

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40P1-69

W. P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. 1-89

HWY. No. _____

LOCATION PROP. COCKSHUTT
RD. BRIDGE, BRANTFORD

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. ONE

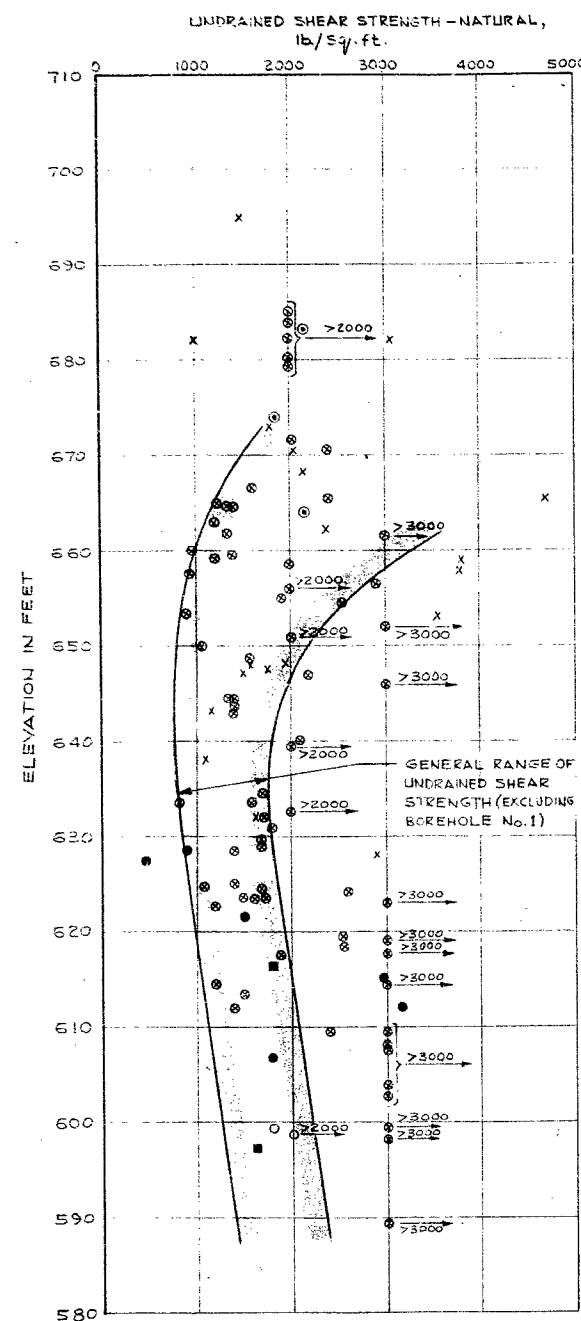
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REPORTS (1 of 3, 2 of 3 & 3 of 3) -

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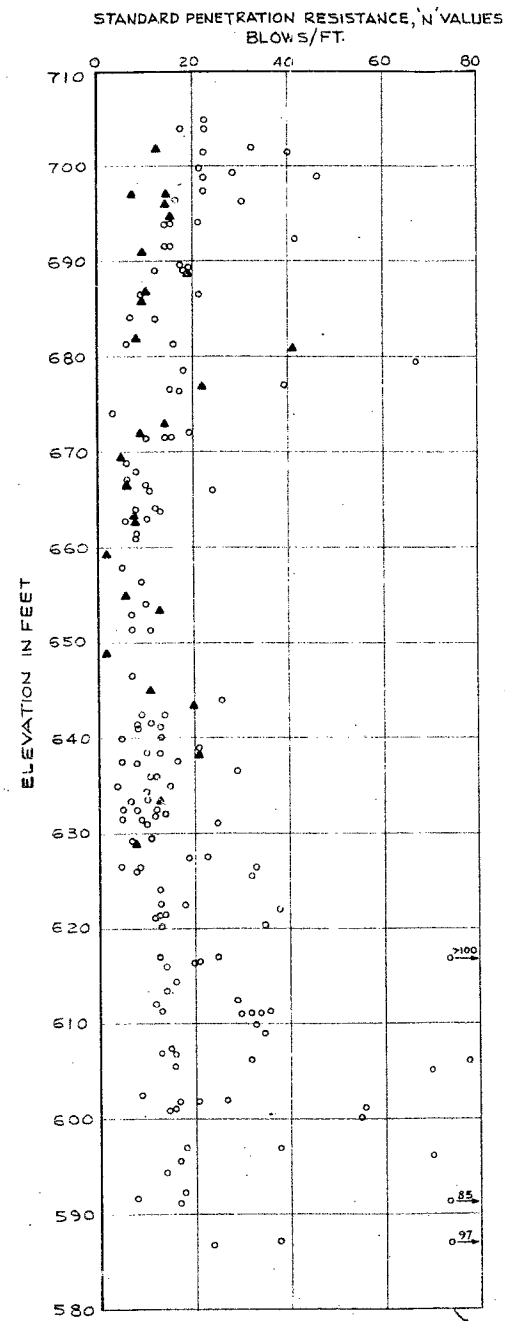
REPORT 3 of 3

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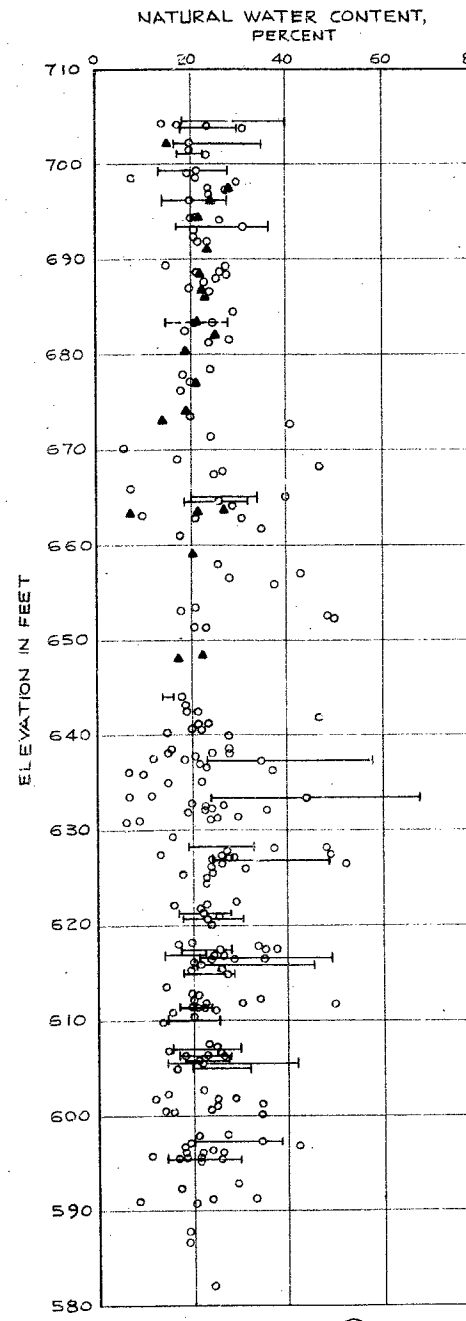
SUMMARY OF ENGINEERING PROPERTIES "LACUSTRINE DEPOSITS" FIGURE 6



PLOT A



PLOT B



PLOT C

LEGEND

PLOT A

- FIELD VANE
 - QUICK TRIAXIAL TEST
 - UNDRAINED DIRECT SHEAR TEST
 - X FIELD VANE
 - ⊙ QUICK TRIAXIAL TEST
- PRESENT INVESTIGATION, BY GOLDER ASSOCIATES.
- PREVIOUS INVESTIGATION, BY OTHERS

PLOT B

- STANDARD PENETRATION RESISTANCE, BLOWS/FT.
 - ▲ STANDARD PENETRATION RESISTANCE, BLOWS/FT.
- PRESENT INVESTIGATION, BY GOLDER ASSOCIATES
- PREVIOUS INVESTIGATION, BY OTHERS.

PLOT C

- Wp W WL
 - Wp W WL
- PRESENT INVESTIGATION, BY GOLDER ASSOC.
- PREVIOUS INVESTIGATION, BY OTHERS

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

40 P1-69
GEOCRE No.

Date: MAY 7, 1975

Golder Associates

Drawn W.E.
Chkd. W.E.
Appd. W.E.

40 P1-69 3 of 3 REPORT



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

40 P1-69

GEOCRE No.

1 of 3

REPORT
TO

McCORMICK RANKIN & ASSOCIATES LIMITED
PRELIMINARY SUBSURFACE INVESTIGATION
PROPOSED REPLACEMENT STRUCTURE
COUNTY ROAD NO. 4
BRANT COUNTY ONTARIO

Distribution:

- 6 copies - McCormick Rankin & Associates Limited,
Port Credit, Ontario.
- 2 copies - H. Q. Golder & Associates Ltd.,
Mississauga, Ontario.

October 1973

73154

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ABSTRACT

The results are presented of a preliminary geotechnical investigation carried out along the south bank of the Grand River adjacent to the existing bridge which carries County Road No. 4 (Cockshutt Road) over the Grand River near Brantford, Ontario. The investigation comprised two deep borings which were put down to determine potential problems related to the stability of the south bank of the river slope.

Available geological information indicates that the principal soils along the south river bank consist of glacio-lacustrine stratified to varved silts and clays with minor lenses of sand and that the bedrock surface is relatively deep. The results of this geotechnical investigation are consistent with the geological information. The layering within the lacustrine soils is extremely irregular and varies from a silty clay with thin silt partings to a primarily cohesionless silt containing one to two inch thick silty clay to clayey silt layers. The undrained shear strength varies from about 1,200 lb/sq. ft. to in excess of 3,000 lb/sq. ft., with an average value of about 1,800 to 2,000 lb/sq. ft.

A preliminary stability analysis of the existing slopes indicates that the factor of safety against rotational slope movement is about 1.2 under the present piezometric conditions. However, under anticipated spring time piezometric conditions, the existing slopes will have a factor of safety of about 1.0 (based upon estimated values for the effective stress shear strength parameters).

Because of the present marginal slope conditions, it is recommended that overall slopes of about 4 (horizontal) to 1 (vertical) should be used for feasibility design computations and that the south abutment for any proposed bridge structure should be founded on steel H-piles driven to a firm founding strata. Prior to carrying out final design, a detailed geotechnical investigation should be carried out to determine the actual effective stress shear strength parameters and pertinent geotechnical properties of the foundation soils.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by McCormick Rankin & Associates Limited, Consulting Engineers to the County of Brant, to carry out a preliminary subsurface investigation along the south bank of the Grand River adjacent to the existing bridge which carries County Road No. 4 (Cockshutt Road) over the Grand River near Brantford, Ontario. The purpose of the preliminary investigation was to determine the subsurface conditions within the south bank and to identify any geotechnical factors which would be significant in selecting an alternate route and location for a replacement bridge structure, or which would be associated with the repair and reconstruction of the existing bridge.

DESCRIPTION OF THE PROJECT

It is understood that the existing County Road No. 4 (Cockshutt Road) bridge over the Grand River Valley is an approximately 620 ft. long structure consisting of four approximately 135 ft. long central spans and two approximately 40 ft. long end spans. It is further understood that the most southerly pier (which is located at about the mid-height of the south valley bank) has moved laterally about two feet, and that the entire bridge appears to be in compression due to horizontal displacement of the south abutment.

Visual examination of the existing south valley bank indicates the presence of numerous old failure zones and large portions of the entire slope appear to be marginally stable as there is evidence of slope creep in addition to the old failure zones. Based on discussions with local residents, it is understood

that a relatively large scale failure of the river bank slope about 100 to 200 ft. upstream from the existing bridge structure took place about 12 years ago.

The location of the existing structure is shown on Fig. 1 (following the text of this report) while general views of the south bank of the Grand River are shown on Fig. 2. A detailed photograph of the relative displacement between the bridge superstructure and the south abutment is shown on Fig. 3.

It is understood that at the present time, detailed engineering studies have not yet been completed with respect to various alternatives for the repair of the structure, or to the possible relocation of County Road No. 4 at alternative locations. Thus the boreholes which have been put down as a part of this investigation will have to be supplemented by detailed borings and associated laboratory testing once the overall engineering aspects of the problem have been assessed. Should it be desirable to relocate the structure, additional borings will have to be put down at foundation locations as well as along the north approach embankment.

AVAILABLE GEOTECHNICAL INFORMATION

Based upon the available geological mapping, the bedrock in the vicinity of the site is located between about elev. 550 and elev. 575 and consists of dolomitic limestone of the Salina Formation, containing shale layers and gypsum inclusions.

South of the existing bridge, the land surface rises

to above elev. 725, and the Grand River has incised into glacio-lacustrine deep water sediments which were mainly created by Glacial Lake Warren. The deposits are mainly stratified to varved silt and clay with minor lenses of sand, with the silt sizes being generally more predominant. Detailed geological studies for a site one mile south-east of Brantford have indicated the presence of at least 200 varves between elevations 650 and 675. Small landslide scarps readily develop in these sediments and old landslides are reported on the geological mapping about 1.5 miles downstream from the site. It has been suggested that slope movement may occur readily along the varves in permeable zones under high piezometric pressures.

North of the existing bridge, the surficial deposits consist of recent alluvium and contain a mixture of silt, sand, gravels with some clay and organic deposits. Ground surface elevation is generally about elev. 650 or lower, while the Grand River was estimated to be at about elev. 625 during the time of this investigation.

Previous geotechnical investigations carried out about 1.5 miles upstream from the site had indicated that the glacio-lacustrine sediments consisted of very stiff to hard brown to grey layered silty clay and clayey silt with some sand and silt seams. A deep clayey silt till was encountered between elev. 650 and 613. The water content of the layered silty clay and clayey silt varies between 20 and 40 per cent, and in some cases, the individual strata are at liquidity indices greater than one.

PROCEDURE

The field work for this investigation was carried out between September 6 and 12, 1973. During this period, two boreholes were put down through the south bank of the existing Grand River Valley at locations which are approximately 500 ft. apart (see Fig. 1 for location). Borehole 1 was put down to determine the general conditions adjacent to the existing abutment, while Borehole 2 was located at one of several possible alternative realignment locations.

The borings were advanced to depths of about 75 ft. and 85 ft. below existing ground surface using a track-mounted power auger supplied and operated by a specialist drilling contractor. In each boring, Standard penetration tests were carried out at about 10 ft. intervals of depth and samples of the subsoil were obtained using conventional 1½ in. I.D. split spoon sampling equipment. At alternating intervals of depth, relatively undisturbed two and three inch diameter thin-walled tube samples were obtained for detailed examination of the soil structure. In addition, the undrained shear strength of the cohesive overburden deposits was determined by means of in situ vane tests. Following completion of augering in BH 2, a dynamic penetration test was driven from the bottom of the borehole to refusal at a depth of about 100 ft. below ground surface.

Following completion of each of the borings, stand-pipes and piezometers were sealed into the open holes to permit monitoring of the groundwater levels within the existing valley bank.

The field work was supervised throughout by a member of our engineering staff who located the borings in the field, directed the drilling and sampling operations and cared for the samples. All of the samples obtained during this investigation were brought to the laboratory for detailed examination and testing.

The ground surface elevation at each of the borehole locations was determined in the field by a member of our engineering staff. The elevations were referenced to a nail on the top of the south end of the west concrete railing in the existing bridge. The elevation of this bench mark was given to us as elev. 675.52 by the County Engineer's Office.

SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the borings together with the results of the in situ sampling and testing are given on the Record of Borehole Sheets which follow the text of this report. Photographs, taken of selected two inch diameter thin-walled Shelby tube samples after being extruded and split longitudinally, are shown on Fig. 4, in conjunction with the results of detailed laboratory identification tests. The results of all the laboratory tests are shown on the Record of Borehole Sheets.

Based on the results of this preliminary investigation, the south bank of the existing Grand River Valley slope is underlain to between about elevations 585 and 600 by an extensive deposit of irregularly layered clayey silt, silty clay, and silt.

As is evident in Fig. 4, the layering is extremely irregular, varying from an essentially silty clay with thin silt partings to a primarily cohesionless silt containing one to two inch thick silty clay to clayey silt layers. Based upon the results of the Atterberg classification tests, the silty clay may be classified as an inorganic clay of high plasticity, (with a liquid limit of about 50 to 60, and a plasticity index of about 30 to 35) whereas the clayey silt layers may be classified as an inorganic clay of low to medium plasticity (with a liquid limit of about 30 to 35, and a plasticity index of about 10 to 15). The Atterberg limit tests also confirmed the presence of non-plastic silt layers.

Based upon the results of the in situ vane tests, the undrained shear strength of the layered deposit varies from about 1,200 lb/sq. ft. to in excess of 3,000 lb/sq. ft., with an average value of about 1,800 to 2,000 lb/sq. ft. There appears to be little variation in undrained shear strength with depth and it is felt that the irregular variation indicated on the Record of Borehole sheets is primarily due to the irregular layering of the deposit. Below elev. 610 in Borehole 1, the in situ vane shear strength is consistently above 3,000 lb/sq. ft.

Based upon the results of the vane shear tests, together with the results of the Standard penetration tests (or 'N' values), the consistency of the layered clayey silt to silty clay is generally stiff to very stiff, and the cohesionless silt layers are in a generally compact state of packing.

Although an extensive laboratory testing program to

determine the effective shear strength parameters of the various strata was outside the scope of this preliminary investigation, shear strength parameters have been estimated based upon measured index properties. It is estimated that the effective angle of shearing resistance of the silty clay layers is about 20 to 22 degrees, the effective angle of shearing resistance of the clayey silt layers is about 25 to 29 degrees, and the effective angle of shearing resistance of the cohesionless silt is about 30 degrees. Within the cohesive layers, the effective cohesion, c' , will vary with the overburden pressures and softening which may have occurred, and it is anticipated that c' is between 100 and 300 lb/sq. ft.

Water levels measured in the standpipes and piezometers sealed into the boreholes (as indicated on the Record of Borehole sheets) indicate that the piezometric water level within the slope varies between about elev. 650 and 655.

DISCUSSION

A section through the existing bridge structure at the south valley wall is shown in Fig. 5, with the information obtained at Borehole 1 transferred to the centreline. At this location, the valley slope has been steepened by the construction of the approach embankment and a section along Line 1, just upstream and to the west of the bridge is shown on Fig. 6. At this location, the average slope is 3.1 horizontal to 1 vertical below the elevation of the roadway.

A stratigraphic section along Line 2 is shown on Fig. 7, while existing valley wall profiles (drawn to a reduced scale) are shown on Fig. 8. Based upon these sections, a

typical slope of 2.9 horizontal to 1 vertical has been used for subsequent approximate analyses of the present slope stability. Similarly, the piezometric elevations recorded at the boreholes have been used in conjunction with Fig. 6 and 7 to determine approximate overall values for the pore pressure coefficient, r_u , where r_u is defined as

$$r_u = \frac{\text{piezometric pressure}}{\text{total pressure}} = \frac{\gamma_w h_w}{\gamma h}$$

An average value of r_u of 0.15 was computed for the groundwater levels observed at the time of this investigation. These groundwater levels were measured near the end of a relatively dry summer, and this value of r_u was therefore interpreted as a minimum value. For spring conditions, when groundwater levels might be relatively high, an upper limiting value of r_u of 0.3 was computed assuming that the upper piezometric surface is controlled at about elev. 675 by the small drainage creek to the south of Borehole 2.

Slope Stability Several recent slope scarps are present upstream and downstream of the site. The upper portion of the scarps may be seen in Fig. 2, where the general view of the bank downstream of the structure indicates a typical slump topography. These observations would indicate that the south slopes are in a marginally stable condition, but it is not known from this investigation if the slope movements have been along lenses in the layered deposits, or have been typical rotational circular arc failures. For preliminary purposes, it has been assumed that the results of average strengths along a circular shearing surface would be similar to the detailed computations along a sliding plane.

Based upon the average undrained shear strength of 1,800 lb/sq. ft., an overall slope height of 87 ft., and an average slope angle of 2.9 horizontal to 1 vertical, the results of a total stress stability analysis indicate a factor of safety of about 1.3 against movement. However, the undrained shear strength in the upper portion of Borehole 1 is typically in the range of 1,400 lb/sq. ft., and for this condition, the factor of safety under total stress conditions is about 1.0.

In order to confirm these trends, a series of effective stress stability analyses were carried out for the range of effective cohesion, c' , and effective shearing resistance ϕ' , which might be anticipated for these soils. The analyses were carried out using stability charts which assume a circular arc failure surface, under the assumption that the results of the circular arc and the results of a sliding plane analysis would be similar under conditions close to failure.

For an $r_u = 0.15$ (which is typical of the present piezometric conditions) a factor of safety of 1.2 would be present for a range of effective stress strength parameters from $c' = 200$ lb/sq. ft. and $\phi' = 20^\circ$, to $c' = 75$ lb/sq. ft. and $\phi' = 25^\circ$. However, for a value of $r_u = 0.30$ (which is probably the condition existing under high spring groundwater conditions), the factor of safety would be about 1.0 (see Fig. 9a).

These results are consistent with the reported results of the slope movement about 12 years ago. The range of effective stress strength parameters from these calculations is similar to those which would be estimated from the results of the classifications tests.

(The consistency of the layered clayey silt, silty clay and silt adjacent to County Road No. 4 is appreciably softer than that for the apparently similar deposit about 1.5 miles upstream. At Borehole 1, the typical Standard Penetration Resistance, or 'N' value, is about 9 blows per foot above elev. 620, while vane shear strengths increase moderately with depth to about 1,500 lb/sq. ft. For the same range in geodetic elevation upstream, the average Standard Penetration Resistance, or 'N' value, is about 60 blows per ft., while the undrained shear strengths in the upper zone are above 2,000 lb/sq. ft. It is suggested that the seepage and piezometric conditions are not similar between the two sites, and that the present difference in slope angles are controlled by these factors).

For design purposes, reconstruction or a new alignment will require slopes which are stable against long term movement. For a value of $c' = 75$ lb/sq. ft.. and $\phi' = 25^\circ$, the variation in factor of safety with slope angle and pore pressure coefficient was computed. The results are shown graphically on Fig. 9(b), where it is indicated that for long term stability with a factor of safety in the order of 1.5, an overall slope angle of 4 horizontal of 1 vertical will be required.

It may be possible to modify the slope angles by the use of drainage control measures such as properly wicked perforated drains and the like. However, it will be necessary to determine the zone or zones of seepage adjacent to the proposed final location before detailed recommendations can be prepared.

Abutment Foundations The design assumptions relative to the stability of the existing south abutment are not known to us,

nor are the as-constructed details available. As designed the south abutment is relatively tall (about 55 ft.) and slender (about 18 ft. wide at the base), and extends for a length of about 35 ft. parallel to the river bank. As such, the present abutment would be subjected to appreciable lateral earth pressures which would develop. At the present bridge location, the overall slope is about 3.1 horizontal to 1 vertical and the slope at this location is only marginally more stable than the sections which have been analysed along line 2. The lateral pressures would be greater than normal against the abutment due to the sloping fill and piezometric pressures, while the passive resistance which might be mobilized on the downstream side would be reduced by the gradual creep of the slope. The amount of movement which has occurred (see Fig. 3) suggests that rotation about the base of the abutment due to horizontal earth pressures has occurred.

In order to minimize the earth pressures which would be associated with any future construction, it is recommended that the abutments be founded on battered end-bearing steel I-piles which are driven through the layered deposits and founded on the underlying till or bedrock. However, these piles will be subjected to unbalanced lateral forces from the earth slope, and it will still be necessary to flatten the overall slope at the abutment to about 4 horizontal and 1 vertical.

The water content of the soils within the upper portion of Borehole 1 is relatively high and possibly on the wet side of the optimum water content for this soil. The material will be marginally suitable for the construction of structural embankments but may be placed on the outer portions of any slopes. It is anticipated that similar conditions will also occur at intermediate

locations between the two boreholes put down as a part of this investigation.

Depending upon the final design location for the bridge structure, it may be possible to improve the piezometric conditions at the site by the installation of horizontal drainage measures to reduce the piezometric pressures. Before details of these drainage measures can be established, it will be necessary to determine the potential critical surfaces of sliding and the continuity of any stratigraphic layers at the site.

RECOMMENDATIONS

For preliminary design purposes, earth slopes should be constructed at overall slopes of 4 horizontal to 1 vertical. Toe protection, including rip rap, will be necessary along the lower portions of the slope at the Grand River in order to minimize any potential for erosion. Abutments and piers within the slopes should be founded on steel H-piles.

Once the final design details have been established, it will be necessary to carry out a detailed geotechnical investigation along the proposed centreline along the north and south approaches as well as within the river. In addition, borings should be put down at the locations of any proposed excavations in order to determine the stratigraphy and piezometric conditions.

At least one of the detailed sampled borings should be put down with large diameter casing using conventional wash-borings techniques so that relatively undisturbed samples may be obtained for detailed laboratory testing.

Periodic measurements should be taken at the piezometers during the coming winter and spring in order to determine the variation in piezometric pressures. In addition, the presently disused well which is located midway between boreholes 1 and 2 should be monitored. These observations will permit more realistic estimates to be made of the maximum piezometric pressures which might be anticipated.



D. L. Townsend, P.Eng.



W. S. Freeman, P.Eng.

DLT:WSF:jb

73154

October 15, 1973

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) <i>Unit weight</i>	
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_p	coefficient of consolidation
T_v	time factor = $c_p t / d^2$ (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

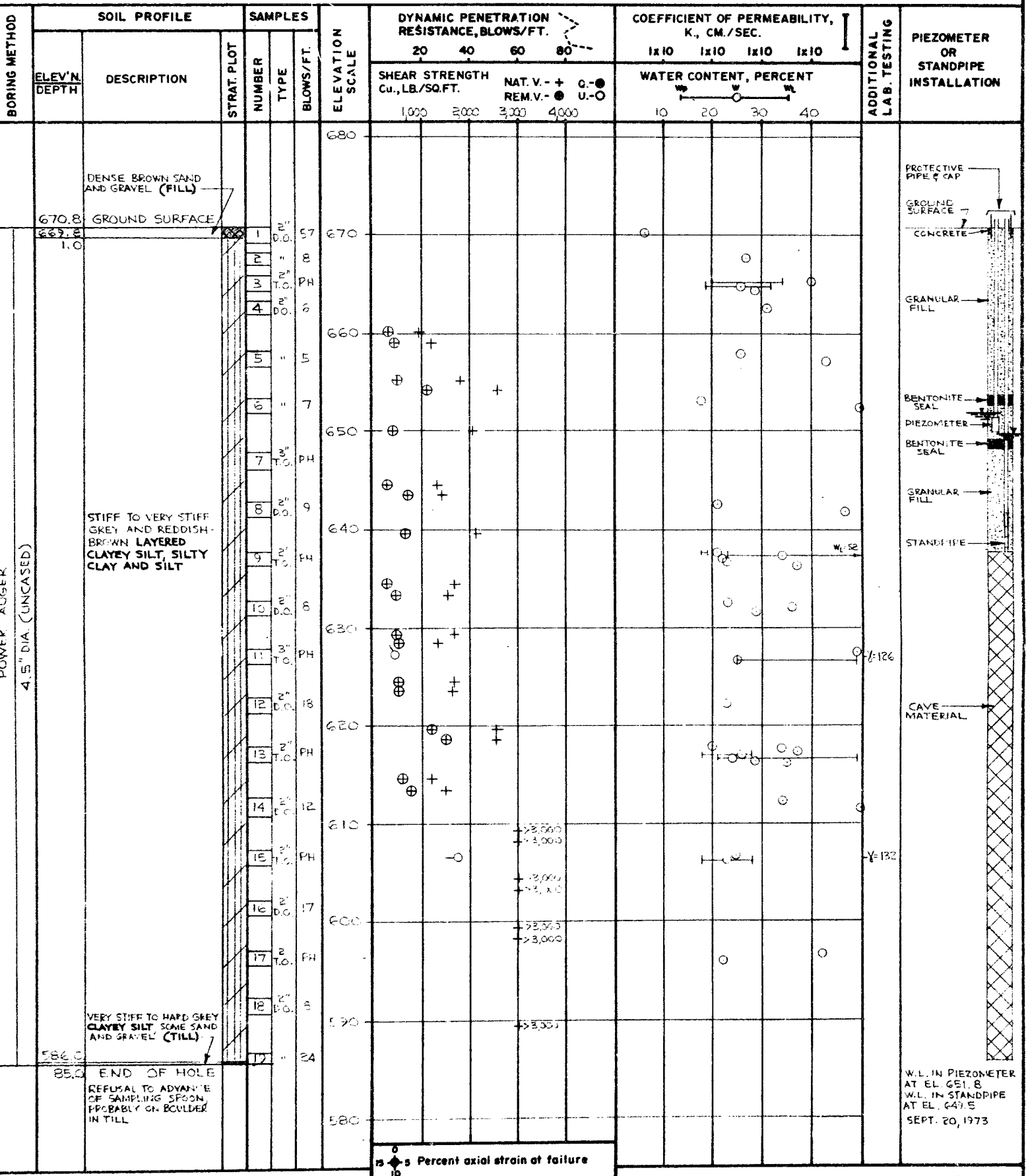
LOCATION See Figure 1

BORING DATE SEPT 6 - 10, 1973

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



Golder Associates

DRAWN J.A.
CHECKED 451

VERTICAL SCALE
1 IN. TO 10 FT.

RECORD OF BOREHOLE 2

LOCATION See Figure 1

BORING DATE

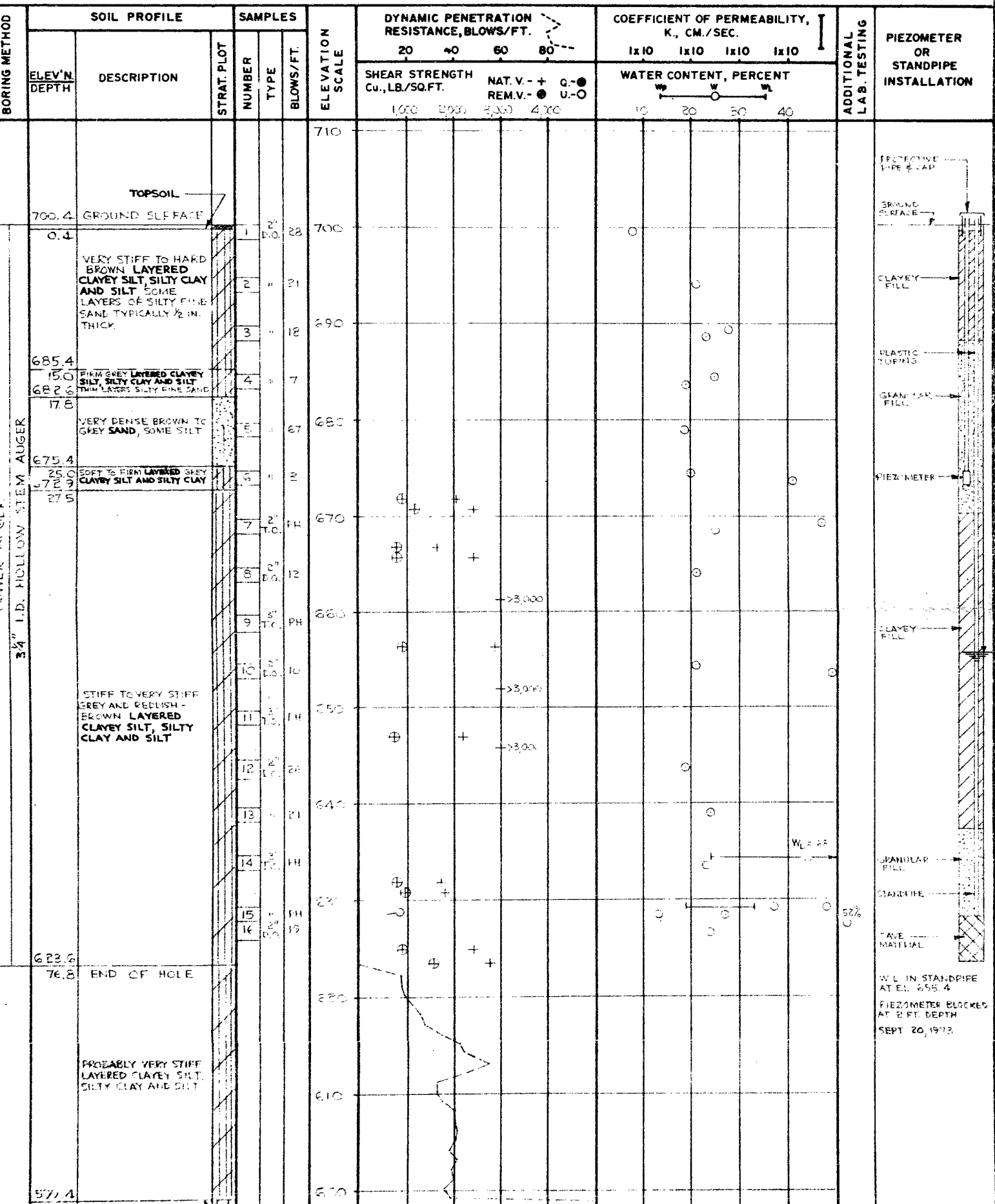
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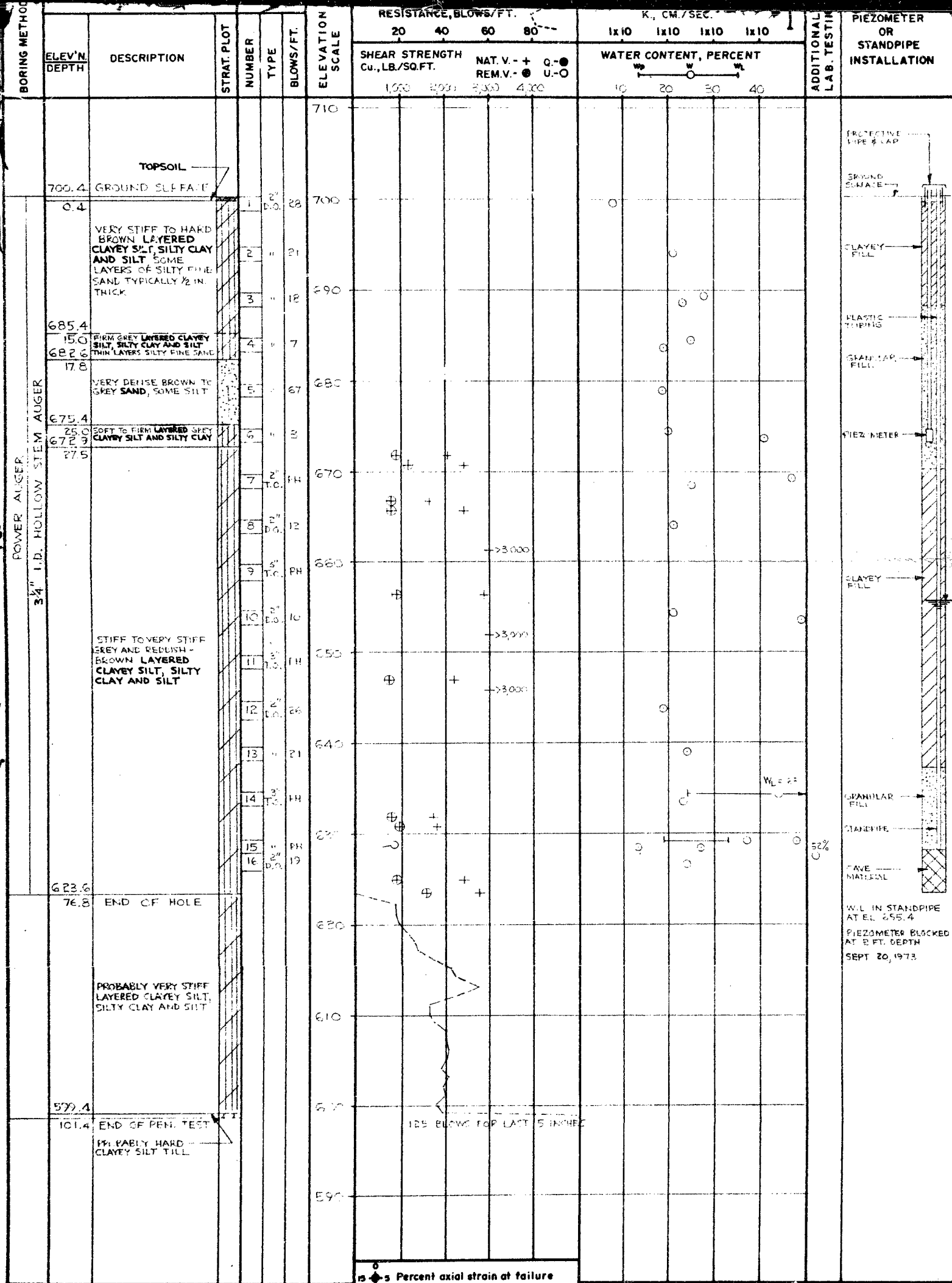
DATUM

GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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





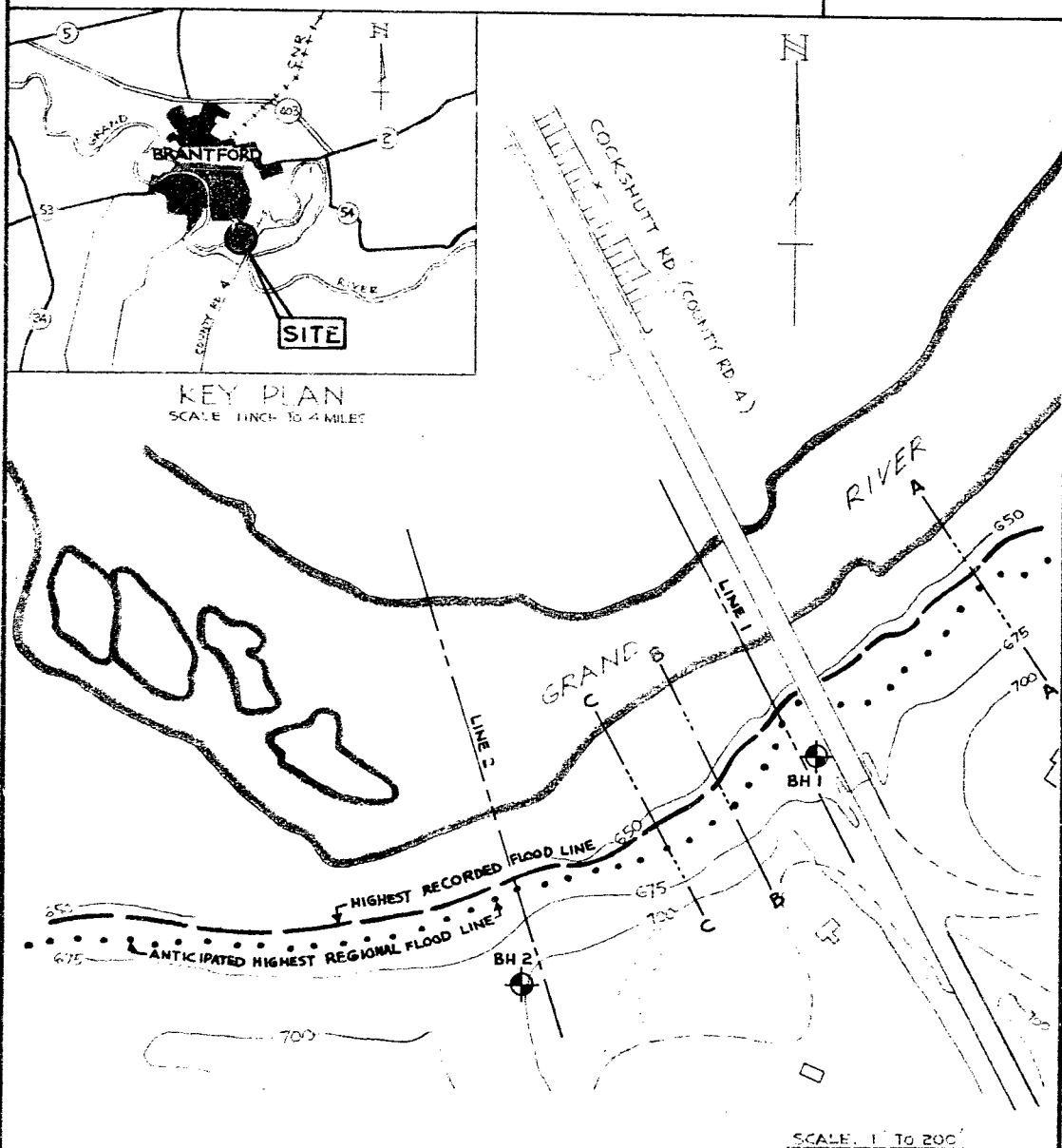
VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN J.A.
CHECKED G.S.

SITE AND BORING PLAN

FIGURE 1



LEGEND



BOREHOLE IN PLAN

REFERENCE

UNCONFINED AND UNDAIED PLAN SUPPLIED
BY MCCORMICK FRANKIN & ASSOCIATES LTD

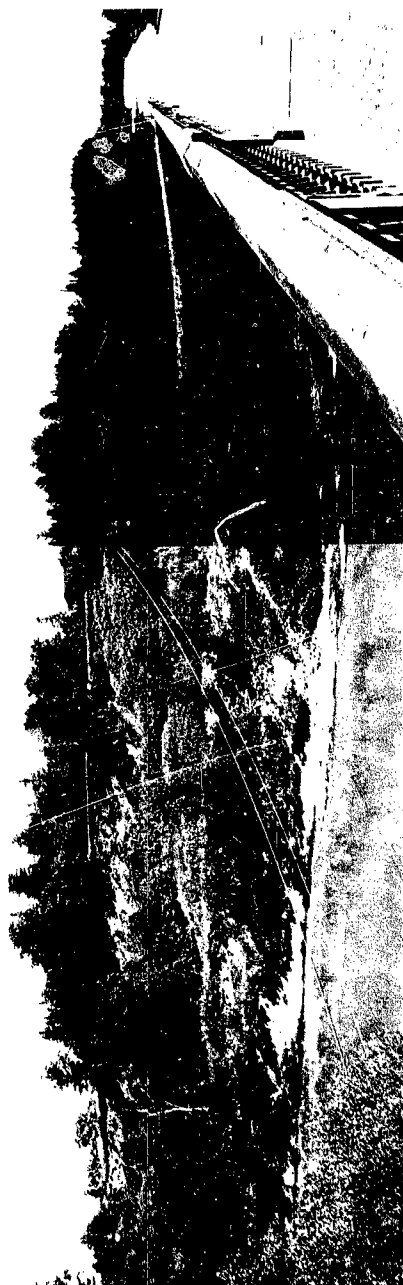
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Golder Associates

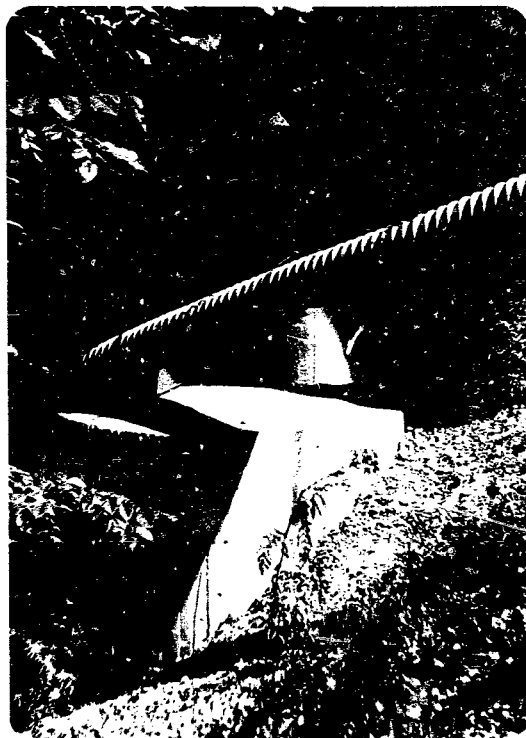
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Chkd. GL
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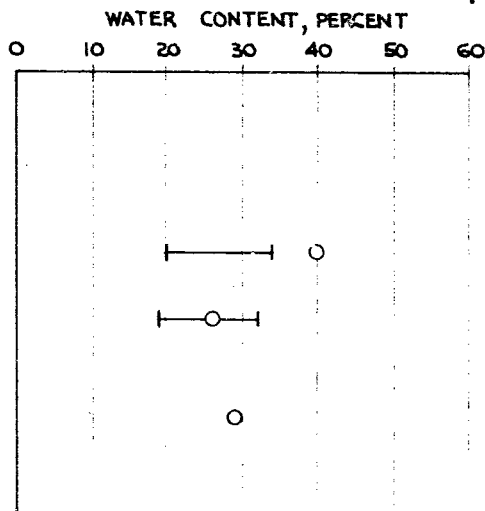
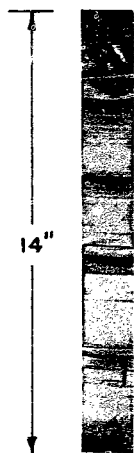
GENERAL VIEW — SOUTH BANK, UPSTREAM OF STRUCTURE



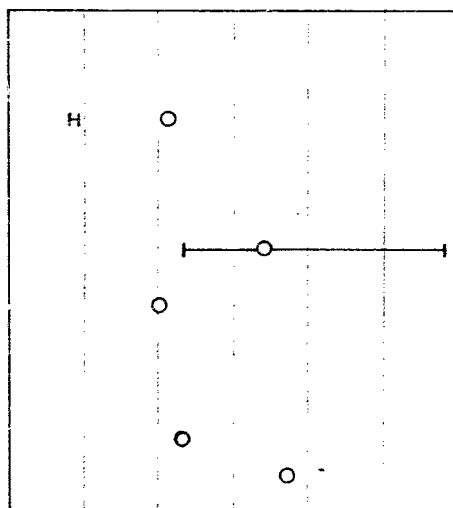
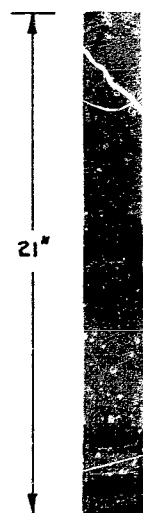
GENERAL VIEW — SOUTH BANK, DOWNSTREAM OF STRUCTURE



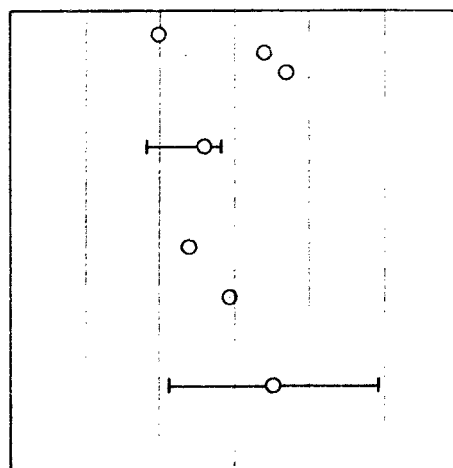
RELATIVE DISPLACEMENT — SOUTH ABUTMENT

DETAILED SOIL
PROPERTIES

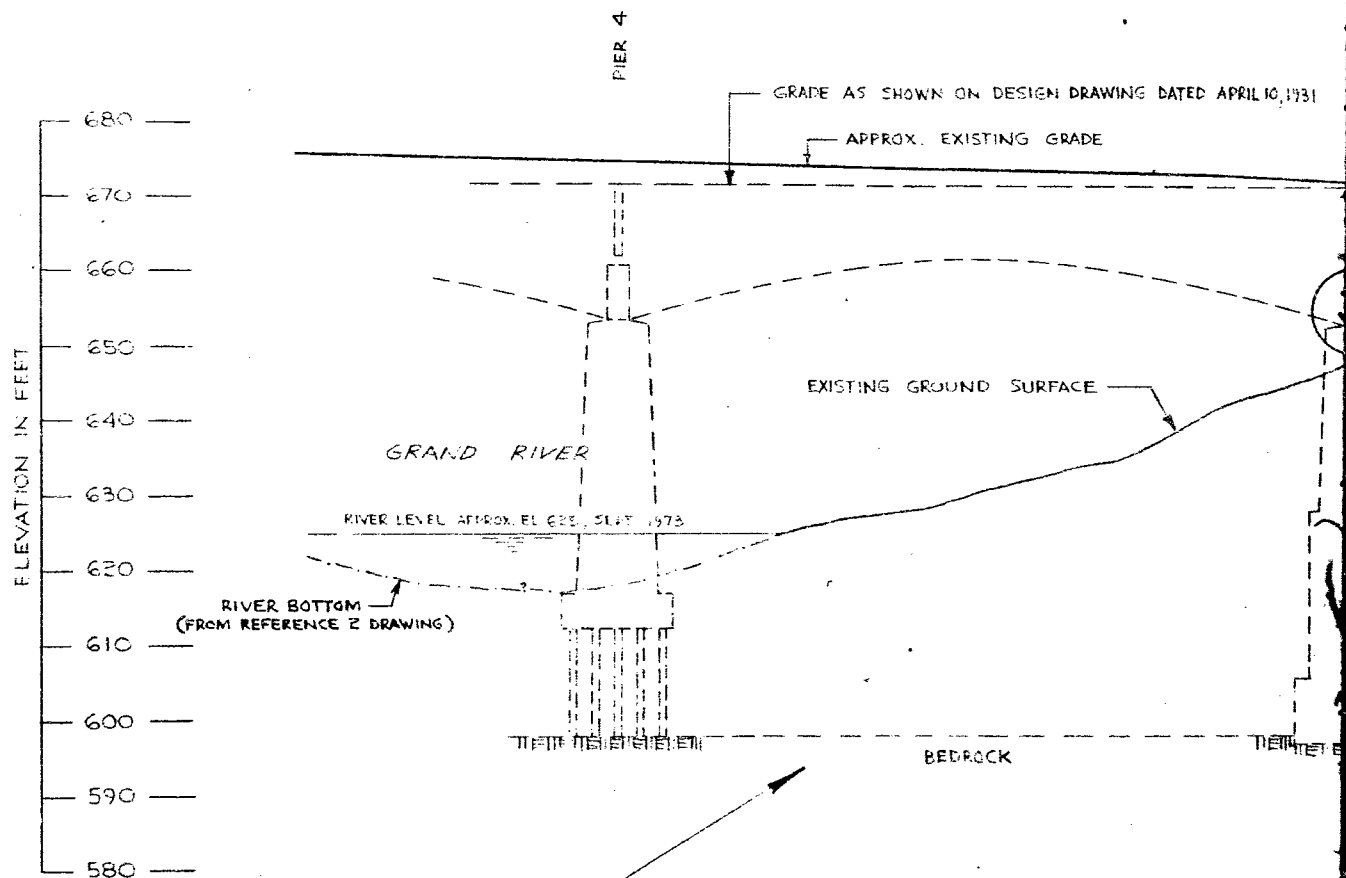
BOREHOLE 1, SAMPLE 3
DEPTH 5.0' TO 6.5'
ELEV. 665.8 TO 664.3



BOREHOLE 1, SAMPLE 9
DEPTH 33.0' TO 34.6'
ELEV. 637.8 TO 636.2



BOREHOLE 1, SAMPLE 13
DEPTH 53.0' TO 54.5'
ELEV. 617.8 TO 615.3



NOTE: ALL BRIDGE STRUCTURE DETAILS AND BEDROCK SURFACE
TAKEN FROM ONTARIO DEPARTMENT OF PUBLIC HIGHWAYS
DESIGN DRAWING No. 1, DATED APRIL 10, 1931.
(ACTUAL AS CONSTRUCTED INFORMATION NOT AVAILABLE)

SOUTH APLTMENT

NOTE




Data regarding the various strains have been obtained at biological laboratories in the U.S. and increasingly because of the interest has been obtained from industrial laboratories as well and have been made known.

SECTION ALONG OF EXISTING

LEGEND

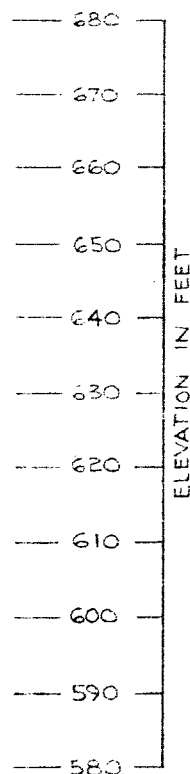
- ① BOREHOLE IN ELEVATION
- 8 STANDARD PENETRATION TEST
- ≡ WATER LEVEL IN PIEZOMETER

STRATIGRAPHY

-  DENSE BROWN SAND AND GRAVEL (FINE)
-  STIFF TO VERY STIFF GREY AND REDDISH CLAYEY SILT, SILTY CLAY AND SILT
-  VERY STIFF TO HARD GREY CLAYEY SILT

REFERENCES

- 1) UNNUMBERED AND UNDATED PLAN BY MCCORMICK RANKIN & ASSOCIATES
- 2) PHILIPS PLANNING & ENGINEERING LIMITED UNNUMBERED DRAWING DATED JAN. 1931 CONSERVATION AUTHORITY, BRANTFORD CROSS SECTIONS 30-35. SUPPLIED BY
- 3) ONTARIO DEPARTMENT OF PUBLIC HIGHWAYS DATED APRIL 10, 1931 - COCKSHUTT BRANTFORD GENERAL PLAN, ELEVATION AND SECTION BY MCCORMICK RANKIN & ASSOCIATES LIMITED



SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONNECTION WITH ACCOMPANYING REPORT

NOTE: FOR LOCATION

Date SEPT. 25, 1973

SECTION ALONG CENTRELINE OF EXISTING BRIDGE

FIGURE 5

LEGEND



BOREHOLE IN ELEVATION



STANDARD PENETRATION RESISTANCE, 'N' VALUE, BLOWS/FT.



WATER LEVEL IN PIEZOMETER OR STANDPIPE, SEPT. 20, 1973

STRATIGRAPHY



DENSE BROWN SAND AND GRAVEL (FILL)



STIFF TO VERY STIFF GREY AND REDDISH-BROWN LAYERED
CLAYEY SILT, SILTY CLAY AND SILT



VERY STIFF TO HARD GREY CLAYEY SILT, SOME SAND AND GRAVEL (TILL)

REFERENCES

- 1) UNNUMBERED AND UNDATED PLAN SUPPLIED BY MCCORMICK RANKIN & ASSOCIATES LTD.
- 2) PHILIPS PLANNING & ENGINEERING LIMITED, JOB No 72010, UNNUMBERED DRAWING DATED JAN. 1973 - GRAND RIVER CONSERVATION AUTHORITY, BRANTFORD FLOODPLAIN STUDIES CROSS SECTIONS 30 - 35. SUPPLIED BY MCCORMICK RANKIN & ASSOC. LTD.
- 3) ONTARIO DEPARTMENT OF PUBLIC HIGHWAYS, DRAWING No 1 DATED APRIL 10, 1931 - COCKSHUTT BRIDGE, BRANTFORD, ONTARIO, GENERAL PLAN, ELEVATION AND SECTION SUPPLIED BY MCCORMICK RANKIN & ASSOCIATES LTD.

40P1-69

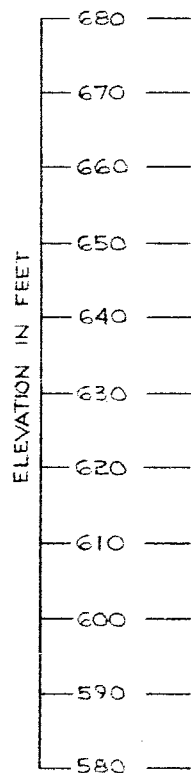
GEOREG No.

NOTE: FOR LOCATION OF SECTION, REFER TO FIG. 1

Date SEPT. 25, 1973

Golder Associates

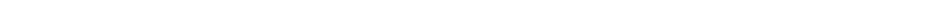
Drawn J.A.
Chkd. U.S.
Appd. [Signature]

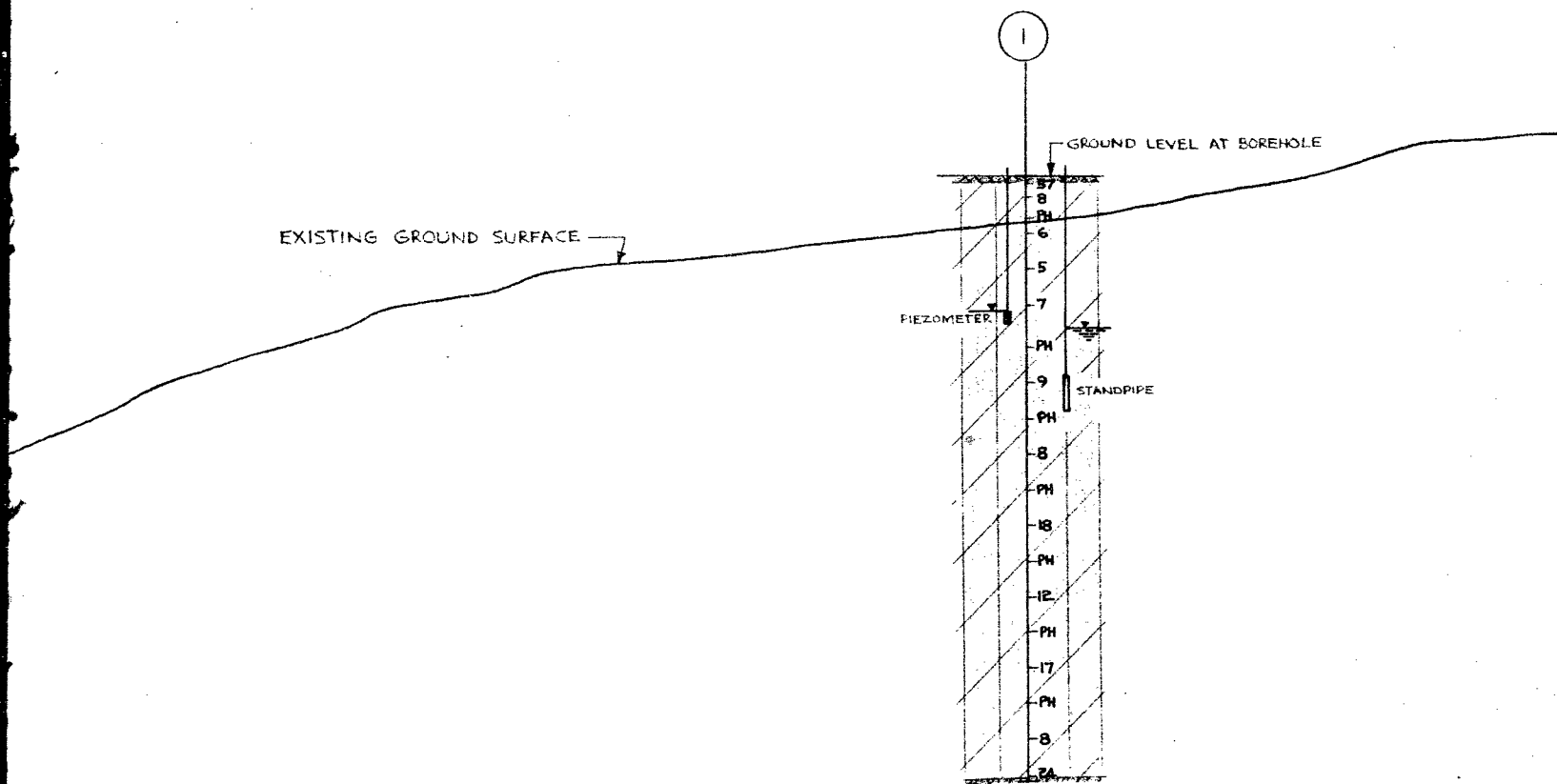


GRAND RIVER

RIVER LEVEL APPROX. EL 625, SEPT. 1973

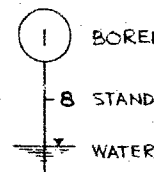
RIVER BOTTOM
(FROM REFERENCE 2 DRAWING)





SCALE: 1" TO 20'

LEGEND

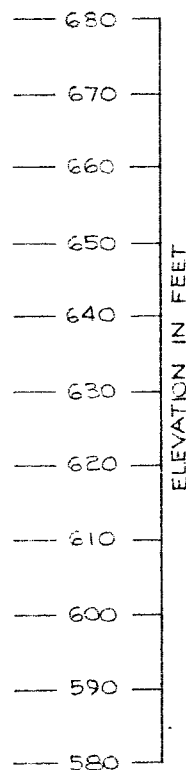


STRATIGRAPHY



NOTE

Date concerning the various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geologic correlation and as may vary from that shown. For complete stratigraphy of each borehole refer to the record of borehole sheet.



REFERENCE

- 1) UNNUMBERED AND UNDATED PLAN SUPPLIED BY MCCORMICK RANKIN & ASSOCIATES LTD.
- 2) PHILIPS PLANNING & ENGINEERING LIMITED, JOB No 72010, UNNUMBERED DRAWING DATED JAN. 1973 - GRAND RIVER CONSERVATION AUTHORITY, BRANTFORD FLOODPLAIN STUDIES, CROSS SECTIONS 30-35, SUPPLIED BY MCCORMICK RANKIN & ASSOCIATES LTD.




NOTE: FOR LOCATION

Date SEPT. 20, 1973

STRATIGRAPHIC SECTION ALONG LINE 1

FIGURE 6

LEGEND

-  BOREHOLE IN ELEVATION
-  STANDARD PENETRATION RESISTANCE, 'N' VALUE, BLOWS/FOOT
-  WATER LEVEL IN PIEZOMETER OR STANDPIPE, SEPT. 20, 1973

STRATIGRAPHY



DENSE BROWN SAND AND GRAVEL (FILL)



STIFF TO VERY STIFF GREY AND REDDISH-BROWN LAYERED
CLAYEY SILT, SILTY CLAY AND SILT.



VERY STIFF TO HARD GREY CLAYEY SILT, SOME SAND AND GRAVEL (TILL)

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.

NOTE: FOR LOCATION OF SECTION REFER TO FIG. 1

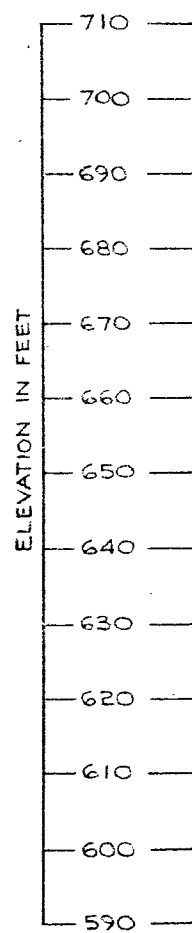
40P1-69

GEOGRAPHIC No.

