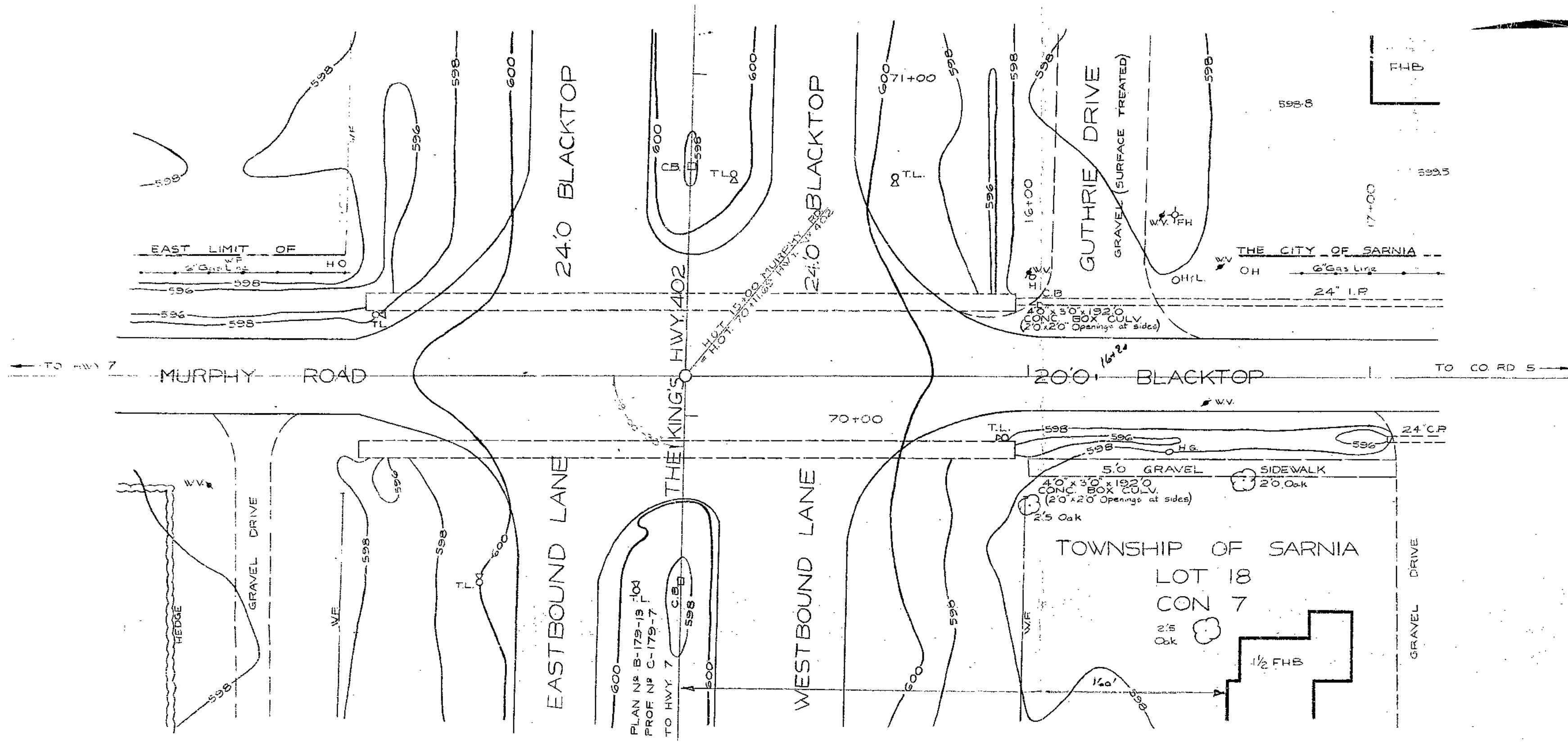
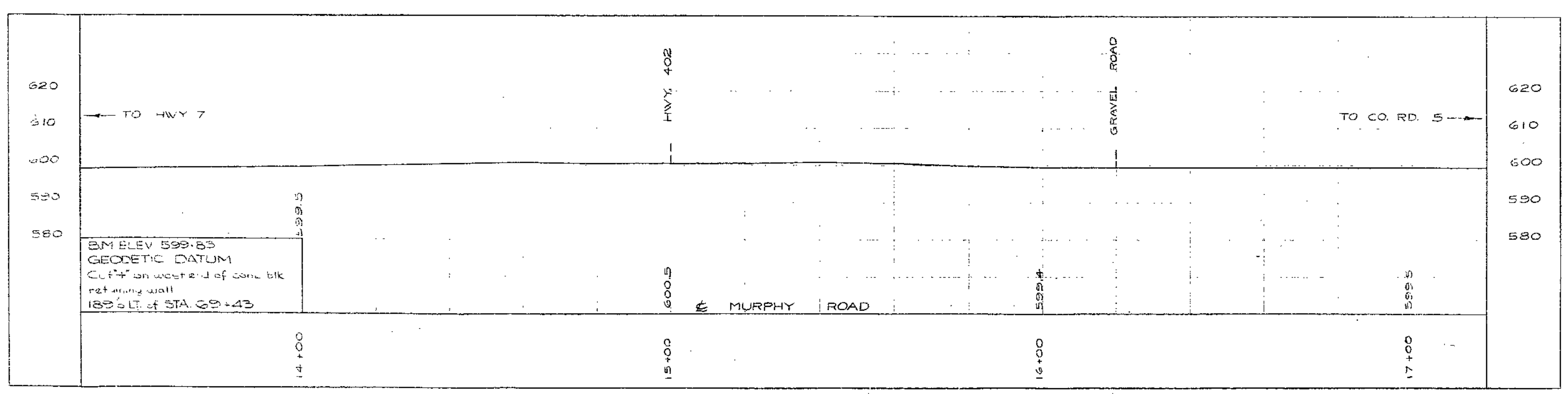
#65-F-229
W.P. #247-65
HWY #402
MURPHY
ROAD

E-4366-1

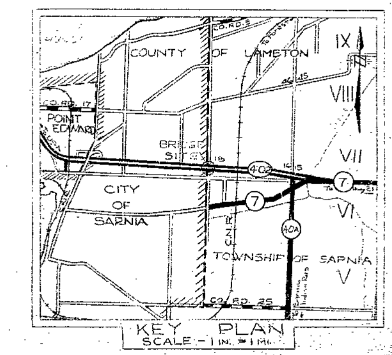
COUNTY of LAMBTON
CITY of SARNIA
LOT 19



PLAN
SCALE - 1 IN. = 20 FT.



PROFILE
H.O.T. 15+00 = 70+11.69 HWY. Nº 402
SCALES ~ HOR. VERT. ~ 1 IN. = 20 FT.



KEY PLAN
SCALE - 1 IN. = 1 MI.

GBM. Nº 2965 ELEV. 595.152
Concrete box culvert under Highway Nº 402 west of C.N.RWY. crossing. Tablet in top of culvert, 6 inches west and 6 inches south of northeast corner of culvert.

W.P. 247-65

DATE	REVISIONS & ADDITIONS	BY	CH'KD

DEPARTMENT OF HIGHWAYS ONTARIO
DESIGN BRANCH
ENGINEERING SURVEYS DIVISION

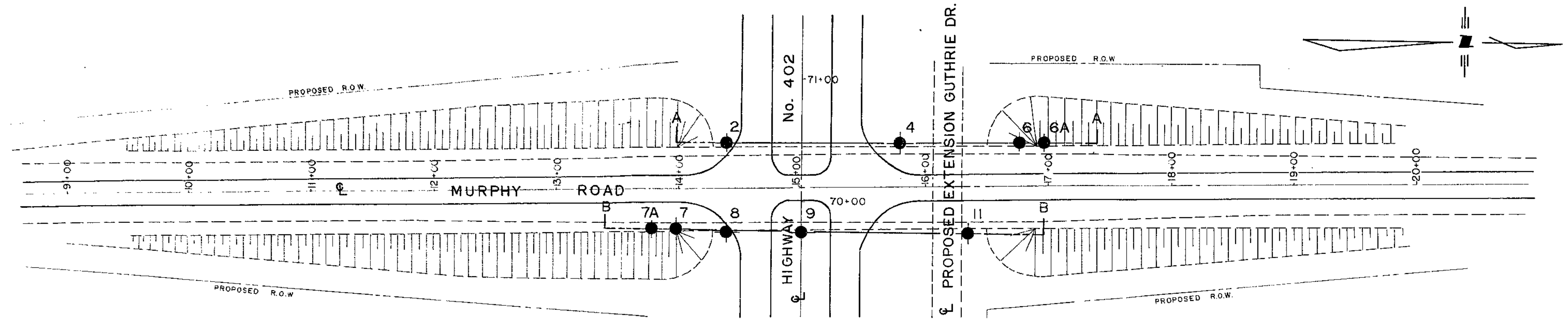
BRIDGE SITE

PROPOSED CROSSING
AT
MURPHY ROAD
AND
THE KING'S HWY. 402
LOT 19 LOT 18 CON. 7
CITY OF SARNIA TOWNSHIP OF SARNIA
COUNTY OF LAMBTON

SCALE	DISTRICT	REGION
AS SHOWN	CHATHAM	SOUTH-WESTERN
W.O. 9332-65-113	Date of Survey DEC. 1965 / Plan JAN. 1966	SITE Nº

SURVEY BY	DRAWN BY
Chief of Party M. MILLER Supervisor W.L. SMYTH	Draftsman E. ROY & B. HOEGY Supervisor J. CAMILLERI
CHECKED BY	PLAN Nº E-4366-1
Draftsman R. HALL Supervisor J. CAMILLERI	

