W.O. 70-F-207C

CREDIT RIVER

& 25TH SIDE ROAD

40P16-11

A. F. DIAS, CONSULTING ENGINEER

SUBSURFACE INVESTIGATION
CREDIT RIVER BRIDGE REPLACEMENT
25TH SIDEROAD 70-F-207M

CALEDON TOWNSHIP ONTARIO

Peel County

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February, 1970 70003

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SOIL AND FOUNDATION ENGINEERS

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February 19, 1970.

A. F. Dias, Consulting Engineer,
338 Queen Street East,
Brampton, Ontario.

Attention: Mr. A. F. Dias, P. Eng.

RE: SUBSURFACE INVESTIGATION,
PROPOSED CREDIT RIVER BRIDGE
REPLACEMENT,
25TH SIDEROAD,
CALEDON TOWNSHIP, ONTARIO.

Dear Sir:

This report presents the results of a subsurface investigation carried out at the site of the proposed Credit River Bridge replacement at the 25th Sideroad, approximately 1-1/2 miles west of Highway 10 in Caledon Township. The purpose of the investigation was to determine the soil and groundwater conditions at the site and to make recommendations for the foundation design of the proposed crossing and pavement design for the re-aligned roadway.

PROCEDURE

Four boreholes were put down during the period January 8 to 14, 1970, at the locations shown on Fig. 1, using

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a trailer-mounted machine drillrig supplied and operated by P.V.K. & Sons Drilling Ltd. under the full time supervision of a member of our engineering staff. Two of the borings at the proposed bridge were taken to depths of 36 ft. while the remaining borings were put down through the existing roadway to depths of about 7 ft. Samples of the subsoil were taken at regular intervals of depth using a 2 in. diameter split spoon sampler. The samples were brought to our laboratory for detailed examination and testing.

Following completion of sampling in BH's 1 and 2, standpipes were installed to permit monitoring of the groundwater level at the proposed bridge.

The soil stratigraphy encountered in the borings is given on the Record of Borehole sheets and an inferred stratigraphic section along the proposed centreline is given on Fig. 1. The results of the laboratory tests are given on the Records of Boreholes and on Figs. 2 and 3.

The locations and ground surface elevations of the boreholes put down during this investigation were obtained by our staff. The borehole elevations are referred to a bench mark consisting of the top of a nail in a root of a 4 ft. diameter stump at the location given on Fig. 1. The elevation of this bench mark is understood to be 1,318.65, referred to Geodetic datum.

SITE AND GEOLOGY

The site is located approximately 2 miles south of Orangeville and 1-1/2 miles west of Highway 10. At this location, the Credit River flows in a southerly direction in a relatively shallow valley having a base width of about 100 ft. with banks rising some 15 to 20 ft. above the valley floor. The river channel is about 20 ft. wide by 4 ft. deep and the depth of water at the time of the investigation was some 1 to 1-1/2 ft.

The site is situated along the Niagara Escarpment physiographic region of Ontario. The Credit River, which has its source to the north of Orangeville, flows in a spillway of a glacial meltwater stream along the edge of the Niagara escarpment. The spillway is bordered by hummocky, bouldery, morainic ridges and deposits of sands and gravels covering the Escarpment rock.

SUBSURFACE CONDITIONS

The detailed soil stratigraphy encountered in each of the borings is shown on the Record of Borehole sheets and a stratigraphy section along the centreline of the roadway is given on Fig. 1. Following is a summarized account of the soil conditions at the site.

The existing roadway embankment at the abutments of the existing bridge consists of about 9 ft. of generally loose silty sand with some gravel. The fill has been placed over about 11 ft. of recent deposits consisting of soft organic silt, loose sands and gravels containing pieces of wood, and soft to stiff clayey silt to silty clay with thin layers of organic silt and pieces of wood. The recent deposits are underlain by dense to very dense sand and gravel.

The groundwater level measured in the standpipes installed in the borings at the proposed bridge at the time of the field investigation was about 1 ft. above the river water level.

DISCUSSION

We understand that the Credit River crossing was to consist of a 40 ft. span rigid frame structure. However, following your review of our draft report dated February 2, 1970, we were advised that in view of the cost of providing sheeted excavations for the proposed bridge footings, two flexible multi-plate steel culverts will be used for the crossing. The grade at the crossing will be raised about 8 ft. above the existing grade and the re-aligned Township roadway at the crossing will have a gravel surface.

PROPOSED CULVERTS

We understand that the proposed culverts will require a total end area of 400 sq.ft. and that two 16 ft. diameter circular, flexible, multi-plate culverts are being considered.