Date SEPT. 20, 1973

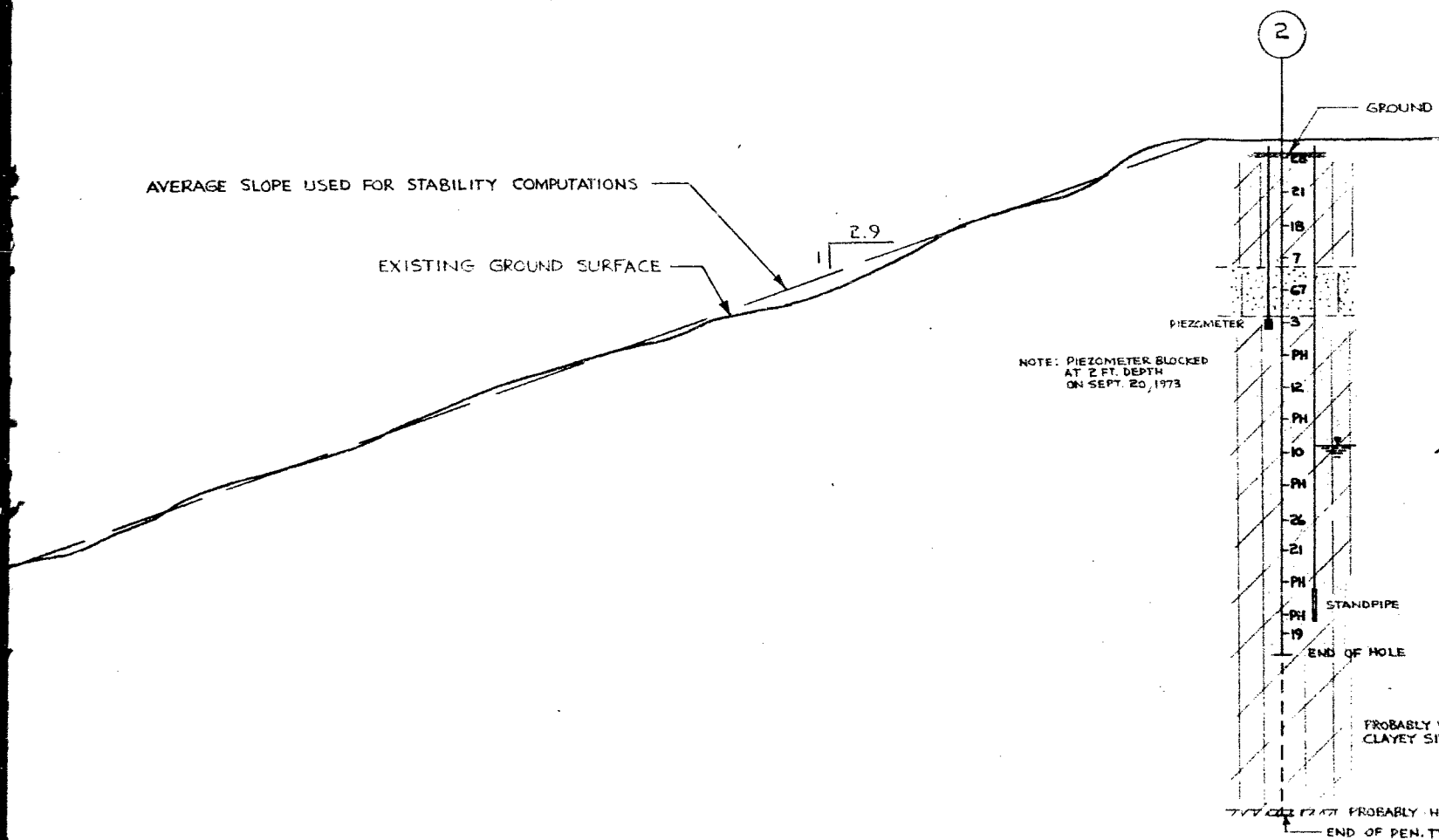
Golder Associates

Drawn J.A.
Chkd. GSE
Appd. DAS

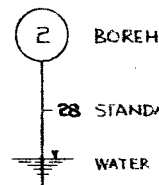


GRAND RIVER
RIVER LEVEL, APPROX. EL. 625, SEPT. 1973
RIVER BOTTOM
(FROM REFERENCE Z DRAWING)

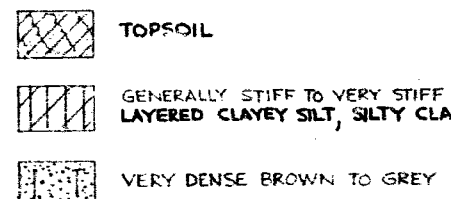
AVERAGE



LEGEND

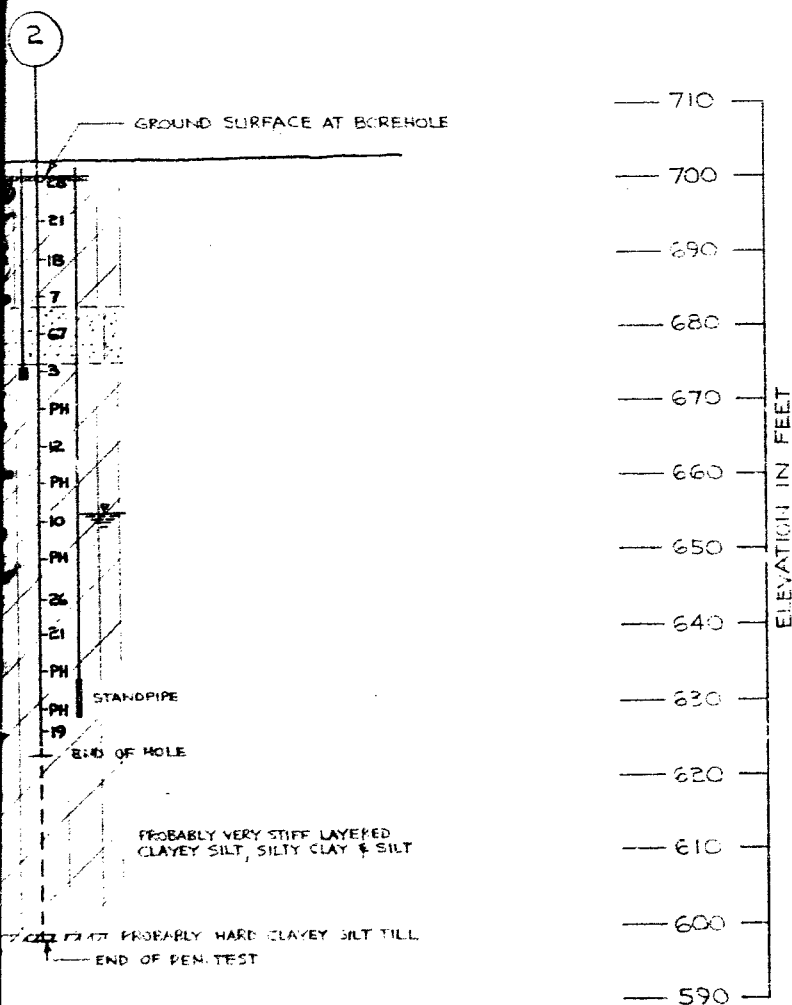


STRATIGRAPHY



NOTE

Date concerning the vertical stratigraphy has been analyzed of borehole locations only. The full stratigraphic section of the borehole has been identified from geological personnel and may vary from that shown. For detailed stratigraphic data borehole locations refer to the record of borehole charts.



REFERENCE

- 1) UNNUMBERED AND UNDATED PLAN SUPPLIED BY MCCORMICK RANKIN & ASSOCIATES LTD
- 2) PHILIPS PLANNING & ENGINEERING LIMITED JOB No 72010 UNNUMBERED DRAWING DATED JAN. 1973 - GRAND RIVER CONSERVATION AUTHORITY, BRANTFORD FLOODPLAIN STUDIES, CROSS SECTIONS 30-35, SUPPLIED BY MCCORMICK RANKIN & ASSOCIATES LTD.

NOTE: FOR LOCAL

Date SEPT. 20, 1973

STRATIGRAPHIC SECTION ALONG LINE 2

FIGURE 7

LEGEND



BOREHOLE IN ELEVATION

28

STANDARD PENETRATION RESISTANCE, 'N' VALUE, BLOWS/FOOT



WATER LEVEL IN PIEZOMETER OR STANDPIPE, SEPT. 20, 1973

STRATIGRAPHY



TOPSOIL



GENERALLY STIFF TO VERY STIFF GREY AND REDDISH-BROWN
LAYERED CLAYEY SILT, SILTY CLAY AND SILT



VERY DENSE BROWN TO GREY SAND, SOME SILT

NOTE:

Date concerning this section, Sept. 20, 1973, was obtained at
borehole location only. The stratigraphic section was
obtained from logs obtained from geological personnel who
may vary from 10 to 15 ft.
For details, refer to the borehole logs and the notes to
the record of borehole logs.

SPECIAL NOTE

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

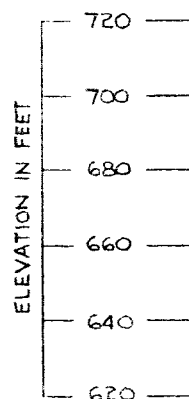
NOTE: FOR LOCATION OF SECTION REFER TO FIG. 1

40P1-69
GEOREG No.

Date SEPT. 20, 1973

Golder Associates

Drawn J.A.
Chkd. WSE
Appd. J.A.



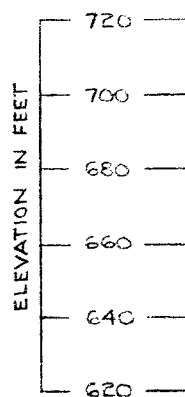
HIGHEST ANTICIPATED REGIONAL FLOOD LINE

HIGHEST RECORDED FLOOD LINE

GRAND RIVER

EXISTING GROUND S

SECTION



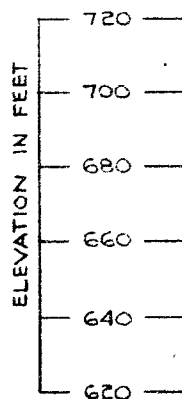
HIGHEST ANTICIPATED REGIONAL FLOOD LINE

HIGHEST RECORDED FLOOD LINE

GRAND RIVER

EXISTING GR

SECTION



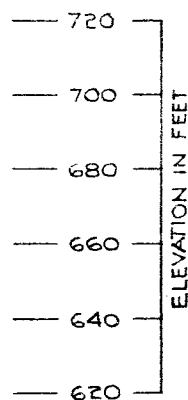
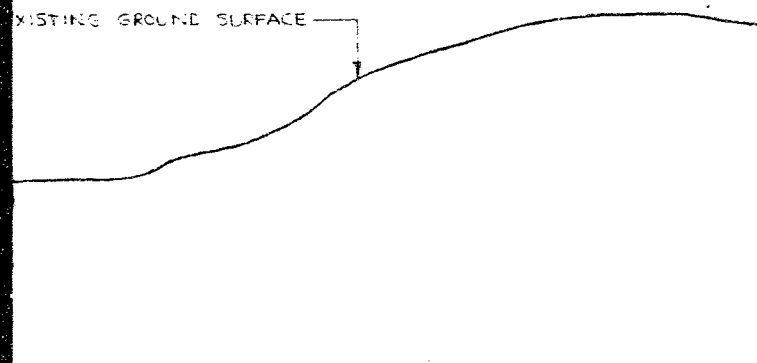
HIGHEST ANTICIPATED REGIONAL FLOOD LINE

HIGHEST RECORDED FLOOD LINE

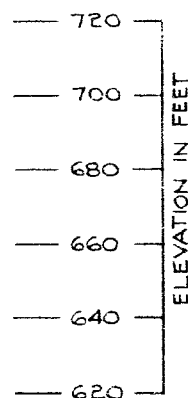
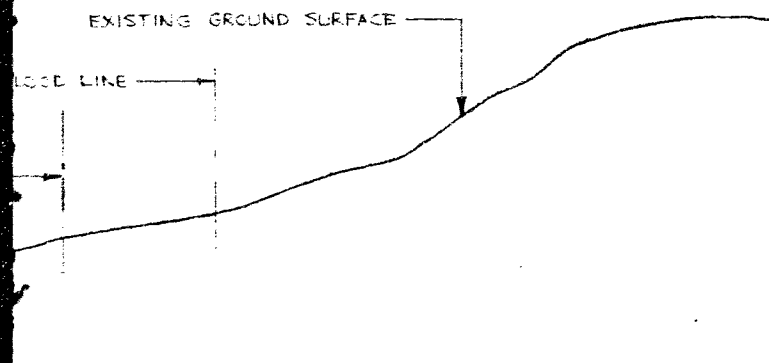
GRAND RIVER

EXISTIN

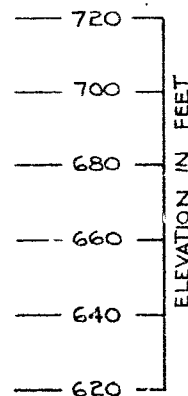
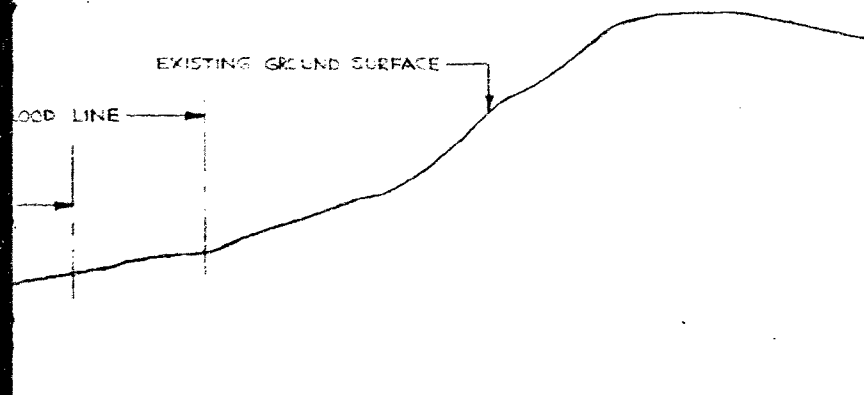
SECTION



SECTION A-A



SECTION B-B



SECTION C-C

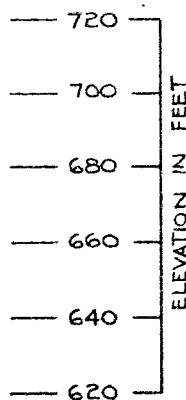
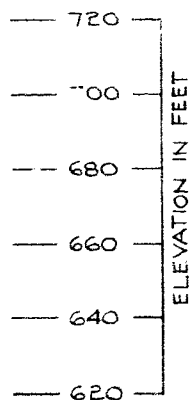
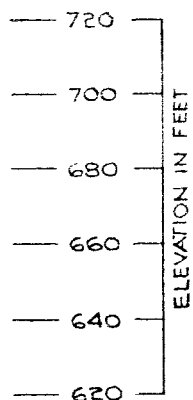
REFERENCE
UNNUMBERED
BY MCCOR

NOTE: FOR LOCAL
REFER
FOR GENERAL

Date SEPT. 25, 1973

TYPICAL SECTIONS OF
SOUTH RIVERBANK

FIGURE 8



SCALE: 1" TO 40'

REFERENCE

UNNUMBERED AND UNDATED PLAN SUPPLIED
BY MCCORMICK RANKIN & ASSOCIATES LTD.

NOTE: FOR LOCATION OF SECTIONS A-A, B-B & C-C
REFER TO FIG. 1

FOR GENERAL VIEWS SEE FIG. 2

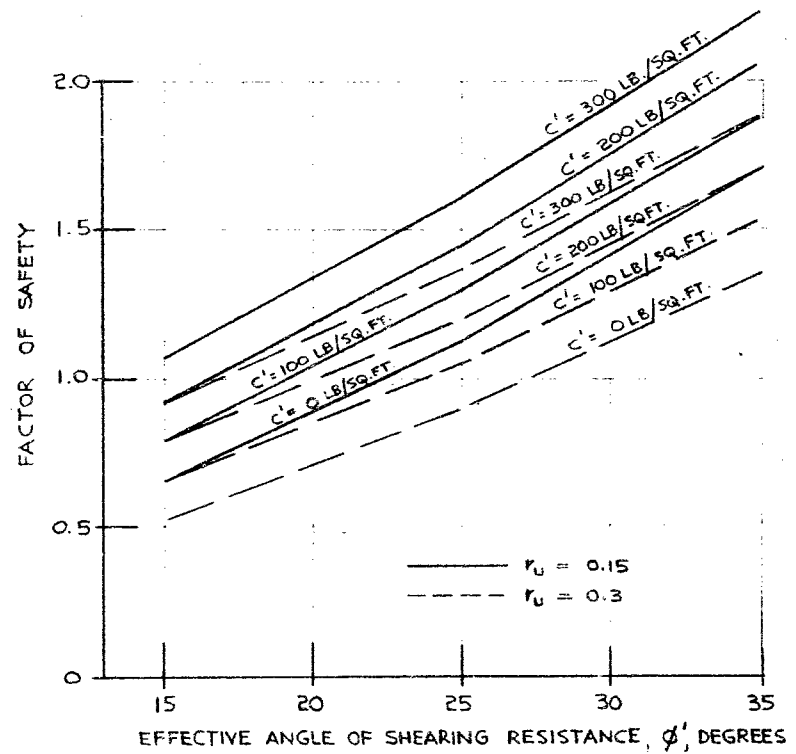
40P1-69
C. J. P.

Date SEPT. 25, 1973

Golder Associates

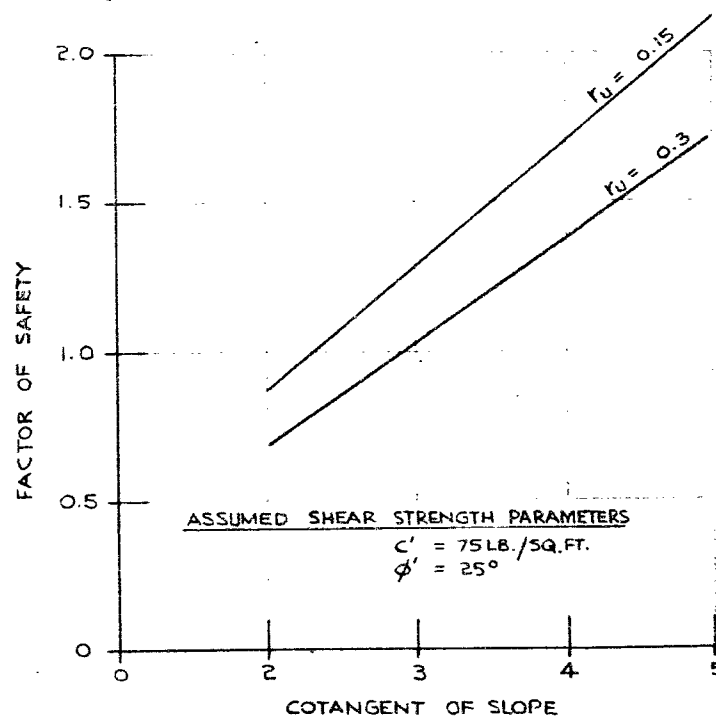
Drawn J.A.
Chkd. W.L.
Appd. J.A.

(a)



RESULTS OF PRELIMINARY STABILITY ANALYSIS OF SECTION
ALONG LINE 2 (SEE FIG. 7)

(b)

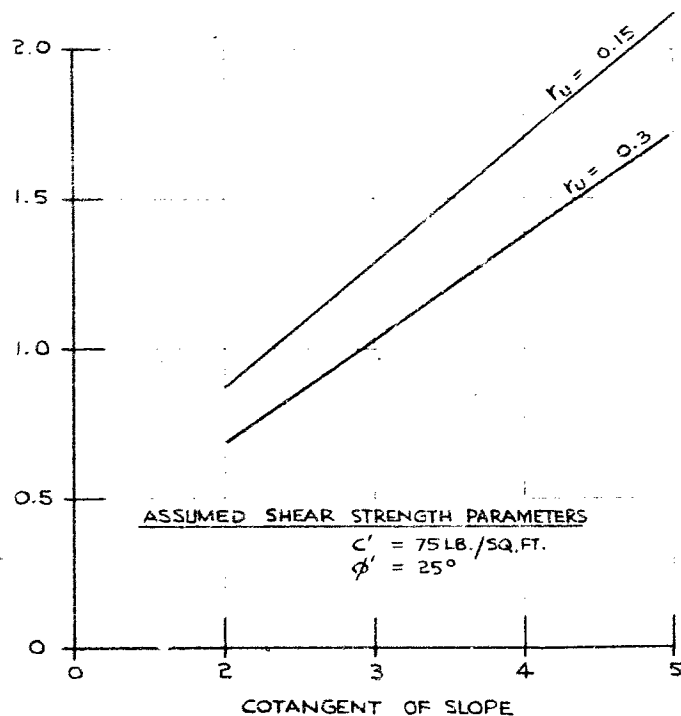


VARIATION OF FACTOR OF SAFETY WITH SLOPE ANGLE AND PORE PRESSURE RATIO ASSUMING PROBABLE SHEAR STRENGTH PARAMETERS.

Date OCT. 15, 1973

Gol

(b)



VARIATION OF FACTOR OF SAFETY WITH SLOPE ANGLE
 AND PORE PRESSURE RATIO ASSUMING PROBABLE SHEAR
 STRENGTH PARAMETERS.

Date OCT. 15, 1973

Golder Associates

40P1-69
 OCT 17 1973

Drawn J.A.
 Chkd. W.F.
 Appd. J.A.



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

INTERIM
REPORT
TO

40 P1-69
GEOCRES No.

2 of 3

McCORMICK, RANKIN & ASSOCIATES LIMITED
SUBSURFACE INVESTIGATION
PROPOSED COCKSHUTT ROAD BRIDGE
GRAND RIVER CROSSING
BRANTFORD ONTARIO

Distribution:

- 8 Copies - McCormick, Rankin & Associates Limited
Mississauga, Ontario
- 2 Copies - H.Q. Golder & Associates Ltd.
Mississauga, Ontario

March, 1975

STRUCTURE SITE No. 40-39

D.T.C. — TORONTO
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STRUCTURAL
OFFICE

741258



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

March 21, 1975

McCormick, Rankin & Associates Ltd.
Consulting Engineers
8 Stavebank Road
MISSISSAUGA, Ontario
L5G 2T4

ATTENTION: Mr. J.L. Malcolm, P. Eng.

RE: PROPOSED COCKSHUTT ROAD BRIDGE
BRANTFORD, ONTARIO

Dear Sirs:

This interim report presents the results of a sub-surface investigation carried out at the above site between December 16, 1974 and January 13, 1975.

Subsequent to preparation of the report in draft form (February, 1975) several factors which were not previously known to us concerning the south valley bank were established.

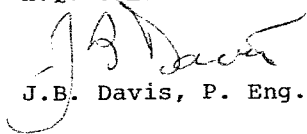
- i) The proposed bridge alignment crosses the area of a major slide which occurred in 1957. This slide was located immediately at the south end of the existing bridge and appears to have occurred in a generally north-easterly direction.
- ii) The south abutment of the existing bridge is reportedly actively moving.
- iii) Consideration is being given to using the material excavated during widening of the existing cut through the crest of the south valley bank as fill for the construction of the proposed north roadway approach embankment.

Further, the results of borings put down during the course of this investigation indicate that the subsurface stratigraphy and, more important, the piezometric conditions within the south valley bank are considerably more complex than was originally anticipated.

Because of the above, it has been agreed to put down additional borings and install additional piezometers in the south valley bank area. Consequently, it was mutually decided to issue the "draft" report as an "interim" report. Recommendations given in this "Interim" report concerning the stability of the south valley bank should be considered tentative and are subject to confirmation by the additional borings and analyses prior to final design.

Yours truly,

H.Q. GOLDER & ASSOCIATES LTD.



J.B. Davis, P. Eng.

JBD:def
741258

Encl.

INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by McCormick, Rankin & Associates Limited, Consulting Engineers to the City of Brantford, and the Brantford Suburban Roads Commission, to carry out a final subsurface investigation at the site of a proposed replacement structure to carry the existing Cockshutt Road over the Grand River, in Brantford, Ontario. The purpose of the investigation was to determine the subsurface conditions at each of the proposed pier and abutment locations and, based on these conditions, to provide engineering recommendations for the geotechnical design and construction of the proposed bridge crossing including an assessment of the overall stability of the present south valley bank.

The results of a preliminary subsurface investigation carried out at the site in September, 1973 are given in our report number 73154 to McCormick, Rankin & Associates Limited dated October, 1973. The condition of the existing bridge structure and the south valley bank as described in this preliminary report are significant.

"It is understood that the existing County Road No. 4 (Cockshutt Road) bridge over the Grand River Valley is an approximately 620 foot long structure consisting of four approximately 135 foot long central spans and two approximately 40 foot long end spans. It is further understood that the most southerly pier (which is located at about the mid-height of the south valley bank) has moved laterally about two feet, and that the entire bridge appears to be in compression due to horizontal displacement of the south abutment.

Visual examination of the existing south valley bank indicates the presence of numerous old failure zones and large portions of the entire slope appear to be marginally stable as there is evidence of slope creep in addition to the old failure zones. Based on discussions with local residents, it is understood that a relatively large scale failure of the river bank slope about 100 to 200 feet upstream from the existing bridge structure took place about 12 years ago."

The results of preliminary stability analyses indicated that the existing south valley bank is only marginally stable and it was recommended that this slope be flattened during construction of the replacement structure. However, it was suggested that, prior to final design, a detailed geotechnical investigation be carried out to determine the effective stress shear strength parameters of the material forming the south valley bank and pertinent geotechnical properties of the foundation soils. The preliminary investigation report should be read in conjunction with this present report.

SITE AND GEOLOGY

It is understood that the proposed replacement structure, a five span continuous concrete bridge with an approximately 2,200 foot long north approach embankment, is to be constructed along an alignment corresponding approximately to "Line 1" shown on Figure 1 of the preliminary investigation report (our report number 73154 dated October, 1973). At the site, the present Grand River valley is some 3,000 feet wide and is bounded on the south by relatively steep (about 3 horizontal to 1 vertical) approximately 75 to 80 foot high slopes which show evidence of past failures (see preliminary report). The present approximately 200 foot wide river channel (normal river level at between elevations 525 and 530) is located along the toe of the south valley bank. The present channel is bounded on the north by a broad relatively flat lying flood plain (ground surface at between about elevations 540 and 545) across which an approximately 25 foot high roadway approach embankment is proposed.

Based on available geological information, the site is located near the western limit of the physiographic region known as the Haldimand Clay Plain, an area characterized by thick deposits of glacio-lacustrine clays and silts laid down in glacial melt-water lakes which occupied the present Lake Erie basin during the retreat of the Wisconsin glaciers. The initial retreat of the glaciers from the Brantford area probably occurred during the latter stages of Cary glacial substage. During this period the site was inundated by glacial Lake Arkona within which glacio-lacustrine deposition could be anticipated. Subsequently, the glacial ice sheet re-advanced to about the Brantford area. Although the exact position of the ice front is not known, some re-working of the Lake Arkona deposits and deposition of glacial till in the Brantford area probably occurred during this advance. During the final retreat of the glacier from the area, the site was inundated by at least one and possibly several glacial lakes (notably glacial Lakes Wayne and Warren).

As glacial retreat continued, the water level in the Lake Erie basin lowered and melt-water streams from the retreating glacier eroded the present Grand River valley through the lacustrine deposits. Subsequent fluvial deposition during the latter stages of the outwash stream and/or recent alluvial deposition by the post-glacial Grand River may be anticipated across the floor of the valley.

As previously noted, the site is located very close to the limit of the glacial lake pondings. Consequently, irregularly stratified and relatively coarse grained (silty) deposits associated with shallow water deposition should be anticipated, particularly at the higher elevations, rather than uniformly layered (varved) highly plastic clays associated with deep water deposition.

PROCEDURE

The field work for this investigation was carried out between December 16, 1974 and January 13, 1975. During this period a total of seven (7) borings (number 101 to 107, inclusive) were put down at the locations shown on Figure 2. The on-shore borings were advancing to depths of between about 40 and 100 feet below existing ground surface using a track-mounted power auger supplied and operated by a specialist drilling contractor from Toronto. Borehole 103, put down within the existing river channel, was advanced from a barrel raft using a light diamond machine drillrig.

In each boring, standard penetration tests were carried out at a maximum of 5 foot intervals of depth and samples of the overburden were obtained using conventional split spoon sampling equipment. Within cohesive soil zones relatively undisturbed samples of the subsoil were obtained using 3 inch diameter thin-walled Shelby tubes and in situ vane tests were carried out between samples to determine the undrained shear strength of the soil.

Groundwater level observations were made in the open borings during the drilling operations and, following the completion of each of the selected borings, a standpipe and/or piezometer was sealed into the open hole to permit monitoring of the stabilized groundwater level(s) across the site.

Details of the drilling and sampling operations, the results of in situ testing and piezometer/standpipe installation are given on the Record of Borehole sheets following the text of this report.

The field work was supervised through by a member of our engineering staff who located the borings in the field, directed the drilling and sampling operations, carried out the in situ testing, and cared for the samples obtained. All of the samples obtained during this investigation were brought to our laboratory in Mississauga for detailed examination and testing.

The borehole locations and the ground surface elevation at each borehole were supplied to us by McCormick, Rankin & Associates Limited. It is understood that the elevations given in this report are referred to Geodetic datum.

SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the borings put down during the course of this investigation are given on the Record of Borehole sheets following the text of this report and a section showing the inferred subsurface stratigraphy along the centerline of the proposed bridge is given on Figure 2. The results of laboratory testing carried out on representative samples of the overburden are shown on the Record of Borehole sheets and on Figures 3 to 12, inclusive. Following is a summarized account of the subsurface conditions encountered within the south valley bank and across the valley floor area.

Subsoil Conditions

a) South Valley Bank

Boreholes 101 and 102, put down through the south valley bank encountered some 27 and 13 feet of fill, respectively. The fill consists of a complex of sandy silt, clayey silt, and silty clay with some gravel and occasional zones of

organic matter and construction debris (asphalt, wood fragments and the like). It appears that the fill was derived from adjacent excavations carried out during construction of the abutments and piers of the existing bridge structure. Based on standard penetrations tests which gave 'N' values ranging between 6 and 24 blows per foot the consistency of the clayey portions of the fill is firm to very stiff and the silty portions are generally compact. In situ vane tests carried out in the cohesive fill in borehole 101 gave undrained shear strengths varying between 880 and in excess of 2,000 pounds per square foot. Natural moisture contents of the fill varied between 7 and 27 per cent and were generally in excess of 18 per cent.

It should be noted that the fill encountered in boreholes 101 and 102 was not encountered in borehole 1 (see Figure 2 for location) put down during the course of the preliminary investigation. Based on site topography, it appears that borehole 1 was put down at or only slightly behind the crest of the natural slopes whereas boreholes 101 and 102 were put down through fill which was placed on the face of the original slope during construction of the existing structure.

At the borehole 101 and 102 locations, the fill is underlain at about elevations 647 and 630, respectively by an extensive stratum of glacio-lacustrine silts and clays. This deposit consists of a complex or irregularly layered red-brown to grey silt, clayey silt and silty clay. The individual layer thickness varies from about 1/8 inch to in excess of about 2 feet and, particularly in the upper portion of the deposit, the layers are badly contorted, possibly due to disturbance by wave action and the like during shallow water deposition.

Within the upper portion of the deposit, silt to clayey silt layers tend to predominate with only a relatively few thin clay layers. With depth, however, the frequency and thickness of the clay layers increases resulting in a predominantly layered clayey silt and silty clay material. The lower portion of the deposit contains a trace to some gravel possibly representing deposition from floating ice blocks carved from the retreating glacier when the ice front was still relatively close to the site.

In both borings put down through the south valley bank, the upper 3 to 4 feet of the deposit appears to have been oxidized and leached, possibly during direct exposure on the slope face prior to placement of the fill, resulting in a predominantly sandy silt material. Further, in the lower 5 to 10 feet of the boring the material appears to have been disturbed or re-worked possibly during a temporary re-advance of the ice sheet during the general retreat of the glaciers from the area. This re-worked material has a generally till-like appearance with only a faint indication of residual layering.

As indicated on Figure 7, the plasticity of the material is typical for a glacial lake clay and, excluding cohesionless silt layers, the material in individual layers ranges from a clayey silt of low plasticity to a silty clay of high plasticity. The in situ water content of the layers generally ranges between about 20 and 40 per cent and the liquidity index of the materials is generally less than about 0.5. However, within occasional layers, a liquidity index of as high as about 1 was determined.

Based on the results of ϕ situ vane tests carried out during the present and previous investigations (our report number 73154, dated October, 1973) and two unconsolidated undrained direct shear tests, the undrained shear strength of the deposit generally ranges between about 1,000 and 2,000 pounds per square foot but increases to in excess of about 3,000 pounds per square foot below about elevation 610 at the abutment location. The variation in vane shear strengths probably reflects the presence of cohesionless silt layers within the deposit as do the occasionally higher strengths reported in boreholes 1 and 2 (our report number 73154, dated October, 1973).

To determine the effective shear strength parameters of the lacustrine complex, three series of consolidated drained direct shear tests were carried out. The results of these individual test series are summarized on Figures 3, 4 and 5. As shown on these figures, the individual test results are extremely variable and indicate that the deposit varies from an effectively cohesionless material ($c' = 0$) with an effective or drained angle of shearing resistance, ϕ' , varying from about 28 degrees to 43 degrees to a material having an apparent cohesion, c' , of about 200 pounds per square foot and an effective angle of shearing resistance, ϕ' , of about 22 degrees.

Considering the highly contorted and irregularly layered structure of this deposit, it is questionable whether the results of an individual laboratory test carried out on an approximately 2 inch square specimen are truly representative of the average shear strength parameters of the deposit (post-failure examination of some of the test specimens indicated the presence of 2 and sometimes 3 different soil materials across the failure plane). However, if all of the individual

test results are considered together (see Figure 6), it appears that, on the average, the deposit will behave as an essentially cohesionless material ($c' = 0$) and, although the effective angle of the shearing resistance of individual zones or layers could range from about 25 to 43 degrees, the average minimum effective angle of shearing resistance, ϕ' , of the layered silt and clay complex is about 29 degrees.

The results of residual strength tests carried out on sample 17, borehole 101, (see Figure 4) indicate that the residual shear strength of the material is about 90 per cent of the available peak strength.

b) Valley Floor Area

Excluding the existing roadway embankment fill and local topsoil deposits, boreholes 104 to 107 inclusive put down across the valley flood plain encountered a surficial silty sand deposit which is probably of recent alluvial origin. Gradation curves of typical samples of this deposit are given on Figure 8. Based on the results of standard penetration tests which gave 'N' values ranging between 4 and 12 blows per foot, the silty sand is in a generally loose state of packing.

Underlying the recent alluvium and the existing river channel (borehole 103), the borings encountered some 5 to 15 feet of sand and gravel. As indicated on Figure 9 (gradation curves) the sand and gravel deposit is relatively clean (silt content of less than about 10 per cent). Based on these gradation curves and field observations, the sand and gravel is relatively pervious with an estimated coefficient of permeability in the order of 10^{-1} to 10^{-2} centimetres per second. The results of standard penetration tests which gave 'N' values ranging between about 12 and 35 blows per foot indicate the sand and gravel deposit is in a compact to dense state of packing.

At the borehole locations, the sand and gravel deposit is underlain at between elevations 613 and 628 by the irregularly layered silt, clayey silt and silty clay stratum previously discussed for the south valley bank area (boreholes 101 and 102). Based on the results of field vane tests and undrained triaxial compression tests carried out on relatively undisturbed samples of the layered clays and silts, the undrained shear strength of the deposit varies between about 1,500 and greater than 3,000 pounds per square foot.

Below about elevation 620 (borehole 107) to elevation 600 (borehole 103), the borings put down across the valley floor encountered the zone of apparently re-worked lacustrine material previously reported in the lower portions of boreholes 101 and 102 put down through the south valley bank. This zone, which was probably re-worked by glacial action during a temporary re-advance of the ice sheet to the Brantford area, consists of clayey silt to silty clay of low to medium plasticity and has a generally till-like texture with only faint indications of residual layering. Based on the results of standard penetration tests which gave 'N' values ranging between 13 and 78 blows per foot (generally about 15 to 30 blows per foot) the consistency of the re-worked zone is variable but the deposit can generally be classified as very stiff to hard.

In boreholes 104, 105 and 106, the re-worked (till-like) material is underlain at a depth of about 30 to 40 feet below ground surface by a deposit of irregularly layered silts, clayey silts and silty clays which is generally similar to the overlying lacustrine deposit. However, this lower deposit is apparently of earlier geological origin (see "Site and Geology") and has been heavily pre-consolidated by glacial over-ride. As a result of this over-consolidation the consistency of the lower lacustrine deposit is hard ('N' values generally in excess of 50 blows per foot).

Bedrock Conditions

The site is underlain at between about elevations 595 and 585 by relatively flat lying dolomitic bedrock of the Salina formation. Although essentially a dolomite, the bedrock contains numerous interbedded shale layers and occasional gypsum layers or inclusions. Further, the rock contains occasional small vugs or solution cavities.

In boreholes 101 and 102 put down through the south valley bank, the rock is capped by some 3 feet of weathered shale but below this cap the rock becomes fairly sound (R.Q.D. of about 60 per cent). Across the valley floor, however, the shale cap was not encountered and the rock was found to be moderately fractured throughout the cored depth.

Groundwater Conditions

Based on the results of groundwater level observations in the open borings during drilling operations and water level readings taken in the piezometers and standpipes following completion of the field work (see Record of Borehole sheets for details) the groundwater conditions across the site are relatively complex.

Across the valley floor area, there appears to be two separate groundwater conditions with a transition occurring through the lacustrine deposits. Because of the pervious nature of the fluvial sand and gravel, the groundwater level in the surficial granular deposits (alluvial silty sand and fluvial sand and gravel) is at or only about 2 to 3 feet above the river water level. Horizontal seepage towards the river is probably occurring through the sand and gravel.

Water level observations during rotary core drilling in the bedrock in borehole 103 and water level readings taken in piezometers sealed within the bedrock in boreholes 104 and 106 indicate that the piezometric water level in the rock across the valley floor area is at about elevation 634 and is artesian with respect to the river water level at the time of the investigation. This artesian groundwater condition is probably due to lateral seepage of groundwater through the rock from the higher ground outside the river valley.

Within the south valley bank area, there is no positive measurement of the piezometric water level in the bedrock (due to caving of the lower portion of borehole 101, a piezometer proposed in the bedrock in this boring was unavoidably destroyed during installation). However based on an extrapolation of the piezometric data from borehole 102 and the probability that the water level in the piezometer installed in borehole 101 reflects the piezometric water level in the bedrock, it appears that the piezometric groundwater level in the rock is at about present ground surface. Further, it appears that upward seepage of groundwater from the bedrock is occurring through the lacustrine deposits into the more pervious upper sandy zone which caps this deposit. An extrapolation of the piezometric data in borehole 102 and observations during drilling in boreholes 101 and 102 indicate that there was no significant head of water in this sandy zone during drilling operations.

Although the sandy zone was not encountered in borehole 1 put down during the previous investigation, water level readings taken in this boring during the present investigation (water level in the piezometer at elevation 657.2 and in the standpipe at elevation 654.5) suggest that in the upper portion of the overburden (and probably the fill) downward seepage of groundwater, probable to the sandy layer, is occurring.

STABILITY OF SOUTH VALLEY BANKExisting Slope

It is understood that some progressive movements are occurring within the existing slope (this should be confirmed and details of the movements should be reviewed prior to finalization of this report) and consequently it would appear that, at least under seasonably high groundwater conditions, the slope is only marginally stable (factor of safety of about unity). Based on the results of the preliminary investigation together with previous slope studies in the area of the site, it appears that the stability of the natural slopes is controlled by effective stress conditions (which consider the influence of groundwater) rather than undrained or total stress conditions.

To obtain an appreciation of the in situ (as opposed to laboratory) effective or drained shear strength parameters of the overburden, the stability of the existing slope was analysed assuming a factor of safety of unity. At the time of this investigation (January, 1975) it appears that upward seepage from the bedrock (piezometric water level at about present ground surface) and downward seepage of surface water was occurring into the upper more pervious portion of the lacustrine complex which appeared to be relatively dry or under very low piezometric head. However, during periods of high river water level and heavy seasonal precipitation, it is probable that the piezometric level in this sandy zone would rise. The maximum seasonal water level in this zone is difficult to determine but probably would not exceed the present ground surface. Consequently, the worst piezometric conditions in the slope would be hydrostatic condition with the water level at the face of the slope.

Preliminary analyses indicated that the critical failure surface could be relatively shallow and would be limited to the lower portion of the existing slope. The results of detailed stability analyses using non-circular, shallow failure surfaces and assuming various piezometric conditions along the failure surface are summarized on Figure 13.

Based on these analyses and assuming that:

- a) the existing slope is in imminent danger of sliding (factor of safety = 1.0)
- b) the piezometric water level is at or only slightly below the ground surface
- c) the soil behaves as a purely frictional material ($c'=0$)

the minimum average effective angle of shearing resistance, of the lacustrine silt and clay complex is about 30 to 35 degrees.

Proposed Slope

The results of the consolidated drained direct shear tests together with the results of a "back" analysis of the existing slope indicate that a realistic minimum average effective (drained) internal angle of shearing resistance, ϕ' , of the essentially cohesionless (under drained conditions) lacustrine deposit is about 30 degrees. Consequently, this value was used to assess the long term stability of the re-graded 4 horizontal to 1 vertical slope proposed in the preliminary subsurface investigation report (our report 73154 dated October, 1973).

As previously discussed, it is difficult to predict the maximum piezometric conditions which will occur in the slope. However, it is anticipated that the maximum average piezometric water level will be less than normal hydrostatic pressures resulting from a groundwater level coincident with the slope face.

The results of preliminary stability analyses carried out using "circular arc" failure surfaces and assuming a piezometric groundwater level at ground surface indicated that the minimum factor of safety against rotational instability was about 1.5 and that this minimum value occurred for relatively shallow failure surfaces passing through to the toe of the slope area. Consequently, the proposed slope was re-analysed assuming relatively shallow non-circular failure surfaces and various piezometric groundwater levels. The results of these analyses are summarized on Figure 14 and indicate that for a piezometric groundwater level at ground surface, the minimum factor of safety against sliding is about 1.4. For a piezometric water level 8 feet below ground surface, the minimum factor of safety increases to greater than 2.

Finally, a non-circular failure surface through the upper portion of the lacustrine deposit was assumed (see Figure 15). The results of this analysis indicates that for a piezometric water level at ground surface, the factor of safety against sliding is about 1.1 but increases rapidly as the piezometric water level is lowered to below the slope face.

Based on the results of the stability analyses, it is our opinion that no major instability of the 4 horizontal to 1 vertical slopes proposed in the preliminary foundation investigation report will occur but that some local straining or shallow sloughing of the slopes could occur if the piezometric water level in the upper portion of the lacustrine silts and clays is at or close to the slope face. Consequently, we recommend that drainage of the upper sandy portion of the

lacustrine deposit be provided. To this end, we suggest that trench drains having a minimum width of 2 feet and extending into the upper portion of the lacustrine deposit be installed at about 25 foot centres along the lower portion of the slope.

Finally, it is recommended that the proposed slope re-grading and drainage extend for a minimum distance of 200 feet upstream and downstream of the proposed bridge.

Erosion Protection

The above discussion regarding slope stability pre-supposes that erosion scour and resultant over-steepening of the toe of the slope is prevented. The requisite slope protection can probably best be achieved by means of gabions installed on the face of the slope. To this end, we recommend that:

a) Blanket gabions be installed into the face of the regraded slopes (regraded to not steeper than 4 horizontal to 1 vertical) and placed on a 1 foot thick bedding pad of properly graded filter material well compacted in place.

b) The gabion blankets extend at least 3 feet above the maximum design flood level of the river.

c) The vertical gabion baskets at the toe of the slope be founded at least 3 feet below the river bed and blanket gabions be installed along the toe on the river bed to prevent bottom scour. These blanket gabions should extend some 5 feet into the river from the vertical gabions.

d) The gabion works be extended a minimum of 200 feet upstream and 100 feet downstream of the bridge.

e) The ends of the gabion sections be anchored into the existing slopes to prevent undercutting and progressive undermining of the gabion.

It should be noted that the vertical gabions should be installed in relatively short sections (less than 20 foot sections) to avoid local overstressing of the slope.

PROPOSED BRIDGE STRUCTURE

It is understood that the existing Cockshutt Road bridge is to be replaced with an approximately 750 foot long, continuous, five span, high level structure.

Foundations

Considering the relatively loose and variable nature of the alluvial and fluvial deposits encountered across the valley floor area, the presence of some 25 feet of variable fill at the south abutment location and the relatively soft (loose) and compressible nature of the lacustrine silt and clay complex, it is considered that the use of spread footings founded on the overburden is not feasible for the support of the proposed continuous bridge structure. Consequently, we recommend that the pier and abutment loads be transferred to the bedrock which underlies the site at between elevations 586 and 595 by means of end-bearing piles.

Although either concrete filled pipe piles or pre-cast concrete piles would be suitable for the foundation support of the proposed pier and abutments, high pore pressures induced

in the lacustrine clay and silt complex during driving of displacement type piles (pipe piles or pre-cast concrete piles) at the south pier and abutment locations, could endanger the overall stability of the south valley bank even following re-grading of the slope. Consequently, we recommend that the proposed piers and abutments be founded on small displacement steel 'H' piles driven to practical refusal on or in the bedrock.

Although no serious problems are anticipated in advancing conventional steel 'H' pile sections to practical refusal on or in the bedrock, some relatively hard driving must be anticipated in penetrating obstructions (cobbles and boulders) within the sand and gravel deposit which underlies the flood plain area and in the lower portion of the overburden and/or the upper weathered and fractured portion of the bedrock. To minimize possible damage to the piles during driving and to minimize contact stresses at the pile tip, we recommend that a relatively heavy pile section, such as a 12 B P 74 section, be employed. Relatively heavy steel 'H' piles (such as a 12 B P 74 section) driven to practical refusal on or in the bedrock (defined as a final set of at least 20 blows per inch over the last 6 inches of driving using a hammer delivering at least 22,500 foot pounds of energy per blow) may be designed using an allowable load of as much as 75 tons per pile. As some fracturing or shattering of the rock (which contains shale layers and gypsum inclusions) must be anticipated during driving, it is recommended that each pile in a group be retapped to the design set following driving of the group.

Construction of Pile Caps

As the groundwater level across the flood plain area is at or only slightly above the river water level, and as it is anticipated that the underside of the pile caps will extend only about 4 to 6 feet below present ground surface, no serious problems are anticipated in excavating for the pile caps at the two northerly pier locations (it is assumed that the north abutment will be perched in the roadway approach fill).

As the existing river channel is underlain by pervious sand and gravel (refer borehole 103) it is recommended that the excavation for the proposed pier within the river channel be carried out within an adequately braced, closed steel sheet pile cofferdam. The sheet piling should be driven to a penetration below the proposed excavation level equal to at least three-quarters of the head of water to be retained or at least 5 feet into the underlying lacustrine clays and silts, whichever is the greater depth. Further, the tops of the sheeting should extend to a sufficient height to prevent over-topping of the cofferdam during flash run-off periods.

As the south pier will be located on the face of the re-graded south valley bank, care must be exercised to avoid local over-steepening of the slope during excavation for the pile cap. To this end, it is suggested that the "up-slope" face of the excavation be cut vertical and supported by means of an adequately braced soldier pile and lagging system. The shoring system should be designed using an earth pressure coefficient of 0.9 to account for the sloping surcharge (this is to be confirmed following discussions with the Consulting Engineers).

No construction problems are anticipated during excavation for the proposed north abutment pile cap.

Falsework Foundations

It is understood that the bridge deck will be cast-in-place and the fresh concrete will be supported in temporary false-work until the concrete sets.

Across the valley floor area, the false-work footings will be founded on the loose to compact silty sand alluvium. Prior to placing the footings, it is recommended that all topsoil and other surficial organic or fill materials be removed along the full length of the structure and the exposed surface of the silty sand proof-rolled with at least 5 passes of a smooth-wheeled vibratory roller weighing at least 5 tons. Any soft or loosened zones exposed during this proof-rolling should be sub-excavated and replaced with compact granular fill. The strip footings for the falsework should be designed using an allowable bearing pressure of not more than 1,000 pounds per square foot and should be a minimum of 12 inches wide. Even using this relatively low bearing pressure, it is anticipated the settlement of the footings during placings and curing of the concrete could be in the order of 1/4 inch to 1/2 inch.

Within the river, the use of conventional falsework footings will not be possible and it is anticipated that the falsework will be supported on beams extending from the pier to temporary grillages or footings at the edge of the river. As these beams will span about 100 feet, relatively large footings will be required to support the falsework loadings. Although settlement is not directly proportional to footing width, these larger footings

can be expected to settle more than the normal falsework footings. Consequently, consideration should be given to limiting the allowable bearing pressure under the larger footings to less than about 1,000 pounds per square foot (the actual reduction will depend on the magnitude of the loads and size of footings involved) or to supporting the ends of the beams on temporary pile foundations.

On the face of the re-graded south valley bank (slope of 4 horizontal to 1 vertical) the falsework footings should be "keyed" into the slope such that the leading edge of the footing is at least 2 feet behind the slope face. As the slope face will probably be underlain by fill, the allowable bearing pressure for the footings should be restricted to not more than 500 pounds per square foot. Settlement of footings designed using this allowable bearing pressure should be negligible during placing and curing of the concrete.

PROPOSED NORTH APPROACH EMBANKMENT

Embankment Stability

The results of this investigation indicate that the proposed embankment area is underlain by loose to compact silty sand and sand and gravel followed by stiff to very stiff lacustrine silts and clays. Provided all topsoil and other surficial organic materials are removed over the full base width of the embankment, no foundation stability problems are anticipated for 25 foot (maximum) high roadway approach embankments constructed using side and end slopes not steeper than 2 horizontal to 1 vertical. Settlement of the foundation subsoil under the full embankment loading should be less than about 2 to 3 inches with the majority of this settlement occurring during the construction period.

Earth Borrow

It is understood that it is presently proposed to employ the material from a cut required through the upper portion of the south valley bank as earth borrow for the construction of the north roadway approach embankment. As no borings were put down in this borrow area (this being outside of our original terms of reference) it is not possible to comment conclusively on the suitability of the excavated material as "engineered" fill.

Based on the results of borehole 1 put down through the upper portion of the valley bank some 500 feet west of the presently proposed alignment during the preliminary investigation (our report 73154 dated October, 1973) it appears that above elevation 670 the material consists essentially of layered clays and silts having an in situ water content between 20 and 30 per cent. As this water content is in excess of the estimated optimum water content for compaction, considerable difficulty would be anticipated in adequately compacting the material. However, as the proposed borrow area is basically a widening of an existing cut, some drainage towards the cut face may have occurred and the in situ water content of the material in the proposed borrow area may be somewhat drier than is indicated by borehole 2.

Prior to final design, it is essential that borings be put down in the proposed borrow area to determine the suitability of the material for use as embankment fill.

H.Q. GOLDER & ASSOCIATES LTD.

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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_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

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	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_α	coefficient of consolidation
T_v	time factor = $c_\alpha t / d^2$ (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_i	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.

POWER AUGER
7" DIA. HOLLOW STEM

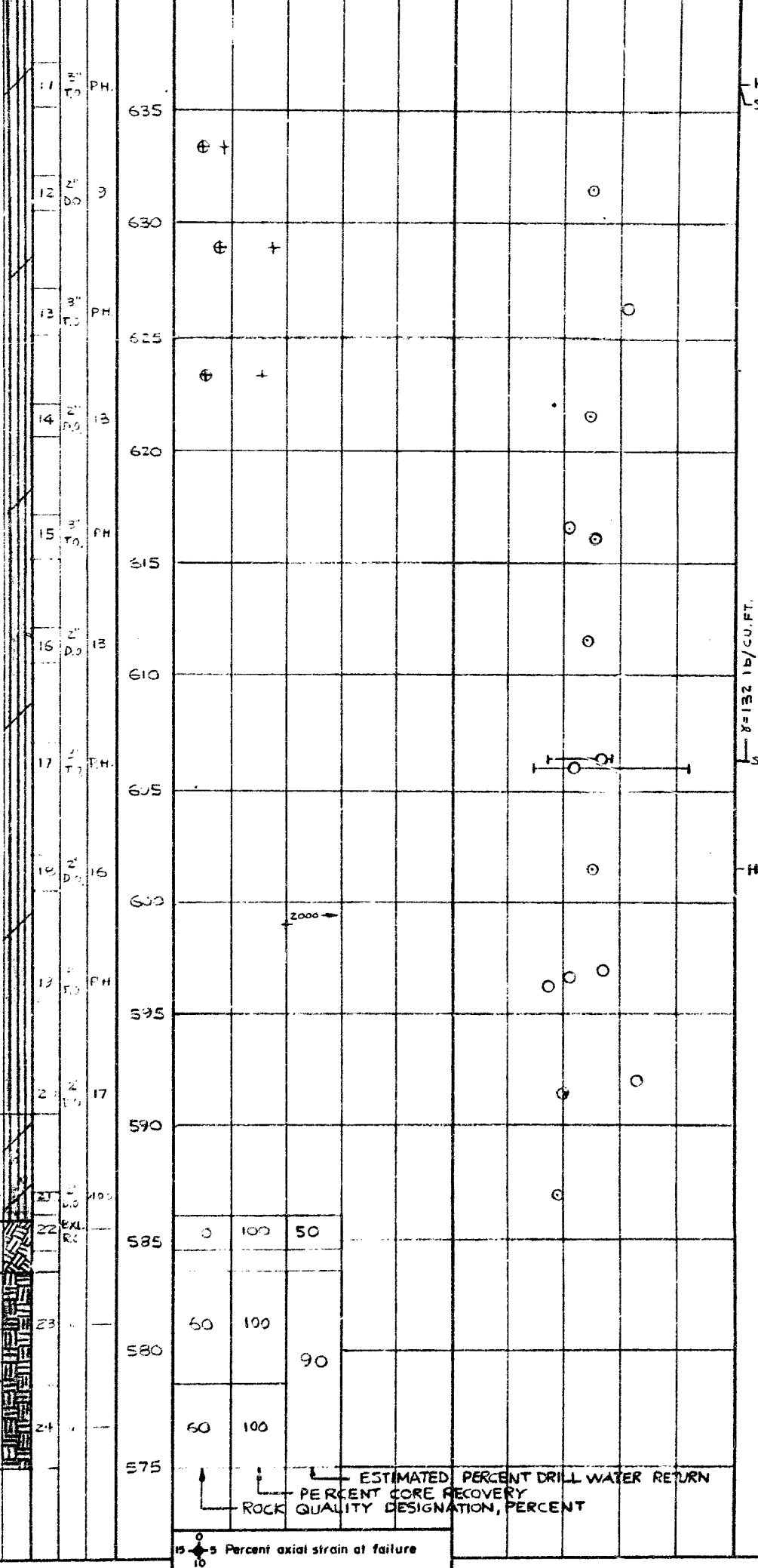
LOOSE TO COMPACT
IRREGULARLY
STRATIFIED SILT
WITH SOME CLAYEY
SILT TO SILTY CLAY
LAYERS BECOMING
WITH DEPTH STIFF
LAYERED CLAYEY
SILT SILTY CLAY
AND SILT

STIFF TO VERY STIFF
RED BROWN TO GREY
OCCASIONALLY FAINTLY
LAYERED CLAYEY SILT
TO SILTY CLAY, TRACE
OF SAND AND GRAVEL
(TILL-LIKE)

HIGHLY WEATHERED
SHALE BEDROCK

FAIRLY SOUND TO
SOUND WITH
FRACTURED ZONES
INTERBEDDED SHALE
AND DOLOMITE,
OCCASIONAL
GYPSUM INCLUSIONS
(BEDROCK)

END OF HOLE



PLASTIC TUBING

GRAVEL

PIEZOMETER

CAVED MATERIAL

WATER LEVEL IN
PIEZOMETER AT
ELEV. 669.0 ON
JAN. 28, 1975

DRAWN
CHECKED

Golder Associates

VERTICAL SCALE
1 IN. TO 5 FT.

