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GEOCRES No. 31 G - 30

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. _____

LOCATION PROP. East CASTOR
RIV. BRIDGE, OSGOOD TWP.

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

NONE

REMARKS: _____



A. J. GRAHAM ENGINEERING CONSULTANTS
SOIL INVESTIGATION
PROPOSED EAST CASTOR RIVER BRIDGE
TOWNSHIP OF OSGOODE
MARVELVILLE ONTARIO.

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June, 1972

72792

Golder Associates

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June 16, 1972.

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Attention: Mr. A. R. Sethna, P. Eng.

RE: SOIL INVESTIGATION,
PROPOSED EAST CASTOR RIVER BRIDGE,
TOWNSHIP OF OSGOODE,
VERNON, ONTARIO.

Dear Sirs:

This letter reports the results of a soil investigation carried out at the bridge crossing of a branch of the East Castor River by Regional Road No.4 in Osgoode Township about 1/2 mile west of Marvelville, Ontario. The purpose of the investigation was to determine the subsoil and groundwater conditions at the site and, based on this information, to make recommendations for the foundation design and construction of a proposed bridge replacement structure.

PROCEDURE

The field work for this investigation was carried out on May 23 and 24, 1972. Two boreholes, one each in the area of the east and west abutment locations of the proposed bridge, were put down to some 8 to 11 ft. below stream level using a machine drill rig supplied and operated by the F. E. Johnston Drilling Co. Ltd., Ottawa. Standard drive open sampling was carried out in the overburden and bedrock was cored in borehole 1 for a depth of about 5 ft. A standpipe was also installed in borehole 1 to measure the groundwater level.

The field work was supervised throughout by a member of our engineering staff.

The locations of the borings, together with a stratigraphic section along the centerline of the proposed bridge are shown on Fig. 1. A detailed log of each borehole is given on the Record of Borehole sheets following the text of this report.

The samples obtained during this investigation were brought to our laboratory for detailed examination. The results of grading tests carried out on samples of the clayey silt and the glacial till at the site are shown on Figs. 2 and 3.

The elevations given in this report are referred to a spot elevation, the location of which was given as the intersection of Regional Road 4 and the road leading south from the bridge site. The elevation of this intersection point was given to us as 231.3 as referred to Geodetic datum.

SITE AND GEOLOGY

The site is located at the crossing of a tributary of the East Castor River by Regional Road No.4, about 1/2 miles west of Marvelville, Ontario. From available geological information it is known that the site is within the physiographic region known as the Winchester Clay Plain. A thin clay cover underlain directly by glacial till exists over much of this area. The underlying bedrock is shown on geological maps to be limestone of the Oxford formation.

SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in the boreholes is given on the Record of Borehole sheets and is illustrated on the stratigraphic section on Fig. 1. Following is a summarized account of the subsurface conditions at the site.

The boreholes were put down through the existing approach embankments and encountered some 5.5 ft. of embankment fill consisting of stiff grey-brown silty clay with some sand, gravel, and occasional boulder size material. Both the east and west approach embankment fills were found to be underlain by some 7 to 8 ft. of stiff grey silty clay. In borehole 1 the upper 4 ft. of the silty clay was weathered to a very stiff grey-brown crust. An Atterberg limit test carried out on a

sample of the grey clay from borehole 1 gave a liquid limit value of 68 and a plasticity index of 45. The corresponding natural water content was 59 percent. With depth in borehole 1, the silty clay becomes layered and more silty as evidenced by the results of a grading test as shown on Fig. 2.

A stratum of glacial till consisting mainly of silty sand with some clay and gravel size material was encountered below the silty clay. The surface of the glacial till as encountered in the boreholes was at depth of 4 to 6 ft. below stream bed level. The results of a grading test on a sample of the till recovered within the standard 1½ in. I.D. split spoon sampler are shown on Fig. 3. Although not encountered in the small diameter boreholes, cobble and boulder size material may be expected within the glacial till stratum. Standard penetration tests carried out in the glacial till gave N values of 96 and 100 blows/ft. indicating a very dense relative density.

Bedrock was encountered below the till in borehole 1 and was cored for a depth of about 5 ft. From the core recovered, the bedrock at the site is considered to be a sound grey limestone.

The water level in the stream at the time of the investigation was at about elevation 221. The ground water level as measured in the standpipe in borehole 1 and the open borehole 2 was some 1 to 3 ft. above the adjacent stream water level.

