#64-F-276M

SUNNINGDALE

BRIDGE OVER

MEDWAY RIVER

LOT 19, CON. V/VI

LONDON TWP.

Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.,
Bridge Division.

Attn: Mr. G.C.E. Burkhardt,
Mun. Bridge Checking Engr.

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

March 11, 1965

Your Memo -- March 10/65

Township of London, Sunningdale Bridge over Medway River, Lot 19, Con. V/VI, County of Middlesex, Structure Site No. 19-235, Your File No. BA 1958.

With reference to your memo of March 10, 1965, regarding the above structure, we herewith submit our comments for your consideration:

The factual soil information in the report is well presented and, in our opinion, adequate. The most significant feature at this site is the presence of artesian conditions which govern the stability of the banks and have also a decisive influence on the choice of the future structure. In connection with this phenomenon, we would like to mention that due to the fact that artesian conditions were not recognized and therefore not considered, two structures in that same general area have failed to the extent that additional spans had to be added.

It is also our opinion that the box culvert represents the most straightforward solution to the relatively difficult problem. The construction problems are minor, and the final product carries the least amount of "unknowns".

In your memo you mention that the culvert is not a good solution from the hydrological point of view. However, it does represent a good solution from the foundation point of view and, therefore, a reconciliation of the two requirements has to be sought. If scour is the problem, we believe that the driving of interlocking steel sheet piling could provide the necessary solution.

We hope that the above will help you to arrive at a satisfactory conclusion. Should there be any additional questions that you would like to discuss, please feel free to call on our Office.

AGS/MdeF

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

P.S. -- The plans are being returned as requested, under separate cover.

MEMORANDUM

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

From: Bridge Division,
Downsview, Ontario

Date: March 10, 1965

Our File Ref.

In reply to

Subject: Township of London, Sunningdale Bridge over Medway River, Lot 19, Con. V/VI, County of Middlesex Structure Site No. 19-235, Our File No. BA 1958

Attached please find one (1) copy of the Foundation Report, by H. Q. Golder and Associates Limited, and one (1) copy of the plans for the above mentioned structure.

The designer has chosen a 3 span concrete box culvert, mainly due to the recommendations outlined in the soils report, but a 3 span box culvert is not a good solution for this site from the hydrology viewpoint. We therefore would appreciate it very much, if we could have your comments, re. the Foundation Report, at your earliest convenience.

For your added information, the designer of the structure, Mr. N. W. Warner, from R.C. Dunn and Associates Limited, has been in contact with Mr. L. G. Soderman, from H.Q. Golder and Associates Limited, re. the driving of sheet piles as scour protection at both ends of the culvert after the construction of the structure. Mr. N. W. Warner was told that no boiling of the soil would occur, provided that the piles are left in place.

Since we do not have enough copies of the plan we would appreciate it very much, if you could send the plans back to us which we are forwarding to you today, as soon as you are finished using them.

G.C.E. Burkhardt

G.C.E.Burkhardt, P.Eng.,
Municipal Bridge Checking Engineer

GCEB/m

B.A. 1958

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
767-9201
763-4103

REPORT

TO

R. C. DUNN & ASSOCIATES LIMITED

ON

SITE INVESTIGATION

PROPOSED SUNNINGDALE BRIDGE

LONDON TOWNSHIP ONTARIO

64-F-276M

Distribution:

10 copies - R. C. Dunn & Associates Limited,
London, Ontario.

2 copies - H. Q. Golder & Associates Ltd.,
Toronto, Ontario.

October, 1964 64094

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FIGURE 1 Boring Plan and Soil Stratigraphy
2 Grain Size Distribution
3 Grain Size Distribution
4 Stability Analyses
5 Stability Analyses

ABSTRACT

The site of the proposed Sunningdale Bridge is located in a broad shallow valley on the road between concessions V and VI south of Arva, Ontario. The soil and water conditions were investigated by means of 4 boreholes. Piezometers were installed in three of these borings to measure the artesian pressures.

The soil conditions at the site consist of a surface veneer of loose river and flood plain deposits overlying a variable thickness of clayey silt and silt till. The till forms a seal over underlying silt and sand strata that carry artesian water. This artesian water is the most significant feature of the site in that it has a pronounced influence on the stability of the bank slopes.

In addition to the tentatively proposed cross section, which was found to be unstable, 4 alternate designs were considered. These include 1) flattening the rip rap slopes and designing a 3 span structure on 2 piers and 2 abutments, 2) founding the abutments on piles and employing a single span, 3) installing drainage trenches to relieve some of the artesian pressure and founding a single span structure on spread footings, and 4)constructing a box culvert.

From both engineering and economic view points, it is considered that alternates 3 and 4 are most applicable to the site. Design details are presented in the report.

INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained to carry out a site investigation at the proposed Sunningdale Bridge on the road between concessions V and VI in London Township south of Arva, Ontario. The purpose of this investigation was to determine the soil conditions at the site and make recommendations on the design of foundations for the structure. The results of the investigation and the recommendations made are contained herein.

PROCEDURE

Three boreholes were put down at the site between August 7 and 17, 1964 to depths of 46 to 61 feet using a machine drillrig. A fourth hole was put down to a depth of 31 feet on September 24 and 25, 1964 to obtain additional information on the complex water conditions prior to final design. Standard two inch split spoon samples were taken at 5 foot intervals in each hole. Piezometers with seals were installed in the three initial holes upon completion.

The locations of the borings are shown on the site plan on Figure 1 along with a section showing the inferred soil stratigraphy. Detailed logs of each borehole are given on the Records of Boreholes at the end of the report.

The elevations of the initial three holes were supplied by R. C. Dunn and Associates Limited and are believed to be referred to a local datum. The elevation of borehole 4 was referenced to borehole 3.

SITE AND GEOLOGY

At the location of the present and proposed bridges, Medway Creek forms a small channel in a broad shallow valley, probably formed by an earlier, considerably larger stream. Valleys of this type generally contain a variable thickness of river deposits and flood plain material underlain in this area by glacial till. It is understood that the present bridge was originally located on a slight bend in the river and at times scour was a problem. A new staight channel was therefore dredged immediately north of the bridge to eliminate this problem. The channel below the present bridge is therefore considerably wider and deeper than it is downstream of the bridge.

SUMMARIZED SOIL CONDITIONS

The soil conditions at the site are detailed on the borehole logs and summarized on the stratigraphic section shown in Figure 1. Soil properties used for the design computations are summarized in the text and on the borehole logs.

Organic Flood Plain Deposits

The floor of the valley is probably covered at most locations by a variable thickness of loose or soft organic silts or sands similar to the upper 5 feet of soil encountered in boreholes 2 and 4 and the upper 2 feet in borehole 1. These deposits were probably laid down by the present stream during various flood stages. Site preparation during construction will include removal of these loose soil types from beneath the structure foundations and approach embankment fill.

Very Stiff Laminated Clayey Silt

In areas where the present river has not caused extensive erosion, it is probable that the organic flood plain deposits are underlain by stiff to very stiff laminated clayey silts similar to the 5 foot layer encountered below the top 2 feet of sand in borehole 1. Visual inspection of the clayey silt exposed in bank slopes downstream of the existing bridge indicated that it is highly fissured. On the basis of tactile examination and two standard penetration resistance values of 24 and 40 blows per foot the stratum is estimated to have an in situ shear strength of about 2,000 lb/sq.ft.

Two Atterberg limit tests run on samples gave liquid limits of 33 and 35 and corresponding plasticity index values of 15 and 17.

For purposes of design, the stratum was assumed to have an in situ unit weight of 135 lb/cu.ft., a drained angle of internal friction of 28 degrees and a drained cohesion intercept of zero.

River Sand and Gravel

At the locations of boreholes 2, 3 and 4 the river flow has eroded away the clayey silt stratum and left behind a variable thickness of loose to compact coarse gravelly and bouldery sand. Three grain size curves for these deposits are shown on Figure 2. In borehole 4, the lower portion of the stratum was very silty. At this point, the river deposits directly overlie the laminated silt stratum and it is possible that artesian water seeping upward from this stratum has brought in silt with it.

For purposes of design the river deposits have been assumed to have an in situ unit weight of 125 lb/cu.ft. and a drained angle of internal friction of 30 degrees.

Clayey Silt Till

A thin layer of clayey silt till underlies the river sands in borings 2 and 3, thickening to the east to about 15 feet in borehole 1 where it underlies the laminated clayey silts. At borehole 4, the till appears to have been completely eroded away and replaced by silty river sands and gravel. Two grain size curves obtained on till samples are plotted on Figure 3.

At the location of borehole 1, the till is estimated to be very stiff to hard, having in situ moisture contents below the plastic limit and standard penetration resistances of 22 to 47 blows per foot. In boreholes 2 and 3, however, two standard penetration resistance values of 8 blows per foot were obtained in the till indicating that, where thin, it appears to have softened under the influence of the upward flowing artesian water from the underlying laminated silt stratum. On the basis of the above observations the till is estimated to have an in situ shear strength varying from about 500 lb/sq.ft. where thin in boreholes 2 and 3 to about 3,000 lb/sq.ft. where thick in borehole 1.