388 6??E
4760000N
17
40 J16W



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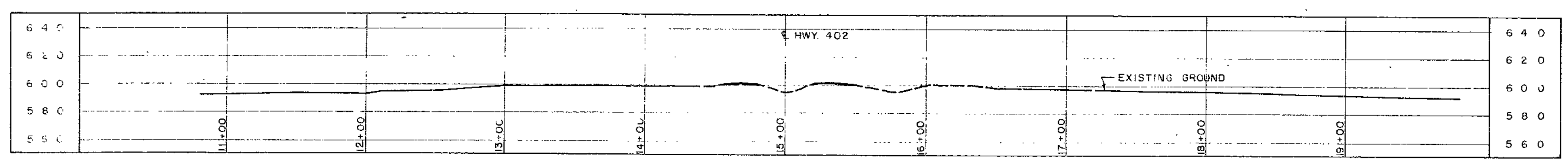
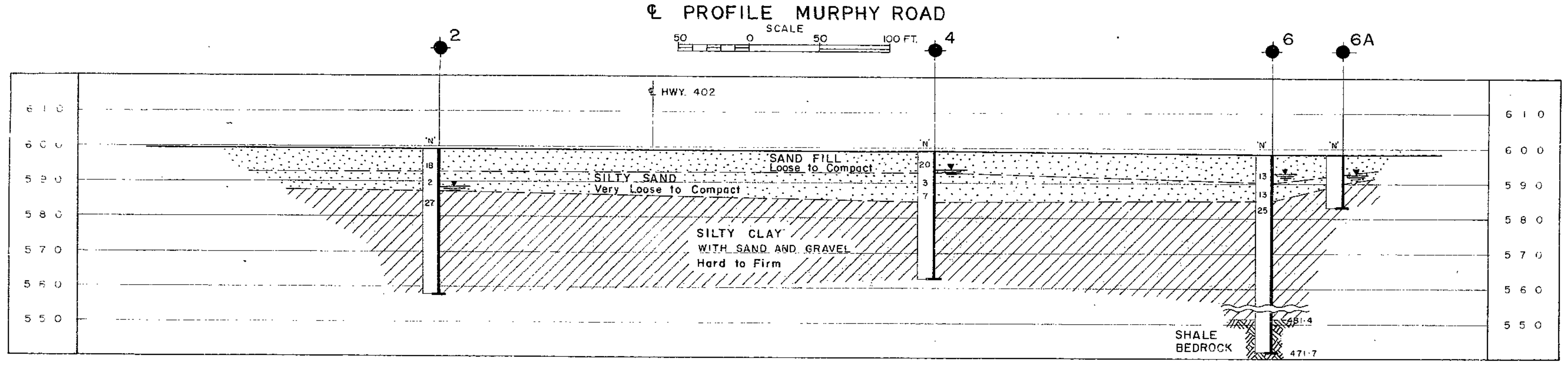
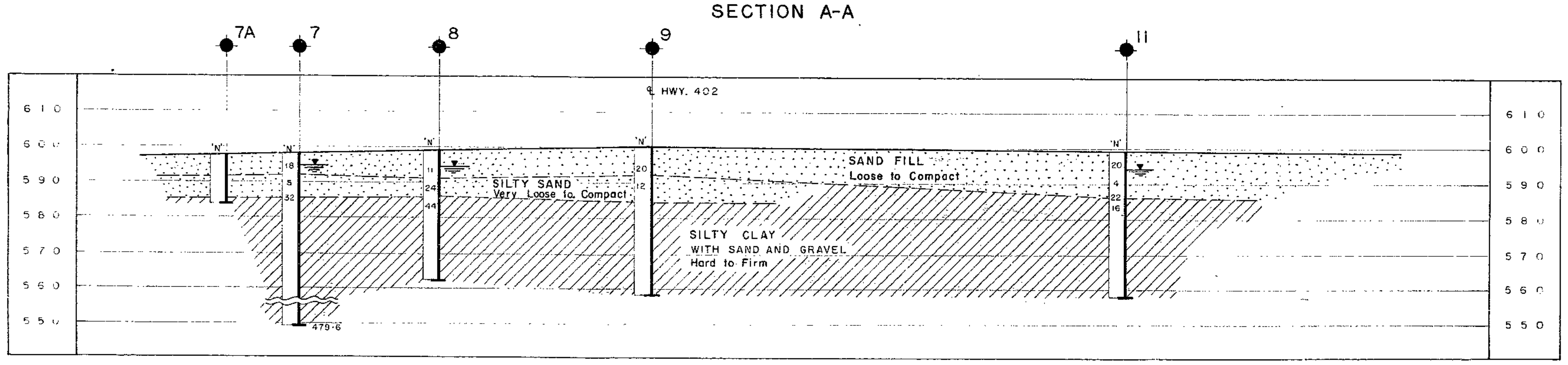


℄ PROFILE MURPHY ROAD
SCALE 50 0 50 100 FT.



SECTION A-A



SECTION B-B
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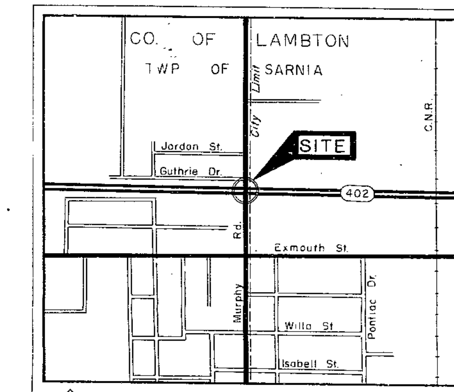
REF. No. 847-E-08



KEY PLAN
SCALE IN FEET 1000 500 0 1000

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. (Jan. 1966)

NO.	ELEVATION	STATION	OFFSET
2	599·3	14+39	37' LT.
4	599·3	15+30	35' LT.
6	598·2	16+76	35' LT.
6A	598·3	16+96	35' LT.
7	598·1	13+99	33' RT.
7A	598·0	13+79	33' RT.
8	599·2	14+79	35' RT.
9	600·2	15+00	36' RT.
11	599·1	16+35	38' RT

- 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
[illegible]	AFS	REVISED MURPHY [illegible] STATIONS OF CHAINAGE.

GEOCON LTD

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

MURPHY ROAD OVERPASS

KING'S HIGHWAY NO. 402 DIST. NO. 1
CO. LAMBTON
TWP. SARNIA LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. BMG	CHECKED HLM	W.P. NO. 247-65	DRAWING NO. T7836-1
DRAWN A.E.L.	CHECKED DBO	JOB NO.	
DATE JAN. 26, 1966		SITE NO.	BRIDGE DRAWING NO.
APPROVED D.B. Cass.		CONT. NO.	

cc: FOUNDATIONS

GEOCON LTD

HEAD OFFICE
420 MICHEL JASMIN, DORVAL, QUEBEC
TELEPHONE 631-9827

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

295 EAST 11TH AVENUE
VANCOUVER 10, B.C.
TEL. 879-2620

Rexdale, Ontario,
February 10th, 1966.

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

W.P. 247-65

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

Re: Soil Conditions and Foundations,
Proposed Murphy Road Overpass,
Highway No. 402,
Sarnia, Ontario.

Dear Sirs:

This letter accompanies our detailed engineering report on the above soil investigation.

The results of the investigation indicate that the overburden at the site consists of up to about 16 feet of loose silty sand and sand fill which overlies an extensive deposit of firm to hard brown to grey silty clay. The clay stratum was observed to have a thickness of about 100 feet and was found to overlie shale bedrock directly. The actual soil conditions are described in detail in the report.

Based on our findings, two alternative foundation types would be suitable for the support of the proposed structure. If the design of the proposed bridge calls for simply supported spans, where some differential settlement between the piers and abutments could be tolerated, then the piers could be founded on the surface of the clay and the abutments founded on spread footings carried within the embankment fill. Since the desiccated crust normally associated with the clay deposit in the Sarnia area is to a large extent absent the net allowable bearing value for the design of the pier foundations would be about one ton per square foot. In addition, the use of spread foundations as a foundation solution would require dewatering measures for excavations through the sand strata and would require also special precautions in the construction of the end of the embankment so that the abutment footings could be safely carried within the embankment fill as discussed in the report. The alternative foundation solution would be to support the abutments and piers on piles end-bearing on bedrock. The use of this foundation solution would, for all practical purposes, eliminate differential settlement and would allow a continuous design for the proposed bridge super-structure, as discussed in the report. A number of recommendations pertinent to the above foundation types, together with the design and construction of the approach embankments are given in the report.

We believe that this report provides all the information required from this investigation. Should you have any questions regarding any aspect of this report, please do not hesitate to call us.

Yours very truly,

GEOCON LTD

M. A. J. Matich per DBO

MAJM/reb

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

T7836

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED MURPHY ROAD OVERPASS

HIGHWAY NO. 402

SARNIA ONTARIO

Distribution:

10 copies - Department of Highways, Ontario, Downsview, Ontario.

3 copies - Geocon Ltd

GEOCON

INDEX

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INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario to carry out a soil investigation at the intersection of Highway 402 and Murphy Road at Sarnia, Ontario. The purpose of the investigation was to determine the soil and ground water conditions as they affect the design of foundations for the proposed bridge structure and the approach embankments.

SUMMARIZED SOIL CONDITIONS

The site investigated is covered by a layer of generally compact brown sand fill about 7 feet thick. The fill is underlain by generally loose fine to medium grey silty sand about 6 feet thick. Underlying the 13 feet of granular soil an extensive deposit of grey silty clay with sand and gravel was encountered. The clay stratum had a maximum observed thickness of 106 feet. About the top 4 feet of the clay stratum has a hard to very stiff consistency below which the clay has a generally stiff consistency increasing to very stiff with depth. The grey silty clay is underlain by black shale bedrock of the Devonian and Mississippian Age. The elevation of bedrock surface is about 480.

DISCUSSION

It is understood that it is proposed to construct an overpass along Murphy Road, overpassing Highway 402 at Sarnia, Ontario. It is also understood that tentative designs call for a five span bridge structure with a maximum finished grade at about elevation 621. With this design the approach embankments will have a maximum height of about 20 feet and will be about 700 feet long with a tentative maximum approach grade of 5 percent. The width of the embankment from shoulder to shoulder will be about 70 feet. In this discussion it is assumed that standard side slopes of 1 vertical to 2 horizontal will be used for the embankments, if stability conditions permit, and also that the embankment fill will be a select granular material.

Embankments

As discussed earlier the approach embankments will be 20 feet above present ground level and will have side slopes of 1 vertical to 2 horizontal. The stability of the embankments was checked by preliminary calculations and for the end-of-construction case, a factor of safety in excess of 1.5 was calculated. The computations were carried out using a unit weight of 130 pounds per cubic foot for the embankment fill and using a minimum undrained shear strength of 1000 pounds per square foot in the clay. The immediate stability of the embankment is therefore considered satisfactory. From previous experience relative to embankment stability in the area it is considered that the long term stability is satisfactory

Embankments (continued)

for the twenty-foot high embankment especially since it will be founded on a 13-foot thick existing granular layer.

Conventional settlement computations were carried out using the results of the consolidation test performed in the clay stratum. A compressionindex Cc of 0.11 and a rebound compression index Cr of 0.015 resulted from the plot of void ratio versus pressure curve. The overconsolidation pressure determined from the above curve was about 1 ton per square foot. However, from published data and previous experience in the area it has been established that the clay in the general area has been overconsolidated by about 0.5 tons per square foot. In the settlement computations therefore the clay was assumed to have an overconsolidation pressure of 0.5 tons per square foot and an overall depth of 100 feet; double drainage was also assumed.