PROPOSED BRIDGE STRUCTURE

a) General

It is understood that it is planned to replace the existing narrow one span concrete structure with a new structure centered a few feet north of the existing bridge centerline. As presently planned, the new bridge will be a simply-supported prestressed concrete girder structure with a single clear span of about 30 ft. It is tentatively planned to raise the roadway grade some 1 to 2 ft. in the vicinity of the new bridge structure.

b) Foundations

It is recommended that the abutments for the proposed bridge at this site be founded on spread footings placed in the sandy silt till which underlies the site at a depth of 4 to 6 ft. below stream bed level. To provide some protection against scour, the footings should be taken down at least 4 ft. below

stream bed level. The N values obtained within the glacial till at the proposed foundation level were in excess of 50 blows/ft. Based on the measured N values, an allowable bearing pressure of 6,000 lb/sq.ft. may be used in the design of spread footings founded on the silty sand till. At this allowable bearing pressure, settlement of the bridge abutments should be negligible provided precautions are taken to prevent loosening of the glacial till at and below foundation grade as discussed below.

In the computation of sliding resistance between a rough concrete footing base and the undisturbed silty sand subsoil, a coefficient of friction of 0.40 may be used in design to which a safety factor of 1.5 should be added.

c) Abutments

Closed end abutments should be backfilled with a well compacted, free-draining and non-frost-susceptible granular material. Provision should be made for drainage from the backfill to prevent hydrostatic or ice pressure build up behind the abutment walls. With full effective drainage of the backfill, a coefficient of active earth pressure, $K_a = 0.3$ and a total unit weight of 135 lb/cu.ft. should be used for the compacted granular fill in the design of the abutments for this simply-supported structure.

d) Approach Embankments

It is understood that the approach embankments are to be raised by 1 to 2 ft. in the vicinity of the new bridge structure. This relatively minor grade raise will not effect the stability of the approach embankments, the safety factor of which will be adequate for 2 horizontal to 1 vertical side slopes. It is also recommended that the embankments be constructed of suitable granular material properly compacted in place. Protection of the side slopes, especially the east embankment where the stream makes a sharp bend, should be provided to some 2 ft. above maximum flood level.

e) Construction Procedures

Providing construction is carried out when the river water level is at or below normal, the excavations for the abutment footings will extend to some 5 to 7 ft. below river water level. Control of river water and groundwater inflow will be required for footing excavations in the essentially

granular till to prevent a reduction in the in situ density of the subsoil at and below foundation level. It should be possible to accomplish this control by pumping from properly filtered sumps. Dykes, consisting of the natural silty clay or sandy silt till, should be constructed on the river side of the excavations to divert the river water flow.

To prevent loosening of the till surface once foundation grade has been reached, it is recommended that the base of the footing excavations be covered with a layer of crushed stone once it is down to grade. It is recommended that the foundation excavations be inspected by a Soils Engineer to ensure the competency of the glacial till once it has been exposed in the footing excavations.

We trust that this report contains sufficient information for your design purposes. If we can be of any further service to you on this project, please call us.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.

R. A. Montgomery
R. A. Montgomery, P. Eng.

RAM/ml
72791
June, 1972.



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

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

(b) Cohesive Soils

<i>Consistency</i>	<i>c_u, lb./sq. ft.</i>
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 hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V field vane test

NOTES:

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

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

LIST OF SYMBOLS

I. GENERAL

π	= 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 discharge
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_s	coefficient of consolidation
T_v	time factor = ct/d^2 (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
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.

