

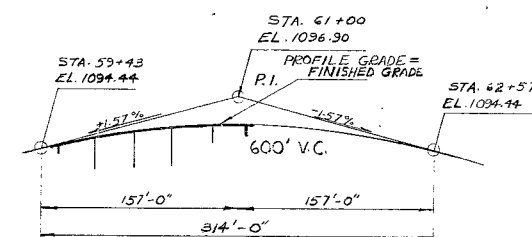
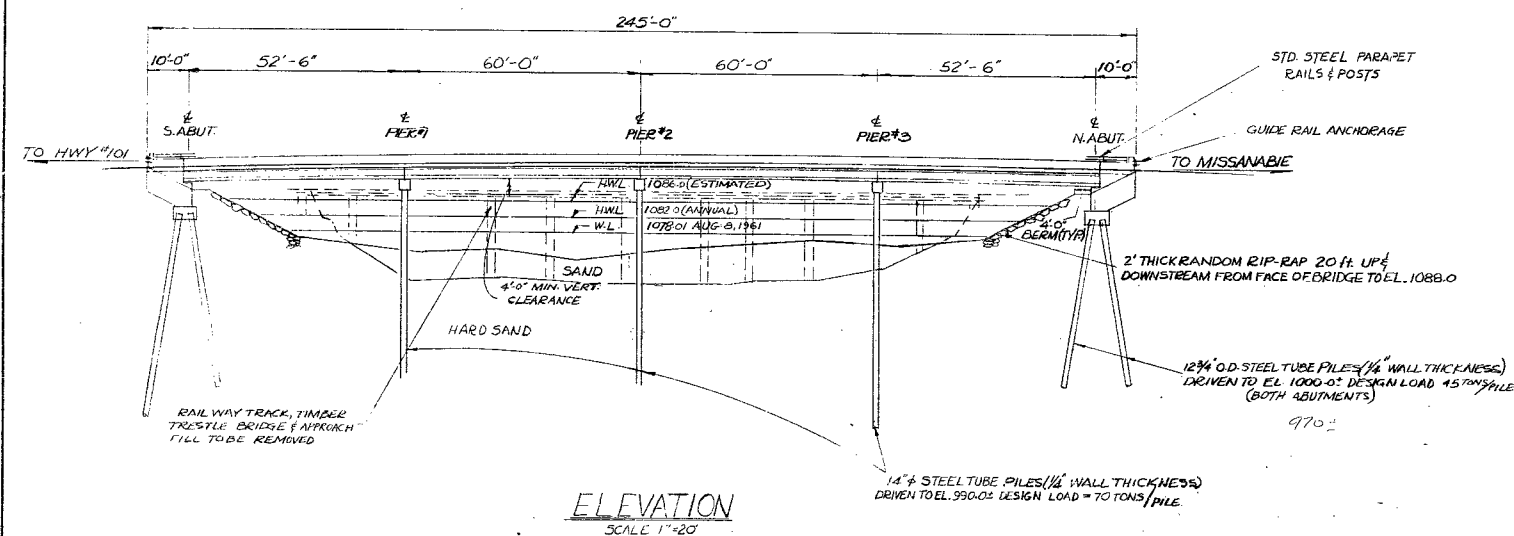
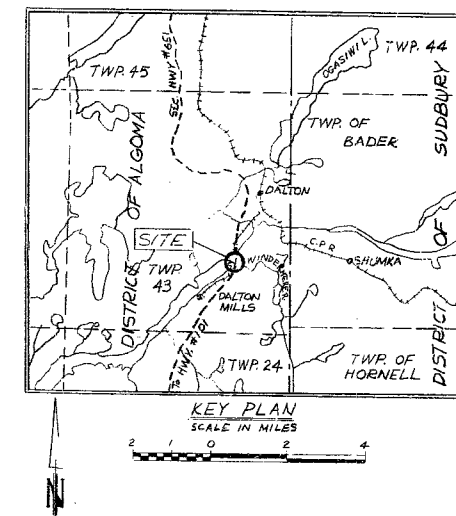
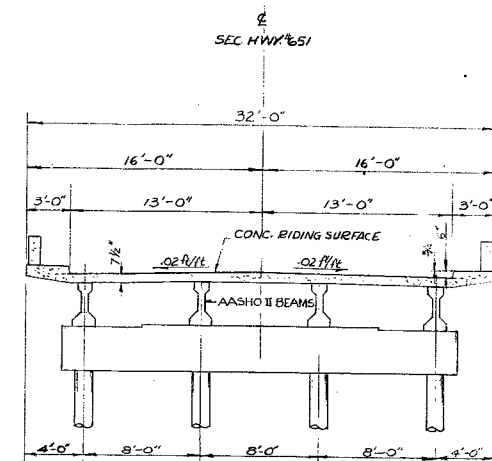
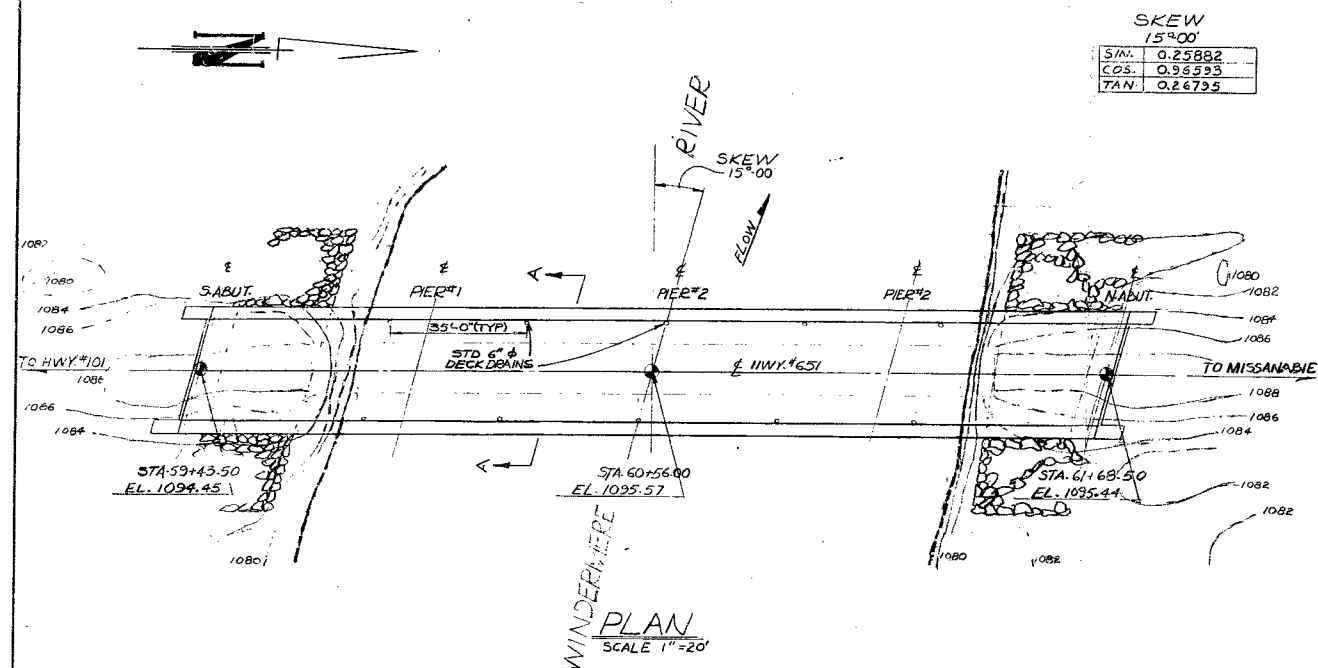
#65-F-226

W.P. #21-65

HWY #651

WINDERMERE

RIVER



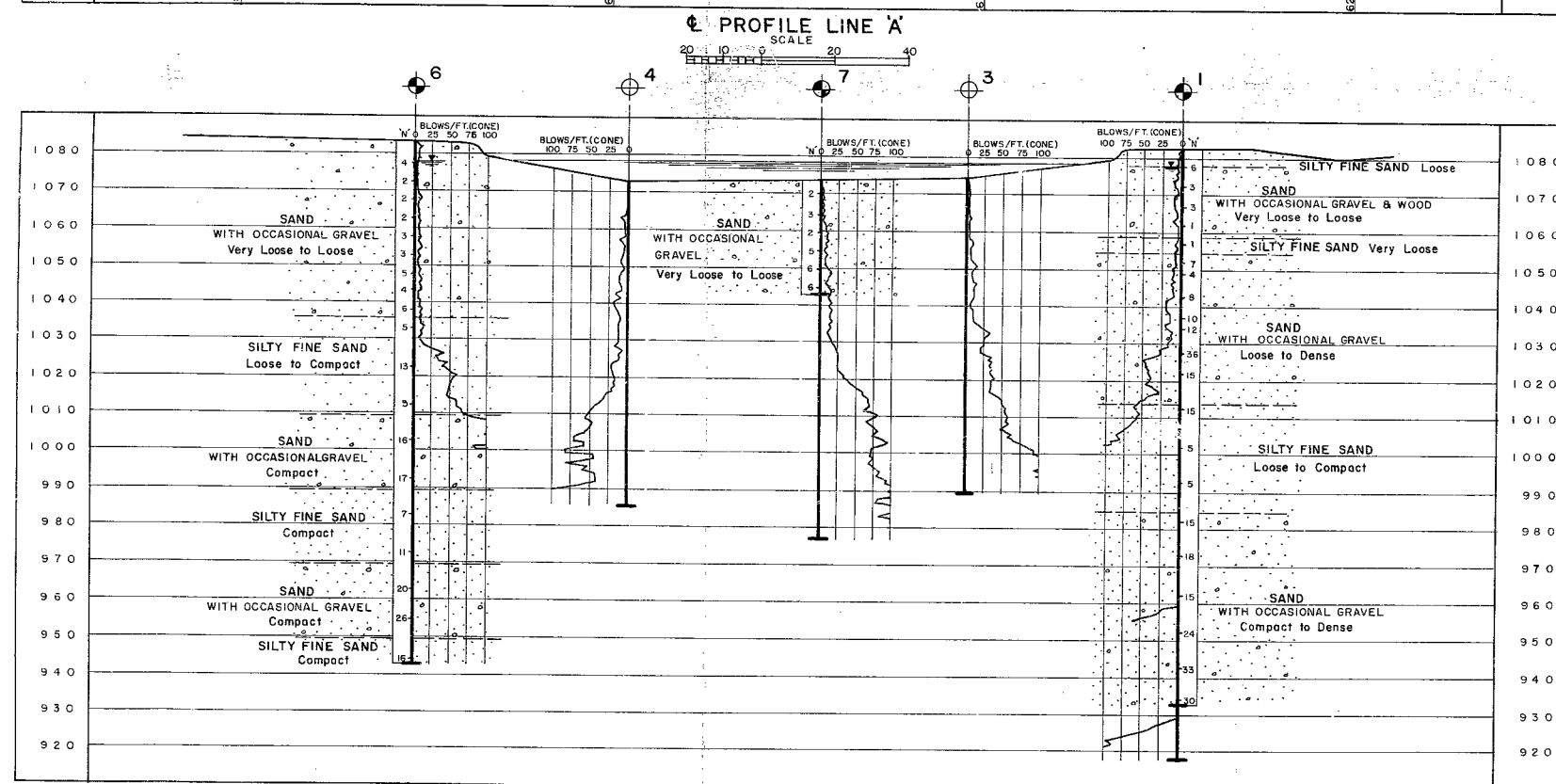
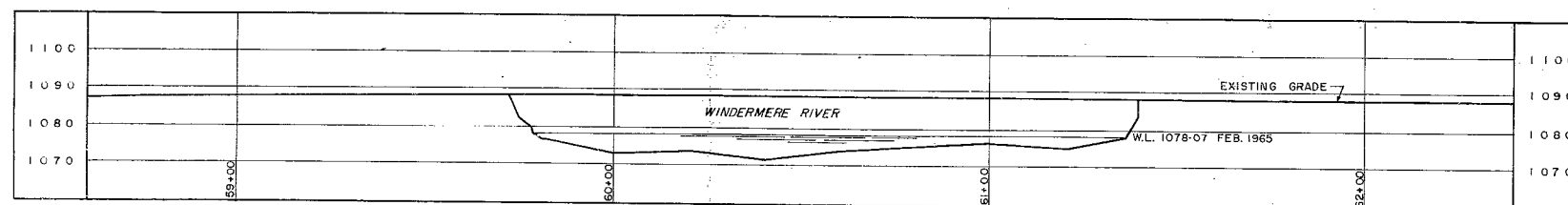
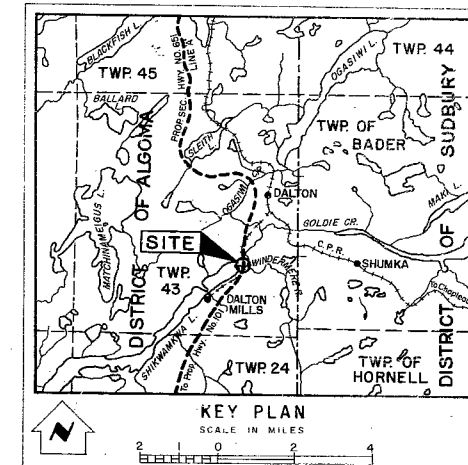
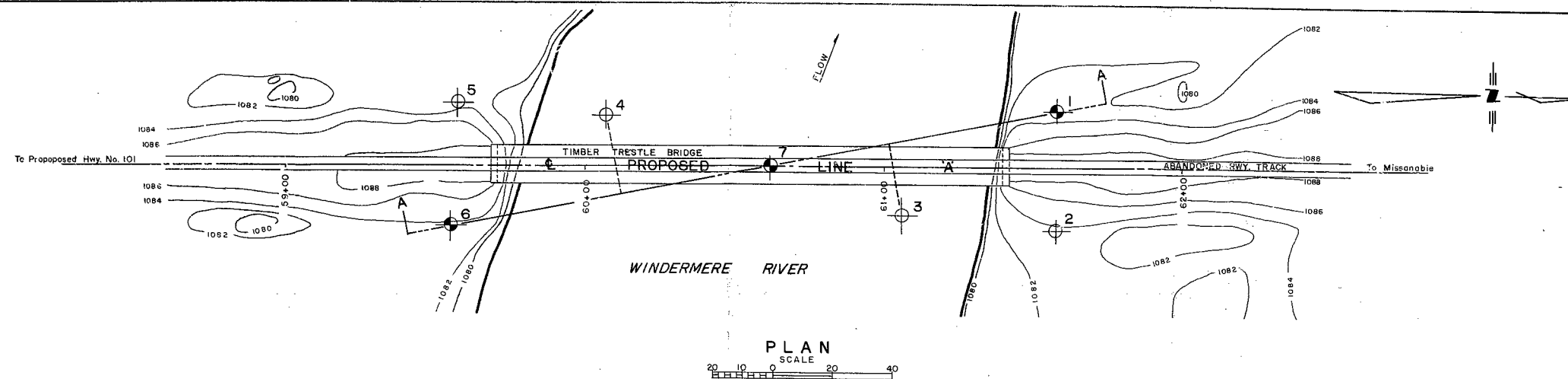
B.M. EL. 1083.28
GEODETIC DATUM
N¹/₂ W IN E. ROOT OF 1.0 PINE
51.0' LT. OF STA. 59+45