Three Atterberg limit tests run on till samples gave liquid limit values varying from 32 to 36 and corresponding plasticity indices varying from 14 to 18. The three natural moisture contents for these samples averaged 17 percent, and were all below the plastic limit.

For purposes of engineering design, the till was assumed to have a unit weight varying from 125 lb/cu.ft. where soft to 135 lb/cu.ft. where very stiff. On the basis of experience with similar materials in this area, the drained angle of shearing resistance was taken as 28 degrees and the cohesion intercept as zero.

Stratified Sandy Silt

As shown on the stratigraphic section, a stratum of laminated silt varying in thickness from about 15 to 22 feet underlies the clayey silt till. A group of grain size curves for this stratum are shown on Figure 2. A close inspection of the grain size data indicated that the silt becomes increasingly sandy with increasing depth. Visual examination of the samples showed that a thin layer of hard clay about 1/16 to 1/4 inches in thickness occurred about every 18 inches in the upper 10 feet or so of the stratum.

Standard penetration resistance or "N" values varying from 26 to greater than 100 blows per foot were obtained. Although variable, the blows were generally high, averaging 52 blows per foot. Much of the variation is probably caused by artesian water in the stratum. The influence of this water is best shown on the log of borehole 4 where the "N" values are very high in the top fine grained part of the stratum, dropping to lower values in the coarser grained part of the stratum where the permeability was high enough to cause piping and a rapid rise in water up the drill casing.

For purposes of engineering design, the sandy silt stratum was assumed to have an in situ weight of 130 lb/cu.ft. and an average angle of internal friction of 35 degrees.

Very Dense Sand and Gravel

A layer of very dense grey to brown silty angular sand and gravel underlies the silt stratum. One grain size analysis run on this material is plotted on Figure 3. Standard penetration resistance values in excess of 60 blows per foot were generally obtained although two low values of 41 and 36 were obtained in borehole 1.

This stratum also carries artesian water as discussed under Water Conditions.

Hard Grey Silt Till

The lower-most stratum encountered in the holes was a hard grey clayey silt till having standard penetration resistance values generally in excess of 100 blows per foot. The upper boundary of this layer varies from about elevation 112 to 115.

WATER CONDITIONS

During drilling operations, artesian water was encountered in the stratified sandy silt stratum in all four boreholes. In borehole 1, artesian water rose up the drill casing to a point about one foot above ground level when the hole penetrated into the lower sand and gravel stratum at a depth of about 37 feet. In borehole 4, water rose up the casing to a level about 4 feet above ground level when the hole had penetrated about 6 feet into the stratified sandy silt stratum. The water rose to a similar elevation each time the hole was advanced to the next sampling depth. In borehole 2, the water level rose in the casing to a point 4 feet above ground surface when the hole penetrated through the thin till layer into the silt stratum. This particular water level was at elevation 162.5 and was the highest level recorded at the site during the period of investigation.

Piezometers were installed in the three initial boreholes as shown on the borehole logs. On August 20, 1964, both piezometers in borehole 1 recorded a water level 1.4 feet below ground surface (elevation 161.0). In borehole 2, the piezometric water level had dropped to a level about 1.0 foot above ground surface or to elevation 159.5. In borehole 3, the piezometer recorded a water level 6.0 feet below the bridge deck or at elevation 160.7.

On September 3, 1964, the writer paid a visit to the site and observed a cloud of light brown silt issuing from the area around the piezometer tube into the stream (borehole 3).

On the basis of the above observations it is believed that there is artesian water in all parts of the stratified sandy silt stratum and in the underlying coarse sand and gravel layer. The layer of clayey silt till probably forms the major seal above the silt stratum although the very thin hard clay layers in the silt stratum probably also contribute.

The observations indicate that the piezometric or artesian water level might be as high as elevation 165 which is about 9 feet above the present river level.

Although the piezometric water level is believed to be normally at about elevation 162, provision has been made in the design to handle water pressures as high as elevation 165.

DISCUSSION

It is understood that the bridge as proposed is a simply supported 2 span structure having an overall length of about 170 feet. The deck of the bridge and the approach embankments are to be at elevation 177 if possible, which is about 10 feet higher than the existing bridge.

From an engineering point of view, the most significant feature of the site is the presence of artesian water in the sandy silt stratum below the upper till. Stability analyses on the existing abutments give a factor of safety less than unity and suggest that the abutments may be creeping. Field observations confirmed that there has been closure of the west abutment against the bridge girders.

Stability considerations influenced by the soil and water conditions indicate that it is desirable to modify the proposed structure. The alternative design schemes considered, all of which eliminate the centre pier, are as follows:

I) Flattening the rip rap slope and designing a 3 span structure on two piers and two abutments.

II) Founding the abutments on piles driven to refusal in the lower till and employing a single 120 foot span.

III) Founding the abutments on spread footings and employing a single 120 foot span. This scheme would also require removal by excavation of some of the till stratum at each abutment to enable relief by drainage of some of the artesian pressure in the silt stratum.

IV) Consideration of a box type culvert as distinct from a bridge.

From both economic and engineering points of view the latter two schemes appear to be the most advantageous. The details are discussed below.

Originally Proposed Bridge Structure

Stability computations performed on the originally proposed cross section are shown on Figure 4a. The stability of the lower part of the rip rap slope is inadequate over the long term (factor of safety = 0.5), and the overall slope has an inadequate long term factor of safety of 1.0.

In addition, construction difficulties which would be induced by the artesian conditions make it desirable to eliminate the centre pier. Computations show that prevention of piping or quick conditions at the centre pier for either a spread footing or a pile cap constructed in the dry would require either an expensive external dewatering system or a deep sheet pile penetration which would be impracticable to achieve by driving in the dense soils at the site.

If a design incorporating a centre pier must be put forward, it would be possible to support this pier on steel H piles or cast-in-place expanded base piles bearing on the lower till. Pile cap construction would involve the placing of tremie concrete at the base of a water filled sheet pile cofferdam driven to a depth sufficient to prevent uplift within the excavation. That piping would indeed occur without these precautions was confirmed visually by the piping occurring into the stream at borehole 3 on September 3, 1964 (see water conditions). Scour protection for the cap would be provided by the sheet piling which would be left in place. We would be pleased to carry out further computations and supply additional design details if it is necessary to construct a centre pier.

Alternate I. - Three Span Structure

This scheme as envisaged would involve lengthening the total span and flattening the rip rap slopes to obtain greater stability. Computations indicate that slopes as flat as 3 horizontal to 1 vertical do not have adequate long term stability because of the combination of artesian water pressures and soil types that exist at this site.

Alternate II. - Single Span Structure - Abutments on Piles

Consideration was given to founding the abutments on steel H piles and employing a single span 120 feet in length. The scheme is illustrated on Figure 4(b).

The abutments would be founded on steel H piles driven vertically to practical refusal in the bottom till or the overlying dense sand and gravel stratum. Safe working loads of 50 tons are estimated for piles driven to this depth. Lateral thrust on the abutment would be resisted by a row of H piles at 2.5 foot centres driven on a 1 on 4 batter as shown.

In Table I, the results of stability computations for this scheme are summarized. In order to obtain an adequate margin of safety the grade has been lowered from elevation 177 to 172. With this grade, one row of batter piles at 2.5 foot centres would be adequate.

TABLE I

ALTERNATE II

STABILITY - SINGLE SPAN STRUCTURE - ABUTMENTS ON PILES

Deep Circle under Abutments (see Figure 4b of report)
Final road grade lowered to elevation 172

Design Assumptions		Factor of Safety	
Batter Piles Included	Piezometric Water Level	West Abutment	East Abutment
No	165	1.0	0.95
No	162	1.2	1.1
*1 row @ 2.5 ft.	165	1.2	1.2
*1 row @ 2.5 ft.	162	1.5	1.4
2 rows @ 2.5 ft.	165	1.5	1.5
2 rows @ 2.5 ft.	162	1.7	1.6

*Recommended design

The wing walls necessary to retain the embankment fill should also be founded on a similar system of piles. Well compacted, clean, free draining granular backfill should be used behind the abutment wall. In order to decrease the weight of the fill and reduce the lateral thrust somewhat it would be advantageous to use light weight cinder fill if available.

A suitably rip rapped berm consisting of free draining granular fill should be placed for a distance of at least 15 feet out in front of the abutments to resist the artesian uplift pressures. The stone used for the rip rap should be large enough to guarantee the prevention of erosion in this part of the river channel.

The overall spacing between the abutments can be altered to obtain a shorter or larger span as desired within limits imposed by hydraulic requirements.