RECORD OF BOREHOLE 1

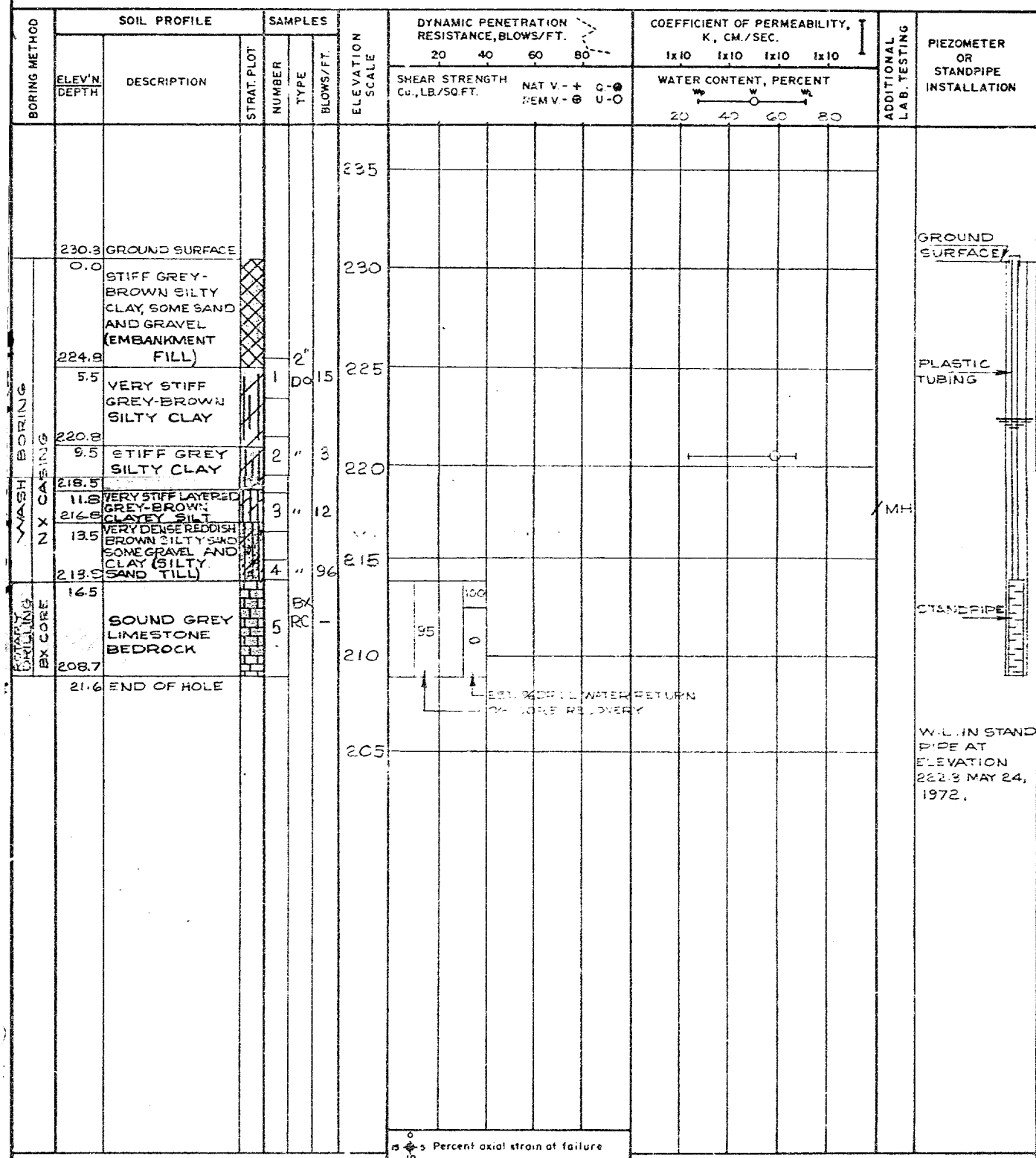
LOCATION See Figure 1

BORING DATE MAY 23, 1972

DATUM GEODETTIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN *G. S.*
CHECKED *W. J.*

RECORD OF BOREHOLE 2




LOCATION See Figure 1

BORING DATE MAY 24, 1972

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, K, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20	40	60	80	1x10	1x10	1x10	1x10		
								SHEAR STRENGTH Cu, LB./SQ.FT.		NAT. V. - + REM. V. - ⊕ Q - ⊕ U - ⊕		WATER CONTENT, PERCENT					
WASH BORING NX CASING		227.6 GROUND SURFACE					230										
	0.0	STIFF GREY-BROWN SILTY CLAY, SOME SAND AND GRAVEL OCCASIONAL BOULDER (EMBANKMENT FILL)					225										
	222.1																
	5.5	STIFF GREY SILTY CLAY		1	2"	5	220										
				2	"	7											
	214.8	VERY DENSE REDDISH BROWN SILTY SAND, SOME GRAVEL (SILTY SAND TILL)					215										
	212.8			3	"	100											
	14.8 END OF HOLE REFUSAL, PROBABLY BEDROCK					210											
								0 15 ⊕ 5 Percent axial strain at failure									
																W.L. IN OPEN HOLE AT ELEVATION 223.6 MAY 24, 1972.	