RECORD OF BOREHOLE 102

LOCATION See Figure 2

BORING DATE JAN. 9, 10, AND 13, 1975

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_v , 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 C_u , LB./SQ. FT.				WATER CONTENT, PERCENT					
								1000 2000 3000 4000				10 20 30 40					
	643.0	GROUND SURFACE					645										
	0.0	COMPACT MOTTLED BROWN SANDY SILT TO CLAYEY SILT TRACE FINE GRAVEL, SOME ZONES OF ORGANIC MATERIAL (FILL)		1	2"	14											
				2	"	13											
				3	"	17											
				4	"	15											
				5	"	12											
	630.0	COMPACT BROWN AND GREY SILT TO SANDY SILT					630										
	13.0																
	627.0			6	"	23											
	16.0																
				7	"	13											
				8	3" TO	PH											
		STIFF TO VERY STIFF RED BROWN TO GREY LAYERED SILT TO CLAYEY SILT AND SILTY CLAY					615										
				9	2" DO	29											
				10	"	15											
				11	"	9											
				12	2" TO	PH											
	595.0	VERY STIFF TO HARD RED GREY CLAYEY SILT TRACE OF FINE GRAVEL (TILL-LIKE)					595										
	48.0				13	2" DO	18										
							590										

POWER AUGER
7" DIA. HOLLOW STEM

GROUND SURFACE

MH

GRAVEL FILL

PLASTIC TUBING

BENTONITE SEAL

PIEZOMETER No. 2
BENTONITE SEAL

GRAVEL FILL

PLASTIC TUBING

BENTONITE SEAL

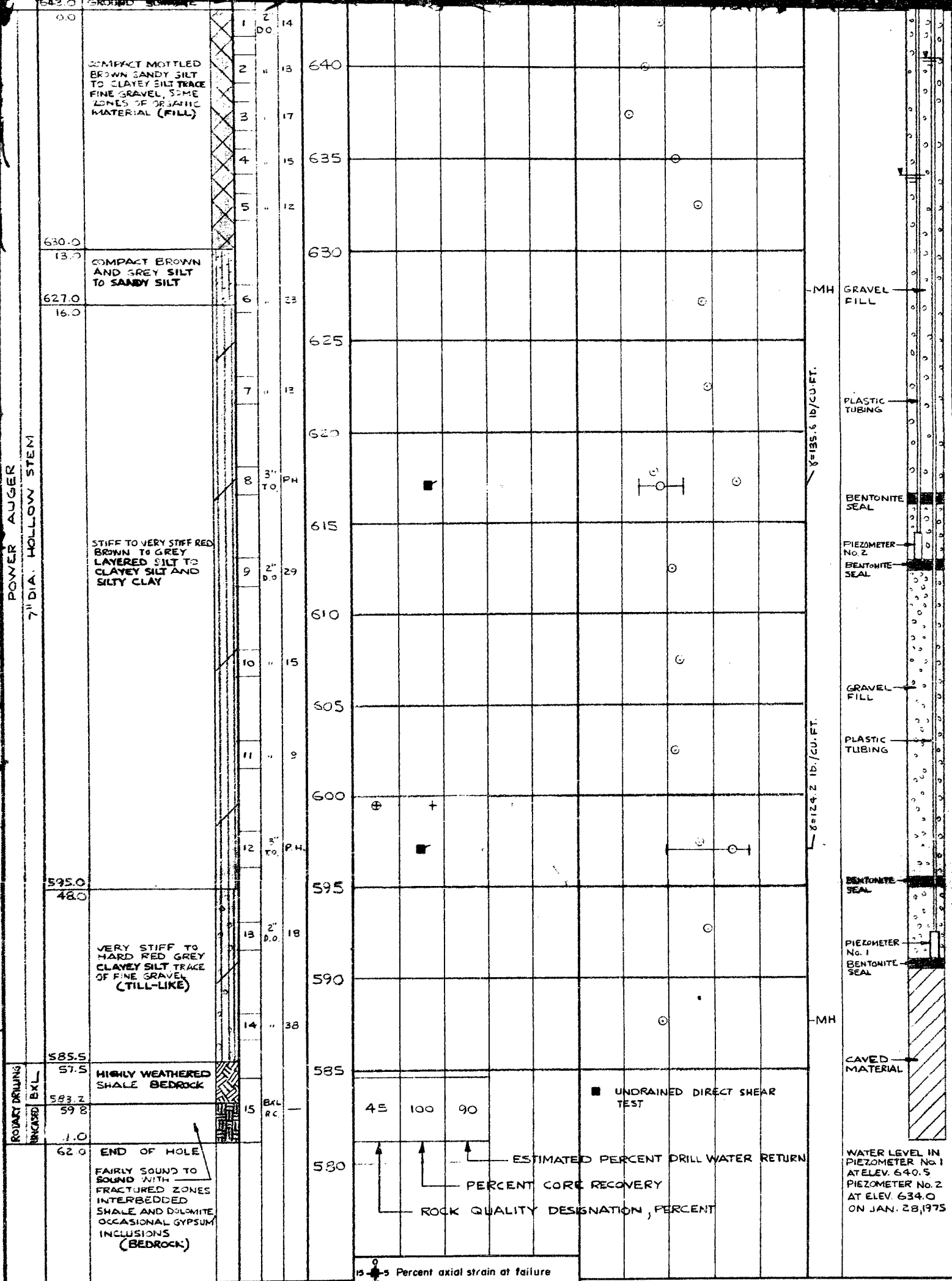
PIEZOMETER No. 1
BENTONITE SEAL

$\gamma = 135.6 \text{ LB./CU. FT.}$

$\gamma = 124.2 \text{ LB./CU. FT.}$

$\gamma = 135.6 \text{ LB./CU. FT.}$

$\gamma = 124.2 \text{ LB./CU. FT.}$



VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN D.M.
CHECKED

RECORD OF BOREHOLE 103

Preliminary

LOCATION See Figure 2

BORING DATE JAN. 2 AND 4, 1975

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_v , 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		
N X CASING	626.5	RIVER LEVEL															
		WATER															
	621.0																
	5.5																
		COMPACT BROWN SAND AND GRAVEL OCCASIONAL SMALL COBBLES		1	2'	35											
				2	"	14											
	613.0			3	"	14											
	13.5			4	"	34											
		STIFF TO VERY STIFF RED BROWN TO GREY IRREGULARLY LAYERED SILT TO CLAYEY SILT WITH OCCASIONAL SILTY CLAY LAYERS		5	"	16											
				6	"	15											
WASH BORING																	
UNCASED																	
ROTARY DRILLING																	
EXL CORE																	
BALANCE																	
END OF HOLE																	

NOTE: WATER LEVEL IN CASING AT ELEV. 632.8 DURING ROTARY CORE DRILLING IN BEDROCK

ESTIMATED PERCENT DRILL WATER RETURN
PERCENT CORE RECOVERY
ROCK QUALITY DESIGNATION, PERCENT

Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN P.M.
CHECKED

RECORD OF BOREHOLE 104

PRELIMINARY

LOCATION See Figure 2

BORING DATE DEC. 23, 24, AND 30, 1974

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_v , 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. - + Q. - ● REM.V. - ● U. - O					
								1000 2000 3000 4000					10 20 30 40				
POWER AUGER 7" DIA. HOLLOW STEM	632.7	GROUND SURFACE		1	2"	5	635									<p>GROUND SURFACE</p> <p>PLASTIC TUBING</p> <p>STANDPIPE</p> <p>LOCAL BACKFILL</p> <p>PIEZOMETER</p> <p>WATER LEVEL IN STANDPIPE AT ELEV. 631.0</p> <p>PIEZOMETER FROZEN TO ELEV. 634.0 ON JAN. 28, 1975</p>	
		LOOSE DARK BROWN SILTY SAND, NUMEROUS ROOTS IN UPPER 2 FT.		2	"	7	630										
	625.5			3	"	5											
	625.5	COMPACT TO DENSE DARK GREY TO BROWN SAND SOME GRAVEL, OCCASIONAL SMALL COBBLES, TRACE OF ORGANIC MATTER IN UPPER ZONE		4	"	13	625										
				5	"	38											
	618.2			6	"	15	620										
		STIFF TO VERY STIFF RED BROWN TO GREY IRREGULARLY LAYERED SILT TO CLAYEY SILT OR AS SILTY CLAY LAYERS		7	3" TO	P.H.	615										
	607.7			8	"	13	610										
		VERY STIFF RED BROWN TO GREY OCCASIONALLY LAYERED CLAYEY SILT TO SILTY CLAY, TRACE OF SAND AND GRAVEL (TILL-LIKE)		9	"	27	605										
	594.2			10	"	38	600										
	VERY STIFF TO HARD RED BROWN TO GREY LAYERED SILT AND CLAYEY SILT AND SILTY CLAY																
	594.2	END OF HOLE				595											
		REFUSAL TO AUGER (PROBABLY BED ROCK)				590											

0

15 5 Percent axial strain at failure

15 0 5 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN D.M.
CHECKED

RECORD OF BOREHOLE 105

PHILLIPS COUNTY

LOCATION See Figure 2

BORING DATE DEC. 22, AND 23, 1974

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_v , 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 C_u , LB./SQ. FT.		NAT. V. - + 0 - ● REM. V. - ● U - ○		WATER CONTENT, PERCENT W_p W W_L					
								1000 2000 3000 4000					10 20 30 40				
POWER AUGER 7" DIA. HOLLOW STEM	641.0	GROUND SURFACE														GROUND SURFACE	
	640.0 639.5 639.0 638.5	LOOSE DARK BROWN SILTY TOPSOIL		1	2"	5	640										
				2	"	8											
		LOOSE BROWN SILTY SAND TO SANDY SILT.		3	"	4	635									MH	
				4	"	5											
	629.0 628.5 628.0 627.5						620										
		DENSE BROWN SAND AND GRAVEL		5	"	32	625									MH	
	623.5 623.0 622.5 622.0																
				6	"	12	620										
		STIFF TO VERY STIFF DARK GREY IRREGULARLY LAYERED SILT AND CLAYEY SILT, OCCASIONAL THIN RED BROWN SILTY CLAY LAYERS		7	"	PH	615										
	610.0 609.5 609.0 608.5																
	VERY STIFF RED GREY CLAYEY SILT, TRACE OF FINE GRAVEL (TILL-LIKE)		8	"	33	610											
607.5 607.0 606.5 606.0																	
	HARD GREY TO RED BROWN SILT TO CLAYEY SILT, OCCASIONAL LAYERS OF SILTY CLAY		9	"	70	605									H		
	VERY DENSE SILTY SAND AND GRAVEL		10	"	85	600											
	595.5	END OF HOLE REFUSAL TO AUGERING PROBABLY BEDROCK		11	"	100	595									CLAY SEAL PIEZOMETER SAND AND GRAVEL STANDPIPE AND PIEZOMETER DESTROYED ON DEC. 27, 1974	

0
15
10

5 Percent axial strain at failure

15 0 5 Percent axial strain at failure
10

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN D.M.
CHECKED

RECORD OF BOREHOLE 106

PRELIMINARY

LOCATION See Figure 2

BORING DATE DEC. 16, 17, 18 AND 20, 1974

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_v , CM./ SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FT.										
								SHEAR STRENGTH Cu, LB./SQ. FT.		NAT. V. - + Q. - ● REM.V. - ● U. - O		WATER CONTENT, PERCENT					
							20	40	60	80	1x10	1x10	1x10	1x10			

GROUND SURFACE

-MH

BACKFILL MATERIAL

-M

PLASTIC TUBING

STANDPIPE

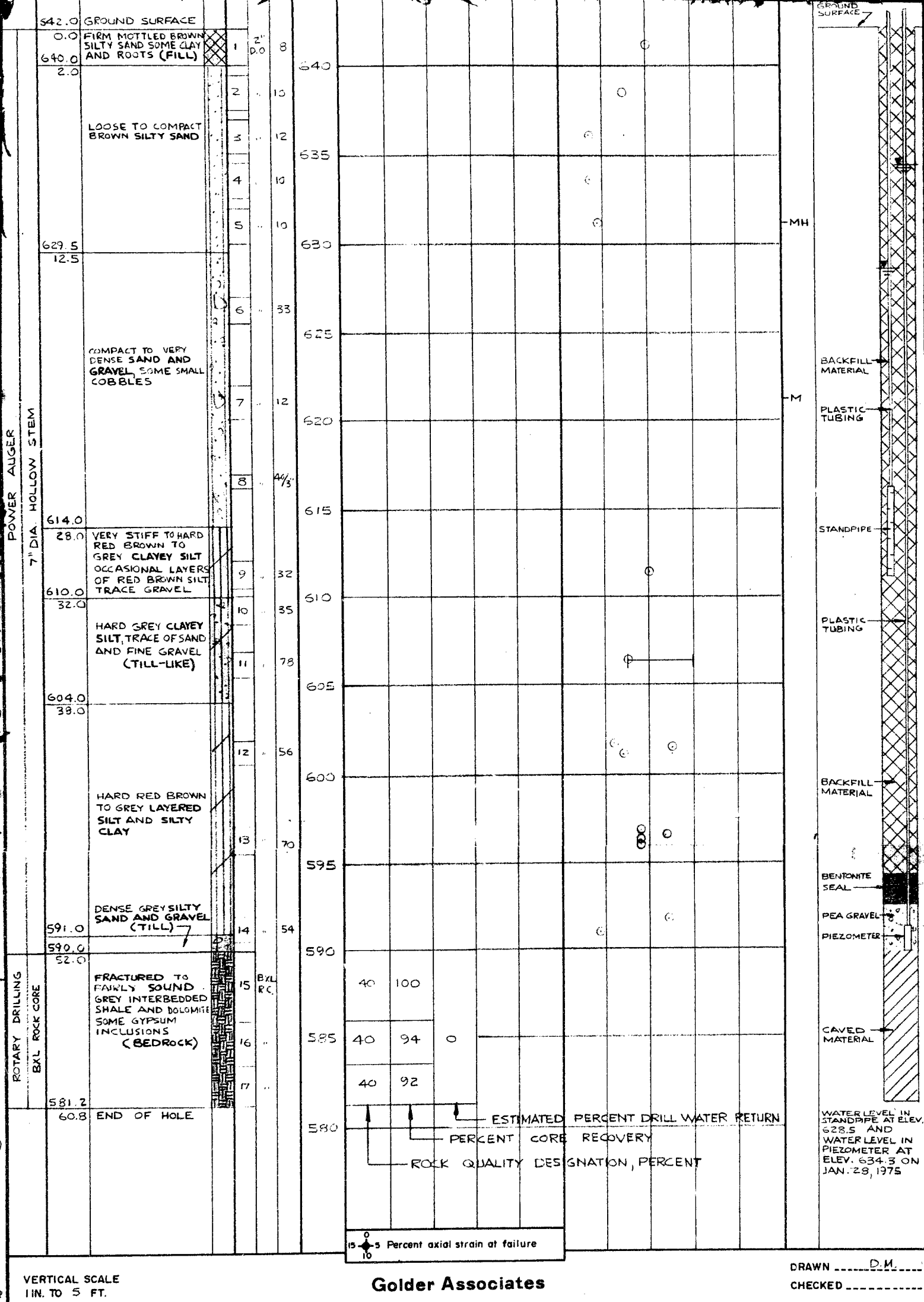
PLASTIC TUBING

BACKFILL MATERIAL

BENTONITE SEAL

PEA GRAVEL

PIEZOMETER



RECORD OF BOREHOLE 107 PRELIMINARY

LOCATION	See Figure	2
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BORING DATE DEC. 31, 1974

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_v , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH				WATER CONTENT, PERCENT					
								C _u , LB./SQ. FT.		NAT. V. - + REM.V. - ⊕ O - ○		w _p		w			w _L
POWER AUGER 7" DIA. HOLLOW STEM	642.0	GROUND SURFACE															
	637.5	LOOSE DARK BROWN SANDY SILT, OCCASIONAL POCKETS OF DARK BROWN SANDY TOPSOIL (FILL)		1	2"	8											
	632.3	COMPACT TO LOOSE BROWN SILTY SAND		2	"	13											
	628.0	COMPACT BROWN SAND AND GRAVEL		3	"	11											
	623.0	STIFF TO VERY STIFF THINLY LAYERED RED BROWN TO GREY SILTY CLAYEY SILT AND SILTY CLAY TRACE GRAVEL		4	"	7											
	619.0			5	"	25											
	615.0			6	"	5											
	610.0			7	3" TO 4"	PH											
	605.5			8	"	10											
	600.0			9	"	10											
	605.5	END OF HOLE															

GROUND SURFACE

PLASTIC TUBING

MH

STANDPIPE

MH

BACKFILL

PLASTIC TUBING

PEA GRAVEL

PIEZOMETER BENTONITE SEAL

CAVED MATERIAL

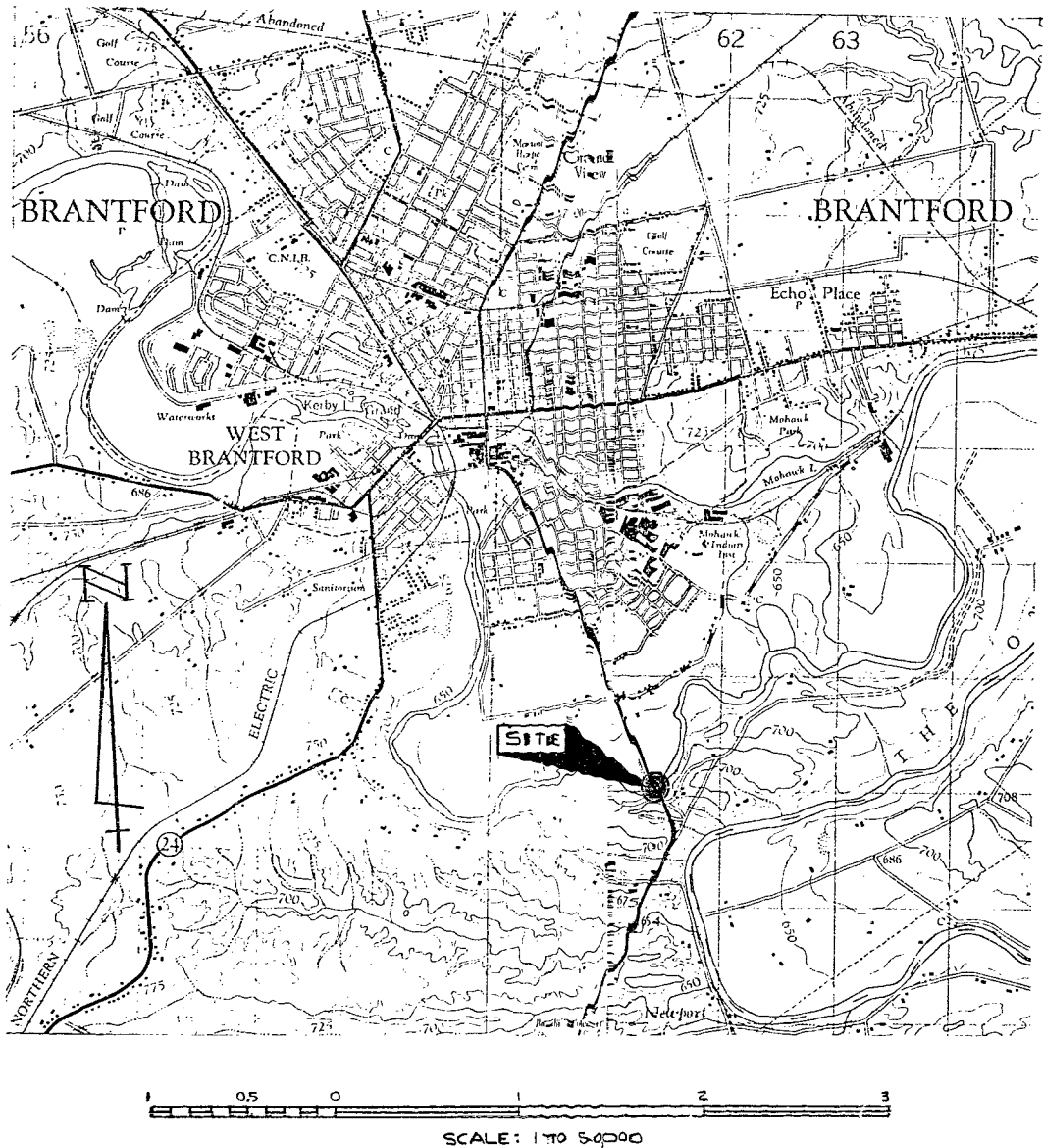
STANDPIPE AND PIEZOMETER DRY ON JAN. 25, 1975

15 0 5 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

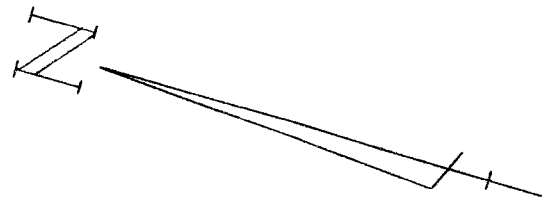
DRAWN _____ D.M.
CHECKED _____



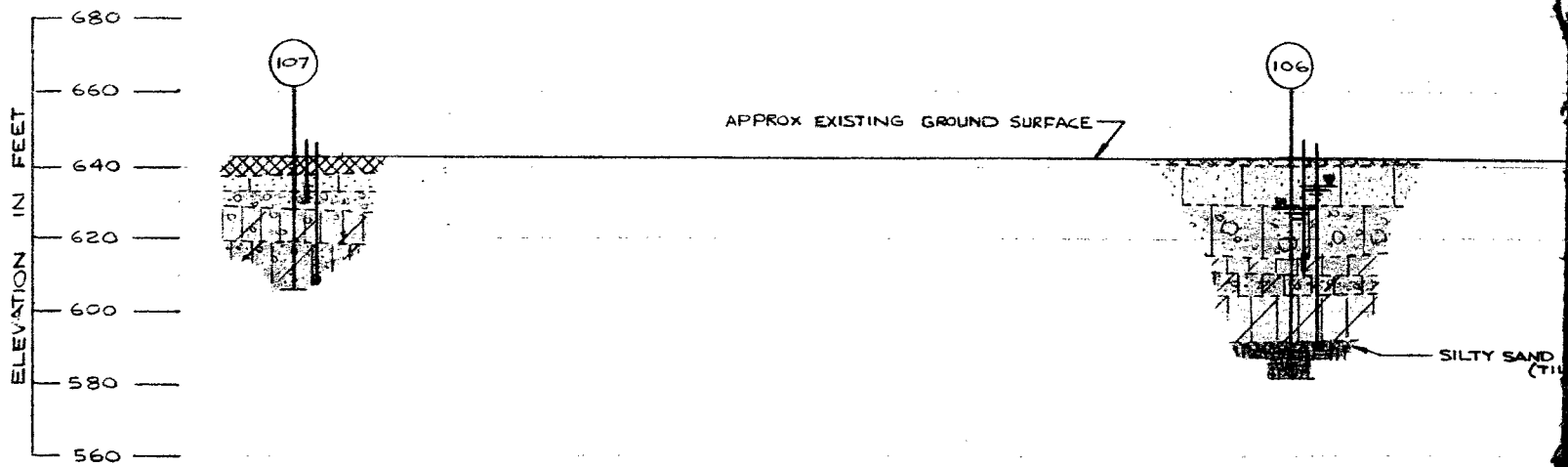
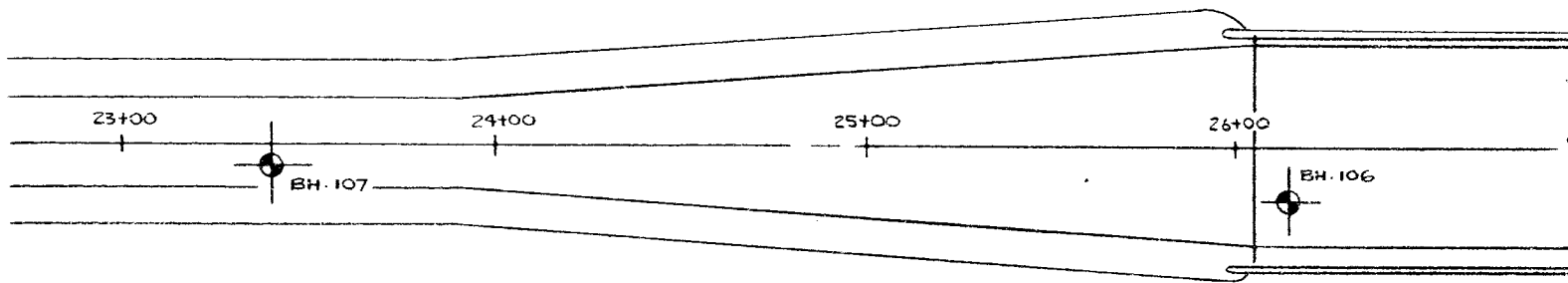
Date JAN. 22, 1975

Golder Associates

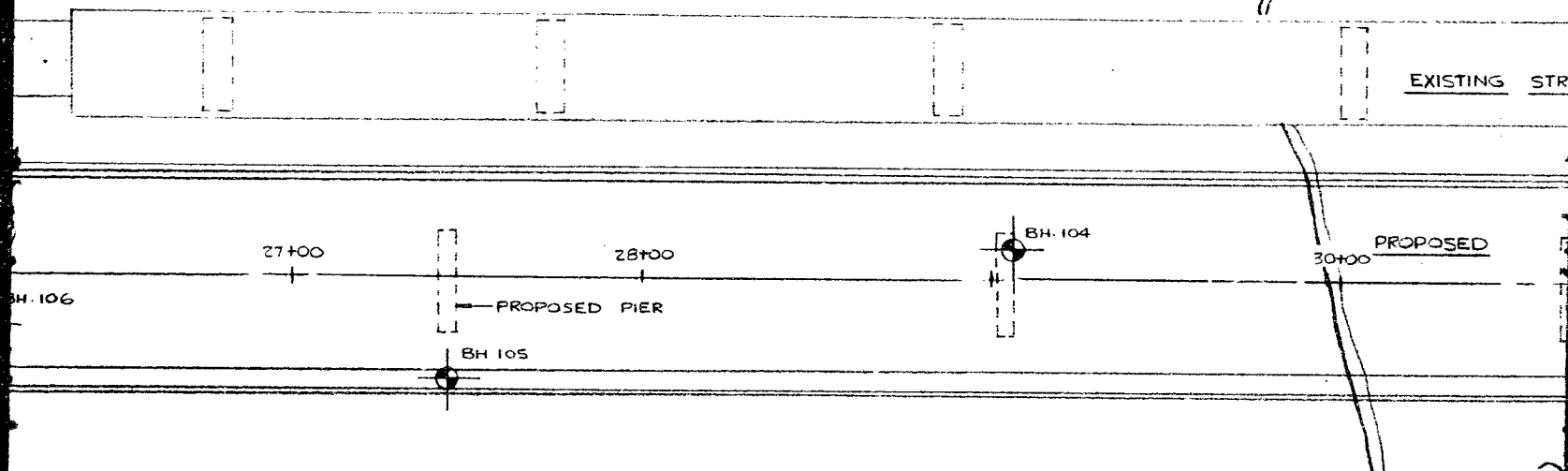
Drawn D.M.
Chkd. _____
Appd. _____



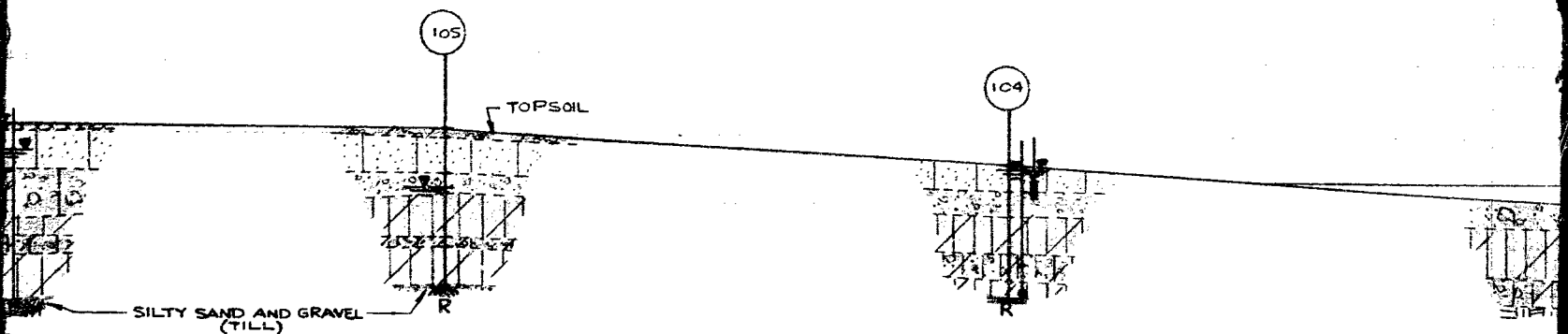
COCKSHUTT



GRAND RIVER



PLAN
SCALE: 1" TO 40'



SECTION ALONG CENTRELINE OF PROPOSED COCKSHUTT BRIDGE

SCALE: 1" TO 40'

GRAND RIVER

EXISTING STRUCTURE

EXISTING PIER

ROAD

PROPOSED

BH 103

BRIDGE

31+00

32+00

BH 102

33+00

BH 1

34+00

BH 101

103

102

101

G.S. AT BH.

SILTY SAND TO
SANDY SILT

ELEVATION IN FEET

680

660

640

620

600

580

560

BLACKSHUTT BRIDGE

RE
C
A
B
E
7

LEGEND

PRELIMINARY

BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION
-GOLDER ASSOC. REPORT No. 73154 DATED OCT. 1973)

BOREHOLE LOCATION IN PLAN (PRESENT INVESTIGATION)



BOREHOLE IN ELEVATION

WATER LEVEL IN BOREHOLE (REFER TO RECORD
OF BOREHOLE SHEETS FOR DETAILS)

STANDPIPE IN BOREHOLE

PIEZOMETER IN BOREHOLE

REFUSAL TO AUGERING (PROBABLY BEDROCK)

STRATIGRAPHY

SANDY SILT TO SILTY CLAY, TRACE SAND AND
GRAVEL, TRACE ORGANIC MATTER (FILL)

LOOSE TO COMPACT BROWN SILTY SAND



COMPACT TO DENSE BROWN SAND AND GRAVEL

FIRM TO VERY STIFF RED BROWN TO GREY LAYERED
SILT, CLAYEY SILT AND SILTY CLAYVERY STIFF TO HARD RED BROWN TO GREY OCCASIONALLY FAINTLY
LAYERED CLAYEY SILT TO SILTY CLAY TR. SAND AND GRAVEL (TILL-LIKE)HARD RED BROWN TO GREY LAYERED SILT, CLAYEY SILT
AND SILTY CLAY

HIGHLY WEATHERED SHALE BEDROCK

FRACTURED TO SOUND GREY INTERBEDDED SHALE
AND DOLOMITE SOME GYPSUM INCLUSIONS (BEDROCK)

SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.REFERENCE: PRELIMINARY PLAN TITLED
COCKSHUTT ROAD BIRKETT LANE TO BLOSSOM
AVENUE CITY OF BRANTFORD COUNTY OF
BRANT SUPPLIED BY MCCORMICK RANKIN
& ASSOCIATES LIMITED. DRAWING No.
702-2. UNDATED.

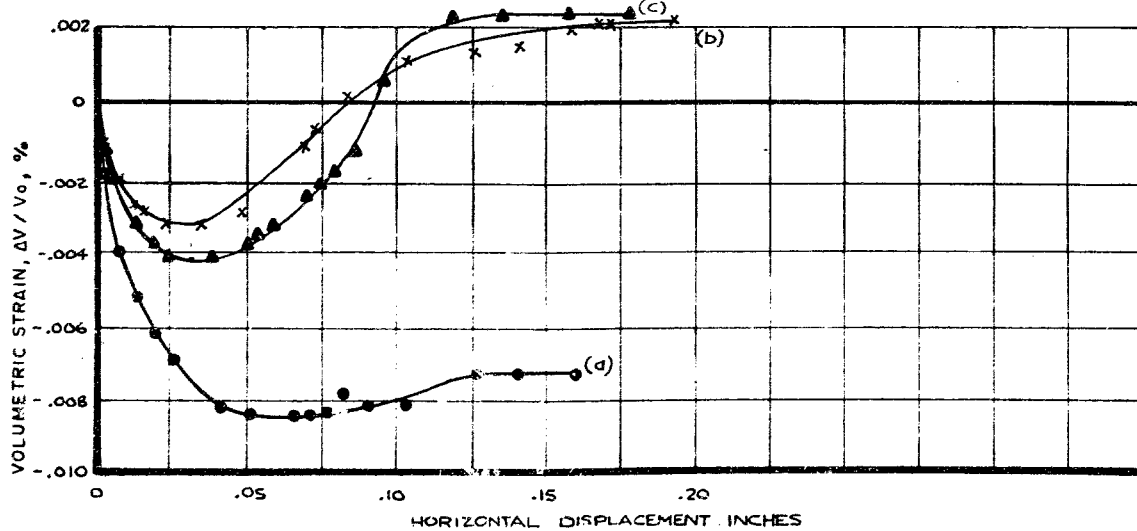
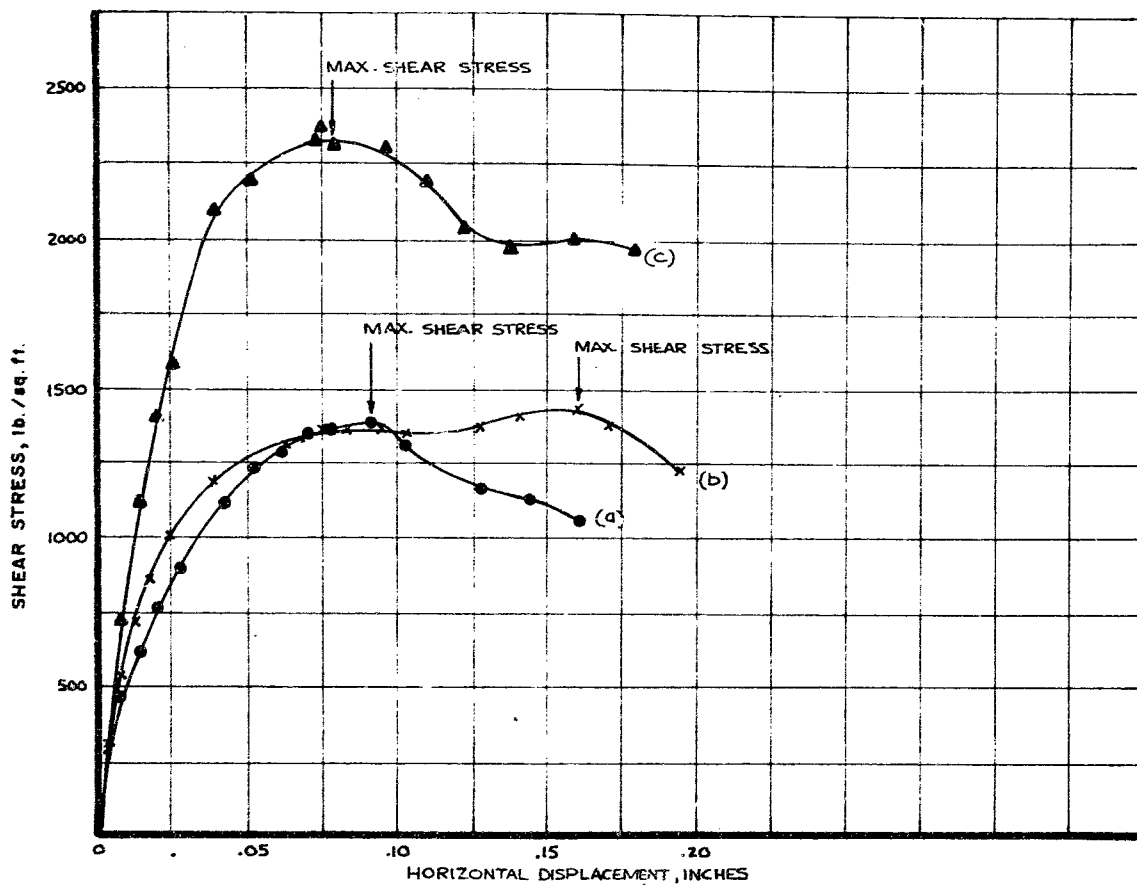
NOTE

Date indicating the various values 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.

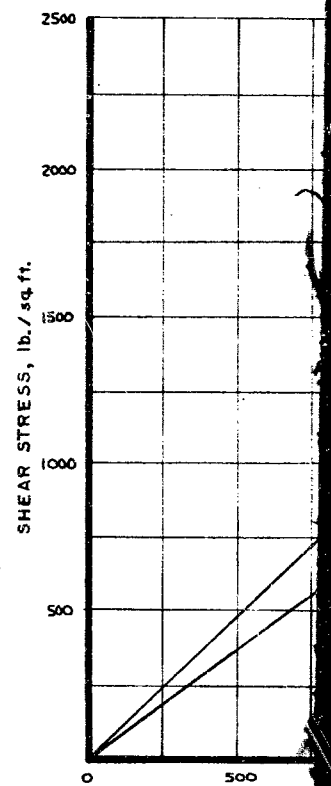
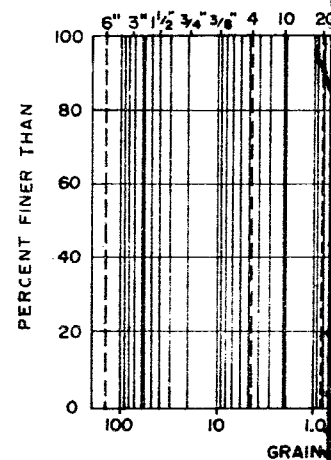
Date FEB. 7, 1975

Golder Associates

Drawn D.M.
Chkd. _____
Appd. _____



COBBLE	GRAVEL	
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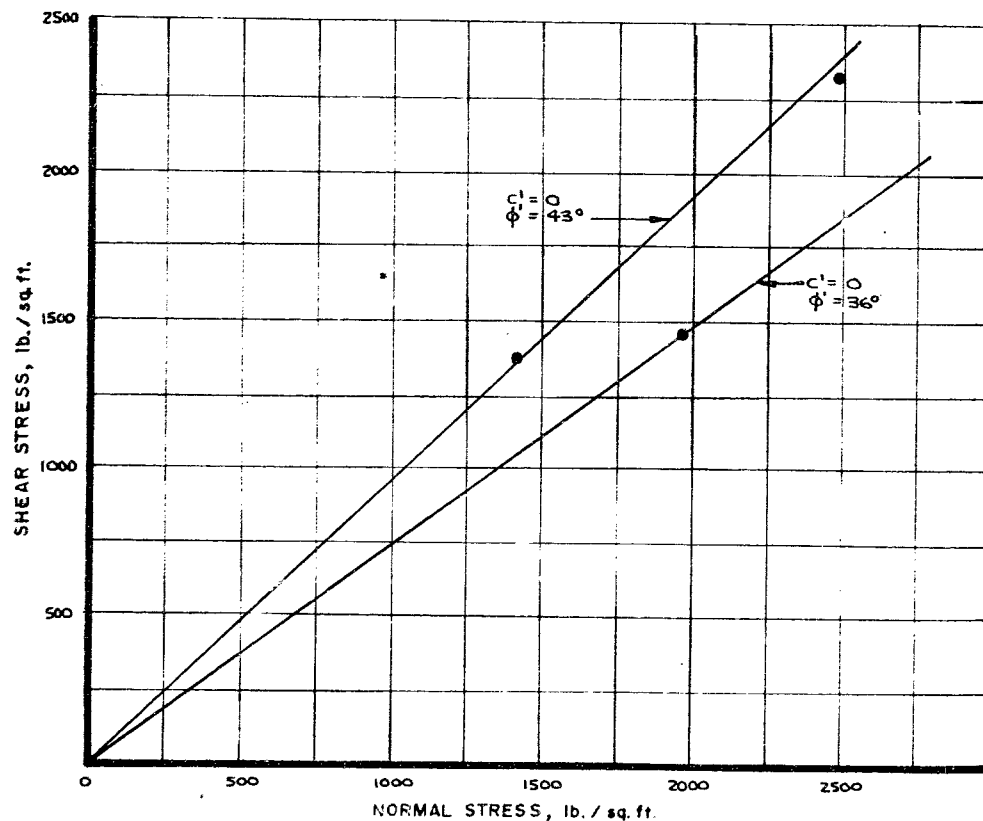
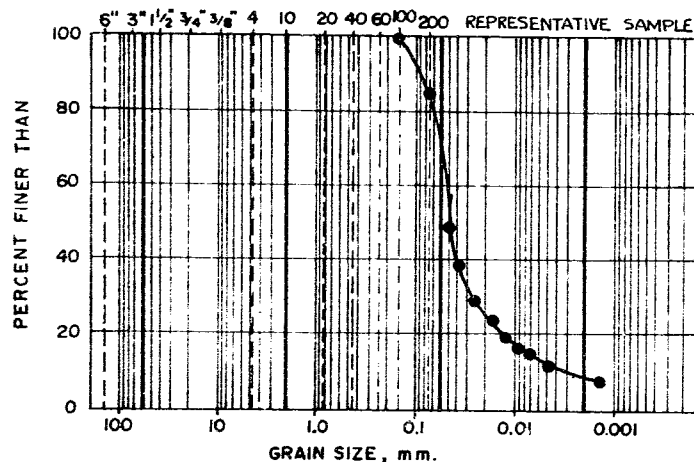


CONSOLIDATED DRAINED DIRECT SHEAR TESTS

PRELIMINARY

COBBLE	GRAVEL	SAND	SILT	CLAY
--------	--------	------	------	------

M.I.T. GRAIN SIZE SCALE



BOREHOLE NUMBER	
SAMPLE NUMBER	
SAMPLE DEPTH, ft.	

SPECIMEN LENGTH, in.	
SPECIMEN HEIGHT, in.	

TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	
	WATER CONTENT, AFTER CONSOLIDATION, %	
	AVERAGE RATE OF STRAIN, % / hr.	
	TIME TO FAILURE, days	
	WATER CONTENT, AFTER TEST, % AVG.	

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	13	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS		
	SECOND PASS		
	THIRD PASS		
	HORIZONTAL DISPLACEMENT, INCHES		
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS		
	SECOND PASS		
	THIRD PASS		

NATURAL WATER CONTENT, w , %	
LIQUID LIMIT, w_L	
PLASTIC LIMIT, w_p	
UNIT WEIGHT, γ_t , lb./cu. ft.	

REMARKS	(a) SANDY SILT AND FINE SAND
	(b) FINE SANDY SILT
	(c) FINE SANDY SILT AND CLAYEY
	SAMPLES TRIMMED FROM 3' INCH

Date FEB. 11, 1975

Golder Associates

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 3

PRELIMINARY

a	b	c	d
→	→	→	→

BOREHOLE NUMBER	101	101	101	
SAMPLE NUMBER	9	9	9	
SAMPLE DEPTH ft	28	28	28	

SPECIMEN LENGTH, in.	2.34	2.34	2.34	
SPECIMEN HEIGHT, in.	.985	.974	.989	

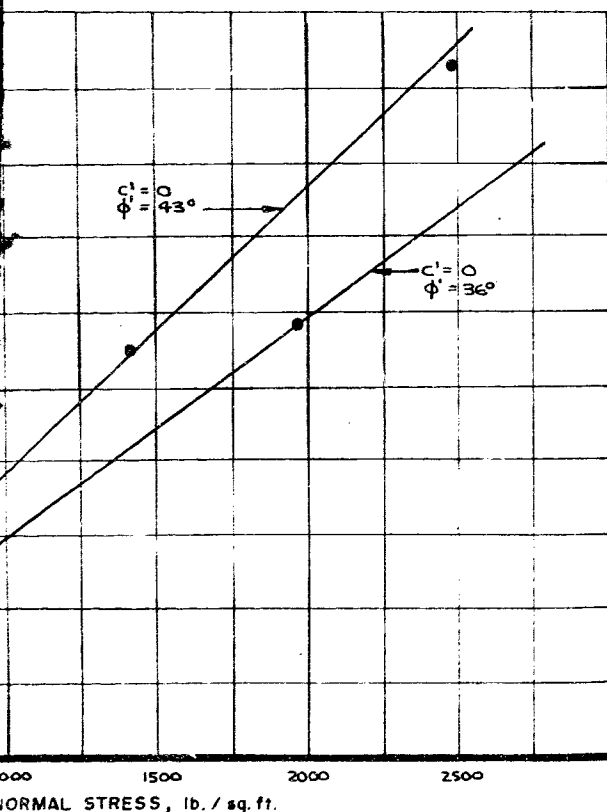
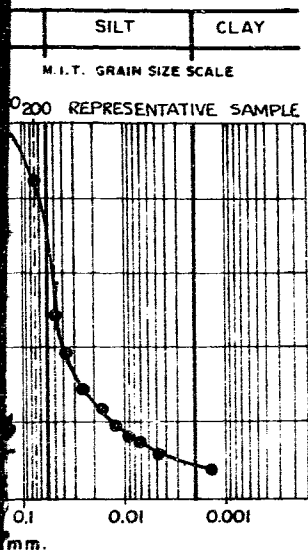
TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	19	19	19	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	1440	1970	2500	
	WATER CONTENT, AFTER CONSOLIDATION, %				
	AVERAGE RATE OF STRAIN, % / hr.	0.5	0.5	0.5	
	TIME TO FAILURE, days	1	2	2	
	WATER CONTENT, AFTER TEST, % AVG.	26	28	26	

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	1380	1440	2320	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	
	HORIZONTAL DISPLACEMENT, INCHES	.080	.1600	.080	
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	

NATURAL WATER CONTENT, w , %	19			
LIQUID LIMIT, w_L	17			
PLASTIC LIMIT, w_p	15			
UNIT WEIGHT, γ_t , lb./cu. ft.	132	132	135	

REMARKS (a) SANDY SILT AND FINE SAND
(b) FINE SANDY SILT
(c) FINE SANDY SILT AND CLAYEY SILT

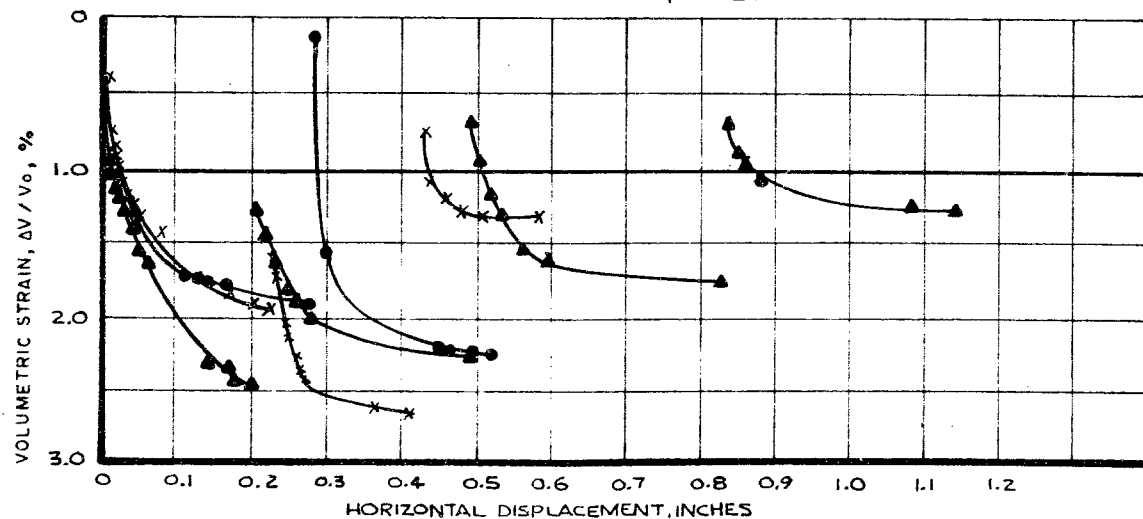
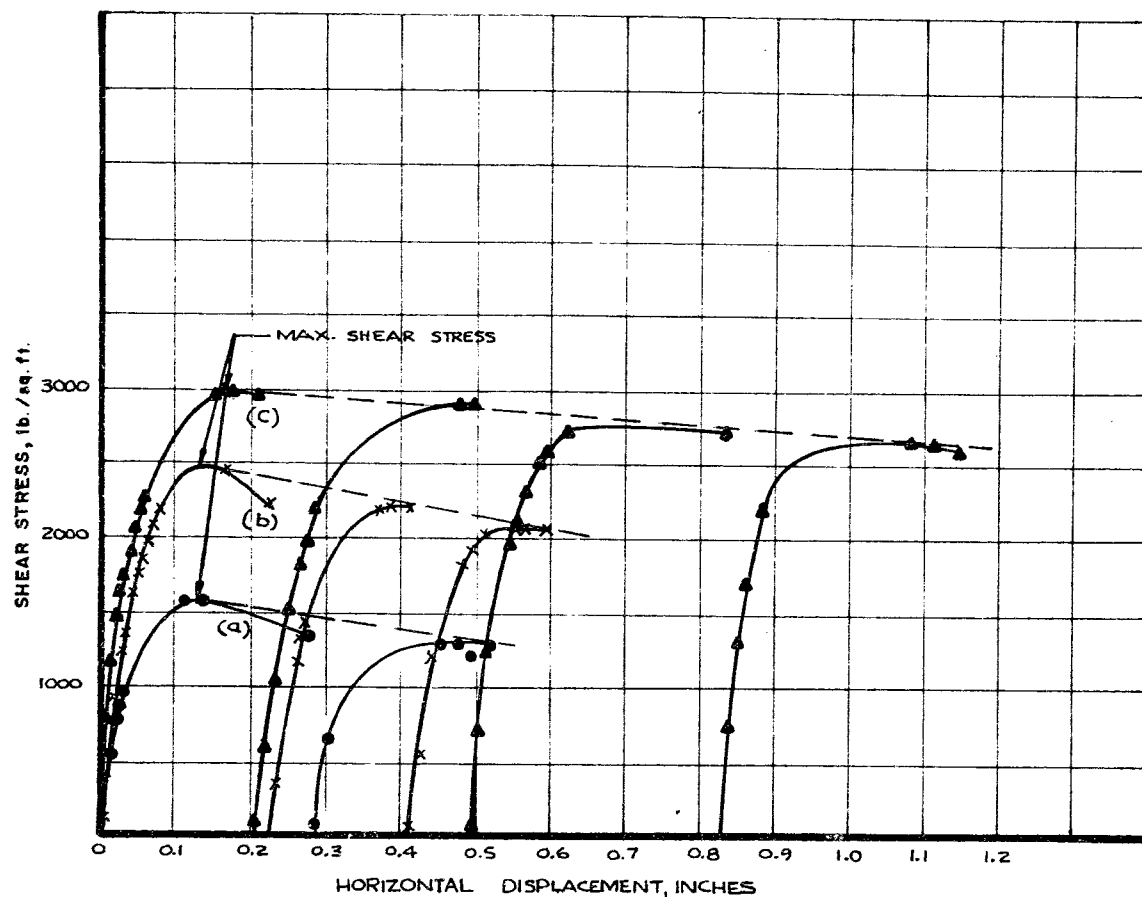
SAMPLES TRIMMED FROM 3" INCH SHELBY TUBE



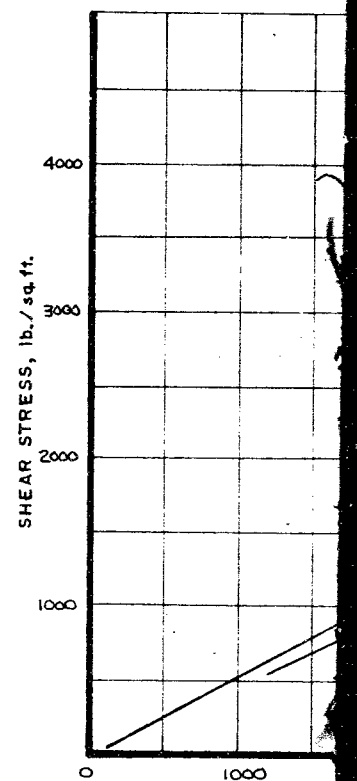
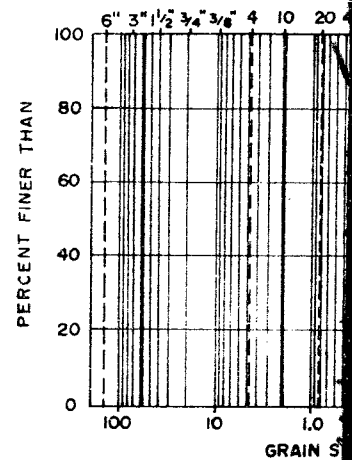
Date FEB. 11, 1975

Golder Associates

Drawn D.M.
Chkd. _____
Appd. _____

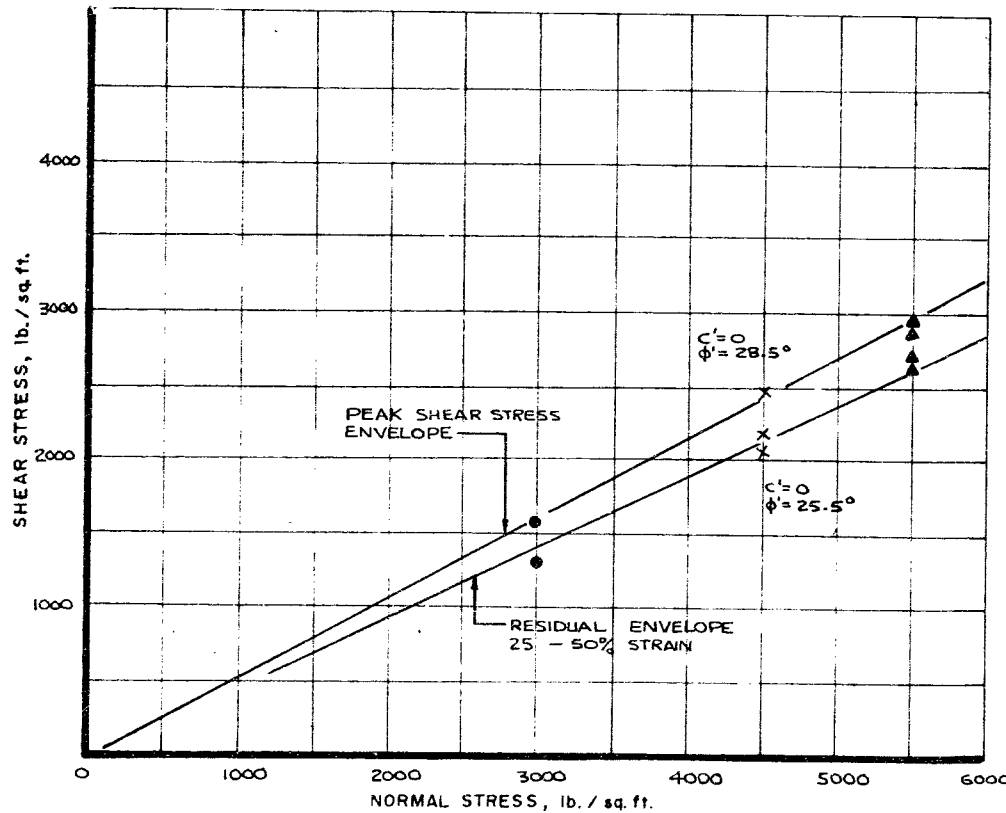
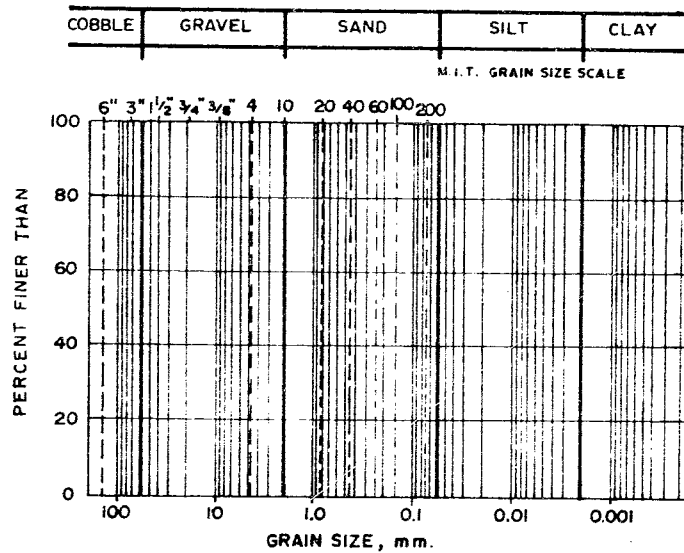


COBBLE	GRAVEL	S
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CONSOLIDATED DRAINED DIRECT SHEAR TESTS

PRELIMINARY



BOREHOLE NUMBER
SAMPLE NUMBER
SAMPLE DEPTH, INCHES RECOVERED

SPECIMEN LENGTH, in.
SPECIMEN HEIGHT, in.

TEST CONDITIONS
WATER CONTENT, BEFORE CONSOLIDATION
NORMAL (CONSOLIDATION) STRESS
WATER CONTENT, AFTER CONSOLIDATION
AVERAGE RATE OF STRAIN, %/MIN
TIME TO FAILURE, days
WATER CONTENT, AFTER TEST

TEST RESULTS
PEAK SHEAR STRESS, lb./sq. ft.
RESIDUAL SHEAR STRESS, lb./sq. ft.
HORIZONTAL DISPLACEMENT, in.
SHEAR STRAIN AT RESIDUAL SHEAR STRESS, %
NATURAL WATER CONTENT, w , %
LIQUID LIMIT, w_L , %
PLASTIC LIMIT, w_p , %
UNIT WEIGHT, γ_t , lb./cu. ft.

REMARKS
SAMPLES TRIMMED FROM IRREGULAR LAYERED LAYERS

Date FEB. 14, 1975

Golder Associates

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 4

PRELIMINARY

a	b	c	d
—●—	—x—	—▲—	

BOREHOLE NUMBER	101	101	101	
SAMPLE NUMBER	17	17	17	
SAMPLE DEPTH, INCHES RECOVERY	66	66	66	

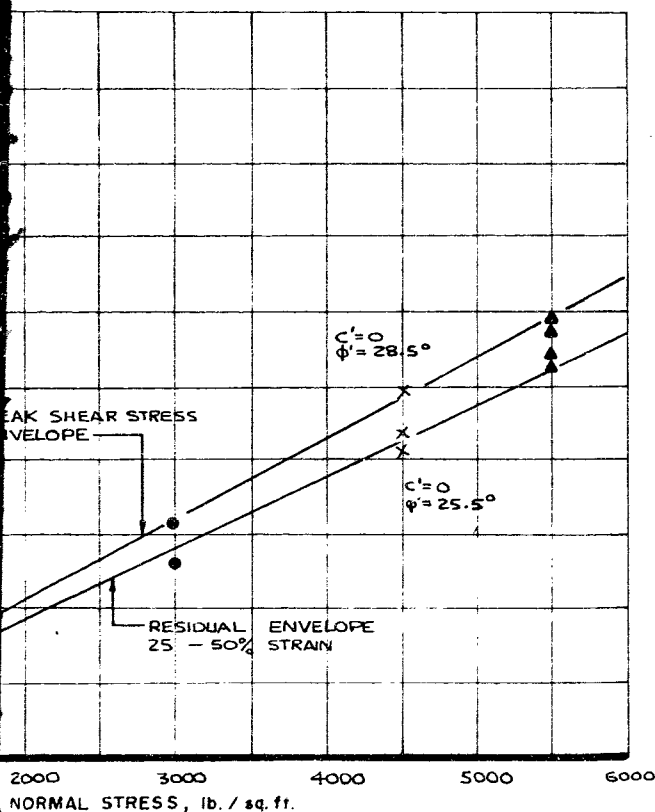
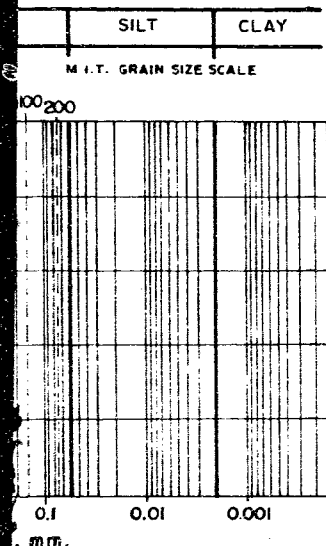
SPECIMEN LENGTH, in.	2.34	2.34	2.34	
SPECIMEN HEIGHT, in.	2.34	2.34	2.34	

TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	26	22	22	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	3000	4500	5500	
	WATER CONTENT, AFTER CONSOLIDATION, %				
	AVERAGE RATE OF STRAIN, % / hr.	.25	.25	.25	
	TIME TO FAILURE, days	2	2	2	
	WATER CONTENT, AFTER TEST, % AVG.	25	21	20	

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	1579	2499	2981	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS	1293	2181	2896	
	SECOND PASS	—	2069	2731	
	THIRD PASS	—	—	2672	
	HORIZONTAL DISPLACEMENT, INCHES	.134	.140	.165	
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS	20.2	17.5	20.4	
	SECOND PASS	—	25.0	27.0	
	THIRD PASS	—	—	46.5	

	SILT	22	22	22	
NATURAL WATER CONTENT, w , %	CLAY	26	27	27	
LIQUID LIMIT, w_L		28	28	30	
PLASTIC LIMIT, w_p		17	18	17	
UNIT WEIGHT, γ_t , lb./cu. ft.		132	132	135	

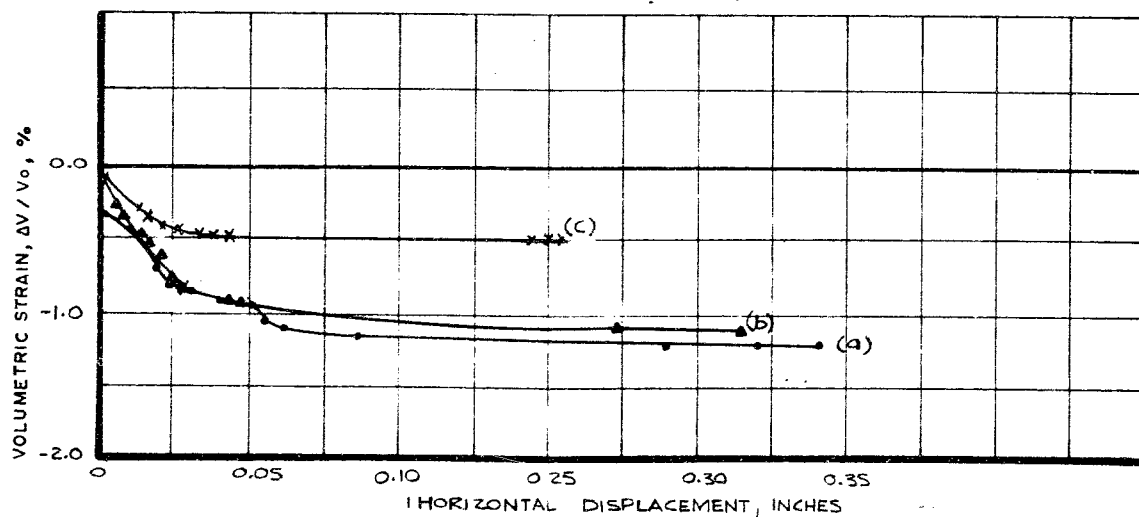
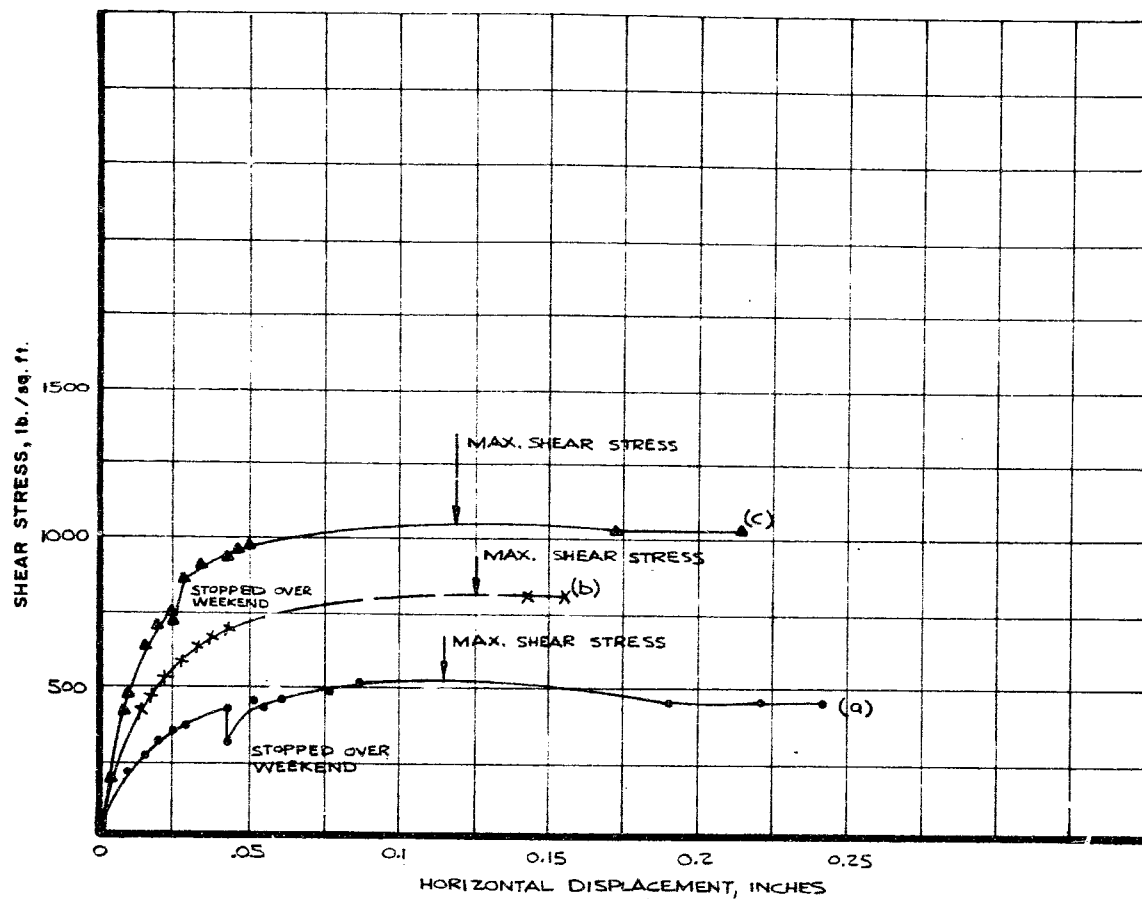
REMARKS SAMPLES TRIMMED FROM 3 INCH SHELBY TUBE
IRREGULAR LAYERED, RED CLAYEY SILT WITH CLAY
LAYERS



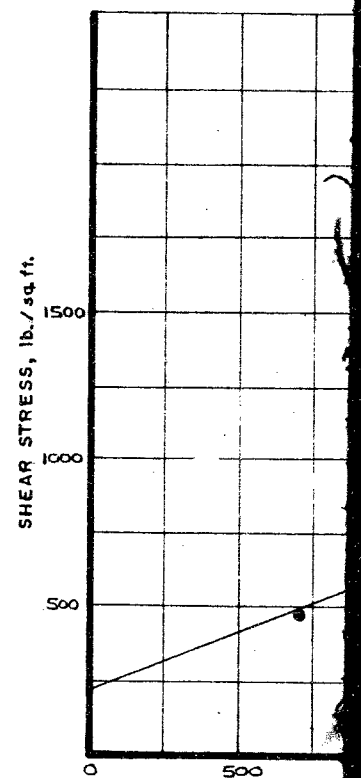
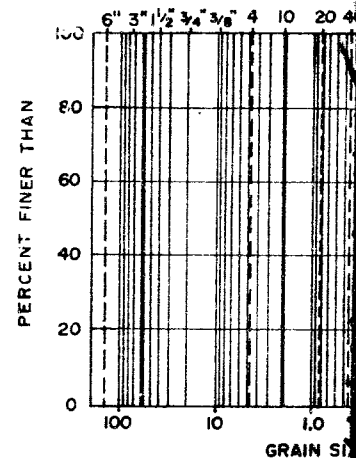
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COBBLE	GRAVEL	SAND
--------	--------	------

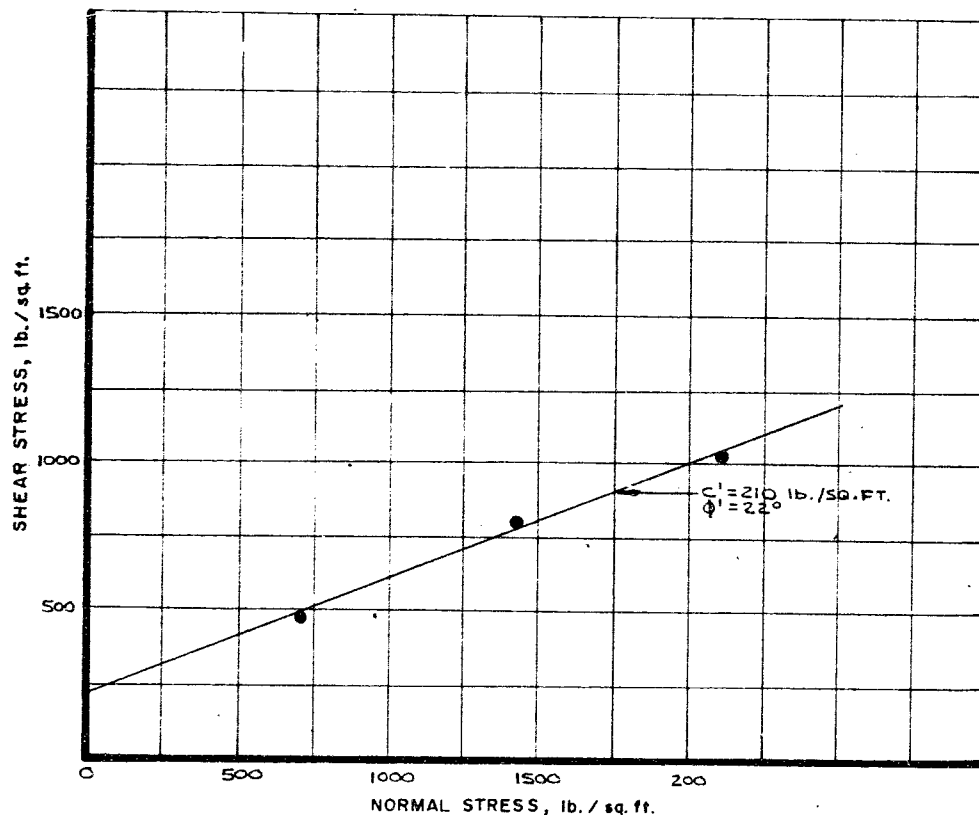
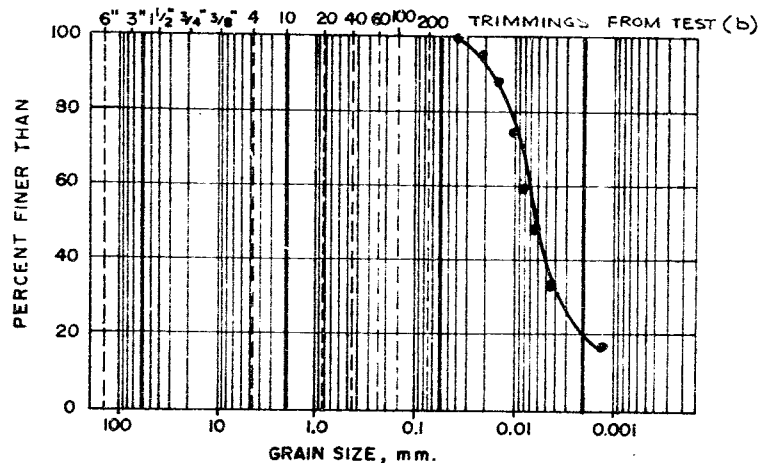


CONSOLIDATED DRAINED DIRECT SHEAR TESTS

PRELIMINARY

COBBLE	GRAVEL	SAND	SILT	CLAY
--------	--------	------	------	------

M.I.T. GRAIN SIZE SCALE



TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.
	WATER CONTENT, AFTER CONSOLIDATION
	AVERAGE RATE OF STRAIN, % / hr.
	TIME TO FAILURE, days
	WATER CONTENT, AFTER TEST, %

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST
	SECOND
	THIRD
	HORIZONTAL DISPLACEMENT, in.
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, %
	FIRST
	SECOND
	THIRD

NATURAL WATER CONTENT, w , %
LIQUID LIMIT, w_L
PLASTIC LIMIT, w_p
UNIT WEIGHT, γ_t , lb./cu. ft.

