

**TO:** Mr. S. McCombie,  
Bridge Planning Engr.,  
Bridge Division.

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

Attention: Mr. A. Watt

**DATE:** October 19, 1965

**OUR FILE REF.**

**IN REPLY TO**

**SUBJECT:**

STABILITY STUDY REPORT BY H. Q. GOLDER & ASSOCIATES LTD.  
Proposed Jones Creek (East Branch) Bridge, Highway 401 -  
Line 'J', near Gananoque, Ontario. District 8 (Kingston).

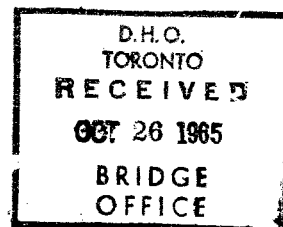
Attached, please find the above-mentioned supplementary report submitted by H. Q. Golder and Associates. The report summarizes the discussions and conclusions reached at our last meeting of October 12, 1965.

It is believed that now all the necessary information for further design is provided. However, should there be any additional questions or problems that you would like to discuss, please feel free to call on our Office.

AGS/MdeF  
Encls. (4)

*Attest*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office  
Gen. Files



## H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

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

October 15, 1965.

Department of Highways, Ontario,  
Foundations Section,  
Hwy. 401 & Keele Street,  
DOWNSVIEW, Ontario.

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

RE: STABILITY STUDY,  
EAST BRANCH JONES CREEK BRIDGE,  
PROPOSED HIGHWAY 401 - LINE "J",  
NEAR GANANOQUE, ONTARIO.

Dear Sirs:

At a meeting held in your offices on September 17, 1965 we were asked to examine the stability of the highway crossing at the above site for a revised grade line at elevation 277. The results of a subsoil investigation which we carried out at this site several years ago for the line "G" crossing are presented in our report 6265, dated December, 1962. With a change in alignment from line "G" to the present line "J", additional borings were put down at the site this year and the results of this additional work are given in our report 65054, dated July, 1965. The grade considered in these two reports was at elevation 263 or some 11 feet

above existing ground surface at the proposed creek diversion location.

This letter which presents the results of the study carried out for the revised grade at elevation 277, or some 25 feet above existing ground surface, should be read in conjunction with our previous reports for this site.

The subsoil conditions at the site are detailed in our previous reports. In summary, the surface cover in the proposed creek diversion area consists of from 10 to 20 feet of very loose to compact silts and sands overlying a stratum of firm to very stiff clayey silt ranging between about 20 and 55 feet in thickness. The clayey silt either rests on bedrock directly or on a relatively thin compact to dense layer of silty sand and gravel overlying the bedrock. An artesian head as high as 4 feet above existing ground surface was encountered within the bedrock and lower portion of the overburden during both site investigations. A simplified soil stratigraphy profile is given on the enclosed Figure 1.

Following are our major comments regarding the revised grade and proposed structure at the site:-

- (i) A rigid arch culvert, as shown on Figure 1, has now

been proposed for the creek diversion channel. The culvert is to be founded on end bearing piles driven to bedrock. As the upper portion of the overburden at the site is generally comprised of loose silts and sands the provision of a piled foundation, although giving sufficient vertical support to the arch, will not necessarily give adequate unyielding support in the horizontal direction. Lateral restraint is necessary for the arch, particularly during the initial stages of embankment construction before fill is placed above the top of the arch. If lateral movement of the pile caps or arch bases takes place the arch structure could crack. To prevent this possibility it is recommended that a twin box culvert, as outlined on Figure 1, should be considered for the crossing instead of a rigid arch culvert. In order to eliminate differential settlement of the culvert, due to the uneven thickness of compressible overburden beneath the creek diversion route, the box culvert should be founded on end bearing piles.

The provision of a box culvert will also eliminate embankment stability problems in the direction perpendicular to the centreline of the creek channel.

(ii) Stability analyses were carried out for the proposed Highway 401 roadway embankment grade at elevation 277. The results of these analyses for the lateral stability of an embankment constructed of granular "B" material are summarized on Figure 2. Taking an average undrained shear strength of 1,500 lb/sq.ft. for the clayey silt subsoil, the factor of safety against an overall rotational failure of a 25 foot high granular fill is computed to be 1.5. However, reference to the summary plot of shear strength on Figure 1 indicates that the undrained shear strength of the clayey silt down to about elevation 225 could be as low as 500 to 700 lb/sq.ft. over portions of the site. Using an average value of 600 lb/sq.ft. above elevation 225 and 1,000 lb/sq.ft. below elevation 225 in the clayey silt, the factor of safety is computed to be unity. This suggests that portions of the proposed embankment could fail during construction. To ensure stability one of the following could be done:-

- (a) Berms, 15 feet high and 40 feet long, provided - this necessitates a longer box culvert.
- (b) The roadway grade lowered to elevation 269 at the

crossing resulting in a stable embankment height of 17 feet.

- (c) Controlled stage construction - that is, initially building the embankment to a safe height and allowing the pore pressure within the subsoil to dissipate with time before increasing the embankment height. It is however understood that this is not a practical solution due to time limitations during the construction period.

- (iii) As rockfill will be readily available from nearby rock cuts for the highway, stability analyses were similarly carried out for a rockfill embankment. The results of these analyses are presented on Figure 3.

- (iv) As indicated on Figures 2 and 3 the stability analyses were carried out for a level ground surface at elevation 252. As the existing creek bed, which is lower than elevation 252, will be in close proximity to the toe of the berm and in some cases to the main embankment itself, it will be necessary to fill in the creek to elevation 252 in the "U" section between the diversion points to the

west of the proposed crossing. Some filling of the creek may also be required immediately to the east of the crossing. It is understood that filling of the existing creek bed with rockfill in the diversion area will be done as a matter of course.

- (v) Along the line "J" route, the valley floor of the existing creek extends for some 1,200 feet (chainage 321+00 to 333+00) between rock outcrops. Since no subsoil information other than at the proposed creek diversion location is available, it must be assumed at this time that the subsoil conditions across the valley are similar. Therefore, for an embankment 25 feet in height above existing ground surface, berms of the size indicated on Figures 2 and 3 will be required across the valley. If the grade is reduced to provide stability without the use of berms, the height of the embankment above existing ground surface across the valley floor must similarly be limited to that shown on the figures.