W.L. IN OPEN HOLE AT ELEVATION 223.6 MAY 24, 1972.

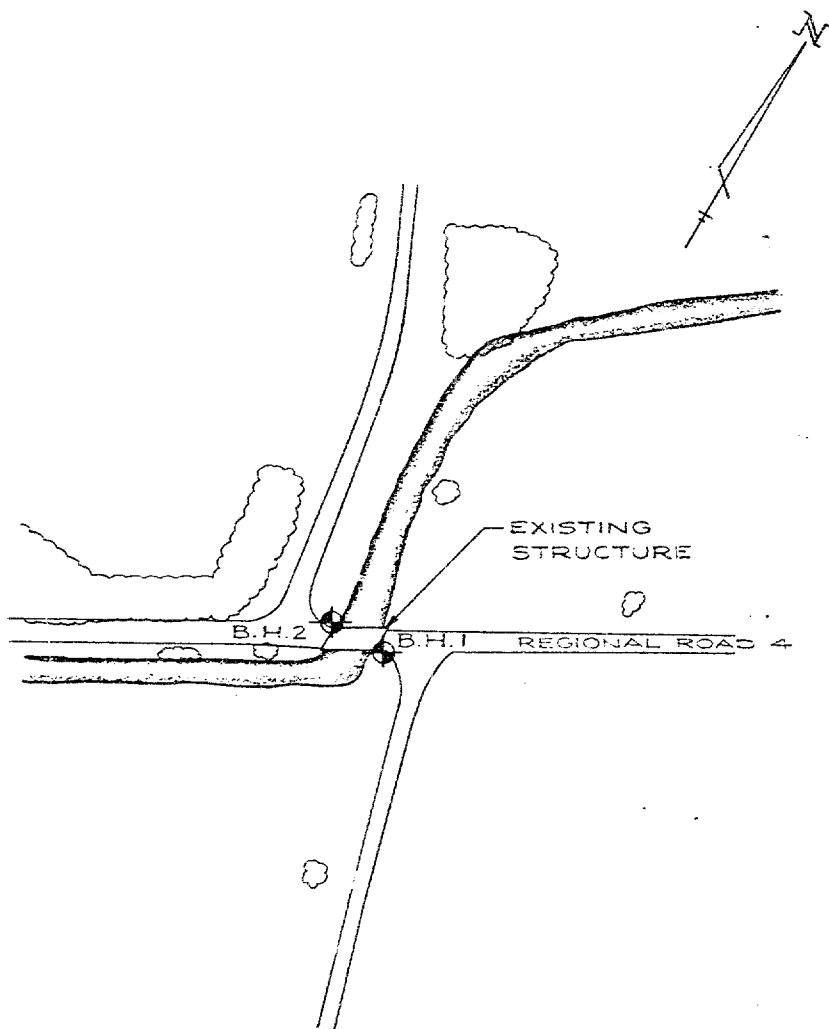
1 MH

0 15 5 Percent axial strain at failure 10

VERTICAL SCALE 1 IN. TO 5 FT.

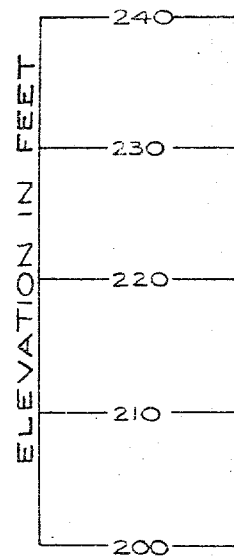
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DRAWN G.E. CHECKED PBA



PLAN

SCALE : 1" To 100'



STIFF
SAND
(EMB.)