[illegible]





REVISIONS			
	DATE	BY	DESCRIPTION

<u>DEPARTMENT OF HIGHWAYS ONTARIO</u> BRIDGE DIVISION			
<u>WINDERMERE RIVER BRIDGE</u> 15.3 MILES NORTH OF HWY #101			
KING'S HIGHWAY No. <u>651</u>		DIST. No. <u>18</u>	
CO. <u>DISTRICT OF ALGOMA</u>			
TWP. <u>43</u>	LOT	CON.	
<u>PRELIMINARY</u>			
APPROVED _____ BRIDGE ENGINEER		SITE No. <u>38C-29</u> CONTRACT No. _____ DRAWING No. <u>D-5709-PI</u>	
DESIGN <u>A.R.</u> DRAWING <u>F.R.</u> DATE <u>SEPT 10 1960</u>	CHECK _____ CHECK _____ LOADING _____	W.P. No. <u>21-65</u> _____ _____ _____	

OCT 27 1966



LEGEND

 Bore Hole
 Cone Penetration Hole
 Bore & Cone Penetration Hole
 Water Levels established at time of field investigation. FEB. 1965

NO.	ELEVATION	STATION	OFFSET
1	1082.8	61+58	20' L
2	1083.2	61+58	21' R
3	1078.8*	61+06	16' R
4	1078.5*	60+07	17' L
5	1083.8	59+57	21' L
6	1083.7	59+54	20' R
7	1079.9*	60+62	ft

* ICE LEVEL

B.M. Elev. 1083.28
 Geodetic Datum
 N and W in Root of Pine

NO.	ELEVATION	STATION	OFFSET
1	1082.8	61+58	20 L
2	1083.2	61+58	21 R
3	1078.8*	61+06	16 R
4	1078.5*	60+07	17 L
5	1083.8	59+57	21 L
6	1083.7	59+54	20 R
7	1079.0*	60+62	4

-- 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.

50 TONS/PILE ABOUT 960.
52 TONS/PILE 3 PILES 900.

REVISIONS			52 TO 54	FILE 3 THERS	900
	DATE	BY	DESCRIPTION		

GEOCON LTD

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

WINDERMERE RIVER

(SEC.) HIGHWAY NO. 651 LINE 'A' DIST NO. 1
DISTRICT OF ALGOMA
TWP. 43 LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD H.L.M.	CHECKED D.R.O.	WP NO. 231-61	MAR 19 1965 T7709-1 BRIDGE DRAWING NO
DRAWN A.E.L.	CHECKED D.R.O.	JOB NO.	
DATE	MAR. 19, 1965	SITE NO	
APPROVED	<i>D.B. Dees.</i>	CONT NO	

W.P.# 21-2

[illegible]

MEMORANDUM

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

From: 208 Simpson Avenue,
FORT WILLIAM, Ontario

Date: November 9, 1964.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 231-61 Now W.P. 21-65
Site 38C-29
Windermere River Bridge
15.3 Mi. North of Hwy. 101
Sec. Rd. 651 - Dist. 18

Attached is a marked up E-plan for this structure, showing the tentative lay-out of the foundations.

Would you please arrange for a Foundation Investigation.

I have assumed that piles would be used. Should your investigation show that a different type of foundation would be more suitable, it may be necessary to alter the lay-out.

The site can be reached from Dalton via a bush-road that ends at the site. There is also an abandoned railway line from Dalton to the site, which could presumably be utilized if required.

FDeV/sp

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

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

April 1965

MEMORANDUM

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

FROM: 208 Simpson Street,
Fort William, Ontario.

DATE: November 16, 1964.

Our File Ref.

IN REPLY TO

SUBJECT: W.P. 21-65 Site 38C-29
Windermere River
Secondary Road 651
District #18

Further to my recent request for a Foundation Investigation, please be informed that the proposed elevation of the grade at the structure is 1,095.

FDeV/es

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

cc. S. McCombie

MISSALVABLE

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

Materials and Testing Division

December 15, 1964

Geocor, Limited,
14 Bass Road,
Burlington, Ontario.

Attention: Mr. H. Latas

- Re: (1) Proposed Goulais River Patrol Yard, Hwy. 532.
✓(2) W.P. 21-65, Sec. Hwy. 651, Windersore River, 15.3 Mi. North
of Hwy. 101.
(3) W.P. 22-65, Sec. Hwy. 651, Ogisiwi Creek, 16.5 Mi. North of
Hwy. 101.
--- District 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 December 15, 1964.

It is understood that a qualified Geils Engineer will be in charge of the field work at all times, and that the drill rig will be mobilized from Sudbury.

Seven copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to January 29, 1965, for the report on the Patrol Yard site, and prior to March 1, 1965, for the others. 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.M.C. 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.M.C. with Crenaflex copies of the drawings.

sent'd. /2 ...

Geoson, Limited.
Attn: Mr. D. Bates.

- 2 -

December 15, 1964

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

Yours very truly,

A. Ruths

HDS/mef

A. Ruths,
MATERIALS & TESTING ENGINEER

cc: Messrs. S. McCombie
H. McArthur
A. A. Ward
B. H. Saint
Mrs. T. Tate
N. D. Smith (2)
Foundations Office ✓
Gen. Files (2)

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

Materials and Testing Division

December 11, 1964

Dominion Soil Investigation Ltd.,
77 Crockford Boulevard,
Scarborough, Ontario.

Attention: Mr. A. Bonca.

Re: (1) Proposed Goulais River Patrol Yard, Hwy. 552.
✓(2) W.P. 21-64, Sec. Hwy. 651, Windermere River, 15.3 Mi. North
of Hwy. 101.
(3) W.P. 22-64, Sec. Hwy. 651, Opasiwi Creek, 16.5 Mi. North of
Hwy. 101.
-- District 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 December 11, 1964.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times, and that the drill rig will be mobilized from Sudbury.

Eleven copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to January 29, 1965, for the report on the Patrol Yard site, and prior to March 1, 1965, for the others. 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 ...

December 11, 1964

Charges for the work performed will be in accordance with your Schedule of Rates, dated July 6, 1964, and invoices to be addressed to the attention of the undersigned.

Yours very truly,

A. Rutka

WDS/MCEP

A. Rutka,
MATERIALS & TESTING ENGINEER

cc: Messrs. S. McCombie
H. McArthur
A. A. Ward
E. F. Saint
Mrs. I. Tate
M. D. Smith (2) ✓
Foundations Office
Gen. Files

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

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

Attention: Mr. S. McCombie

April 12, 1965

**FOUNDATION INVESTIGATION REPORT BY GECCON, LTD. -
Proposed Windermere River Bridge on Hwy. #651,
South of Sault Ste. Marie, Ontario, District #18.
W.P. 21-65**

We have reviewed the Foundation Investigation Report prepared by Geccon, Limited, for the above-mentioned job, and submit the following comments:

The Consultant has recommended piled foundations for the proposed structure and has prepared several tables showing theoretical capacities for various pile sizes. Our experience has shown that in this particular type of soil with an apparently low relative density, actual pile capacities are generally higher than the theoretical capacities. We would strongly recommend, therefore, that pile load tests be carried out in order to ascertain the most suitable type of pile and the appropriate design loads. For preliminary design purposes, we would suggest that the values from Meyerhof's equation given in the Consultant's table be used.

Your attention is drawn to the Consultant's comments pertaining to dewatering. However, since the structure will be founded on piles, we believe this to be a construction problem only. Should additional information be required regarding the above-mentioned project, please contact this office.

MD/MdeP
Attach.

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

M. Devata
for A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

Foundations Office.
Gen. Files.

cc: Foundations Office (Rm. 110)

Mr. F. DeVisser,
Regional Bridge Location Engr.,
Regional Office (North-Western
Region - Port William).

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

November 24, 1966

Windermere River Bridge,
15.3 Miles N. of Hwy. 101,
Hwy. 651, District #18,
W.P. 21-65 (Report by Geocon, Ltd.)

A meeting was held at the Bridge Office on November 14, 1966, between the members of the Foundation Section and the Bridge Design Section, to discuss the Preliminary Plan of the above mentioned structure.

It was concluded that the following recommendations should be adopted for the design of the structure foundations.

The proposed structure should be supported on large displacement (12-3/4" O.D. tubular) piles driven to estimated tip elevations 970 and 960 at the abutment and pier locations, respectively. In such a case, a safe load of 45 tons per pile for the abutments and 50 tons per pile for piers, can be used for design purposes. The pile driving during construction, should be controlled by the use of the Hiley Formula as per current D.H.O. Standards DD 1218 and DD 1219.

MD/MdeF

cc: Foundations Office
Gen. Files

M. Devata

M. Devata,
SUPERVISING FOUNDATION ENGR.

For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

*Note: The above ^{tip} elevations should be reached
Any prior stoppage of piles should
be approved by Foundations
Discussion with Gus*

Dec 19/66

AGS

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Eng.,
Laboratory Building,
Downsview, Ontario.

From: Bridge Division,
777 Memorial Avenue,
P. O. Box 1170,
Port Arthur, Ontario,
Date: November 1, 1966.

Our File Ref.

IN REPLY TO

SUBJECT:

Site 38C-29, W. P. 21-65,
Windermere River Bridge,
15.3 Miles N. of Hwy. 101,
Hwy. 651. District 18.