With the above approach using the Cr value for applied loads up to 0.5 tons per square foot and the Cc value for applied loads greater than 0.5 tons per square foot, a maximum settlement along the centreline of the embankment of 7 inches was determined. It is however, known from previous experience in the Sarnia

Embankments (continued)

area that conventionally computed settlements are on the high side. Therefore, a settlement computation was carried out using the Cr value over the whole range of the applied load and a maximum settlement along the centreline of the embankment of 2 inches was determined. From the results of the above settlement computations and experience, it is recommended that allowance be made for at least 4 inches of ultimate consolidation settlement. About 2 inches of immediate settlement will occur as the embankment is raised to final grade due to settlement of the sand and elastic settlem ent of the clay.

Based on the coefficient of consolidation measured in the consolidation test and assuming two way drainage, the computed time for 90 percent consolidation is about 50 years. It is believed however, that this value is on the high side and should be in the order of 30 years since time settlement records of a large grain elevator in Sarnia are in agreement with this.

Foundations

The following points are relative to foundation design and construction and placement of engineered fill irregardless of the foundation type used.

Foundations (continued)

(a) All footings or pile caps subject to frost action should be protected by at least 4 feet of earth cover.

(b) The base of footing excavations which expose the grey silty clay stratum should be protected by a thin mud mat of lean concrete placed soon after the excavation has reached final grade since the silty clay is susceptible to softening from surface water and construction traffic.

(c) Where footings are founded on sand and gravel pads or in the embankment as discussed below, the engineered fill underlying the footings should be select well-graded granular material compacted to at least 100 percent of modified A. A. S. H. O. density for the full depth of the fill.

(d) All topsoil in the area of the proposed embankment should be removed prior to embankment construction.

(e) Backfill behind abutments should be free-draining and compacted as in(c) above.

(f) The sand layer overying the silty clay stratum is not suitable as a foundation material.

Foundations (continued)

Two possible solutions present themselve., as alternative foundation types.

(i) Spread Footings:

The bridge structure could be supported entirely on spread footings. In this case it would be necessary to found the pier footings on the surface of the natural clay stratum, and the abutment footings within the embarkment. An alternative for founding the pier footings on pads of well compacted select granular material is discussed below.

The significant stratum at the site for shallow spread pier footings is the hard to firm silty clay. The shear strength profile for the clay is given on Figure 1 in Appendix II. Using this profile and considering footings founded on the surface of the silty clay stratum at about elevation 585 a net allowable bearing value of 1.0 tons per square foot may be used. This value incorporates a recommended factor of safety of 3 against general shear failure. The above foundation solution will involve excavation in the granular layer overlying the silty clay to a maximum depth of about 15 feet. Since the water level in the granular

Foundations (continued)

(i) Spread Footings: (continued)

layer is about 5 feet below ground level it will be necessary to provide special dewatering measures. In this instance, the excavations could be carried out using steel sheet pile cofferdams with dewatering by the procedure of pumping from filter equipped sumps or by open cut with dewatering of the sand using a "sanded-in" vacuum well point system.

In view of the relatively small net allowable bearing value for the pier foundations on the clay stratum, consideration could be given to founding the pier footings at a higher elevation than given before and filling between the underside of footing and the surface of the clay with select well graded sand and gravel compacted to at least 100 percent of Modified Proctor density. In this case a net allowable bearing value of 2 tons per square foot could be used with the provision that the increase in pressure on the surface of the natural clay stratum figured on the basis of a 1 horizontal to 2 vertical load distribution below footing level should not exceed 1.0 tons per square foot. The pads of well graded sand and gravel should extend at least 2 feet beyond the edges of each footing and have side slopes no

Foundations (continued)

(i) Spread Footings: (continued)

steeper than 1 horizontal to 1 vertical.

In view of the low net allowable bearing value, the embankment surcharge at the abutments will preclude the practical use of the abutment footings carried on the surface of the natural clay stratum. However, the abutment footings could be founded high in the embankment with the underside of the footings 4 feet below ground level, that is, at maximum depth of frost penetration. A net allowable bearing value of 2.0 tons per square foot could be used in this case provided the underlying embankment fill meets the requirements specified in section (c) above. Also, it is necessary that the requirements for the distance between the top edge of the embankment and the nearest footing edge be met. For instance, for footings between 5 and 10 feet wide founded 4 feet below ground level, with a bearing value of 2 tons per square foot, a distance between embankment edge and footing edge of 10 feet is required. For footing widths outside this range the required distance should be re-checked.

Foundations (continued)

(i) Spread Footings: (continued)

Under the bridge pier loadings the consolidation settlement that will occur is computed to be in the order of 2 inches. In addition elastic settlement of the piers in the order of one inch will occur during construction. Therefore 2 inches of residual settlement of the superstructure supported by the piers will occur during the life of the structure. The embankments and therefore the abutments founded in the embankments will undergo 4 inches of consolidation settlement, as discussed, resulting in a differential settlement of 2 inches between the abutments and their adjacent piers during the life of the structure. It is assumed that this amount of movement could be tolerated from traffic considerations and the differential settlement could be accommodated in design by hinging the approach spans at piers adjacent to the embankments, and using simply-supported construction generally.

(2) Piles:

The bridge structure could be supported entirely on piles, in which case the supporting girders could be made continuous over the piers. Depending on the type of

Foundations (continued)

(2) Piles: (continued)

piled foundation selected, settlement of the abutments and piers due to structure loads and due to consolidation of the clay from the embankment surcharge could for all practical purposes be eliminated. Also, the use of a piled foundation would eliminate the difficulties associated with the construction of spread footings. The most suitable pile type would be one which derives its capacity in end-bearing on the shale bedrock. Friction piles would not have the desired effect as the clay stratum as a whole would be consolidating under the embankment and bridge loading.

A suitable pile type in this instance would be a non-displacement type such as a steel H pile or a displacement pile such as a tubular steel pile driven closed-end. In the event that displacement type piles are used it is recommended that preaugering be carried out for a depth equal to at least three-quarters of the anticipated pile length to avoid excessive horizontal drift of the pile tip. From our recent experience in the Sarnia area it is known that the tips of 10 inch diameter tubular steel piles driven closed-end without preaugering may drift horizontally as much as 15 feet over a length

Foundations (continued)

(2) Piles: (continued)

of 100 feet.

The use of a piled foundation would result in differential movement between the embankment and the abutments due to consolidation settlement under the embankment and this settlement would have to be compensated for by periodic maintenance. For design of the above piles negative skin friction should be allowed for the in the clay stratum.

CONCLUSIONS AND RECOMMENDATIONS

1) The site is covered by a layer of granular soil about 13 feet thick made up of about 7 feet of compact brown sand fill underlain by about 6 feet of loose grey silty sand. The granular layer is underlain by about 100 feet of generally stiff grey silty clay with sand and gravel. The clay stratum is underlain by black shale bedrock.

2) The proposed approach embankments with a maximum height of 20 feet and side slopes of 2:1 will be stable. However, it is estimated that under the maximum height, the embankments will undergo a total consolidation settlement of 4 inches.

3) The allowable bearing value on the surface of the clay stratum for the design of spread footings is 1.0 tons per square foot as discussed in the report.

4) If spread footings are used for the piers and abutments the structure will undergo total and differential settlements as discussed in the report. An alternative that would eliminate settlement of the bridge structure would be to found the piers and abutments on piles end-bearing on the bedrock.

PERSONNEL

The field work for this investigation was carried out under the supervision of Mr. B. M. Ghadiali, P. Eng. This report was written by Mr. H. L. MacPhie, checked by Mr. D. B. Oates, P. Eng. and reviewed by Mr. M. A. J. Matich, P. Eng.

H.L. MacPhie per DBO.

HLM/reb

H. L. MacPhie, P. Eng.,
Senior Soils Engineer.

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL CONDITIONS

WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

PROCEDURE

The field work for this investigation was commenced on January 4th, 1966 and completed on January 20th, 1966. A total of 9 boreholes were put down using a mobile power auger. Initially 12 boreholes numbered 1 to 12 were proposed, that is, two boreholes at each of the two abutments and four piers. However, in the process of the field work boreholes 1, 3, 5, 10 and 12 were deleted and augerholes 6A and 7A were added. Two inch Shelby tube samples were taken in the clay stratum and two inch split spoon samples were taken in both the clay stratum and the granular soil overlying the clay stratum . A piezometer was installed in borehole 2.

Detailed logs of the boreholes are presented on the Office Reports on Soil Exploration in Appendix I. The location of the boreholes together with the inferred soil stratigraphy are shown on Drawing T7836-1 located in the pocket at the rear of this report.

The laboratory testing of the soil samples was carried out in the soil mechanics laboratory of Geocon Ltd in Toronto. The results of the laboratory testing are plotted on the Office Reports on Soil Exploration in this Appendix and on the Figures in Appendix II. The soil samples remaining after testing will be stored until February 1st, 1967, at which time you will be contacted for instructions regarding their disposal.

All elevations in this report are referred to Geodetic datum. The bench mark used in this investigation is located 100 feet right of Highway 402, chainage 69+79. The bench mark is the top of a Department of Highways, Ontario monument and has a quoted elevation of 598.91.

SITE AND GEOLOGY

The site investigated is located at the intersection of Highway 402 and Murphy Road at Sarnia, Ontario. The ground surface in the area of the site is generally flat lying at about elevation 600.

From available geological information and previous experience in the area, it is known that the area is covered by a thick stratum of silty clay. Generally, the clay has a hard to stiff desiccated crust and is underlain by shale bedrock of the Devonian and Mississippian Age. The clay stratum contains sand and gravel sizes and is sometimes referred to as a clay till.

SOIL CONDITIONS

The principal soil strata encountered in the boreholes are as follows:

Topsoil

A layer of topsoil 3 inches thick was encountered at the location of boreholes 7 and 11. At the time of investigation the actual extent of the thin topsoil cover could not be determined accurately because the site was snow covered.

Loose to Compact Brown Sand Fill

A layer of brown sand fill was found to cover the site and was encountered in all the boreholes. The thickness of the fill layer varied from 5.5 feet at borehole 6A to 13 feet at borehole 11 and had an average thickness of about 7 feet. The fill was generally brown but was brown to grey brown in borehole 11. The sand fill layer contained occasional fine gravel traces and silt traces; sand and gravel fill was encountered at borehole 9. It is believed that the fill was placed to bring the intersection of Murphy Road and Highway 402 to final grade.

One mechanical analysis test was carried out on a typical sample from the fill and the results are plotted on Figure 2 of Appendix II. The grain size distribution curve indicates that the material consists of about 15 percent gravel sizes and 85 percent sand sizes.