VERY



STIFF



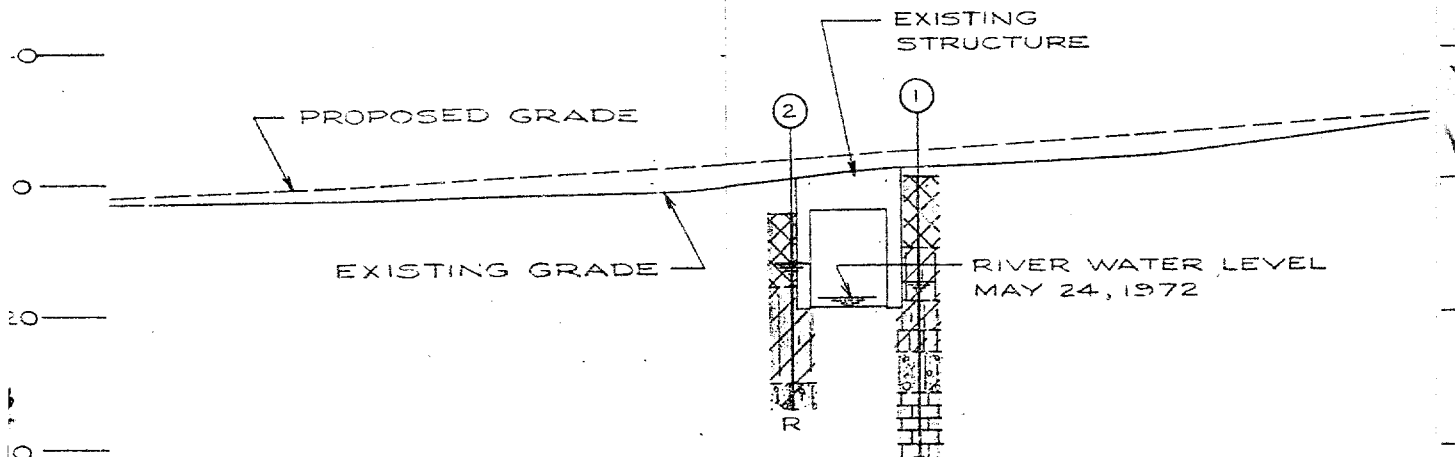
VERY
CLAY



VERY
SOME



SOUN



SECTION ALONG & ROAD

SCALE: HORIZ. 1" TO 50'
VIRT. 1" TO 10'

STRATIGRAPHY

STIFF GREY BROWN SILTY CLAY, SOME
SAND AND GRAVEL, OCCASIONAL BOULDERS
(EMBANKMENT FILL)

VERY STIFF GREY BROWN SILTY CLAY

STIFF GREY SILTY CLAY

VERY STIFF LAYERED GREY BROWN
CLAYEY SILT

VERY DENSE REDDISH BROWN SILTYSAND,
SOME GRAVEL AND CLAY (SILTY SAND TILL)

SOUND GREY LIMESTONE BEDROCK

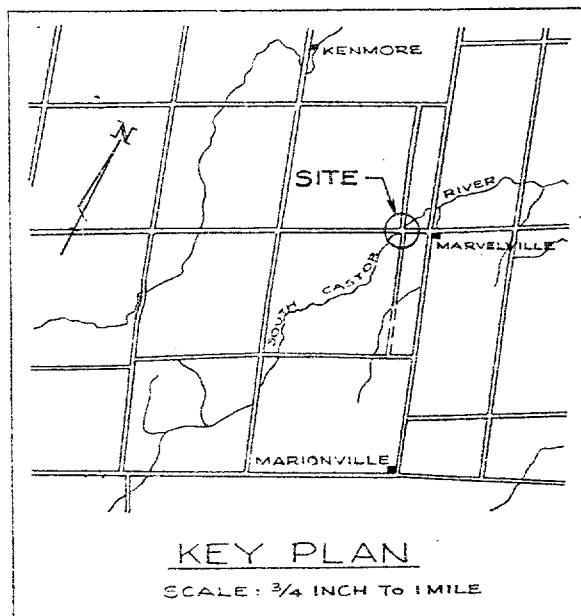
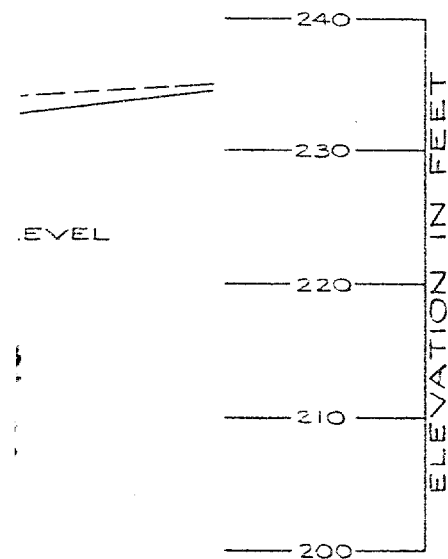
LEGEND