Attached is one print of preliminary plan for
the subject structure, for your information.

If you have any comments, please let me know
before November 11, 1966.

FDV/mcr
Enc.

F. DeVisser
F. DeVISSER,
Regional Bridge Location Eng.

Windermere River

W.P. 231-68 21-68 P. 231-68

Piers

Given: 18" ϕ Piles $\therefore A_c = 1.77 \text{ ft}^2$ $\therefore A_s = \frac{4.71 L}{50} = .0942 L$
F.S. = 3

$$q_u = 4 N_c A_c + \bar{N} A_s / 50$$

From BH #6 Geocon Ltd

	\bar{N}	$A_s / 50$
1070 - 1030	$\bar{N} = 4$	15.1 Tons
1030 - 1010	$\bar{N} = 11$	20.7 Tons
1010 - 990	$\bar{N} = 16$	30.2 Tons
990 - 970	$\bar{N} = 9$	16.9 Tons
		82.

Desire $q_{all} = 60 - 70 \text{ tons}$ say 65 tons

$$\therefore q_u = 195 \text{ tons}$$

Try pile tip between 990 & 970

Use $N_c = 11$

$$q_u = 4 \times 11 \times 1.77 + 66.0 + 9 \times .0942 L = 195 \text{ tons}$$

$$\therefore -78 - 66 + 195 = 0.848 L$$

$$\therefore L = \frac{51}{.848} = 60' \text{ No Good}$$

But at 970 $N = 20$

$$\therefore 4 N_c A_c \text{ becomes } 142 \text{ tons}$$

$$\therefore L = \frac{195 - 142}{.848} = \text{Negative}$$

\therefore Use 18" ϕ piles to 970'

$$q_{all} = \frac{142 + 83}{3} = \frac{225}{3} = 75 \text{ Tons}$$

$$[q_{all} (\text{basis of BH \#1}) = 62.4 \text{ Tons}] \text{ See P.2}$$

②

WP 231-61

Abutments

Given: $12\frac{3}{4}" \text{ } \phi \text{ Piles}$ $\therefore A_c = 0.885 \text{ } \phi$ $A_{s/50} = 0.0668 L \text{ } \phi$
 $F.S. = 3$
 $q_u = 4 N_c A_c + \bar{N} A_{s/50}$

From BH #6 Geaccon Ltd.

Layer	\bar{N}	$\frac{\bar{N} A_s}{50}$	$4 N_c A_c$
1080 - 1030	4	13.4	
1030 - 1010	11	14.7	
1010 - 990	16	21.4	56.6
990 - 970	9	12.0	38.9
		61.5	

Desire $q_{all} = 40 \text{ to } 45 \text{ Tons}$ Say 42.5 $\therefore q_u = 127.5$

\therefore Pile tip above 970 $q_u \approx 100.4$

Pile tip at 970 $N = 20$ $\therefore q_u = 132.4 \text{ Tons}$

\therefore Use $12\frac{3}{4}" \text{ } \phi$ to 970'

$q_{all} = 44 \text{ Tons}$

Check on basis of BH #1

Abutment - $q_u = 56.6 + 10.7 + 26.0 + 8.3 + 15.0 = 116.6$ $q_{all} = 38.9 \text{ Tons}$

Pier - $q_u = 113.3 + 15.1 + 36.7 + 11.8 + 21.2 = 208.1$ $q_{all} = 69.4 \text{ Tons}$

GEOCON LTD

HEAD OFFICE

420 MICHEL JASMIN, DORVAL, QUEBEC

TELEPHONE 631-9827

Rexdale, Ontario,
March 29th, 1965.

DISTRICT OFFICES

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

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

Department of Highways, Ontario,
Downsview, Ontario.

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

12 66 Re: Soil Conditions and Foundations
Proposed Windermere River Bridge
Sec. Highway 651- W.P. 21-65
Sault Ste. Marie, Ontario

Dear Sirs:

This letter accompanies our detailed report on the
above investigation.

We find that the overburden consists of at least 165
feet of very loose to dense silty fine sand and sand with gravel. In
general, the upper 30 feet of the deposit is very loose. The soil
conditions encountered are described in detail in the report.

In view of the generally very loose nature of the
upper 30 feet of the overburden, spread footings are not considered
a practical foundation solution for the proposed bridge. The most
suitable foundation is believed to be the use of piles of either the
driven friction or cast-in-place pedestal type, as discussed in the
report. Recommendations covering pile design and foundation and
embankment construction are given in the report.

We believe that this report presents all the information
required from this investigation. Should you have any questions or
if we can be of further assistance otherwise, please do not hesitate to
call us.

Yours very truly,
GEOCON LTD

M. A. J. Matich per D. J. O.

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

MAJM/reb

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INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter dated December 15th, 1964, Work Permit No. 22-65 to carry out a foundation investigation for the proposed crossing at Windermere River, approximately 2 miles south-west of Dalton, Ontario.

The purpose of the investigation was to determine the soil conditions at the site of the proposed crossing as required for the design of the proposed structure .

SUMMARIZED SOIL CONDITIONS

The site is underlain by an extensive deposit of granular material having a thickness in excess of 165 feet. The deposit consists of alternate layers of silty fine sand and sand having occasional gravel. In general, the upper few feet of this deposit contains organic traces, particularly within the river. The upper 30 feet of the deposit has a very loose relative density. The relative density below 30 feet ranges generally from loose to compact with depth.

At the time of the investigation, the water level in the river and the adjacent river banks was at about elevation 1078. At a depth of about 90 feet a slight artesian pressure, 2 feet above normal water level, was encountered towards the base of a silty

GEOCON

fine sand layer.

DISCUSSION

General

It is understood that it is proposed to construct a bridge over the Windermere River at the site investigated. As presently planned the bridge will consist of four spans located on Line A, as shown on Department of Highways, Ontario Drawing No. E-4033-1. It is further understood that the proposed elevation of the grade at the structure will be 1095.

Foundations

The soil conditions at the site consist of a deep granular deposit consisting of silty fine sand and sand with occasional gravel. The relative density of the deposit ranges generally from very loose to compact with depth, with the upper 30 feet being very loose and containing at the surface, organic material.

Because of the very loose relative density of the surface strata and the fact that the soil is scour susceptible, the site is not considered suitable for the practical use of spread footings for the foundations of the proposed structure. The most suitable foundation solution appears therefore to be the use of piles.

Foundations (continued)

In view of the excessive depth to bedrock, the most suitable pile type would be a displacement type which derives its support partly by side friction and partly by point resistance within the granular overburden. Suitable displacement pile types would be treated timber piles, steel pipe piles driven closed end or pre-cast or cast-in-place concrete piles. Irrespective of the pile type chosen, it is recommended that a pile test be performed on a representative pile to verify the pile design working load. Further, all pile caps subject to frost action should be provided with a minimum of 6 feet of protective earth cover.

For initial selection of allowable pile diameters and lengths, preliminary design computations have been carried out for straight shaft circular piles of different diameters, using three approaches. (Meherhof 1956.¹ "Penetration Tests and Bearing Capacity of Cohesionless Soils." Journal of the Soil Mechanics and Foundation Division A.S.C.E. No. SMI-Mansur and Kaufman 1956.² "Pile Tests, Low Sill Structure, Old River, La." Journal of the Soil Mechanics and Foundation Division, A.S.C.E. No. SM4 -Terzaghi and Peck.³ "Soil Mechanics in Engineering Practice"). The results incorporating a computed factor of safety of 2.5 are given on Figures 4, 5 and 6 in Appendix II. The values indicated

Foundations (continued)

are for individual piles. In no case, should the working load of the pile exceed the safe load as determined from the structural properties of the pile material. As can be seen from the computations the three methods give widely varied results for a given pile diameter and length and as such indicate clearly the advisability of performing a load test to verify the pile design working load assumed in design.

A number of factors are relevant to choosing which of the above methods is more suited to the soil conditions at the site. The approach of Mansur and Kaufman was developed from a series of pile load tests in sand having a relative density similar to the overburden at this site below elevation 1040 and as such the estimate of the resistance developed in skin friction in the overburden above elevation 1040 is probably too high using their approach. A feature of such an empirical approach is that there is no standard by which an adjustment may be made to allow for changes in the soil conditions. The method suggested by Meyerhof relies entirely on "N" values which can be significantly affected by changes in composition of the overburden and by slight disturbance at the bottom of the borehole particularly if slight artesian pressure exists within the overburden. While the dynamic penetration tests indicate clearly

Foundations (continued)

the low relative density of the overburden, the "N" values recorded are probably on the low side. It is suggested therefore that for preliminary design the values given by Terzaghi and Peck be used and confirmed by a load test as discussed, which will also reflect slight artesian condition encountered at depth. The load test results should be adjusted if necessary for group action in foundation design.

In view of the generally granular nature of the overburden, consideration could be given to the use of compacted cast-in-place pedestal piles with bases formed in the sand and gravel strata.

The overburden at the site is susceptible to scour. Therefore, bridge foundations should be protected against undermining by such means as provision of rip-rap protection or extending piles a sufficient distance below maximum possible scour depth. The extent of scour depends on the maximum flood level in the river, the hydraulics of the river channel in the vicinity of the bridge, pier orientation and the like. The hydraulic considerations involved are beyond the scope of this report. Published data (Ref. 3), indicates that the depth of scour below the low water channel may be of the order of four times the greatest known rise of river level. Therefore, if no suitable rip-rap protection is provided, to the piers, it should be assumed that scouring to at least the depth indicated may occur. The computed pile capacities as given above assume that adequate surface rip-rap will be provided.

Approach Embankments

It is recommended that the surface organic material be stripped from beneath approach embankments. With a proposed grade at 1095, the overall height of embankment relative to adjacent ground level will be about 12 feet. In view of the granular nature of the subsoil, it is considered that the stability of the approach embankments with side slopes of 1 vertical to 1.5 horizontal or flatter will be adequate. However, it is recommended that rip-rap be provided where necessary to above maximum high river level to prevent scour.

It is recommended that the backfill to abutments consist of well compacted free draining non-frost susceptible clean granular material. With this provision, a coefficient of lateral earth pressure of 0.5 is recommended for the case of pile supported abutments.

Construction

Should excavations be required below water level at the pier or abutment locations, they will involve excavation within the very loose sand or silty sand and therefore, some means will be required to control water inflow. For this purpose a "sanded-in" well point system could be used or alternatively the excavation could be sheeted and dewatered by pumping from filter equipped sumps maintained ahead of excavation. Where excavations are required within

Construction (continued)

the river berms built to above river level would be required if well points only are used. Similar berms might also facilitate construction if sheeting was used. Sheeting, if used, should be carried down below excavation level a distance at least equal to the maximum head differential likely to be encountered. Irrespective of the method used, construction should be carried out in sections to minimize constriction of river flow.

CONCLUSIONS AND RECOMMENDATIONS

- 1) The site is covered by an extensive deposit of silty fine sand and sand with occasional gravel. The maximum depth of this deposit as encountered was 165 feet. The relative density ranged generally from very loose to compact with depth.
- 2) At the time of investigation the water level in the river and the adjacent river bank was at about elevation 1078. Slight artesian pressure, 2 feet above normal water level, was encountered at depth within the overburden.
- 3) The low relative density and scour susceptibility of the surficial soil strata at the site preclude the practical use of shallow spread footings. The most suitable foundation solution therefore, is considered to be the use of friction or pedestal piles as indicated

in the report. Computed pile loads are given as a guide to preliminary design, and it is recommended that the design load be established by test loading representative piles with adjustment for group effect if necessary.

4. Construction of piers and pile caps will probably require excavation below water level. Recommendations are given on possible measures to handle unwatering of such excavations.