- (vi) Settlement of the roadway embankment will take place due to consolidation of the subsoil. For a 25 foot high fill it is estimated that the total settlement could be of

the order of 9 inches. The portion of this settlement contributed by compression of the upper silts and sands should take place during construction. The majority of the remaining portion, comprising about  $1/3$  to  $1/2$  the total estimated settlement, due to long term consolidation in the underlying clayey silt should occur within about 1 to  $1\frac{1}{2}$  years. Differential settlement across the embankment, resulting from the variable thickness of compressible deposits overlying the bedrock, could be significant in the first few months following construction.

- (vii) If the proposed roadway grade at the crossing is lowered from elevation 277 to about elevation 269, the alternative to a box culvert could be a 3 span bridge structure. Three spans would be necessary to accommodate berms on either side of the diversion channel as the embankment height above the channel bottom at elevation 244 would be about 25 feet. The length of berms required would be of the same order as given on Figures 2 and 3 with a top elevation of about 259 or some 15 feet above the channel bottom.



We trust that the information given in this letter is sufficient for your requirements. If you require additional information, please give us a call.

Yours truly,  
H. Q. GOLDER & ASSOCIATES LTD.,



J. L. Seychuk, P.Eng.



JLS:HDG  
65054-1  
October 15, 1965.

## 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

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

<i>WH</i>	sampler advanced by static weight—weight, hammer
<i>PH</i>	sampler advanced by pressure—pressure, hydraulic
<i>PM</i>	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<sub>w</sub>, 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

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer <sup>1</sup>
<i>Q</i>	undrained triaxial <sup>2</sup>
<i>R</i>	consolidated undrained triaxial <sup>2</sup>
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test

#### NOTES:

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

<sup>2</sup>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
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{sy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	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 = $c_s t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

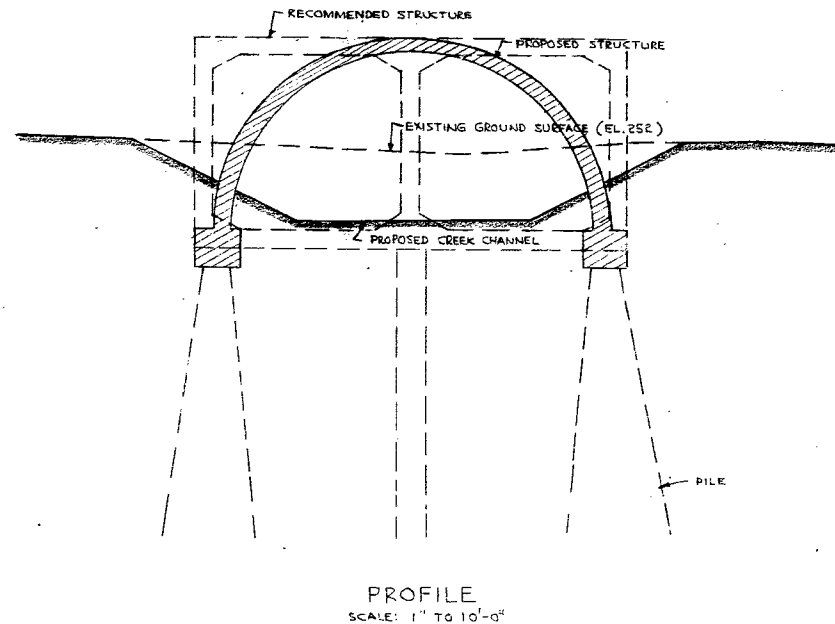
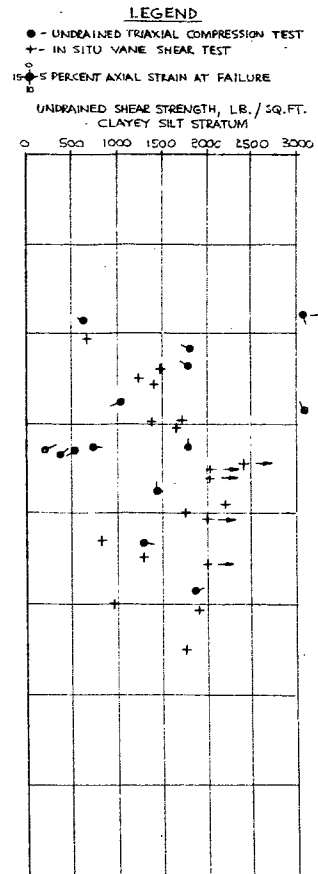
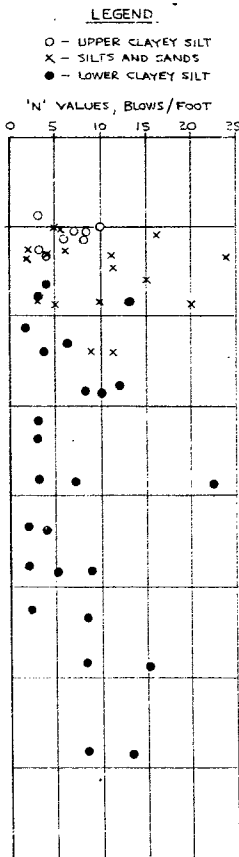
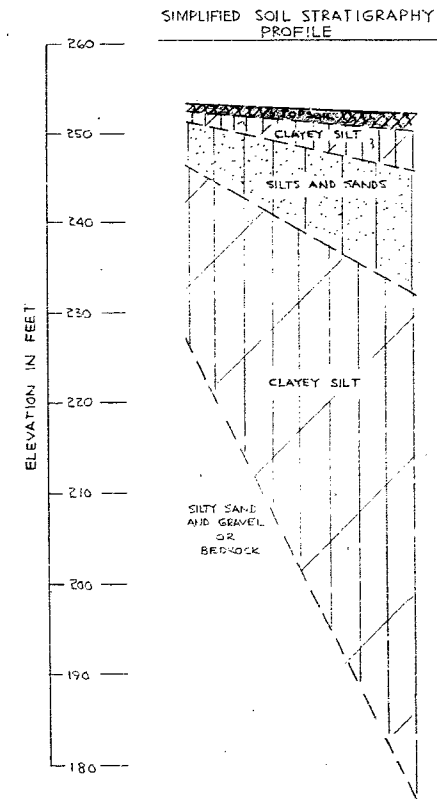
#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_t$	sensitivity

$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

\*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.