REMARKS	SAMPLES TRIMMED
	720 lb./sq. ft. SHEAR
	1440 lb./sq. ft. SHEAR
	2179 lb./sq. ft. SHEAR

Date MAR. 3, 1975

Golder Associa

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 5

PRELIMINARY

a	b	c	d
•	x	Δ	

BOREHOLE NUMBER	101	101	101	
SAMPLE NUMBER	11	11	11	
SAMPLE DEPTH, INCHES RECOVERY	7-8	9-10	11-12	

SPECIMEN LENGTH, in.	2.34	2.34	2.34	
SPECIMEN HEIGHT, in.	.994	.939	.971	

TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	23	22	19	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	720	1440	2179	
	WATER CONTENT, AFTER CONSOLIDATION, %				
	AVERAGE RATE OF STRAIN, % / hr.	.25	.25	.25	
	TIME TO FAILURE, days	2	2	2	
	WATER CONTENT, AFTER TEST, %	24	25	24	

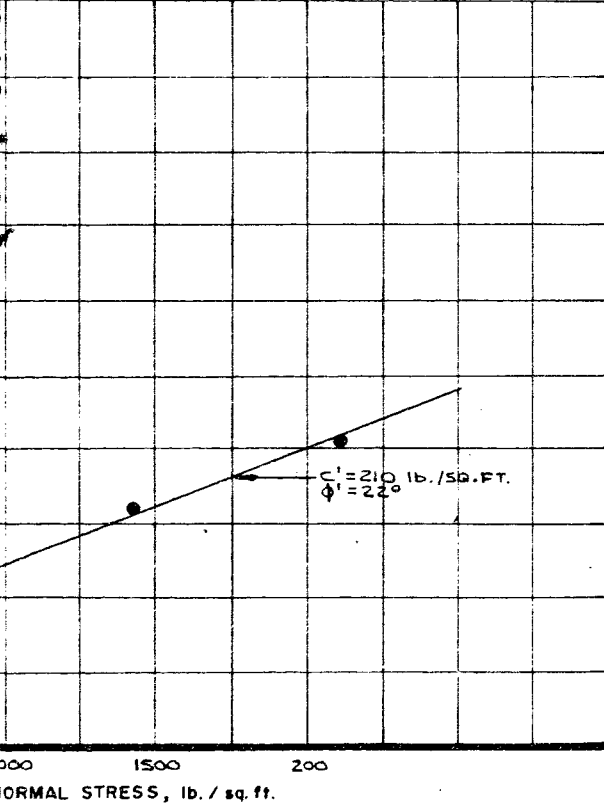
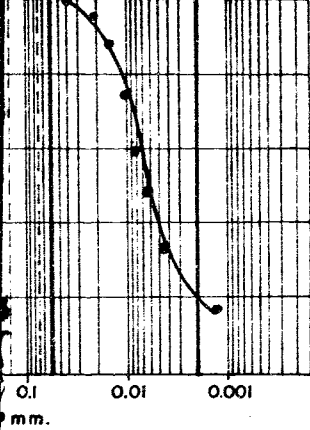
TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	480	807	1030	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	
	HORIZONTAL DISPLACEMENT, INCHES	.115	.125	.120	
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	

NATURAL WATER CONTENT, w , % AVG.	23	22	19	
LIQUID LIMIT, w_L	26	23	22	
PLASTIC LIMIT, w_p	17	17	14	
UNIT WEIGHT, γ_t , lb./cu. ft.	128	139	135	

REMARKS SAMPLES TRIMMED FROM 3" SHELBY TUBE
720 lb./sq. ft. SHEAR PLANE - CLAYEY SILT TO SILT
1440 lb./sq. ft. SHEAR PLANE - CLAYEY SILT TO SILT
2179 lb./sq. ft. SHEAR PLANE - CLAYEY SILT

SILT CLAY
M.I.T. GRAIN SIZE SCALE

200 TRIMMINGS FROM TEST (b)



Date MAR. 3, 1975

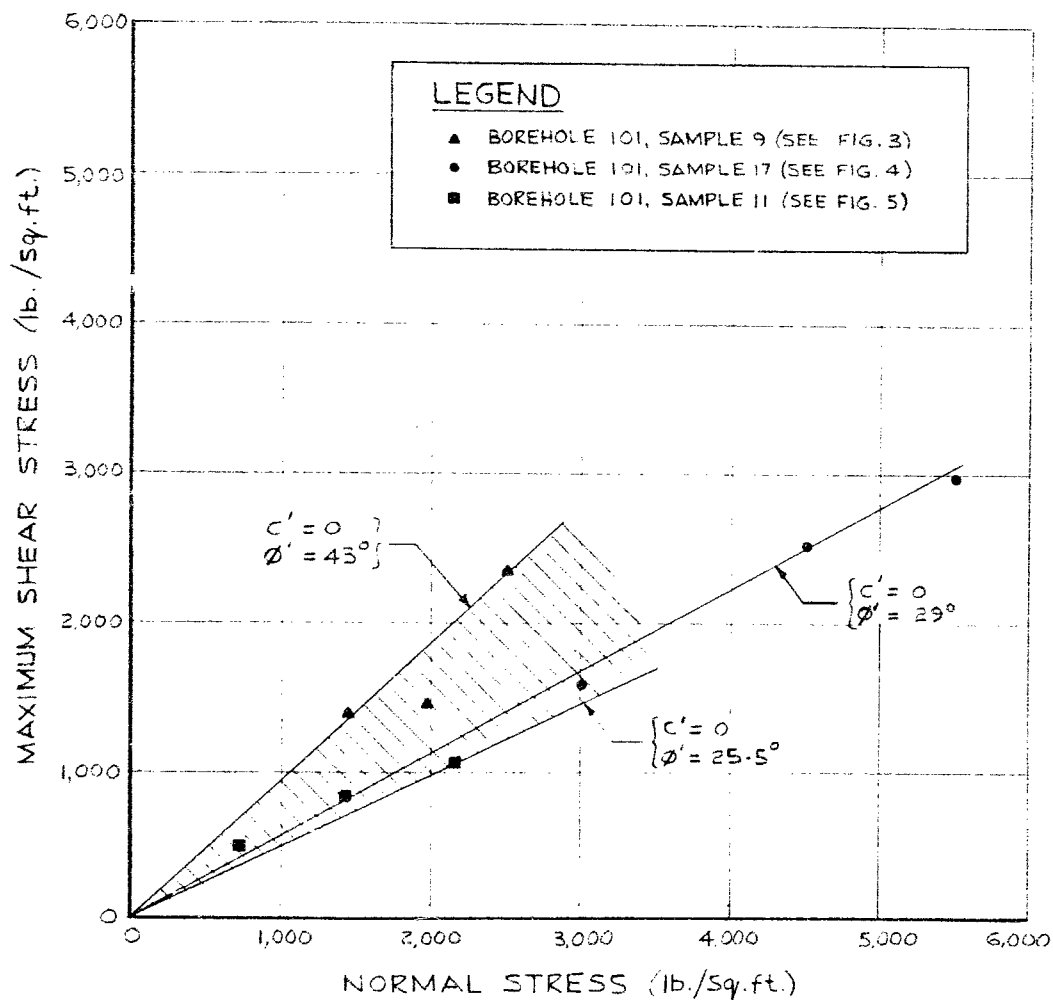
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SUMMARY OF CONSOLIDATED DRAINED DIRECT SHEAR TEST

FIGURE 6

PRELIMINARY

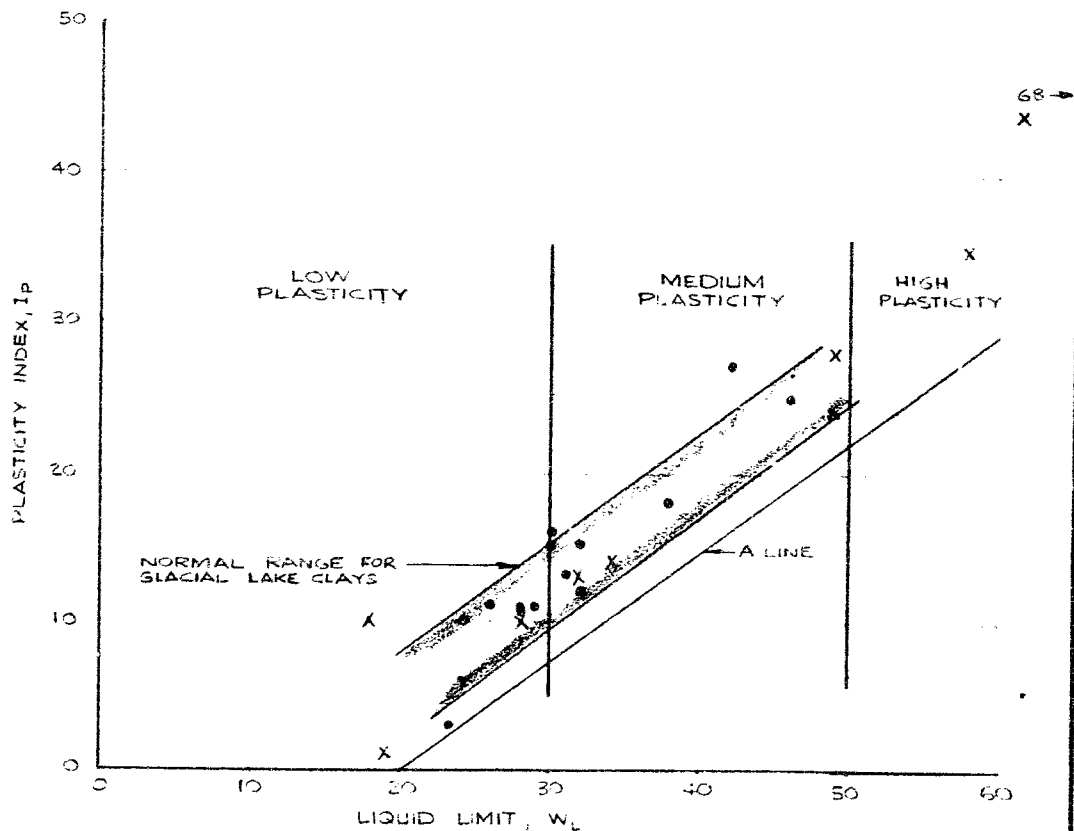


Date MAR. 3, 1975.

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Chkd _____
Appd _____

PRELIMINARY



LEGEND

- PRESENT INVESTIGATION
- X PREVIOUS INVESTIGATION

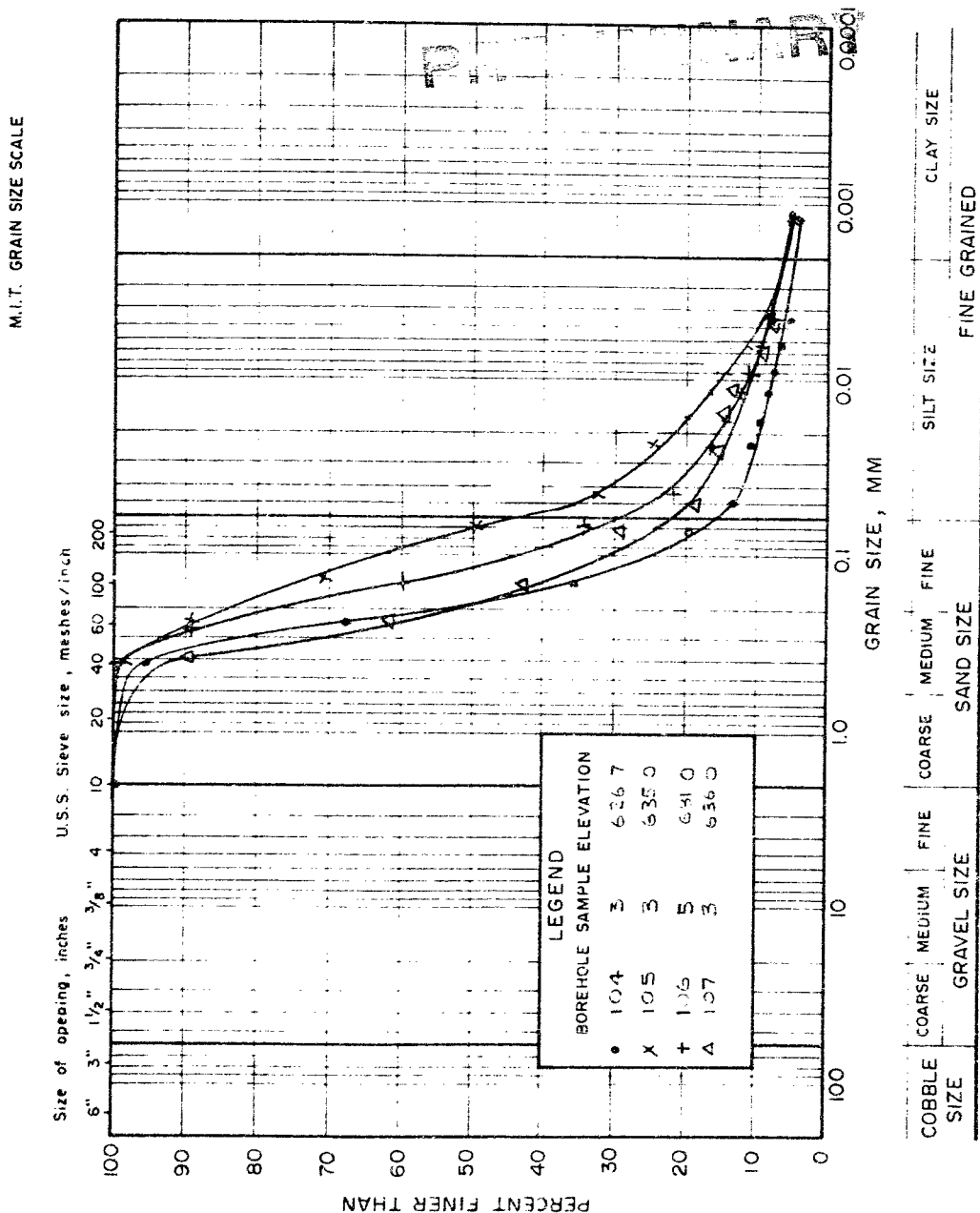
Date FEB. 11, 1975

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 Ck'd
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GRAIN SIZE DISTRIBUTION SILTY SAND

FIGURE 8

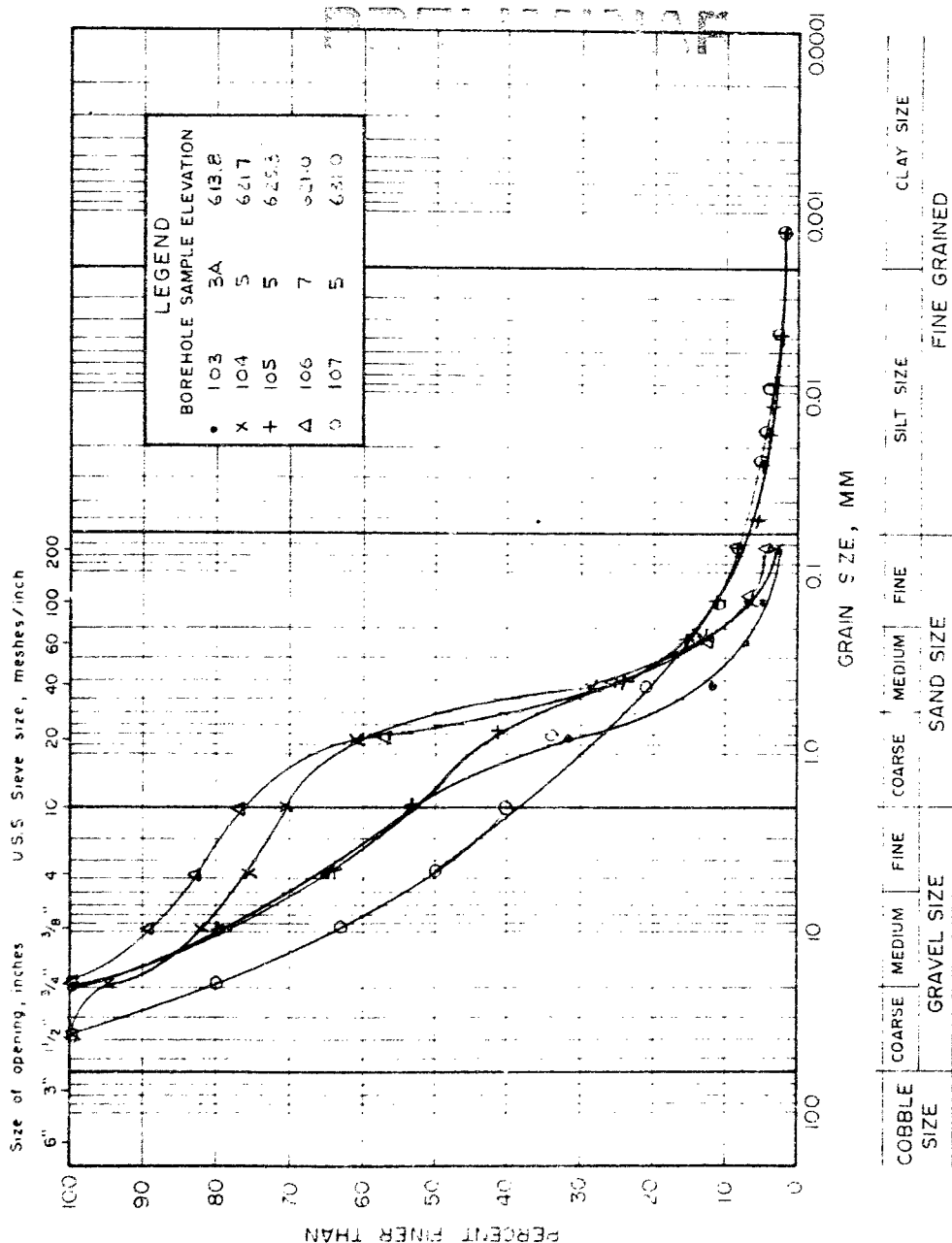


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GRAIN SIZE DISTRIBUTION SAND AND GRAVEL

FIGURE 3

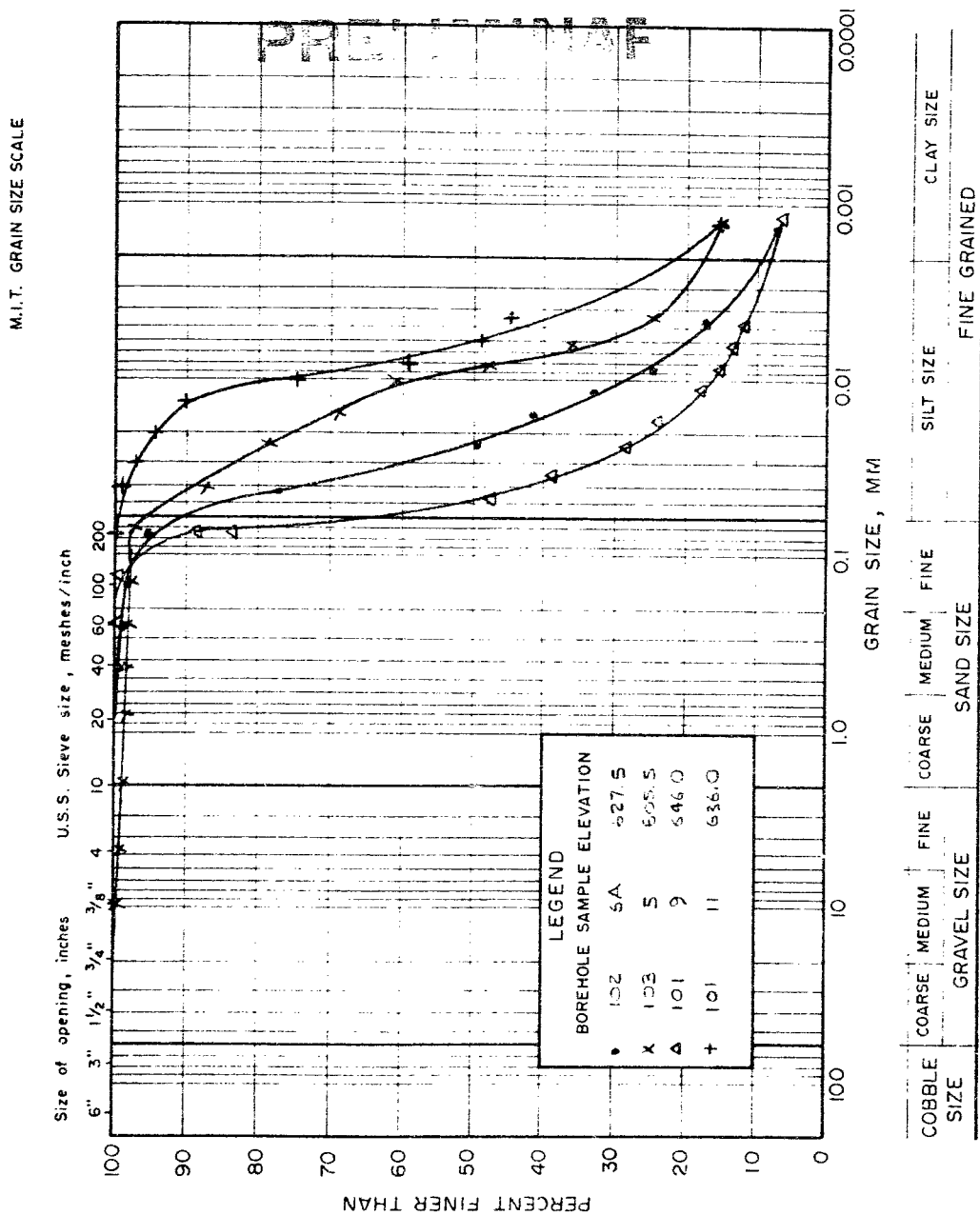
M.I.T. GRAIN SIZE SCALE



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GRAIN SIZE DISTRIBUTION SANDY SILT TO CLAYEY SILT

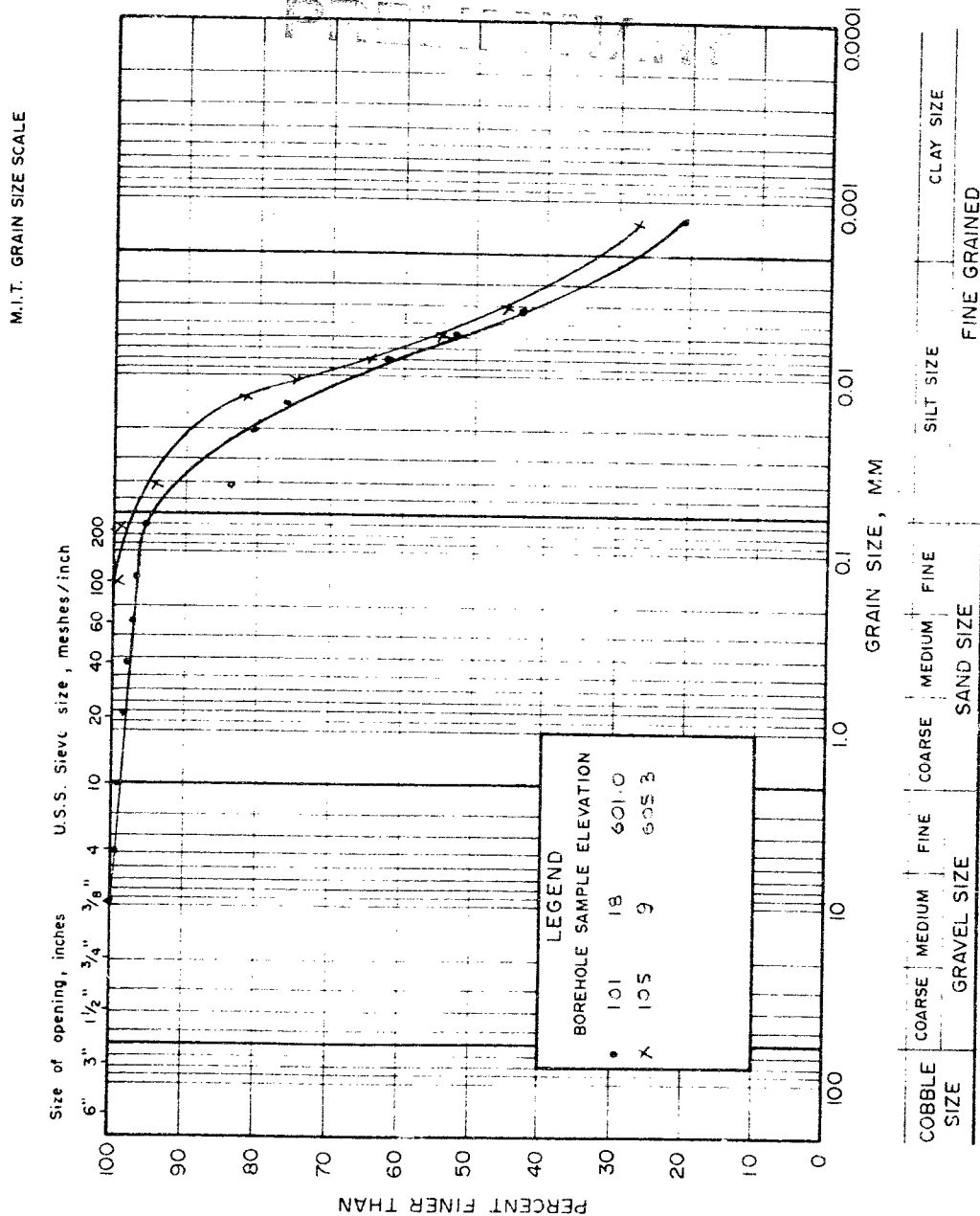
FIGURE 10

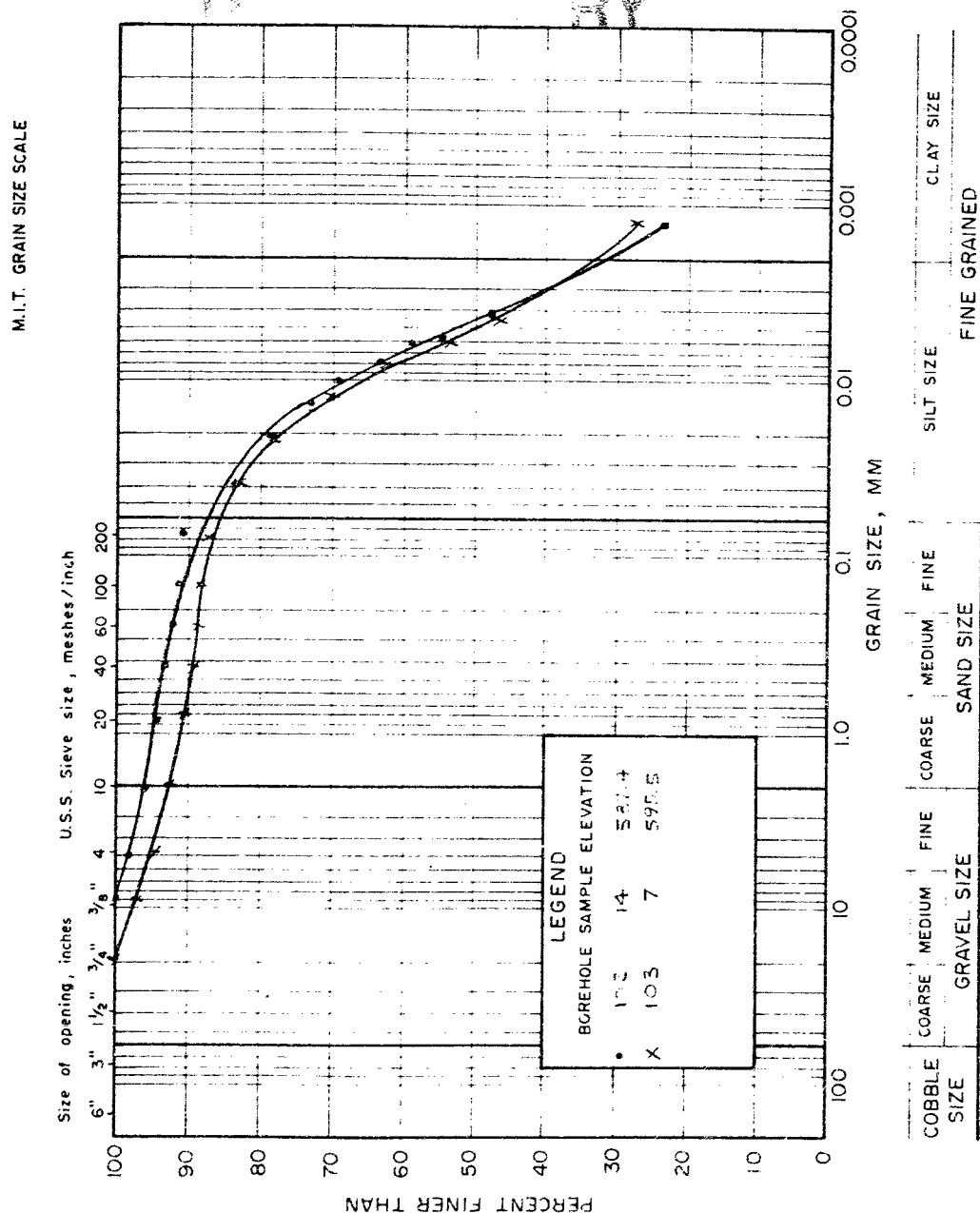


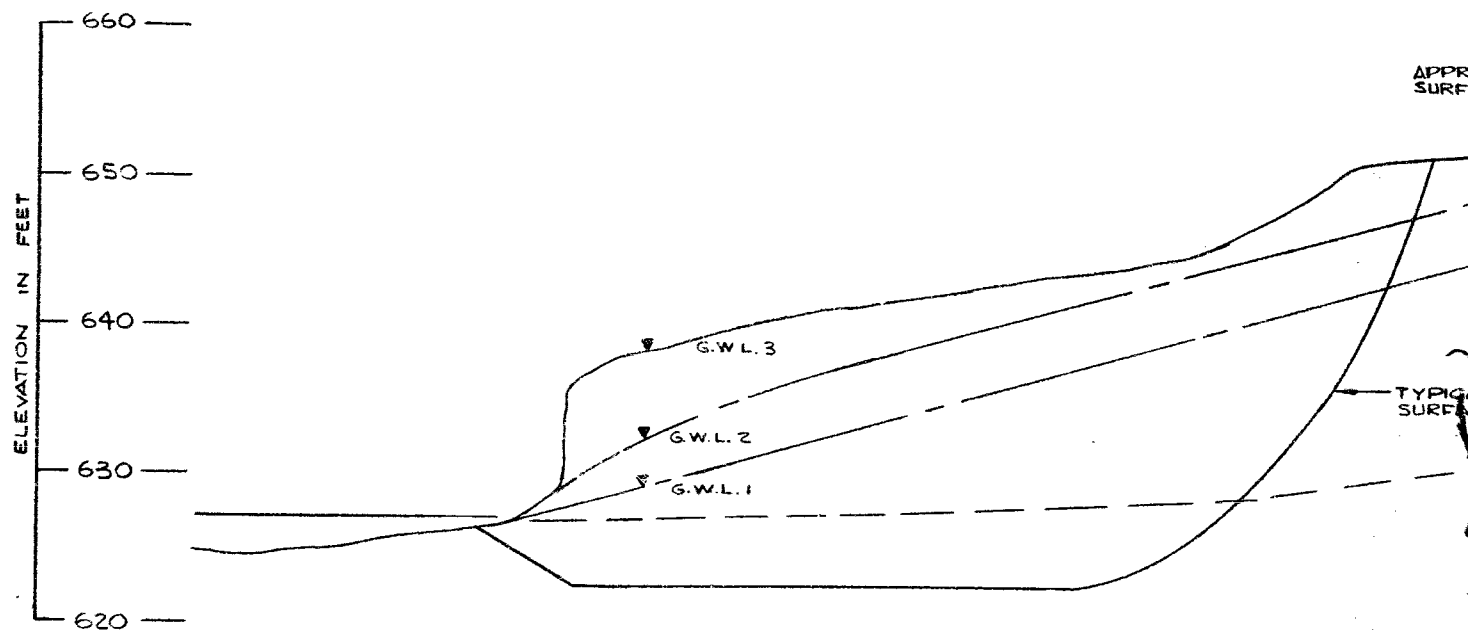
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GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY

FIGURE 11

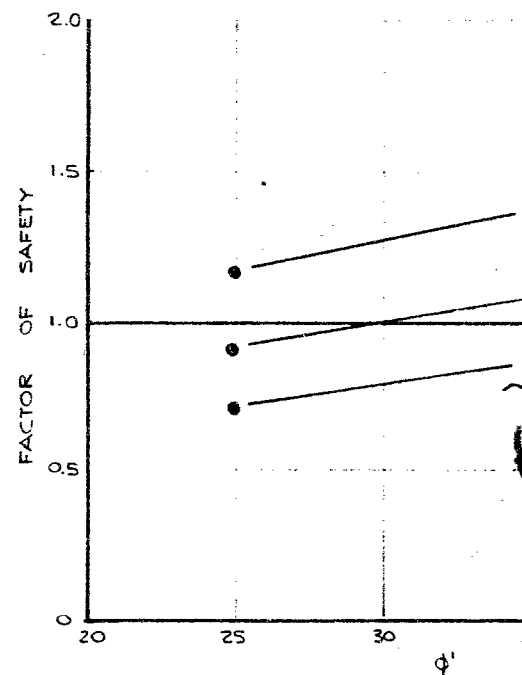
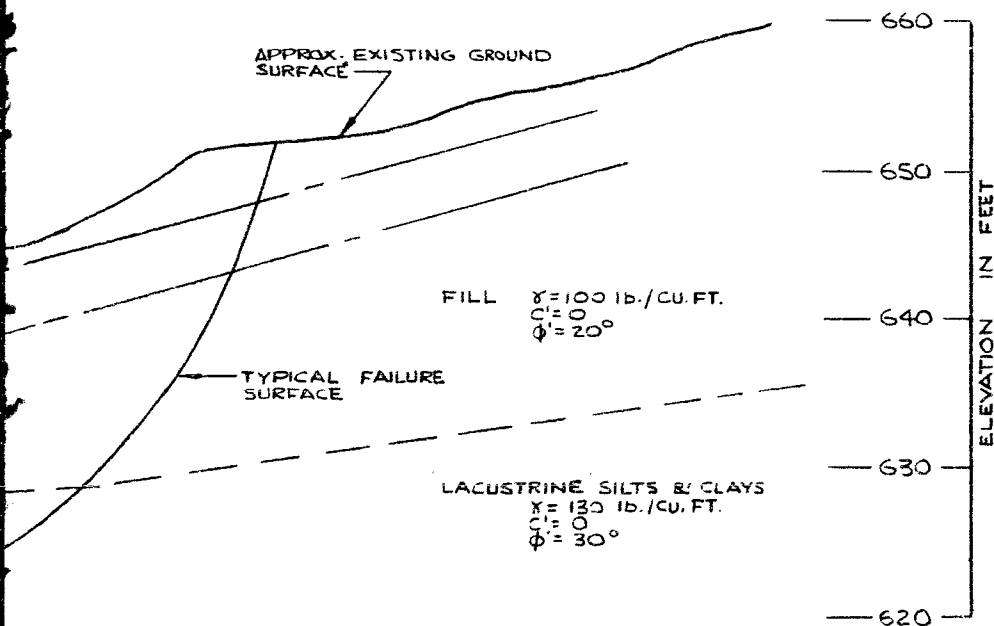






SCALE: 1" TO 10'

PRELIMINARY

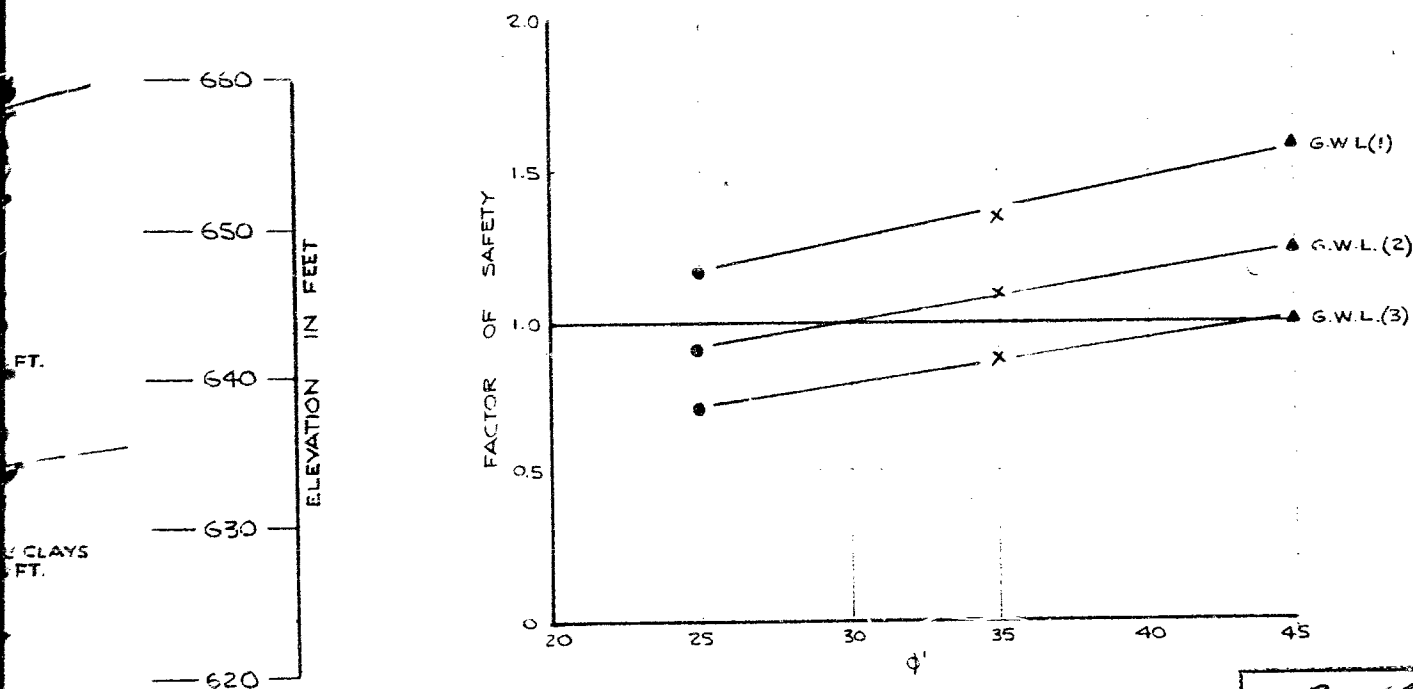


SCALE: 1" TO 10'

Date MAR. 4, 1975

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PRELIMINARY



40 P1-69

GEOCRE No.

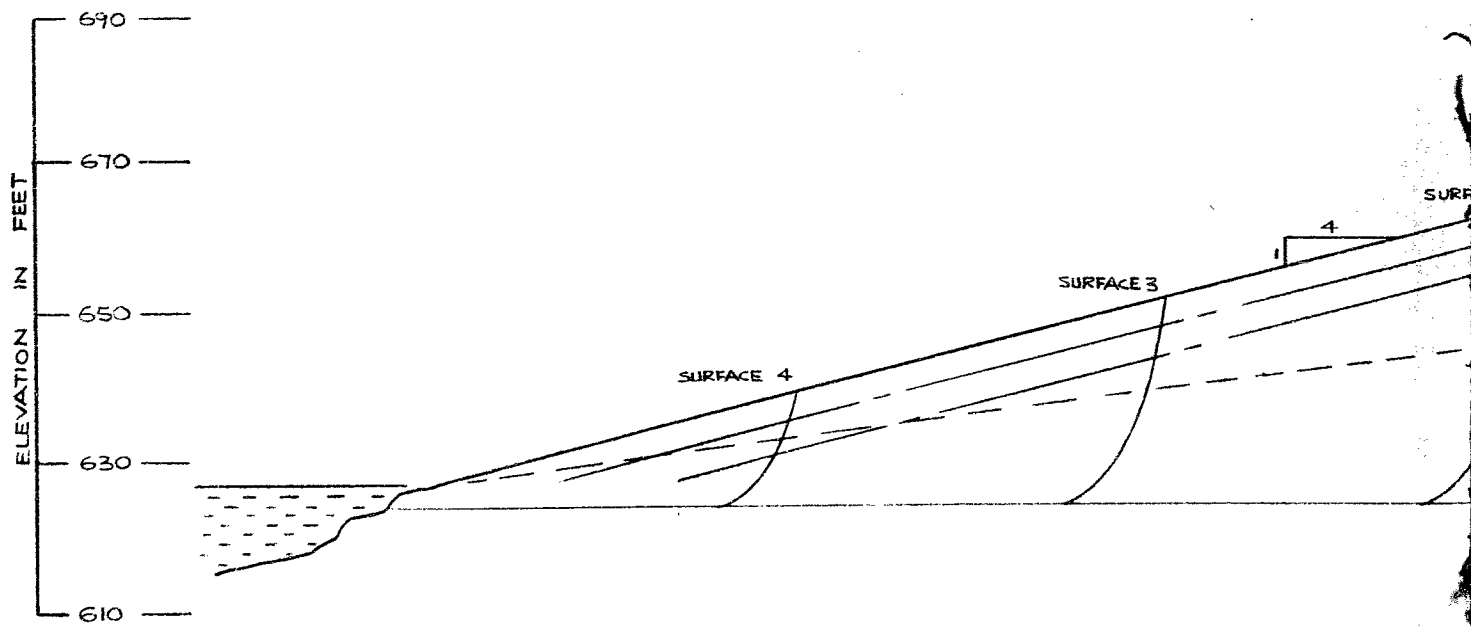
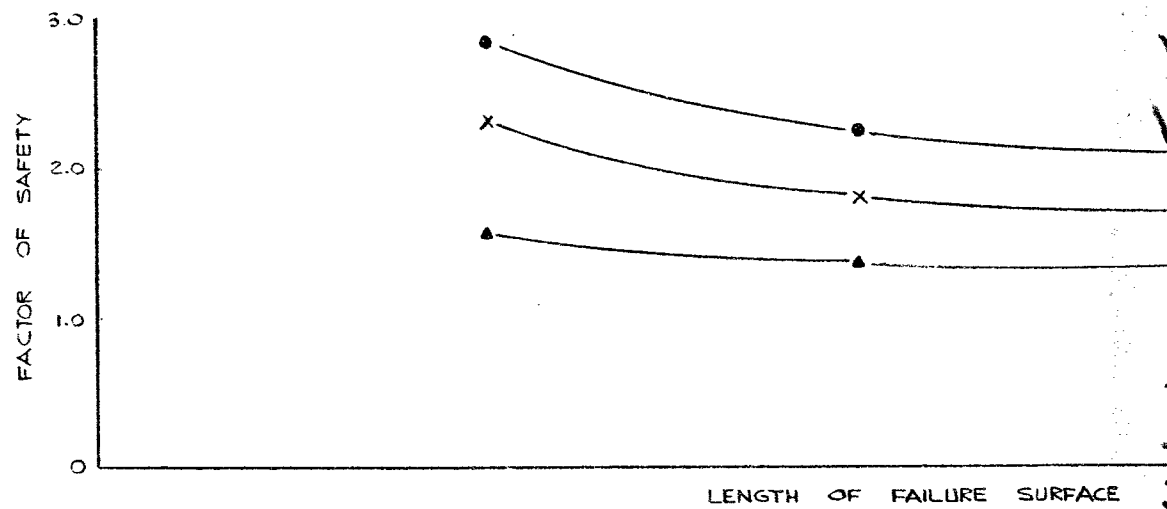
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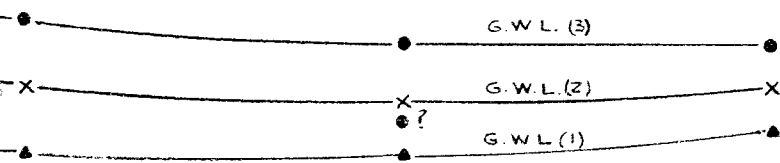
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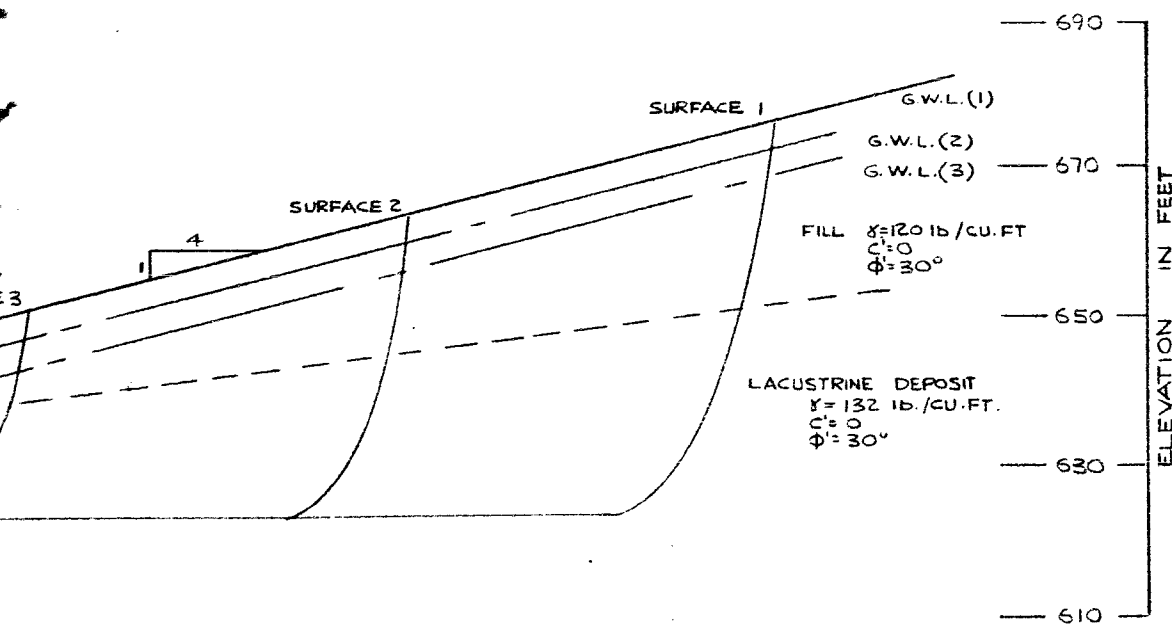
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OF FAILURE SURFACE



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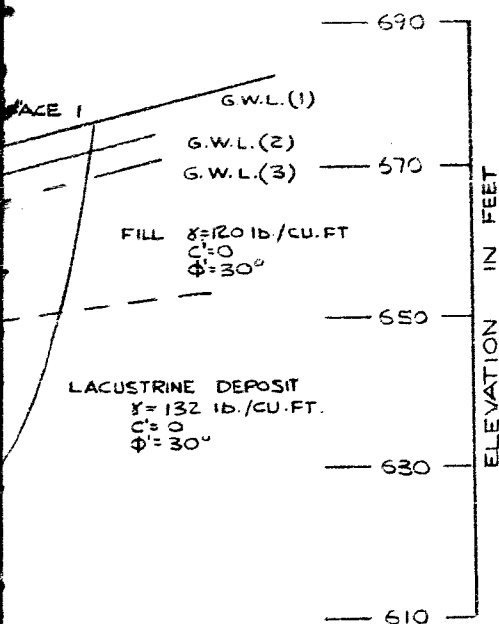
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Gold

SUMMARY OF STABILITY ANALYSIS
- PROPOSED 4 HORIZONTAL & 1 VERTICAL

FIGURE 14

PRELIMINARY



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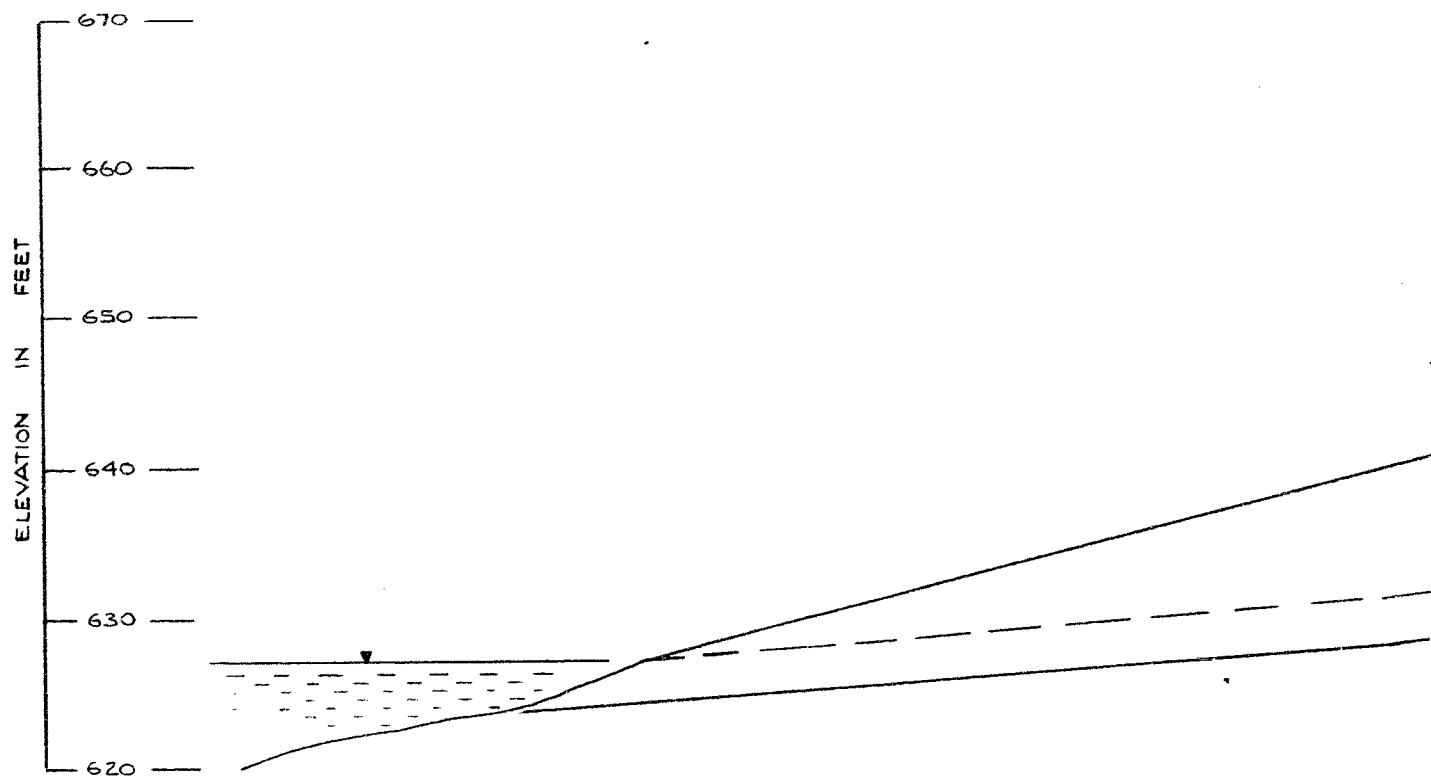
SCALE: 1" TO 20'

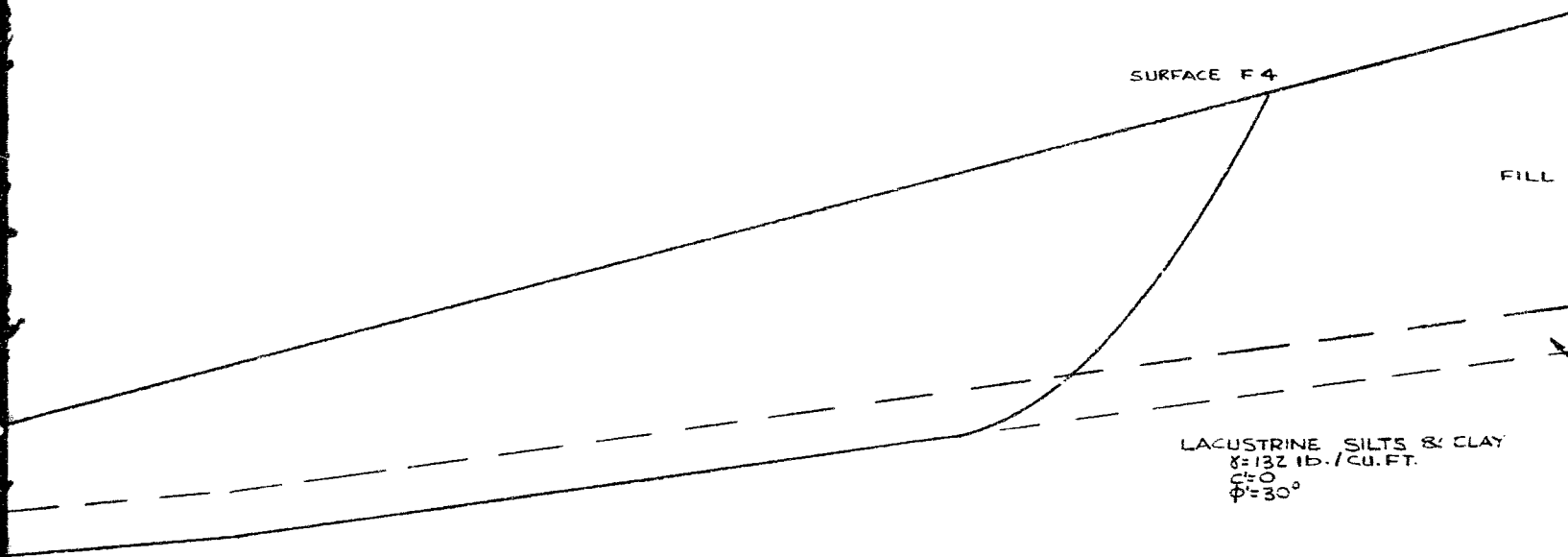
40 P1-69
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Date MAR 4, 1975

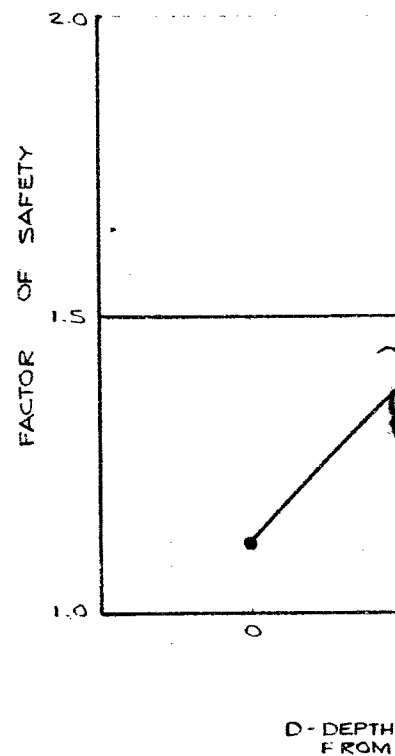
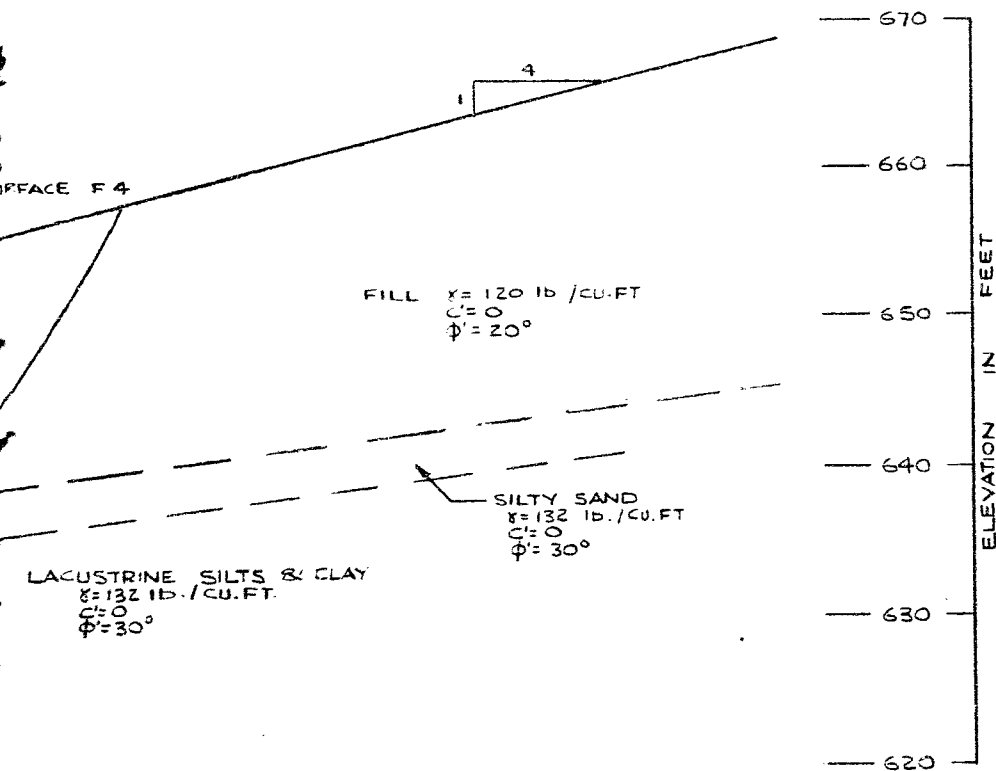
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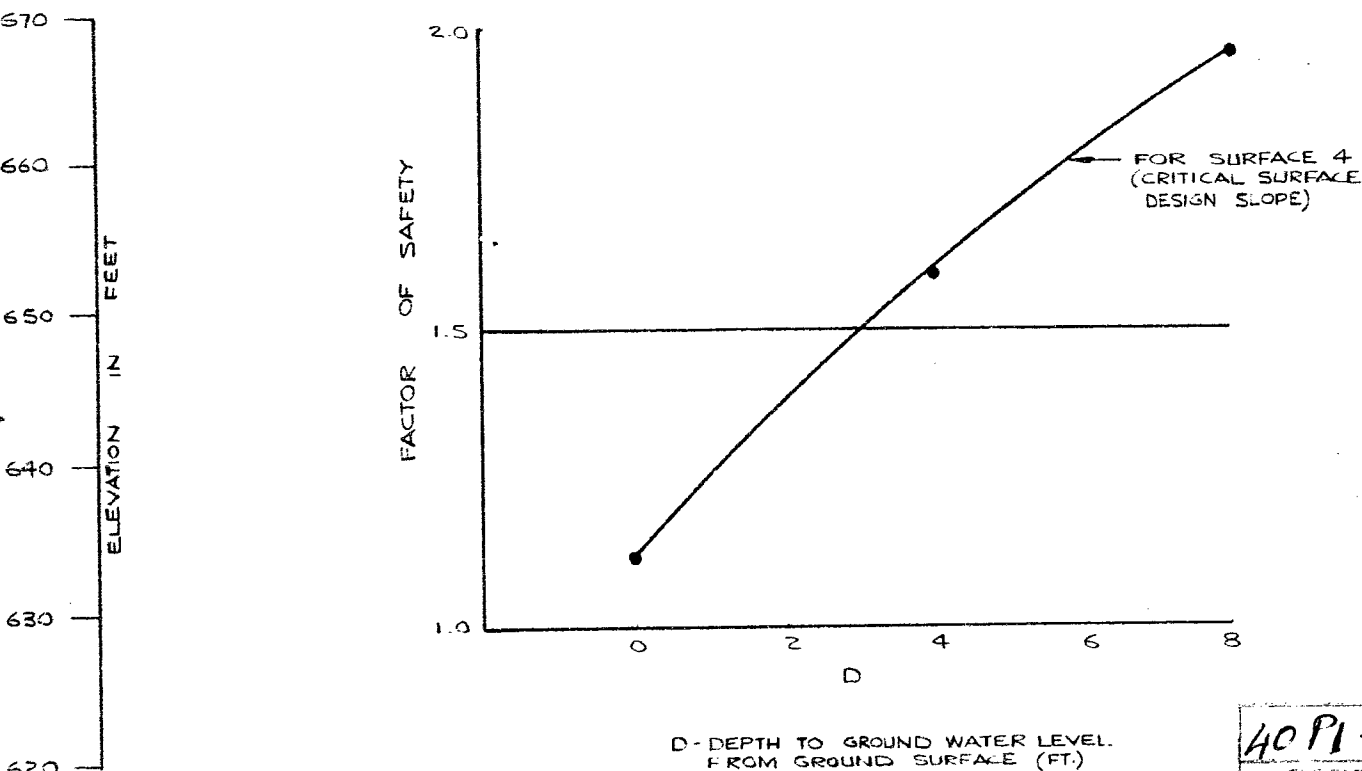




SCALE : 1" TO 10'



PRELIMINARY



40 P1 - 69
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Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

40 P1-69

CHOCRES No.