Consideration might also be given to the use of two flexible pipe-arch multi-plate culverts; two 20 ft. span by 13 ft. rise pipe-arch culverts would be required to provide the necessary area. One advantage of pipe-arch culverts is that the rise of a pipe-arch is less than the diameter of a circular culvert having the same end area, consequently, a pipe-arch will be operating at full capacity at a considerably lower depth of flood water than will a circular culvert.

It is recommended that the culverts be placed on a 30 in. minimum thickness of bedding material consisting of free draining sand and gravel placed immediately following removal of the river bed deposit. A detailed inspection of the subgrade should be made prior to placement of the bedding pad to ensure that there are no extensive zones of soft and compressible organic material. The bedding pad material should be compacted in 9 in. lifts to 100 per cent of the standard Proctor optimum dry density. Allowing for a 4 ft. clear space between culverts, the minimum recommended width of bedding pad for two 16 ft. diameter culverts is 52 ft., and 65 ft. for two 20 ft. span pipe-arch culverts.

The backfill adjacent to the culverts above the bedding pad should be free draining and non-frost-susceptible granular material, free of particles larger than 3 in. in diameter, placed in lifts not exceeding 9 in. in loose thickness. Each lift should be compacted to 100 per cent of the standard Proctor optimum dry density using hand tampers at the haunches and sides of the culverts. The backfill should be brought up at the same rate along the sides of the culverts and carried to a height of at least 1 ft. above the top of the culverts before any relaxation in compaction control is considered or before heavy compaction equipment is allowed over the culverts.

Some settlement of the culvert can be expected due to the presence of the thin organic layers and pieces of wood in the underlying recent deposits. We estimate that the settlement beneath the central part of the roadway could be of the order of 3 to 4 in. and about 1 in. beneath the toe of the embankment fill. To accommodate the anticipated differential settlement, it is recommended that the culvert be placed with a 3 in. camber in the middle.

RIGID FRAME BRIDGE (ALTERNATIVE)

The recent deposits, which extend about 8 ft. below the river bed as shown on Fig. 1, are not suitable for the foundation support of a rigid frame structure. The underlying dense to very dense sand and gravel deposit is a competent founding stratum and the bridge structure may be placed on spread footings founded within the upper portion of the deposit at about elevation 1,295. An allowable bearing pressure of 4 tons/sq.ft. may be used in design of a spread footing having a minimum width of 4 ft. Provided that the in situ density of the sand and gravel at and below founding elevation is not reduced during construction, the total settlement of footings founded as discussed above should be negligible.

Since the excavation for spread footings founded within the dense sand and gravel will be taken some 10 ft. below the river water level, some form of groundwater control will be required to permit construction in the dry and to prevent 'boiling' and loosening of the base of the excavation due to upward seepage of water. This control may be achieved by means of adequately braced and strutted closed steel sheet piling driven to a minimum depth of 6 ft. below the final footing excavation level (excavation level assumed to be elevation 1,295).

Backfill behind the abutments should consist of at least 4 ft. (measured horizontally) of free draining and non-frost-susceptible granular material. Provision should be made for drainage of this backfill in order to prevent hydrostatic or ice pressure build-up behind the abutments. With full effective drainage behind the abutments, an at-rest earth pressure coefficient, K_o, = 0.4 and a total unit weight of 135 lb/cu.ft.

(using submerged unit weight below the groundwater level) may be used in design of the abutments.

ROADWAY

The roadway grade across the Credit River valley will be raised by up to 8 ft. Since there will not be any borrow material available from within the limits of this contract, we recommend that the necessary imported material consist of well graded sand and gravel. The granular embankment materials should be placed in 9 in. thick lifts and each lift should be compacted to 95 per cent of the standard Proctor optimum dry density. The upper foot of the embankment fill, which will form the subgrade for the granular pavement materials, should be compacted to 100 per cent of the standard Proctor optimum dry density.

The natural subgrade beneath the existing roadway, as indicated in BH's 3 and 4, consists of loose to compact silts and sands which are moderately to highly frost-susceptible. In order to eliminate potential frost heave in the new roadway, it would be necessary to remove the silts and sands for the full depth of frost penetration (about 4 to 5 ft.) and replace with free-draining and non-frost-susceptible granular material. However, assuming that some frost heave can be tolerated and that provision will be made for yearly maintenance, we suggest that the gravel surface roadway consist of 4 in. of sand cushion (such as concrete sand) placed over the compacted silty subgrade, followed by 20 in. of D.H.O. granular 'B' with less than 5 per cent silt sizes, topped with 6 in. of D.H.O. granular 'A'. The roadway materials should be compacted to at least 100 per cent of the standard Proctor optimum dry density. The subgrade, subbase, and base course should be crowned in order to promote drainage from the roadway into ditches.

In areas where the granular pavement materials are placed over granular embankment fill, the 4 in. sand cushion can be eliminated. Consideration can also be given to reducing or eliminating the 20 in. thick layer of granular 'B' over the embankment provided that the imported embankment material forming the roadway subgrade meets the D.H.O. granular specifications.