# SUMMARY PLOT OF ENGINEERING PROPERTIES OF SUBSOIL

FIGURE 1

## REFERENCES

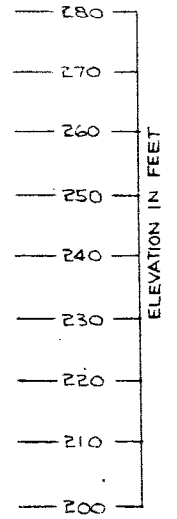
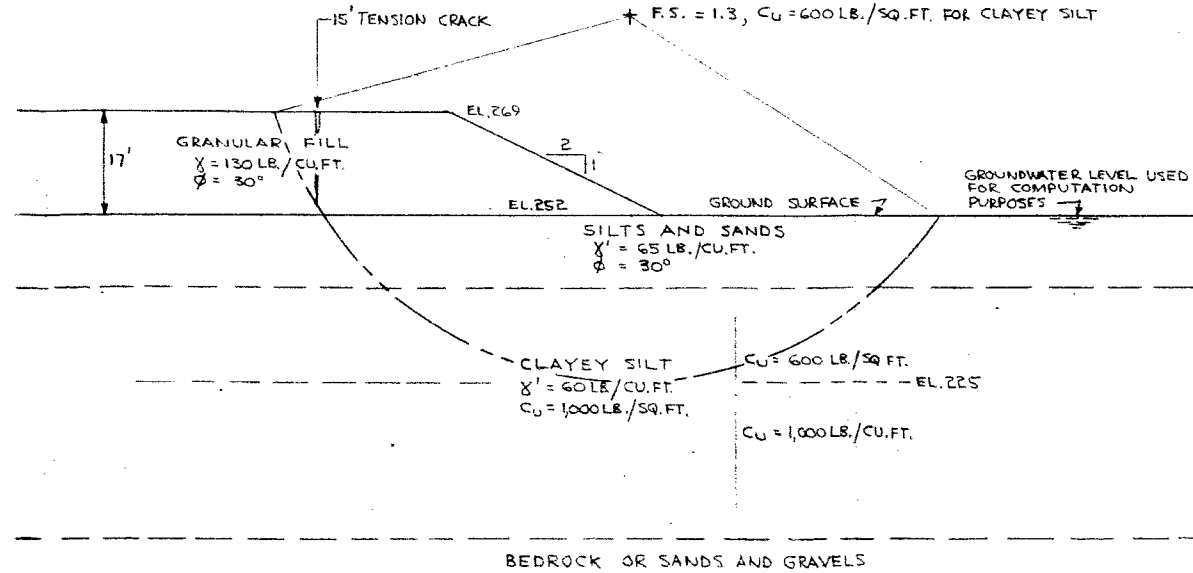
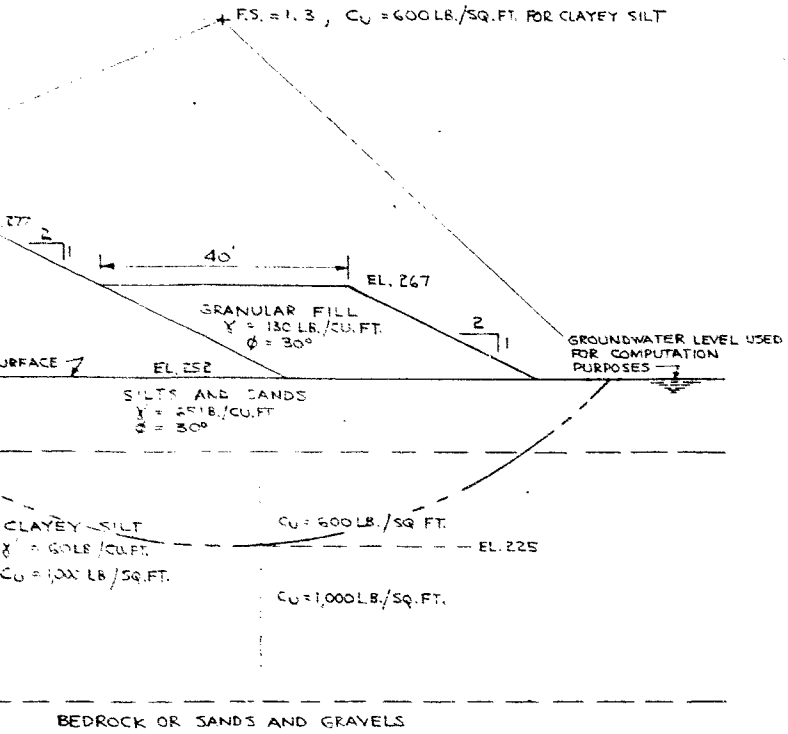
1. H. Q. GOLDER & ASSOCIATES LTD. REPORT NO 6265 TITLED SOIL CONDITIONS AND FOUNDATIONS - PROPOSED JONES CREEK (EAST BRANCH) BRIDGE, HIGHWAY 401 - LINE 'G' DATED DEC. 1962.
2. H. Q. GOLDER & ASSOCIATES LTD. REPORT NO 65054 TITLED ADDITIONAL BORINGS - PROPOSED JONES CREEK (EAST BRANCH) BRIDGE, HIGHWAY 401 - LINE J DATED JULY, 1965.
3. DEPARTMENT OF HIGHWAYS, ONTARIO - PLAN NO E-4606-1 BRIDGE SITE, PROPOSED CROSSING AT PROPOSED DIVERSION OF EAST BRANCH OF JONES CREEK, KING'S HIGHWAY 401 PROPOSED REVISION LINE 'J', DATED JUNE 1965.