Loose to Compact Brown Sand Fill (continued)

Standard penetration resistances determined in the fill ranged from 4 to 22 blows per foot with an average of 16 blows per foot. Based on the above "N" values the relative density of the fill layer ranges from loose to compact and is generally compact. Standard penetration resistances in the sand fill were not determined in augerholes 6A and 7A since the purpose of these augerholes was to determine the boundaries of the fill layer at the respective locations.

Very Loose to Compact Grey Silty Sand

Underlying the brown sand fill is a stratum of grey silty sand. This stratum was encountered in all the boreholes with the exception of borehole 11 where the grey sand stratum was not encountered. The thickness of the stratum ranged from 2.5 feet at borehole 6A to 8.5 feet at borehole 4 and had an average thickness of 6 feet. From visual observation the grey silty sand stratum was predominantly composed of fine to medium sand sizes with silt and at borehole 7 the stratum graded from a silty sand at the top of the stratum to a sandy silt at the bottom of the stratum. The stratum was generally grey in colour but was brown to grey at borehole 4. The natural grey silty sand stratum was distinguished from the overlying fill layer due to the definite change in colour

Very Loose to Compact Grey Silty Sand (continued)

and grain size distribution between the two layers.

One mechanical analysis test was carried out on a typical sample of the grey silty sand stratum and the results are plotted on Figure 3 of Appendix II. The grain size distribution curve indicates that the silty sand consists of 86 percent sand sizes, and 14 percent silt sizes. Examination of all the samples recovered indicates that in general, the stratum has a slightly higher silt content than indicated above.

Standard penetration resistances determined in this stratum ranged from 2 to 24 blows per foot and had an average of 8 blows per foot. Based on the above "N" values the relative density of this stratum ranges from very loose to compact and is generally loose. Standard penetration resistances were not determined in the grey silty sand stratum in augerholes 6A and 7A since the purpose of these augerholes was to determine the boundaries of this stratum at the respective locations.

Hard to Firm Grey Silty Clay with Sand and Gravel

Underlying the grey silty sand at the location of 8 boreholes and the brown sand fill at the location of borehole 11 is an extensive deposit of grey silty clay with sand and gravel. Eight

Hard to Firm Grey Silty Clay with Sand and Gravel (continued)

boreholes were terminated in this stratum and the stratum was fully penetrated at borehole 6. The maximum observed thickness of the silty clay stratum was 106 feet at borehole 7 where boring and sampling was carried out to a depth of 37 feet below ground level after which the borehole was advanced to refusal, without sampling, to a depth of 118.5 feet below ground level. It is believed that the thickness of the clay stratum is generally about 100 feet over the site due to the relatively level nature of the bedrock surface in the area. The brown desiccated crust generally encountered in this geological area was not observed at this site except in borehole 2.

Atterberg limits were determined on three samples of the silty clay stratum. Values of liquid limit ranged from about 27 to 31 with an average of 28 and values of plastic limit ranged from about 14 to 16 with an average of about 15. The corresponding natural moisture contents ranged from about 18 to 20 with an average of about 19. The results of the above limit values indicate that the clay is inorganic and of low to medium plasticity.

Wet unit weights determined on samples from the silty clay stratum ranged from 130 to 137 pounds per cubic foot with an average value of 132 pounds per cubic foot.

Hard to Firm Grey Silty Clay with Sand and Gravel (continued)

The sensitivity of the clay as determined from laboratory vane shear tests was found to range from 1.5 to 7 and generally from 1.5 to 4 with an average value of about 3.

Unconfined compression tests were carried out on representative samples of the stratum and resulting undrained shear strengths were found to vary from 800 to 1660 pounds per square foot. Laboratory vane tests gave results of undrained shear strengths ranging from 860 to greater than 2000 pounds per square foot. The results of the above strength tests are shown as a plot of shear strength versus depth in Figure 1 in Appendix II. Figure 1 also shows the strength versus depth profile obtained by Geocon Ltd for the Department of Highways, Ontario at Highway 402 and Modeland Road. This latter profile was obtained using in-situ vane testing. Both types of tests show a similar scatter and magnitude of shear strength values.

Standard penetration resistances determined in the top part of the clay stratum gave "N" values ranging from 16 to 44 blows per foot with an average of 29 blows per foot. Two further standard penetration resistances determined at depth in borehole 6 gave "N" values of 7 and 18.

Hard to Firm Grey Silty Clay with Sand and Gravel (continued)

From the above results of undrained shear strengths and "N" values it is inferred that a thin discontinuous layer of hard to very stiff clay about 4 feet thick exists at the top of this stratum below which the consistency ranges generally from stiff to very stiff with depth.

One consolidation test was carried out on a sample from the stratum and the results are shown on Figure 4 of Appendix II. The compression index, Cc, was found to be 0.11 and the rebound compression index, Cr, was found to be 0.015.

Black Shale Bedrock

Underlying the grey silty clay stratum at the location of borehole 6 sound black shale bedrock was encountered. The bedrock was cored in BX size for about 10 feet. Bedrock was encountered at about elevation 481. Refusal to augering was encountered at about elevation 480 in borehole 7 and it may be inferred that this is bedrock surface. As a general rule, in the Sarnia area the bedrock and clay are usually spearated by a layer of sand and gravel. This layer was not observed in the two holes which were taken down to bedrock or refusal.

WATER CONDITIONS

During the field work a water level was observed in the granular layer overlying the clay stratum, about 5 feet below ground level, that is, at about elevation 594. A piezometer was installed in the silty clay stratum encountered at borehole 2 and the water level observed on February 3rd, 1966 was 11 feet below ground level at about elevation 588. The observations of the water level in the piezometer indicate that the water level was rising slowly over a period of about one week. For all practical purposes the ground water level, at the time of the investigation, was at elevation 594. At borehole 6 where bedrock was encountered, it was found that gas pressure existed and was of sufficient magnitude to displace the water in the borehole in the form of a jet of water rising about 8 feet above ground level.

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 T7836 BORING # 2 And 4 DATUM GEODETIC CASING BX.
BORING DATE JAN. 13-20/66 REPORT DATE JAN. 25, 1966 COMPILED BY AEL CHECKED BY H.L.M.
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 PCF
- 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

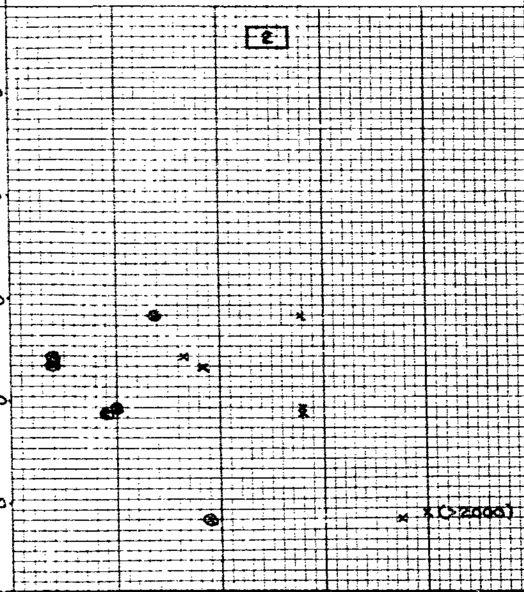
SHEAR STRENGTH - PSF. LAB. VANE x (Nat.) ⊗ (REm.) V_L - UNCON. COMP. ▲ (U) 500 1000 1500 2000
WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw 10 20 30 40
DYNAMIC PENETRATION TEST BLOWS PER FOOT

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

Boring 2

ELEVN. / DEPTH	DESCRIPTION
599.3 / 0.0	GROUND LEVEL
	COMPACT BROWN SAND FILL
592.8 / 6.5	VERY LOOSE GREY SILTY SAND
587.3 / 12.0	VERY STIFF TO FIRM BROWN TO GREY SILTY CLAY WITH SAND AND GRAVEL
557.8 / 41.5	END OF HOLE

Water conditions: BENTONITE PLUG; W.L. @ EL. 588.5 FEB. 3, 1966; PIEZOMETER

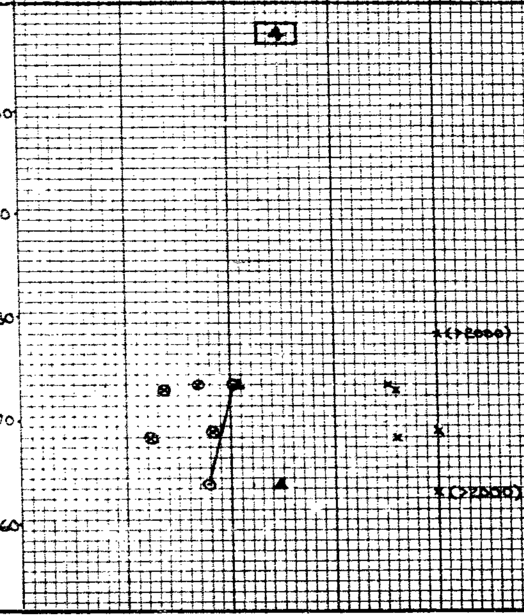


OTHER TESTS	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2" DO	1	18
	" DO	2	2
	" DO	3	27
V_L	" TO	4	HYD. PUSH
V_L V_L	" TO	5	"
V_L V_L	" TO	6	"
	" TO	7	"
V_L V_L	" TO	8	"

Boring 4

ELEVN. / DEPTH	DESCRIPTION
599.3 / 0.0	GROUND LEVEL
	COMPACT BROWN SAND FILL
593.3 / 6.0	VERY LOOSE TO LOOSE BROWN TO GREY SILTY SAND
584.8 / 14.5	VERY STIFF TO STIFF GREY SILTY CLAY WITH SAND AND GRAVEL
562.8 / 36.5	END OF HOLE

Water conditions: W.L. @ EL. 594.0 JAN. 13, 1966



OTHER TESTS	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2" DO	1	20
	" DO	2	3
	" DO	3	7
	" TO	4	PUSH 4" 22/8"
V_L	" TO	5	PUSH
γ = 130 V_L	" TO	6	"
V_L V_L	" TO	7	"
γ = 130 V_L	" TO	8	"

APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7836 BORING # 6 DATUM GEODETIC CASING BX.
BORING DATE JAN. 6-11/66 REPORT DATE JAN. 17, 1966 COMPILED BY AEL CHECKED BY HLM
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