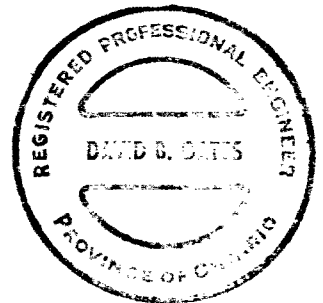
PERSONNEL

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

DBO/reb

D. B. Oates

D. B. Oates, P. Eng.,
District Soils Engineer.



APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL CONDITIONS

WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

GEOCON

PROCEDURE

The field work for this investigation was carried out between January 23rd and February 28th, 1965. As discussed later, the site is located about 2 miles south-west of Dalton, Ontario and about 1 mile, south-west of the investigation carried out for the crossing over Ogasiwi Creek. The work for this investigation was carried out immediately after the work at the Ogasiwi Creek was finished. At the time of investigation there was no suitable small bulldozer available in Dalton or Missanabie. In any event, the bridge between the two sites was not capable of supporting heavy equipment and in fact some temporary support was necessary to support the drill rig. The move from Ogasiwi Creek to Windermere River was therefore made using two horses. Because of the deep snow cover of about 4 feet and the steep hills between the two sites, 7 days were required to break a road and move all the equipment into the site. For comparison, only about 3 days were necessary to bring the equipment out to Dalton. Transport from Dalton to the site for the field crew was by Skidoo.

Three boreholes, with adjacent dynamic penetration tests, were put down using a standard skid mounted machine drill rig to depths of 37, 141 and 150 feet. The deepest borehole was extended to a depth of about 165 feet by a penetration test carried out in the bottom of the hole. An additional 4 dynamic penetration test were also put down using the same equipment.

GEOCON

Sampling of the overburden was carried out at 5 foot intervals using a two inch split spoon with a foot valve to assist recovery. Because of the generally loose relative density of the strata, recovery was not always achieved with the split spoon. As a result, slotted tube samples were generally obtained after the first attempt at recovery with the split spoon.

A complete log of each borehole and penetration test is given on the Office Reports on Soil Exploration in this Appendix. The locations of the boreholes together with the inferred soil stratigraphy are shown on Drawing T7709-1, located in the pocket at the rear of this report.

The results of the laboratory testing are shown on the Figures in Appendix II. All samples remaining after testing will be stored until April 1st, 1966 at which time you will be contacted for instructions regarding their disposal.

All elevations are referred to Geodetic datum. The location of the bench mark used is shown on Department of Highways, Ontario Drawing No. E-4033-1. The bench mark has an elevation of 1083.28 and is a nail and washer in the east root of a 1 foot diameter Pine Tree located about 51 feet left of station 59+45.

The site is located about 2 miles south-west of Dalton, Ontario. Access to the site is by a gravel road which terminates at the site. The new structure will replace the existing four span timber trestle which crosses the Windermere River and which forms part of an abandoned railway link between Dalton and Dalton Mills. The river in the area of the bridge was about 160 feet wide at the time of investigation; ground level is generally flat lying between elevations 1080 and 1084, with the existing approach embankments at about elevation 1088.

At the time of investigation the river level was at about elevation 1078. Information taken from the Department of Highways, Ontario Drawing No. E-4033-1 indicates that high river level every 5 years is at about elevation 1086 at which time the surrounding ground will be submerged.

From available geological information and work in the general area, it is known that the overburden cover is composed of sands and silts of post glacial fluvial origin. Previous work in the general area indicates that the bedrock is an igneous intrusive grey to pink granite of the Archean Era of Precambrian Period.

The principal soil strata encountered in the boreholes are described below. Because of the difficulty of obtaining relatively undisturbed samples of the overburden and also its variable composition, the boundaries and descriptions of the various strata are probably idealized. Because of the method of deposition, a more detailed definition of continuous layers was not possible and in any event would probably not be realistic. It is believed however, that the inferred strata described below and on Drawing T7709-1 are for all practical purposes representative of the stratigraphy.

Topsoil

Boreholes 1 and 6, put down on the north and south sides of the river respectively, encountered about 6 inches of topsoil. The topsoil was observed to consist of sand with a high root and organic content.

Very Loose to Compact Brown and Grey Silty Fine Sand

Underlying the topsoil in borehole 1, is a 5 feet thickness of brown silty fine sand. This surficial layer was not encountered in either of the other two boreholes, at the surface. However, three silty fine sand layers were encountered at depth within boreholes 1 and 6. The upper layer ranged in thickness in boreholes 1 and 6 from 4 to 26 feet, respectively, and was encountered at depths below ground level of 24 and 48 feet, respectively. Borehole 7 was terminated at a

Very Loose to Compact Brown and Grey Silty Fine Sand (continued)

depth of about 37 feet and as such was probably not deep enough to indicate conclusively whether this layer was continuous. The middle layer ranged in thickness in the above boreholes from 29 to 20 feet, respectively, and was encountered at depths of 69 and 94 feet, respectively. Slight artesian pressure was encountered below this layer in borehole 1, as discussed later. The presence of the lower silty fine sand layer was inferred by the results of the penetration test from the bottom of borehole 1; borehole 6 was terminated within this layer. Penetration of this layer in each borehole was about 4 to 6 feet.

Mechanical analysis tests were carried out on samples obtained from this stratum in borehole 1. One sample was taken from the surficial layer, encountered only in borehole 1, and the remaining samples were taken from within the middle layer described above. The results are plotted on Figure 1 in Appendix II. The samples contained 33 to 47 percent silt sizes, with the remainder of the grading being in the fine sand range.

Standard penetration tests carried out in the above layers gave "N" values ranging from 1 to 15 blows per foot. Average "N" values for the three main layers described above, were 7, 9 and 16 blows per foot. The relative density ranges from very loose to compact and is generally loose to compact.

Very Loose to Loose Brown to Grey Sand with Organic and Wood Traces and Occasional Gravel

Underlying the silty fine sand in borehole 1 and the river bottom in borehole 7, is a stratum which consists generally of sand. At various depths within the stratum the gradation includes gravel sizes, usually in the fine gravel range. This layer is identical in its general composition to the layers encountered at depth within the overburden and which are described below. However, it is described separately herein because of its occasional wood and organic content. The organic content is confined generally to the borehole put down in the river where 6.5 feet of brown sand with organic traces was encountered. Borehole 1, however, encountered wood above a depth of 14 feet below ground level. The colour change from brown to grey occurred within this stratum in borehole 1 at a depth below ground level of about 8 feet. The presence of a distinct colour boundary indicates that the wood was probably not deposited during a process of scour in recent times as was initially considered to be possible.

A mechanical analysis was carried out on a sample from this stratum and the results plotted on Figure 2 in Appendix II. The sample contained 26 percent gravel sizes and 74 percent sand sizes.

Standard penetration tests carried out in this stratum gave "N" values of 2 and 3, indicating a very loose relative density.

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Very Loose to Dense Brown to Grey Sand with
Occasional Gravel

The layer defined separately above because of its organic content forms part of a deeper deposit of sand. Within the depth encountered, this extensive deposit of sand is separated into three layers by silty fine sand layers. The layers ranged individually from 10 to 48 feet, 41 to 20 feet and 63 to 20 feet in boreholes 1 and 6, respectively. Borehole 7 was terminated within the upper sand layer at a depth below river bottom of about 32 feet.

Mechanical analysis tests were carried out on samples from this stratum and the results plotted on Figure 3 in Appendix II. The results reflect the variable composition of these layers. The samples ranged from a gravelly sand which contained 40 percent fine gravel sizes, 40 percent coarse sand sizes, 10 percent medium sand sizes and 10 percent fine sand and silt sizes to a fine sand containing 16 percent medium sand sizes and 14 percent silt sizes.

Standard penetration tests carried out in the above layers gave "N" values ranging from 1 to 36. Average "N" values for the three layers were 4, 14, 23 respectively with depth.

WATER CONDITIONS

VIII

At the time of the investigation the water level in the river was at about elevation 1079. Observations of the ground water level in boreholes 1 and 6 confirm this elevation. However, when borehole 1 was at a depth of 90, that is close to the base of a silty fine sand layer, slight artesian pressure was encountered. The artesian head equalized in the casing about 2 feet above normal water level.

Information from the Department of Highways, Ontario Drawing E-4033-1 indicates that the high annual river level is at elevation 1082 and that high river level, every 5 years, is at elevation 1086.

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

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

ELEVATION AND DEPTH

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

WATER CONDITIONS

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

DESCRIPTION

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

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

STRATIGRAPHIC PLOT

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

ELEVATION SCALE

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

GRAPHS

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

OTHER TESTS

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

SAMPLES

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

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

GEOCON

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7709 BORING # 1 DATUM GEODETIC CASING BX.
 BORING DATE FEB. 1-10/65 REPORT DATE MAR. 3, 1965 COMPILED BY AEL. CHECKED BY _____
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



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

SAMPLE TYPES

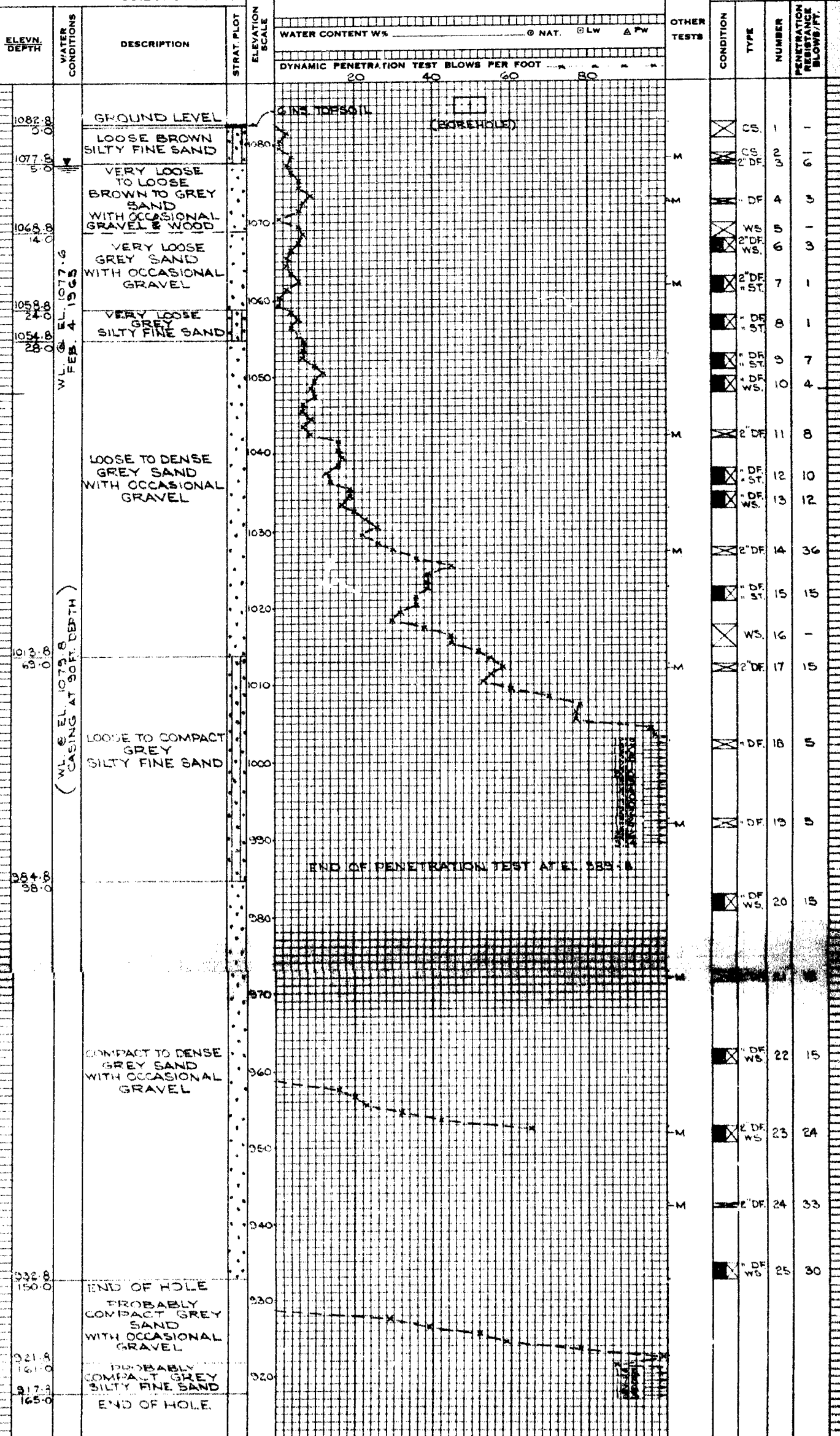
F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