Drawn: Oct. 14, 1965

GOLDER & ASSOCIATES

Made J.A.  
 Chkd J.A.  
 Appd J.A.





TERMS FOR PROPOSED GRADE AT EL.277

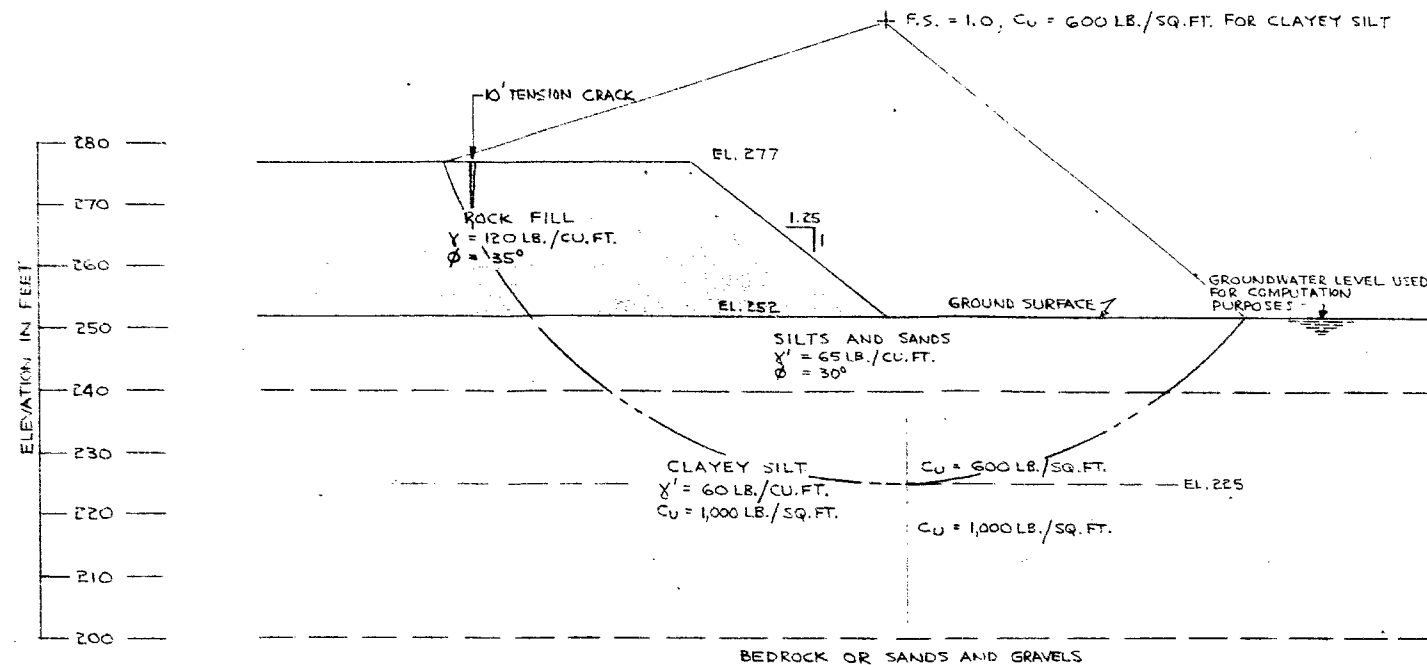
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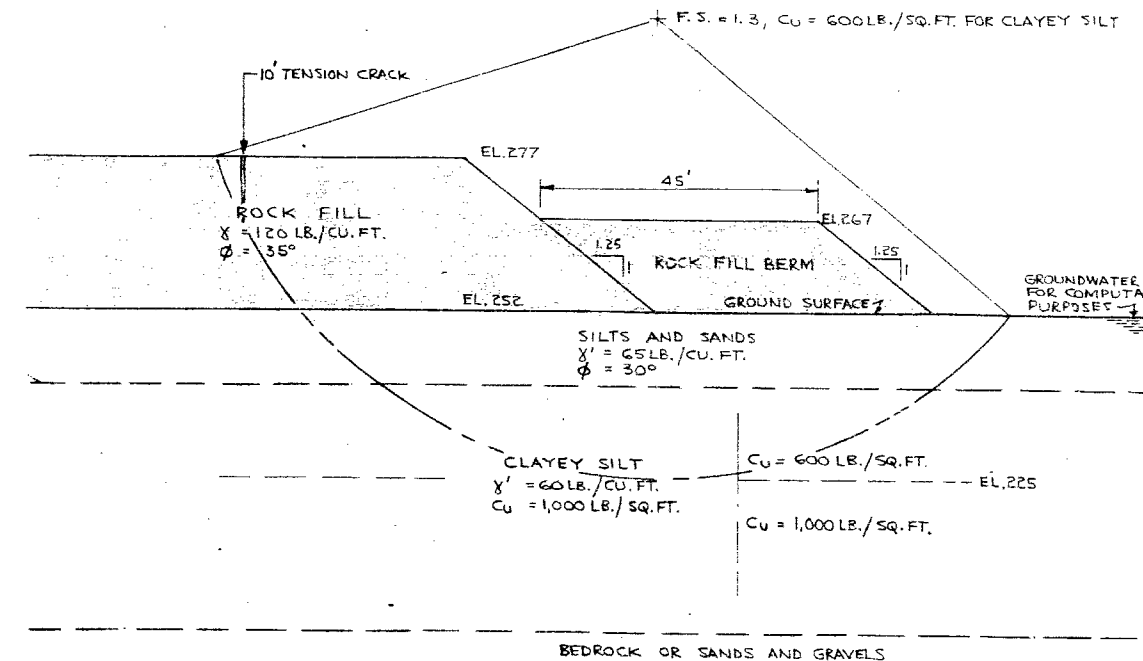
Drawn: Oct. 8, 1965

GOLDER & ASSOCIATES

Made: J.A.  
 Chkd: J.A.  
 Appd: J.A.