Alternate III. - Single Span Structure - Abutments on Spread Footings

In order to found the abutments on spread footings and provide adequate safety against long term movement of the abutments without installing excessively large berms, it would be necessary to install a drainage system to relieve the artesian pressure in the silt stratum. The drainage scheme shown on Figure 5 involves excavating a portion of the till at each abutment and backfilling the area with sand and gravel fill.

As can be seen from the drawing, the softened claysy silt till would be completely removed from below the west abutment which would then be founded directly on the sand and gravel backfill. A cantilever wall similar to that shown would be a suitable structure and could be designed using an allowable soil bearing value of 2 tons/ sq.ft. The overall factor of safety of this abutment with the 15 foot berm installed as shown on Figure 5 would be about 1.2.

At the location of the east abutment the very stiff clayey silt stratum and the silt till stratum have a combined thickness of about 20 feet. Excavation to the silt stratum below this abutment would have to extend to about elevation 140 which is about 12 feet below the river level. The very stiff consistency of the till and the depth of the excavation would make this a very difficult operation. A different scheme is presented on Figure 5 for the east abutment, namely, extending the berm to a total width of 30 feet and excavating to the silt stratum below the berm rather than below the abutment. The abutment itself would be founded on the till stratum using an allowable bearing value for design of 2 tons/sq.ft. In view of the different stress-strain characteristics of the very stiff silt till and the compact sand and gravel filling the trench, it is recommended that the trench slope in the till be cut at 2 horizontal to 1 vertical to minimize shearing strains thereby reducing the possibility of softening of the till under the abutment and embankment loads. Using the foregoing precautionary measures the abutment would have a long term factor of safety against shallow failure of about 1.2 and against deep seated failure of greater than 1.3.

The berms should consist of free draining granular backfill suitably rip rapped to prevent erosion at flood stage.

It should be noted that this design involves a lowering of the final road grade to elevation 172 in order to ensure an adequate factor of safety on long term slope stability.

Alternate IV. - Box Type Reinforced Concrete Culvert

A reinforced concrete box culvert is considered to be the least problematical type of structure for this particular site. A box culvert design would obviate many of the difficulties foreseen in connection with the design and construction of a conventional bridge, e.g. (1) drainage system to relieve artesian pressures in the abutment areas, (2) piping occurring at the centre pier location, and (3) grade reduction to ensure approach embankment stability.

The proposed grade (i.e. elevation 177.0 feet) need not be reduced if a box culvert design is made. The overall length of the culvert should be such that the ends extend into the existing river banks. If this is not done it may be necessary to provide short berms at right angle to road centre line to ensure stability of the approach fills. The decision to place approach fill berms can best be made during construction when excavation has progressed sufficiently to allow inspection of soil conditions in the approach fill areas.

The base of the culvert should be at least 2 feet thick and reinforced top and bottom to provide adequate resistance to the shear stresses imposed by the embankment. The fill placed next to the culvert and in the area close to the river bank should consist of well compacted free draining non-frost susceptible granular material. Side slopes must be rip rapped to an elevation at least 3 feet above maximum flood elevation.

Approach Embankments

The approach fills for the bridge are presently designed with 2 on 1 side slopes. Computations indicate that these slopes should be satisfactory provided all soft organic surface materials are stripped off prior to placing new fill.

Visual inspection of the flood plain area showed it to contain depressions, especially north of the present roadway, which may be filled with soft organic deposits. Special care should be taken to remove these materials if encountered during construction. If the deposits are deep enough that their removal causes local piping, it may be necessary to place small berms rather than remove all of the soft material. Design of these berms should be deferred until the local soil conditions are determined during construction. In any event, it is recommended that the land surface at the toe of the embankments be brought to elevation 160 which corresponds approximately to the present valley floor elevation.

SUMMARY AND CONCLUSIONS

1. The most significant feature of the site is the presence of artesian water in the sandy silt stratum immediately below a thin layer of glacial till.

2. Stability of the existing structure is believed to be marginal. The existing abutments are believed to be creeping inwards as evidenced by closure of the abutments against the ends of the bridge girders.

3. Stability considerations make it necessary to modify the proposed design. The alternate schemes presented all eliminate the centre pier as it would be very difficult and costly to build for the reasons outlined in the report.

4. From both economic and design points of view it is believed that either Alternate III or Alternate IV provides a satisfactory solution to the stability problems at the site. Alternate III consists of providing drainage trenches to relieve some of the artesian pressures and founding the structure on spread footings. Alternate IV is a rigid concrete box culvert design. Details of these and the other alternates are given in the report.

for R. M. Quigley, P. Eng.

October, 1964
64094

for L. G. Soderman, P. Eng.

GOLDER & ASSOCIATES

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 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_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	in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	effective angle of shearing resistance, or friction	
c_u	apparent cohesion*	in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
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.

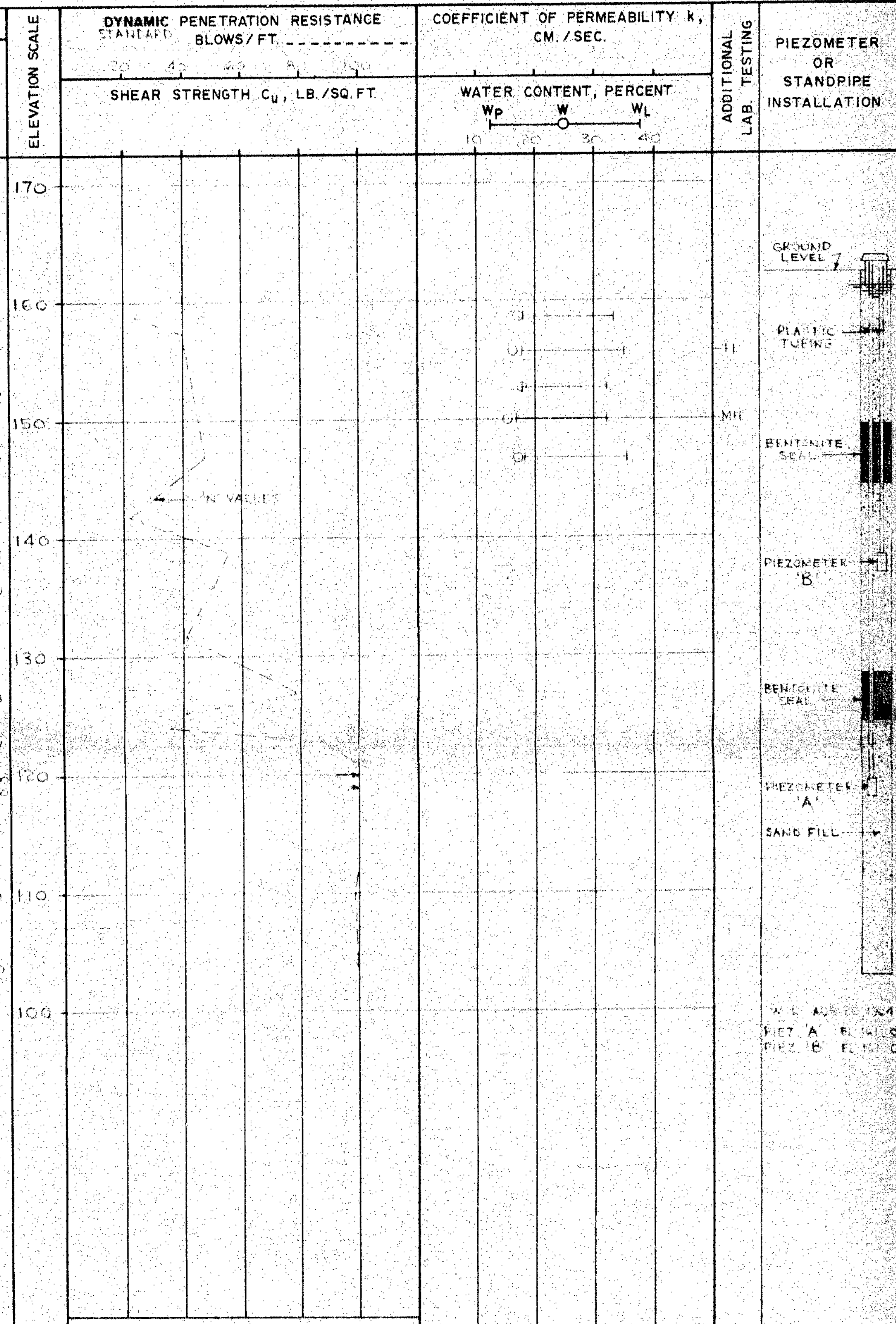
RECORD OF BOREHOLE 1

LOCATION See Figure 1 | BORING DATE AUG. 7-12, 1964 | DATUM LOCAL

BOREHOLE TYPE WASH BORING | BOREHOLE DIAMETER BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES | PEN. TEST HAMMER WEIGHT --- LB. DROP --- INCHES