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

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7709 PEN. TEST 2 DATUM GEODETIC CASING ---
 BORING DATE FEB 17 / 65 REPORT DATE MAR. 2, 1965 COMPILED BY AEL CHECKED BY ---
 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

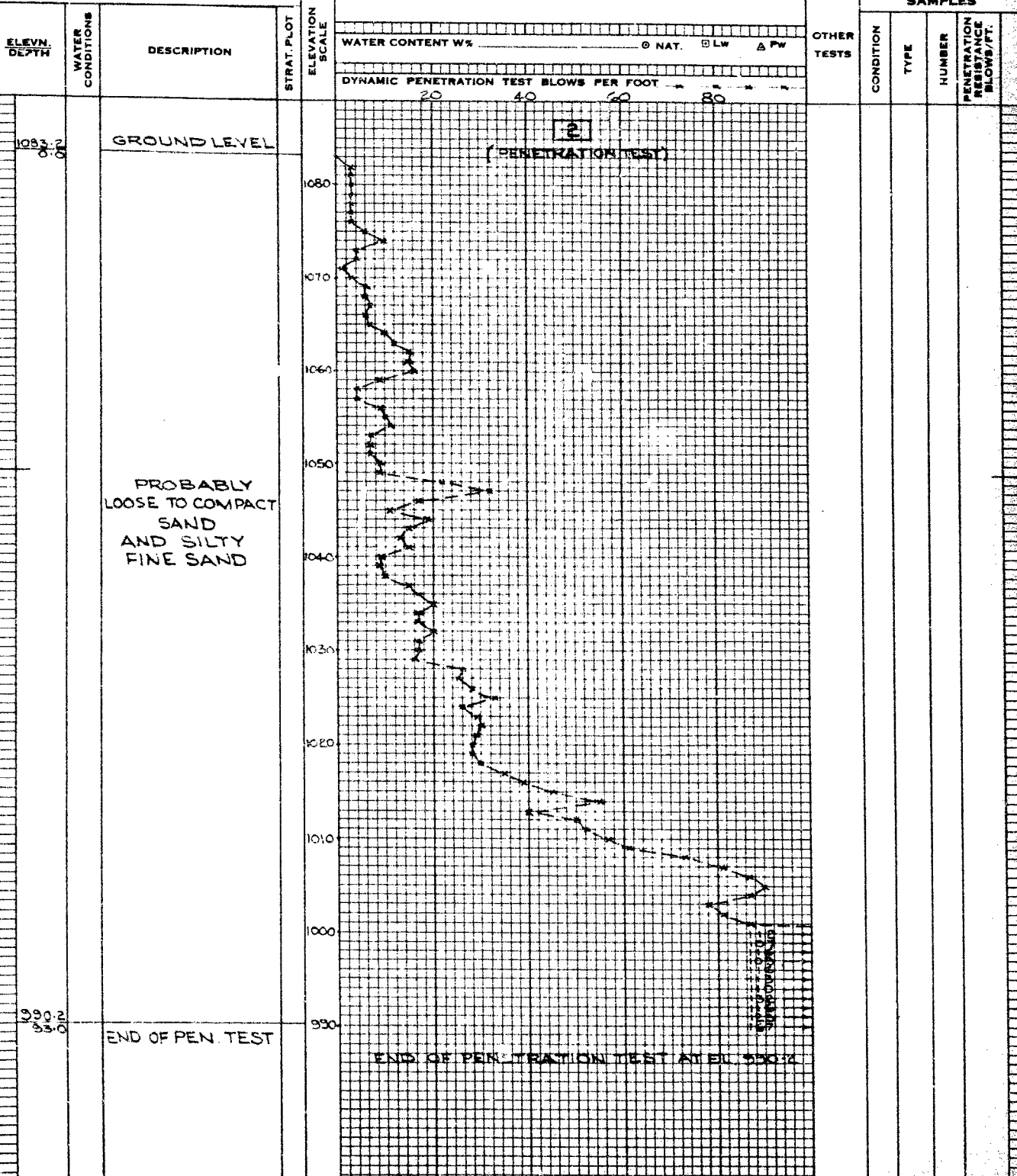
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 L.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

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



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT I7709 PEN. TEST 3 DATUM GEODETIC CASING ---
 BORING DATE FEB. 18, 1965 REPORT DATE MAR. 11, 1965 COMPILED BY A.E.I. CHECKED BY ---
 HAMMER WT. 140 LBS. CROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY)

SAMPLE CONDITION



SAMPLE TYPES

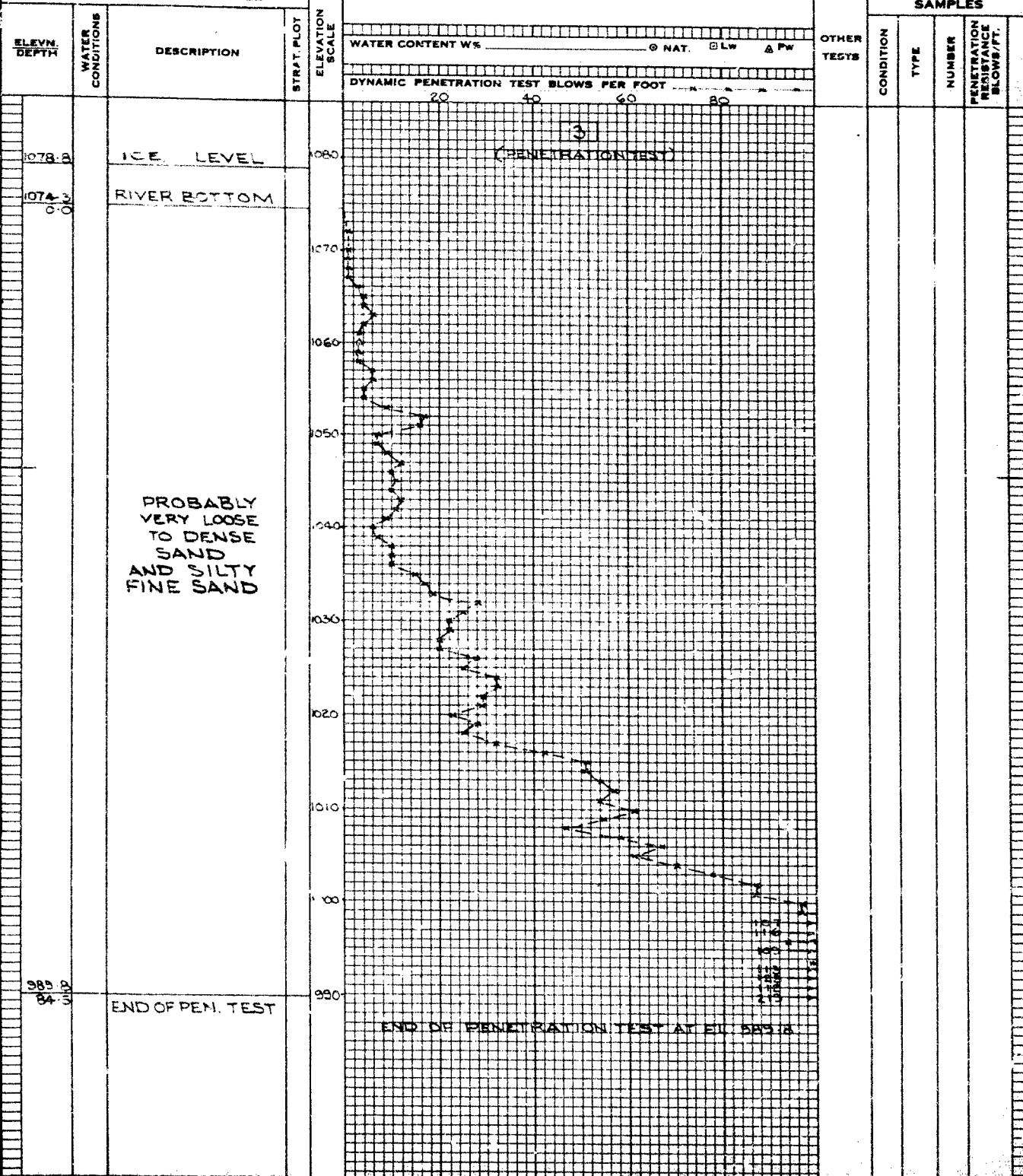
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

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

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7709 PEN. TEST 4 DATUM GEODETIC CASING ---
 BORING DATE FEB 18, 1965 REPORT DATE MAR 11, 1965 COMPILED BY A.E.L. CHECKED BY ---
 HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

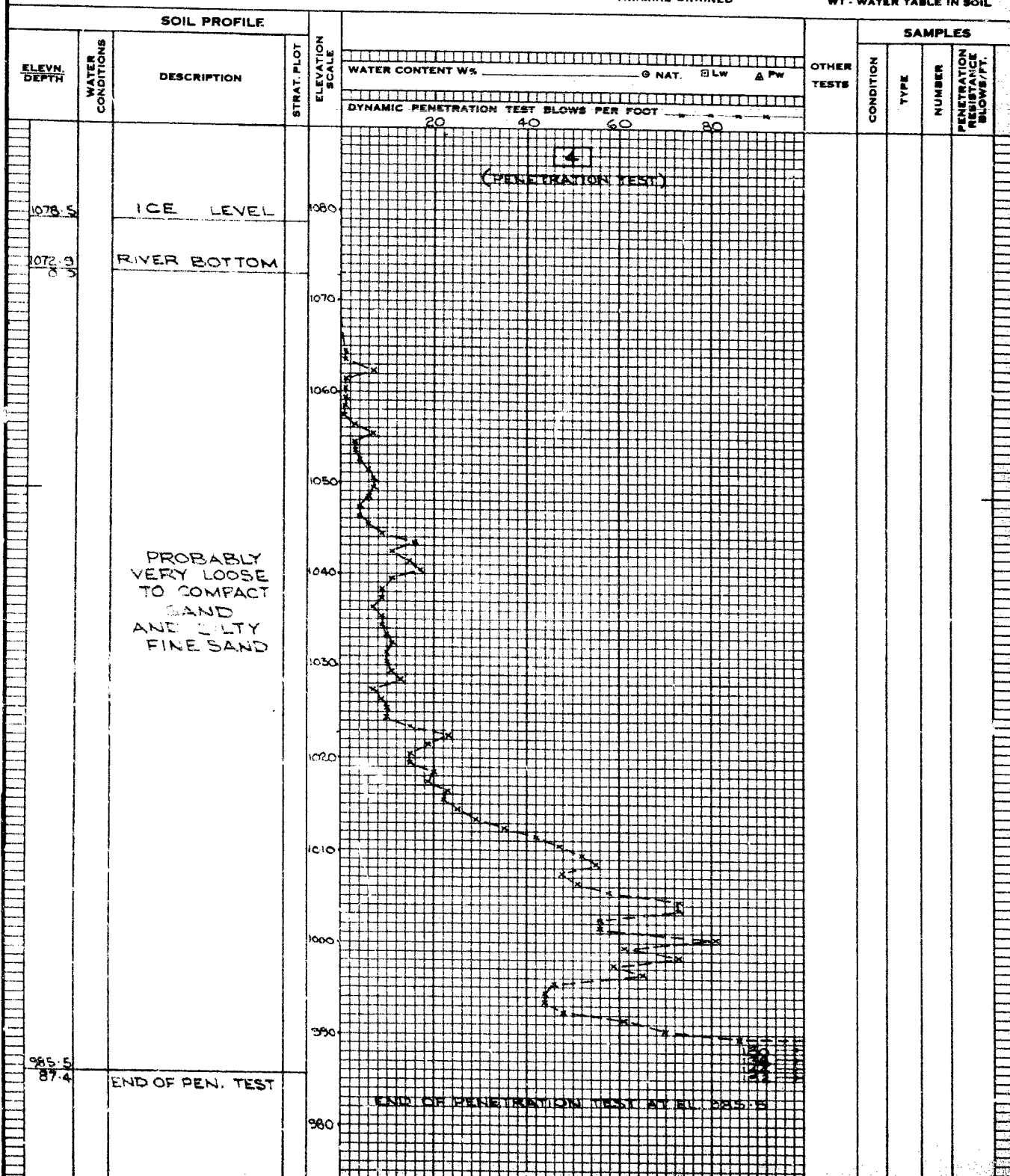
SAMPLE TYPES

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



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7709 PEN. TEST 5 DATUM GEODETIC CASING ---
 BORING D. YE FEB. 18, 1965 REPORT DATE MAR. 11, 1965 COMPILED BY AEL CHECKED BY ---
 HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