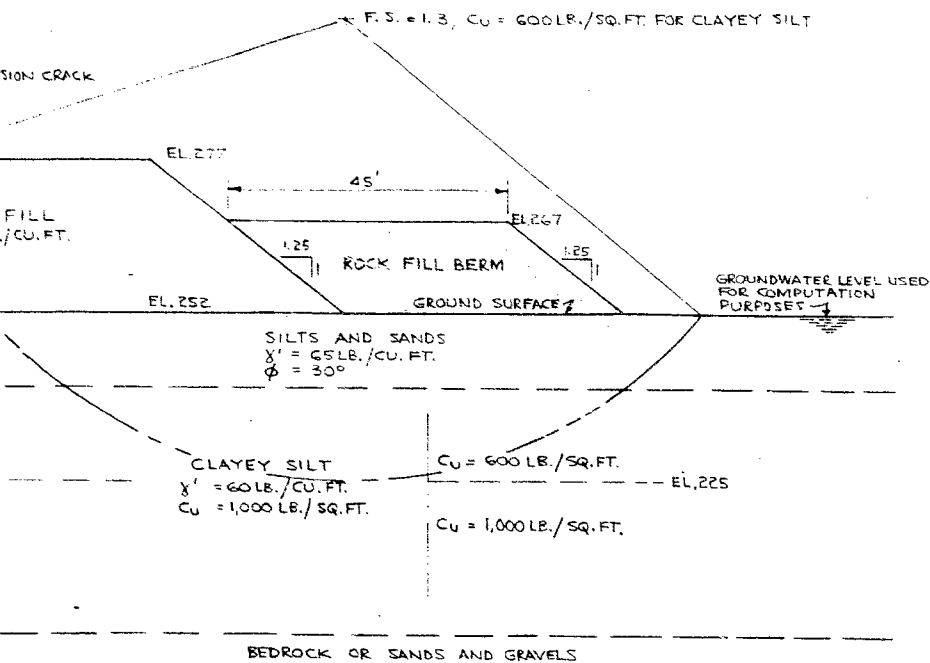


PROPOSED GRADE AT EL. 277



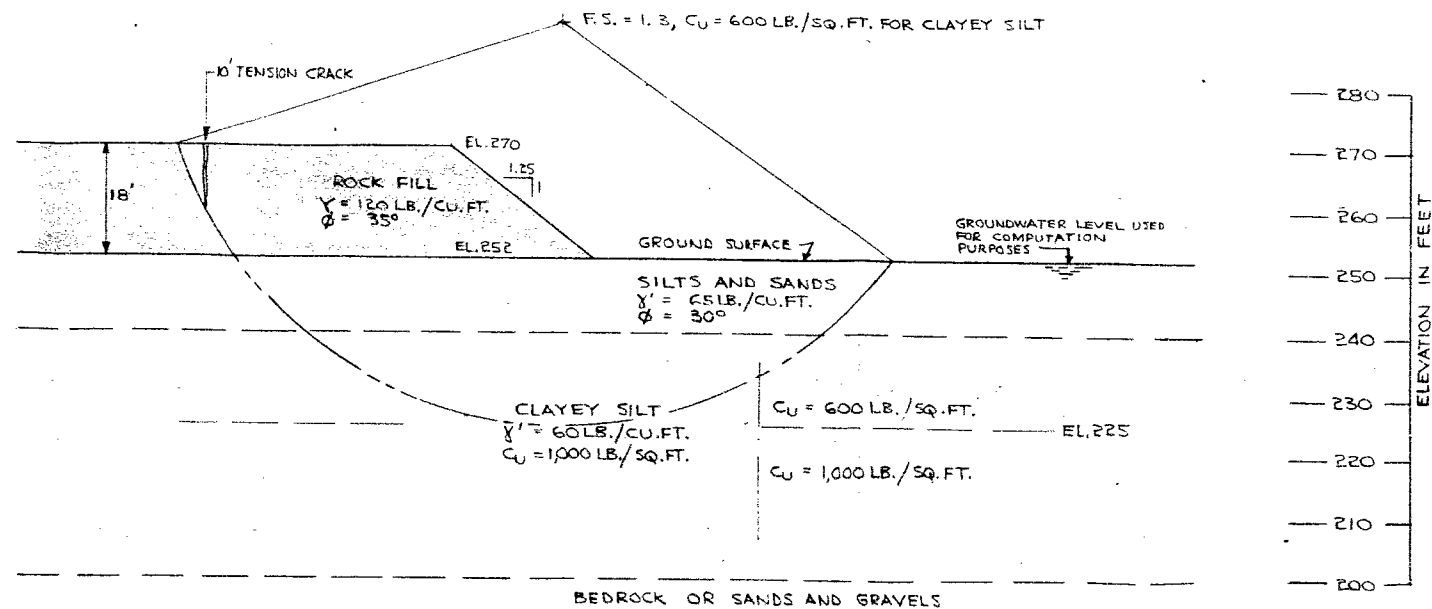
RECOMMENDED BERMS FOR PROPOSED GRADE AT EL. 277

SCALE: 1" TO 20'



ENDED BERMS FOR PROPOSED GRADE AT EL. 277

SCALE: 1" TO 20'



RECOMMENDED SECTION WITHOUT PROVISION OF BERM

Drawn: Oct. 8, 1985

GOLDER & ASSOCIATES

Made by: J.A.  
 Chkd: J.A.  
 Appd: J.A.



## MEMORANDUM

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

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

Attention: Mr. S. McCombie

DATE: July 27, 1965

OUR FILE REF.

IN REPLY TO

## SUBJECT:

FOUNDATION INVESTIGATION REPORT BY:  
H. Q. Golder and Associates, Limited.  
Proposed Jones Creek (East Branch) Bridge,  
Highway 401 - Line "J", Gananoque, Ontario.  
W.P. 179-61                      --                      District 8

Attached, please find the above-mentioned report submitted by the Consultant, H. Q. Golder and Associates.