- BOREHOLE IN
- BOREHOLE IN
- WATER LEVEL
MAY 24, 1972
- R REFUSAL, PRO

SPECIAL NO
THIS DRAWING IS TO BE RE
WITH ACCOMPANYING REPO

BORING PLAN AND SOIL STRATIGRAPHY

FIGURE 1



LEGEND

BOREHOLE IN PLAN

BOREHOLE IN ELEVATION

WATER LEVEL IN ELEVATION,
MAY 24, 1972

REFUSAL, PROBABLY BEDROCK

REFERENCE:

SITE PLAN AND PROFILE SUPPLIED BY
A.J. GRAHAM ENGINEERING
CONSULTANTS LTD.

31G-30

GEOCRE No.

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.

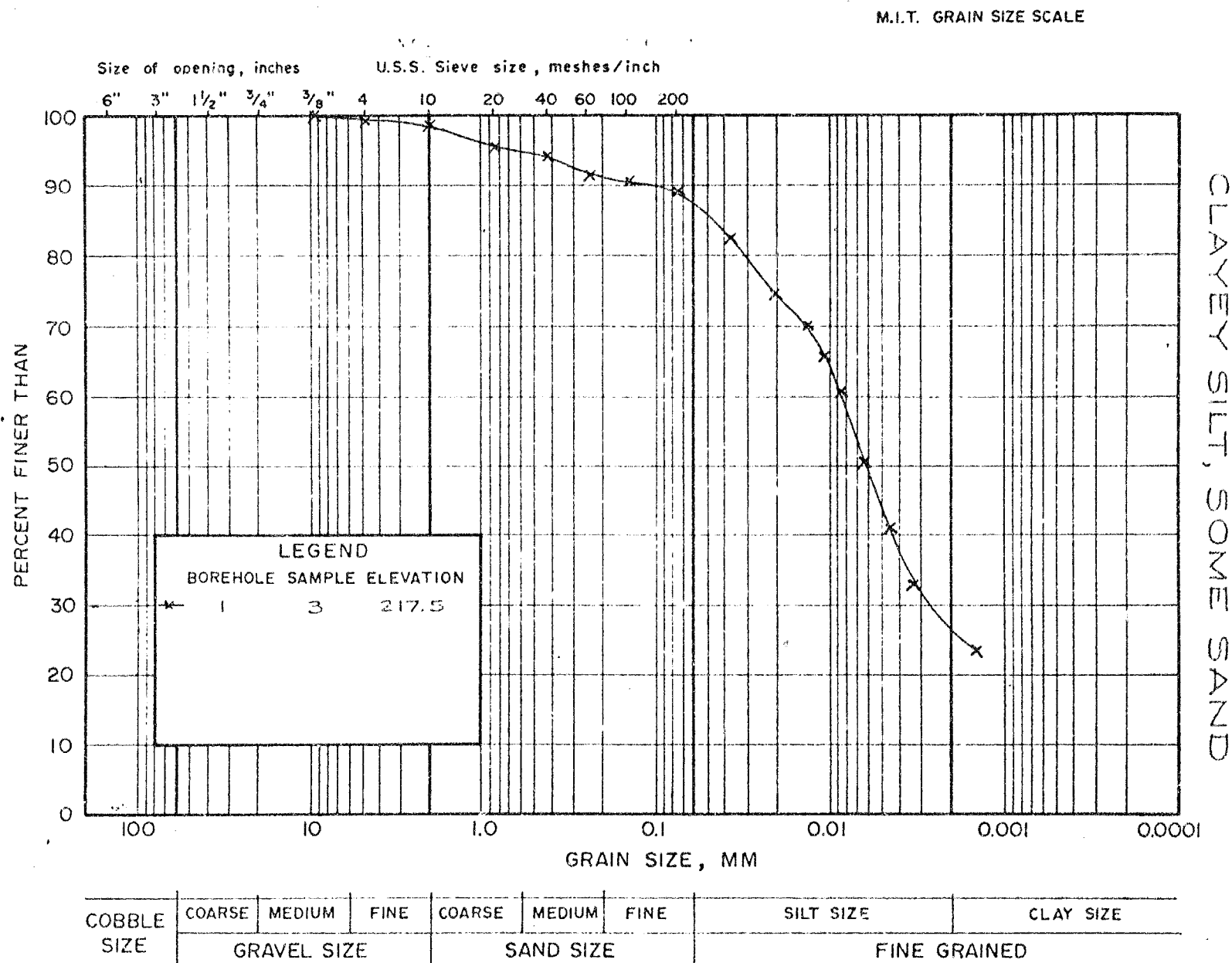
SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