We trust that this report provides sufficient information for your design purposes. If you have any questions regarding this report or if we can be of any further assistance to you on this project, please call us.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.

L R Lahti

L. R. Lahti, P. Eng.

LRL:jg
70003

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$

e = base of natural logarithms 2.7183

$\log_e a$ or $\ln a$, natural logarithm of a

$\log_{10} a$ or $\log a$, logarithm of a to base 10

t time

g acceleration due to gravity

V volume

W weight

M moment

F factor of safety

II. STRESS AND STRAIN

u pore pressure

σ normal stress

σ' normal effective stress ($\bar{\sigma}$ is also used)

τ shear stress

ϵ linear strain

ϵ_{xy} shear strain

ν Poisson's ratio (μ is also used)

E modulus of linear deformation (Young's modulus)

G modulus of shear deformation

K modulus of compressibility

η coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

γ unit weight of soil (bulk density)

γ_s unit weight of solid particles

γ_w unit weight of water

γ_d unit dry weight of soil (dry density)

γ' unit weight of submerged soil

G_s specific gravity of solid particles $G_s = \gamma_s/\gamma_w$

e void ratio

n porosity

w water content

S_r degree of saturation

(b) *Consistency*

w_L liquid limit

w_P plastic limit

I_P plasticity index

w_S shrinkage limit

I_L liquidity index $= (w - w_P)/I_P$

I_C consistency index $= (w_L - w)/I_P$

e_{max} void ratio in loosest state

e_{min} void ratio in densest state

D_r relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

h hydraulic head or potential

q rate of discharg

v velocity of flow

i hydraulic gradient

k coefficient of permeability

j seepage force per unit volume

(d) *Consolidation (one-dimensional)*

m_v coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$

C_c compression index $= -\Delta e/\Delta \log_{10} \sigma'$

c_v coefficient of consolidation

T_v time factor $= c_v t/d^2$ (d, drainage path)

U degree of consolidation

(e) *Shear strength*

τ_f shear strength

c' effective cohesion intercept; ϕ' effective angle of shearing resistance, or friction — in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$

c_u apparent cohesion*; ϕ_u apparent angle of shearing resistance, or friction — in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$

μ coefficient of friction

S_t sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

Form G.A. - D - 1

Project No. 70003

RECORD OF BOREHOLE 1

LOCATION See Figure 1 BORING DATE JANUARY 8-18, 1970 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: WASH BORING NX CASING

ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
1314.5 / 0.0	GROUND SURFACE				
	LOOSE, WITH DENSE ZONES, BROWN MIXTURE SILTY SAND, SOME GRAVEL AND OCCASIONAL COBBLES (FILL)	1	2" DO	5	
		2	"	47	
1306.0 / 8.5	SOFT DARK GREY ORGANIC SILT AND SANDY SILT	3	"	7	
1304.5 / 10.0	LOOSE BROWN TO GREY SAND AND GRAVEL, TO SAND WITH SOME SILT AND GRAVEL, PIECES OF WOOD (RECENT DEPOSIT)	4	"	7	M
		5	"	6	
		6	"	10	M.H.
1298.5 / 16.0	SOFT TO FIRM GREY SILTY CLAY, TRACE SAND AND ORGANIC MATERIAL, PIECES OF WOOD (RECENT DEPOSIT)	7	"	4	
1295.5 / 19.0	VERY DENSE GREY BROWN SAND AND GRAVEL, TRACE SILT			100/6"	
		8	"	78	M.H.
		9	"	154/10"	
		10	"	133	
		11	"	49	
1278.0 / 36.5	END OF HOLE				

LOST DRILLING WASH WATER AT ELEV. 1307 AND AT 1304.5

PIEZOMETER OR STANDPIPE INSTALLATION: CAP; GROUND SURFACE; GRAVEL BACKFILL; CAVED MATERIAL; STANDPIPE; CAVED MATERIAL

WATER LEVEL IN STANDPIPE AT ELEV. 1305.6 JAN. 14, 1970

Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

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DRAWN E.Z.S.