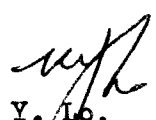
We have reviewed the report and found the factual data both adequate and well presented. The conclusions and recommendations are straightforward and do not require any comments. However, should there be any queries in connection with this project, please feel free to contact our Office.

KYL/MdeF

Attach.

cc: Messrs. B. R. Davis (2)  
H. A. Tregaskes  
D. W. Farren  
J. Ford  
E. A. Cash  
J. E. Gruspier  
A. Watt

Foundations Office  
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K. Y. Lo,  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.

D.H.O.  
TORONTO  
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BA1564A  
H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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W. P. 179-61

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

ADDITIONAL BORINGS

PROPOSED JONES CREEK (EAST BRANCH) BRIDGE

HIGHWAY 401 - LINE "J"

GANANOQUE

ONTARIO

Distribution:

- 10 copies - Department of Highways, Ontario,  
Toronto, Ontario.
- 2 copies - H. Q. Golder & Associates Ltd.,  
Toronto, Ontario.

July, 1965

65054

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FIGURE 1 - Boring Plan and Soil Stratigraphy Sections	

## ABSTRACT

The results of a subsurface investigation to extend a previous investigation for the proposed Highway 401 crossing of Jones Creek (East Branch) are reported. Because of a change in the alignment of the proposed Highway 401 (line G to line J), the position of the bridge structures at the crossing was changed and the present investigation consisting of two additional boreholes was carried out.

The subsurface conditions encountered by the additional borings are substantially the same as those reported in previous investigation (our report 6265, dated December, 1962). The additional area of the site is underlain by up to 10 feet of silts and sands with a trace of organic matter. Below this is firm to very stiff clayey silt about 38 feet in thickness. In one borehole the clayey silt stratum extends to bedrock while in the other about 4 feet of compact silty sand and gravel occurs between the clayey silt and bedrock. The bedrock is a sound hard grey granitic type rock and was proven by coring drilling in both boreholes.

A piezometer installed in the bedrock in one borehole indicated an artesian head of about 4 feet above ground surface.

It is recommended that the proposed structure be founded on steel "H" piles with reinforced tips driven to bedrock. Loads of 70 tons per pile are suggested for 12 inch "H" piles driven to practical refusal.

The approach embankments should be constructed with side slopes no steeper than 2 horizontal to 1 vertical.

Settlements of the proposed structures and approach embankments, if founded as recommended, should be minor.

## INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario, to carry out additional borings for the proposed Jones Creek (East Branch) bridge structures for the proposed Highway 401, line "J", near Gananoque, Ontario. A subsoil investigation was carried out at the site along the line "G" alignment in 1962 (our report 6265, dated December, 1962). Since the revised alignment, line "J", crosses the Jones Creek (East Branch) some 30 to 40 feet north of the previously investigated line, additional borings were put down to confirm the subsurface conditions at the line "J" crossing.

## PROCEDURE

The field work for this investigation was carried out between May 28, and June 3, 1965. During this period 2 boreholes with accompanying dynamic penetration tests were put down to a depth of about 60 feet using a machine drillrig supplied and operated by the F.E. Johnston Drilling Co. Ltd. The field work was supervised throughout by a member of our engineering staff.

The locations of the borings put down during this investigation and during the previous investigation are shown on Figure 1 located in a pocket following the Records of Boreholes. A

detailed log for each boring put down in this investigation is shown on the Records of Boreholes following the text of this report. Sections of the inferred soil stratigraphy across the site are shown on Figure 1.

The samples obtained during the investigation were brought to our laboratory for detailed examination. No laboratory testing was carried out.

The elevations used in this report are referred to Geodetic Datum. The elevations and borehole locations were given to us by the Department of Highways, Ontario.

#### SITE AND GEOLOGY

The site of the proposed bridge structures to carry Highway 401 over the Jones Creek (East Branch) is located some 8 miles west of Brockville, Ontario and some 0.2 miles south of Highway 2 in the Township of Front of Yonge in the County of Leeds, Ontario.

The site is presently unoccupied and is generally fairly flat except where it is cut by the existing creek channel which is some 10 feet lower than the surrounding ground surface. At the site the creek is some 30 feet wide.

From the previous site investigation and from available geological information, it is known that the proposed site is located within the physiographic region known as "Leeds Knobs and Flats". This region consists primarily of scattered bedrock outcrops between which lie water laid deposits of clays, sands and gravels. The plain in which the site is located has been modified by the action of the east Branch of Jones Creek and is a floodplain for the creek.

Bedrock in this area consists of various types of altered sedimentary rocks, crystalline limestones and dolomites; gneisses and quartzites of Precambrian age, which are intruded, metamorphosed and deformed by bodies of granite, syenite and other igneous rocks. The surface elevation of bedrock can vary appreciably within small areas.

#### SUBSURFACE CONDITIONS

The subsurface conditions encountered in these two boreholes are essentially the same as those encountered in the 1962 investigation.

The soil profile, as shown on the accompanying Figure 1 that includes the information from the previous investigation, consists of topsoil underlain by a layer of firm to stiff mottled grey and brown clayey silt. This layer is 3.5 feet thick at borehole 13 and 2.0 feet thick at borehole 14. Beneath the clayey silt

is loose to compact grey sand with a trace to some silt and gravel and a trace of organic matter in some places. This stratum is very nearly 6 feet thick at both boreholes.

The sand is underlain by firm to very stiff clayey silt with some silty clay, sandy silt and sand layers up to several inches thick. This stratum is 38 feet thick at both boreholes. The clayey silt has an average sensitivity of about 6. At borehole 14 the clayey silt stratum extends to bedrock. At borehole 13, 4.4 feet of compact grey silt sand and gravel was encountered between the clayey silt and the bedrock.

Bedrock was proven in both boreholes by taking approximately 10 feet of core in AXT size. The bedrock is a sound hard grey granitic type rock with occasional fissures.

A piezometer was installed in the bedrock at borehole 14 and sealed just above the bedrock. The highest water level recorded was taken on June 9, 1965 and indicated that there was an artesian condition in the lower portion of the borehole with a head of water about 4 feet above the ground surface. This artesian condition and the head measured agrees with that obtained in the 1962 site investigation.