3 of 3

REPORT

TO

McCORMICK, RANKIN & ASSOCIATES LIMITED
SUBSURFACE INVESTIGATION
PROPOSED COCKSHUTT ROAD BRIDGE
BRANTFORD ONTARIO

STRUCTURAL SITE NO. 1-89

D.T.C. — TORONTO
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May, 1975

741258

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SYNOPSIS

This report details the factual results of a subsurface investigation carried out at the site of the proposed Cockshutt Road bridge over the Grand River in Brantford, Ontario; summarizes the results of our review and assessment of previous slope failures which have occurred at the site; and presents our engineering recommendations for the geotechnical design and construction of the new bridge, including remedial measures to stabilize the existing slope. Comments regarding reported movements of the south abutment of the existing bridge are also provided.

The investigation was carried out in two stages. During the first stage (December 16, 1974 to January 13, 1975) a total of 7 borings were put down at the proposed pier and abutment locations to obtain subsurface information for foundation design purposes. During the second stage of the investigation (March 26 to April 20, 1975), a total of 9 additional borings were put down in the south valley bank area to:

- obtain additional samples of the subsoil
- permit installation of additional piezometers
- permit installation of slope indicator casings
- examine the condition of the south abutment of the existing bridge.

The site is located at the southern edge of the Grand River valley and its most significant feature is the approximately 80 ft. high slopes which form the south bank of the valley. This bank shows considerable evidence of past slope stability failures (ref. Figure 2) and a review of the history of the site indicates that problems with slope movements have been experienced since the mid-1800's. The valley bank consists of thick deposits (in excess of 100 ft.) of normally or near normally consolidated glacio-lacustrine silts and clays; the upper portion of which have been desiccated to a very stiff consistency. Near the slope face, the lacustrine sequence has been disrupted by a failure which reportedly occurred across the proposed bridge alignment in 1957. Across the valley floor area, the glacio-lacustrine silts and clays are overlain in turn by glacio-fluvial sands and gravels and recent alluvial sands and silts. The site is underlain at between elevations 585 and 595 by dolomitic bedrock of the Salina formation. The bedrock is known to contain gypsum layers and inclusions and, locally, solution of these inclusions has resulted in voids in the bedrock. Within the bedrock, groundwater seepage is occurring towards the river valley and within the overburden downward and lateral groundwater seepage is occurring. Groundwater conditions in the overburden are complicated by the

SYNOPSIS

(continued)

presence of the slide debris near the face of the existing slope.

Examination of aerial photographs (ref. Figure 2) indicates the presence of numerous large failure scars along the south valley bank. One such failure reportedly occurred downstream of and across the existing bridge alignment in 1945 and caused an approximately 2 ft. movement of the existing bridge abutment. Another failure occurred immediately upstream of the existing bridge (across the presently proposed alignment) in 1957. Analyses of these failures (particularly the 1957 failure) indicate that they are deep-seated and occur when the bank is steepened by toe erosion to an overall slope of about $1\frac{1}{2}$ to 2 horizontal to 1 vertical. The resulting slide debris slumps to a marginally stable configuration (about 3 horizontal to 1 vertical slope). With time, however, the debris retrogresses through toe erosion and local instability until another deep-seated failure occurs.

To stabilize the present valley bank, we recommend that the existing slopes be flattened to not steeper than 4 horizontal to 1 vertical for a distance of at least 400 ft. upstream and 200 ft. downstream of the proposed bridge, that erosion of the regraded slope be prevented by gabions or the like, and the relief of artesian pressures on the bedrock beneath the toe of the regraded slopes be provided. Even with slope flattening, however, some straining within the slope must be anticipated. Consequently, we recommend that the performance of the regraded slopes be monitored and that provision be made to accommodate minor movement of the proposed south pier and abutment.

The proposed bridge may be founded on small displacement steel 'H' piles driven to practical refusal on or in the bedrock. Relatively heavy 'H' sections (12 BP 74 or heavier section having a reinforced lower section fitted with a drive shoe) driven to a final set of 20 blows/in. using a hammer delivering at least 40,000 ft.-lb of energy per blow may be designed using an allowable load of 75 tons per pile. Considering the possible presence of open voids in the rock, however, we recommend that, as a precaution, provision be made for over-driving the piles in the rock after the design set has been achieved in an attempt to shatter the roof of any shallow cavity and for proof-drilling of the rock after the piles have been driven and retapped to the design set.

Alternatively, the piers and abutments may be founded on pre-bored cast-in-place concrete piles socketted at least $1\frac{1}{2}$ pile diameters into sound rock (i.e. below any weathered zone

SYNOPSIS

(continued)

or open voids or fractures). Provided the sockets are inspected and proof-drilled, such piles may be designed using an allowable load of 75 tons per square foot of cross-sectional shaft area.

The results of this investigation indicate that, in situ, the material in the proposed south roadway approach cut (through the crest of the south valley bank) is too wet to permit proper compaction. However, the material could be used for the construction of the approximately 30 ft. (maximum) high north roadway approach embankment across the existing flood plain if the material is air-dried in the field to a water content within about 2 per cent of the optimum water content for compaction. Although no foundation stability problems are anticipated for approach embankment side slopes as steep as 2 horizontal to 1 vertical, somewhat flatter slopes may be required for erosion protection purposes.

A boring put down through the south abutment of the existing bridge indicates that, if the abutment is broken and sheared, the most probable location of this break is at about elevation 637, about 16 ft. below the top of the abutment. Readings taken in the slope indicator casings installed beside and behind the abutment do not indicate any overall slope movements; in fact, the casing installed beside the abutment indicates movement into rather than out of the slope. It is postulated that the reported abutment movements may be due to internal expansion and contraction of the bridge structure itself rather than to slope instability or lateral earth pressures on the abutment.

1.0 INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by McCormick, Rankin & Associates Limited, Consulting Engineers to The City of Brantford and the Brantford Suburban Roads Commission, to carry out a final subsurface investigation at the site of a proposed replacement structure to carry the existing Cockshutt Road over the Grand River in Brantford, Ontario.

Originally, the purpose of this investigation was to determine the subsurface conditions at each of the proposed pier and abutment locations and, based on these conditions, to provide engineering recommendations for the geotechnical design and construction of the proposed bridge crossing, including an assessment of the overall stability of the present south valley bank. The preliminary results of this stage of the investigation are presented in our interim report to McCormick, Rankin & Associates Limited (Interim report 741258, "Subsurface Investigation, Proposed Cockshutt Road Bridge, Grand River Crossing, Brantford, Ontario", dated March, 1975).

As the investigation progressed, we learned that movements of the existing south valley bank and the south abutment of the existing bridge were being monitored by J.D. Lee Engineering Limited. The results of this monitoring reportedly indicated that erratic but progressive movements of at least the face of the existing south valley bank were occurring and, more significantly, that movement (both translational and rotational) of the existing abutment was occurring. Further, during this period, we were provided with considerable additional and more detailed information

concerning the history of valley bank failures at the site. Based on this additional information, and the results of the first stage work, it was agreed with the Consulting Engineers, the City of Brantford, the Brantford Suburban Roads Commission and the Ministry of Transportation and Communications that additional investigation and monitoring of the reported slope movements was required. This report details the factual results of the first stage investigation, and provides engineering recommendations for the final design and construction of the proposed bridge crossing.

The results of a preliminary subsurface investigation consisting of two boreholes put down through the existing south valley bank along possible alternative alignments for the proposed bridge in September, 1973 are presented in our report "Preliminary Subsurface Investigation, Proposed Replacement Structure, County Road No. 4, Brant County, Ontario" to McCormick, Rankin & Associates Limited (our report 73154, dated October, 1973).

2.0 DESCRIPTION OF PROJECT

It is understood that the proposed replacement structure is to be located approximately parallel to and about 60 ft. (centreline to centreline) west of (upstream of) the existing bridge. The new bridge is to have an overall length of about 780 ft. and is to be a five-span structure (pre-cast beams) with a 160 ft. long centre span, 150 ft. long intermediate spans and about 125 ft. long end spans. The proposed deck level varies from about elevation 679 at the south abutment to about elevation 669 at the north abutment. Thus, the new bridge deck will be at or only slightly above the present bridge level.

In the south roadway approach, the existing cut through the crest of the valley bank is to be widened to the west. This will involve excavation to a depth of some 20 to 25 ft. and it is presently proposed to use the excavated material for construction of the north roadway approach embankment.

The north roadway approach will require an approximately 2,200 ft. long embankment across the river flood-plain (ground surface generally between about elevations 640 and 650). This embankment will have a maximum height of about 30 ft. above the existing flood plain and will encroach upon the existing approach embankment. The existing embankment is to be incorporated into the new embankment to provide relatively flat downstream side slopes for erosion protection purposes.

It is understood that the design flood level for the Grand River is at about elevation 655. Thus, during peak flood periods, the northern portion of the approach embankment will be overtopped.

3.0 HISTORY OF SITE

The following history of the site is based on discussions with Mr. C.G. Spencer, Brant County Engineer, and with J.D. Lee Engineering Limited; examination of old reports, correspondence and drawings provided to us by Mr. Spencer; examination of aerial photographs taken at various times since 1948; and, of great significance, a paper by Dr. R.F. Legget entitled "Stabilizing the South Pier of the Cockshutt Bridge" published in the April, 1946 edition of Engineering and Contract Record.

3.1 History of Bridge Structures

To the best of our knowledge, the present bridge is the fourth structure to be constructed on approximately the same alignment. The first two structures (built in the mid to late 1800's) were low level structures with inclined access roads along the face of the approximately 85 ft. high south valley bank (side-hill cut). Although little is known of these structures, some problems were experienced probably due to floods and movements of the south valley bank.

The third bridge (built about the turn of the century) was a high level steel truss structure with a deck at about elevation 670. The centreline of the structure coincides closely with the centreline of the presently proposed bridge. Access to the south end of the bridge was via an approximately 30 ft. deep cut through the crest of the south valley bank and apparently a local approach fill on the slope face. Remnants of the approach cut can still be seen at the site. Evidence suggests that this bridge suffered structural distress due to, among other things, large longitudinal compressive stress induced by lateral (northward) movement of the south piers and abutment (located within the south valley slope).

The fourth or present bridge was constructed about 1932. Due to the history of problems with previous bridges a fairly extensive search was reportedly carried out for an alternative site (a detailed topographical map of this entire stretch of the river was reportedly prepared in 1931 but no copies have been found to date). It is reported that no significantly better site could be located and, considering land acquisition and road re-alignment costs, it was decided to build the bridge about 50 ft. east of the then existing steel truss bridge.

The existing bridge (deck at about elevation 672) is a multi-span reinforced concrete arch structure with pile supported piers and mass concrete abutments which extend down to bedrock. The south abutment is about 70 ft. high and except for the upper part, was reportedly constructed in a series of about $7\frac{1}{2}$ ft. high sections; 4 ft. long sections of steel rail being installed across each construction joint (cold joint) as shear connectors. It is understood that during construction of the south abutment, bedrock was encountered 6 ft. deeper than anticipated and consequently the excavation had to be taken below the tips of the steel sheet piling used to support the sides of the excavation.

The existing bridge has suffered serious structural distress due in part to horizontal displacement of the south abutment. It is reported that during a slope failure in 1945 (see also section 3.2) the top of the south abutment moved about $1\frac{1}{2}$ to 2 ft. relative to the bridge seat. Based on field observations, the investigators apparently concluded that the abutment broke and rotated about a construction joint at about elevation 612, that is about 26 ft. above the base of the abutment.

Following the movement, a collar was installed at the top of the abutment to fix the deck structure to the abutment. Subsequently, it is understood that the entire bridge structure has been "pushed" about 6 in. to the north. Thus the total abutment movement to date is about 30 in. or more.

Recent monitoring by J.D. Lee Engineering Limited reportedly indicates that the abutment (and deck) is still moving. The abutment had reportedly been rotating until last fall (1974) when the abutment reportedly straightened slightly due to shear displacement across the postulated break at about elevation 612.

3.2 History of Slope Failures

It is understood that a major slope failure occurred immediately downstream (east) of the existing bridge in 1945 and that it was this failure that caused the sudden, massive movement of the south abutment (other reports suggest that the abutment had "crept" about 6 in. prior to the massive movement). At or before the failure, the overall valley slope was apparently in the order of $1\frac{1}{2}$ horizontal to 1 vertical (approximately 80 to 90 ft. overall slope height). Although little factual information is available concerning this failure, it is understood that some movement was noted before the failure (this is consistent with pre-failure "creep" of the abutment) and that some slope re-grading (unloading) was carried out. Additional slope grading was carried out after the failure. Although no details of pre-slide or post-slide geometry are available it is understood that the ground movement was typical of a deep seated, rotational instability.

A second major slide occurred immediately upstream (west) of the existing bridge in 1957. Here, there is reasonably good pre-slide and post-slide geometry (including 1948 and 1964 aerial photographs) and a meaningful "back analysis" of the slide can be carried out. The slide apparently did not seriously affect the existing bridge but did remove part of the roadway approach. The new (or presently proposed) bridge crosses this old failure zone.

It should be noted that the aerial photographs indicate the presence of numerous other (and presumably older) slides along this stretch of the river. At least one of these slides (located a few hundred feet upstream of the bridge) appears to have extended several hundred feet back from the crest of the valley bank.

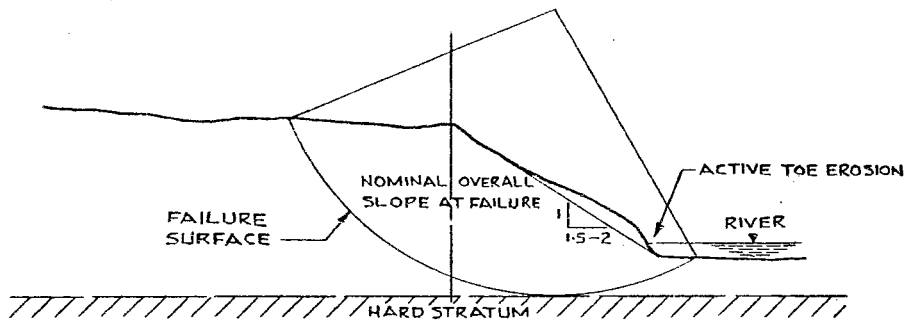
It should also be noted that the present site is located about a mile downstream of Tutela Heights where there is a long history of slope failures.

3.3 Probable Failure Mechanism

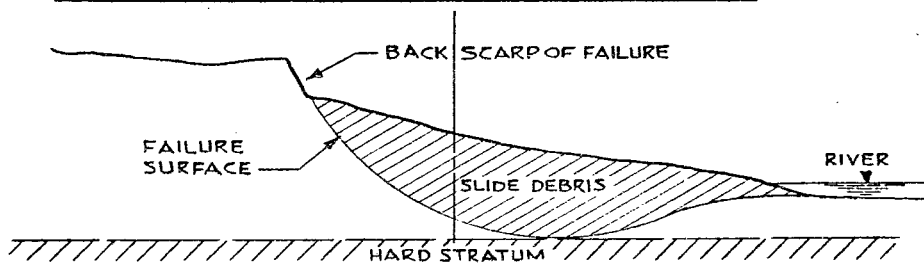
Based on our review of the history of past failures in the vicinity of the site, the following failure mechanism is postulated; the various stages of bank retrogression are illustrated on the following page:

- i) As a result of toe erosion by the river and local instability, the south valley bank becomes steepened to an overall slope in the order of $1\frac{1}{2}$ to 2 horizontal to 1 vertical.
- ii) The resulting over-steepened, approximately 80 ft. high slope fails along a more or less conventional deep-seated circular or non-circular failure surface (for convenience a circular arc failure is illustrated). Such failures are probably relatively sudden (although warning of the impending failure in the form of accelerating creep and the appearance of tension crack will probably be present) and are typically large. Examination of aerial photographs suggest that the failures may extend as much as 600 to 800 ft. back from the crest of the valley bank.
- iii) The failure proceeds until the slide debris reaches a stable configuration. Aerial photographs suggest that the debris can vary from as steep as about 3 to 4 horizontal to 1 vertical to a relatively flat plateau

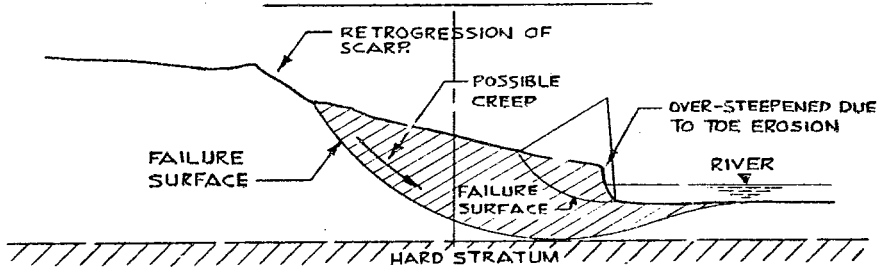
ILLUSTRATION OF POSTULATED FAILURE MECHANISM SOUTH VALLEY WALL



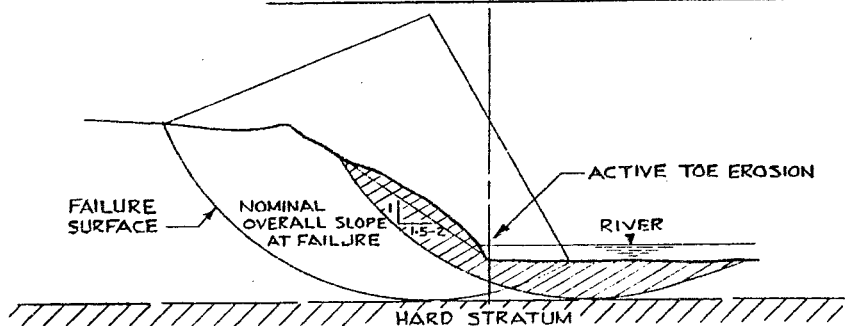
MAJOR FAILURE OF OVER-STEEPENED SLOPE



POST FAILURE CONDITION



RETROGRESSION OF SLOPE DEBRIS



SECOND MAJOR FAILURE OF OVER-STEEPENED SLOPE

only a few feet above the river level (the flatter debris being generally associated with the larger slides). As a result of the failure, the river is generally diverted from its original position.

- iv) As a result of erosion by the river, the toe of the slide debris becomes oversteepened. This oversteepening probably results in local instability of the toe of the debris slope, particularly as the height of the eroded face increases, and may result in "creep" or slumping along the original failure surface.
- v) As a result of progressive toe erosion/instability the valley bank becomes steepened until the overall slope decreases to approximately $1\frac{1}{2}$ to 2 horizontal to 1 vertical at which time a second deep-seated major instability occurs.

It should be noted that the actual overall slope at which major instability occurs and the extent of the failure is dependent upon the engineering properties of the subsoil and the prevailing groundwater conditions at the location of the failure. Both of these conditions will vary significantly along the river bank.

4.0 SITE AND GEOLOGY

4.1 Description of Present Site

The site of the Cockshutt Road Bridge is located on the upstream side of the "neck" of a major meander of the Grand River referred to as "The Oxbow" (see Figure 1). As a result, approximately $8\frac{1}{2}$ river miles downstream of the site

the Grand River passes within about 2,000 ft. south of the site. An over-view of the general area would suggest that, left unattended, the river will, with time erode through this "neck" and eliminate the oxbow.

The Grand River valley is some 1 to 2 miles wide and includes a large portion of the City of Brantford. The northern portion of the valley is characterized by a very flat overall slope, averaging less than about 1 per cent, but the valley is bounded on the south (the site of the presently proposed bridge) by the relatively steep and potentially unstable slopes discussed previously.

At the proposed bridge crossing, the existing valley bank is characterized by approximately 10 to 15 ft. high relatively steep (near vertical upstream of the bridge site), lower slopes, approximately 40 to 50 ft. high intermediate slopes (slopes between about 3 and 5 horizontal to 1 vertical) and about 20 to 30 ft. high relatively steep upper slopes (typical 1 to 1½ horizontal to 1 vertical). These three slopes probably represent the erosion steepened toe, the face of old slide debris and the back-scarp of old failures respectively (see also Section 3.3).

A fairly recent (1972) aerial photograph showing the old failure scarps along the stretch of the river presently under consideration is given on Figure 2.

4.2 Geology of Area

Based on available geological information, the site is located near the western limit of the physiographic region known as the Haldimand Clay Plain, an area characterized by thick deposits of glacio-lacustrine clays and silts laid down in glacial melt-water lakes which occupied the present Lake Erie basin during the retreat of the Wisconsin glaciers.

The initial retreat of the glaciers from the Brantford area probably occurred during the latter stages of Cary glacial substage. During this period the site was inundated by glacial Lake Arkona within which glacio-lacustrine deposition could be anticipated. Subsequently, the glacial ice sheet re-advanced to about the Brantford area. Although the exact position of the ice front is not known, some re-working of the Lake Arkona deposits and deposition of glacial till in the Brantford area probably occurred during this advance. During the final retreat of the glacier from the area, the site was inundated by at least one and possibly several glacial lakes (notably glacial Lakes Wayne and Warren).

As glacial retreat continued, the water level in the Lake Erie basin lowered and melt-water streams from the retreating glacier eroded the present Grand River valley through the lacustrine deposits. Subsequent fluvial deposition during the latter stages of the outwash stream and/or recent alluvial deposition by the post-glacial Grand River may be anticipated across the floor of the valley.

As the site is located very close to the limit of the glacial lake pondings; irregularly stratified and relatively coarse grained (silty) deposits associated with shallow water deposition should be anticipated, particularly at the higher elevations, rather than uniformly layered (varved) highly plastic clays associated with deep water deposition.

5.0 FIELD PROCEDURES

As previously noted (see Section 1.0) the field work for this investigation was carried out in two stages. During the first stage of the investigation, a total of seven (7) detailed sampled borings were put down at the proposed pier and abutment locations. During the second

stage of the investigation, additional detailed sampled borings were put down through and behind the existing south valley bank, within the proposed south approach cut/borrow area and through the south abutment of the existing bridge. In addition, two slope indicator casings were installed to permit monitoring of movements within the south valley bank. Details of the drilling and sampling operations, the results of in situ testing and piezometer/standpipe/slope indicator casing installations are given on the Record of Boreholes sheets following the text of this report.

The field work for both stages of this investigation was supervised throughout by members of our engineering staff who located the borings in the field, directed the drilling and sampling operations, carried out the in situ testing, and cared for the samples obtained. All of the samples obtained during this investigation were brought to our laboratory for detailed examination and testing.

The borehole locations and the ground surface elevation at each borehole were supplied to us by McCormick, Rankin & Associates Limited. It is understood that the elevations given in this report are referred to Geodetic datum.

5.1 Stage I Investigation

The field work for the first stage of the investigation was carried out between December 16, 1974 and January 13, 1975. During this period a total of seven (7) borings (numbers 101 to 107, inclusive) were put down at the locations shown on Figure 3. The on-shore borings were advancing to depths of

between about 40 and 100 ft. below existing ground surface using a track-mounted power auger supplied and operated by a specialist drilling contractor from Toronto. Borehole 103, put down within the existing river channel, was advanced from a barrel raft using a light diamond machine drillrig.

In each boring, standard penetration tests were carried out at a maximum of 5 ft. intervals of depth and samples of the overburden were obtained using conventional split spoon sampling equipment. Within cohesive soil zones relatively undisturbed samples of the subsoil were obtained using 3 inch diameter thin-walled Shelby tubes and in situ vane tests were carried out between samples to determine the undrained shear strength of the soil.

Groundwater level observations were made in the open borings during the drilling operations and, following the completion of drilling, a standpipe and/or piezometer was sealed into selected holes to permit monitoring of the stabilized groundwater level(s) across the site.

5.2 Stage II Investigation

The field work for the second stage of the investigation was carried out between March 26, and April 20, 1975 and on April 28, 1975. During this period, a total of nine (9) borings (numbered 201 to 209, inclusive) were put down in the south valley bank area to:

- obtain additional samples of the subsoil
- permit installation of additional piezometers
- permit installation of slope indicator casings
- examine the conditions of the south abutment of the existing bridge

The locations of these additional borings are shown on Figure 3. The borings were advanced to depths of between about 45 and 135 ft. below existing grade using two (2) track-mounted power auger drillrigs supplied by specialist drilling contractors from Toronto. Both of these drillrigs were fully equipped for drilling (hollow stem augers) and sampling in the overburden and for rotary core drilling in the bedrock and one of the drillrigs (a heavy duty diesel powered machine) was also equipped for rotary mud drilling and advancing larger diameter cased borings for the installation of slope indicator casings.

5.2.1 Proposed Approach Cut/Borrow Area

Four (4) of the borings (numbered 206 to 209, inclusive) were put down in the area of the proposed approach cut through the crest of the south valley bank to determine the suitability of the material for use as compacted fill. In three (3) of the borings (boreholes 206, 207 and 209) standard penetration tests were carried out at 2½ to 5 ft. intervals of depth and samples of the overburden were obtained using conventional 1½ in. I.D. split spoon sampling equipment. These relatively "disturbed" samples were subsequently tested to determine the in situ water content of the proposed borrow material. Two of these borings (numbered 206 and 207) were terminated at a depth of about 45 ft. below the present ground surface while the third boring (numbered 209) was sampled to bedrock to obtain a complete soil profile behind the south valley slope (see also section 5.2.3). Following completion of each of boreholes 206 and 207, piezometers were sealed into the open holes at/or below the proposed cut grade to permit monitoring of the groundwater level(s).

The fourth boring (borehole 208) was put down to a depth of about 30 ft. to obtain bulk auger samples for laboratory compaction testing purposes.

5.2.2 Investigation of Existing Abutment Movements

To investigate the reported movements of the existing south valley bank and the south abutment of the existing bridge, two slope indicator casings (boreholes 202 and 205) and additional piezometers (borehole 203) were installed through the existing slope and the existing abutment was rotary core drilled for its full height (borehole 204).

Borehole 202 - Slope Indicator Casing No. 1: Slope Indicator Casing No. 1 was installed in line with and about 10 ft. west of the existing bridge abutment to monitor slope (soil) movements which could be causing movement of the abutment. The hole was advanced uncased to a depth of about 69 ft. (6 ft. into bedrock) by rotary tricone drilling (4 in. dia. size) using a heavy mud slurry to support the walls of the hole. During drilling in the bedrock, serious loss of drilling mud into an open void or fracture occurred and it became impossible to maintain a "mud" level above a depth of about 14 ft. below ground surface. This loss of drilling mud together with a forced delay due to unseasonably cold weather resulted in collapse of the uncased hole. Repeated attempts to re-establish the hole with "mud" were unsuccessful and it was eventually necessary to case the hole with "H" casing. The slope indicator casing was successfully installed through the drill casing on April 8, 1975. However, due to unavoidable caving of the walls of the hole during withdrawal of the drill casing, it was not possible to completely grout the slope indicator casing in place.

The initial deflected shape of the slope indicator casing was established on April 10, 1975 and is shown on the Record of Borehole sheet for borehole 202. As indicated on the borehole log, the casing was relatively straight but sloped down towards the south-east at a slope of about 3 degrees.

Borehole 205 - Slope Indicator Casing No. 2: To act as a check on the first slope indicator casing and to provide more complete data concerning the magnitude and direction of possible slope movements, a second slope indicator casing (borehole 205) was installed from the west shoulder of the existing road about 12 ft. south of the end of the existing bridge. Due to the problems experienced with caving during installation of the first slope indicator, it was decided to case the overburden portion of the second installation. Consequently, an initial pilot hole was advanced (augered) to a depth of about 85 ft. below ground surface and an "H" casing driven to a depth of 80 ft. The hole was then advanced by rotary tricone drilling to a depth of about 95 ft. (some 12 ft. into bedrock). Again, loss of drilling fluid into an open void or fracture in the rock was experienced. Slope Indicator Casing No. 2 was successfully installed and grouted (during withdrawal of the drill casing) on April 14 - 15, 1975 and an initial reading was obtained on April 16, 1975. The initial deflected shape of the casing is shown in the Record of Borehole sheet for borehole 205. This casing was installed relatively straight but slopes down towards the north at an average slope of about 1 degree.

Borehole 203 - Piezometer Installations: To permit monitoring of the groundwater level(s) in the slope in the vicinity of the south abutment of the existing bridge, borehole 203 was put down adjacent to Slope Indicator Casing No. 1 (borehole 202). Initially, it was proposed to advance this boring unsampled into bedrock to permit installation of piezometers. During the course of the field work, however, it was decided to obtain continuous samples through the zone of reported abutment movement (about elevation 612) to attempt to locate the plane of movement of the 1945 slide. Details of the sampling operations and piezometer installations are given on the Record of Borehole sheet for borehole 203 and a detailed water content profile obtained from the relatively undisturbed samples is given on Figure 9.

Borehole 204 - Coring of Existing Abutment: To determine the condition of the concrete in the south abutment of the existing bridge and, in particular, to attempt to determine if the abutment is broken and sheared, borehole 204 was drilled through the full height of the abutment and into the underlying bedrock. This hole was put down from the bridge deck and was advanced through the abutment by rotary core drilling in "Nx" core size ('L' series barrel). In general, no serious problems were encountered in advancing this hole to a depth of about 95 ft. (about 5 ft. into bedrock). However, a near vertical, ½ in. thick steel plate was encountered at a depth of about 84 ft. and serious damage to the diamond studded core bits occurred during drilling through this obstruction.

It should be noted that complete loss of drillwater occurred at a depth of about 40 ft. and that circulation was not re-established below this depth. As examination of the core indicated the presence of possible "open" fractures

below 30 ft., (see Appendix I to this report) it was agreed to carry out pressure packer tests in selected portions of the boring (the water "take" during packer testing being an indirect indication of the "openness" of the fractures).

Packer testing was initiated on April 19, 1974. However, during testing between depths of 65 and 70 ft., the packer assembly became wedged in the hole and subsequently broke at a depth of about 70 ft. Attempts to recover the broken packer on April 19, 1975 were unsuccessful. On April 28, 1975 a drillrig returned to the site, to attempt to recover the broken packer assembly. These attempts proved unsuccessful and it was necessary to abandon about 25 ft. of drillrod in the hole. The remainder of the packer tests were carried out on April 28, 1975. The results of these tests are summarized on the Record of Borehole sheet for borehole 204.

5.2.3 Additional Borings, South Valley Bank Area

To obtain additional information regarding the engineering properties of the subsoil and the groundwater conditions in and behind the existing south valley bank, one detailed sampled boring (borehole 201) was put down at the toe of the slope close to the existing river and one of the borings put down in the proposed south approach cut/borrow area (borehole 209 - see also Section 5.2.1) was sampled to and core drilled into, the bedrock (a total depth of about 135 ft.). In each of these borings, standard penetration tests were carried out at 2½ to 5 ft. intervals of depth and samples of the subsoil were obtained using conventional 1½ in. I.D. split spoon sampling equipment. In softer cohesive portions of the overburden, the split spoon samples were augmented with 2 in. and 2 7/8 in. dia. thin walled Shelby tube samples and, where possible, the undrained shear strength of the subsoil was determined using in situ vane

testing equipment. Bedrock was proved by rotary core drilling in 'Bx' core size ('L' series barrel). Following completion of each boring, piezometers were sealed into the open hole to permit monitoring of the groundwater level(s) in the bedrock and the overburden. It should be noted that excessive unavoidable caving of the lower portion of the overburden in borehole 209 prevented the installation of additional piezometers in this boring.

6.0 SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the boreholes put down during both stages of this investigation is shown on the Record of Borehole sheets following the text of this report. Also shown are Record of Borehole sheets for the two borings put down during the preliminary (1973) investigation (boreholes 1 and 2 - for location see Figure 3). It should be noted that minor stratigraphic changes have been made in the logs for the preliminary borings (originally presented in our report 73154, dated October, 1973) and the logs for the first stage borings (originally presented in preliminary form in our interim report 741258, dated March, 1975). These modifications are restricted to geological interpretation of the boring results and are based on additional information which became available during the second stage of the investigation.

A stratigraphic section showing the inferred subsurface conditions along the centreline of the proposed reconstruction is shown on Figure 4 and an illustrative section showing the inferred subsurface conditions through the 1957 slide is

shown on Figure 5. A summary of the engineering properties of the subsoil forming the south bank of the valley is shown on Figure 6 and the results of laboratory testing carried out on representative samples of the subsoil are given on the Record of Boreholes sheets and on Figures 7 to 20, inclusive.

The results of a previous investigation carried out by others in connection with a proposed roadway re-alignment immediately south of the presently proposed bridge crossing were provided to us by the Brantford Suburban Roads Commission (report by Associated Geotechnical Services Ltd. entitled "Report on Soil Investigation for the Proposed Alignment of Cockshutt Road, County of Brant" dated December 12, 1962). Undrained shear strength data and laboratory compaction test data presented in this previous report for borings put down in the vicinity of the presently proposed bridge site (boreholes 1, 1a, 2, 3 and 4) have been considered in preparation of this report.

6.1 Subsoil Conditions

6.1.1 South Bank of Grand River

With the exception of borehole 1 put down during the preliminary (1973) investigation, all of the boreholes put down through the flank of the south valley slope encountered varying types and depths of fill which was probably placed as the result of the construction activities which have taken place at the site in connection with the several former bridges and/or regrading of the slope after the 1957 slide. In borehole 205, 15 ft. of gravel in a matrix of sandy silt was encountered. As much as 18 ft. of fill comprised of reworked clayey silt occurred in boreholes 202 and 203; some or all of this fill possibly having been

placed as backfill for the excavation of the adjacent bridge abutment. In boreholes 101, 102 and 201, up to 25 ft. of reworked clayey silt fill was encountered below ground surface.

The major stratum encountered in the south bank area is a layered deposit of silt, clayey silt and silty clay. The sequence of the layers is extremely irregular. Individual layers range in thickness from several inches to a small fraction of an inch. Although the layering in undisturbed portions of the stratum is essentially horizontal, individual layers are distorted and convoluted. Local shear planes also occur across the layers, possibly the result of shear failures during deposition but, in some cases, also the possible result of the regional bank instability. The trend of composition of the stratum is from predominately silt or clayey silt near the surface to stratified clayey silt and silty clay with depth. A small gravel content generally also occurs at depth.

In boreholes 206 to 209, inclusive, which were put down behind the crest of the valley slope, the upper 10 to 15 ft. of the stratum is weathered to a brown color and consists of layered clayey silt and sandy silt with numerous thin fine sand partings. The maximum thickness of the lacustrine stratum was encountered in borehole 209, where it was found to extend to a depth of 115 ft.

Some surficial weathering of the material has also occurred where the deposit is at or near the face of the slope of the south bank of the river. Thus in boreholes 101 and 102, the upper 3 to 6 feet of the stratum beneath the fill is brown sandy silt with layers of clayey silt and silty sand. In borehole 201, two such brown sandy silt

layers were encountered, one immediately beneath the surficial fill and the other at a depth of 21 ft.

A detailed examination of the layered clayey silt stratum at borehole 201 revealed that the bedding planes of the soil which occurred between the two sandy silt layers were inclined at up to 45 degrees to the horizontal. Further, the bedding planes in the layered clayey silt and silty clay encountered in borehole 201 were inclined at up to 35 degrees to the horizontal to a depth at about 40 ft. Photographs of these inclined layers are shown on Figure 8. It is concluded that the soil in this borehole (which is located adjacent to the present river bank) consists of old slide debris to at least elevation 600.

A detailed examination of the series of continuous, undisturbed samples which were obtained in borehole 203 between depths of 30 and 41.5 ft. (elevation 618.5 to 607) showed that the layering of these samples was approximately horizontal. Based on these observations we conclude that the soil in borehole 203 has not been disturbed by the adjacent slide movements. It should also be noted that the 11.5 ft. interval which was examined in detail in borehole 203 was chosen to correspond to the depth at which the reported failure of the adjacent south bridge abutment had occurred in 1945. No evidence of a failure plane was noted in the soil samples, and the detailed water content profile which was taken along the samples (as plotted on Figure 9) shows no abnormal trend in water content, which would normally be associated with a major slip surface.

As indicated on Figure 7 the plasticity of the material is typical for a glacial lake clay and, excluding cohesionless silt layers, the material in individual layers ranges from a clayey silt of low plasticity to a silty clay of high plasticity. The in situ water content of the layers (see Figure 6) generally ranges between about 20 and 40 per cent and the liquidity index of the materials is generally less than about 0.5. However, in occasional layers a liquidity index of as high as about 1 was determined.

As shown on Figure 6, the results of in situ vane tests carried out during the present and previous investigations and undrained triaxial and direct shear tests carried out on relatively undisturbed samples indicated that, below about elevation 640 to 670, the lacustrine deposit tends to be near normally consolidated. Within this lower zone, however, local, more heavily pre-consolidated (stiffer) zones occur (e.g. borehole 1). The minimum undrained shear strength occurs between about elevations 620 and 660 where strengths as low as about 900 to 1,000 lb/sq. ft. but more typically about 1,000 to 1,500 lb/sq. ft. were recorded. The average undrained shear strength below about elevation 670 (excluding the local stiffer zones) is about 1,200 to 1,800 lb/sq. ft. and the average remoulded shear strength in this zone is about 500 lb/sq. ft. (sensitivity of about 2 to 4).

Above about elevation 640 to 670, the deposit has been slightly over-consolidated, by desiccation, and is stiffer than the non-desiccated material. Above about elevation 670, the undrained shear strength is consistently greater than about 2,000 lb/sq. ft.

Excluding the lower portion of borehole 1 which appears to be a local depositional feature (possibly the result of a floating ice mass) the occasional higher vane shear strengths probably reflect the presence of cohesionless silt layers within the otherwise cohesive deposit.

To determine the effective shear strength parameters of the lacustrine complex, three series of consolidated drained direct shear tests were carried out. The results of these individual test series are summarized on Figures 10, 11, and 12. As shown on these figures, the individual test results are extremely variable and indicate that the deposit varies from an essentially cohesionless material ($c' = 0$) with an effective or drained angle of shearing resistance, ϕ' , varying from about 28 degrees to 43 degrees, to a material having an apparent cohesion, c' , of about 200 lb/sq.ft. and an effective angle of shearing resistance, ϕ' , of about 22 degrees.

Considering the highly contorted and irregularly layered structure of this deposit, it is questionable whether the results of an individual laboratory test carried out on an approximately 2 inch square specimen are truly representative of the average shear strength parameters of the deposit (post-failure examination of some of the test specimens indicated the presence of 2 and sometimes 3 different soil materials across the failure plane). However, if all of the individual test results are considered together (see Figure 13), it appears that, on the average, the deposit will behave as an essentially cohesionless material ($c' = 0$) and, although the effective angle of the shearing resistance of individual zones or layers could range from about 25 to 43 degrees, the average minimum effective angle of shearing resistance, ϕ' , of the layered silt and clay complex is about 29 degrees.

The results of residual strength tests carried out on sample 17, borehole 101, (see Figure 11) indicate that the residual shear strength of the material is about 90 per cent of the available peak strength.

In borehole 209, a 12 ft. thick stratum of hard, thinly bedded clayey silt was encountered below a depth of 115 ft. and overlying bedrock. This stratum is similar to that encountered at depth in some of the boreholes in the valley floor area (see Section 6.1.2). The results of standard penetration tests, which gave 'N' values in excess of 85 blows/ft., indicate that the stratum has been consolidated under glacial ice pressure in the geologic past.

6.1.2 Valley Floor Area

Excluding the existing roadway embankment fill and local topsoil deposits, boreholes 104 to 107 inclusive, which were put down across the valley flood plain, encountered a surficial silty sand deposit which is probably of recent alluvial origin. Gradation curves of typical samples of this deposit are given on Figure 16. Based on the results of standard penetration tests which gave 'N' values ranging between 4 and 12 blows/ft. the silty sand is in a generally loose state of packing.

Underlying the recent alluvium and the existing river channel (borehole 103), the borings encountered some 5 to 15 feet of sand and gravel. As indicated on Figure 17 (gradation curves) the sand and gravel deposit is relatively clean (silt content of less than about 10 per cent). Based on these gradation curves and field observations, the sand and gravel is relatively pervious with an estimated coefficient

of permeability in the order of 10^{-1} to 10^{-2} centimetres per second. The results of standard penetration tests which gave 'N' values ranging between about 12 and 35 blows/ft. indicate the sand and gravel deposit is in a compact to dense state of packing.

At the borehole locations, the sand and gravel deposit is underlain at between elevations 613 and 628 by the irregularly layered silt, clayey silt and silty clay stratum which was previously discussed in detail for the south valley bank area. The stratum has a slight gravel content with depth. Based on the results of field vane tests and undrained triaxial compression tests carried out on relatively undisturbed samples of the layered clays and silts, the undrained shear strength of the deposit varies between about 1,500 and greater than 3,000 lb/sq.ft.

In boreholes 104, 105 and 106, the layered clayey silt and silty clay is underlain at a depth of about 30 to 40 feet below ground surface by a deposit of irregularly layered silts, clayey silts and silty clays which is generally similar to the overlying lacustrine deposit. However, this lower deposit is apparently of earlier geological origin (see section 4.2) and has been heavily pre-consolidated by glacial over-ride. As a result of this over-consolidation the consistency of the lower lacustrine deposit is hard ('N' values generally in excess of 50 blows/ft.).

6.1.3 Proposed Borrow Area

As previously noted (see section 2.0) it is presently proposed to utilize the material from the south roadway approach cut as borrow for the construction of the north approach embankment. To determine the suitability of this

material for use as "engineered" fill, four boreholes (numbered 206, 207, 208 and 209) were put down within the proposed approach cut (see section 5.2.1). The results of these borings indicate that the approach cut will be excavated through the upper portion of the extensive lacustrine silt, clayey silt and silty clay deposit which underlies the site (see section 6.1.1). Within the cut area, the upper 10 to 15 ft. of the deposit has been weathered to a brown colour.

To determine the compaction characteristics of the lacustrine material, two standard Proctor compaction tests were carried out on representative bulk samples of the material from depths of 5 and 15 ft. in borehole 208. The results of these tests are presented in Figures 14 and 15 and gave maximum standard Proctor dry densities of 105 and 112 lb/cu.ft. at optimum water contents of about 18 and 15 per cent, respectively. This range of optimum water content agrees with the range of results obtained from Harvard Miniature compaction tests carried out on samples from the same area by Associated Geotechnical Services Limited in 1962.

Natural water contents were obtained on all samples from the upper 20 ft. in boreholes 206, 207 and 209. These show that the natural water content varies generally from about 20 to 28 per cent with an average value of about 22 per cent. The water contents of auger samples taken in borehole 208 have been disregarded because the samples appear to have been affected by surface water encountered while drilling the hole. Typical Atterberg limits on the more plastic portions of samples from the upper 20 ft. in boreholes 206 to 209 gave liquid limits which ranged from 22 to 40 and plasticity indices which varied from 21 to 4. Thus the deposit varies from a clay of medium plasticity to a basically non-plastic silt.

Based on the results of the present and previous investigations, the in situ water content of the proposed borrow material is some 5 to 10 per cent wet of the optimum water content for compaction. Consequently, proper compaction of this material at its natural water content will be extremely difficult, if not impossible.

6.2 Bedrock Conditions

The site is underlain at between about elevations 585 and 595 by relatively flat lying dolomitic bedrock of the Salina formation. Although essentially a dolomite, the bedrock contains numerous interbedded shale layers and some gypsum layers or inclusions. Based on available geological information and our previous experience, it is known that locally the Salina formation can contain voids (either open or infilled) formed by the solution of gypsum layers or inclusions by groundwater. These voids can vary from small vugs to cavities as much as several feet thick.

Borings put down through the south valley slope generally encountered an upper 2 to 3 ft. thick cap of weathered and fractured shale but below this cap the rock is generally fairly sound (R.Q.D. in excess of about 60 per cent). Across the valley floor, the shale cap was not encountered and the rock was found to be moderately fractured throughout the cored depth.

It should be noted that in four of the borings put down into the bedrock during the second stage of the investigation (boreholes 202, 203, 205, and 209), complete loss of drilling fluid (mud slurry or water) occurred into open fractures or

voids in the bedrock (in borehole 202 the void was as much as 6 in. thick). Although it is not considered practical to attempt to define the areal extent of possible voids or open fractures in the bedrock by exploratory borings during the pre-design stage of the project, the results of the borings indicate that the occurrence of relatively extensive infilled or open voids in the bedrock is possible.

6.3 Groundwater Conditions

Based on the results of groundwater level observations in the open borings during drilling operations and water level readings taken in the piezometer and standpipe installations during and following completion of the field work (see Record of Borehole sheets for details) the groundwater conditions across the site are relatively complex.

6.3.1 Valley Floor Area

Across the valley floor area, there appears to be two separate groundwater conditions with a transition occurring through the lacustrine deposits. Because of the pervious nature of the fluvial sand and gravel, the groundwater level in the surficial granular deposits (alluvial silty sand and fluvial sand and gravel) is at or only about 2 to 3 ft. above the river water level. Horizontal seepage towards the river is probably occurring through the sand and gravel.

Water level observations during rotary core drilling in the bedrock in borehole 103 and water level readings taken in piezometers sealed within the bedrock in boreholes 104 and 106 indicate that the piezometric water level in the rock across the valley floor is at about elevation 634 and is artesian with respect to the river water level at the

time of the investigation. This artesian groundwater condition is probably due to lateral seepage of groundwater through the rock from the higher ground outside the river valley.

6.3.2 South Valley Bank Area

Within the south valley bank (i.e. behind the slope face) the subsoil is essentially horizontally layered and seepage is occurring from the higher ground towards the valley slope. Near the slope face, however, the seepage is interrupted by the debris from old slides, and, in the immediate vicinity of the bridge, by relatively impervious fill. Further, seepage towards the existing mass concrete bridge abutment (which was reportedly backfilled to rock with granular material) probably influences the groundwater conditions close to the abutment. Because of these factors, the piezometric groundwater levels within the overburden slope are extremely complex and vary both vertically and areally.

Within the bedrock, the groundwater conditions are more uniform. Predictably, lateral seepage is occurring from the higher ground to the south towards the river valley. Based on the results of this investigation, the piezometric water level in the bedrock varies from about elevation 656 in borehole 209, put down some 200 ft. behind the crest of the valley bank to about elevation 637 in borehole 201 put down at the edge of the river (see Figure 4). This head loss corresponds to a hydraulic gradient through the bedrock of about 5 per cent. It should be noted that except during periods of river flooding, the piezometric water level in the rock beneath the river area is artesian with respect to

the river level indicating that upward seepage of groundwater from the rock into the river is occurring. It should further be noted that during flood stages the piezometric water level in the rock, at least in the lower portion of the slope, reflects fractionally the rise in river level.

Despite the variation in groundwater levels in the overburden, the average phreatic surface in the lower portion of the slope (boreholes 102, 201 and 203) appears to be within a few feet of the piezometric water level in the underlying bedrock and, coincidentally, within a few feet of the present ground surface.

Within the undisturbed material behind the 1957 failure, downward seepage of surface water into the bedrock and lateral seepage towards the existing approach cut and the face of the valley bank is occurring. Behind the crest of the existing valley bank (boreholes 206, 207 and 209) the piezometric water level varies with depth from a phreatic surface at about elevation 680 to 685 to a level corresponding to the piezometric pressure in the underlying bedrock (see Figure 21).

Within the upper portion of the intermediate slide debris slope, downward and lateral seepage similar to that discussed above also appears to be occurring. At a given section, the piezometric water level appears to vary from a phreatic surface at about the existing slope face (at shallow depth) to a level corresponding to the piezometric level in the underlying bedrock (in the lower portion of the overburden).

7.0 STABILITY OF EXISTING SOUTH VALLEY BANK

7.1 "1945" Failure

As previously noted (see section 3.2) a major slide occurred immediately downstream of the existing bridge in 1945. The slide zone apparently extended across the bridge alignment as the slide caused serious lateral displacement of the top of the abutment. Although there is little hard factual information concerning this failure, Legget (1946) indicates that at the time of failure, the overall slope of the valley bank was about $1\frac{1}{2}$ horizontal to 1 vertical and that the failure appeared to be a conventional deep-seated rotational movement. However, it is understood that no borings were put down to establish the subsoil conditions and no analysis of the failure was carried out.

Based on available topographic mapping, it appears that at the time of the 1945 failure the overall height of the slope was about 80 feet. Based on this slope height, an overall slope of $1\frac{1}{2}$ horizontal to 1 vertical and the subsurface conditions established during the present investigation, preliminary total stress stability analyses indicate that the average operating undrained shear strength of the lacustrine clay deposit at the time of the failure (factor of safety equal to one) was about 1,600 to 1,800 lb/sq.ft. This agrees closely with the upper limit of the range of available undrained shear strength measured during the present investigation (see Figure 6).

7.2 "1957" Failure

In 1957, a major failure occurred immediately upstream of the existing bridge (see Section 3.2). Based on available

photographs, this failure extended into the approach to the existing bridge but reportedly did not cause any significant movement of the existing abutment. The back scarp of this failure, although probably somewhat degraded, is still clearly visible upstream of the present bridge.

Considerable information concerning the pre-slide and post-slide geometry for this failure is available in the form of a detailed topographic map prepared in 1931 (Drawing No. Mun 4201-2, "Cockshutt Bridge, Brantford, Ontario - Contour Plan and General Elevation" dated April 10, 1931 and revised September 1, 1931), Grand River Conservation Authority Flood Line Mapping prepared in 1967 and aerial photographs taken in 1948, 1962, 1967 and 1972. Based on the available information, our "best estimate" of the pre-slide and post-slide geometry is shown on Figure 22. As indicated on this figure, the average overall slope prior to failure was about $1\frac{1}{2}$ to 2 horizontal to 1 vertical and the failure extended about 100 ft. back from the original crest of the bank.

Initially, the failure was "back-analysed" using the "sliding wedge" method of analyses based on conventional Rankine earth pressure theories. The results of these analyses, both total stress and effective stress shear strength parameters, are summarized in Figure 23.

Based on total stress shear strength parameters and assuming a uniform undrained shear strength in the lacustrine deposit, the "critical" failure plane will extend as deep as possible (i.e. down to a hard stratum - in this case, the bedrock). The minimum average undrained shear strength along the basal failure plane required for stability (i.e. factor of safety equal one) is about 2,800 lb/sq.ft. for an average

overall slope of $1\frac{1}{2}$ horizontal to 1 vertical and about 2,100 lb/sq.ft. for an average overall slope of 2 horizontal to 1 vertical. As the average available shear strength near the base of the lacustrine deposit is only about 1,800 to 2,000 lb/sq.ft., an average 2 horizontal to 1 vertical overall slope would, at best, be only marginally stable and a $1\frac{1}{2}$ horizontal to 1 vertical slope would be decidedly unstable.

Based on a "best guess" groundwater level in the slope and assuming a purely frictional material (i.e. $c' = 0$) the results of the effective stress sliding wedge analysis indicate that the "critical" plane of failure is somewhat higher than for the case of the total stress analysis and that the angle of shear resistance, ϕ , required for stability is about 25 degrees for the case of a 2 horizontal to 1 vertical slope and about 29 degrees for a $1\frac{1}{2}$ horizontal to 1 vertical slope. For the case of the steeper slope, the required, ϕ , agrees closely with the average measured, ϕ , from laboratory tests (see Figure 13).

The 1957 failure was also "back-analysed" assuming a conventional circular arc failure surface and using published slope stability charts (Terzaghi and Peck, 1948). The results of this analysis (based on total stress conditions) are summarized on Figure 24 and indicate that for a deep-seated failure surface the required average undrained shear strength for stability is about 1,700 to 1,800 lb/sq.ft. ($1\frac{1}{2}$ to 2 horizontal to 1 vertical overall slope). As the average measured undrained shear strength below about elevation 670 (see Figure 6) is only about 1,500 lb/sq.ft. failure could be anticipated for slopes flatter than apparently existed at the time of the actual failure.

It should be noted from Figure 6 that the lowest actual measured undrained shear strengths occur between about elevations 620 and 660 (minimum average shear strength of about 1,200 lb/sq.ft.) However, based on the results of the total stress stability analyses (see Figure 24) and considering the presence of the stiffer overlying crustal material (estimated average undrained shear strength of about 3,000 lb/sq.ft. above elevation 670), a deep-seated failure surface extending to or close to the bedrock will probably be more critical despite the trend for increasing strength with increasing depth (see Figure 6).

Finally, a conventional "trial and error" total stress stability analysis was carried out using the "best estimate" pre-slide slope geometry (see Figure 22) and average minimum measured shear strengths. Two conditions were considered.

- (i) A deep-seated failure surface with an average undrained shear strength of 1,500 lb/sq.ft. between elevations 670 and 586 (the bedrock surface).
- (ii) A relatively shallow failure surface with an average undrained shear strength of 1,200 lb/sq.ft. between elevations 670 and 620 (the assumed upper surface of a stiff layer).

The results of these analyses are summarized on Figure 25.

As indicated on Figure 25, for the case of a deep-seated failure surface and an average undrained shear strength of 1,500 lb/sq.ft. below elevation 670, the minimum computed factor of safety is 1.0 and the theoretical "critical" failure surface coincides very closely to the "best estimate" of the

actual post-slide geometry. For the case of the shallower failure surface, the computed minimum factor of safety is about 1.1 for an average shear strength of 1,200 lb/sq.ft. between elevations 620 and 670.

Based on the above analyses, it is our opinion that:

- (i) the 1957 failure is typical of the past failures along this portion of the Grand River,
- (ii) the failures are "triggered" by toe erosion and local toe instability,
- (iii) when the overall slope is steepened to about $1\frac{1}{2}$ to 2 horizontal to 1 vertical deep-seated rotational instability of the entire bank occurs. These failures can best be analysed in terms of undrained shear strengths (total stress conditions),
- (iv) below the upper desiccated material (i.e. below about elevation 670) the average undrained shear strength of the lacustrine deposit is about 1,500 lb/sq.ft.
- (v) assuming the lacustrine deposit to be cohesionless ($c' = 0$) the average effective internal angle of shearing resistance, ϕ' , of the material is about 28 to 29 degrees.

7.3 Stability of Present Slope

The existing slope may be broadly sub-divided into three parts;

- (i) the toe area which, at least locally, may be over-steepened by river erosion,
- (ii) an intermediate slope (generally about 3 horizontal to 1 vertical) across the debris from previous failures, and

- (iii) an upper steep scarp representing the back face of the previous failures.

As part of the investigation of the existing abutment movements, two slope indicators were installed through the existing slope to monitor overburden movements. Details of the slope indicator installations are given in the Record of Borehole sheets for boreholes 202 and 205 (slope indicator casings numbered 1 and 2, respectively) and the results of periodic readings taken in the casings since the time of installation are summarized on Figure 26.

Based on monitoring to date (obtained over an approximately $3\frac{1}{2}$ week period) it is our opinion that the readings in slope indicator casing number 2, installed in the existing roadway approach, do not indicate any significant slope movement. The maximum recorded lateral displacement of the casing is about $\frac{1}{4}$ in. which is about the limit of accuracy of the instrument over a length of 100 feet and the recorded movements are erratic (i.e. the readings indicate that the top of the casing moved to the south-west, then back to the north-east, then to the south, south-east and, on May 7, 1975, the top of the casing was within 0.1 in. of the starting point). It should be noted, however, that a $3\frac{1}{2}$ week period is too short to expect any real indication of long term creep type movements. Consequently, it is recommended that additional readings be taken in this installation at about one month intervals for a period of 6 months to a year to determine whether a pattern of progressive long-term movements is established.

Readings taken to date in slope indicator number 1, installed adjacent to the existing bridge abutment, indicate that movement is occurring towards the south-east (i.e. the movement is basically into, rather than out of, the slope).

The recorded movement to date at the top of the casing is about 0.9 in. and the pattern of movement is relatively uniform. However, the readings indicate that the entire casing is tilting rather than indicating a local zone or plane of movement.

The pattern of movement indicated by slope indicator number 1 (i.e. movement into the slope), if true, is not indicative of slope creep or instability. Although a possible explanation for a movement into the slope is discussed in section 8.2. we have not, to date, been able to definitely explain a movement of this type.

To confirm the pattern of movement recorded to date in slope indicator number 2, we recommend that:

- (i) additional readings be taken in this installation at about 2 week intervals,
- (ii) the precise plan location and elevation of the top of the casing be established by ground survey at the time of each reading. This will require the establishment of a remote reference point (bench mark) and precision survey procedures which are outside our field of expertise.

In addition to the monitoring, a theoretical assessment of the stability of the existing slope within the 1957 slide area was carried out. Two mechanisms of slope instability or creep may be postulated.

- (i) Creep along the original failure plane due to erosion (unloading) at the toe of the slope.
- (ii) Local instability of the lower portion of the slope due to oversteepening by toe erosion.

Based on the theoretically critical failure surface shown on Figure 25 and the "best estimate" of the post-slide geometry, the minimum average remoulded or residual undrained shear strength along the failure plane required for stability of the slide debris (factor of safety equal one) is about 800 lb/sq.ft. This agrees generally with the upper range of the remoulded shear strength of the lacustrine clays and silts as determined by field vane tests. Although some thixotropic regain in shear strength with time may be anticipated, the results of this investigation indicate that, in undisturbed areas, the ratio of undrained shear strength to effective overburden pressure is about 0.2 to 0.3. Consequently, based on the slide geometry, the average strength along the failure surface will probably not increase to more than about 1,000 lb/sq.ft. Thus, assuming no toe erosion, the factor of safety against failure along the original failure surface will increase only to about 1.1 to 1.2. Should the toe of the slide debris slope erode to approximately the original toe of slope location shown on Figure 25, the factor of safety along the original failure surface will be reduced by about 5 to 10 per cent. Consequently, even relatively minor erosion of the toe of the slide debris could cause a significant increase in straining or creep along the original failure surface.

A similar analysis carried out for the slide geometry illustrated on Figure 5, representative of the filled area close to the existing slope, also gave a probable factor of safety of about 1.1 to 1.2 along the inferred original failure surface. Similarly, in this area, progressive creep along the original failure surface could be anticipated as a result of toe erosion.

Total and effective stress stability analyses carried out assuming local instability of the lower portion of the slope due to toe erosion indicated that such failure would be typically fairly shallow and would be controlled by effective stress parameters. The results of an effective stress stability analysis carried out for the present slope geometry along the proposed roadway centreline are summarized on Figure 27, and analyses carried out for a fairly typical 3 horizontal to 1 vertical slide debris slope are summarized on Figure 28. The results of these analyses indicate that under severe groundwater conditions, the slope would be, at best, only marginally stable.

In summary, the slide debris forming the lower portion of the slopes in the area of the proposed bridge is stable if toe erosion is prevented. However, with toe erosion, local shallow instability of the lower portion of the slope and deep-seated progressive creep type movements along the original failure plane must be anticipated.

8.0 NOTES ON EXISTING BRIDGE ABUTMENT

8.1 Condition of Abutment Concrete

To assess the condition of the south abutment of the existing bridge and, in particular, to attempt to determine if the abutment is broken and sheared (see also section 3.1), the abutment was core drilled for its full height (borehole 204). A detailed structural log locating and commenting on all structural defects observed in the concrete core is given in Appendix I to this report. To determine the degree of "openness" of various defects observed in the core, pressure packer permeability tests were carried out over selected sections of the borehole. The results of these tests are summarized on the Record of Borehole sheet for borehole 204 as "take" (expressed as gallons/min. for various applied total hydraulic pressures) and as an "equivalent" coefficient of permeability. It should be noted that both of these values are expressed as an average "take" or permeability along the entire test section (i.e. the concrete is considered as a homogeneous mass) whereas the actual "take" probably occurred through isolated defects within the otherwise relatively impervious concrete.

Excluding the heavily reinforced concrete collar which we understand was installed at the top of the abutment in 1945, the concrete forming the south abutment of the existing bridge is generally sound and intact (generally 100 per cent core recovery and R.Q.D. of 90 to 100 per cent). The concrete in the lower approximately 20 ft. of the abutment (i.e. below about elevation 603.5) is somewhat more porous than the upper concrete and contains some construction debris (wood, steel and the like). The abutment is underlain at

elevation 584.8 by fairly sound dolomitic bedrock of the Salina formation. An approximately 7 in. thick "void" (probably infilled) was encountered between the sound concrete and the rock surface. In our opinion, this "void" represents a local condition resulting from debris (possibly a lump of clay) left in the base of the excavation at the time of the initial concrete pour.

Although the abutment concrete is generally intact, the core contains numerous defects (see Appendix I). The majority of these defects are clearly drilling fractures (caused by unavoidable vibration of the core barrel during drilling) as they were "fresh" breaks which could be precisely reassembled. However, a few defects evidenced minor weathering (staining) and/or could not be reassembled. None of the defects showed evidence of horizontal shear displacement.

Based on detailed examination of the core and correlation of the observed defects with measured pressure packer test "takes", there appears to be only one throughgoing open defect in the abutment; the two closely spaced and weathered breaks at about elevation 637.1. It should be noted that a core break which could not be reassembled was observed at elevation 610.6; close to the elevation at which a "break" in the abutment had been postulated by previous investigators (see Section 3.1). However, the results of a pressure packer test carried out over this portion of the boring (test no. 3) showed very little "take" indicating that the "break", if natural, was tight (i.e. it had not opened due to rotation of the upper portion of the abutment).

In summary, the results of borehole 204 indicate that, if the existing abutment is broken, the most probable level of the break is at about elevation 637.