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Project No. 70003

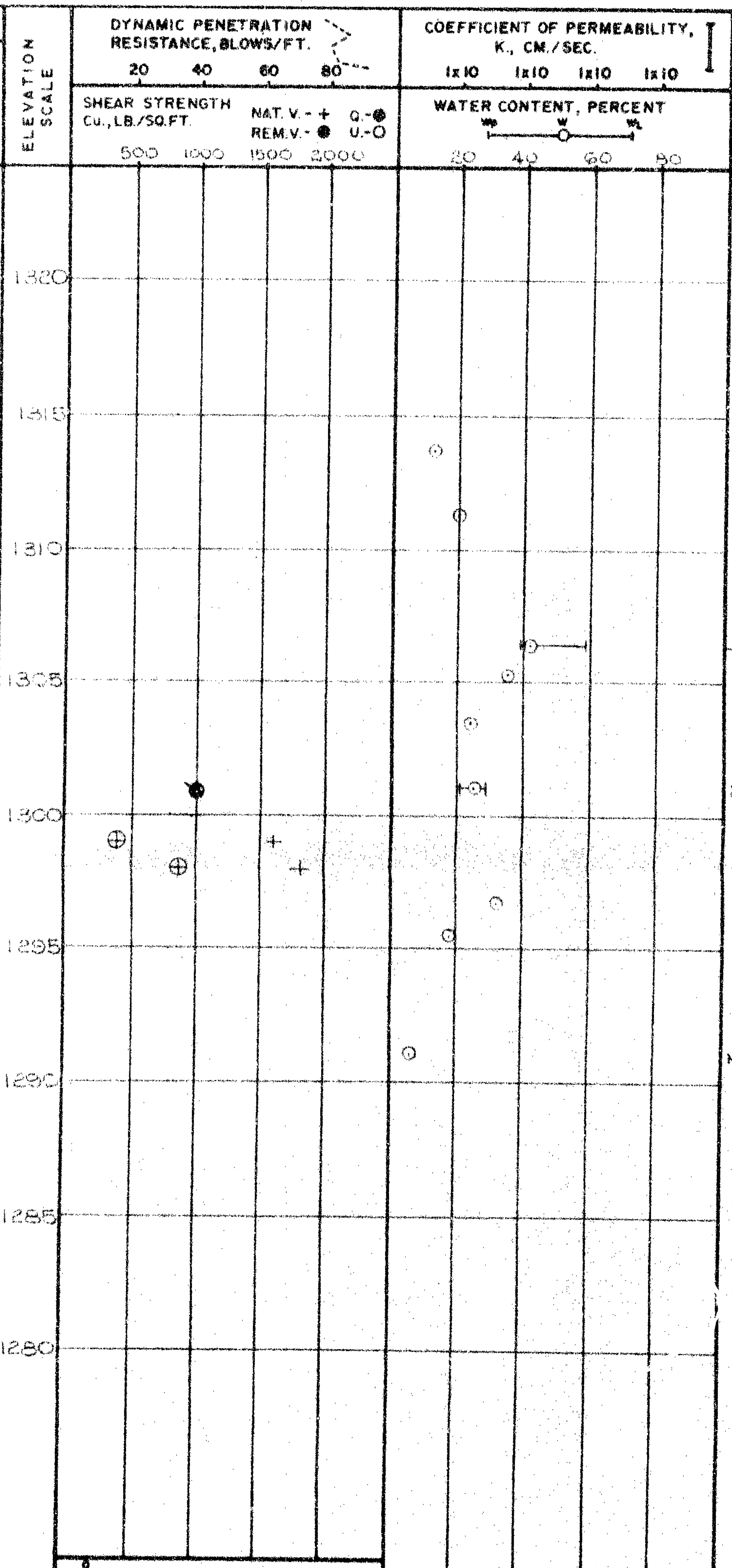
RECORD OF BOREHOLE 2

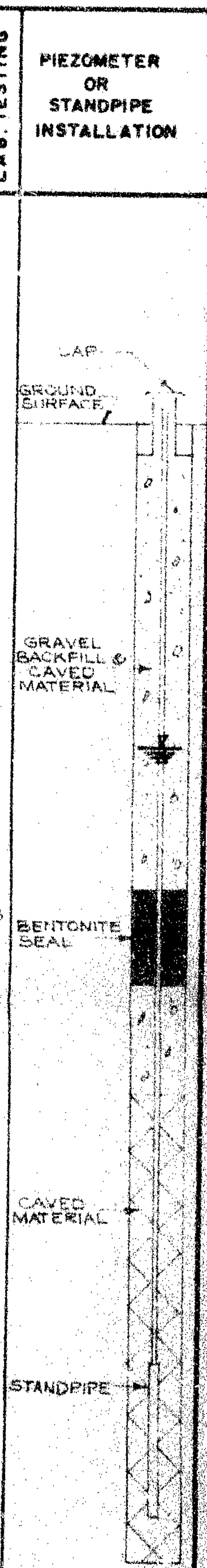
LOCATION See Figure 1 BORING DATE JANUARY 13-14, 1970 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: WASH BORING, NX CASING

ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT	ADDITIONAL LAB. TESTING
1318.9 / 0.0	GROUND SURFACE				
	LOOSE BROWN SILTY SAND, SOME GRAVEL, FROZEN TO ELEV 1314.5 (FILL)	1	2" O.D.	140	ORGANIC CONTENT 4%
		2	TO		
		3	2" O.D.	8	
1307.4 / 9.5	SOFT GREY ORGANIC SILT AND SILTY FINE SAND, PIECES OF WOOD (FLOOD PLAIN DEPOSIT)	4	"	4	
1303.9 / 13.0	FIRM TO STIFF GREY LAYERED CLAYEY SILT AND SILTY CLAY, OCCASIONAL 1/8" THICK DARK GREY ORGANIC SILT LAYERS (RECENT DEPOSIT)	5	"	10	
		6	2" TO	PH	γ=123
		7	2" O.D.	34	
1295.9 / 21.0	DENSE TO VERY DENSE BROWN SAND BECOMING SAND AND GRAVEL, TRACE SILT	8	"	86	M.H.
		9	"	120	
		10	"	49	
1280.4 / 36.5	END OF HOLE				





WATER LEVEL IN STANDPIPE AT ELEV. 1306.4 JAN. 14, 1970

VERTICAL SCALE 1 IN. TO 5 FT.

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RECORD OF BOREHOLE 3 & 4

LOCATION See Figure I — BORING DATE JANUARY 14, 1970 — DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB, DROP 30 IN. — PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

Column headings: BORING METHOD | SOIL PROFILE (ELEV'N. / DEPTH, DESCRIPTION, STRAT. PLOT) | SAMPLES (NUMBER, TYPE, BLOWS/FT.) | ELEVATION SCALE | DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. (20, 40, 60, 80); SHEAR STRENGTH Cu, LB./SQ.FT. NAT. V. - + Q - ● REM.V. - ● U - O | COEFFICIENT OF PERMEABILITY, K, CM./SEC. (1x10, 1x10, 1x10, 1x10); WATER CONTENT, PERCENT Wp W Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Borehole 3

Boring method: DRY SAMPLING, NX CASING

ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
1326.4	GROUND SURFACE			
1325.9 / 0.5	SAND & GRAVEL (FILL)			
	LOOSE BROWN SAND, SOME SILT, TRACE CLAY (FROZEN TO ELEV. 1324)	1	C.S.	—
1321.4 / 5.0		2	2" D.O.	8
1319.4 / 7.0	HARD GREY CLAYEY SILT WITH SAND LAYERS	3	"	31
	END OF HOLE			

Elevation scale: 1325, 1320

Additional lab testing: M.H.

WATER LEVEL IN OPEN HOLE AT ELEV. 1321 JAN. 14, 1970

Borehole 4

Boring method: DRY SAMPLING, NX CASING

ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
1323.1	GROUND SURFACE			
1322.6 / 0.5	SAND & GRAVEL (FILL)			
	COMPACT BROWN STRATIFIED SILT AND SAND BECOMING SILT BELOW ELEV. 1317.6 (FROZEN TO ELEV. 1320.5)	1	C.S.	11
		2	2" D.O.	
1316.6 / 6.5		3	"	9
	END OF HOLE			

Elevation scale: 1325, 1320, 1315

WATER LEVEL IN OPEN HOLE AT ELEV. 1318 JAN. 14, 1970

15 ◆ 5 (0, 10) Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

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DRAWN E.Z.S.