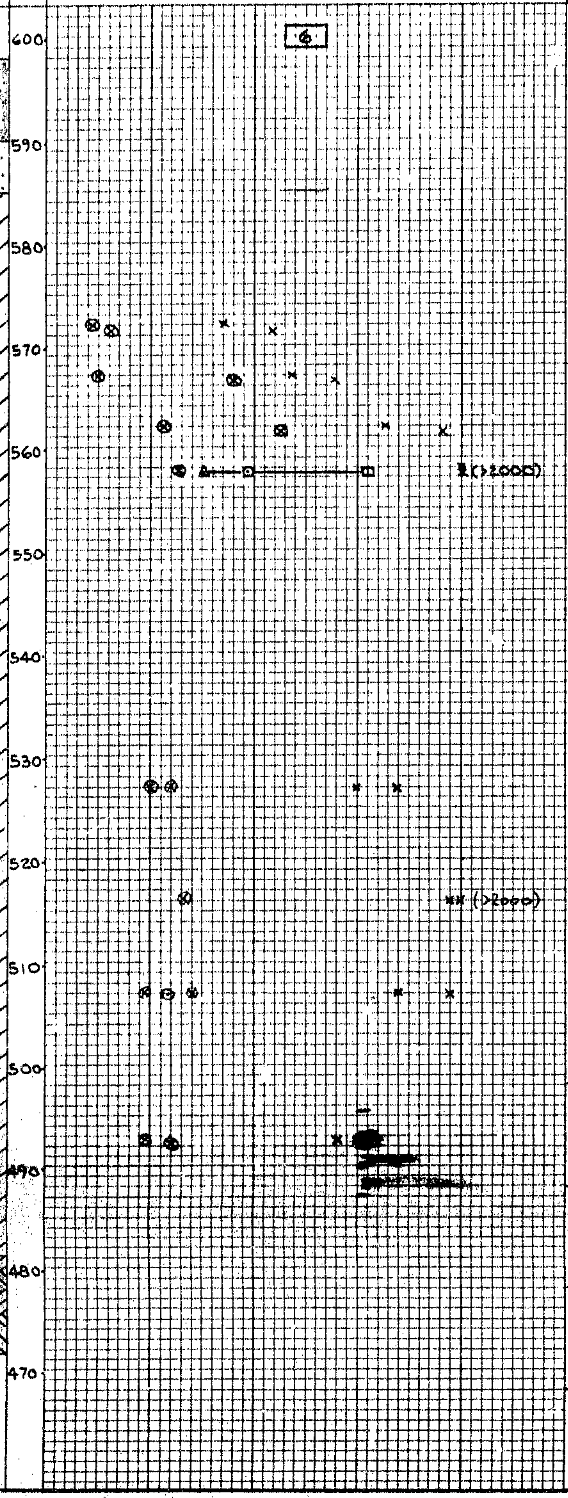
SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

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

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
598.2 / 0.0		GROUND LEVEL
		COMPACT BROWN SAND FILL
590.2 / 8.0		COMPACT GREY SILTY SAND
585.2 / 13.0	WL. @ EL. 592.7 JAN. 6, 1966	VERY STIFF TO FIRM GREY SILTY CLAY WITH SAND AND GRAVEL
481.4 / 116.8		BLACK SHALE BEDROCK
471.7 / 126.5		END OF HOLE

SHEAR STRENGTH - PSF.
LAB. VANE x (Nat.) ⊗ (Rem.) VL
500 1000 1500 2000
WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw
10 20 30 40
DYNAMIC PENETRATION TEST BLOWS PER FOOT



OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		2" DO	1	13
		" DO	2	13
		" DO	3	25
		" TO	4	HYD. PUSH
VL		" TO	5	"
VL		" TO	6	"
VL		" TO	7	"
VL		" TO	8	"
		" DO	9	7
		" DO	10	18
		" TO	11	HYD. PUSH
		" TO	12	PUSH
		" TO	13	"
VL		" TO	14	HYD. PUSH
RC. RECOV 100%		BX. RC.	15	–

APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7836 BORING # 6A And 7 DATUM GEODETIC CASING BX

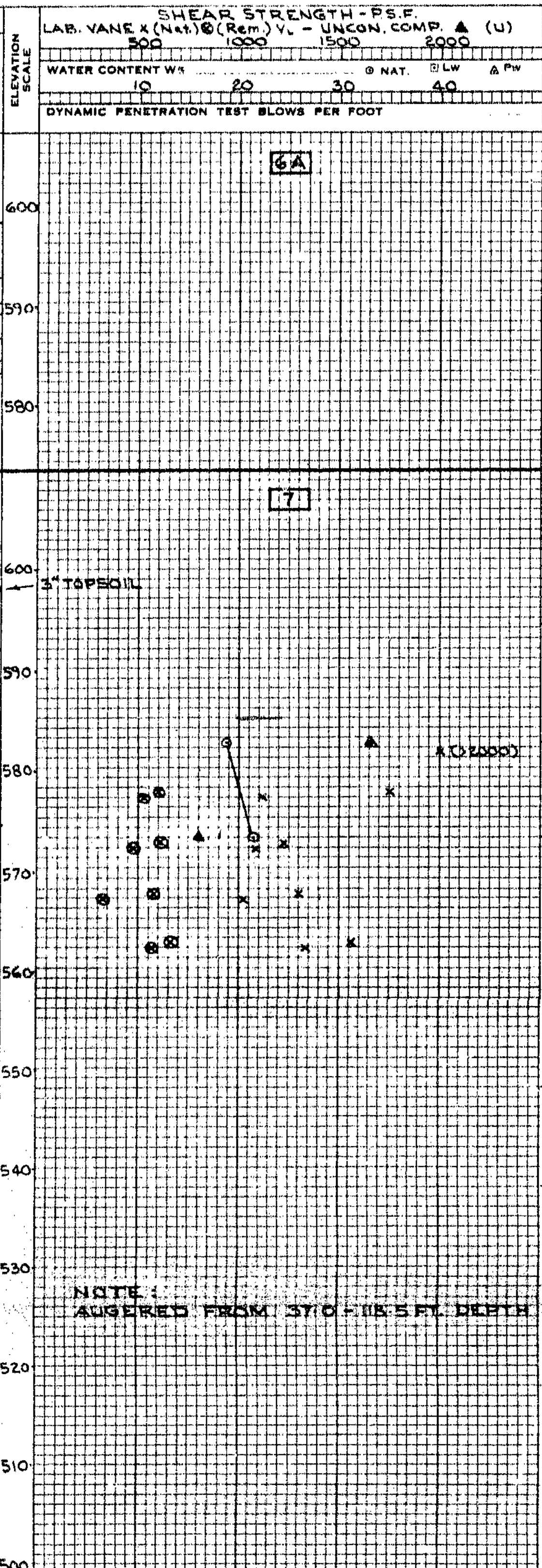
BORING DATE JAN. 13-19/66 REPORT DATE JAN. 17, 1966 COMPILED BY A.E.L. CHECKED BY H.L.M.

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION	SAMPLE TYPES		ABBREVIATIONS	
DISTURBED	A.S. - AUGER SAMPLE	F.S. - FOIL SAMPLE	V - IN-SITU VANE TEST	γ - WET UNIT WEIGHT - P.C.F.
FAIR	S.T. - SLOTTED TUBE	S.O. - SLEEVE-OPEN	M - MECHANICAL ANALYSIS	K - PERMEABILITY
GOOD	W.S. - WASHED SAMPLE	S.F. - SLEEVE-FOOT VALVE	U - UNCONFINED COMPPESSION	C - CONSOLIDATION
LOST	D.O. - DRIVE-OPEN	T.O. - THIN WALLED OPEN	Qc - TRIAXIAL CONSOLIDATED UNDRAINED	
	D.F. - DRIVE-FOOT VALVE	R.C. - ROCK CORE	Q - TRIAXIAL UNDRAINED	WL - WATER LEVEL IN CASING
	C.S. - CHUNK SAMPLE		S - TRIAXIAL DRAINED	WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
6A							
598.3 / 0.0	WL @ EL. 592.8 JAN. 13 1966	GROUND LEVEL					
		BROWN SAND FILL					
592.8 / 5.5							
590.3 / 8.0		GREY SILTY SAND					
		GREY SILTY CLAY WITH SAND AND GRAVEL					
583.3 / 15.0		END OF HOLE					
7							
598.1 / 0.0		GROUND LEVEL					
		COMPACT BROWN SAND FILL	M	DISTURBED	2"DO	1	18
592.1 / 6.0		LOOSE GREY SILTY SAND TO SANDY SILT		DISTURBED	"DO	2	5
585.6 / 12.5	WL @ EL. 594.7 JAN. 14, 1966			DISTURBED	"DO	3	32
		HARD TO FIRM GREY SILTY CLAY WITH SAND AND GRAVEL	γ=150 V_L	GOOD	"TO	4	PUSH 12" 15/10"
			V_L V_L	GOOD	"TO	5	PUSH
			γ=133 V_L V_L	GOOD	"TO	6	"
			V_L V_L	GOOD	"TO	7	"
			V_L V_L	GOOD	"TO	8	"
561.1 / 37.0							
		GREY SILTY CLAY WITH SAND AND GRAVEL					



APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7836 BORING # 7A DATUM GEODETIC CASING —
BORING DATE JAN. 19/66 REPORT DATE JAN. 25, 1966 COMPILED BY A E L. CHECKED BY H. L. M.
SAMPLER HAMMER WT. — LBS. DROP — INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN 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 — WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw — DYNAMIC PENETRATION TEST BLOWS PER FOOT

7A

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	ELEVATION SCALE	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
598.0 / 0.0		GROUND LEVEL	600					
592.0 / 6.0		BROWN SAND FILL			DISTURBED	AS	1	—
585.5 / 12.5		GREY SILTY SAND	590		DISTURBED	AS	2	—
14.0		GREY SILTY CLAY WITH SAND AND GRAVEL			DISTURBED	AS	3	—
		END OF HOLE	580					

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7836 BORING # 8 And 9 DATUM GEODETIC CASING BX.
BORING DATE JAN. 4-5, 1966 REPORT DATE JAN. 10, 1966 COMPILED BY AEL. CHECKED BY HLM
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