8.2 Possible Causes of Reported Movements

Three possible causes of the reported abutment movements (rotation and shear displacement) can be postulated.

i) Lateral Earth Pressures Resulting from Deep-Seated Instability of the Existing Valley Bank: As previously discussed (see section 7.3), the slope upstream of the existing bridge (i.e. in the area of the 1957 failure) is only marginally stable and some progressive and deep-seated creep type movements along the original failure surface are probable as a result of toe erosion. However, it is understood that this failure did not significantly effect the existing abutment. In the area of the 1945 failure (i.e. immediately downstream of the existing bridge) the slope was flattened to about 4 horizontal to 1 vertical immediately after the failure and a wide stabilizing berm was provided at about elevation 660 to 670. In our opinion, this regrading should have been sufficient to stabilize the failure as no significant toe erosion is evident in the downstream area. Further, readings to date in the slope indicator casings installed behind and adjacent to the abutment (see Figure 26 and pertinent discussion in section 7.3), although of limited duration, do not indicate the presence of any progressive downhill movements such as would be associated with deep-seated bank instability.

ii) Conventional Lateral Earth Pressures Acting Against the Abutment: Considering the relatively steeply sloping ground surface behind and in front of the abutment, the net lateral earth pressure (based on conventional Rankin earth pressure theory) acting against the abutment are large and, if the abutment is broken, the resulting thrust at the top of the abutment and shear force across the break will be

significant (see Figure 29). Based on the results of borehole 204, put down through the existing abutment, the most probable location of the break, if one exists, appears to be at about elevation 637 (i.e. about 16 ft. below the top of the abutment). Assuming the backfill behind the abutment is drained to river water level, the thrust at the top of the abutment, broken at this level, would be about 130 kips and the shear force across the break would be about 230 kips.

As it is understood that the total weight of the bridge deck and abutment at about elevation 637 is about 3,500 kips and as the coefficient of friction ($\tan \phi$) of concrete on concrete is probably in the order of 0.3, shear displacements across a break at this level are unlikely (factor of safety of about 4).

iii) Structural Forces Within the Bridge Itself: During the 1945 slope failure, the south abutment moved about 18 in. to the north. This could be expected to induce large compression forces on the deck structure which would tend to close and jamb any expansion joints. Following the failure, the deck structure was fixed rigidly to the south abutment by means of a reinforced concrete collar; the deck being more or less free to move across the remaining piers and abutment. Consequently, it would appear that normal thermal expansion of the bridge deck would tend to "push" the south abutment towards the south (i.e. into the slope) and normal thermal contraction would tend to "pull" the abutment towards the north (i.e. out of the slope). Due to normal earth pressures acting against the south abutment, this alternating "push-pull" action could result in a gradual creep of the deck

and the top of the south abutment towards the north, particularly if the south abutment is in fact sheared at some point. Although a detailed analysis of this type of movement is outside of our field of expertise this mechanism could explain the otherwise inexplicable southward tending movements indicated by slope indicator casing number 1 installed adjacent to the abutment (see Figure 26 and pertinent discussion in section 7.3). If the abutment is being "pushed" into the slope by thermal expansion of the deck, this abutment movement could result in southward movement of the surrounding soil and thus the slope indicator casing. In this regard, it is interesting to note that the approximate air temperatures at the time of the various readings in slope indicator number 1 were:

April 10, 1975	- 35° to 40° F
April 16, 1975	- 45° to 50° F
April 21, 1975	- 35° to 40° F
April 30, 1975	- 50° to 60° F
May 7, 1975	- 60° to 70° F

9.0 STABILIZATION OF SOUTH VALLEY BANK

9.1 Stability Considerations

As previously discussed, the existing slopes forming the south valley bank are the results of old failures and, due to toe erosion by the river, the slide debris forming the lower portion of the slope are only marginally stable. To stabilize the slopes in the vicinity of the proposed bridge, we recommend that the existing bank be flattened.

Based on the results of this investigation and back-analyses of old slides (notably the slide which occurred in 1957 - see section 7.2) the average undrained shear strength of the lacustrine clays and silts is about 3,000 lb/sq.ft. above elevation 670 and about 1,500 lb/sq.ft. below elevation 670. Based on this available shear strength, the overall factor of safety (F.S.) against deep-seated rotational instability (total stress stability analyses) for various overall slope angles, β , will be:

Cotan β = 3	F.S. = 1.2
Cotan β = 4	F.S. = 1.3
Cotan β = 5	F.S. = 1.5

Despite the history of past failures along this section of the river, it is our opinion that the degree of confidence in the shear strength data is sufficiently high from the "back-analyses" of known failures that an overall factor of safety against deep-seated rotational instability of as low as about 1.3 can be accepted for design. On this basis, we suggest that the overall slopes be regraded to no steeper than 4 horizontal to 1 vertical.

With a factor of safety of 1.3 against deep-seated rotational instability some local elastic overstress is inevitable within the slope and some deep-seated strain or creep type movement will occur, particularly within the old (1957) slide debris. Although the potential for such movements could be reduced by further flattening of the slopes (to say 5 or 6 horizontal to 1 vertical) it is our opinion that the strain experienced by the suggested 4 horizontal to 1 vertical slope will not be of sufficient magnitude to justify flatter slopes.

Straining within the regraded slope would result in movement of the south pier and abutment of the proposed bridge. Although such movements are expected to be small, (less than about 2 to 3 in.) we suggest that, as a precaution;

- (i) provision be made on the design to accommodate as much as 3 in. of horizontal movement between the pier (abutment) and the bridge structure,
- (ii) the existing slopes be regraded as early as possible prior to construction of the bridge and,
- (iii) movements, if any, within the regraded slope be monitored by means of slope indicator casings installed immediately behind (up-slope) of the pier and abutment locations (we suggest that a slope indicator casing be installed behind the pier excavation in any case to monitor the performance of the temporary excavation shoring.

The results of laboratory consolidated drained direct shear tests together with the results of a "back" analysis

of the existing slope indicate that a realistic minimum average effective (drained) internal angle of shearing resistance, ϕ' , of the essentially cohesionless (under drained conditions) lacustrine deposit is about 29 degrees. This value was used to assess the long term stability of the regraded (4 horizontal to 1 vertical) slope.

In assessing the stability of the regraded slope in terms of effective stress conditions, the most difficult problem at this site is predicting the probable groundwater conditions within the slope. Based on available piezometric data, however, it is anticipated that at relatively shallow depth within the slope the maximum average piezometric water level will be less than normal hydrostatic pressure resulting from a groundwater level coincident with the slope face.

The results of preliminary stability analyses carried out using "circular arc" failure surfaces and assuming a piezometric groundwater level at ground surface indicated that the minimum factor of safety against rotational instability is about 1.5 and that this minimum value occurred for relatively shallow failure surfaces passing through to the toe of the slope area. Consequently, the proposed slope was re-analysed assuming relatively shallow non-circular failure surfaces and various piezometric groundwater levels. The results of these analyses are summarized on Figure 30 and indicate that for a piezometric groundwater level at ground surface, the minimum factor of safety against sliding is about 1.4. For a piezometric water level 8 ft. below ground surface, the minimum factor of safety increases to greater than 2.

As shallower and shallower failure surfaces are assumed, the analyses degenerates to an "infinite" slope condition with a groundwater level at ground surface. For this case, the factor of safety against sliding is equal to the tangent of the slope angle divided by $\frac{1}{2} \tan \phi$. For a 4 horizontal to 1 vertical slope and a ϕ , of 29 degrees. this gives a factor of safety of about unity. As the water level is lowered to below the slope face, however, the factor of safety increases rapidly to about 2. Consequently, to minimize surficial creep of the slope, we recommend that positive steps be taken to ensure that the phreatic surface in the lower portion of the slope is maintained at least 4 ft. below the slope face. This can be readily accomplished in conjunction with the provision of erosion protection along the lower portion of the slope (see Figure 31 and section 9.2)

As regrading of the slope will probably not significantly affect the piezometric pressure in the underlying bedrock, at the toe of the regraded slope piezometric pressures in the rock could be artesian with respect to the regraded slope face. Although this excess pressure will dissipate through the lacustrine deposit due to upward seepage into the river channel, the rate of dissipation could be extremely erratic within the old slide debris and it is possible that significant artesian pressures could persist to within a few feet of the face of the slope. Consequently, as a precaution, we suggest that a line of gravity relief drains be installed into the bedrock along the toe of the regraded slope. The drains should extend at least 20 ft. into the bedrock and should be at least 3 in. diameter. The holes should be inclined back into the slope

at an angle of about 30 degrees to the vertical and should be cased through the overburden (driven casing). The casing should be seated into the rock by driving and the drains advanced as open holes into the rock. The drains should discharge into the bottom of the river and the tops of the drains should be fitted with removable screens and should be equipped with flap valves to minimize contamination of the drains. A suggested method of installing the drains is illustrated on Figure 31. Initially, the drains should be installed at about 50 ft. centres and the performance of the system monitored by means of piezometers. Should this monitoring indicate that insufficient drawdown is achieved by the initial system, additional drains should be installed. Finally, it is recommended that the system be inspected periodically and that the wells be flushed (by high pressure water jetting) at least annually (after spring flooding).

It should be noted that, locally, the existing slope may dip below the recommended overall 4 horizontal to 1 vertical slope. In the lower half of the slope, such depressions should be filled to the final slope profile with well graded and free draining granular material. Local depressions on the upper half of the slope may be left but intermediate slopes above such depressions should be trimmed to not steeper than 2 horizontal to 1 vertical.

9.2 Erosion Protection

It must be stressed that the preceding discussion is predicated upon the fact that erosion of the toe of the slope is prevented. Examination of the existing banks suggest that the most serious erosion occurs due to toe scour under normal flow conditions rather than under flood conditions. To

strengthen the toe area and to promote slope drainage at the toe, we suggest that the toe of the regraded slopes be protected by a "vertical" basket gabion wall backed by suitably graded granular filter material(s) or that the thickness of highly compacted granular filter material be increased by steepening the toe of the slope to 2 horizontal to 1 vertical as illustrated on Figure 31. In either case, heavier gabion units and/or rip-rap protection may be required at the toe to prevent mechanical damage of the slope protection by ice flow impact and the like. As indicated on Figure 31, the width of the resulting berm at about elevation 635 can be varied to accommodate variations in the existing slope geometry while maintaining a "smooth" channel alignment (i.e. the "toe of the slope" line can be established to best suit river hydraulics and the berm width varied to marry this "toe of slope" with the regraded existing slopes).

The face of the regraded slope should be protected by means of blanket gabions placed on a bedding pad of properly graded filter material as illustrated on Figure 31. To ensure that the phreatic surface is maintained at least 4 ft. below the slope face (see section 9.1) we recommend that the total thickness of gabion blanket and filter blanket be at least 4 ft. This protection should extend at least 5 ft. above the maximum design flood level. Further, as a precaution, we suggest that blanket gabions be installed on the bed of the river for a distance of about 20 ft. in front of the toe of the slope.

Finally, it is stressed that the slope protection must be inspected and, if necessary, repaired periodically (at least once a year after spring flood run-off).

9.3 Lateral Extent of Slope Stabilization

We recommend that the entire 1957 slide area be regraded to an overall slope of not steeper than 4 horizontal to 1 vertical and protected against river scour. Consequently, slope stabilizations as discussed above, will be required for a minimum distance of about 400 ft. upstream of the proposed bridge centreline. Downstream of the bridge, we suggest that the slope stabilization be extended about 200 ft. from the proposed centreline.

It should be noted that upstream of the 1957 slide area, the river is impinging more or less directly onto the toe of the debris from an earlier slide (see Figure 2). At present, the toe of the slope has been eroded to a near vertical scarp some 10 ft. high. The results of borehole 2 put down behind the crest of the valley bank during the preliminary (1973) investigation indicate that the subsurface conditions in this area are similar to those in the 1957 slide area. Consequently, if left unprotected, the existing slide debris slope will retrogress through toe erosion and local instability until the overall slope is steepened to about $1\frac{1}{2}$ to 2 horizontal to 1 vertical at which time a major deep-seated rotational instability will occur.

Although toe erosion and local instabilities per se in this area should not adversely affect the proposed bridge, major retrogressive failure could involve a significant portion of the recommended slope stabilization works (slope flattening, gabions and the like). In this case, unless immediate remedial measures are taken, successive erosion and local instabilities could quite rapidly cause retrogression of the slope into the bridge site.

It should be noted that although erosion several hundred feet upstream of the site poses a potential danger to the proposed bridge, the retrogression process outlined above will require a considerable time span (probably tens of years). Consequently, we do not feel that stabilization measures in this area are essential at the time of construction of the bridge. However, within a few years, some form of toe protection should be installed (flattening of the slope is not considered necessary as local creep type movements in this area are not critical). In the meantime, this area should be kept under careful observation and the rate of toe erosion monitored such that local remedial measures can be installed before a serious failure occurs.

10.0 BRIDGE FOUNDATIONS

Considering the relatively loose and variable nature of the alluvial and fluvial deposits encountered across the valley floor area, the presence of variable fill and slide debris in the south valley bank area and the relatively soft and compressible nature of the lacustrine silt and clay complex, the use of spread footings founded in the overburden is not feasible for the foundation support of the proposed continuous bridge structure. Consequently, we recommend that the pier and abutment loads be transferred through the overburden and onto the bedrock, which underlies the site at between about elevations 585 and 595, by means of piles. Either driven piles or relatively large diameter pre-bored, cast-in-place concrete piles may be used for the support of the proposed bridge. However, with either type of pile, special precautions must be taken during construction to prevent settlement due to the presence of possible voids or cavities in the foundation bedrock.

10.1 Driven Piles

Although either concrete filled pipe piles or pre-cast concrete piles would be suitable for the foundation support of the proposed pier and abutments, high pore pressures induced in the lacustrine clay and silt complex during driving of displacement type piles (pipe piles or pre-cast piles) at the south pier and abutment locations, could endanger the overall stability of the south valley bank, even following regrading of the slope. Consequently, we recommend that the proposed piers and abutments be founded on small displacement steel 'H' piles driven to practical refusal on or in the bedrock.

Although no serious problems are anticipated in advancing conventional steel 'H' pile sections to practical refusal on or in the bedrock, some relatively hard driving must be anticipated in penetrating obstructions (cobbles and boulders) within the sand and gravel deposit which underlies the flood plain area and in the lower portion of the overburden. Further, very hard driving will be required to seat the piles onto or into the bedrock (see following discussion of driving criteria). To minimize possible damage to the piles during driving, we recommend that a relatively heavy pile section, such as a 12 BP 74 section, be employed. Relatively heavy steel 'H' piles (such as a 12 BP 74 section) driven to practical refusal on or in the bedrock (see following discussion of driving criteria for piles) may be designed using an allowable load of as much as 75 tons per pile.

It should be noted that four (4) of the borings put down during the second stage of the investigation encountered cavities or open fractures within the bedrock. As previously discussed (see Section 6.2) complete delineation of possible cavities by exploratory borings during the pre-design investigation is not feasible. Cavities are known to exist in the rock so it must be assumed that a fairly extensive cavity could occur directly beneath a pile group. During driving of individual piles, it is possible that, should a cavity exist, the roof of the cavity could be sufficiently strong to "hang-up" the piles but that under the static loading of several piles the roof could collapse. However, all of the voids or "open" zones encountered in the borings occurred within the upper few feet of the bedrock. In this case, it should be possible to intentionally shatter the rock and "break down" the relatively thin roof of such cavities by the use of high driving energies during driving of the piles (This is

particularly true across the valley floor area where the overburden is relatively thin and the energy absorbed in overcoming skin friction on the piles will be low). Consequently, we recommend that the following piling requirements and driving criteria be used. (Subject to review after driving of the initial piles):

- (i) A heavy pile section (12 BP 74 section minimum) should be employed.
- (ii) Each of the piles should be fitted with a hardened steel drive shoe (e.g. a Pruyt point) to minimize damage to the pile tips during driving.
- (iii) The lower 10 ft. of each pile should be reinforced by welding 3/8 in. (minimum) thick steel plate to the web and flanges.
- (iv) The piles should be driven using a hammer having a rated energy of about 40,000 ft.-lb per blow (e.g. a Delmag D-22 or equivalent).
- (v) The piles should be driven onto or into the bedrock to a final set of at least 20 blows/in. with an average set of at least 15 blows/in over the last 12 in. of driving.
- (vi) After the design set is achieved, the piles should be given a minimum of 50 blows of the hammer (40,000 ft.-lb of energy per blow), no matter what the resistance, in an attempt to shatter the roof of any potential cavity. If the piles advance at greater than 0.05 in./blow during this proof-driving the piles should be driven until the design set is again

achieved. The process of proof-driving should be repeated until no significant additional penetration is achieved. If no significant advance (less than about 0.05 in./blow) occurs during proof-driving, the piles may be left after the 50 blows have been applied.

- (vii) As the object of the proof-driving is to intentionally shatter the rock, it is imperative that each pile in a group be retapped to the design set following driving of all of the piles in the group.

Although it is felt that the above driving procedure should shatter the roof of any voids or cavities near the surface of the rock, the possibility of a relatively large void at depth can not be entirely discounted. Consequently, as a precaution, we recommend that the bedrock at each pier and abutment location be proof-drilled following driving and retapping of all of the piles on the group (each pier and abutment is considered to be a pile group for this discussion). To this end, we recommend that a minimum of two (2) test holes be put down at each pier and abutment location. These test holes should be advanced at least 20 ft. into the bedrock by rotary core drilling in minimum Nx core size (approximately 3 in. dia. drill hole). A core must be obtained for examination and each hole should be water tested (pressure packer permeability test) to assess the "openness" of the rock.

Should the pile driving records (pile penetration and the like) and/or the results of the proof-drilling and water testing indicate the presence of large open voids or cavities beneath a pile group(s) it may be prudent to fill (grout) the voids prior to installation of the pile cap(s). However, we

are not aware of any precedent for grouting of the Salina formation bedrock beneath piled foundations or of any cases of structural failures attributed to the collapse of cavities in this rock. Consequently, although the possible necessity of grouting should be borne in mind, we suggest that no provision for such work be made at this time. In any case, we would recommend that this work be carried out on a "time and materials" basis by an approved specialist grouting contractor rather than as part of the general contract.

10.2 Bored Piles

As an alternative to driven piles, consideration should be given to founding the proposed piers and abutments on relatively large diameter pre-bored, cast-in-place concrete piles socketted into the sound bedrock. The advantage of this type of pile is that the foundation bedrock within and below the socket can be inspected and, if necessary, the piles can be carried deeper into the rock to avoid the possible problems associated with open voids or cavities in the rock (see Section 10.1).

The piles may be advanced and cased through the overburden and into the bedrock by any approved method (pre-boring with "mud", churn drilling or the like), the casing being seated into the bedrock by driving. The holes should then be advanced a minimum of $1\frac{1}{2}$ pile diameters or 5 ft., whichever is the greater depth, into sound bedrock by either rotary or percussion drilling. The socket thus formed must be inspected and should any open fractures or cavities be encountered in the rock, the piles should be extended at least $1\frac{1}{2}$ pile diameters below the defect. Further, the competency of the rock below the base of the piles must be proven for a depth of at

least 2 pile diameters by rotary core drilling (minimum Bx core size).

To facilitate inspection and to permit cleaning of the sides of the socket and concreting of the pile "in the dry", an attempt should be made to dewater each pile unit. Considering the fractured nature of the rock, however, it is probable that dewatering of the piles will not be feasible. In this case, the sockets will have to be cleaned and inspected underwater (by divers, underwater camera or the like) and the concrete placed by the tremie method. Under no condition, should concrete be placed in a partially dewatered shaft if significant groundwater inflow from the rock is occurring.

Providing the walls and base of the socket are cleaned and the pile is socketted at least $1\frac{1}{2}$ pile diameters into sound rock, the piles may be designed using an allowable load of 75 tons per square foot of cross-sectional pile area. Thus, a 30 in. diameter pile (cross-sectional area about 5 sq. ft.) may be designed using an allowable load of about 375 tons.

It should be noted that the groundwater in the Salina formation is known to contain high sulphate concentrations. Consequently, it is recommended that the piles be constructed using sulphate resistant cement.

The steel casing or liner installed through the overburden should be left in place.

11.0 NORTH APPROACH EMBANKMENT

11.1 Embankment Stability

The results of this investigation indicate that the proposed embankment area is underlain by loose to compact silty sand and sand and gravel followed by stiff to very stiff lacustrine silts and clays. Provided all topsoil and other surficial organic materials are removed over the full base width of the embankment, no foundation stability problems are anticipated for 30 ft. (maximum) high roadway approach embankments constructed using side and end slopes not steeper than 2 horizontal to 1 vertical (see also section 11.2). Settlement of the foundation subsoil under the full embankment loading should be less than about 2 to 3 in. with the majority of this settlement occurring during the construction period.

During flood periods, the end of the approach embankment will probably be subjected to serious erosion forces and, under the maximum design flood, the lower, northern portion of the embankment will be overtopped. Consequently, to prevent erosion of the fill, the "nose" of the embankment will probably have to be protected by rip-rap, gabions or the like. The size and extent of this protection will depend upon hydraulic consideration which are outside the scope of this investigation. Further, it is recommended that, as a precaution, the side and end slopes be flattened from the 2 horizontal to 1 vertical slope required for foundation stability to about 3 horizontal to 1 vertical. All slopes not protected by rip-rap or the like should be sodded or seeded and mulched to prevent erosion by floods and surface water run-off.

11.2 Earth Borrow

It is understood that it is presently proposed to employ the material for the south approach cut as earth borrow for the construction of the north approach embankment. However, the results of borings put down within the proposed approach cut/borrow area (see section 6.1.3) indicate that the in situ water content of the proposed borrow material is generally some 5 to 10 per cent wet of the optimum water content for compaction. Consequently, proper compaction of this material at its natural water content will be extremely difficult if not impossible.

Three possible alternatives for the construction of the north approach embankment may be adopted.

(i) The material from the south approach cut may be placed at its in situ water content and compacted to 100 per cent of the standard Proctor density at the placement water content (this may be as low as 80 to 85 per cent of the standard Proctor density at optimum water content). In this case, we estimate that the undrained shear strength of the compacted fill will be in the order of 500 lb/sq.ft. Consequently, to maintain stability of the fill itself, embankment side and end slopes should be made not steeper than about $3\frac{1}{2}$ horizontal to 1 vertical in the higher portions of the embankment tapering to about 2 horizontal to 1 vertical for embankment heights of less than 10 ft. Further, the local borrow should be capped by a least 5 ft. of highly compacted selected granular fill (such as M.T.C. Granular 'B') to provide adequate pavement strength. With this procedure, long term settlement of the pavement due to consolidation of

the fill could be as much as about 6 to 10 in. Considering the anticipated construction problems (the "wet" fill material will be extremely difficult to handle and compact) and the long term maintenance problems, this procedure is not recommended.

(ii) The embankment could be constructed during a reasonably dry period(s) and an attempt made to dry the borrow material to within about 2 per cent of the optimum water content by air drying. Although this procedure is completely dependent upon the weather, it is understood that economic consideration requires the use of the material for the south approach cut if at all feasible. If air drying of the material is adopted, we suggest that:

- (a) if possible, the work be carried out in late summer (August, September);
- (b) as large an area as possible be exposed in the borrow pit and on the embankment and the surface of the exposed material be loosened and worked by harrowing and raking to facilitate aeration;
- (c) the material be excavated from the borrow area and spread on the embankment in thin lifts (say 6 to 9 in. thick) by working back and forth across the areas in a methodical pattern;
- (d) the surface of the borrow area and embankment be sloped at all times to promote rapid run-off of surface water if rain occurs; and
- (e) if rain is predicted, the surface of both the borrow pit and embankment be "blinded" by rolling with a smooth wheeled steel roller to promote rapid run-off and minimize infiltration.

Following drying of each lift (6 to 9 in. thickness) the lift should be compacted to at least 95 per cent of standard Proctor density at optimum water content by several passes of a static sheeps-foot or peg-foot roller weighing at least 10 tons. Provided 95 per cent of the maximum standard Proctor density is achieved in the field, long-term settlement due to consolidation of the fill should be less than 1 in. and no special precautions regarding the roadway pavement structure should be required.

(iii) The material from the proposed south approach cut be wasted or used for site grading purposes (where compaction is not critical) and the proposed north approach embankment constructed using imported fill material which can be properly compacted. For ease of construction, granular borrow material (e.g pit run sand or sand and gravel having a silt content of less than 20 per cent) is recommended. The fill should be placed in relatively thin horizontal lifts and each lift compacted to 95 per cent standard Proctor density at optimum water content.

12.0 CONSTRUCTION FEATURES AND MONITORING

12.1 South Valley Bank Grading

We recommend that the existing south valley bank be regraded to the proposed 4 horizontal to 1 vertical slope before construction of the bridge foundations commences as construction activity could reduce the stability of the present only marginally stable slope (see section 7.3). During slope regrading, material must be removed from the top of the slope first and the slope flattened to its final configuration from the "top down". At no time should slope regrading operations result in a steepening of the existing slope.

It is understood that the existing bridge is to be maintained in service until the new structure is completed. Consequently, during slope regrading, it will be necessary to either:

- a) leave the existing roadway approach as a local promintory on the face of the regraded 4 horizontal to 1 vertical slope or,
- b) regrade the existing slope across the present bridge to 4 horizontal to 1 vertical and construct a temporary approach span to the existing bridge.

Considering the serious concern which has been expressed regarding the structural integrity of the existing bridge, we favour the second alternative (i.e. provide a temporary approach span) as any promintory on a slope is a "point of weakness" and thus the regrading could result in an increase in lateral pressure on the existing abutment.

If the existing slope is to be flattened across the existing bridge site and a temporary approach span provided, we recommend that this portion of the regrading be carried out prior to the regrading of the adjacent slope. As previously discussed, the regrading should be carried out working from the top of the slope towards the bottom. Extreme care must be exercised during excavation beneath the existing structure and close to the existing abutment.

It is recommended that the south end of the temporary approach span (i.e. adjacent to the existing bridge) be supported on steel 'H' piles driven to practical refusal in the bedrock. As the anticipated foundation loads are relatively light, we do not feel that precautionary grouting of the rock is required (see also section 10.0). The north end of the approach span can be founded on a temporary footing or grillage founded within the existing roadway approach. This footing should be located at least 10 ft. behind the crest of the regraded slope and may be designed using an allowable bearing pressure of about $1\frac{1}{2}$ tons/sq.ft.

Should it be decided to leave a local promintory on the face of the regraded slope to provide access to the existing bridge, we recommend that the side slopes of this promintory be cut no steeper than 2 horizontal to 1 vertical. During slope regrading, the existing slope indicator installations should be protected and prior to regrading at least one and preferably two additional casings should be installed on the east side of the existing bridge to permit monitoring of any lateral or northward movements in the promintory.

It should be noted that if the bridge approach is left in place, the side slopes of the promintory will extend into

or across the south pier pile cap excavation. Consequently, local steepening of the slope and/or provisions of adequately braced shoring will be required in this area. It is understood from discussions with the Consulting Engineers, that the pile cap excavation will extend about 10 ft. into the side slope. Thus an approximately 5 ft. vertical face (in addition to the excavation into the 4 horizontal to 1 vertical slope itself) will be required. We suggest that the toe of the roadway promintory side slope be supported by means of an adequately braced soldier pile and lagging system designed using an earth pressure coefficient of 0.9 to account for the sloping surcharge. The soldier piles should be placed and concreted into pre-bored holes. The use of a steel sheet pile bulkhead, either cantil  ver or braced sheeting, is not recommended as the sheeting penetration will create a line of weakness in the slope. Movement of the promintory slope should be closely monitored during excavation and construction of the pier foundation. The existing slope indicator casings (boreholes 202 and 205) could be used for this purpose and, to this end, these installations should be protected during construction.

12.2 Construction of Pile Caps

As the groundwater level across the flood plain area is at or only slightly above the river water level, and as it is anticipated that the underside of the pile caps will extend only about 4 to 6 ft. below present ground surface, no serious problems are anticipated in excavating for the pile caps at the two northerly pier locations (it is assumed that the north abutment will be perched in the roadway approach fill).

As the existing river channel is underlain by pervious sand and gravel (refer borehole 103) it is recommended that

the excavation for the proposed pier within the river channel be carried out within an adequately braced, closed steel sheet pile cofferdam. The sheet piling should be driven to a penetration below the proposed excavation level equal to at least one-half of the head of water to be retained or at least 5 ft. into the underlying lacustrine clays and silts, whichever is the lesser depth. Further, the tops of the sheeting should extend to a sufficient height to prevent over-topping of the cofferdam during flash run-off periods.

Assuming the existing south river bank is regraded across the existing bridge site and a temporary approach span is provided, the south pier will be located on the face of the regraded slope. Care must be exercised to avoid local over-steepening of the slope during excavation for the pile cap. To this end, it is suggested that the "up-slope" face of the excavation be cut vertical and supported by means of an adequately braced soldier pile and lagging system. The shoring system should be designed using an earth pressure coefficient of 0.5 to account for the sloping surcharge. As previously noted (see section 9.1) we recommend that a slope indicator casing be installed behind the shoring. This slope indicator should be closely monitored during excavation for the pile cap and the shoring system modified, if and as required, movements occur during excavation.

No construction problems are anticipated during excavation for the proposed north abutment pile cap.

12.3 Monitoring During Pile Driving Operations

As discussed in section 10.1, it is recommended that, if the bridge is founded on driven piles, relatively small displacement steel 'H' piles be employed to minimize possible

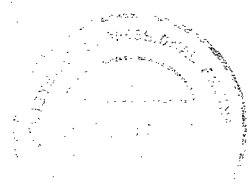
build-up of pore pressures in the lacustrine clays and silts during pile driving operations. However, considering the hard driving required for these piles, we recommend that, as a precaution, "no volume change" piezometers be installed at various depths and at various radial distances from the south pier and south abutment pile groups to monitor any possible pore pressure build-up. Should these piezometers indicate a significant increase in pore water pressure during pile driving (say greater than about 2 or 3 lb/sq. in. increase) driving operations should be suspended until the excess pressure dissipates.

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APPENDIX I

DETAILED STRUCTURAL LOG - BOREHOLE 204

PROPOSED COCKSHUTT ROAD BRIDGE

BRANTFORD

ONTARIO

ELEVATION (FEET)	DEPTH BELOW TOP OF DECK (FEET)		SAMPLE (RUN) NUMBER	PERCENT CORE RECOVERY	R.Q.D., PERCENT	SYMBOLIC LOG	NOTES
657	14						
	14.9'						Top of abutment at elev. 656.5
656	16		2	70	70		15.6' - break - can reassemble but concrete above and below break is poor
655							16.1 - 3/4" D. smooth rod - fits bottom of SA2 but recovered in SA3 no recovery - poor concrete
			3				16.6 - break - some paper on surface
654							16.8 - break - can reassemble
			4	100	100		17.1 - 1"D. smooth rod - fits bottom of SA3 and top of SA4
653	18						Broken concrete below
							16" stick recovered in SA4
652							18.4 - 3/4"D. smooth rod - recovered in SA5 - cannot reassemble runs 4 & 5
							18.5 - smooth weathered joint - probably top of original abutment
651	20						19.5 - 2 x 3/8"D. smooth rods & break - can reassemble break
							19.9 - 3/8"D. smooth rod & break - can reassemble break
650			5	100	97		20.4 - 2 x 3/8"D. smooth bars & break - can reassemble break
							21.0 - break - can reassemble (aggregate)
							21.1 - 3/8"D. smooth rod (concrete intact)
							21.6 - 3/8"D. smooth rod (concrete intact)
649	22						
648							23.6 - break - can reassemble
647	24						24.2 - break - porous concrete - cannot reassemble
646							
645	26		6	100	80		25.6 - break - can partially reassemble but some concrete missing
							25.9 - break - can partially reassemble but some concrete missing
644							27.1 - break - can reassemble
	28						27.9 - break - can reassemble
643							28.2 - break - can reassemble
							28.5 - break - can reassemble
							28.6 - break - can reassemble

(continued)

ELEVATION (FEET)	DEPTH BELOW TOP OF DECK (FEET)		SAMPLE (RUN) NUMBER	PERCENT CORE RECOVERY	R.Q.D., PERCENT	SYMBOLIC LOG	NOTES
643	28						
642							29.2 - break - can reassemble
641	30		7	100	92		30.0 - break - can reassemble
640							31.1 - break - can reassemble
							31.6 - break - can reassemble
	32						31.9 - break - can reassemble
639							
638							33.1 - break - porous concrete-some grinding-cannot reassemble break
	34						33.8 - break - can reassemble (aggregate)
637			8	100	98		34.3 - double fractures 1/2' apart-broken core-cannot reassemble-some staining or weathering along fractures
636							
635	36						35.7 - break - minor grinding but can reassemble aggregate
634							37.0 - break - can reassemble (aggregate)
							37.6 - break - can reassemble
633	38						38.7 - break - can reassemble
632			9	100	100		
631	40						40.0 - break - can reassemble
630							40.7 - break - can reassemble
	42						
629							42.2 - break - minor grinding - can reassemble

(continued)

ELEVATION (FEET)	DEPTH BELOW TOP OF DECK (FEET)		SAMPLE (RUN) NUMBER	PERCENT CORE RECOVERY	R.Q.D. PERCENT	SYMBOLIC LOG	NOTES
629	42						42.2 - break - minor grinding - can reassemble
628							
627	44		10	100	100		44.5 - break - minor grinding - cannot reassemble
626							45.7 - break - minor grinding - can reassemble
625	46						46.1 - break - can reassemble
624							46.9 - break - can reassemble
623	48						47.4 - break - can reassemble
622							
621	50		11	100	100		50.7 - break - can reassemble (aggregate)
620							
619	52						51.9 - break - can reassemble (aggregate)
618							52.5 - break - minor grinding - can reassemble (aggregate)
617	54						53.6 - 53.8 - break & honey comb concrete - possibly as much as 2" of core lost
616			12	97	97		54.5 - break - can reassemble (aggregate)
615	56						55.9 - break - can reassemble (aggregate)
							56.7 - break - can reassemble

(continued)

ELEVATION (FEET)	DEPTH BELOW TOP OF DECK (FEET)	SAMPLE (RUN) NUMBER	PERCENT CORE RECOVERY	R.Q.D. PERCENT	SYMBOLIC LOG	NOTES
615	56	12	97	97		55.0 - break - can reassemble (aggregate)
614						56.7 - break - can reassemble
613	58					57.3 - break - can reassemble
612						57.7 - break - minor grinding-cuttings on top of SA13 can reassemble
611	60	13	100	100		59.4 - break - can reassemble
610						60.8 - break - minor tortional grinding-porous concrete can only partially reassemble core
609	62					
608						62.8 - break - can reassemble
607	64					63.0 - break - can reassemble
606						
605	66	14	100	94		64.2 - break - can reassemble
604						64.9 - break - can reassemble
603	68	15	60	0		65.8 - break - minor grinding - can reassemble (aggregate)
602						67.9-68.1 - aggregate-top & bottom of zone honeycombed
601	70	16	100	91		68.3 - break - top of SA16 covered with cuttings can reassemble when cuttings removed
						69.4 - break - can reassemble
						70.8 - break - trace of wood fibres-can at best only partially reassemble
						70.9 - break - can reassemble (continued)

ELEVATION (FEET)	DEPTH BELOW TOP OF DECK (FEET)		SAMPLE (RUN) NUMBER	PERCENT CORE RECOVERY	R.Q.D., PERCENT	SYMBOLIC LOG	NOTES
601	70	CONCRETE BELOW ABOUT ELEV. 603.5 POOR QUALITY THAN CONCRETE ABOVE	16	100	91		70.8 - break - trace of wood fibres-can at best only partially reassemble
600							70.9 - break - can reassemble
							71.4 - break - porous concrete - can reassemble
599	72		17	100	94		72.4 - break - porous concrete - can reassemble
							72.6 - break - porous concrete - can reassemble
598							73.0 - break - can reassemble
597	74						74.0 - break - slightly porous concrete - can reassemble
596							75.0 - break - slightly porous concrete - can reassemble
595	76						76.1-76.2 - 1" thick board at about 12° to horizontal- good concrete contact
594			18	100	83		77.3 - break - can reassemble
593	78						78.0 - break - trace of wood fibres-possible partial void- can only partially reassemble core
592							78.2 - break - minor grinding - can reassemble (aggregate)
591	80						79.7 to 80.0 - zone of highly porous broken concrete
590							81.0 - break-porous concrete-wood fibre-can only partially reassemble
589	82		19	100			81.5 - break - can reassemble
588							82.0 - break - can reassemble
							82.6 - break - can reassemble
587	84						83.3 - break-partial void or core loss-can only partially reassemble
			20	0	0		83.6 - wood fibre - no break
							83.9 - 84.3 - 1/2" thick steel plate inclined at about 80° to horizontal-appears to be good concrete contact but core badly broken
							84.8 - break - can reassemble

(continued)

ELEVATION (FEET)	DEPTH BELOW TOP OF DECK (FEET)		SAMPLE (RUN) NUMBER	PERCENT CORE RECOVERY	R.Q.D., PERCENT	SYMBOLIC LOG	NOTES
587	84						
586			21	100	100		84.8 - break - can reassemble
							85.6 - break - can reassemble
	86						Bottom of Sound Concrete
585			0	0			Void (infilled)
584							Bedrock Surface
583	88						
582							
581	90						
580							

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>TC</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_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

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	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 = $c_s t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion intercept
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_i	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 2

LOCATION See Figure 43

BORING DATE

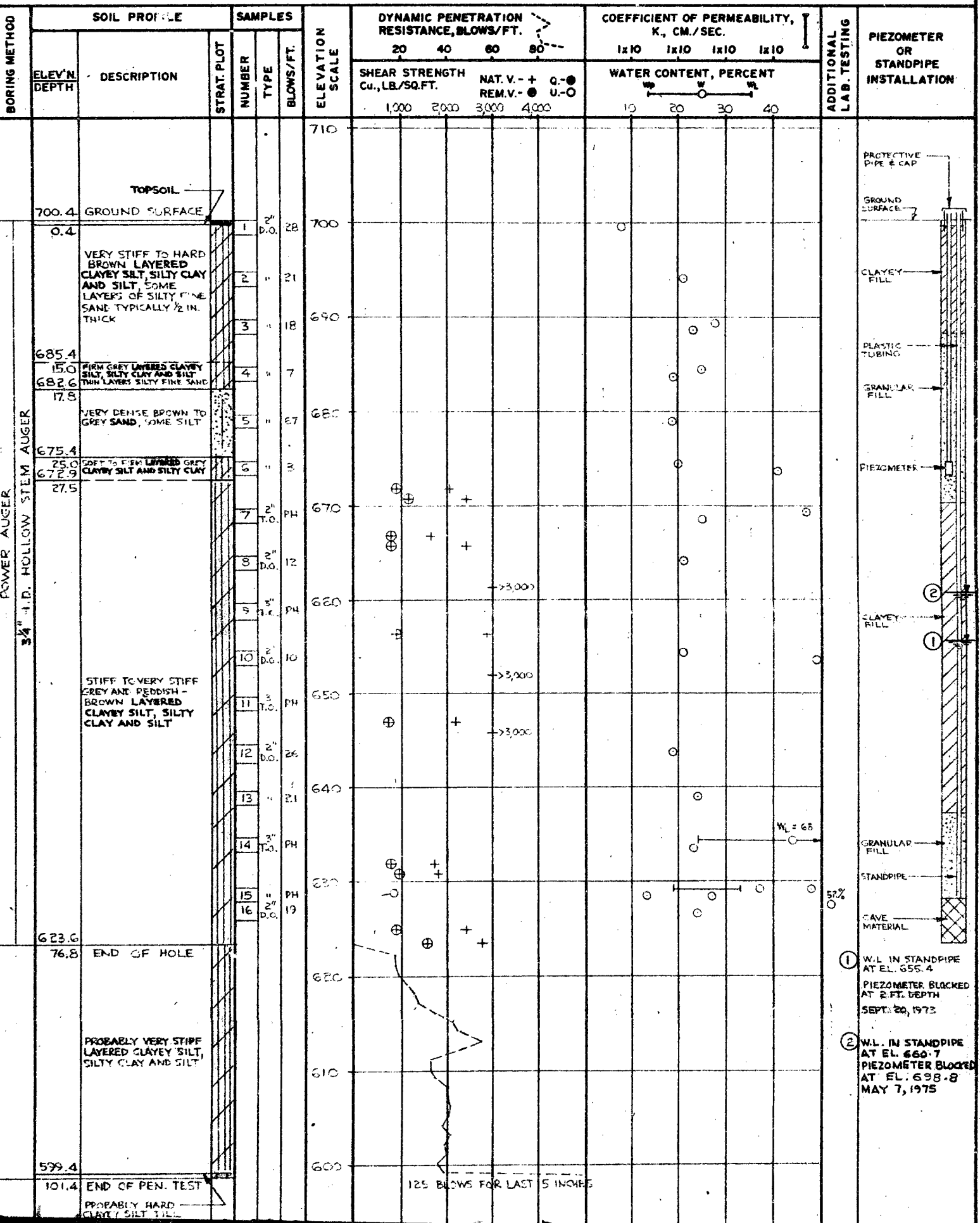
SEPT. 11-12, 1973

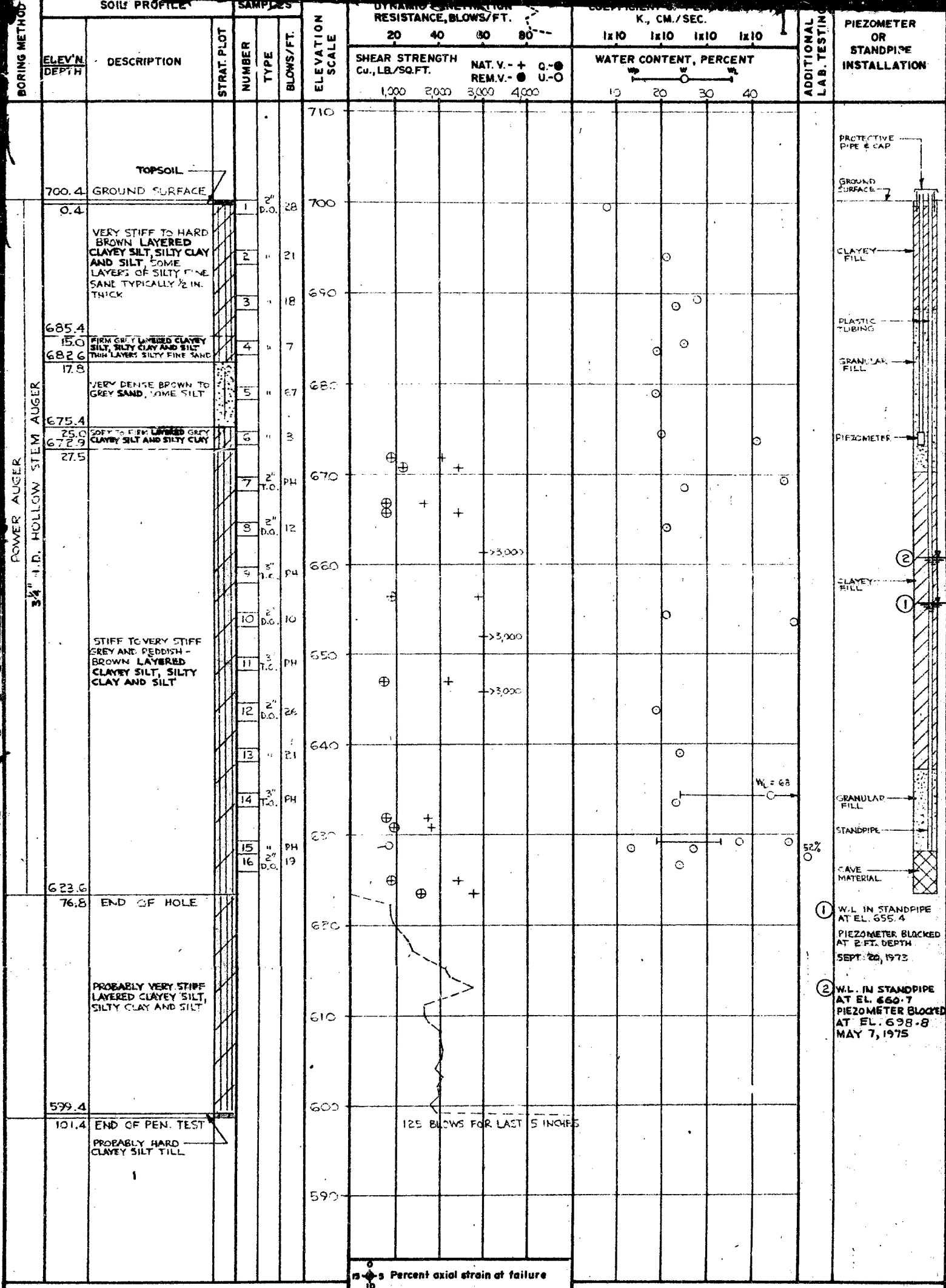
DATUM

GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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





RECORD OF BOREHOLE 101

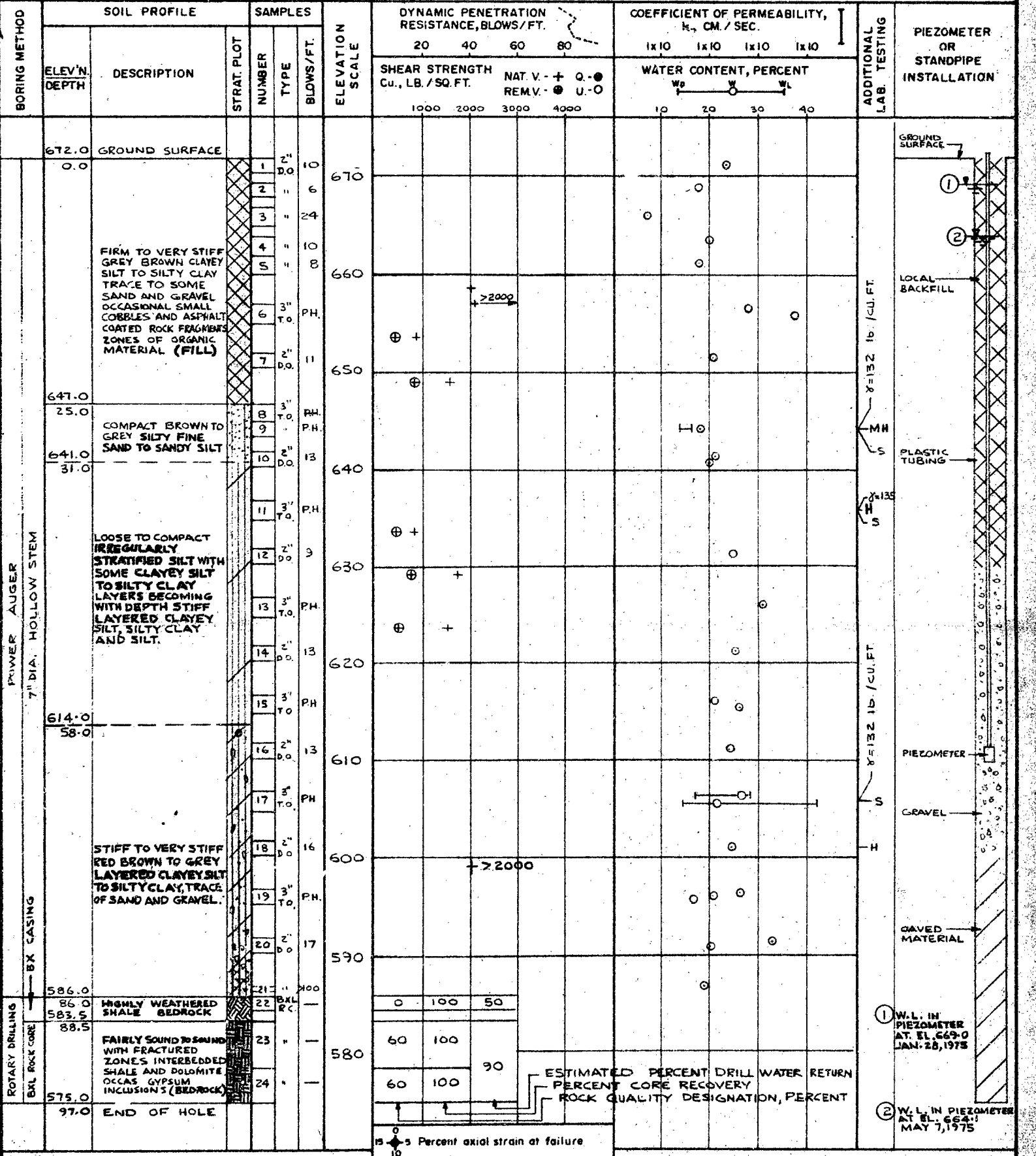
LOCATION See Figure 3

BORING DATE JAN. 2 - 9, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



RECORD OF BOREHOLE 102

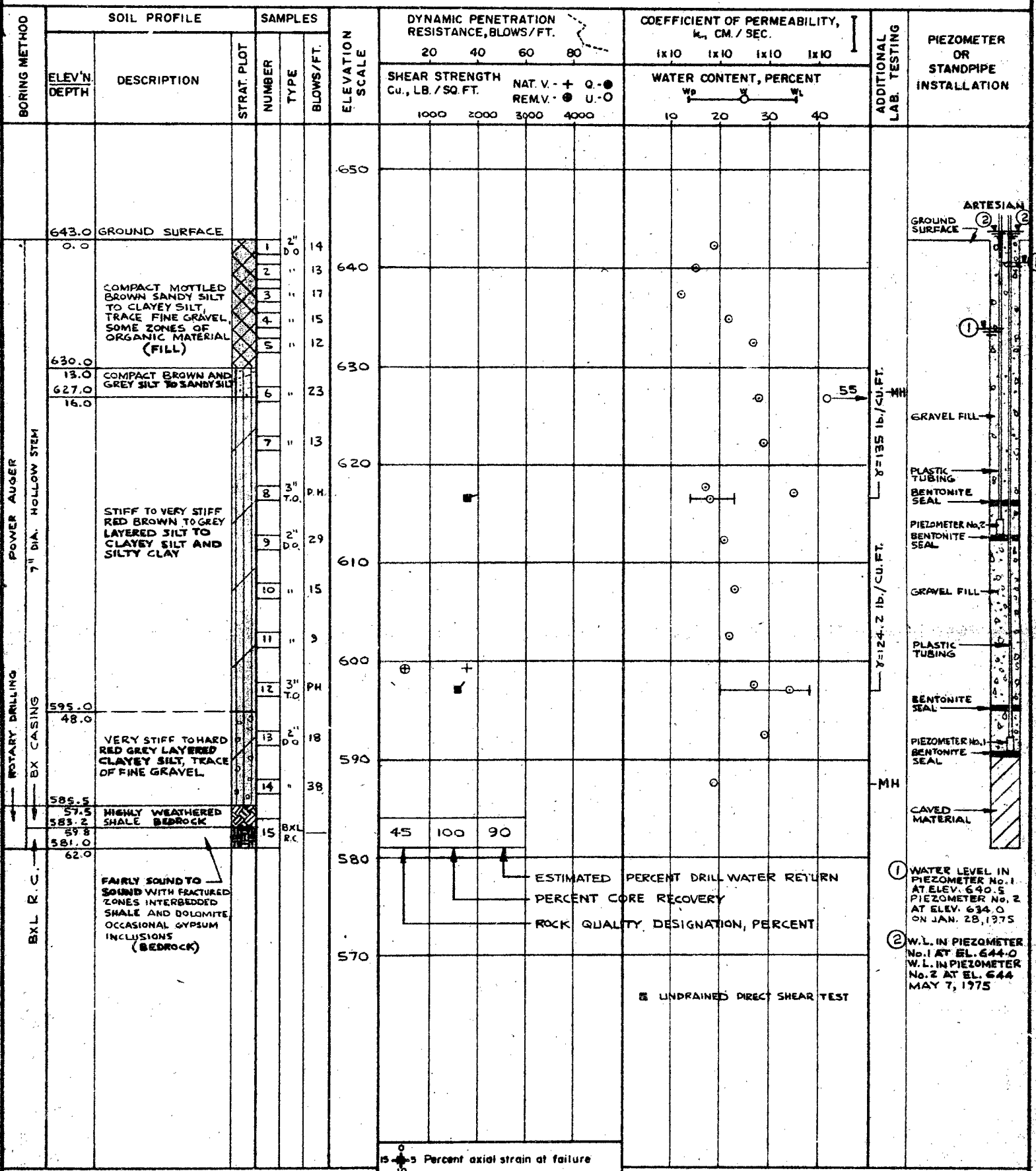
LOCATION See Figure 3

BORING DATE JAN. 9, 10 & 13, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

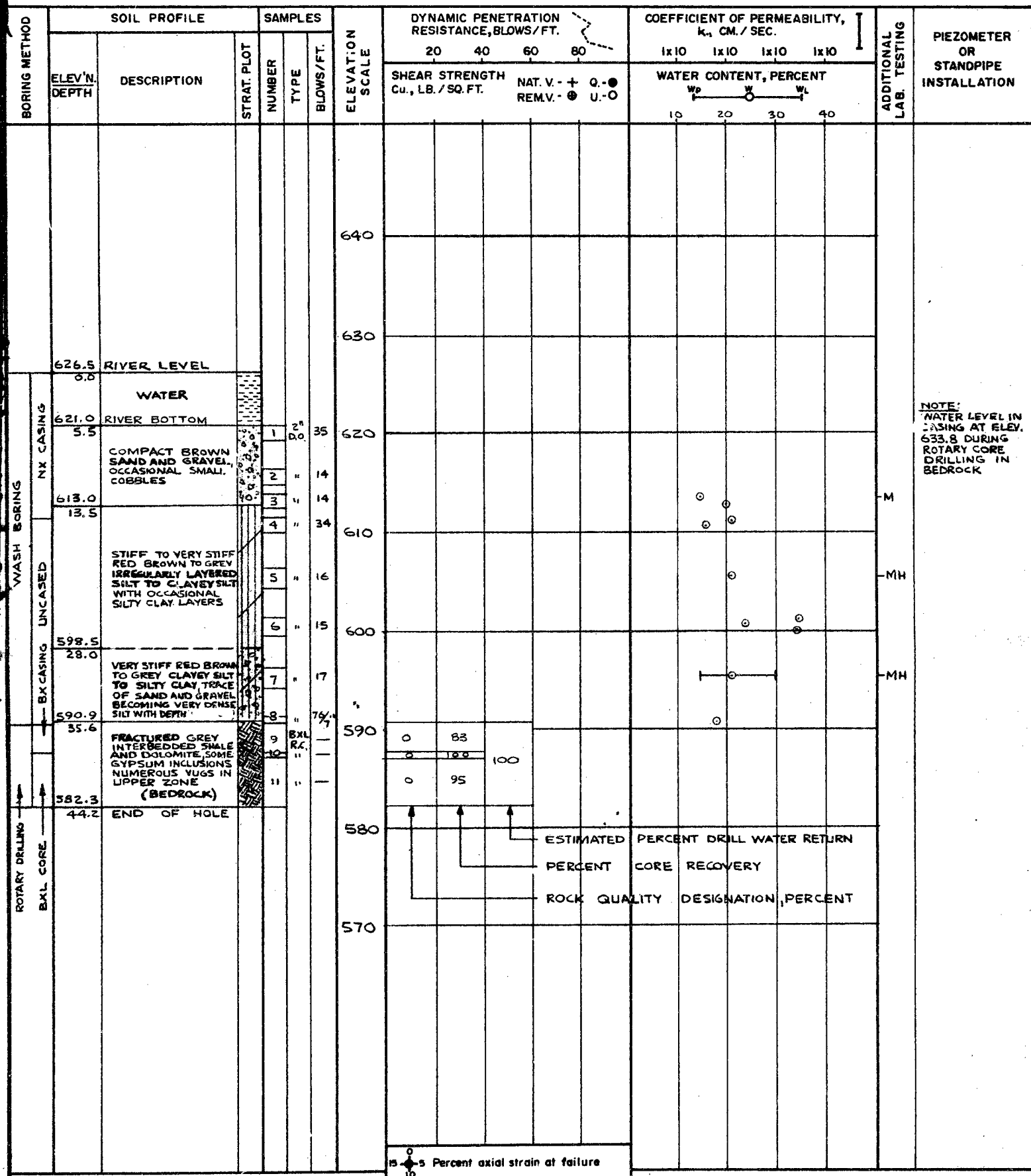


VERTICAL SCALE
LIN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED R.G.

LOCATION	See Figure 3	BORING DATE	JAN. 2 & 4, 1975	DATUM	GEODETIC
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.			PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.		



VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED RG

RECORD OF BOREHOLE 104

LOCATION See Figure 3 BORING DATE DEC. 23, 24 & 30, 1975 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_v , 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 C_u , LB./SQ. FT.				WATER CONTENT, PERCENT					
								1000 2000 3000 4000					10 20 30 40				
POWER AUGER 7" DIA. HOLLOW STEM	632.7	GROUND SURFACE					640										
	0.0	LOOSE DARK BROWN SAND TO SILTY SAND, NUMEROUS ROOTS IN UPPER 2 FT.		1	2" D.O.	5	630										
	625.5			2	"	7											
	7.2	COMPACT TO DENSE DARK GREY TO BROWN SAND SOME GRAVEL OCCASIONAL SMALL COBBLES, TRACE OF ORGANIC MATTER IN UPPER ZONE		3	"	5											
	618.2			4	"	13											
	14.5			5	"	38											
		STIFF TO VERY STIFF RED BROWN TO GREY IRREGULARLY LAYERED SILT TO CLAYEY SILT OCCASIONAL SILTY CLAY LAYERS		6	"	13											
	607.7			7	3" P.H.		620										
	25.0			8	2" D.O.	13											
		VERY STIFF RED BROWN TO GREY OCCASIONALLY LAYERED CLAYEY SILT TO SILTY CLAY, TRACE OF SAND AND GRAVEL		9	"	27											
	599.7						610										
	33.0	VERY STIFF TO HARD RED BROWN TO GREY LAYERED SILT, CLAYEY SILT AND SILTY CLAY		10	"	38											
	594.2						600										
	38.5	END OF HOLE					590										
		REFUSAL TO AUGER PROBABLY BEDROCK															

ARTESIAN

GROUND SURFACE

PLASTIC TUBING

STANDPIPE

PLASTIC TUBING

LOCAL BACKFILL

BENTONITE SEAL

PEA GRAVEL

PIEZOMETER

① WATER LEVEL IN STANDPIPE AT ELEV. 631.0
PIEZOMETER FROZEN TO ELEV. 634.0 ON JAN. 28, 1975

② W.L. IN STANDPIPE AT EL. 630.4
W.L. IN PIEZOMETER AT EL. 633.7
MAY 7, 1975

0
15 30 45 Percent axial strain at failure

0
 15 10 Percent axial strain at failure

VERTICAL SCALE
 1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
 CHECKED R.G.

LOCATION See Figure 3 BORING DATE DEC. 20 & 23, 1974 DATUM GEODETIC
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

DRAWN D. M.
CHECKED RG

RECORD OF BOREHOLE 106

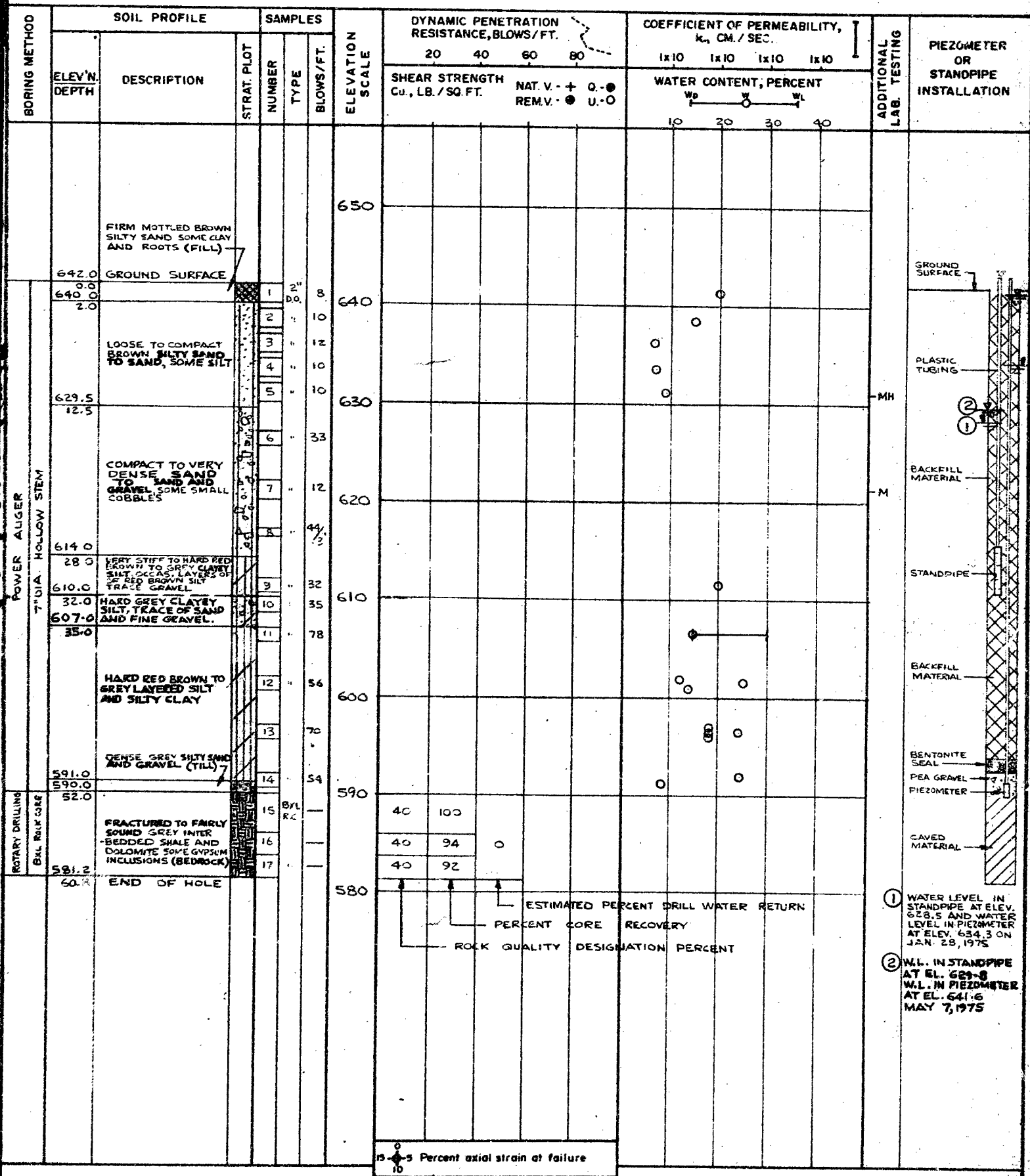
LOCATION See Figure 3

BORING DATE DEC. 16, 17, 18 & 20, 1974

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED R.G.