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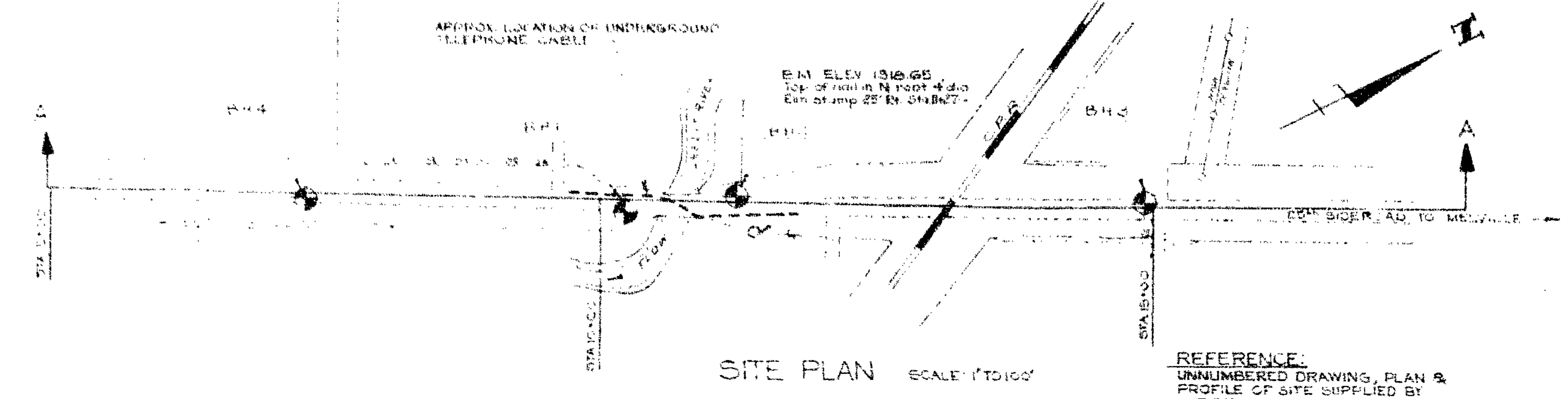
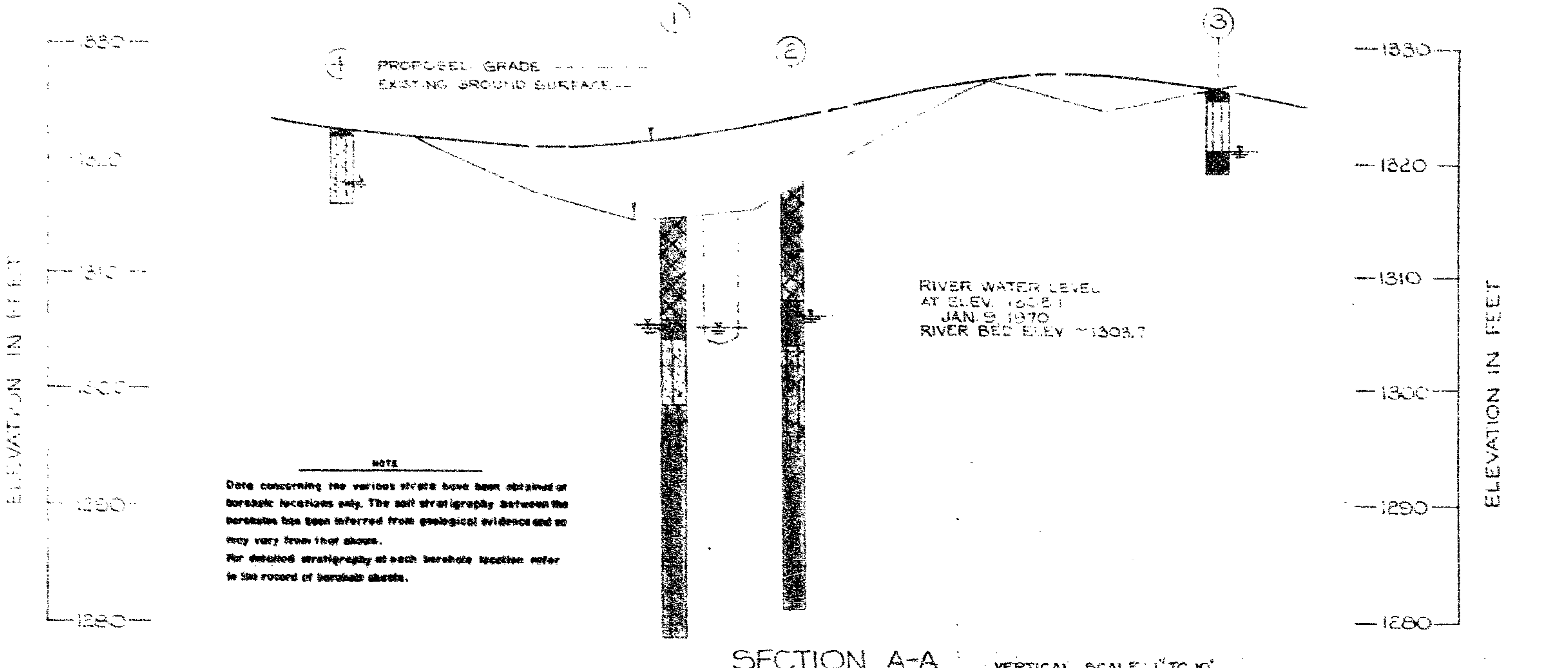
BORING PLAN AND STRATIGRAPHIC SECTION

FIGURE 1



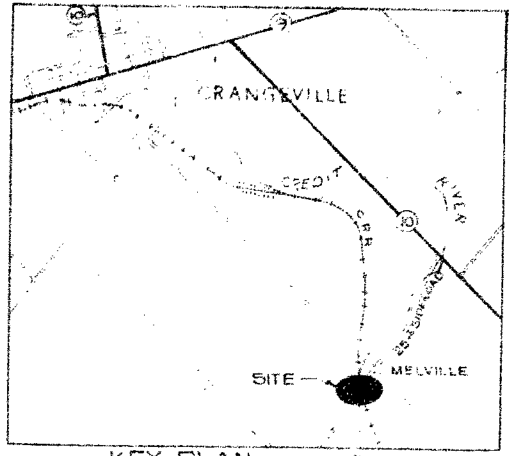
SITE PLAN SCALE: 1" TO 100'



NOTE

Data concerning the various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geological evidence and so may vary from that shown.
For detailed stratigraphy at each borehole location refer to the record of borehole sheets.

