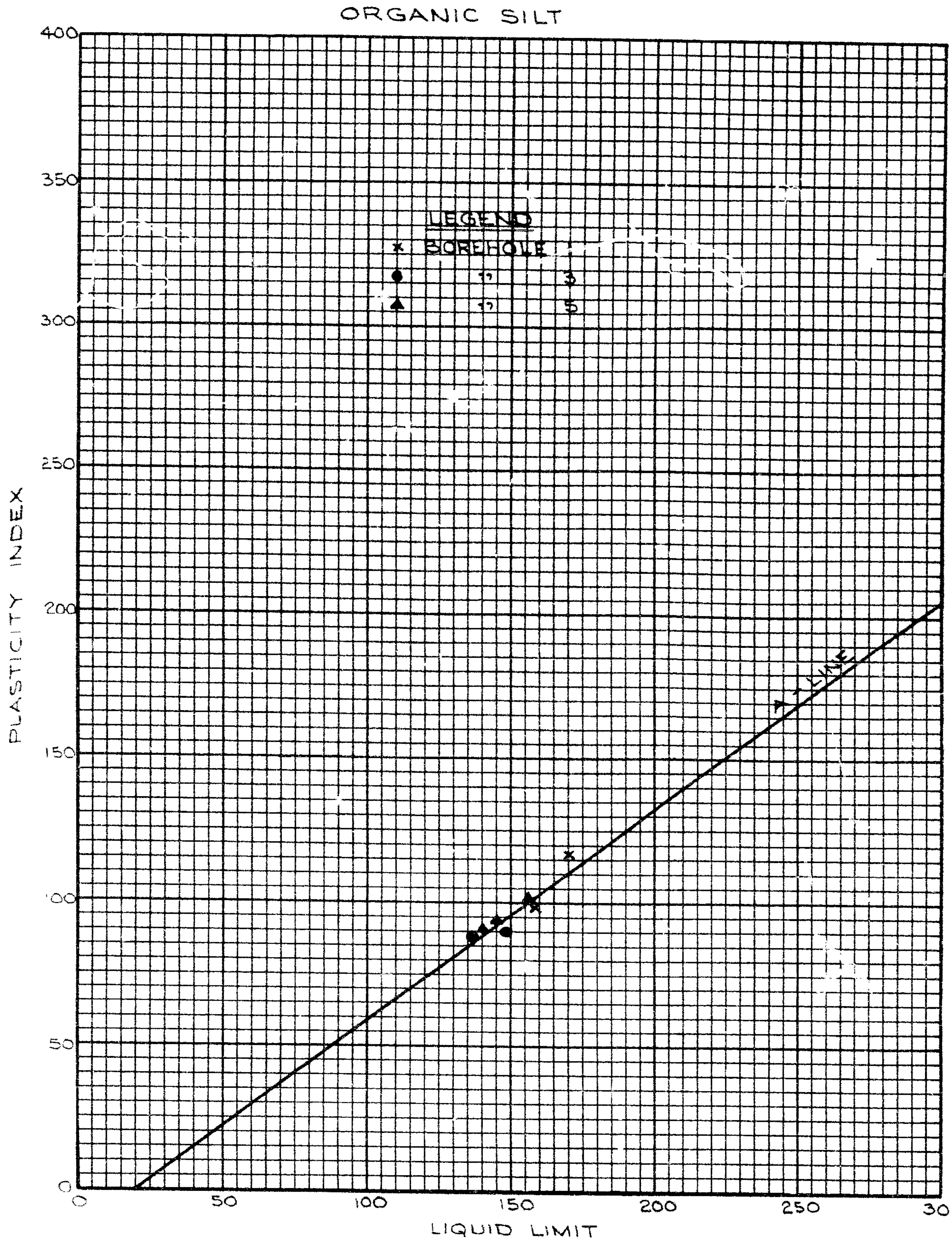
M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION

APPENDIX 11
FIGURE 1
PROJECT T7723

PLASTICITY CHART

APPENDIX II
FIGURE 2
PROJECT T7723



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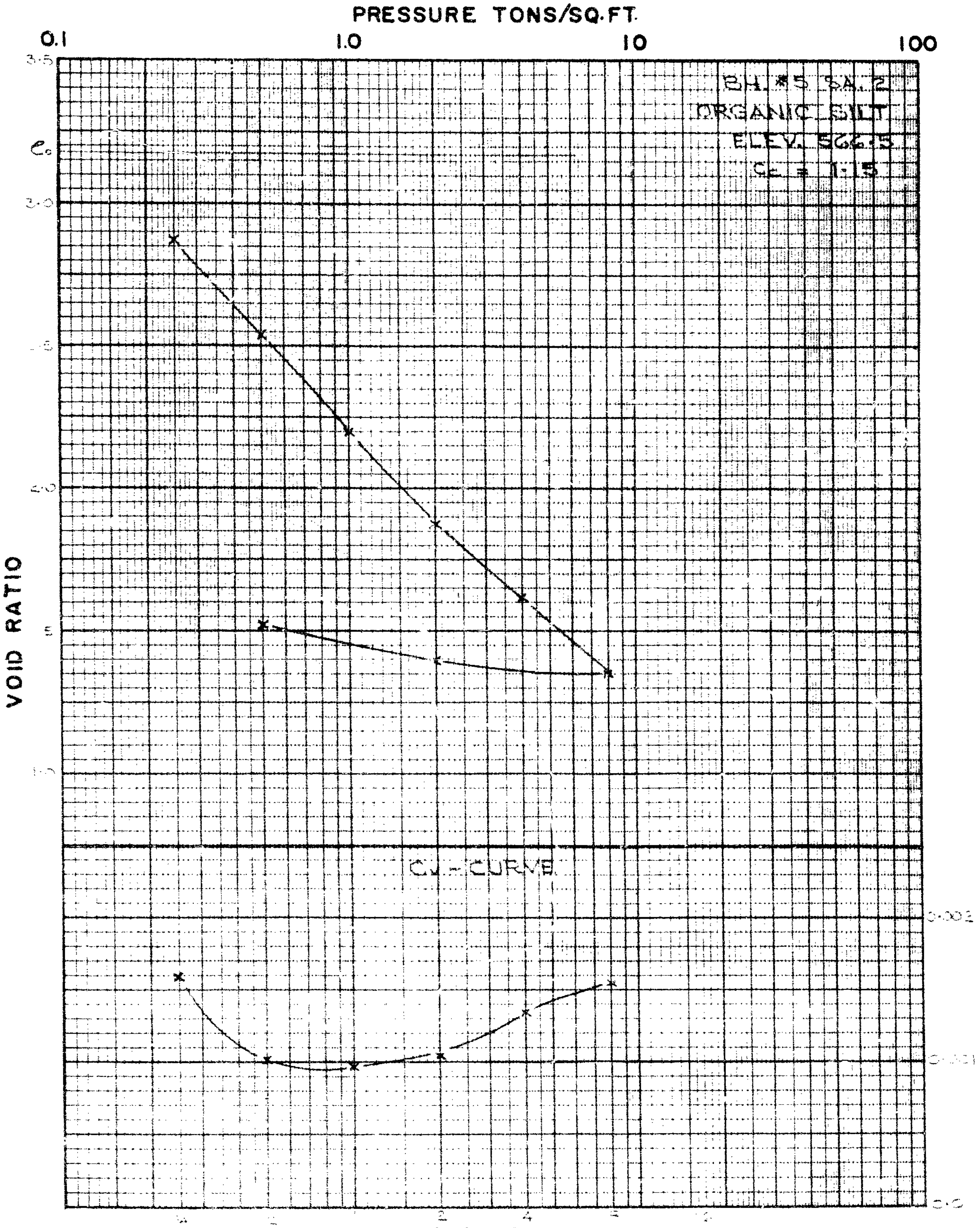
VOID RATIO-PRESSURE DIAGRAM
CONSOLIDATION TEST