RECORD OF BOREHOLE 107

DATUM GEODETIC

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

[illegible]

DRAWN D.M.
CHECKED EG

RECORD OF BOREHOLE ZOI

LOCATION See Figure 3

BORING DATE MAR. 26 TO APRIL 2, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

[illegible]

VERTICAL SCALE
LIN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED B.G.

RECORD OF BOREHOLE 202

SLOPE INDICATOR CASING No.1

LOCATION See Figure 3

BORING DATE

APRIL 1-8, 1975

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	INITIAL SHAPE OF CASING NORTH-SOUTH DIRECTION * (APRIL 10, 1975)	INITIAL SHAPE OF CASING EAST - WEST DIRECTION * (APRIL 10, 1975)	ADDITIONAL LAB. TESTING	SLOPE INDICATOR CASING INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.					
ROTARY DRILLING N X CASING	648.5	GROUND SURFACE						<div>DEFLECTION OF CASING (FEET)</div> <div>SOUTH NORTH WEST EAST</div> <div></div>	<div>DEFLECTION OF CASING (FEET)</div> <div>SOUTH NORTH WEST EAST</div> <div></div>	<div>LID WITH HINGES AND PADLOCK</div> <div>GROUND SURFACE</div> <div>CEMENT SEAL</div> <div>PEA GRAVEL</div> <div>CAVED MATERIAL</div> <div>LOCAL BACKFILL</div> <div>CAVED MATERIAL</div> <div>JOINTS OR COUPLING 5 FT. SECTIONS</div> <div>LOCAL BACKFILL</div> <div>TIGHT FIT IN BEDROCK</div>	<div>W.L. IN SLOPE INDICATOR CASING AT EL. 638.3 MAY 7, 1975.</div>
	640.5 8.0	SILT AND CLAYEY SILT (PROBABLY FILL)					640				
	631.5 170	HOLE CAVED PROBABLY SILT OR SANDY SILT (POSSIBLY FILL)					630				
	618.5 30.0	PROBABLY STIFF CLAYEY SILT AND SILTY CLAY.					620				
	613.5 35.0	HOLE CAVED PROBABLY SILT OR SANDY SILT					610				
	595.5 53.0	PROBABLY STIFF CLAYEY SILT AND SILTY CLAY					600				
	589.5 59.0	PROBABLY VERY STIFF CLAYEY SILT SOME GRAVEL					590				
	585.2 63.3	SHALE BEDROCK (POSSIBLY BOULDER)					580				
	579.5 69.0	FRACTURED TO FAIRLY SOUND DOLOMITE (BEDROCK)					570				
		END OF HOLE									

* SLOPE ASSUMED TO RUN EAST-WEST FOR SLOPE INDICATOR READING PURPOSES

VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED R.G.

RECORD OF BOREHOLE 203

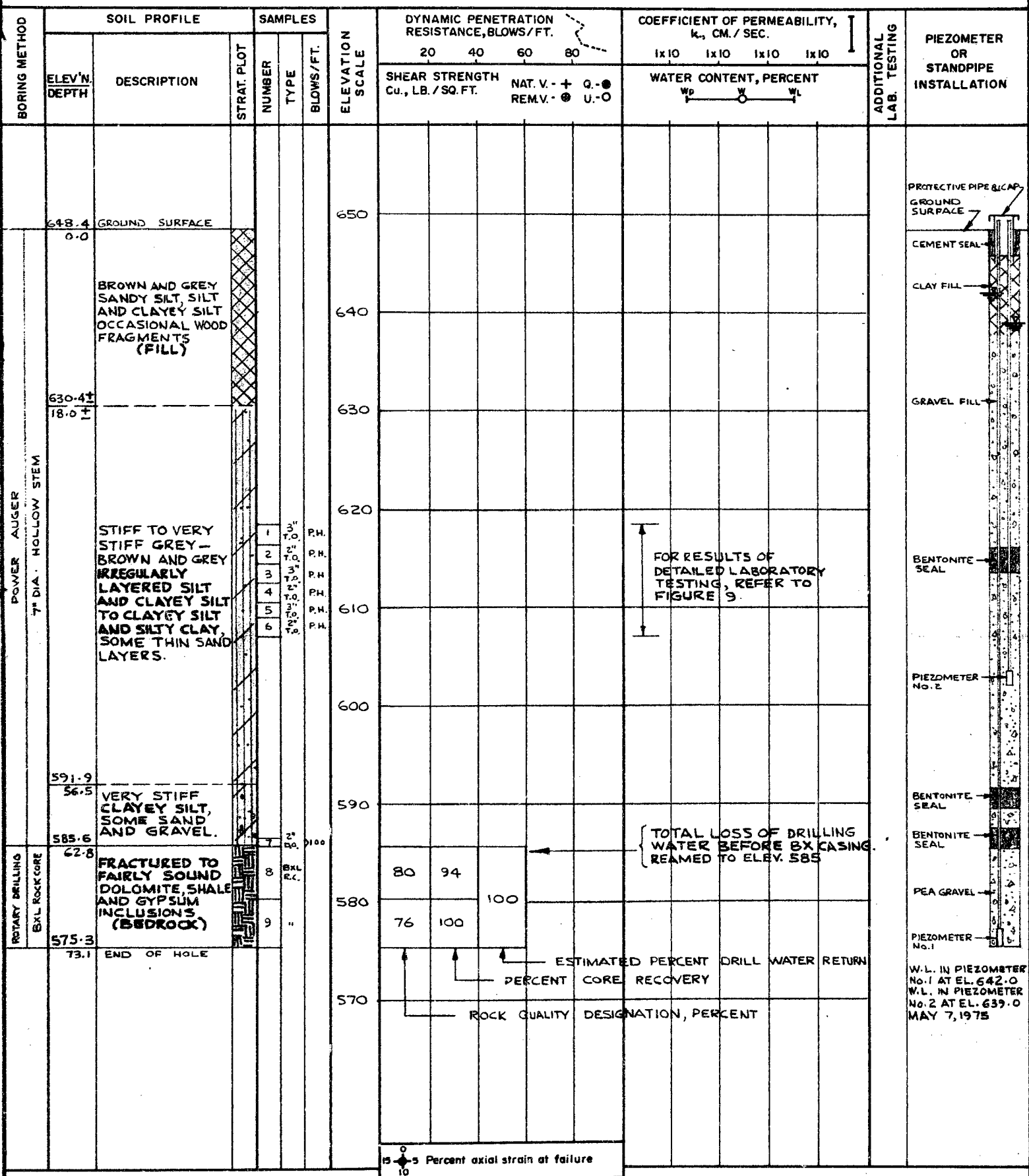
LOCATION See Figure 3

BORING DATE APRIL 9-11, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED RG

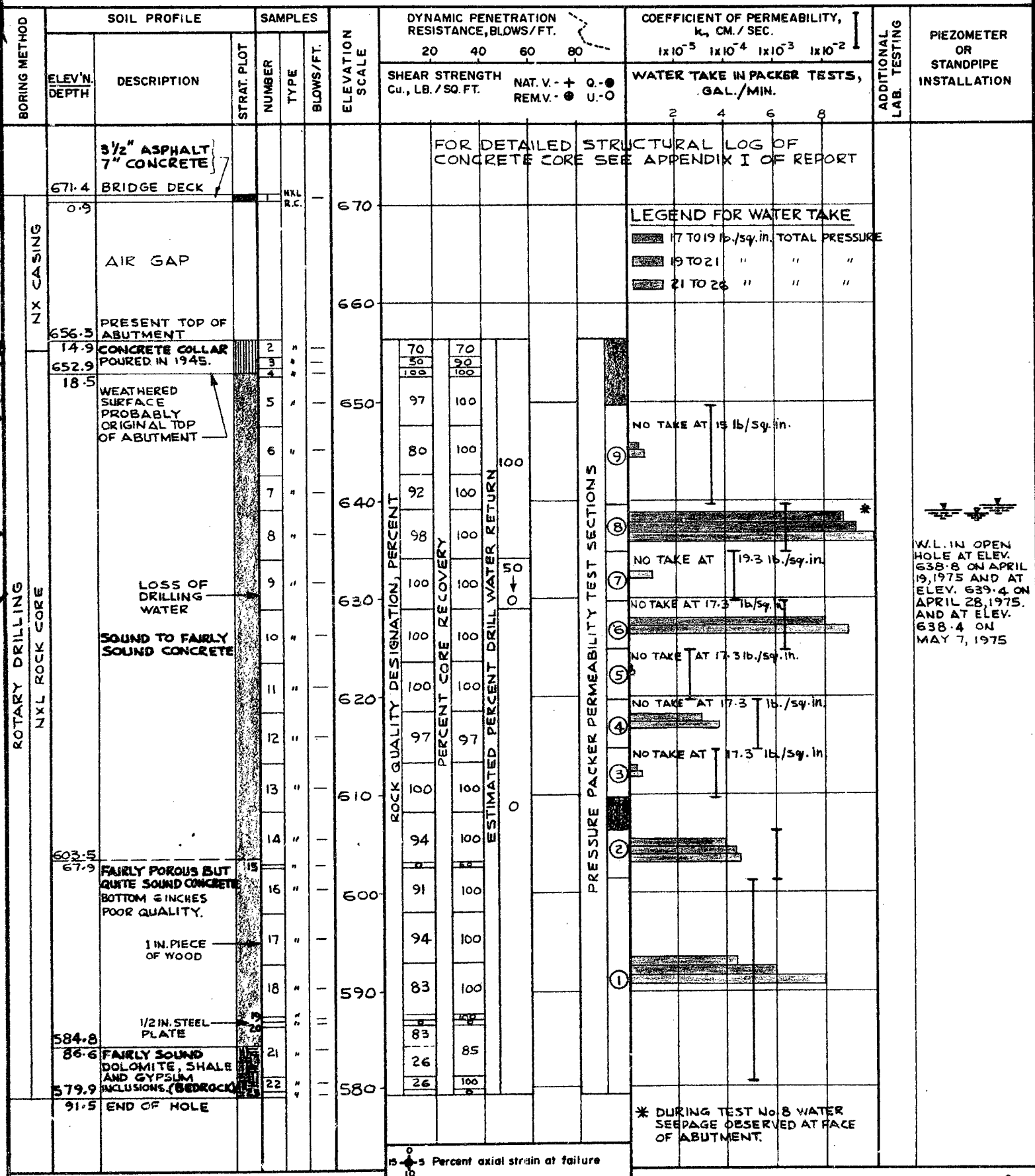
RECORD OF BOREHOLE 204

LOCATION See Figure 3

BORING DATE APRIL 10 TO 12, 16, 19 & 28, 1975 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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


 VERTICAL SCALE
 1 IN. TO 10 FT.

Golder Associates

 DRAWN *m.j.b.*
 CHECKED *B.G.*

RECORD OF BOREHOLE 205

SLOPE INDICATOR CASING No. 2

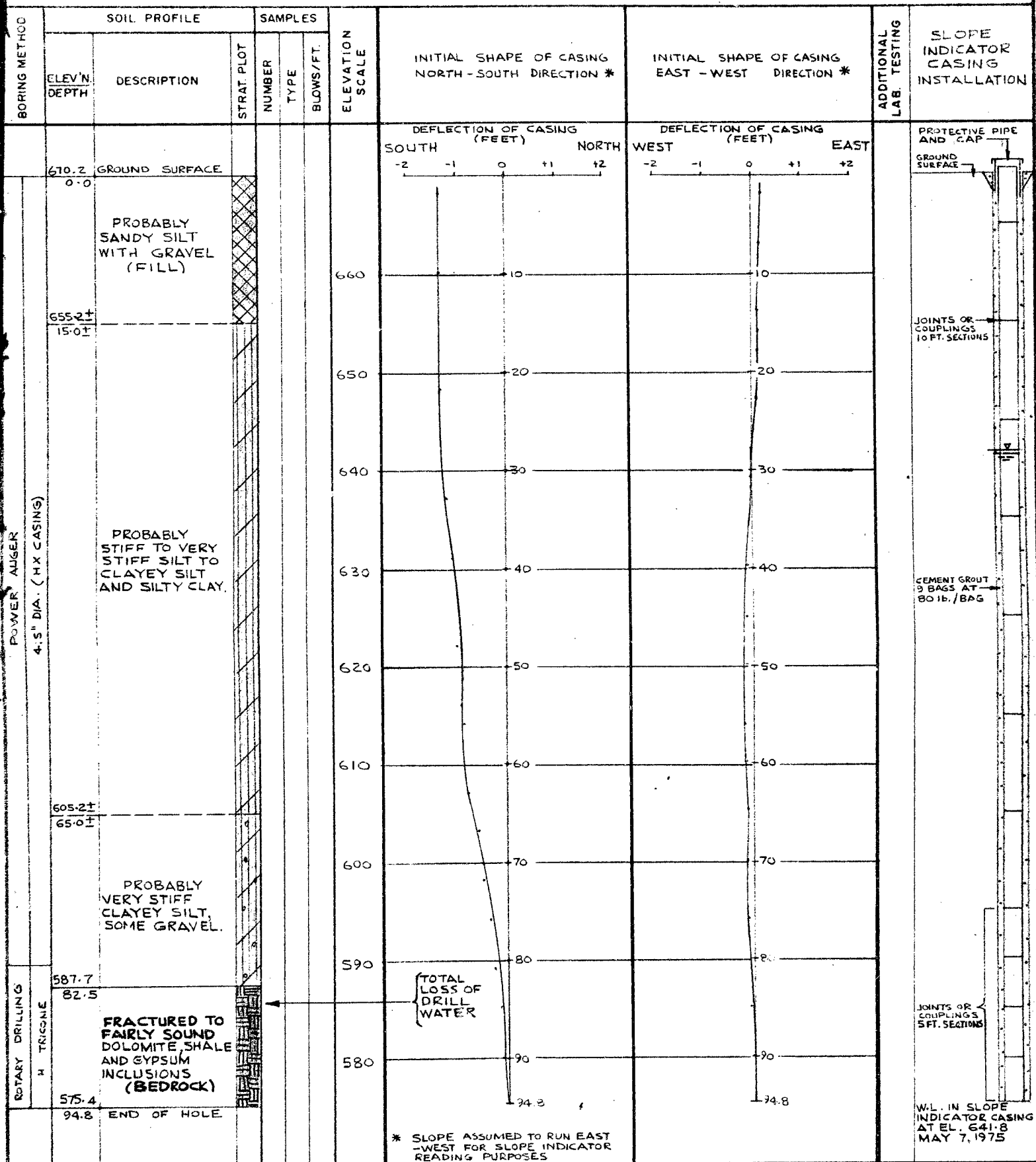
LOCATION See Figure 3

BORING DATE APRIL 11, 12, 14 & 15, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB, DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB, DROP 30 IN.


 VERTICAL SCALE
1 IN TO 10 FT

Golder Associates

 DRAWN D.M.
CHECKED BG

RECORD OF BOREHOLE 206

LOCATION See Figure 3

BORING DATE

APRIL 2, 3 & 4, 1975

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_v , 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 C_u , LB. / SQ. FT.				WATER CONTENT, PERCENT					
								1000	2000	3000	4000	10	20	30	40		
							720										
							710										
	708.0	GROUND SURFACE															
	0.5																
		VERY STIFF BROWN CLAYEY SILT WITH SANDY SILT LAYERS		1	2"	22											
				2	"	32											
				3	"	21	700										
				4	"	22											
	693.0			5	"	41											
	15.0			6	"	17	690										
				7	2"	P.H.											
				8	"	P.H.											
		VERY STIFF TO STIFF BROWNISH GREY TO GREY IRREGULARLY LAYERED CLAYEY SILT AND SILTY CLAY, SOME THIN SAND LAYERS.		9	2"	39											
				10	"	19											
				11	2"	P.H.	670										
				12	2"	P.H.											
				13	"	8											
	662.0																
	46.0	END OF HOLE					660										
							650										

POWER AUGER
(7.5" DIA. HOLLOW STEM)

TOP SOIL

GROUND SURFACE

CLAY BACKFILL

PEA GRAVEL

PIEZOMETER 2

PEA GRAVEL

BENTONITE SEAL

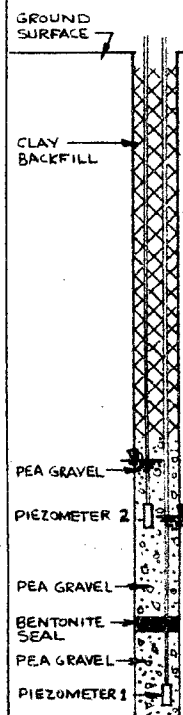
PEA GRAVEL

PIEZOMETER 1

W.L. IN PIEZOMETER NO. 1 AT EL. 676.8 AND IN NO. 2 AT EL. 680.5 ON MAY 7, 1975

0 5 Percent axial strain at failure

0 5 Percent axial strain at failure



W.L. IN PIEZOMETER No. 1 AT EL. 676.8 AND IN No. 2 AT EL. 680.5 ON MAY 7, 1975

VERTICAL SCALE 1 IN. TO 10 FT.

Golder Associates

DRAWN A.M. CHECKED B.G.

LOCATION See Figure 3

BORING DATE

APRIL 7, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED BG

RECORD OF BOREHOLE 209

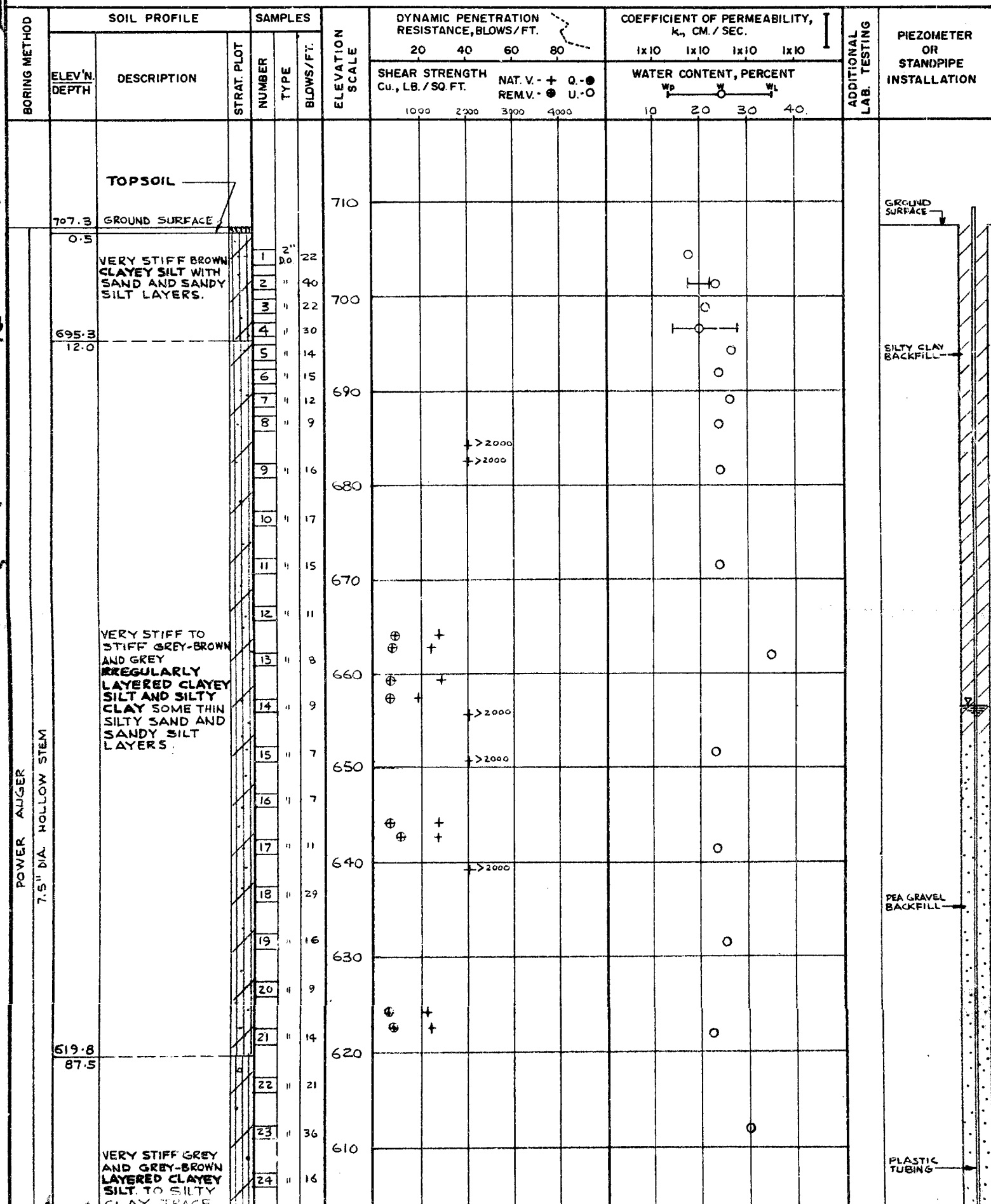
LOCATION See Figure 3

BORING DATE APRIL 16, 17, 19 & 20, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



POWER AUGER
7.5" DIA. HOLLOW STEM

ROTARY DRILLING
BXL ROCK CORE

VERY STIFF TO
STIFF GREY-BROWN
AND GREY
IRREGULARLY
LAYERED CLAYEY
SILT AND SILTY
CLAY SOME THIN
SILTY SAND AND
SANDY SILT
LAYERS

VERY STIFF GREY
AND GREY-BROWN
LAYERED CLAYEY
SILT TO SILTY
CLAY, TRACE
GRAVEL

HARD RED BROWN TO
GREY LAYERED
CLAYEY SILT AND
SILTY CLAY

FRACTURED TO
FAIRLY SOUND
DOLOMITE, SHALE
AND GYPSUM
INCLUSIONS (BEDROCK)

519.8
87.5

592.3
115.0

580.3

127.0

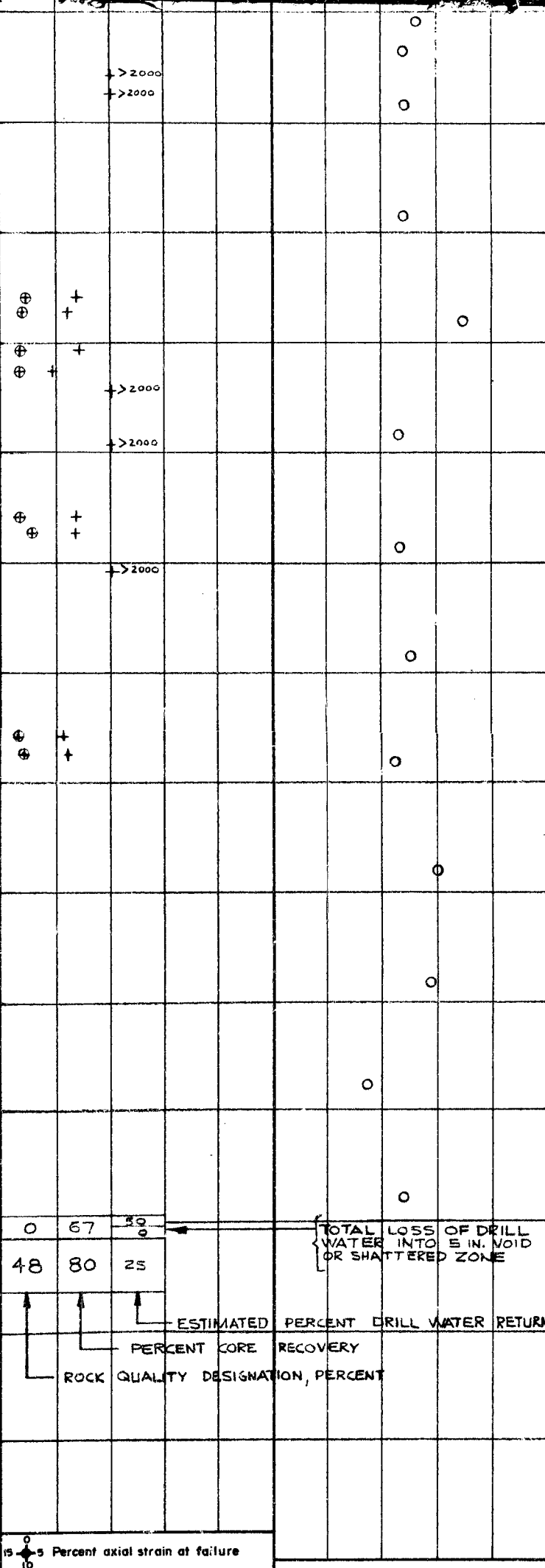
573.4

133.9

END OF HOLE

7	"	12
8	"	9
9	"	16
10	"	17
11	"	15
12	"	11
13	"	8
14	"	9
15	"	7
16	"	7
17	"	11
18	"	29
19	"	16
20	"	9
21	"	14
22	"	21
23	"	36
24	"	16
25	"	21
26	"	18
27	"	85
28	"	97
29	"	100
30	BXL	R.C.
31	"	—

690
680
670
660
650
640
630
620
610
600
590
580
570
560



PEA GRAVEL
BACKFILL

PLASTIC
TUBING

BENTONITE
SEAL

PIEZOMETER

W. L. IN PIEZOMETER
AT ELEV. 656.5
ON MAY 7, 1975

0
15 5 Percent axial strain at failure
10

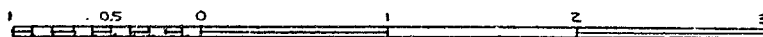
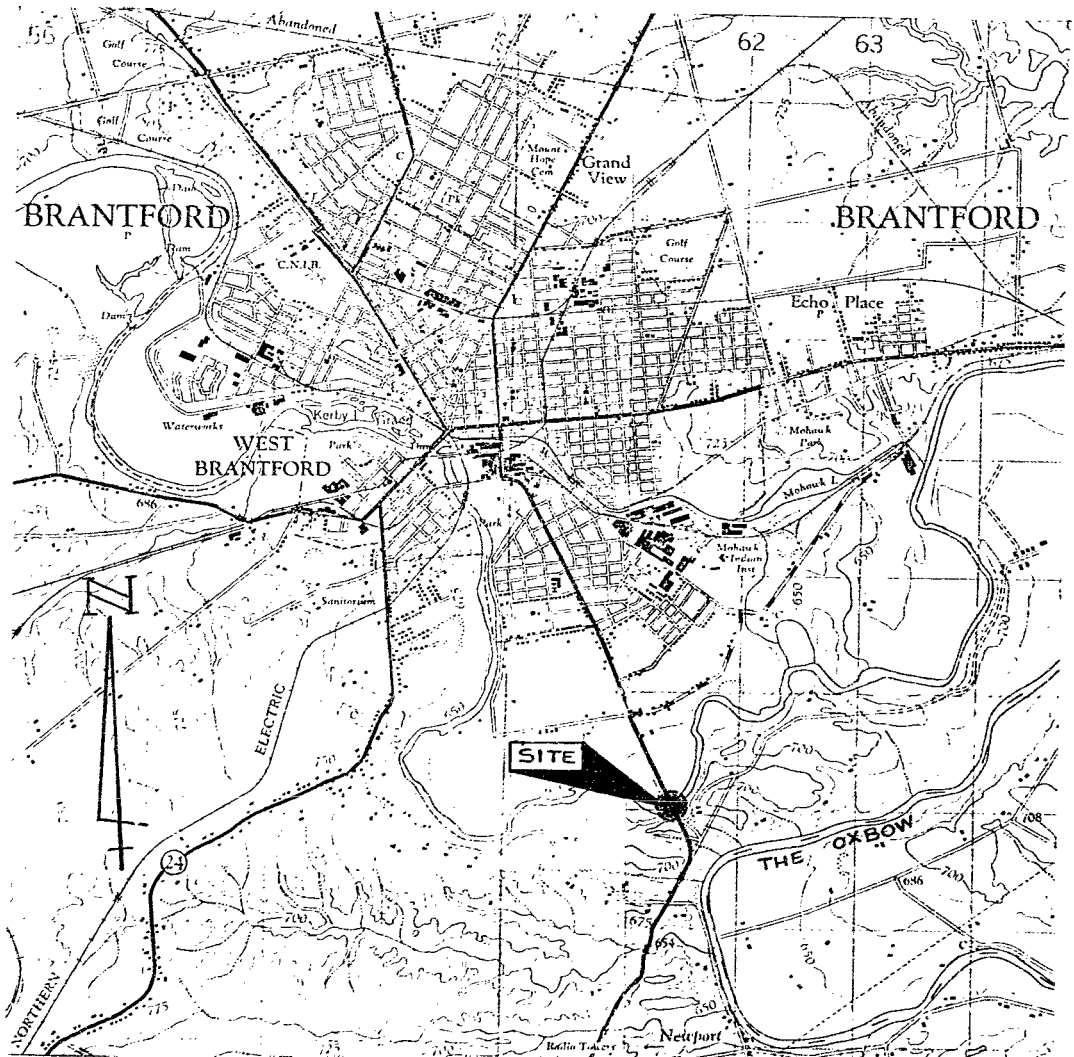
VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN M.
CHECKED BG

KEY PLAN

FIGURE 1

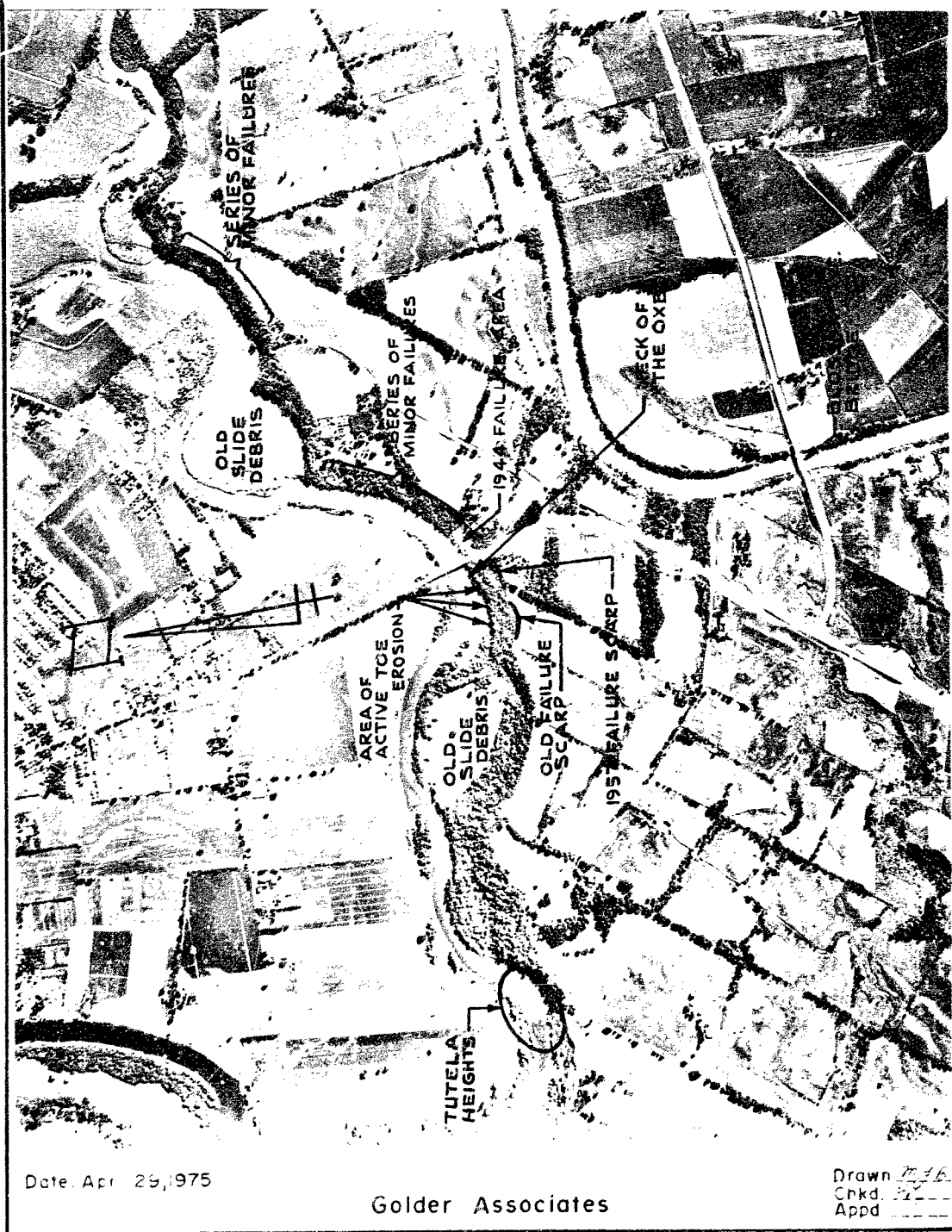


SCALE: 1 to 50,000

Date JAN. 22, 1975

Golder Associates

Drawn D.M.
 Chkd. AK
 Appd. _____

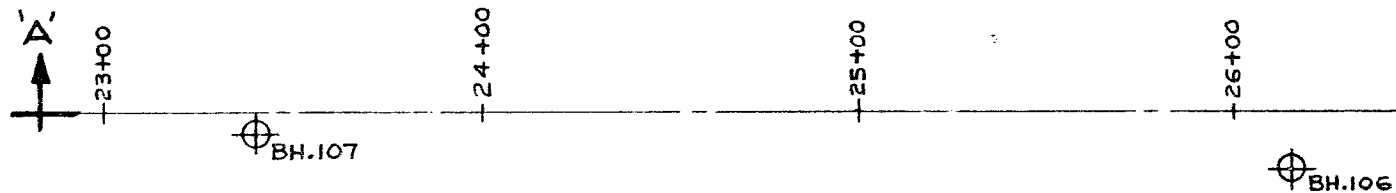


Date: Apr. 29, 1975

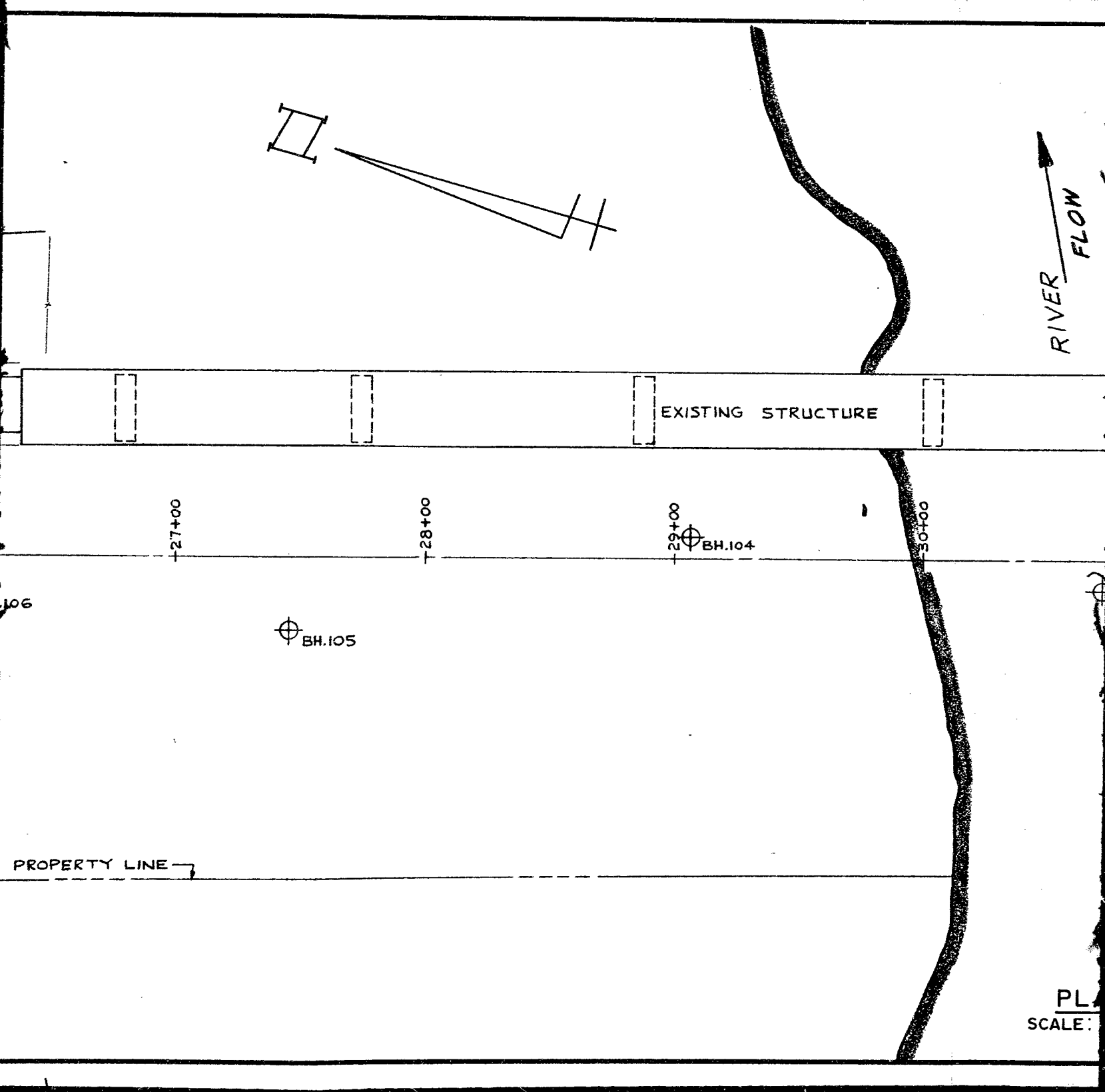
Golder Associates

Drawn *mfb*
 Chkd. *---*
 Appd. *---*

R/W



PROPOSED PROPERTY LINE



PL
SCALE:

RIVER
FLOW

R/W

EXISTING

BH.204

BH.202
S.I. No. 1

BH.203

BH.205
S.I. No. 2

BH.1

BH.103

BH.102

BH.101

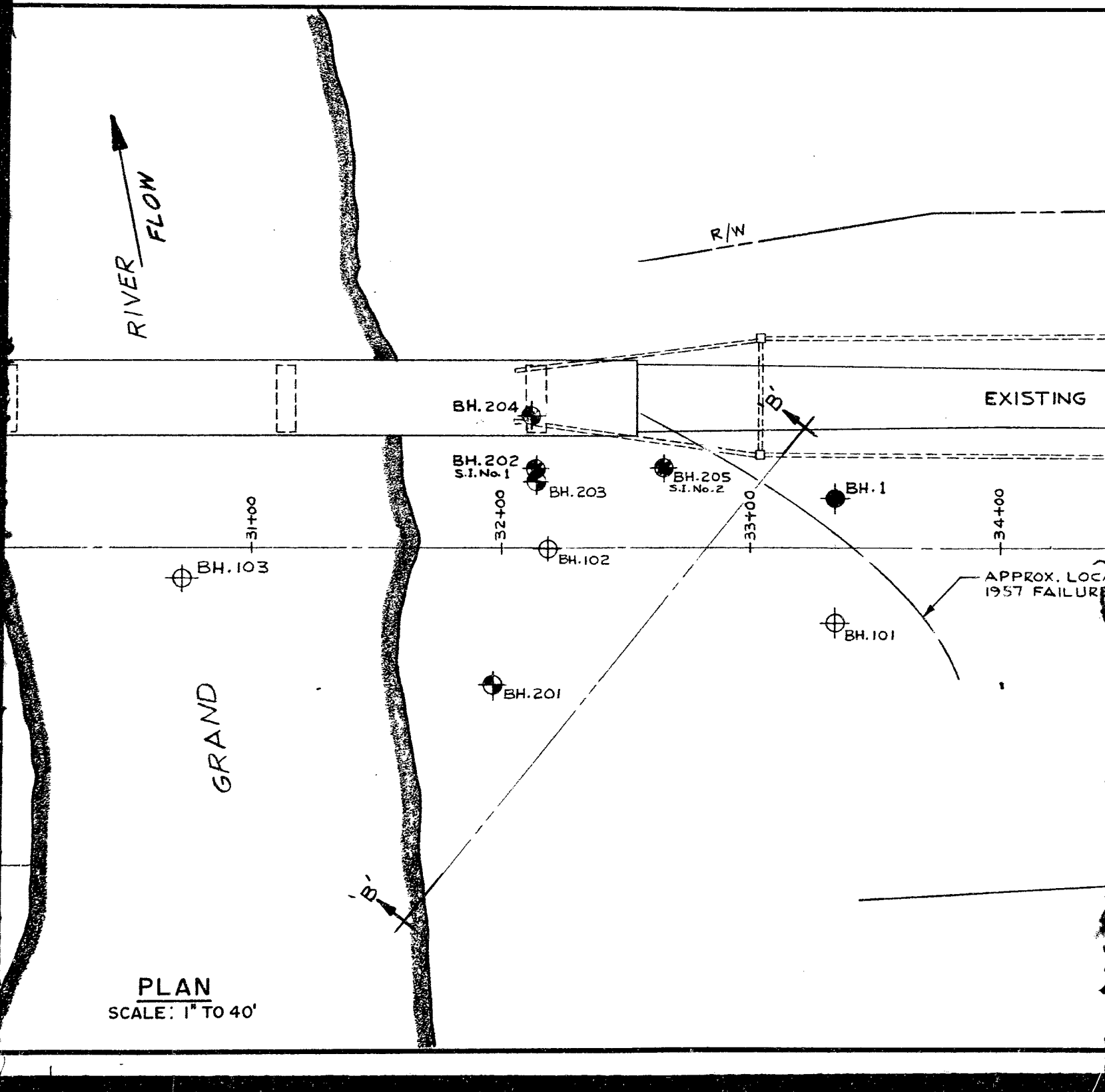
APPROX. LOC.
1957 FAILURE

BH.201

GRAND

PLAN

SCALE: 1" TO 40'



RIVER
FLOW

R/W

EXISTING

BH.204

BH.202
S.I. No. 1

BH.203

BH.205
S.I. No. 2

BH.1

BH.103

BH.102

APPROX. LOC.
1957 FAILURE

BH.101

BH.201

GRAND

PLAN

SCALE: 1" TO 40'

EXISTING COCKSHUTT ROAD

34+00

35+00

36+00

CENTRELINE PROP.

37+00

COCKSHUTT ROAD

00+8E

APPROX. LOCATION OF
1957 FAILURE SCARP

BH.209

BH.208

BH.206

FOUNDATION

BH.207

PROPOSED PROPERTY LINE

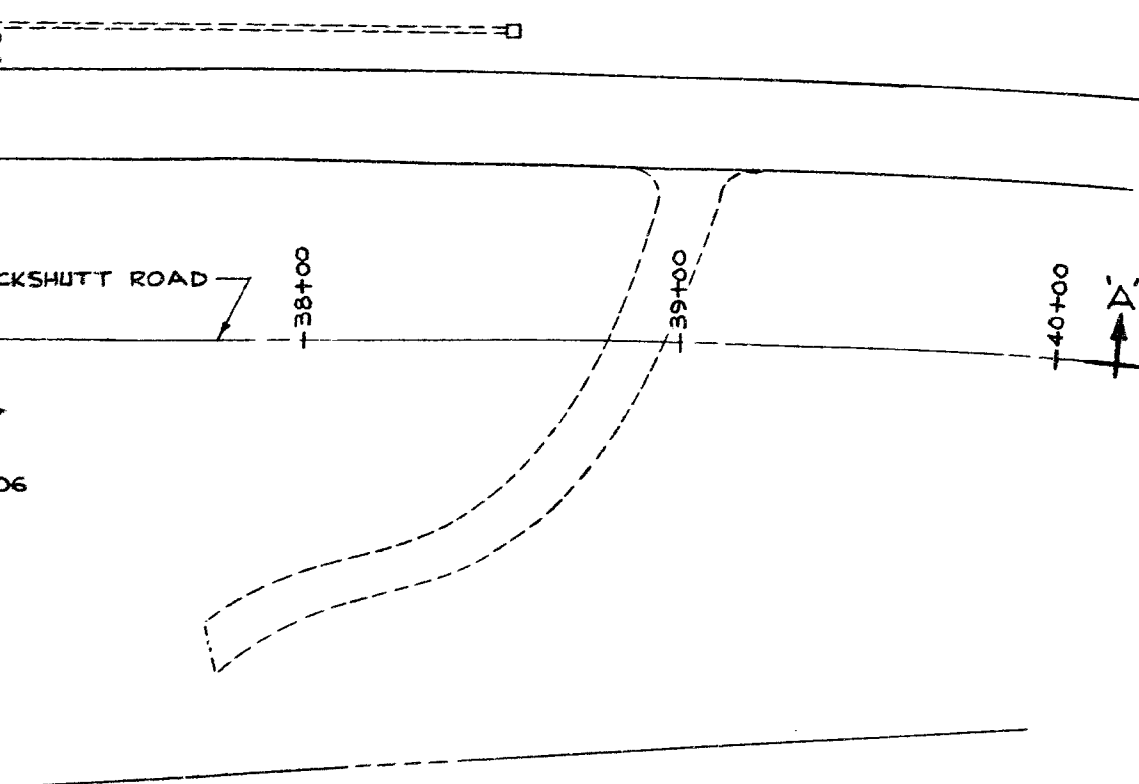
BH. 2

APPROX. 250 FT. UPSTREAM
OF CENTRELINE AT CREST OF
VALLEY BANK.

BORING PLAN

LEGEND

- BOREHOLE IN PLAN 201 TO 209 INCLUSIVE
- BOREHOLE IN PLAN WITH SLOPE INDICATOR
- BOREHOLE IN PLAN —STAGE 101 TO 107 INCLUSIVE
- BOREHOLE IN PLAN, BH. 18-2



REFERENCE:

McCORMICK, RANKIN & ASSOCIATES
FOR BRANTFORD SUBURBAN ROAD
OF BRANTFORD, "COCKSHUTT ROAD"
BLOSSOM AVENUE, PROPERTY A
NUMBER 702, SHEETS 3 AND 4

SPECIAL NOTE
THIS DRAWING IS TO BE USED
WITH ACCOMPANYING REPORT

NOTE:

FOR STRATIGRAPHIC SECTION
FIGURE 4 AND FOR SECTION
FIGURE 5.

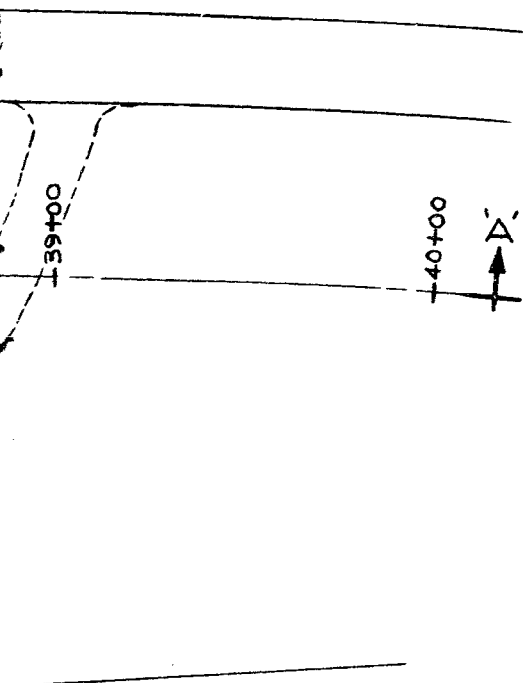
Date: APRIL 29, 1975

Golder Associates

LEGEND

- | | | | |
|---|--|------------------|--------------------------|
| <ul style="list-style-type: none"> ● BOREHOLE IN PLAN
201 TO 209 INCLUSIVE ● BOREHOLE IN PLAN
WITH SLOPE INDICATOR ● BOREHOLE IN PLAN — STAGE I
101 TO 107 INCLUSIVE ● BOREHOLE IN PLAN, BH. 18-2 | STAGE II

PREVIOUS INVESTIGATION
GOLDER ASSOC. REPORT No.
73154, DATED OCTOBER, 1973 | }
}
}
} | PRESENT
INVESTIGATION |
|---|--|------------------|--------------------------|

REFERENCE:

McCORMICK, RANKIN & ASSOCIATES LIMITED DRAWINGS
 FOR BRANTFORD SUBURBAN ROADS COMMISSION, CITY
 OF BRANTFORD, "COCKSHUTT ROAD", BIRKETT LANE TO
 BLOSSOM AVENUE, PROPERTY ACQUISITION, DRAWING
 NUMBER 702, SHEETS 3 AND 4 OF 7, UNDATED.

SPECIAL NOTE

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

NOTE:

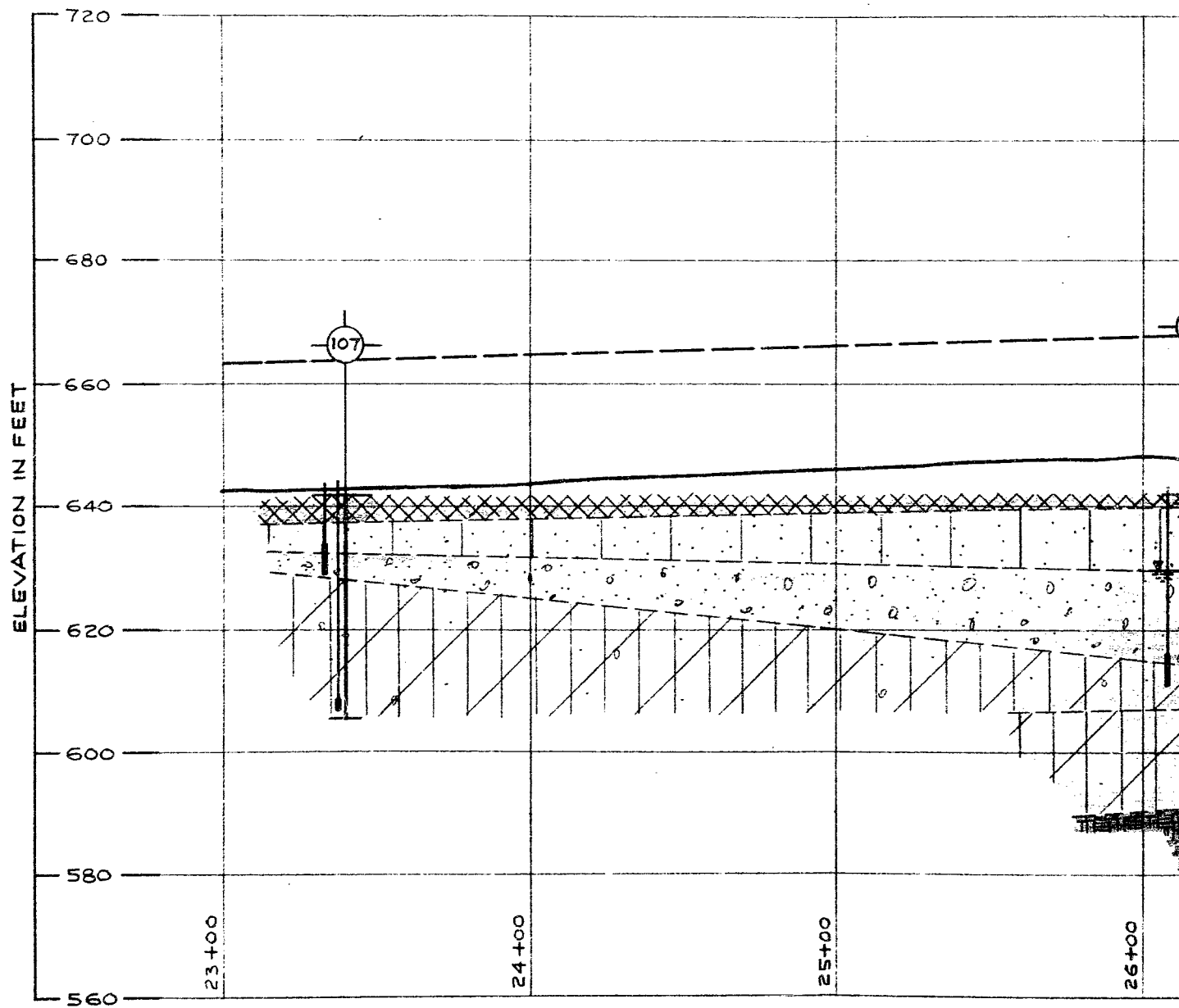
FOR STRATIGRAPHIC SECTION 'A'—'A' REFER TO
 FIGURE 4 AND FOR SECTION 'B'—'B' REFER TO
 FIGURE 5.

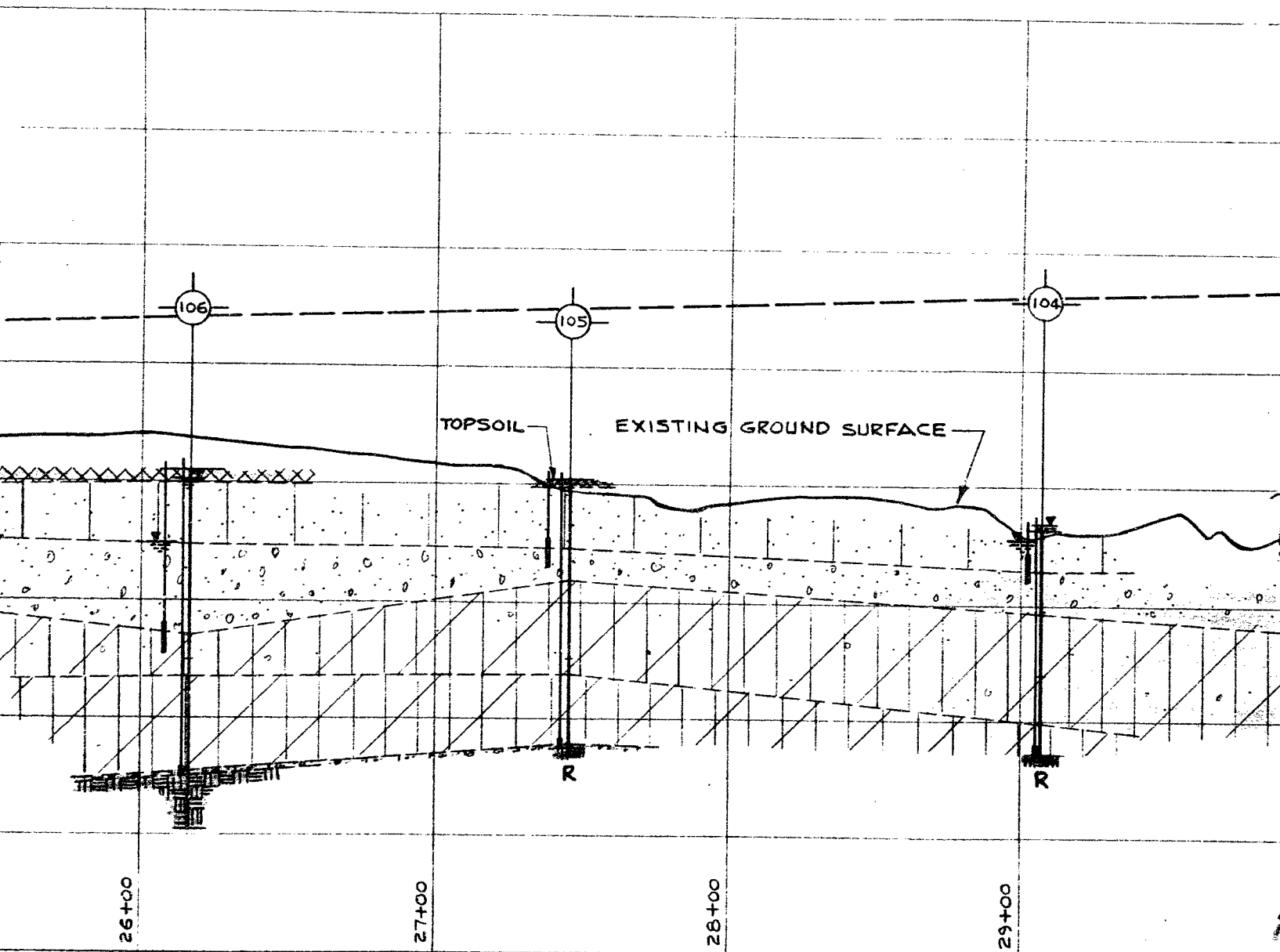
40P1 - 69
 GEOCRES No.

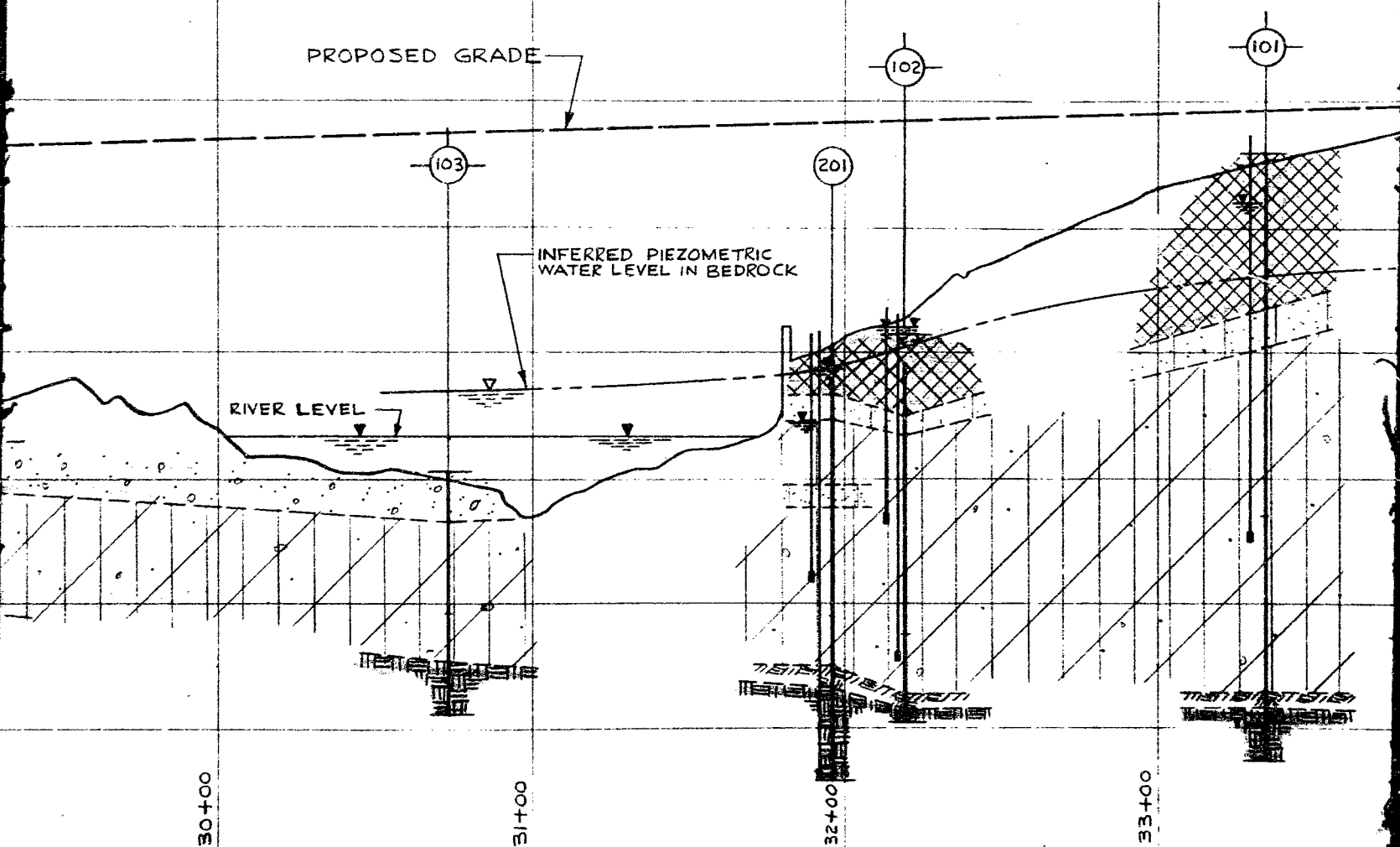
Date: APRIL 29, 1975

Golder Associates

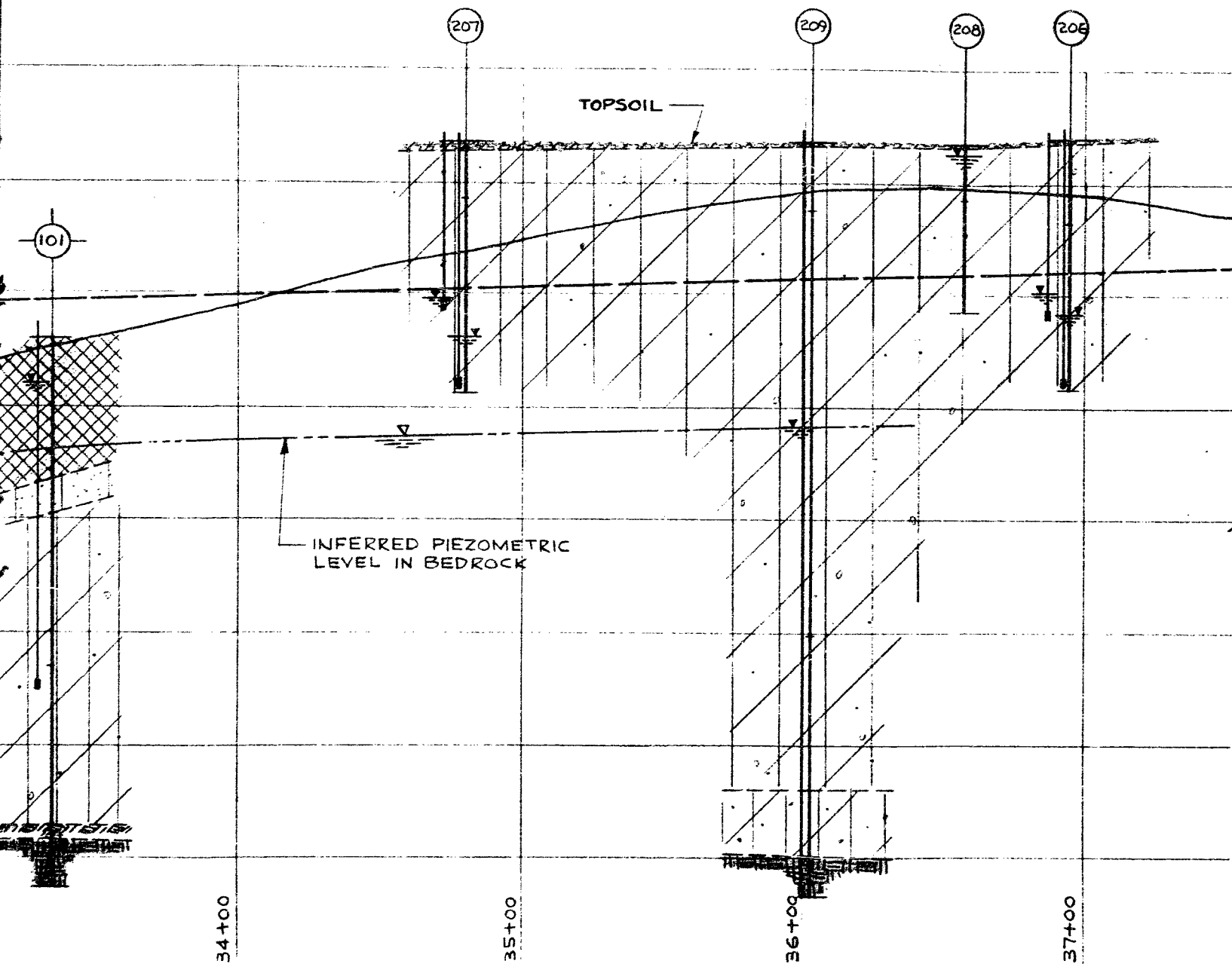
Drawn M.B.
 Chkd. R.G.
 Appd. [Signature]







SCHEMATIC SECTION 'A-A'





EXISTING GROUND SURFACE

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 of each borehole location refer to the record of borehole sheets.