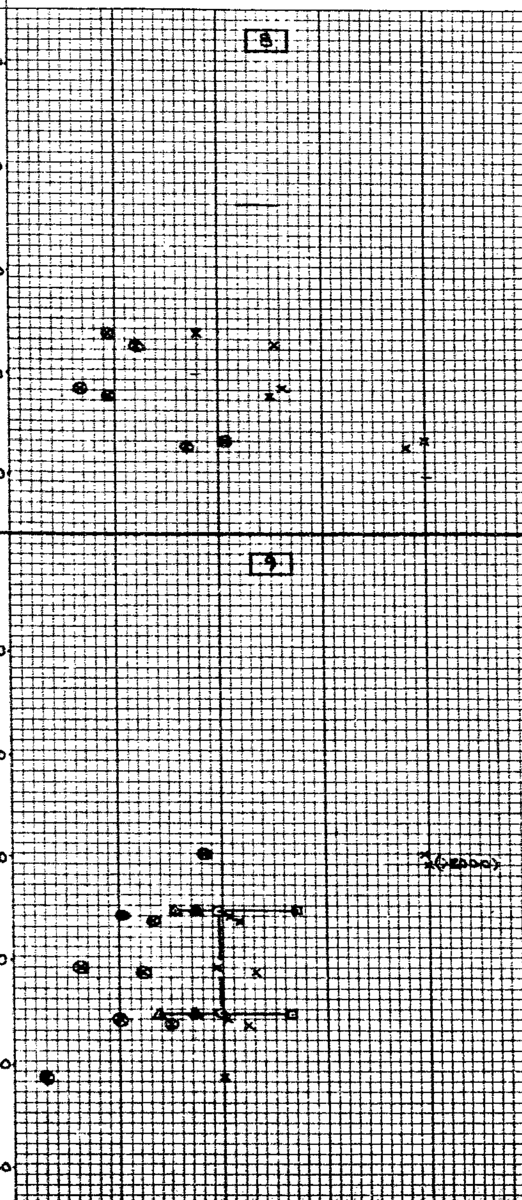
SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

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-PCF.
- 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 — UNCON. COMP. ▲ (U) 500 1000 1500 2000
WATER CONTENT W% ⊙ NAT. ⊡ Lw △ Pw 10 20 30 40
DYNAMIC PENETRATION TEST BLOWS PER FOOT



Boring 8

ELEVN. DEPTH	DESCRIPTION	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
599-2 / 0-0	GROUND LEVEL					
	COMPACT BROWN SAND FILL			2"DO	1	11
591-2 / 8-0	COMPACT GREY SILTY SAND	M		"DO	2	24
586-2 / 13-0	HARD TO FIRM GREY SILTY CLAY WITH SAND AND GRAVEL			"DO	3	44
				"TO	4	HYD. PUSH
		V_L		"TO	5	"
		V_L		"TO	6	"
		V_L		"TO	7	"
562-2 / 37-0	END OF HOLE					

Water conditions: WL. @ EL. 594.7 JAN. 4, 1966

Boring 9

ELEVN. DEPTH	DESCRIPTION	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
600-2 / 0-0	GROUND LEVEL					
	COMPACT BROWN SAND AND GRAVEL FILL			2"DO	1	20
592-2 / 8-0	COMPACT GREY SILTY SAND			"DO	2	12
				"DO	3	—
584-2 / 16-0	HARD TO FIRM GREY SILTY CLAY WITH SAND AND GRAVEL	V_L		"TO	4	HYD. PUSH
		γ=132 V_L		"TO	5	"
		V_L		"TO	6	"
		γ=132 V_L		"TO	7	"
		V_L		"TO	8	"
558-2 / 42-0	END OF HOLE					

APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

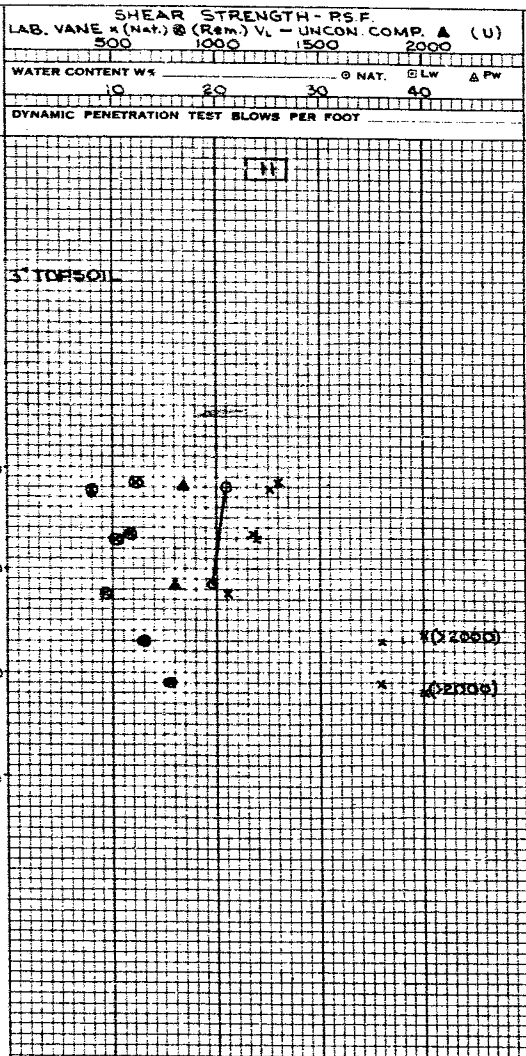
CONTRACT T7836 BORING # 11 DATUM GEODETIC CASING BX
BORING DATE JAN. 12 /66 REPORT DATE JAN. 18, 1966 COMPILED BY AEL CHECKED BY HLM
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

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

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
599.1 / 0.0		GROUND LEVEL; 3" TOPSOIL					
	▽	COMPACT TO LOOSE BROWN AND GREY BROWN SAND FILL		Disturbed	2" DO	1	20
				Disturbed	" DO	2	4
585.8 / 13.3				Disturbed	" DO	3	22
	W.L. @ EL. 594.0 JAN. 12, 1966	VERY STIFF TO FIRM GREY SILTY CLAY WITH SAND AND GRAVEL		Disturbed	" DO	4	16
			γ=131 V_L	Good	" TO	5	PUSH
			V_L	Good	" TO	6	"
			γ=137 V_L	Good	" TO	7	"
			V_L	Good	" TO	8	"
557.6 / 41.5			C V_L	Good	" TO	9	"
		END OF HOLE					