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.
162.4	GROUND LEVEL			
0.0	LOOSE BROWN SAND			
2.0	VERY STIFF BROWN CLAYEY SILT	1	2" D.O.	24
		1A	CA.	40
		2	D.O.	—
155.4 / 7.0	VERY STIFF BROWN TO GREY CLAYEY SILT TILL	3	"	43
		4	"	45
		5	"	47
		6	"	22
140.4 / 22.0	DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT	7	"	55
		8	"	50
		9	"	41
		10	"	78
126.4 / 36.0	VERY DENSE GREY SAND AND GRAVEL	11	"	41
		12	"	36
		13	"	87
		14	"	>100
		15	"	>100
112.1 / 50.3	HARD GREY CLAYEY SILT TILL	16	"	99
		17	"	100
102.9 / 59.5	END OF HOLE			



VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED

RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE AUG. 12 13, 1964 DATUM LOCAL

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX & NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

ELEVN. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.
158.5	GROUND LEVEL			
0.0	LOOSE BROWN ORGANIC SILTY SAND	1	2" D.O.	5
153.5 / 5.0	LOOSE TO COMPACT BROWN GRAVELLY SAND	2	"	19
148.5		3	"	8
147.0 / 11.5	FIRM TO STIFF GREY CLAYEY SILT TILL	4	"	78
		5	"	67
	DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT	6	"	55
		7	"	47
125.5 / 33.0	VERY DENSE GREY SAND AND GRAVEL	8	"	87
		9	W.S.	—
		10	D.O.	>100
115.5 / 43.0	HARD GREY CLAYEY SILT TILL	11	"	>100
112.2 / 46.3	END OF HOLE			

STRAT. PLOT | ELEVATION SCALE: 160, 150, 140, 130, 120, 110

~~DYNAMIC~~ STANDARD PENETRATION RESISTANCE BLOWS/FT. ----------
20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.

N-VALUES

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT
W_P W W_L
10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

GROUND LEVEL
CEMENT SEAL
PLASTIC TUBING
BENTONITE SEAL
PIEZOMETER
SAND FILL
W.L. IN PIEZOMETER AT EL. 154.5 AUG. 1964

15 ○ 5 Percent axial strain at failure
10

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED

RECORD OF BOREHOLE 3

LOCATION See Figure I | BORING DATE AUG. 13-14, 1964 | DATUM LOCAL

BOREHOLE TYPE WASH BORING | BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES | PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

Soil Profile

ELEVN. / DEPTH	DESCRIPTION
166.7 / 0.0	EXISTING BRIDGE DECK
155.3 / 11.4	RIVER LEVEL
152.3 / 14.4	RIVER BOTTOM
	LOOSE BROWN SAND, GRAVEL AND BOULDERS
147.7 / 19.0	FIRM TO STIFF GREY CLAYEY SILT TILL
143.7 / 23.0	DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT (OCCASIONAL CLAY AND SAND LAYERS)
121.7 / 45.0	VERY DENSE BROWN SAND AND GRAVEL
114.9 / 51.8	HARD BROWN CLAYEY SILT TILL
105.7 / 61.0	END OF HOLE

Samples

NUMBER	TYPE	BLOWS / FT.
1	2" S.S.	8
2	"	8
3	"	26
4	"	53
5	"	45
6	"	99
7	"	>100
8	"	>100
9	"	>100
10	"	>100
11	"	>100

ELEVATION SCALE: 170, 160, 150, 140, 130, 120, 110, 100

DYNAMIC PENETRATION RESISTANCE
STANDARD BLOWS / FT. ----------
20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT
W_P W W_L
10 20 30 40

ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

PLASTIC TUBING
RIVER BOTTOM
SAND FILL
BENTONITE SEAL
PIEZOMETER
GRAVEL FILL
W.L. IN PIEZOMETER AT EL. 167 AUG. 25, 1964

15-○-5 Percent axial strain at failure
10

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED

PROJECT No. 64224

RECORD OF BOREHOLE 4

LOCATION See Figure 1 BORING DATE SEPT. 24 1964 DATUM LOCAL

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

ELEVN. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE
					160
156.2	GROUND LEVEL				
0.0	LOOSE ORGANIC SILTY SAND AND GRAVEL	1	2" O.D.	6	
		2	"	5	
151.2 / 5.0	COMPACT BROWN TO GREY SILTY SAND AND GRAVEL	3	"	20	150
		4	"	10	
		5	"	41	
144.2 / 12.0	VERY DENSE GREY STRATIFIED SILT BECOMING SANDY AND LESS DENSE WITH DEPTH	6	"	75	
		7	"	65	
		8	"	56	140
		9	"	28	
		10	"	36	
		11	"	31	130
125.7	VERY DENSE GREY SILTY SAND AND GRAVEL				
124.7 / 31.5	END OF HOLE	12	"	64	
					120

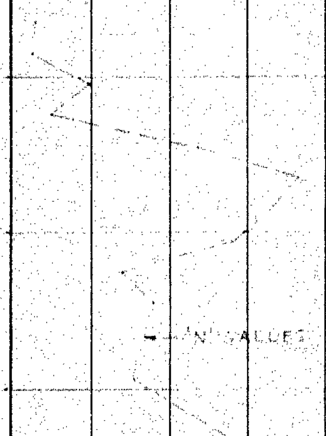
STANDARD PENETRATION RESISTANCE BLOWS/FT. 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING



PIEZOMETER OR STANDPIPE INSTALLATION

W.L. @ EL. 1[illegible] WITH CASING AT 18' DEPTH SEPT. 24 1964

W.L. @ EL. [illegible] WITH CASING AT [illegible] DEPTH SEPT. 25 1964

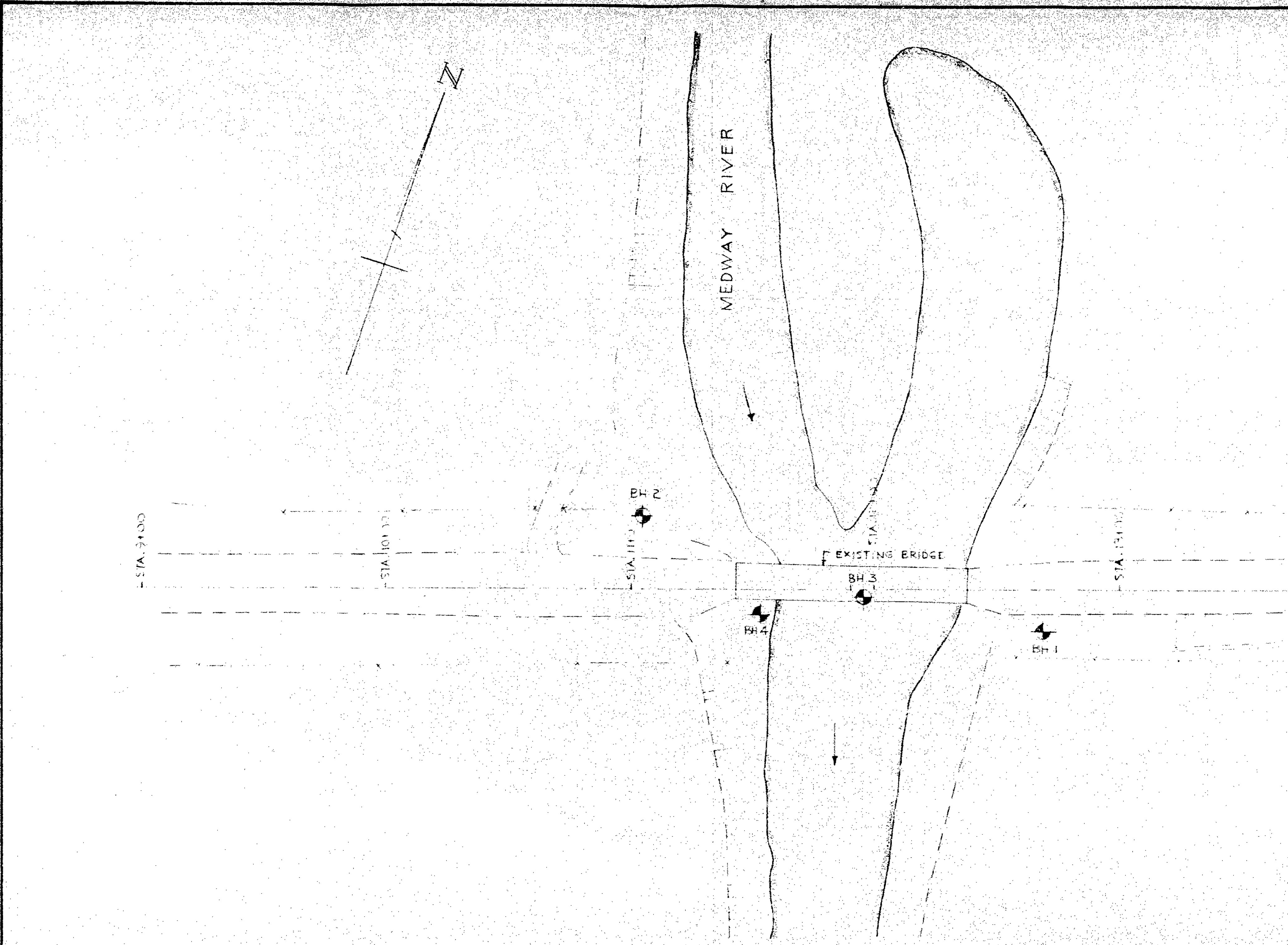
15 ◇ 5 Percent axial strain at failure 10

VERTICAL SCALE 1 INCH TO 10 FT.