SECTION A-A VERTICAL SCALE: 1" TO 10' HORIZONTAL SCALE: 1" TO 100'



KEY PLAN SCALE: 1" TO 0.8 MILES

LEGEND

BOREHOLE IN PLAN

BOREHOLE IN ELEVATION

WATER LEVEL IN BOREHOLE JAN. 14, 1970

STRATIGRAPHY

LOOSE TO COMPACT BROWN **SILTY SAND** SOME GRAVEL TO **SAND AND GRAVEL (FILL)**

LOOSE TO COMPACT BROWN **SILT** AND **SAND**

SOFT DARK GREY **ORGANIC SILT** AND **SILTY SAND**, PIECES OF WOOD **(FLOOD PLAIN DEPOSIT)**

LOOSE BROWN TO GREY **SAND** AND **GRAVEL** TO **SAND**, SOME SILT AND GRAVEL, PIECES OF WOOD **(RECENT DEPOSIT)**

SOFT TO STIFF GREY LAYERED **CLAYEY SILT** TO **SILTY CLAY** WITH 1/8 IN. THICK ORGANIC SILT LAYERS **(RECENT DEPOSIT)**

DENSE TO VERY DENSE BROWN **SAND** AND **GRAVEL** TRACE SILT

Date JAN. 22, 1970

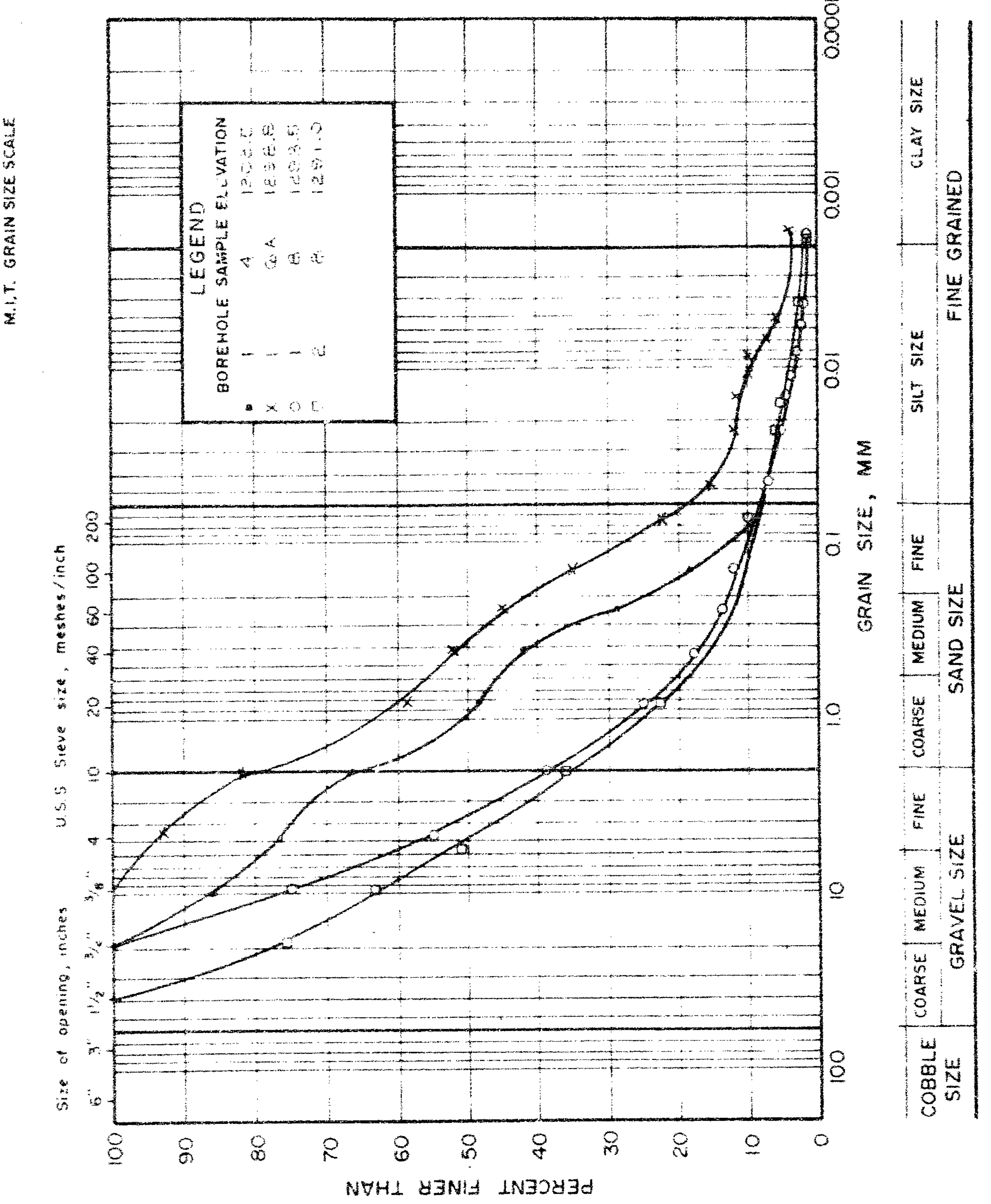
Golder Associates

Drawn
Chkd.
Appd.

PROJECT No. 79003

GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL

FIGURE 2



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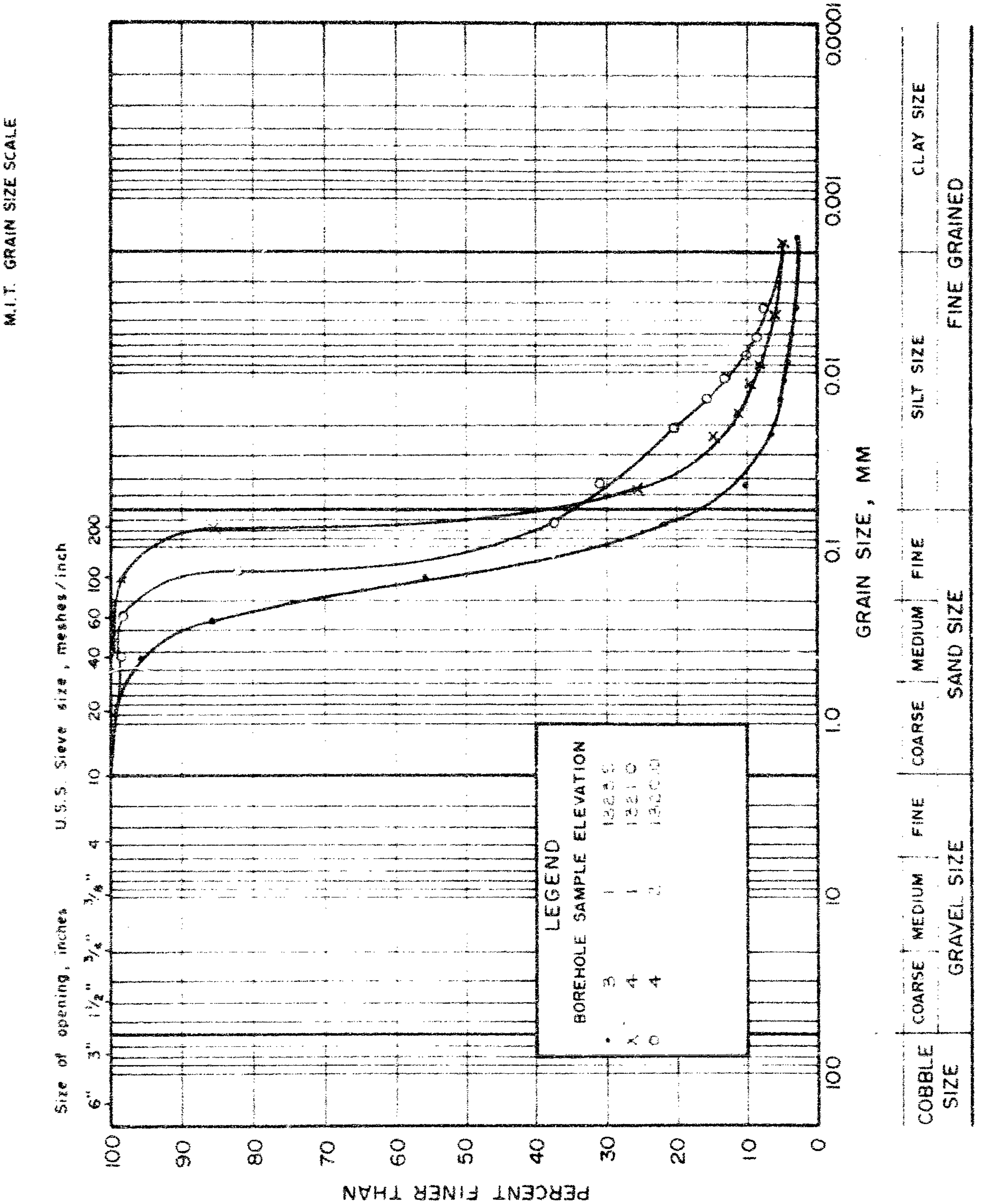
GRAIN SIZE DISTRIBUTION
SILT AND SAND

FIGURE 3



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