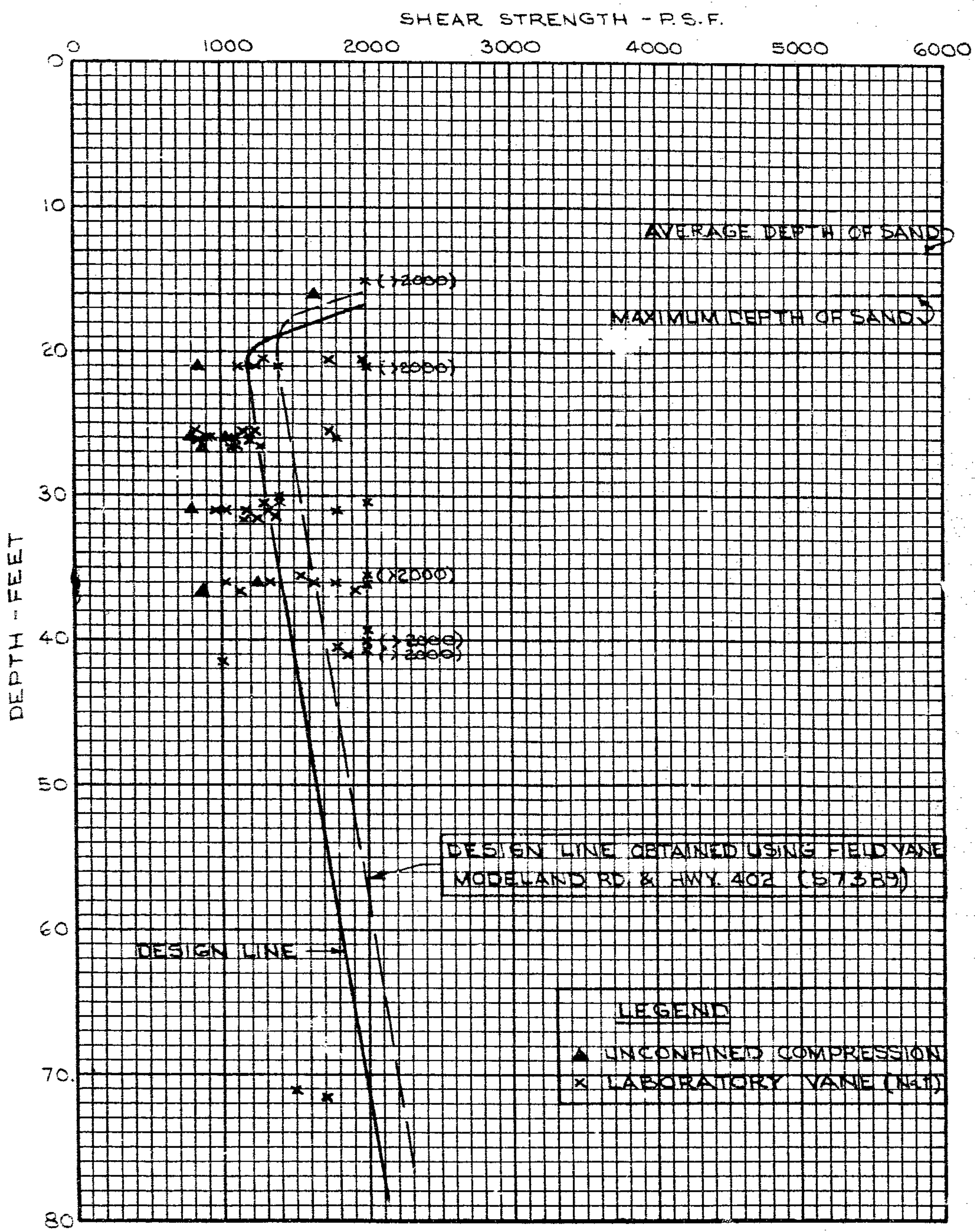
APPENDIX II

FIGURES - LABORATORY TESTING

GEOCON

SHEAR STRENGTH versus DEPTH

APPENDIX II
FIGURE 1
PROJECT T7836



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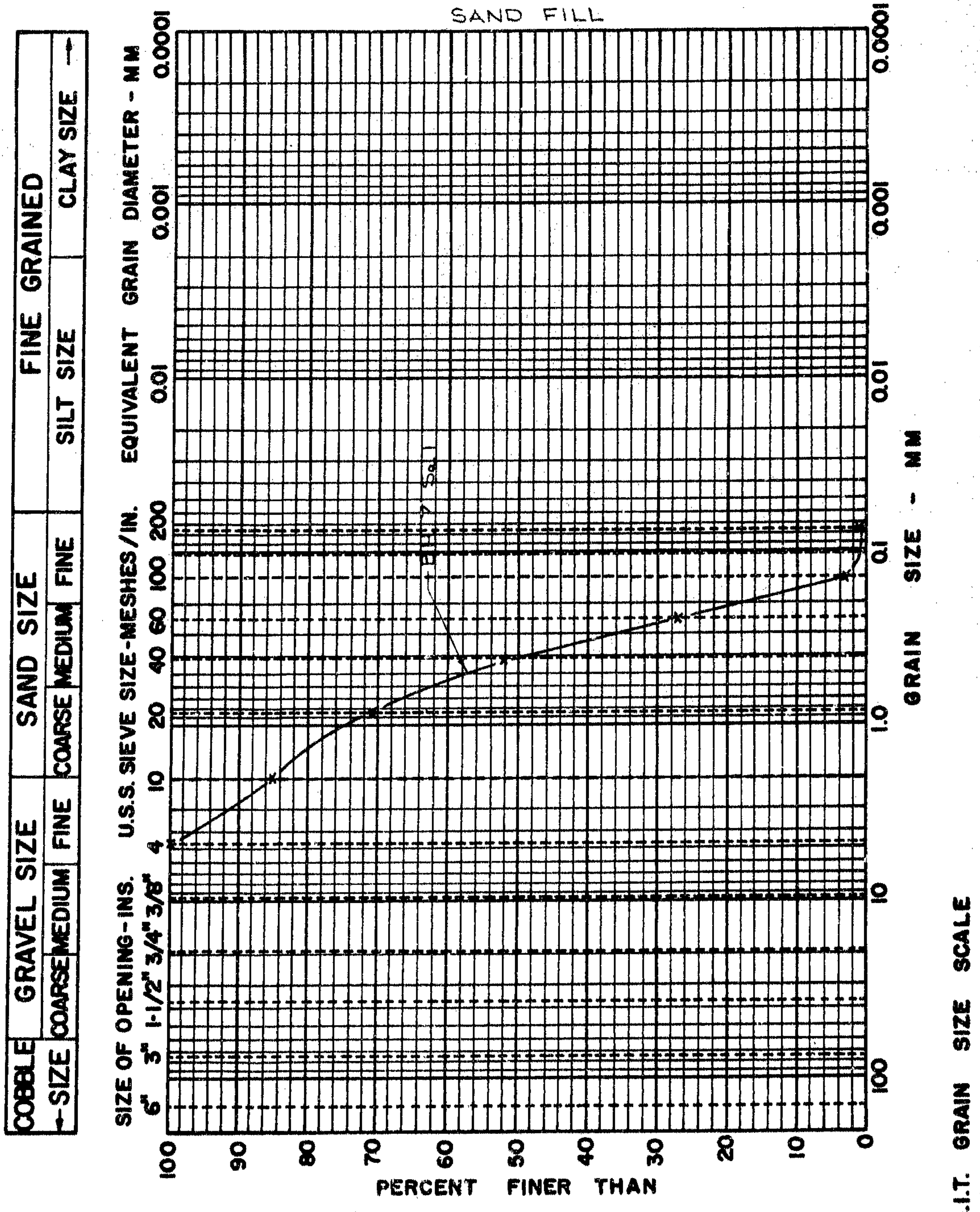
GRAIN SIZE DISTRIBUTION

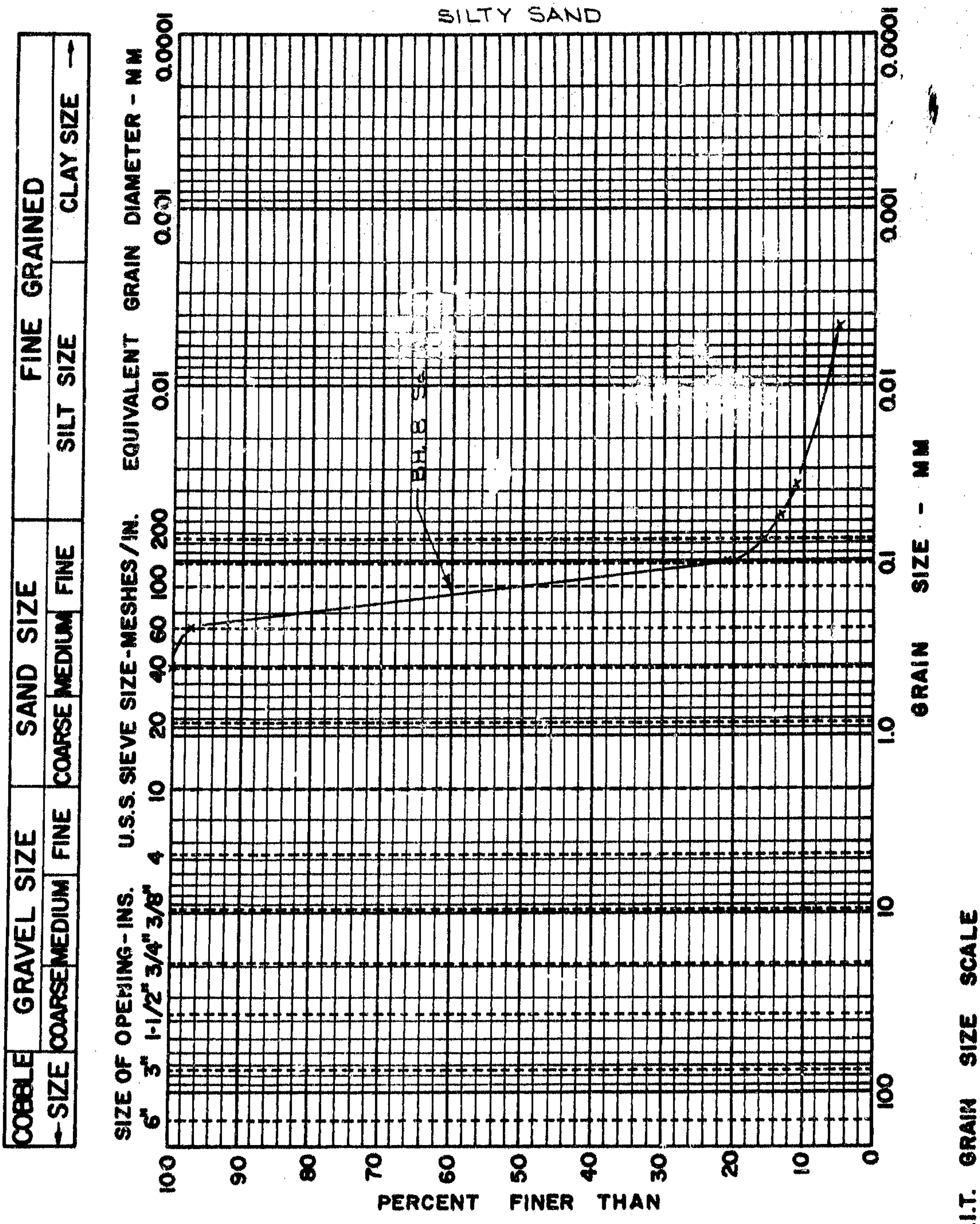
APPENDIX II
FIGURE 2
PROJECT T7836



GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 3
PROJECT T7836



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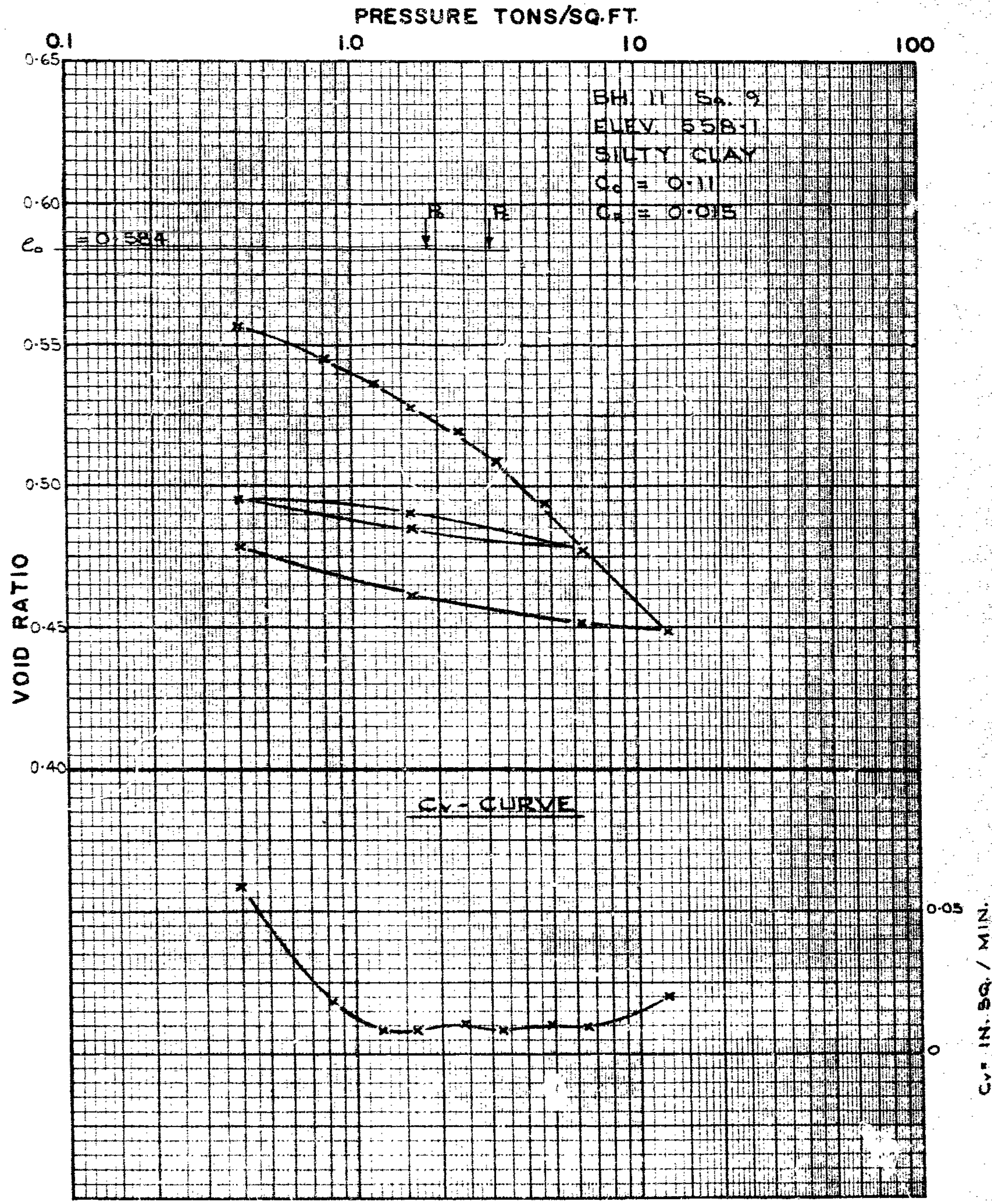
VOID RATIO-PRESSURE DIAGRAM
CONSOLIDATION TEST

APPENDIX II
FIGURE 4
PROJECT T7836



GEOCON

cc: Mr. H. Szymanski

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

December 30, 1965

Materials and Testing Division

[illegible], Limited,
[illegible] Road
[illegible]dale, Ontario.