APPENDIX II
FIGURE 3
PROJECT T7723



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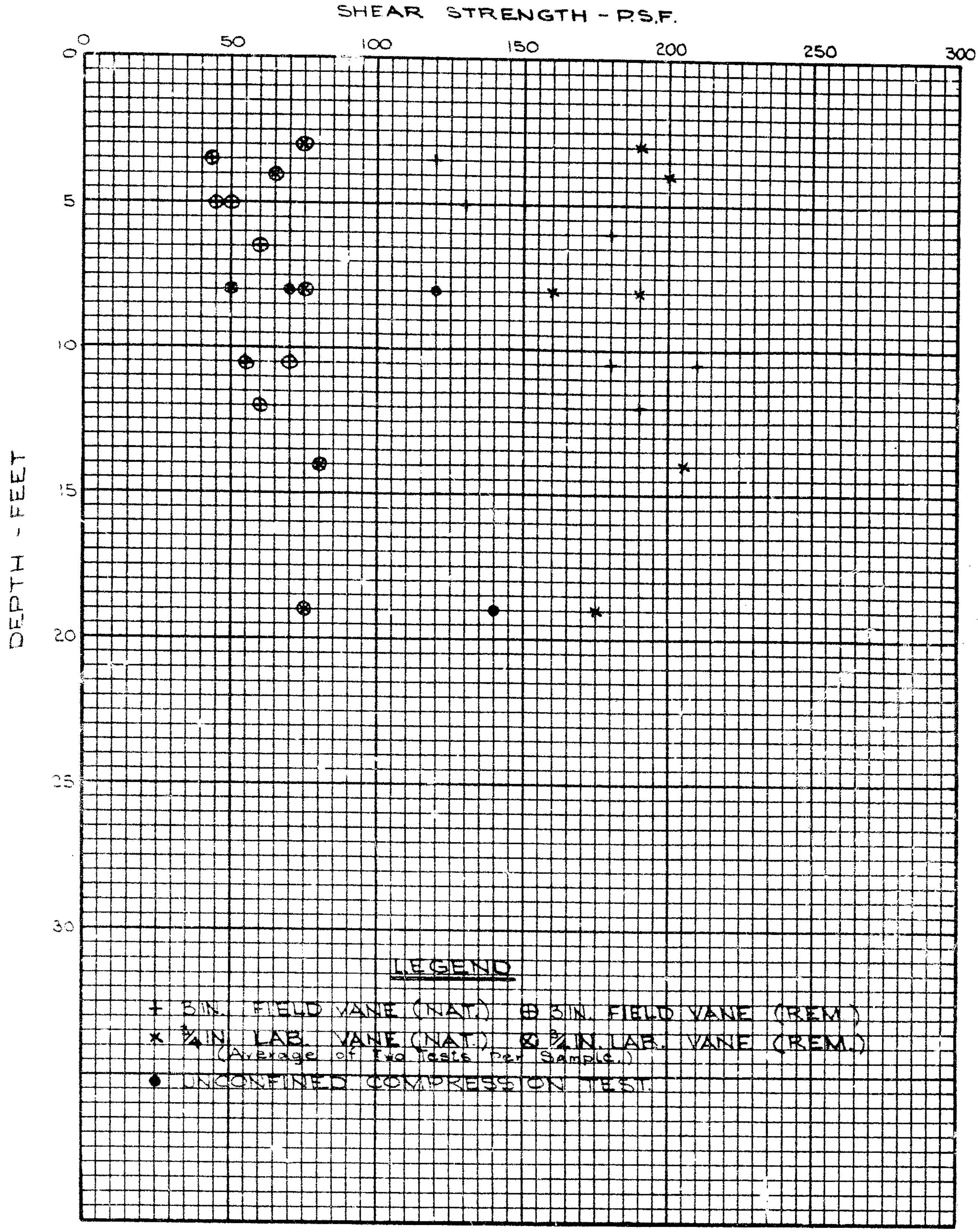
SHEAR STRENGTH vs. DEPTH

APPENDIX II
FIGURE 4
PROJECT T7723



GEOCON

cc: Foundations Office (Rm. 110)

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

Materials and Testing Division

January 25, 1965

Geocon, Limited,
14 Haas Road,
Rexdale, Ontario.