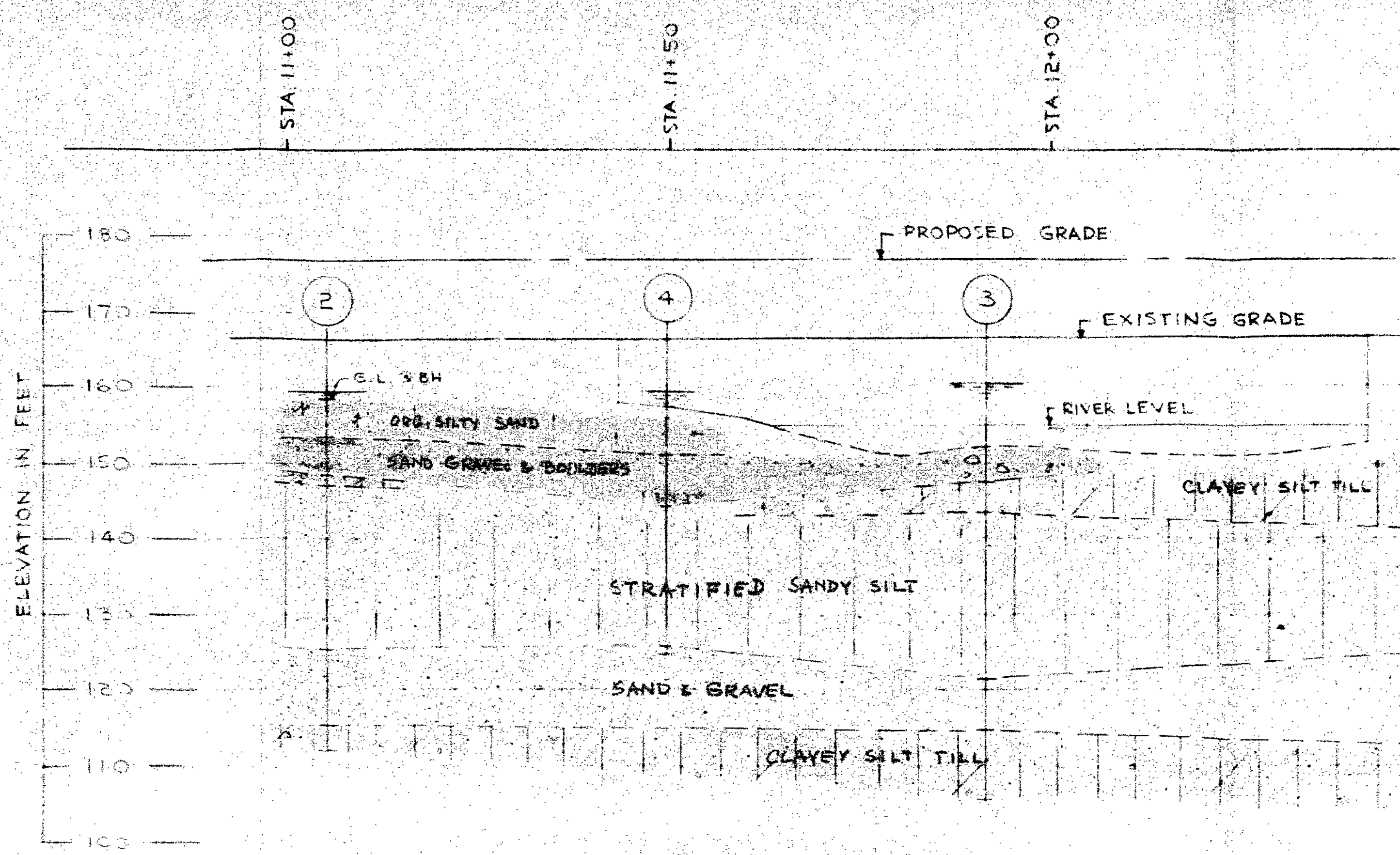
GOLDER & ASSOCIATES

DRAWN A.

CHECKED



PLAN
SCALE: 1" TO 40'-0"



SCHEMATIC SECTION ALONG CENTRELINE OF ROAD

SCALE: 1" TO 20'-0"

STRATIGRAPHY

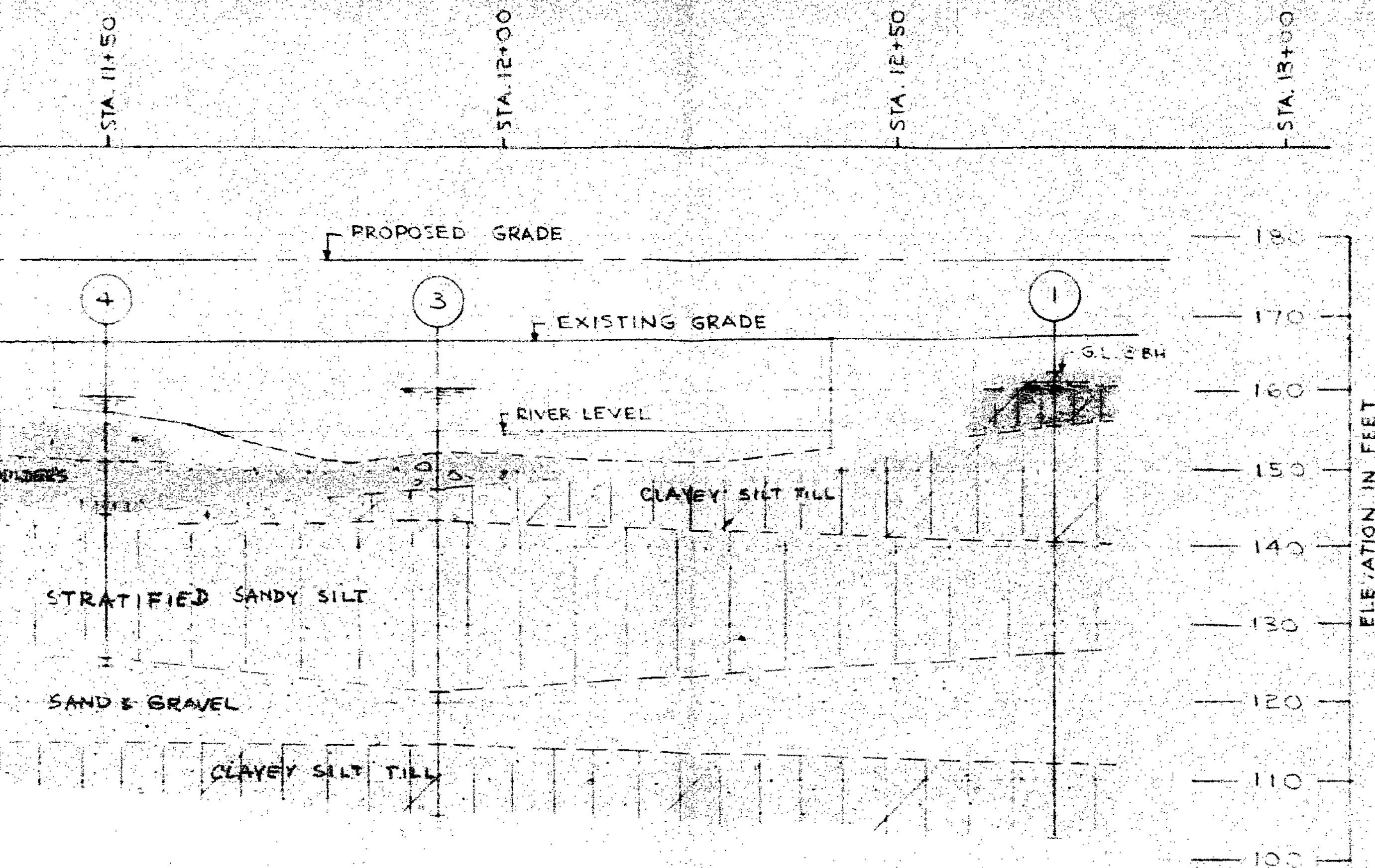
- LOOSE BROWN SAND
- LOOSE BROWN ORGANIC SILTY SAND
- VERY STIFF BROWN CLAYEY SILT
- LOOSE TO COMPACT BROWN SAND, GRAVEL AND BOULDERS
- FIRM TO VERY STIFF BROWN TO GREY CLAYEY SILT TILL
- DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT
- VERY DENSE GREY AND BROWN SAND AND GRAVEL
- HARD GREY AND BROWN CLAYEY SILT TILL

REFERENCE

1. R.C. DUNN & ASSOCIATES LTD. JOB Nº 64-116, PLAN OF SUNNINGDALE BRIDGE, LOCATED ON ROAD BETWEEN CONC. V AND CONC. VI AT LOT 19.
2. R.C. DUNN & ASSOCIATES LTD. JOB Nº 64-116, PROFILE OF SUNNINGDALE BRIDGE.