Attention: Mr. [illegible] Gates

Re: W.P. 247-65, Site [illegible]-238,
Murphy Road Underpass,
Highway 402, District #1 (Chatham).

Dear Sir:

Please consider this your authority to carry out a foundation investigation at the above-mentioned site. One print of Drawing No. 247-[illegible]-08 was handed to your Mr. [illegible] Gates on December 29, 1965. The proposed location of the subject structure is marked in red on the drawing.

We are advised that a 6-inch watermain runs along Murphy Road and under Hwy. 402. There is also a link leading along Guthrie Drive. You are advised to check the precise location of this and other possible utilities, with the local appropriate authorities.

According to our information, the survey of this proposed interchange was started on December 13, 1965, and we believe that it could be completed by the time you start the investigation. You are requested to check this situation with Mr. Peter Peacock, Construction Engineer, District No. 1, Chatham, located at: 60 [illegible] Drive, Chatham - Phone: [illegible] 2-1400, who can also provide you with all the necessary assistance and put you in touch with the right authority regarding your possible problems.

The job is of an urgent character and we understand that you will commence the field work right after the New Year. We would appreciate receiving ten (10) copies of the final report as soon as possible, but certainly before February 11, 1966. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawing accompanying the foundation report, showing the location of borings, the inferred subsoil conditions, etc., is to become a contract drawing, you are requested to prepare it in accordance with the D.H.O. standards. To enable you to do this, we are supplying you with a sample drawing

cont'd. /2 .....

Geocon, Limited,
Attn: Mr. D. Oates.

December 30, 1965

with all the necessary explanations, together with a linen sheet for your drawing. You are also requested to provide us with a Cronaflex copy of the drawing.

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

We are attaching Purchase Order J 36203, covering the purchase of any new material required for this work, in order that you may use this as a basis for exemption from the Federal Tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

A. Rutka

A. Rutka,
MATERIALS & TESTING ENGINEER

AR:dg
Attach.

cc: Messrs. S. McCombie
A. Gater
F. C. Brown
J. Roy
H. Konings
Mrs. I. Steinberg
A. Crowley
M. Szymanski (2)
Foundations Office
Gen. Files (2)

cc: Foundations Offic (Rm. 110)

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

Attention: Mr. H. McCombie

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

February 21, 1966

FEB 21 1966

FOUNDATION INVESTIGATION REPORT BY:
Geocon, Ltd., Consulting Engineers -
Proposed Murphy Road Overpass, Hwy. 402,
Sarnia, Ontario, District #1 (Chatham).
W.P. 247-65 -- Site 14-238

Attached, please find the above mentioned report prepared and submitted by the consultant, Geocon, Ltd.

We have reviewed the report and have found the factual information adequate and well presented.

Due to the presence of a relatively thick, loose layer of granular soil which would have to be excavated, spread footings can hardly be considered as the right solution. The consultant has recommended as an alternative, steel H-piles driven to bedrock some 120 ft. below ground surface. As yet another alternative, we would recommend the use of 45-ft. long timber piles which could support a safe load of 25 tons each. These piles should have a 14-inch butt and 8 - 10-inch tip. It should be a matter of economics as to which type of piles should be chosen.

Your attention is drawn to the incorrect chainage of the Murphy Road as given in the report and the accompanying drawing. There is a 500-ft. difference - i.e., chainage 15+00 shown on the drawing should read 20+00. The error has resulted from the incorrect chainage shown on the drawing 647-B-08 which was given to the consultant prior to the commencement of the field work. It is suggested that you make the necessary corrections on the Cronaflex copy from which the prints will become part of the contract drawings. This memo accompanies all copies of the report and therefore, changes in the report and on the drawing are not necessary.

AGS/mdeF
Attach.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
A. Cator
P. C. Brown
J. Roy
A. Watt
Foundations Office
Gen. Files

Department of Highways Ontario

Copy for the information of

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

Mr. R. McIntyre,
Office Project Design Engineer,
London Regional Office,
LONDON, Ontario.

Bridge Division,
Downsview, Ontario.

February 21, 1966.

ATTENTION: Mr. M. Dodge.

W.P. 247-65, Site 14-328,
Murphy Road Underpass,
Hwy. 402 - Dist. 1.

To confirm the information given to me today, the centre line of Murphy Road crosses Hwy. 402 at Station 15 + 00 as shown on Bridge Site Plan X 4366-1.

MZ/sp

M. Zoltay,
for G. Scott,
Regional Bridge Location Eng.

cc. A. Stermac
S. McCombie
G. Scott
A. Watt

April 14/66.

Sub : W P 247-65
Murphy Rd Underpass
Hwy 402 , Dist #2
Report by Geocon

Q :- Bridge Office (Mr K. Bassi) requested the foundation section to review the subsoil conditions and advise whether a semi continious structure will be suitable or not at this location.
(End spans simply supported and Intermediate spans continious over the piers).

Ans. In our opinion the piers will ~~settle~~ have uniform settlements. Continious in that portion seems to be reasonable.
End spans being simply supported can tolerate any possible differential settlements between abutment and end pier.

M. Devata
April 14/66.

As ~~per~~ requested by Mr K. Bassi, we have reviewed the subsoil conditions and advised that 20 tons / pile ~~at~~ for #14 timber piles may be used for abutment foundations and 25 tons for pier footings. The piles will be 45 ft long.

M. Devata
July 14/66.

cc: Foundations Office (Rm. 110)

Mr. S. McCombie,
Bridge Planning Engr.,
Bridge Division.

Attention: Mr. A. P. Watt,
Regional Bridge
Location Engr.

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

May 10, 1966

Murphy Road Underpass -
Hwy. 402, District 1 (Chatham),
W.P. 247-65

We have reviewed your preliminary plan of the Murphy Road Underpass. We have no further comments pertaining to the foundations of the proposed structure.

M. Devata

M. Devata,
SENIOR FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MD/MdeP

cc: Foundations Office
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

From: Bridge Division,
Downsview, Ontario.

Date: April 29th, 1966.

Our File Ref.

In reply to:

Subject: W.P. #247-65, Bridge Site #14-328,
Murphy Road Underpass,
at Sarnia East Limits,
Hwy. #402, District #1.

Enclosed please find one copy of the preliminary plan D-5911-P1 for the above structure.

Would you kindly review the bridge foundations proposed and inform the Bridge Office if they are satisfactory.

APWatt

A.P. Watt, P. Eng.,
Regional Bridge Location Engineer.

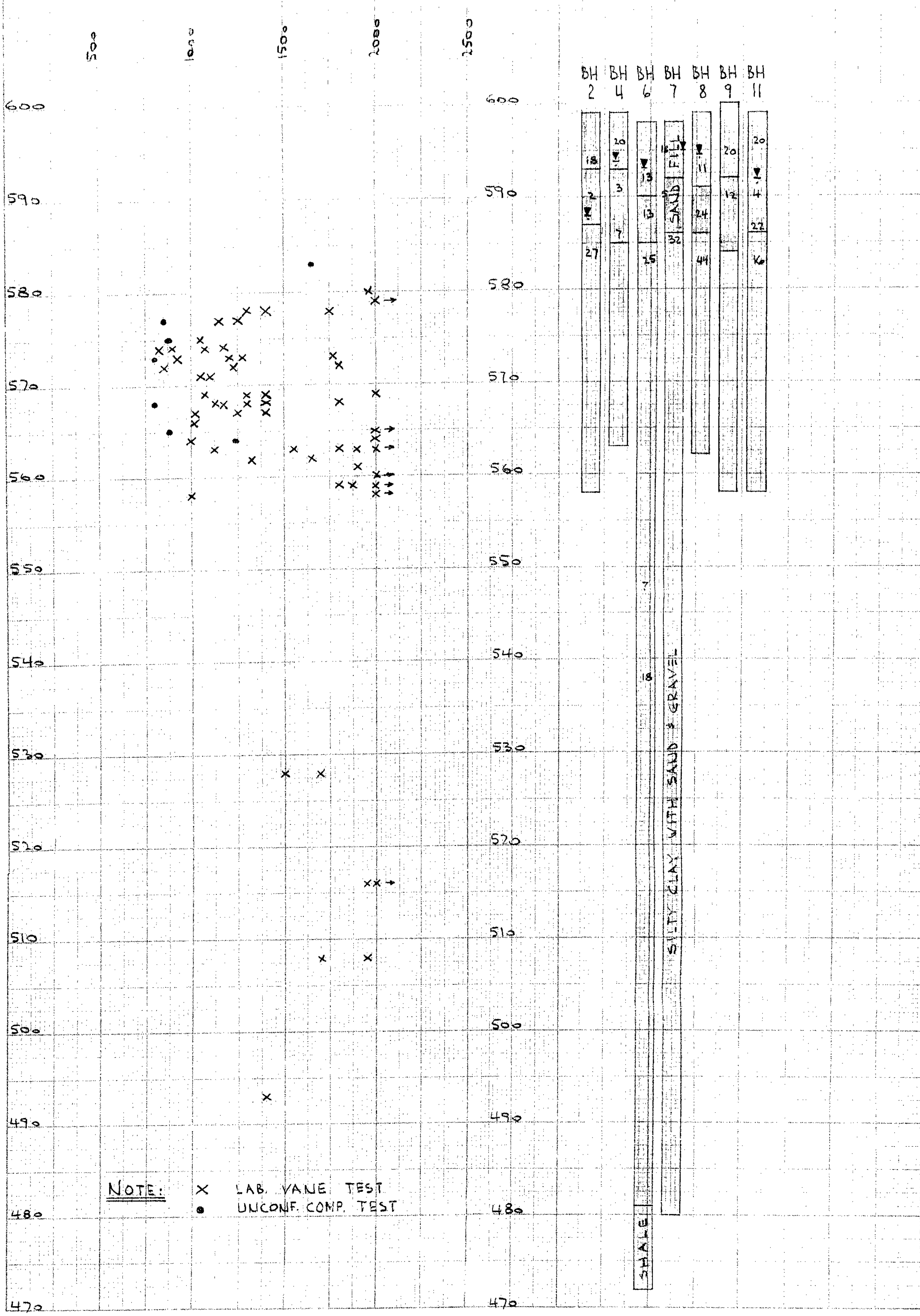
APW/cew
Encl.

W.P. 247-65 Murphy Rd O'pass
Semicontinuous structure over piers -
Simply supported end spans
45 ft long timber piles
20 T/pile at abutments
25T/pile at piers

If stage construction used and fill instrumented maybe after say 6 months design of structure could be changed to continuous throughout.

W.P. 247-65

MURPHY ROAD & HWY. 402 SARNIA



63-F-12
W.P. 53-63

C.N.R. AND HWY. 40A - SARNIA