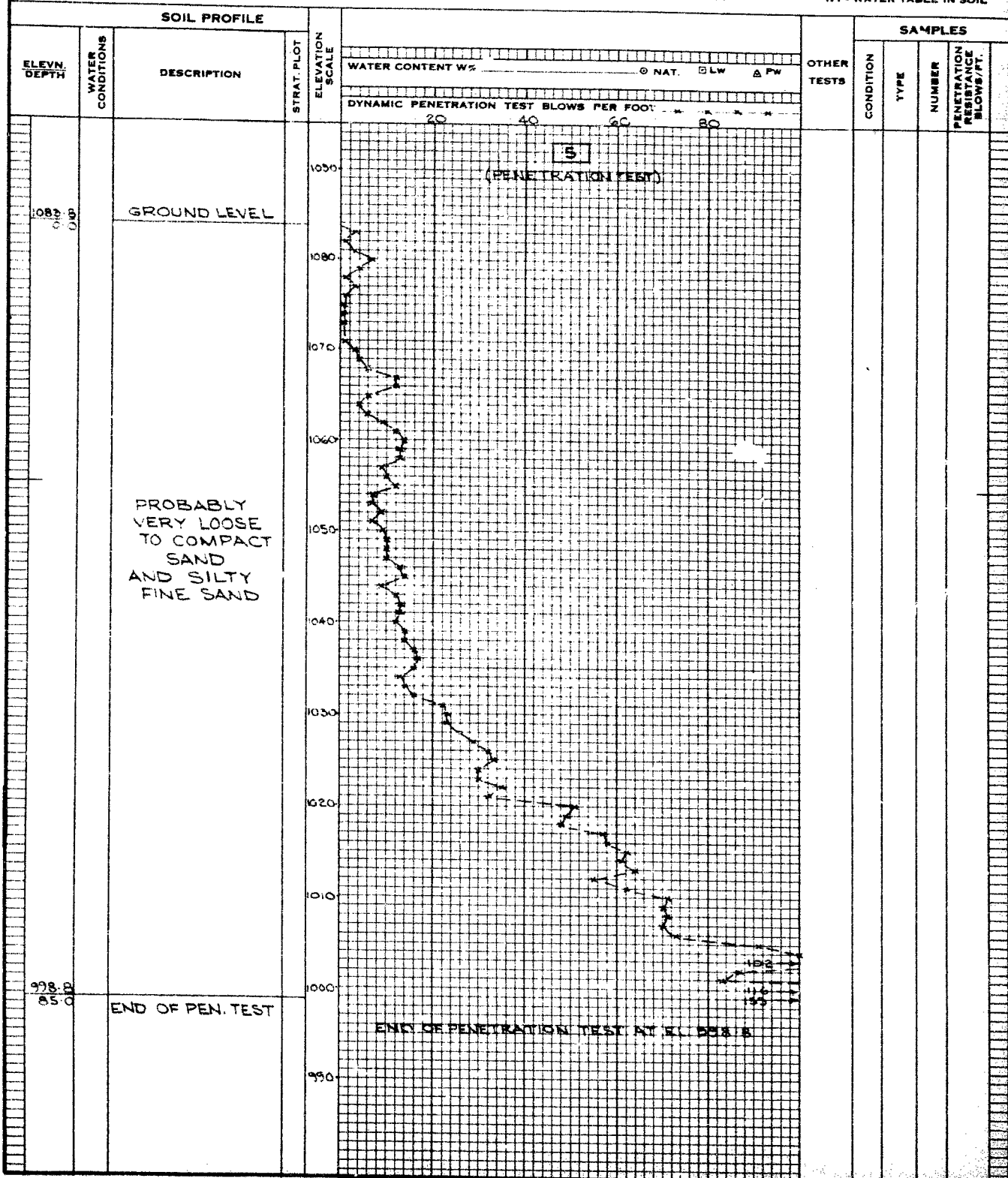
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

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



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 17703 BORING # 6 DATUM GEODETIC CASING 3X
 BORING DATE FEB. 12-22/65 REPORT DATE MAR. 11, 1965 COMPILED BY AEL CHECKED BY _____
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES PENETRATION RESISTANCES CONVERTED TO BLOWS OF 2000 IN - LBS. ENERGY

SAMPLE CONDITION



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

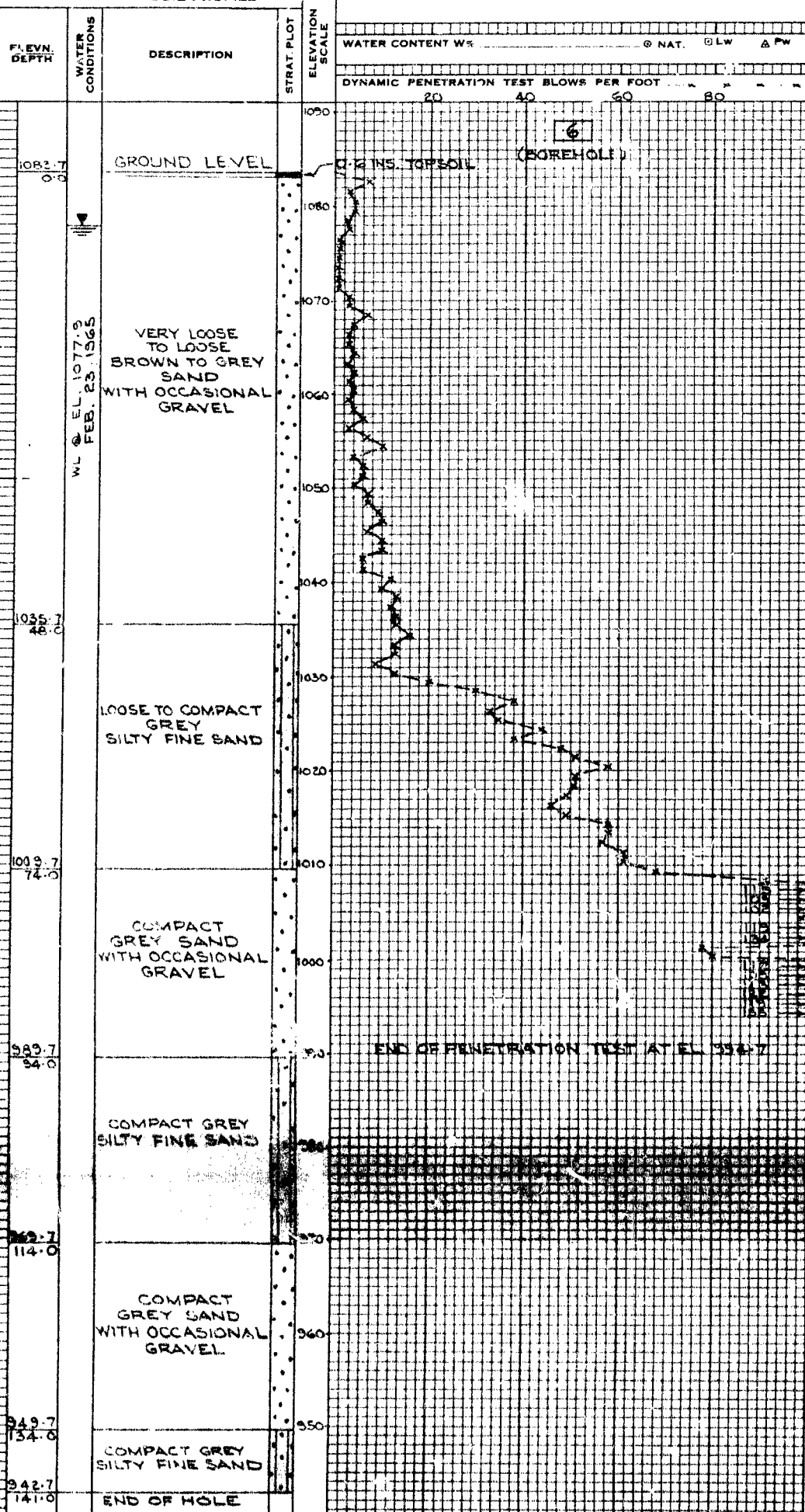
SAMPLE TYPES

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

SOIL PROFILE



SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
CS	1	1	
2" DF	3	4	
" DF ST	4	2	
" DF ST	5	2	
" DF ST	6	2	
" DF ST	7	3	
" DF ST	8	3	
" DF ST	9	5	
" DF	10	4	
" DF	11	6	
" DF	12	5	
" DF ST	13	13	
" DF	14	9	
" DF	15	16	
" DF ST	16	17	
" DF ST	17	7	
" DF ST	18	18	
" DF ST	19	20	
WS	20	-	
2" DF 20A	20A	26	
" DF ST	21	16	

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7709 BORING # 7 DATUM GEODETIC CASING BX.
 BORING DATE FEB. 24, 1965 REPORT DATE MAR. 11, 1965 COMPILED BY AEL CHECKED BY _____
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



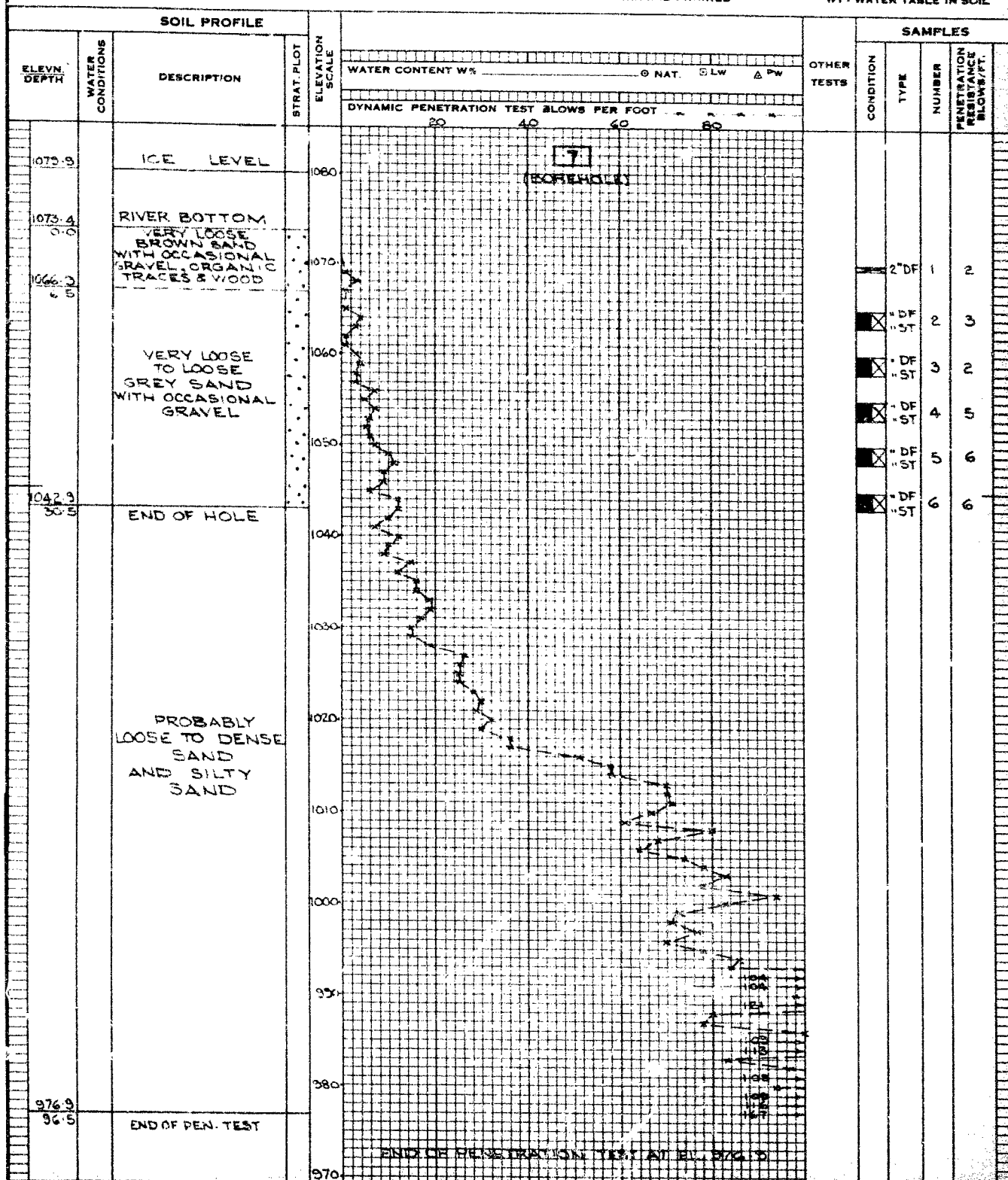
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