BORING PLAN AND SOIL STRATIGRAPHY

FIGURE 1

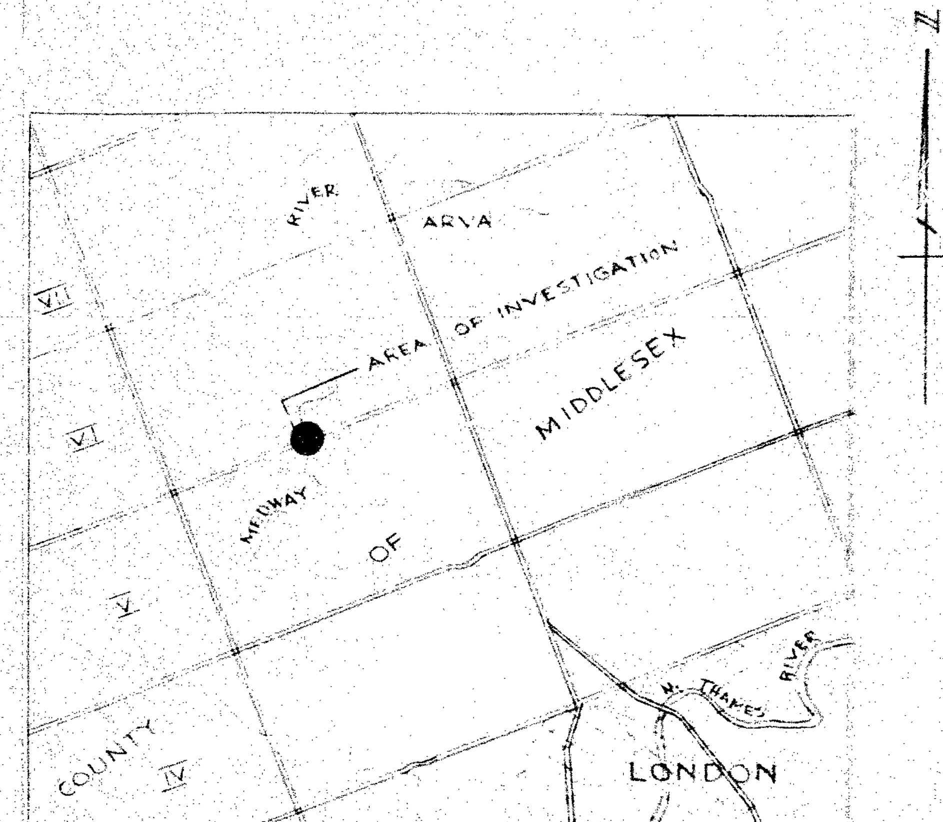


...MATIC SECTION ALONG CENTRELINE OF ROAD

SCALE: 1" TO 20'-0"

STRATIGRAPHY

- LOOSE BROWN SAND
- LOOSE BROWN ORGANIC SILTY SAND
- VERY STIFF BROWN CLAYEY SILT
- LOOSE TO COMPACT BROWN SAND, GRAVEL AND BOULDERS
- FIRM TO VERY STIFF BROWN TO GREY CLAYEY SILT TILL
- DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT
- VERY DENSE GREY AND BROWN SAND AND GRAVEL
- HARD GREY AND BROWN CLAYEY SILT TILL



KEY PLAN

SCALE: 1" TO 0.8 MILES (APPROX.)

LEGEND

- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- W.L. IN PIEZOMETER - AUG. 20, 1964