## DISCUSSION

As boreholes 13 and 14, the two additional boreholes covered by this report, extend and substantially confirm the previous investigation, the foundation recommendations remain essentially as previously discussed. For convenience the major points of discussion in our previous report are presented herewith.

Because of the depth of excavation that would be necessary to reach suitable foundation conditions for spread footings, it is recommended that the proposed structure be founded on end bearing piles driven to bedrock. The upper surface of bedrock is irregular and probably has some near vertical faces as noted in nearby outcrops. Therefore pile lengths may vary from about 20 feet to greater than 70 feet. As the length of piles necessary can not be accurately predicted, steel "H" piles are recommended to facilitate extension of the piles if required during driving. It is suggested that reinforced tips be provided for the steel "H" piles in order that they may bite into any sloping bedrock surface rather than slip along the surface when driven.

For 12 x 12 inch, 53 pound steel "H" piles with reinforced tips, a design load of 70 tons per pile may be used providing the piles are driven to a set of at least 12 blows per inch with a hammer developing 20,000 foot pounds of energy per blow.

Pile caps should be founded at least 5 feet below the proposed bottom of the Jones Creek (East Branch) diversion to avoid possible scour. As the groundwater level is close to ground surface the excavations for the pile caps should be carried out inside a steel sheet piled cofferdam which is driven at least 5 feet into the clayey silt stratum to prevent basal instability of the excavations.


Settlement of the bridge structures, if founded as recommended above, will be negligible.

Free draining granular backfill should be placed behind the proposed abutments. This backfill should extend at least 4 feet horizontally away from the abutment walls and have provision for drainage to ensure that no excess hydrostatic or ice pressures build up behind the walls. In the design of the abutments it is recommended that an earth pressure coefficient,  $K$ , of 0.3 be used, provided that some minor movement of the top of the abutment can be accommodated.

The approach embankments to the proposed bridge structures will have a height of about 11 feet above the existing ground surface at the proposed bridge abutments. The embankments will have a maximum height of about 14 feet above ground level at station 319+00 on line "G".

The embankments may be constructed of well compacted granular borrow or rockfill. The side slopes of the embankments should not exceed 2 horizontal to 1 vertical to ensure the overall and surficial stability of the embankments. All topsoil should be removed prior to construction of the embankments.

The embankments will settle due to consolidation of the subsoil under the additional weight of the fill. The portion of the settlement contributed by compression of the upper silts and sands is estimated to be about 3 to 4 inches; this should largely take place during construction. To estimate the probable settlement due to consolidation of the clayey strata, a consolidation test was carried out on a sample of a silty clay layer in this material. The results of this test are given on Figure 11 of our previous report. Based on this and assuming that the clayey silt stratum is largely over-consolidated, we estimate that the probable settlement below the centre of a low embankment about 14 feet in height above the existing ground level due to consolidation of the clayey strata should be about 1 to 2 inches. The major portion of this settlement should occur in the first 6 to 12 months after construction.

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J. L. Seychuk, P.Eng.

FAD:HB  
65054  
July, 1965

**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

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

<i>WH</i>	sampler advanced by static weight—weight, hammer
<i>PH</i>	sampler advanced by pressure—pressure, hydraulic
<i>PM</i>	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<sub>u</sub>, 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

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer <sup>1</sup>
<i>Q</i>	undrained triaxial <sup>2</sup>
<i>R</i>	consolidated undrained triaxial <sup>2</sup>
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test

#### NOTES:

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

<sup>2</sup>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
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	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 = $c_s t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

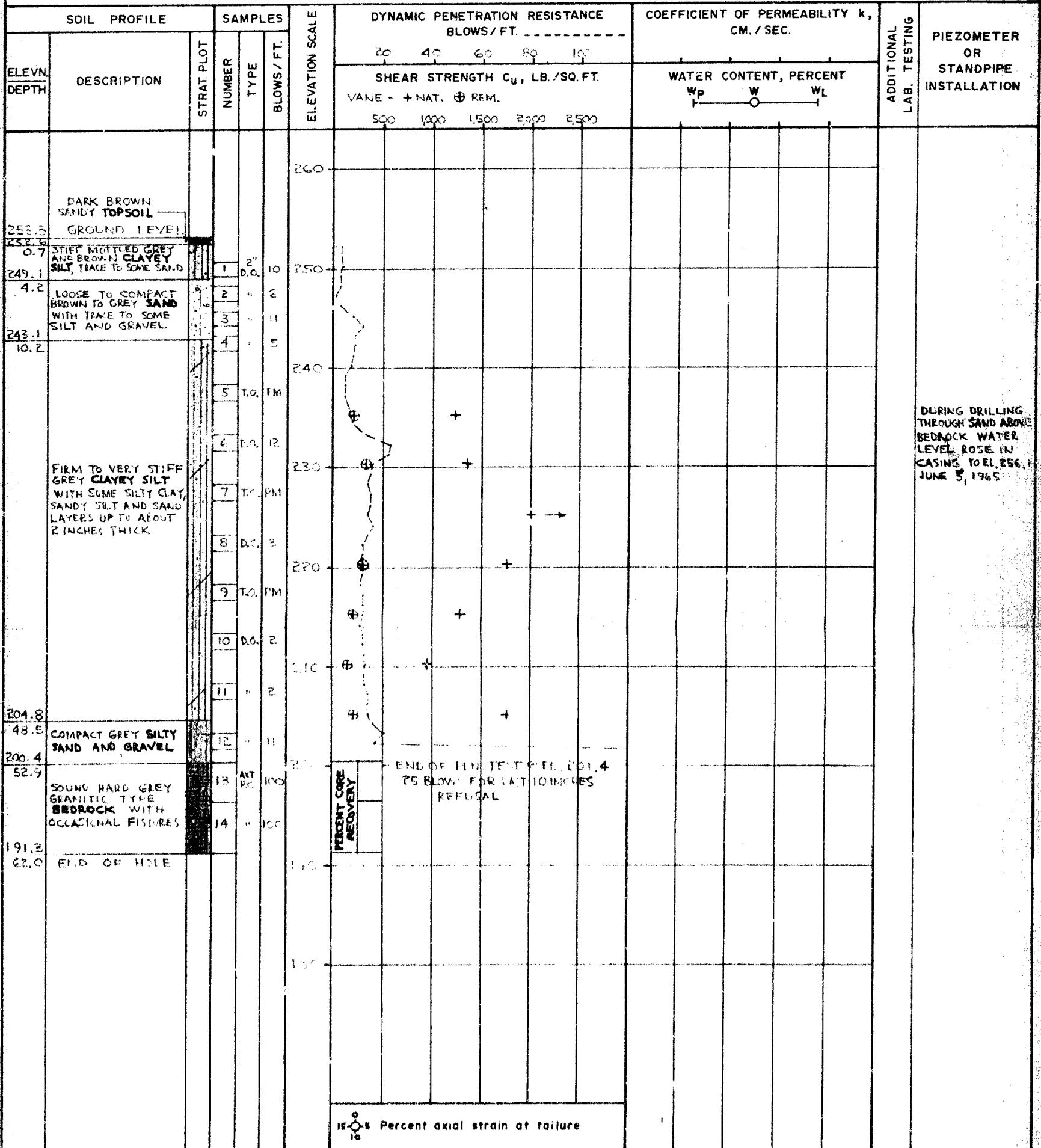
#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
	intercept
$\phi'$	effective angle of shearing resistance, or friction
$\left. \begin{array}{l} c' \\ \phi' \end{array} \right\} \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array}$	
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\left. \begin{array}{l} c_u \\ \phi_u \end{array} \right\} \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array}$	
$\mu$	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 13