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
 OC. - TRIAXIAL CONSOLIDATED UNDRAINED
 O. - TRIAXIAL UNCONFINED
 S. - TRIAXIAL DRAINED
 7. - WET UNIT WEIGHT
 K. - PERMEABILITY
 C. - CONSOLIDATION
 WL. - WATER LEVEL IN CASING
 WT. - WATER TABLE IN SOIL



APPENDIX II

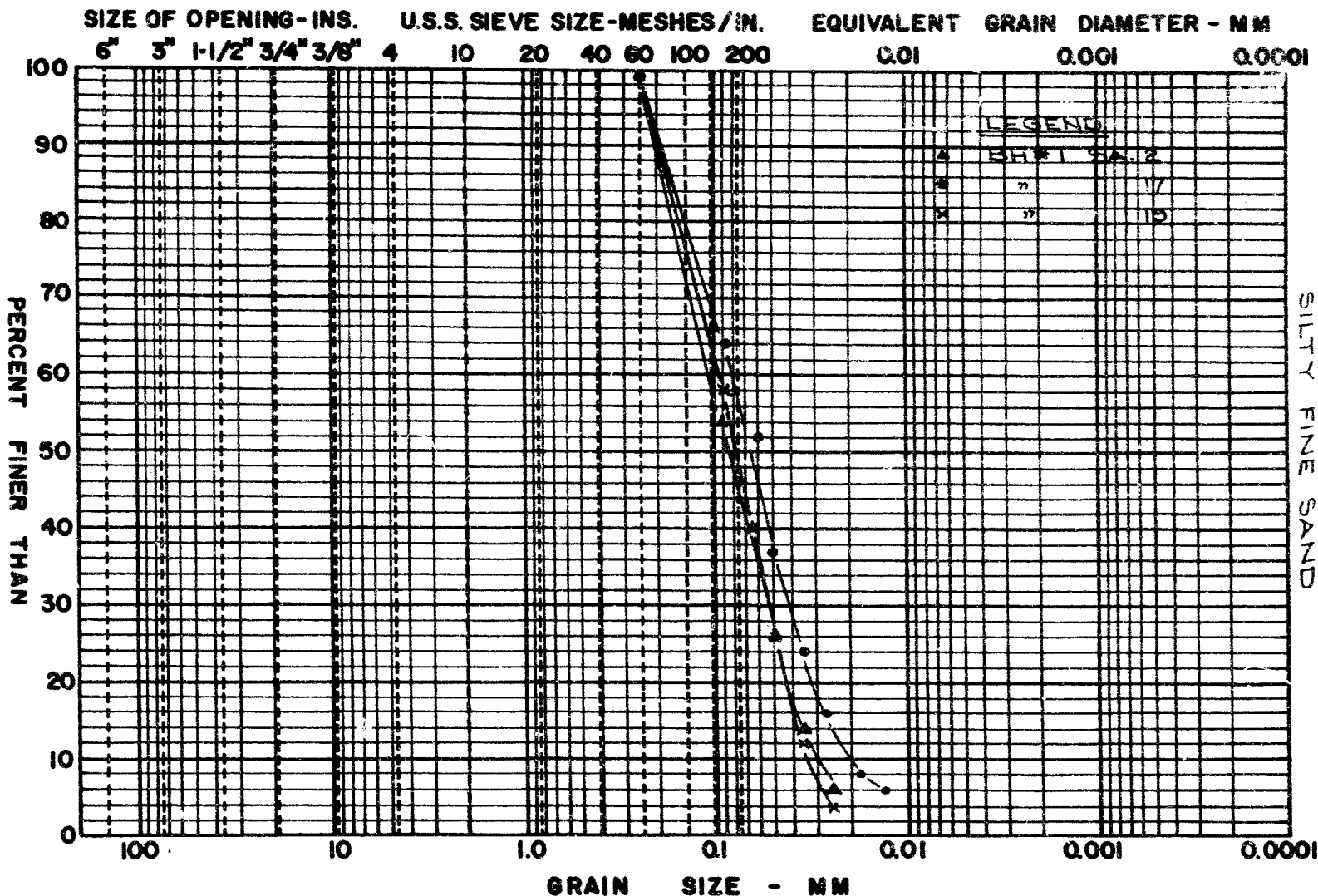
FIGURES - LABORATORY TESTING

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX 11
FIGURE 1
PROJECT T7709

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



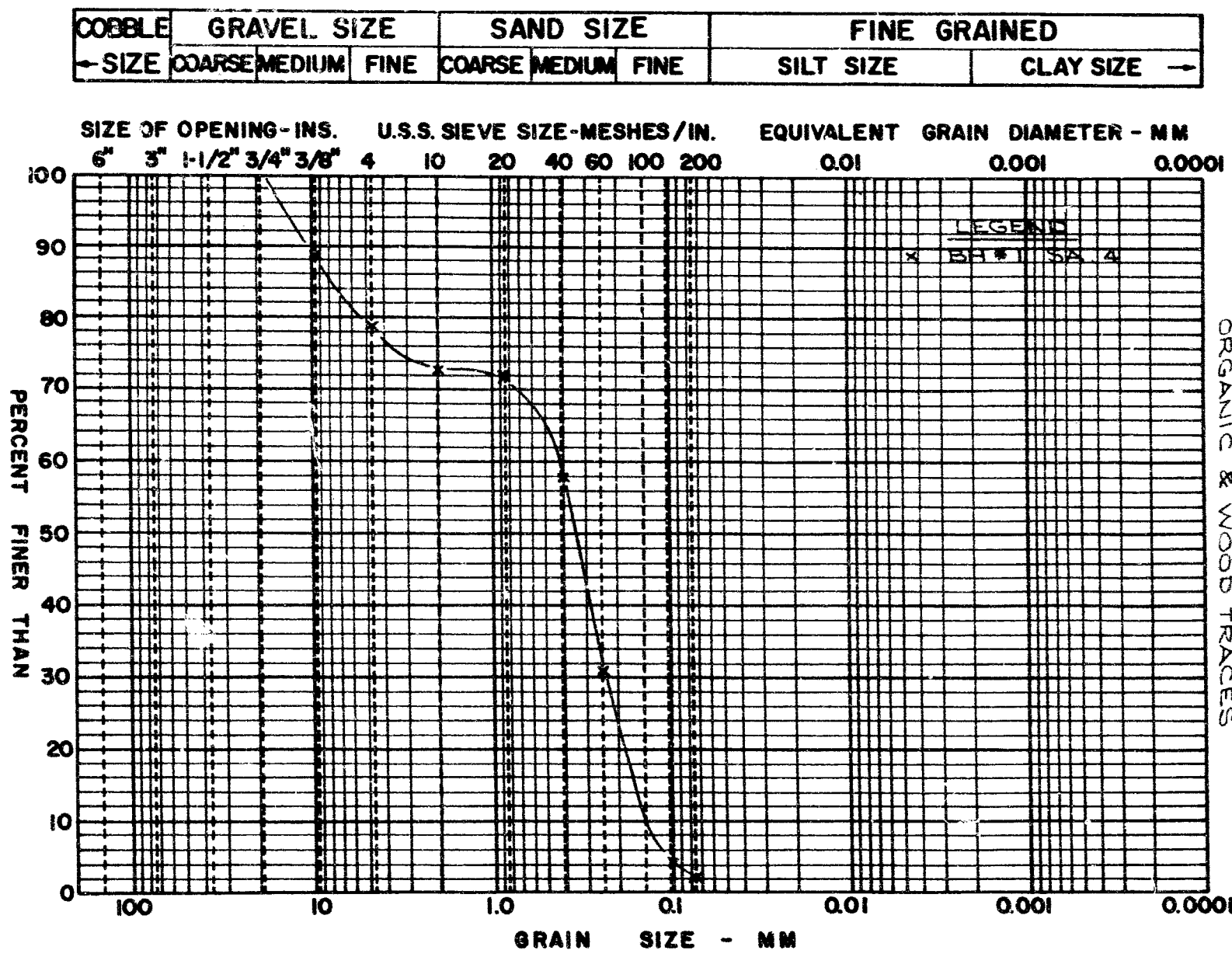
M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 2
PROJECT T7709

SAND WITH OCCASIONAL GRAVEL,
ORGANIC & WOOD TRACES



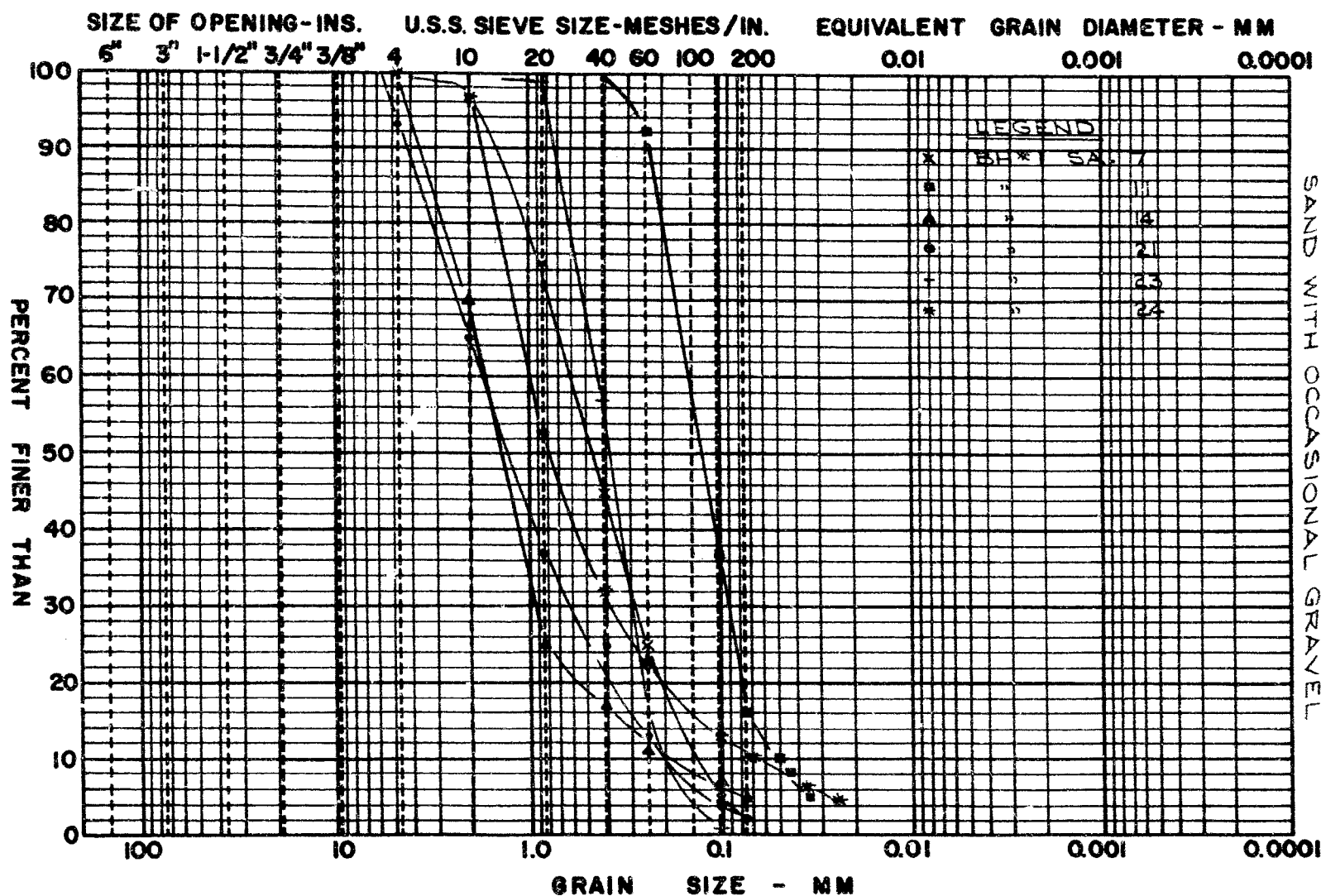
M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 3
PROJECT T7709