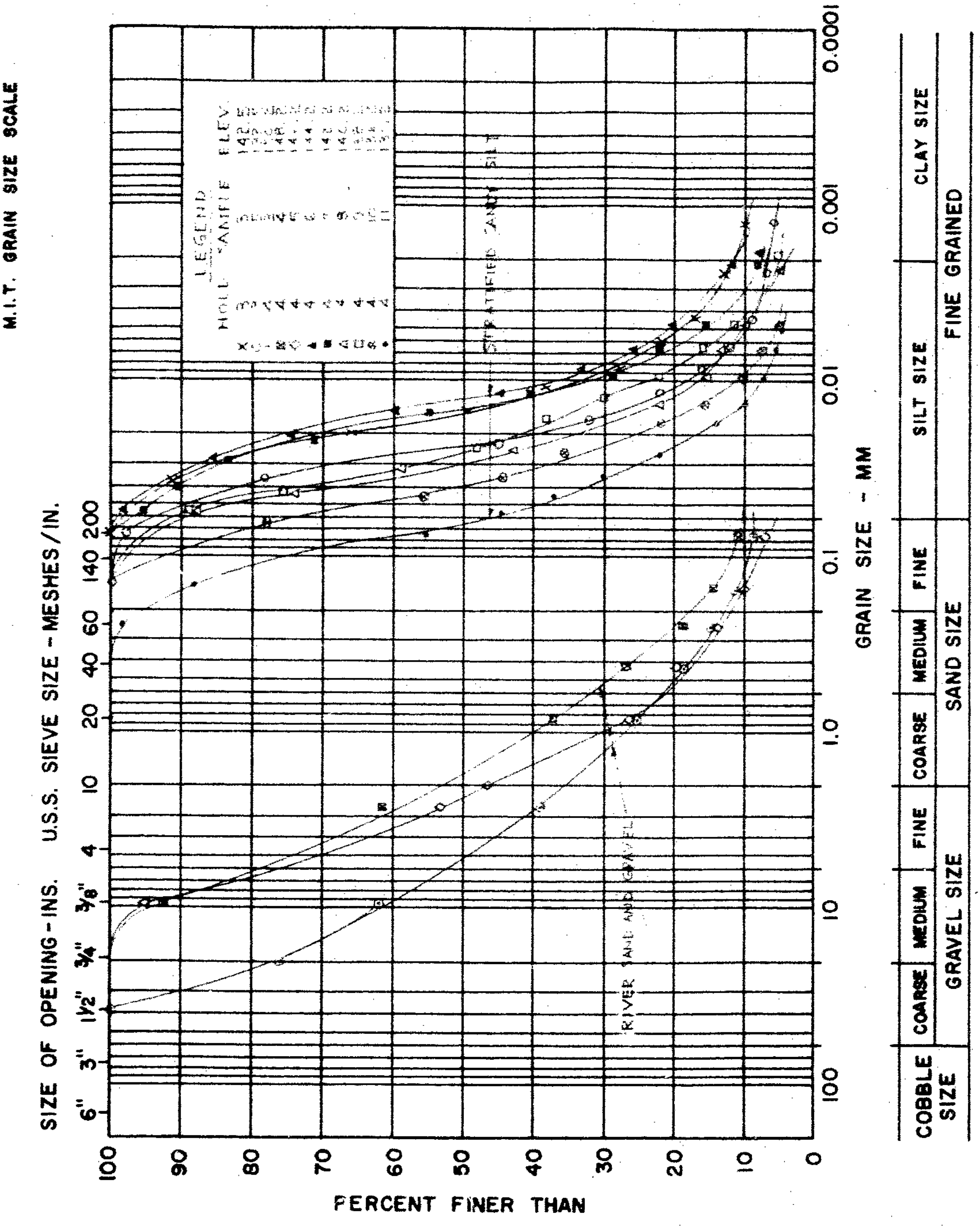
GOLDER & ASSOCIATES

Made J.A.
Chkd. RHG
Appd LCS

PROJECT No. 64094

GRAIN SIZE DISTRIBUTION

FIGURE 2

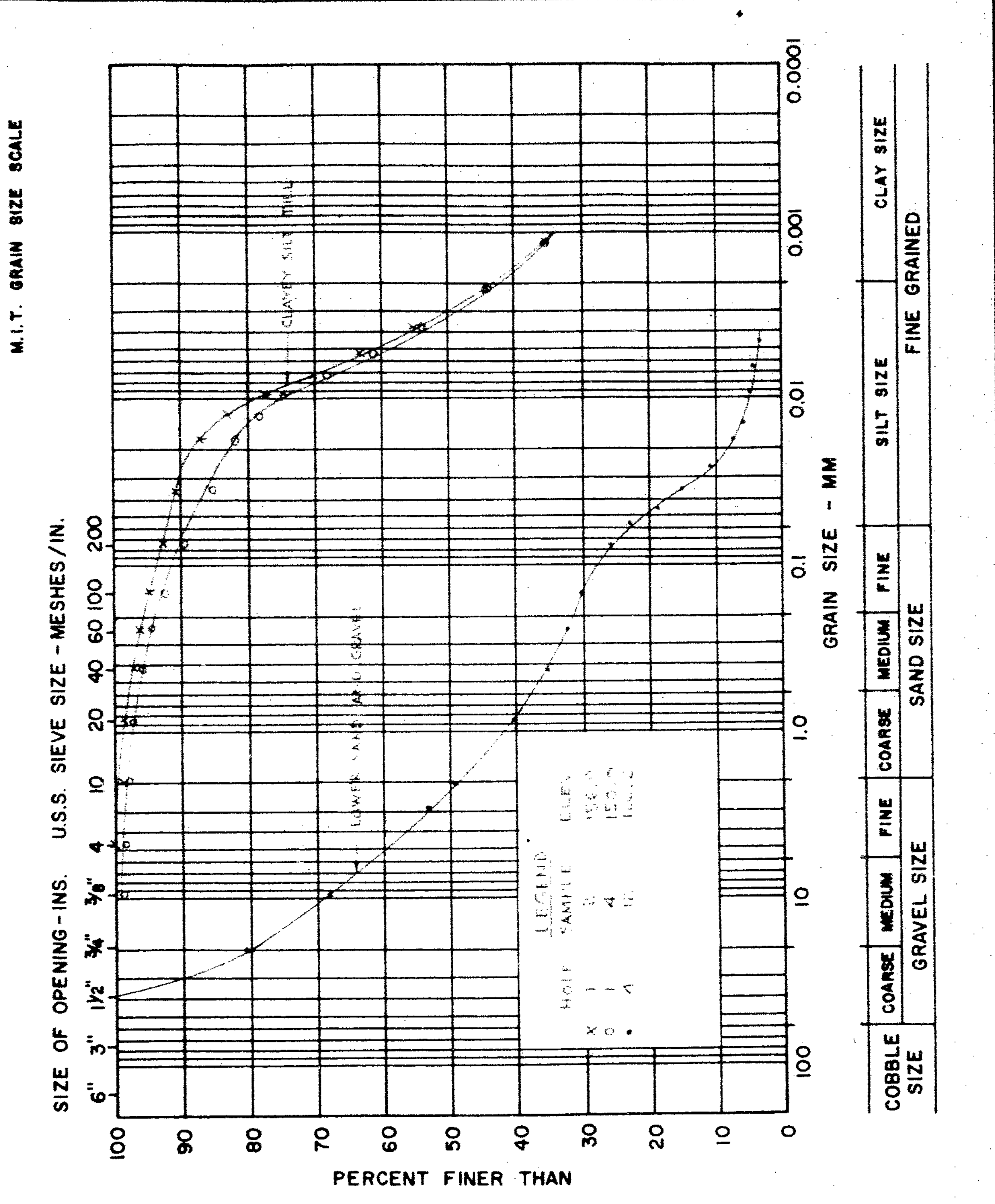


GOLDER & ASSOCIATES

PROJECT No.

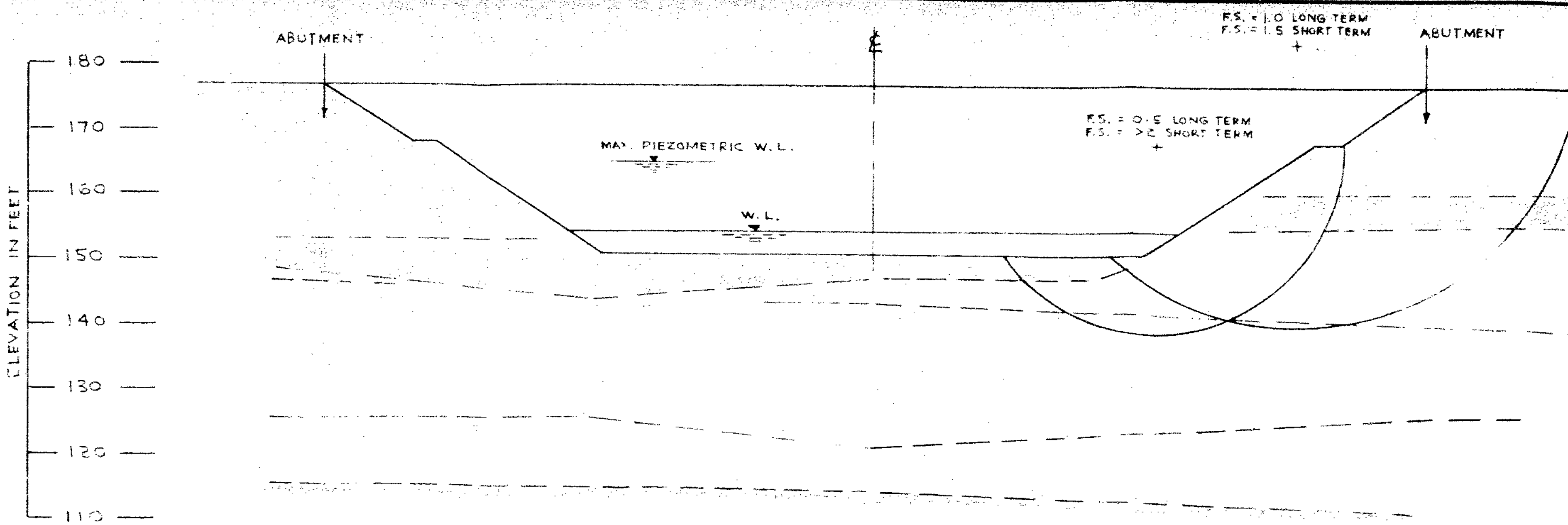
GRAIN SIZE DISTRIBUTION

FIGURE 3



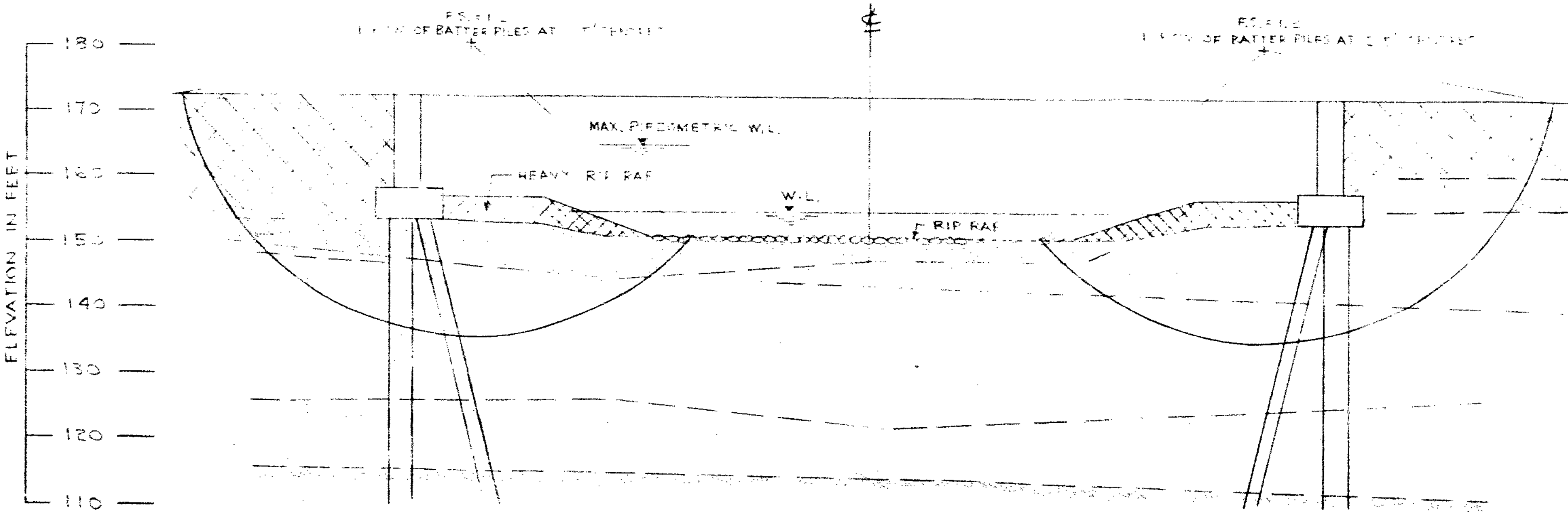
GOLDER & ASSOCIATES

(a)