Date JUNE 14, 1972

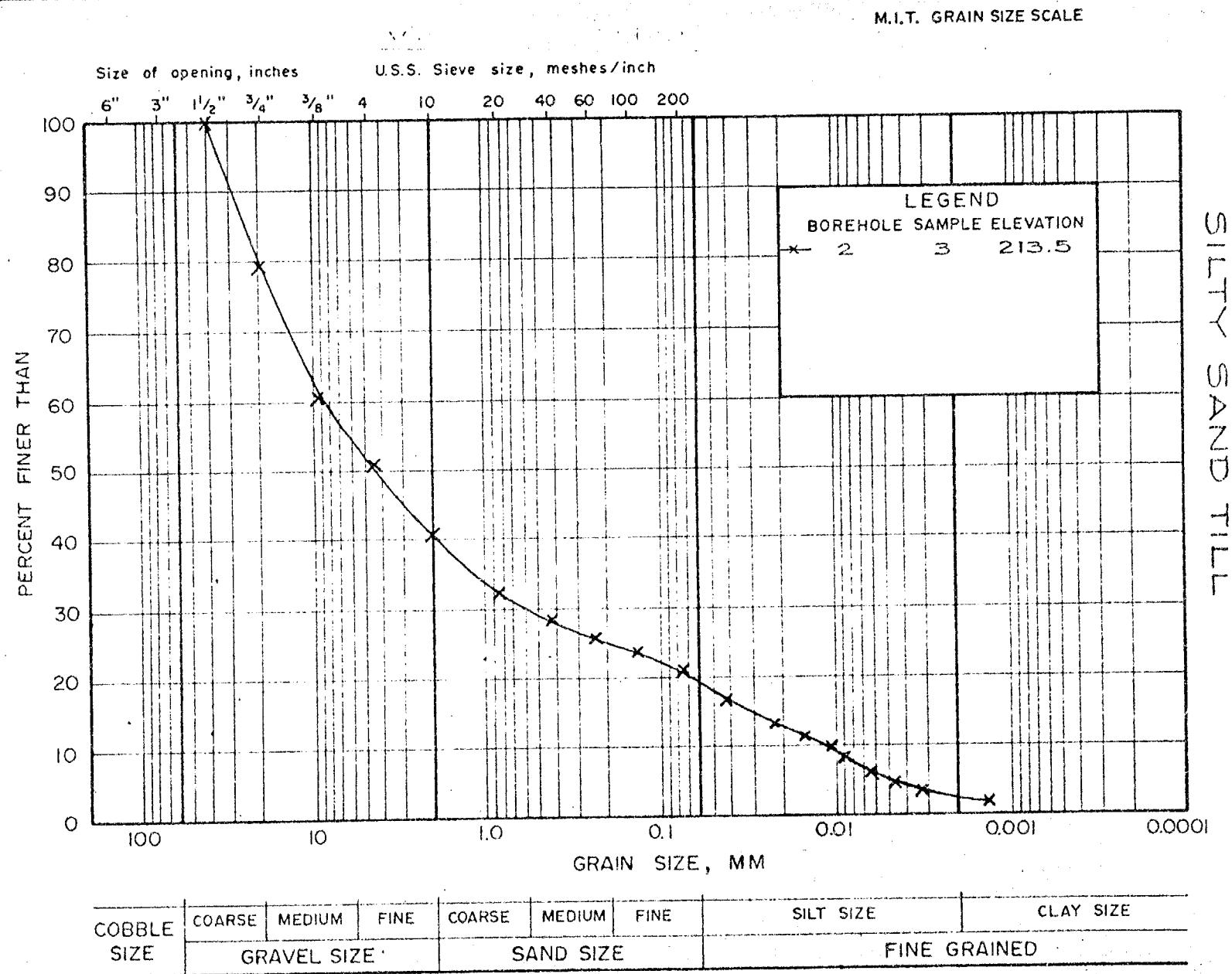
Golder Associates

Drawn D.A.
Chkd. RA
Appd. _____



GRAIN SIZE DISTRIBUTION

FIGURE 2



GRAIN SIZE DISTRIBUTION

FIGURE 3