Attention: Mr. D. Eaton

Desbarats River

Re: W.P. 904-64, Hwy. 17, ~~[illegible] River~~ Bridge in Desbarats, Site 38S-179.
Hwy. 129, Nebskwashi River Bridge in Chapleau, Site 46-256. -- Dist. 18, Sault Ste. Marie --

Dear Sir:

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on January 19, 1965.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Eleven copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to February 17, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the D.H.O. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide the D.H.O. with Cronaflex copies of the drawings.

cont'd. /2 ...

Charges for the work performed will be in accordance with your Schedule of Rates, dated March 4, 1960, and invoice to be addressed to the attention of the undersigned.

Yours very truly,

AR.

A. Rutka,
MATERIALS & TESTING ENGINEER

AR:/MCa?

cc: Messrs. S. McCombie
F. De Visser
H. McArthur
A. A. Ward
G. F. Saint
R. D. Smith (2)
Mrs. T. Tate

Foundations Office
Gen. Files (2)

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: 208 Simpson Street,
FORT WILLIAM, Ontario.

Date: January 15, 1965.

Our File Ref.

In reply to

Subject: W.P. 904-64 - Site 385-179
Desbarats ~~Walker~~ River Bridge in Desbarats
Hwy. 17 - Dist. 18

Enclosed please find one print of plan E-4535-1 for the subject structure, on which is marked the location of the footings for a new structure.

Would you please have a foundation investigation carried out, in order to determine the type of foundation required for the structure, and the stability of the fill to the east of the proposed bridge.

S. McCombie
for F. DeVisser,
Regional Bridge Location Engineer.

FDeV/sp

cc. R. Fitzgibbon
N. D. Smith
S. McCombie

JOB GIVEN
TO SEGUN
JAN 19 —

cc: Foundations Office (Rm. 110).

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

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

Attention: Mr. A. McCamble

March 11, 1965

FOUNDATION INVESTIGATION REPORT BY:
Geocon, Limited, Consulting Engineers.
Proposed Desbarats River Bridge, Hwy. 17,
Site 385-179, District 18, W.P. 904-64

Attached, please find the above-mentioned report submitted by the Consultant, Geocon, Ltd. We have reviewed the report and herewith submit our comments for your consideration:

The main characteristic of the site is the presence of a very soft layer of organic silt overlying a dense to very dense till layer which is, in turn, underlain by sound Rhyolite bedrock. The thickness of the two soil layers varies considerably and the rock level is also very irregular.

The upper surface of the till layer within the area of the proposed new river crossing varies between elevations 562.5 (B.H. 3) and 570.7 (B.H. 2), thus sloping from West to East. Bedrock has only a few inches of cover at the North-east corner of the East abutment, while at the South-east corner there is more than 30 feet of overburden. Soundings to establish the way bedrock is sloping between these two points would have been very helpful.

Because of the variable bedrock elevation, it is impractical to found the structure directly on bedrock. The till is definitely competent to support 3.0 T/sq.ft. However, the excavation and de-watering may present certain problems due to

(a) the relatively high normal ground water table; and

(b) the artesian water head with its origin in the till layer.

We feel that consideration should be given to pouring a tremie seal within the sheeted excavation should the pumping prove to be impractical, or if it creates boiling conditions.

Bedrock outcrops only a short distance to the North of the proposed crossing, and the Consultant therefore suggests that re-locating the structure should be considered. We concur with this suggestion.

cont'd. /2 ...

Should there be any other questions that you would like to discuss, please feel free to call on our Office.

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
H. McArthur
A. A. Ward
E. R. Saint
F. De Visser
A. Watt

Foundations Office
Gen. Files