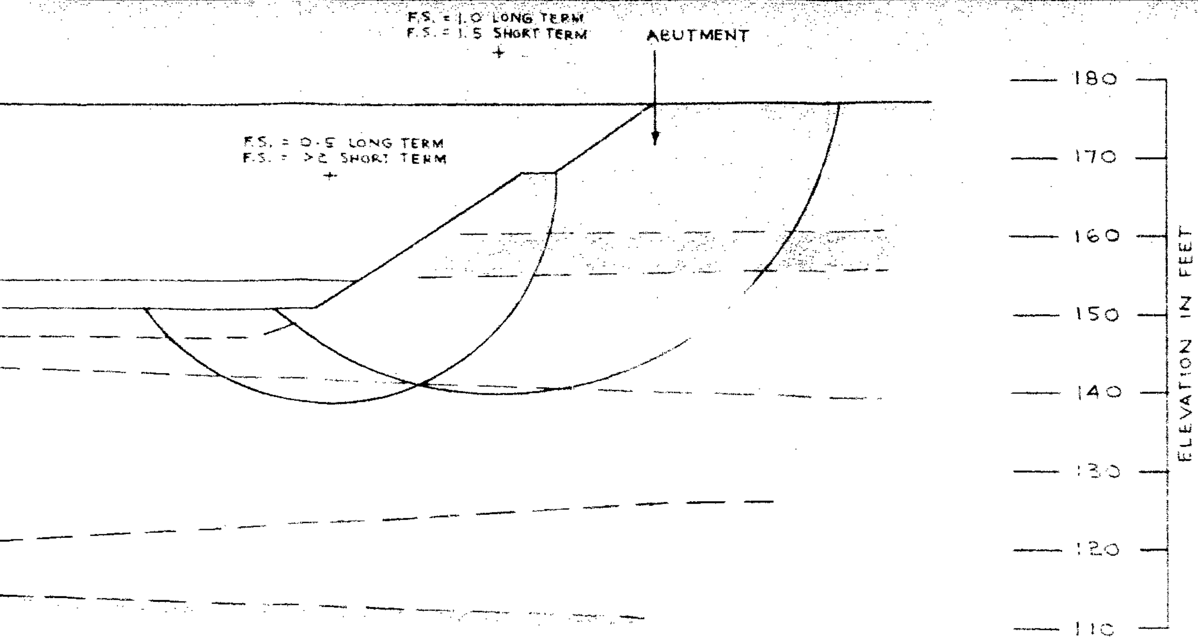


STABILITY ANALYSES - ORIGINALLY PROPOSED SECTION ALONG CENTRELINE OF ROADWA

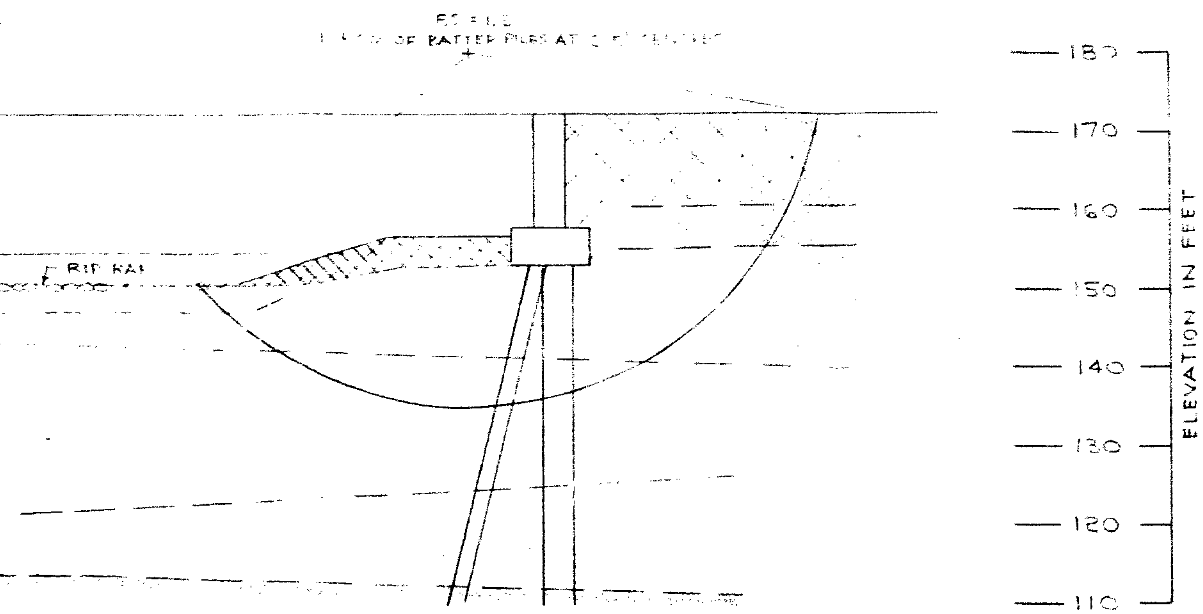
(b)



ABUTMENTS ON PILES - ALTERNATE II



ECTION ALONG CENTRELINE OF ROADWAY



STABILITY ANALYSES
PROPOSED ABUTMENT DESIGN

FIGURE 4

STRATIGRAPHY

- VERY STIFF BROWN CLAYEY SILT
- LOOSE TO COMPACT BROWN SAND, GRAVEL AND BOULDERS
- FIRM TO VERY STIFF BROWN TO GREY CLAYEY SILT TILL
- DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT
- VERY DENSE GREY AND BROWN SAND AND GRAVEL
- HARD GREY AND BROWN CLAYEY SILT TILL

REFERENCE

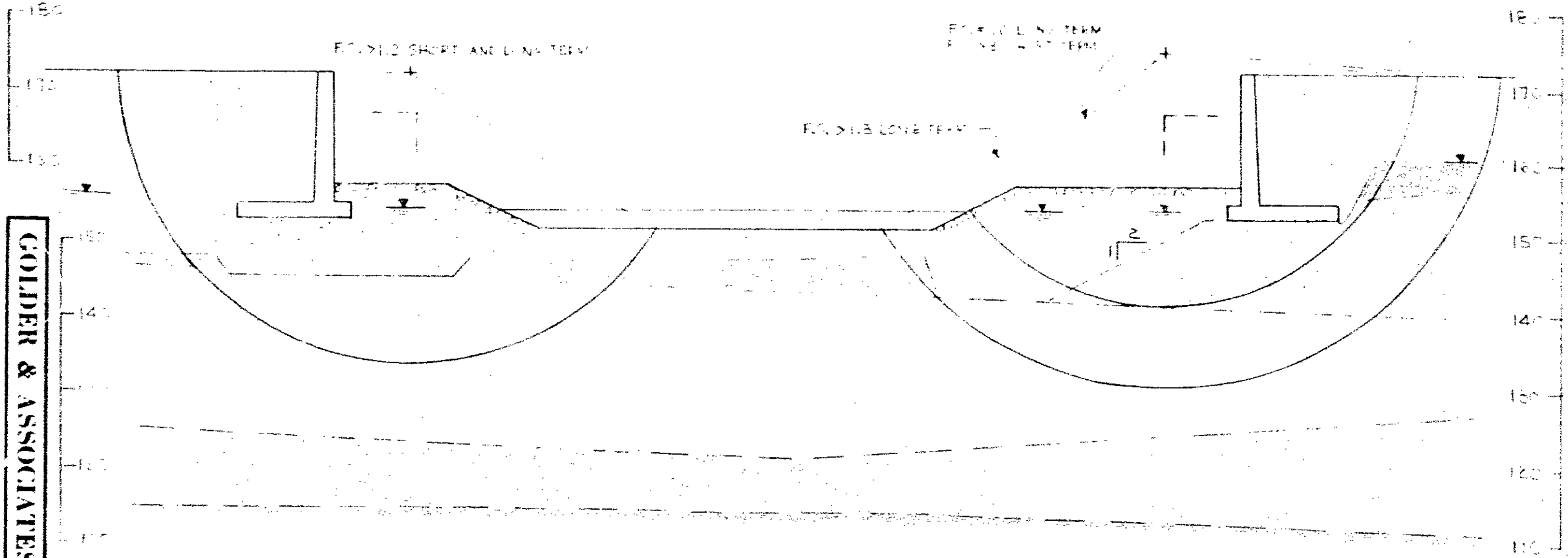
R.C. DUNN & ASSOCIATES LTD. JOB No 64-116, PROFILE OF SUNNINGDALE BRIDGE.

SCALE: 1" TO 20'-0"

GOLDER & ASSOCIATES

Made J.A.
Chkd. DHQ
Appd. [illegible]

PROJECT No. [illegible]



SINGLE SPAN STRUCTURE - ABUTMENTS ON SPREAD FOOTINGS
ALTERNATE III

STRATIGRAPHY

- VERY STIFF BROWN CLAYEY SILT
- FIRM TO VERY STIFF BROWN TO GREY CLAYEY SILT
- DENSE TO VERY DENSE GREY STRATIFIED SANDY SILT
- VERY DENSE GREY AND BROWN SAND AND GRAVEL
- HARD GREY AND BROWN CLAYEY SILT TILL

SCALE: 1" To 20'-0"

FIGURE 5

GOLDER & ASSOCIATES

Made J.A.
Chkd.
Appd.