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



M.I.T. GRAIN SIZE SCALE

GEOCON

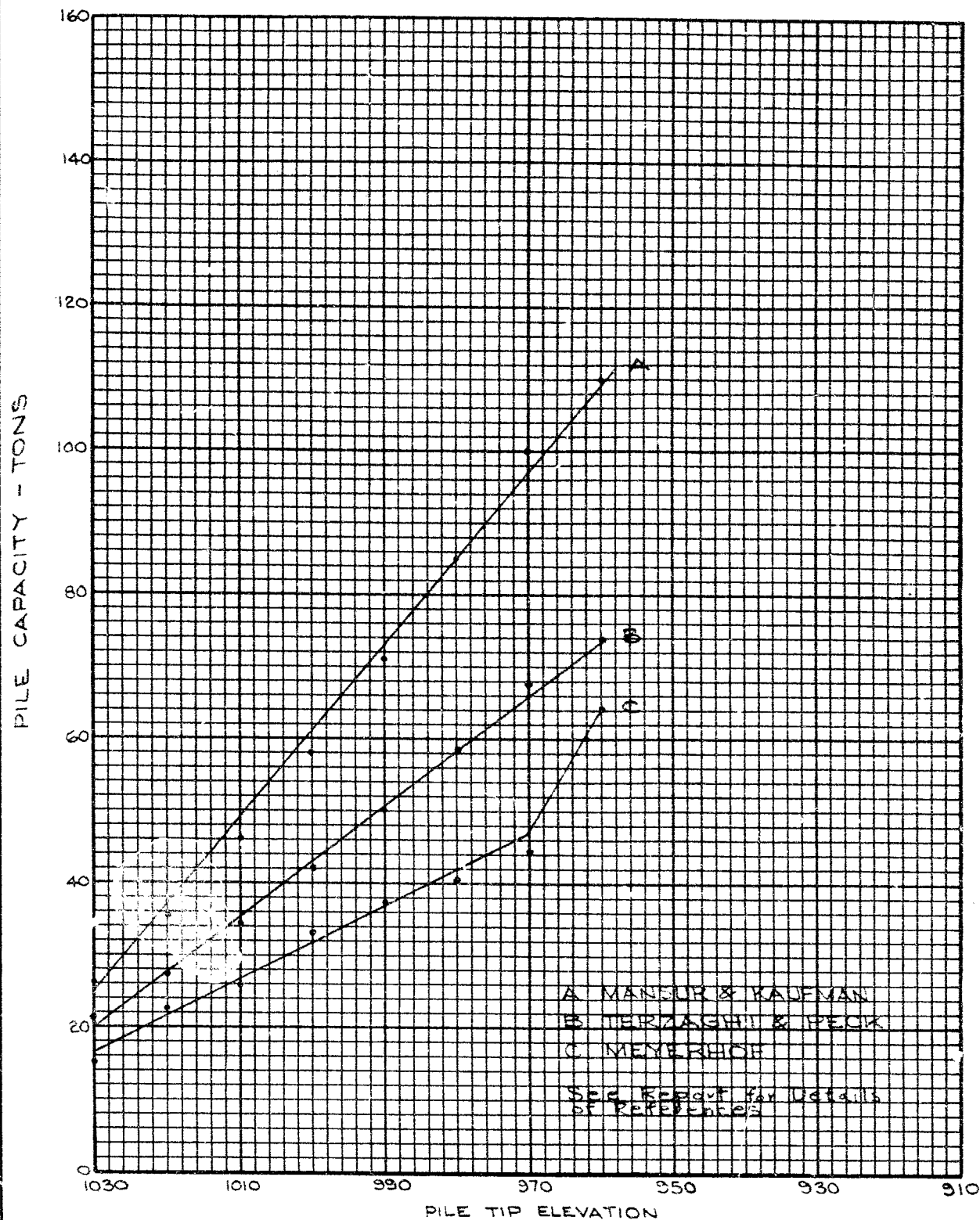
PILE CAPACITY vs. PILE TIP ELEVATION

12 IN. DIA. PILE

APPENDIX II

FIGURE 4

PROJECT T7709

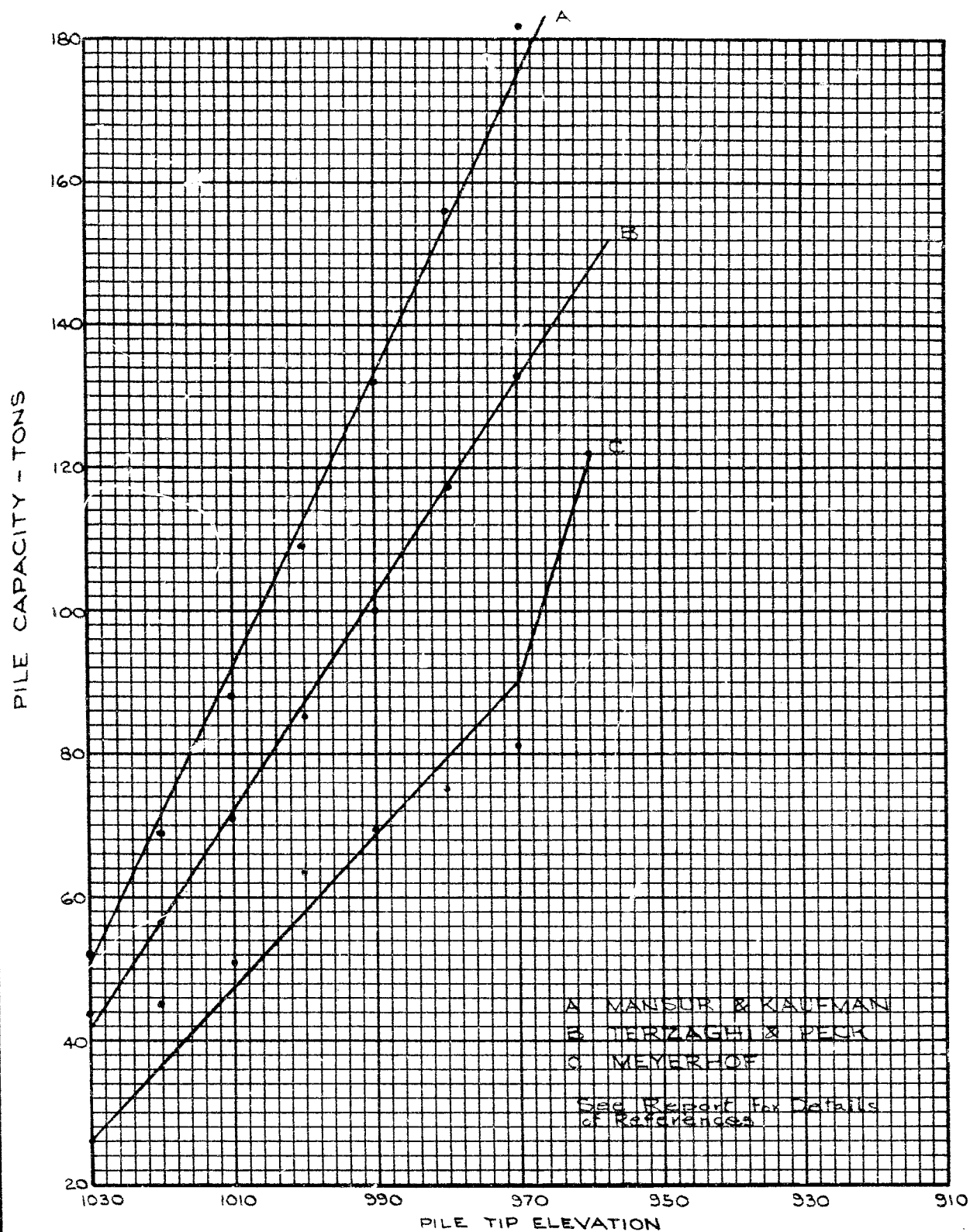


GEOCON

PILE CAPACITY vs. PILE TIP ELEVATION

18 IN. DIA. PILE

APPENDIX II
FIGURE 5
PROJECT T7709

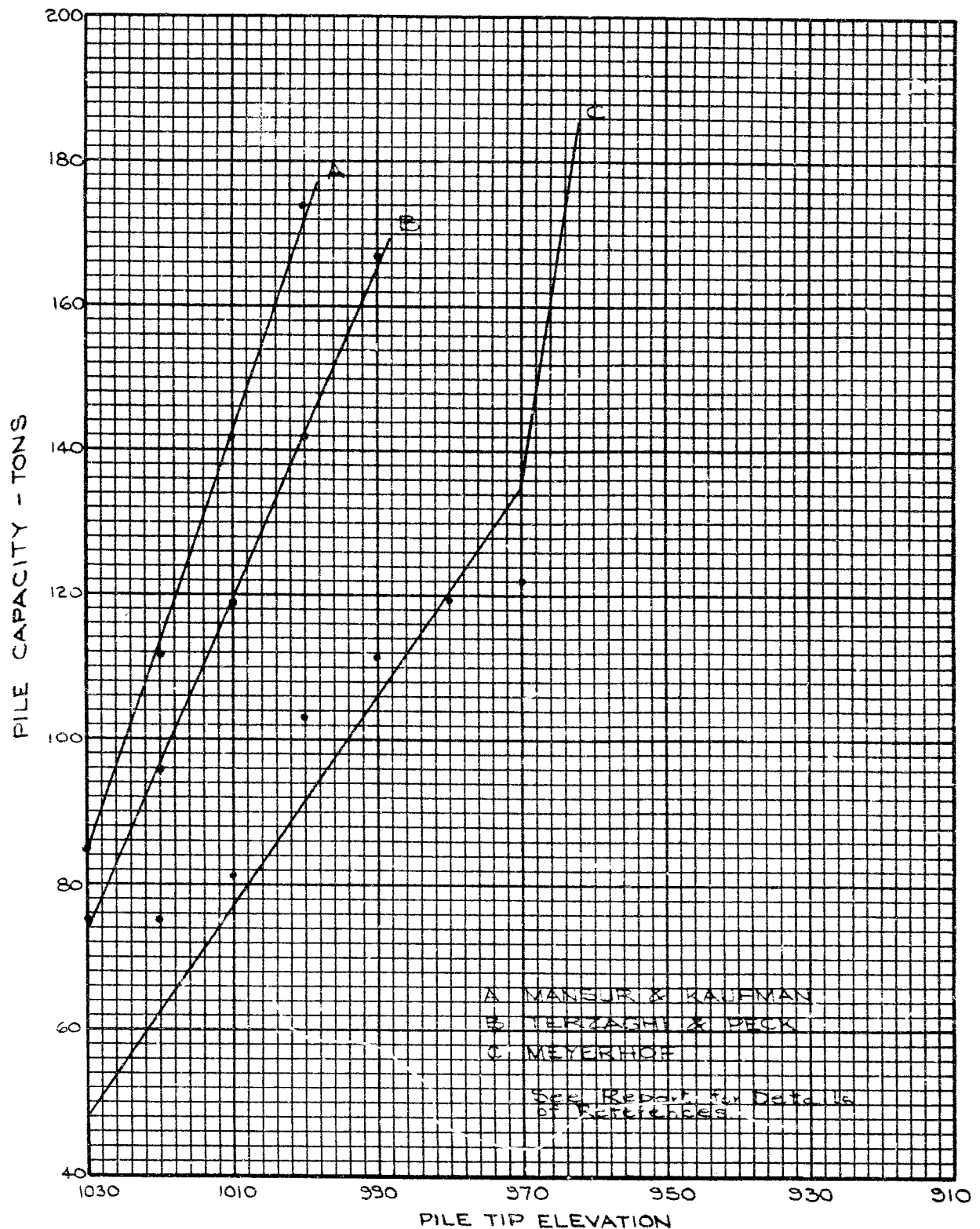


GEOCON

PILE CAPACITY vs. PILE TIP ELEVATION

24 IN. DIA. PILE

APPENDIX II
FIGURE 6
PROJECT T7709



GEOCON