LOCATION See Figure 1 BORING DATE JUNE 1-3, 1965 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX & AX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.  
CHECKED

## RECORD OF BOREHOLE 14

LOCATION See Figure 1

BORING DATE MAY 28 - JUNE 1, 1965

DATUM

GEODETIC

BOREHOLE TYPE

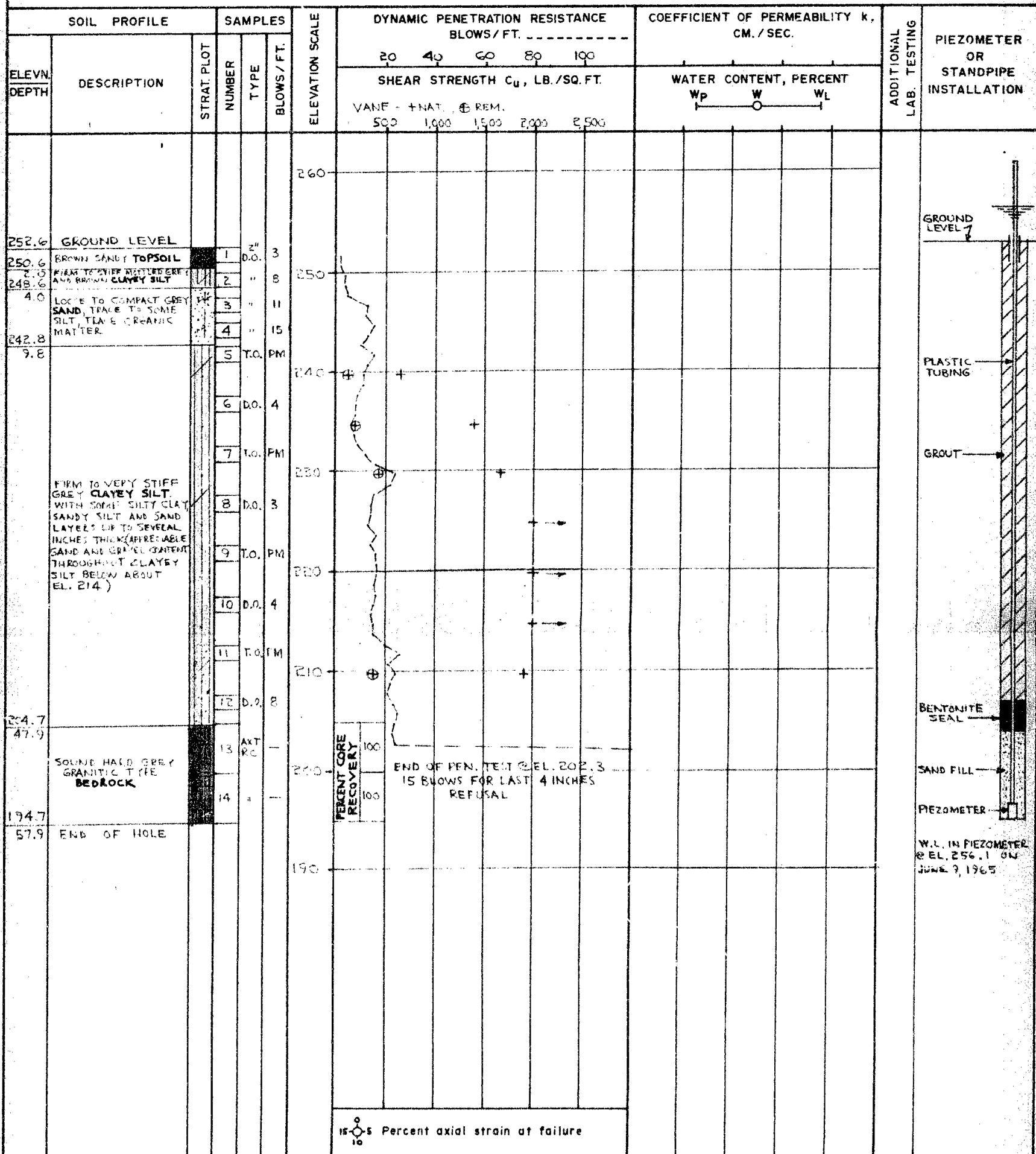
WASH BORING

BOREHOLE DIAMETER

NX, BX &amp; AX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE  
1 INCH TO 10'-0"

GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED J.B.