SPECIAL NOTE

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

NOTE:

FOR LOCATION OF SECTION REFER
TO FIGURE 3.

38+00

39+00

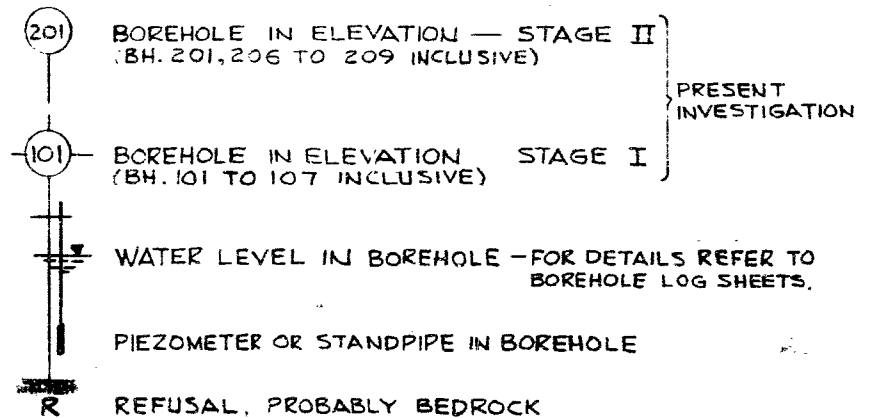
40+00

41+00

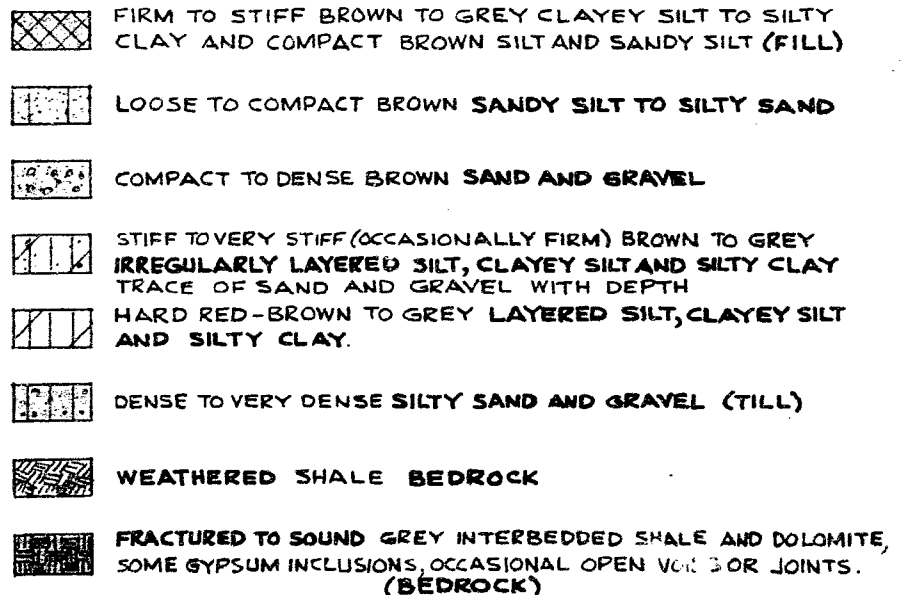
STRATIGRAPHIC SECTION ALONG PROPOSED CENTRELINE

FIGURE 4

LEGEND



STRATIGRAPHY



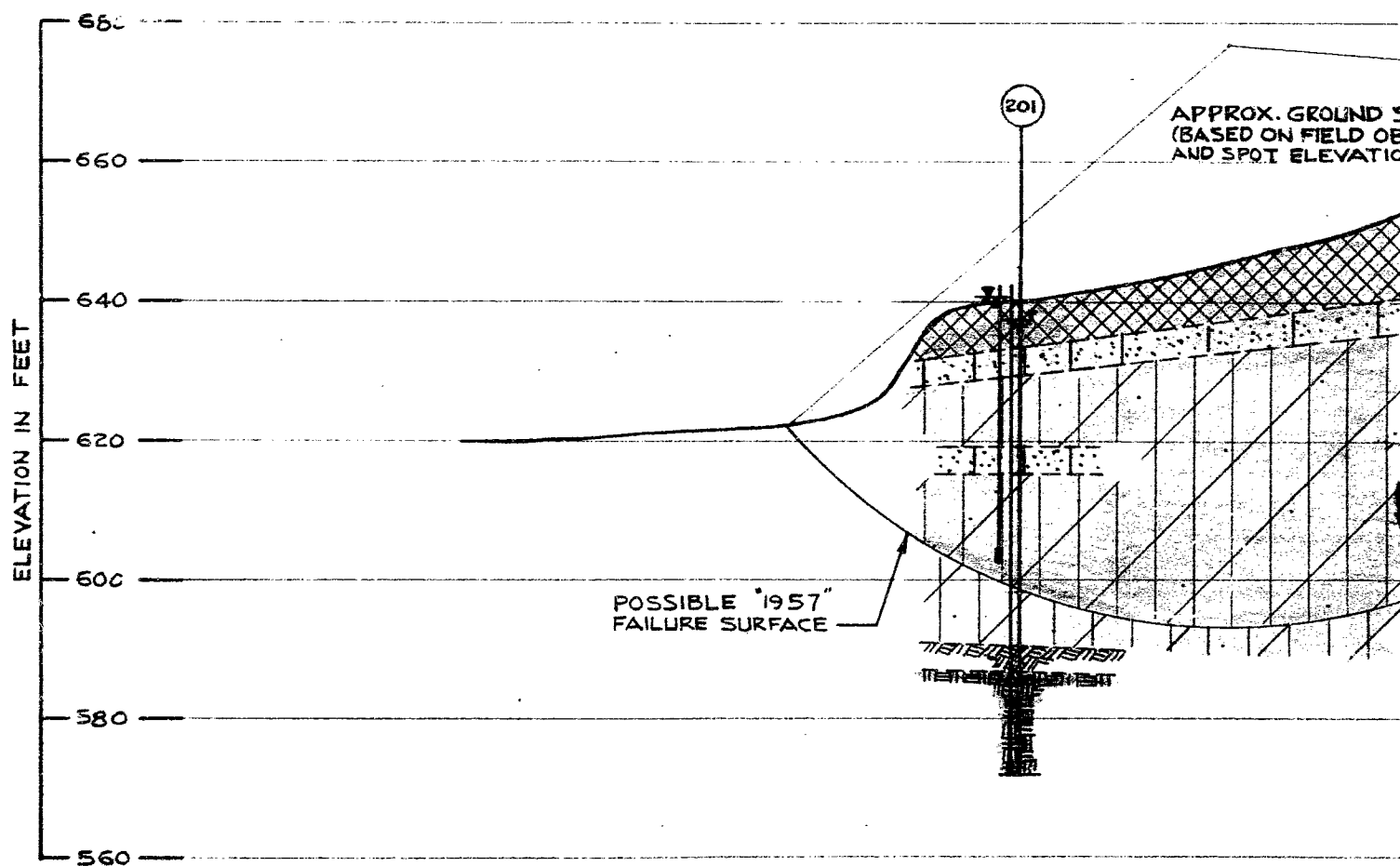
SCALE: -
 Horizontal: - 1" TO 40'
 Vertical: - 1" TO 20'

40P1 - 69
 GEOLOGICAL No.

Date: MAY 6, 1975

Golder Associates

Drawn *2/2/75*
 Chkd. *BC*
 Appd. *4/8*



SECTION 'B'-B'
SCALE: 1" TO 20'

SPECIAL NOTE

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

NOTE:

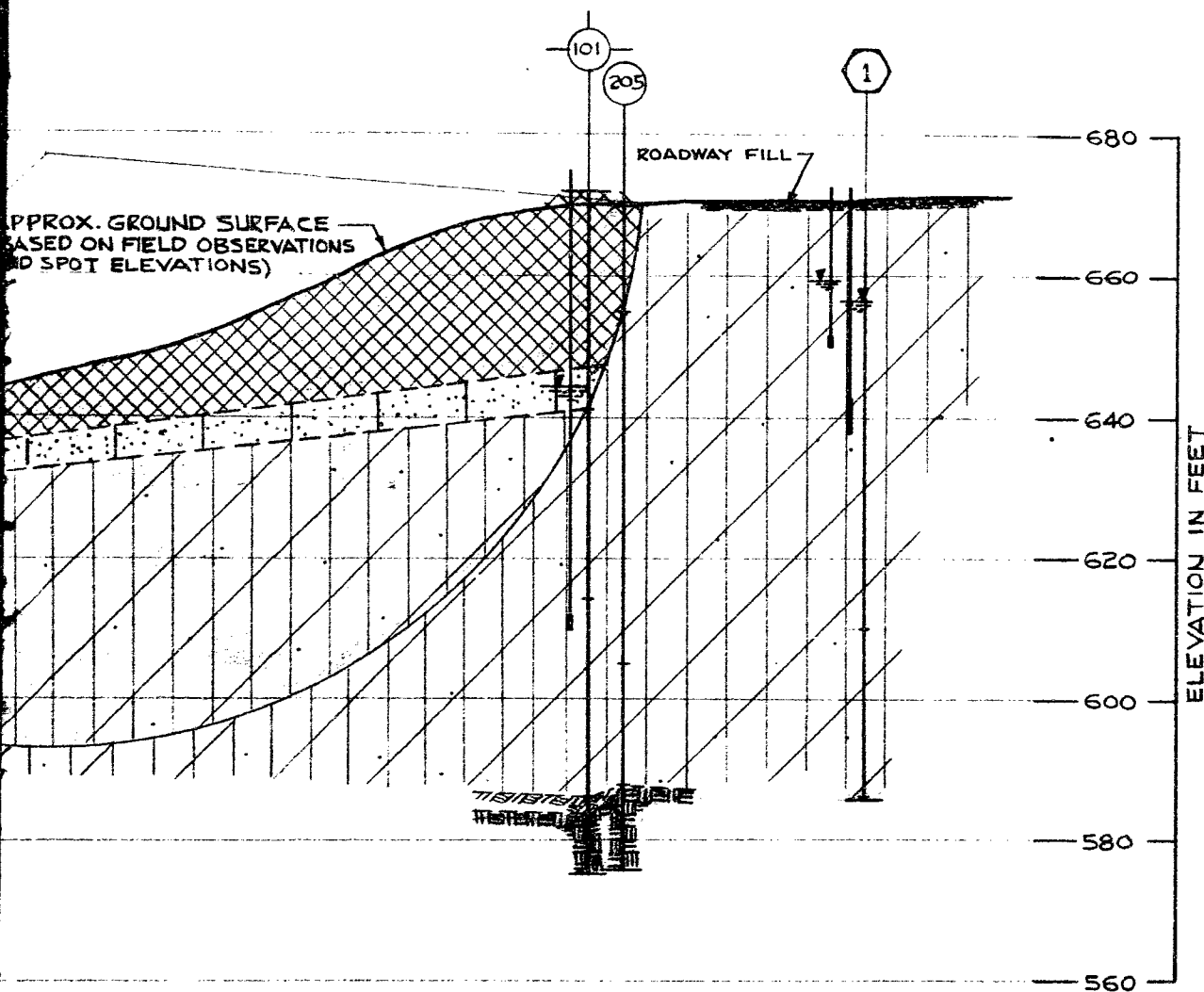
FOR LOCATION OF SECTION REFER TO

LEGEND

- (201) BOREHOLE IN ELEV
- (101) BOREHOLE IN ELEV
- (1) BOREHOLE IN ELEV
- WATER LEVEL IN B
- PIEZOMETER OR STA

SIMPLIFIED STRAT

- FILL REPORTEDLY
- SANDY SILT ZO
- ESSENTIALLY HOR
SILTS AND CLAYS
- ERRATICALLY BE
CLAYS (SLIDE DE
- WEATHERED BED
- FAIRLY SOUND TO



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.

Date: MAY 6, 1975

ILLUSTRATIVE SECTION THROUGH "1957" FAILURE

FIGURE 5

LEGEND

- 201

BOREHOLE IN ELEVATION - STAGE II
- 101

BOREHOLE IN ELEVATION - STAGE I
- 1

BOREHOLE IN ELEVATION -

{PREVIOUS INVESTIGATION
GOLDER ASSOC. REPORT No.
73154, DATED OCTOBER, 1973}
- WATER LEVEL IN BOREHOLE - FOR DETAILS REFER TO BOREHOLE LOG SHEETS.
- PIEZOMETER OR STANDPIPE IN BOREHOLE

SIMPLIFIED STRATIGRAPHY

- FILL REPORTEDLY PLACED AFTER "1957" FAILURE
- SANDY SILT ZONES
- ESSENTIALLY HORIZONTALLY BEDDED LACUSTRINE SILTS AND CLAYS.
- ERRATICALLY BEDDED LACUSTRINE SILTS AND CLAYS (SLIDE DEBRIS)
- WEATHERED BEDROCK
- FAIRLY SOUND TO SOUND BEDROCK

NOTE

Notes 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.

Date MAY 6, 1975

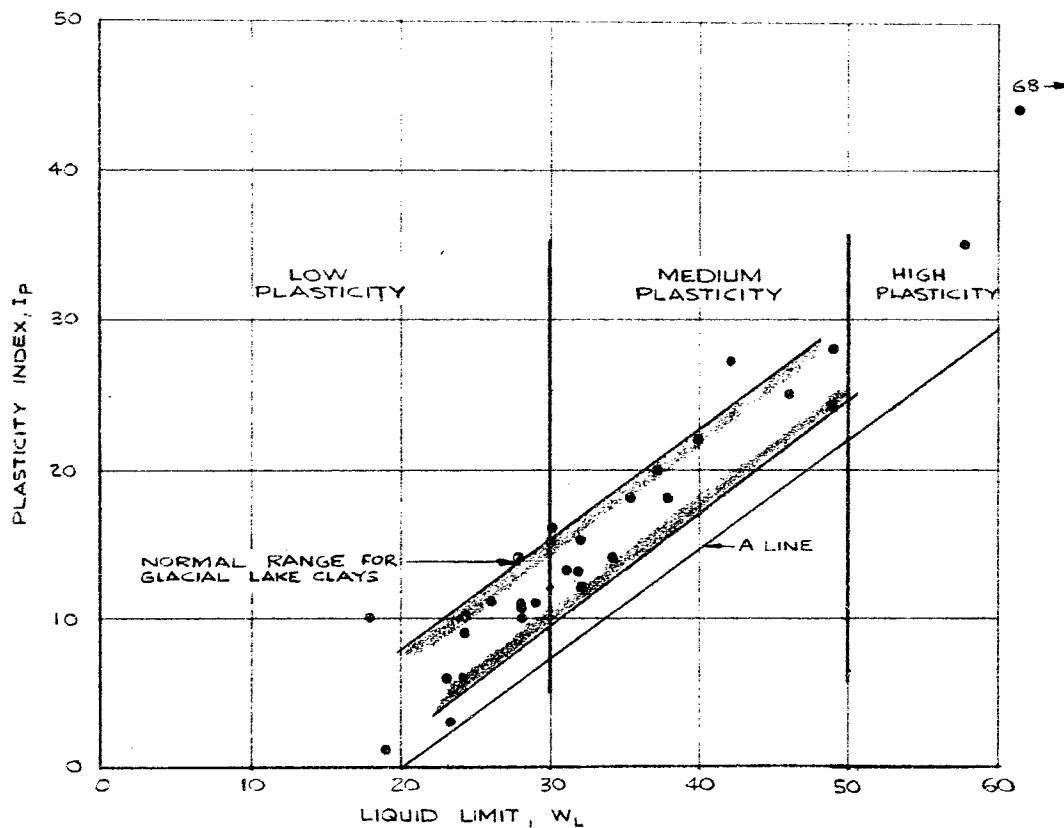
Golder Associates

40P1-69
GEOCREP No.

Drawn m.y.B.
Chkd BC
Appd AS

PLASTICITY CHART LAGUSTRINE DEPOSITS

FIGURE 7



LEGEND

- ATTERBERG LIMITS
(PRESENT INVESTIGATION)

Date FEB. 11, 1975

Golder Associates

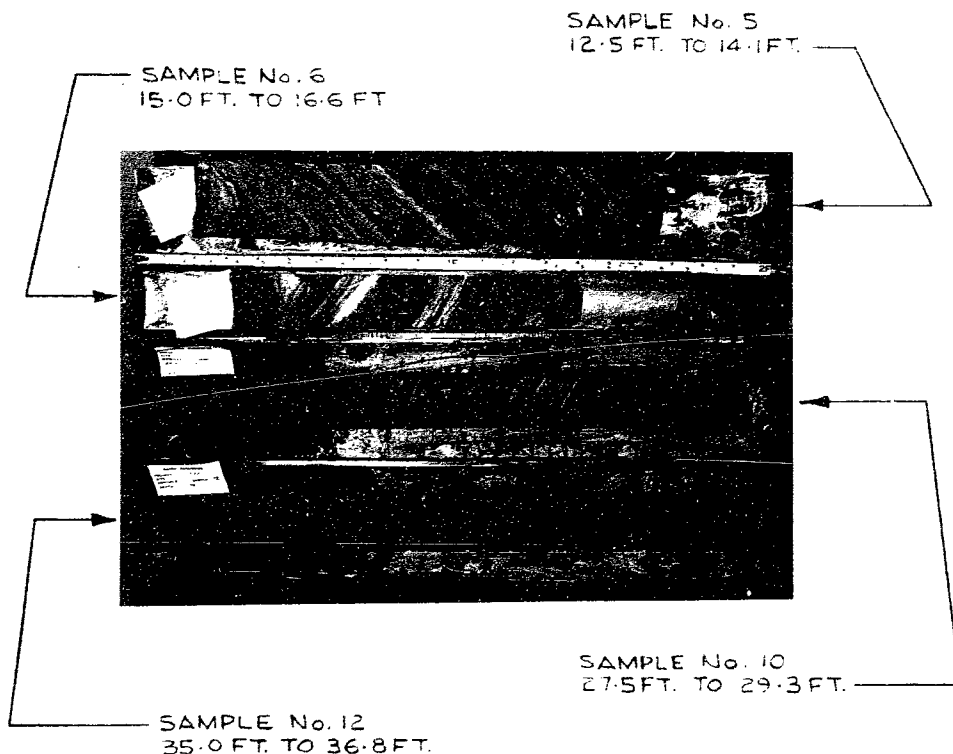
Drawn D.M.
Chkd. BG
Appd. [Signature]

PROJECT No. 741258

Form G.A. - D - 4

PHOTOGRAPH OF SAMPLES
SHOWING DISTURBED LAYERING
(BOREHOLE 201)

FIGURE 8



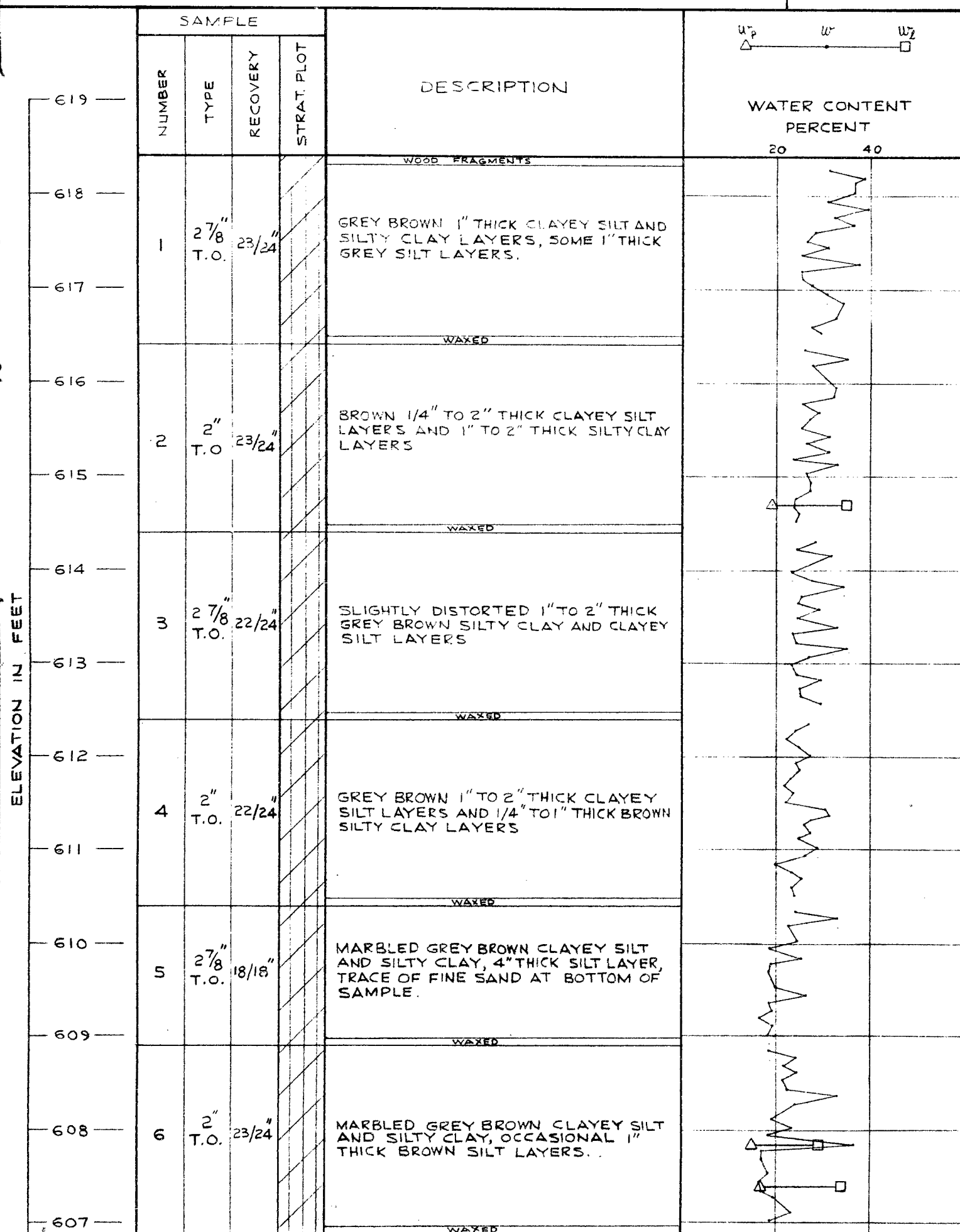
Page MAY 13, 1975

Golder Associates

Drawn *20-j-8*
Chkd
Appd

SUMMARY OF WATER CONTENTS - BH. 203 DEPTH 30' TO 41.5'

FIGURE 9



ELEVATION IN FEET

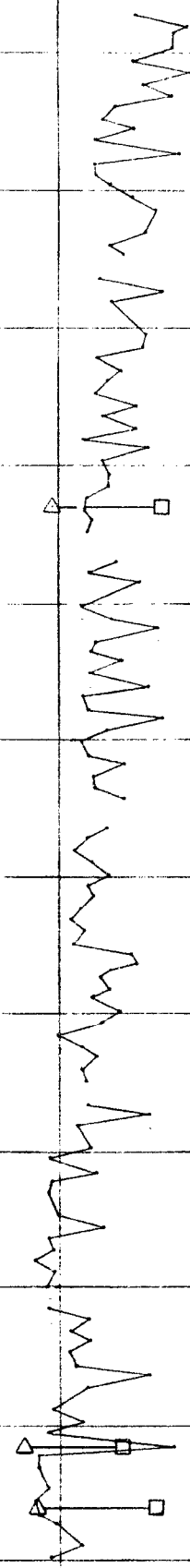
619 —
618 —
617 —
616 —
615 —
614 —
613 —
612 —
611 —
610 —
609 —
608 —
607 —
606 —

SAMPLE				DESCRIPTION	WATER CONTENT PERCENT
NUMBER	TYPE	RECOVERY	STRAT. PLOT		
1	2 7/8" T.O.	23/24"		WOOD FRAGMENTS	
				GREY BROWN 1" THICK CLAYEY SILT AND SILTY CLAY LAYERS, SOME 1" THICK GREY SILT LAYERS.	
2	2" T.O.	23/24"		WAXED	
				BROWN 1/4" TO 2" THICK CLAYEY SILT LAYERS AND 1" TO 2" THICK SILTY CLAY LAYERS	
3	2 7/8" T.O.	22/24"		WAXED	
				SLIGHTLY DISTORTED 1" TO 2" THICK GREY BROWN SILTY CLAY AND CLAYEY SILT LAYERS	
4	2" T.O.	22/24"		WAXED	
				GREY BROWN 1" TO 2" THICK CLAYEY SILT LAYERS AND 1/4" TO 1" THICK BROWN SILTY CLAY LAYERS	
5	2 7/8" T.O.	18/18"		WAXED	
				MARbled GREY BROWN CLAYEY SILT AND SILTY CLAY, 4" THICK SILT LAYER, TRACE OF FINE SAND AT BOTTOM OF SAMPLE.	
6	2" T.O.	23/24"		WAXED	
				MARbled GREY BROWN CLAYEY SILT AND SILTY CLAY, OCCASIONAL 1" THICK BROWN SILT LAYERS.	
				WAXED	

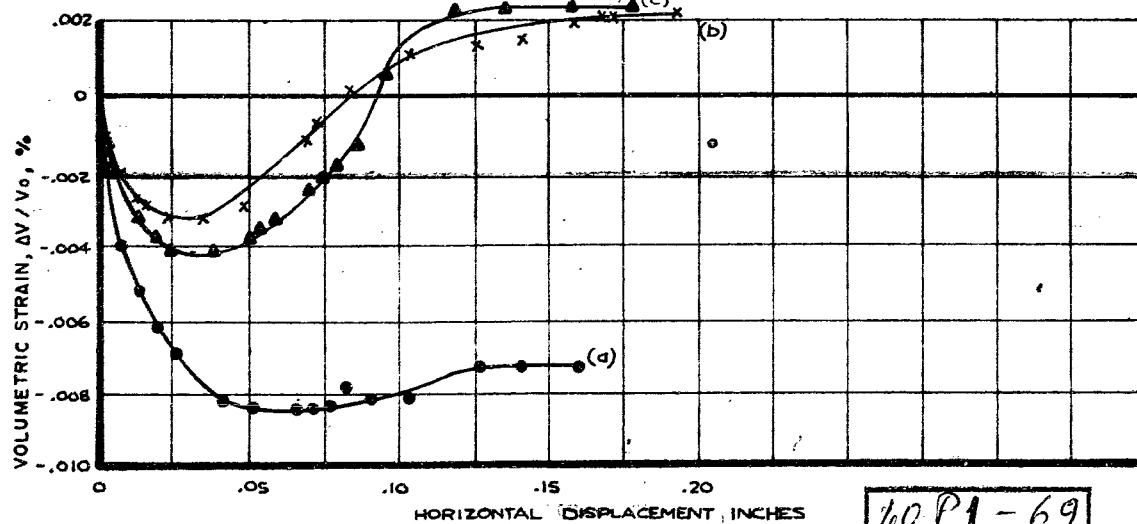
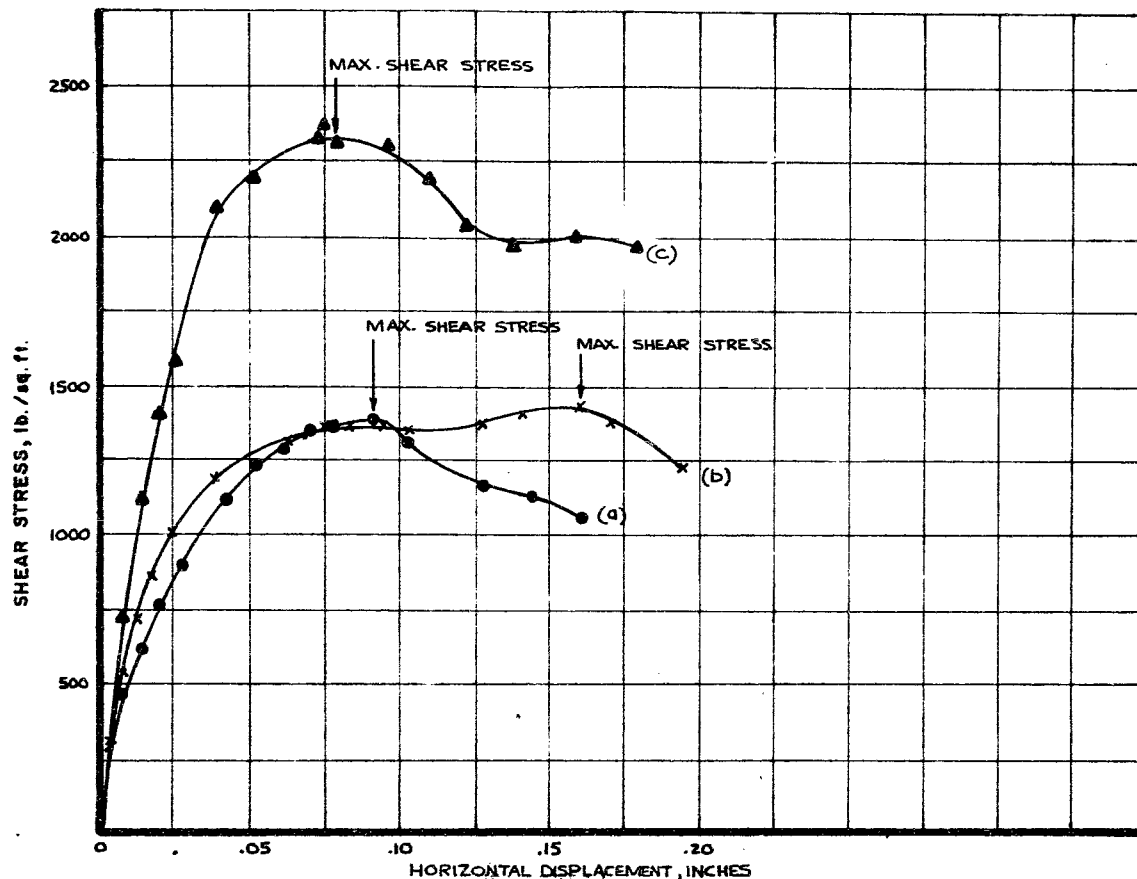
u_p u_r u_z
△ • □

WATER CONTENT
PERCENT

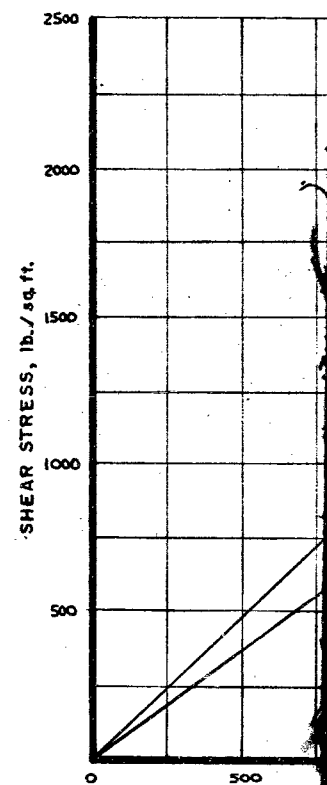
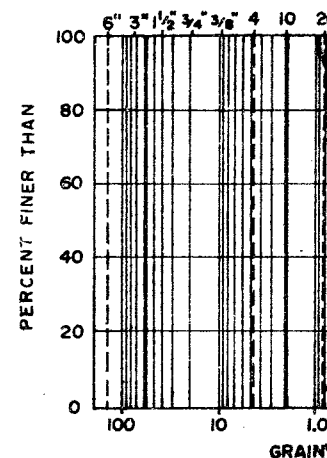
20 40



VERTICAL SCALE: 1" TO 1'



COBBLE	GRAVEL	

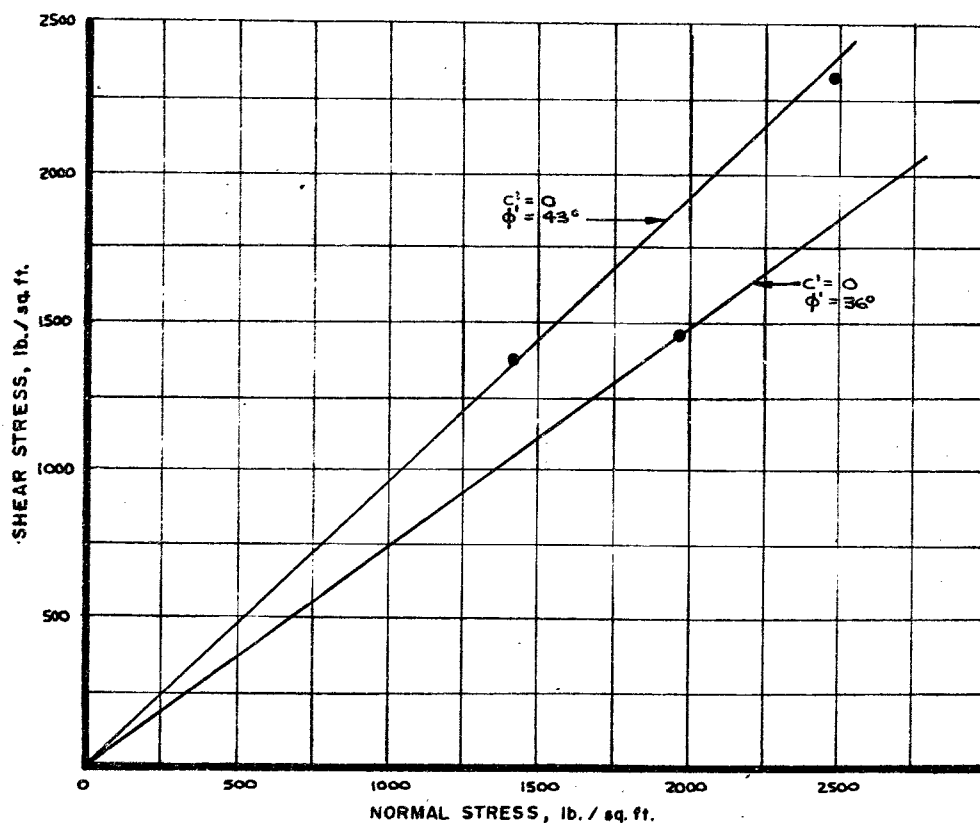
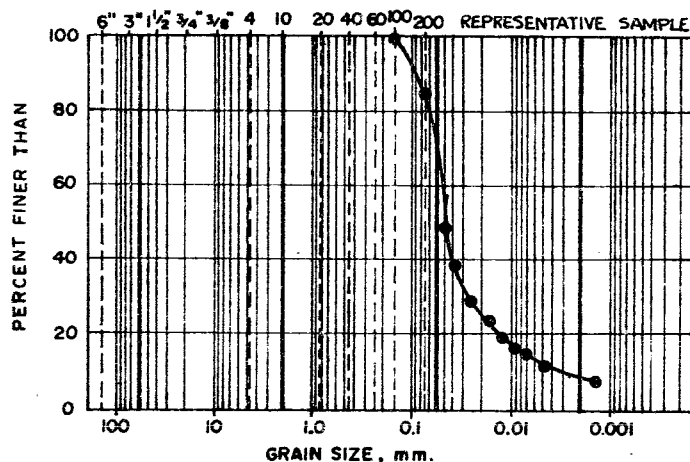


40 P1-69
GEOCREES No.

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

COBBLE	GRAVEL	SAND	SILT	CLAY
--------	--------	------	------	------

M.I.T. GRAIN SIZE SCALE



BOREHOLE NUMBER

SAMPLE NUMBER

SAMPLE DEPTH, ft.

SPECIMEN LENGTH, in.

SPECIMEN HEIGHT, in.

TEST CONDITIONS

WATER CONTENT, BEFORE CONSOLIDATION

NORMAL (CONSOLIDATION) STRESS, lb.

WATER CONTENT, AFTER CONSOLIDATION

AVERAGE RATE OF STRAIN, % / hr.

TIME TO FAILURE, days

WATER CONTENT, AFTER TEST, %

TEST RESULTS

PEAK SHEAR STRESS, lb./sq. ft.

RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST

SECOND

THIRD

HORIZONTAL DISPLACEMENT, INCHES

SAT MAX. SHEAR STRESS AT STRESS

SHEAR STRAIN AT RESIDUAL SHEAR STRESS

FIRST

SECOND

THIRD

NATURAL WATER CONTENT, w , %

LIQUID LIMIT, w_L

PLASTIC LIMIT, w_p

UNIT WEIGHT, γ_t , lb./cu. ft.

REMARKS (a) SANDY SILT AND

(b) FINE SANDY SILT

(c) FINE SANDY SILT AND

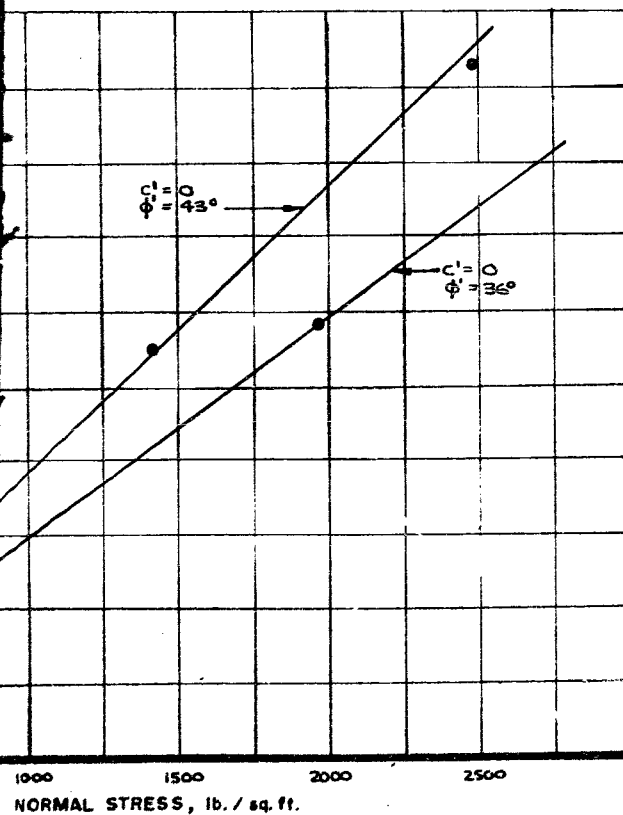
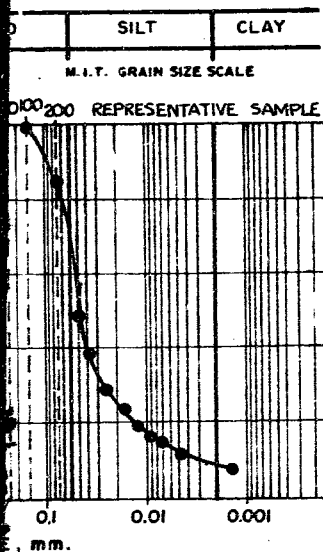
SAMPLES TRIMMED FROM

Date FEB. 11, 1975

Golder Associa

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 10



a b c d

← * → -

BOREHOLE NUMBER	101	101	101	
SAMPLE NUMBER	9	9	9	
SAMPLE DEPTH, ft.	28	28	28	

SPECIMEN LENGTH, in.	2.34	2.34	2.34	
SPECIMEN HEIGHT, in.	.985	.974	.989	

TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	19	19	19	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	1440	1970	2500	
	WATER CONTENT, AFTER CONSOLIDATION, %				
	AVERAGE RATE OF STRAIN, % / hr.	0.5	0.5	0.5	
	TIME TO FAILURE, days	1	2	2	
	WATER CONTENT, AFTER TEST, % AVG.	26	28	26	

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	1380	1440	2320	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	
	HORIZONTAL DISPLACEMENT, INCHES AT MAX. SHEAR STRESS	.080	.1600	.080	
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	

NATURAL WATER CONTENT, w , %	19			
LIQUID LIMIT, w_L	17			
PLASTIC LIMIT, w_p	15			
UNIT WEIGHT, γ_t , lb./cu. ft.	132	132	135	

REMARKS (a) SANDY SILT AND FINE SAND
(b) FINE SANDY SILT
(c) FINE SANDY SILT AND CLAYEY SILT

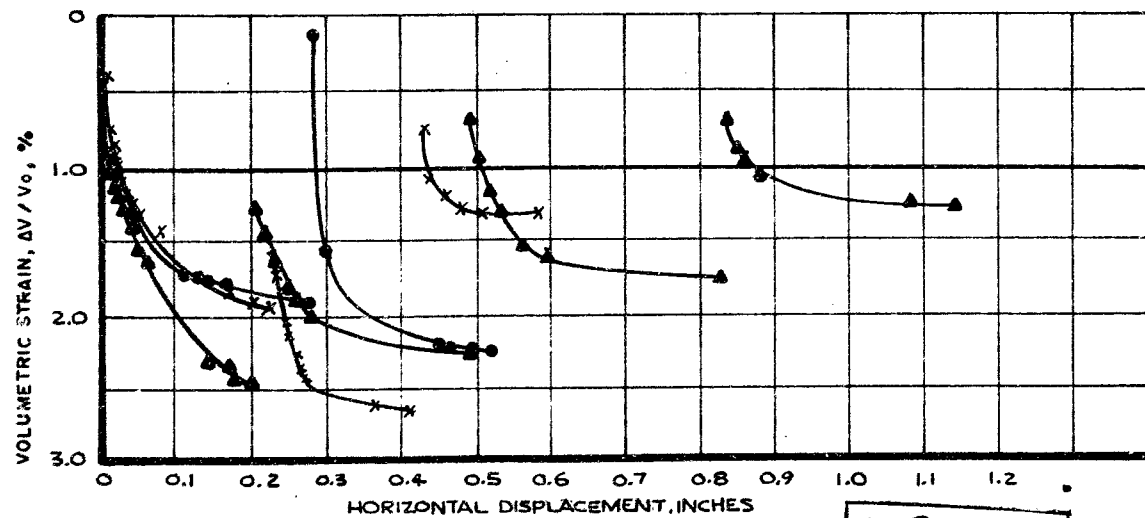
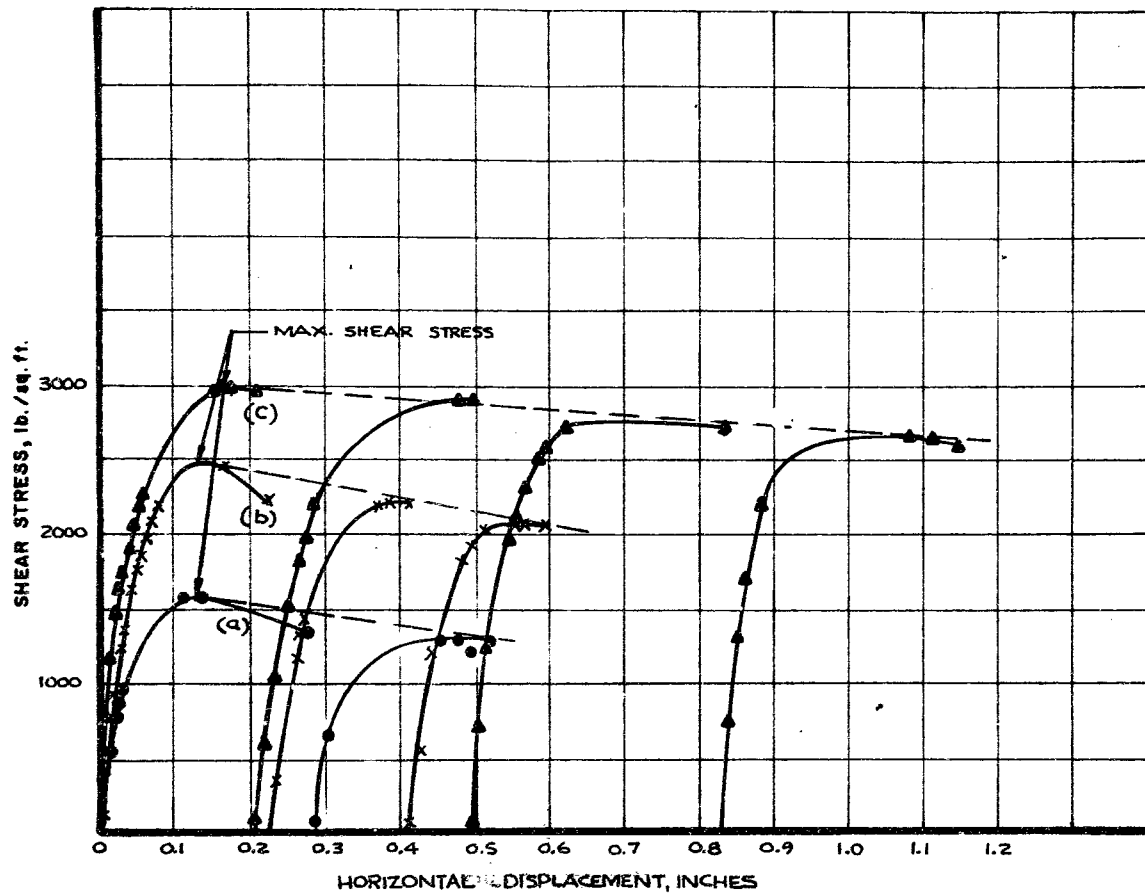
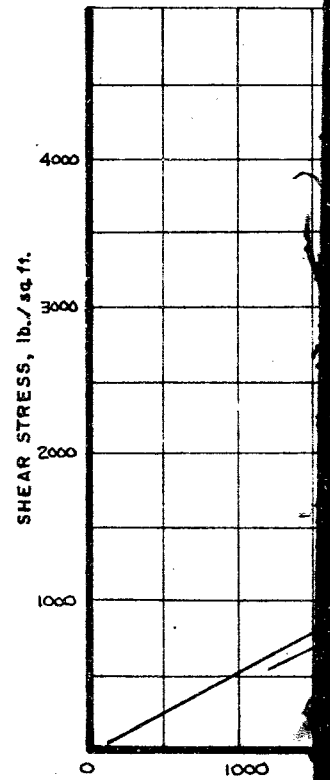
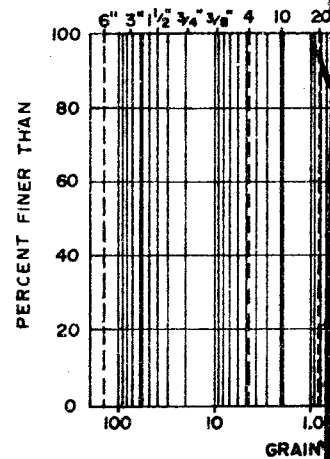
SAMPLES TRIMMED FROM 3 INCH SHELBY TUBE

Date FEB. 11, 1975

Golder Associates

Drawn D.M.
Chkd. BE
Appd. AS

COBBLE	GRAVEL	
--------	--------	--

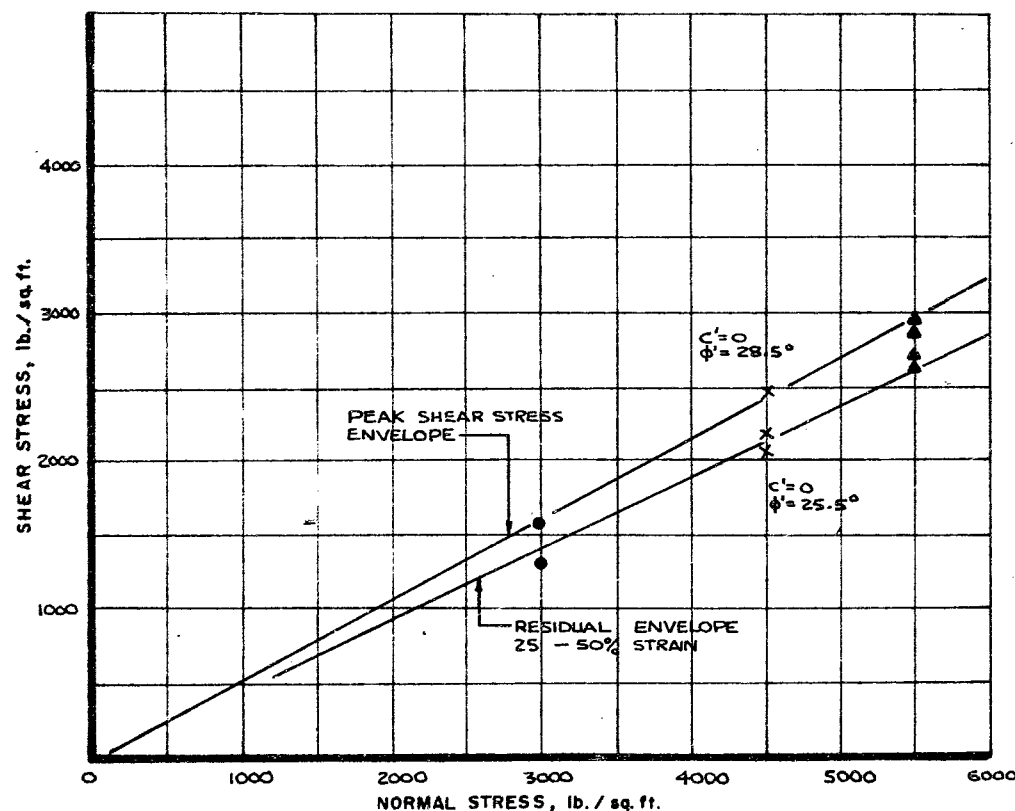
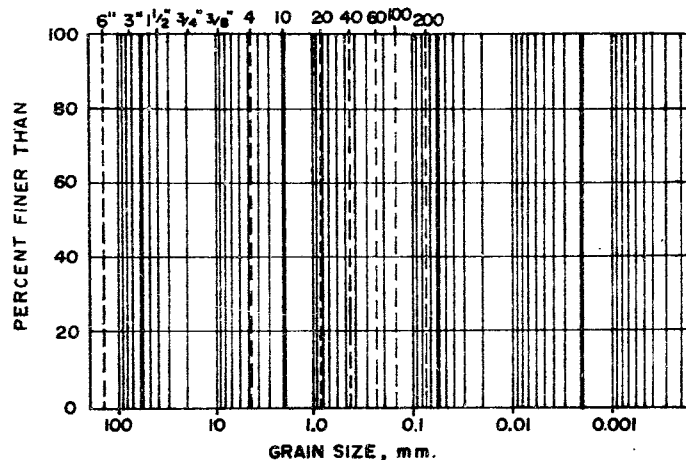


40 P1-69
GEOCRE No.

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

COBBLE	GRAVEL	SAND	SILT	CLAY

M.I.T. GRAIN SIZE SCALE



BOREHOLE NUMBER
SAMPLE NUMBER
SAMPLE DEPTH, INCHES

SPECIMEN LENGTH, in.
SPECIMEN HEIGHT, in.

TEST CONDITIONS
WATER CONTENT, BEFORE
NORMAL (CONSOLIDATION)
WATER CONTENT, AFTER CO
AVERAGE RATE OF STRAI
TIME TO FAILURE, days
WATER CONTENT, AFTER,

TEST RESULTS
PEAK SHEAR STRESS, lb.
RESIDUAL SHEAR STRESS, lb.
HORIZONTAL DISPLACEMENT
AT MAX. SHEAR STRESS
SHEAR STRAIN AT RESIDUAL

NATURAL WATER CONTENT
LIQUID LIMIT, w_L
PLASTIC LIMIT, w_p
UNIT WEIGHT, γ_t , lb./cu.

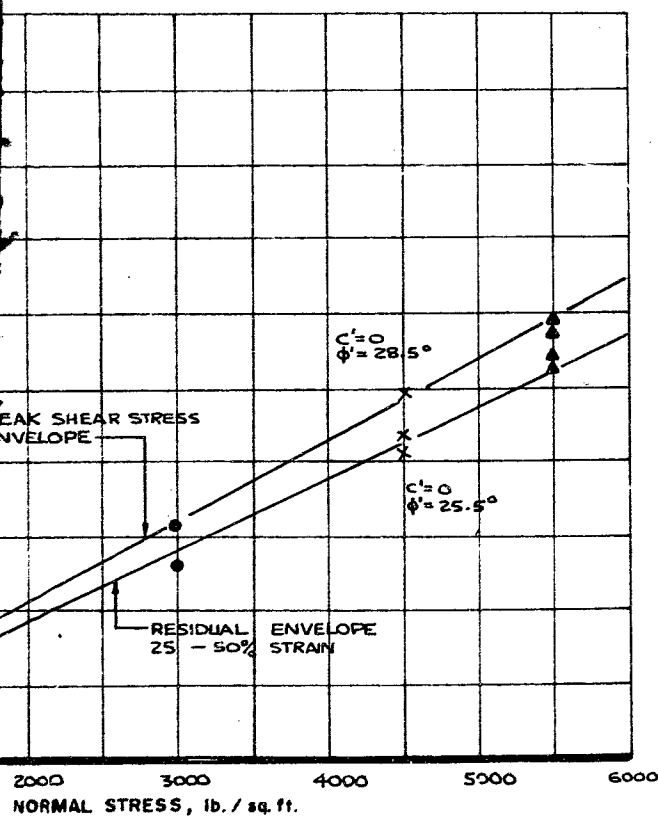
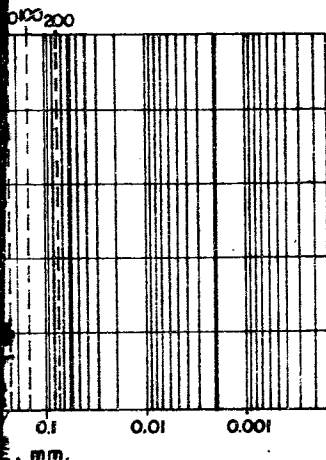
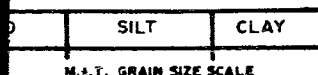
REMARKS SAMPLES TRIM
IRREGULAR LAYERS

Date FEB. 14, 1975

Golder

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 11



a	b	c	d
—●—	—x—	—▲—	

BOREHOLE NUMBER	101	101	101	
SAMPLE NUMBER	17	17	17	
SAMPLE DEPTH, INCHES RECOVERY	66	66	66	

SPECIMEN LENGTH, in.	2.34	2.34	2.34	
SPECIMEN HEIGHT, in.	2.34	2.34	2.34	

TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	26	22	22	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	3000	4500	5500	
	WATER CONTENT, AFTER CONSOLIDATION, %				
	AVERAGE RATE OF STRAIN, % / hr.	.25	.25	.25	
	TIME TO FAILURE, days	2	2	2	
	WATER CONTENT, AFTER TEST, % AVG.	25	21	20	

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	1579	2499	2981	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS	1293	2181	2896	
	SECOND PASS	—	2069	2731	
	THIRD PASS	—	—	2612	
	HORIZONTAL DISPLACEMENT, INCHES AT MAX. SHEAR STRESS	.134	.140	.165	
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS	20.2	17.5	20.4	
	SECOND PASS	—	25.0	27.0	
	THIRD PASS	—	—	46.5	

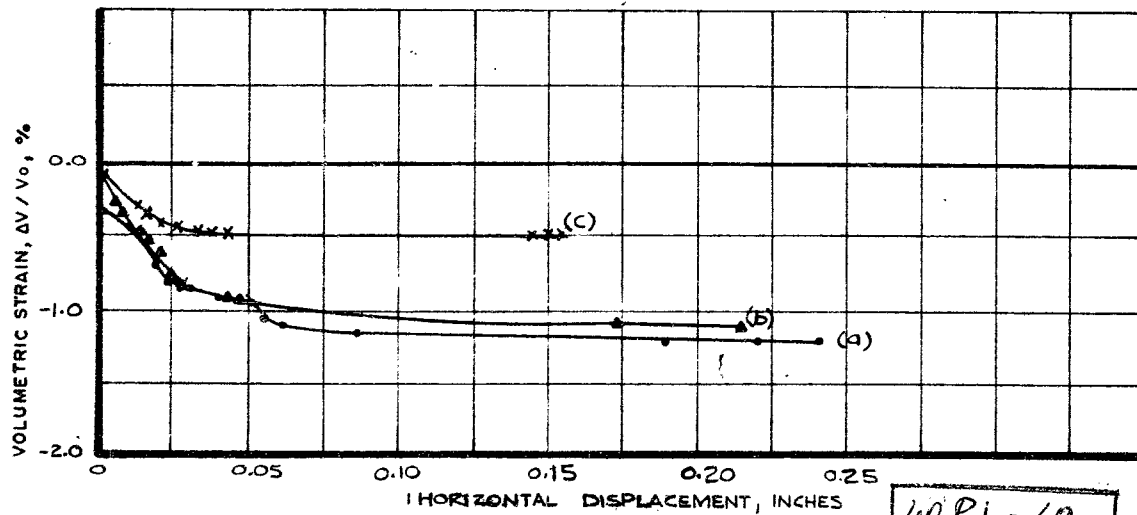
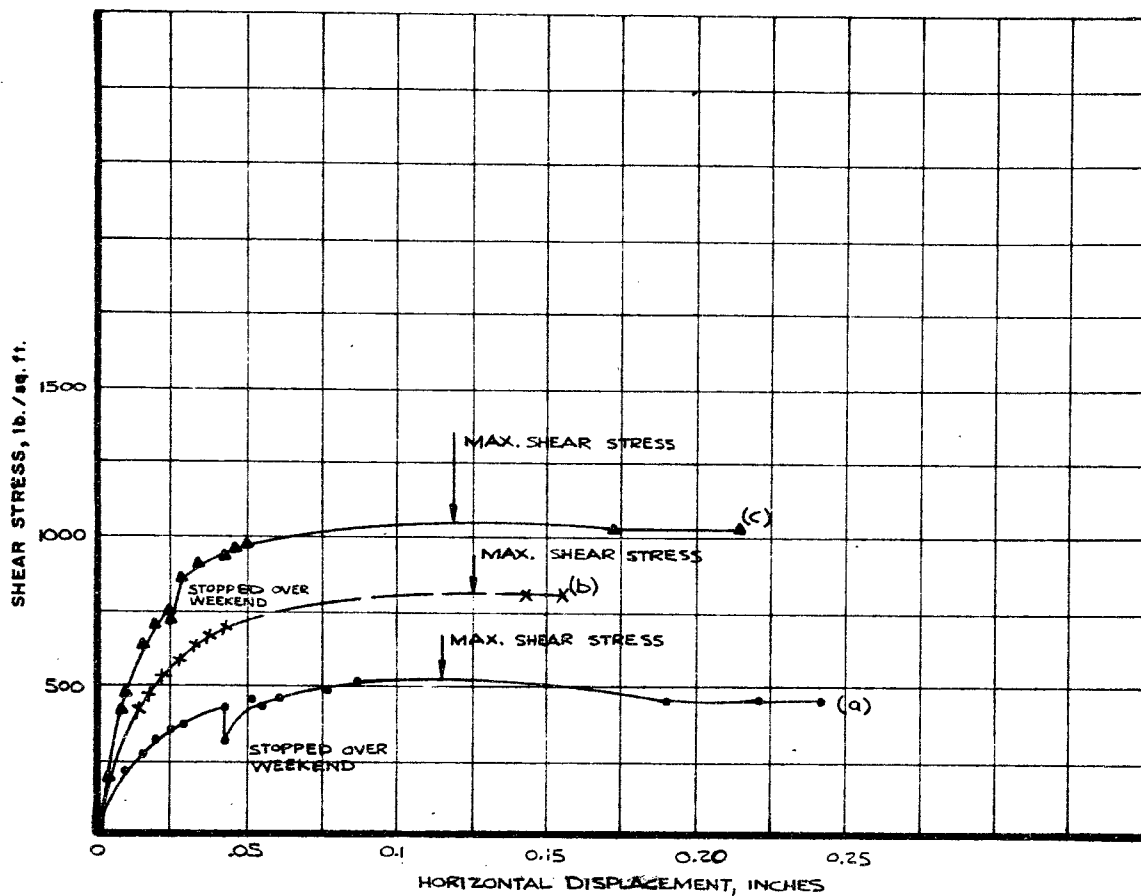
	SILT	22	22	22	
NATURAL WATER CONTENT, w , %	CLAY	26	27	27	
LIQUID LIMIT, w_L		28	28	30	
PLASTIC LIMIT, w_p		17	18	17	
UNIT WEIGHT, γ_t , lb./cu. ft.		132	132	135	

REMARKS SAMPLES TRIMMED FROM 3 INCH SHELBY TUBE
IRREGULAR LAYERED, RED CLAYEY SILT WITH CLAY
LAYERS

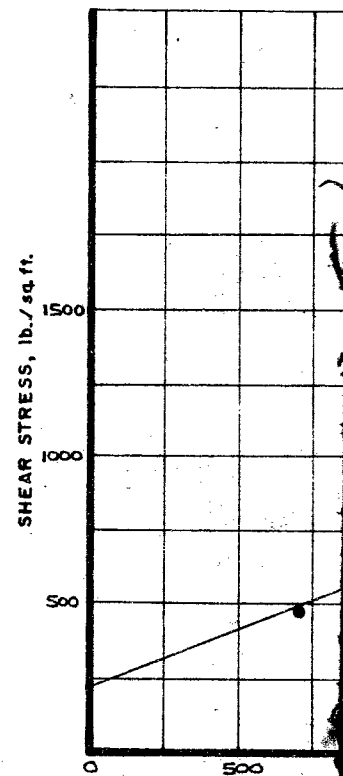
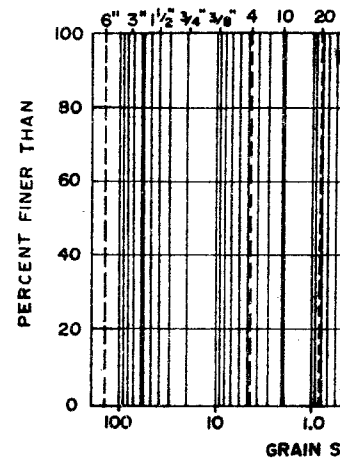
Date FEB. 14, 1975

Golder Associates

Drawn D.M.
Chkd. SE
Appd. JR



COBBLE	GRAVEL	
--------	--------	--



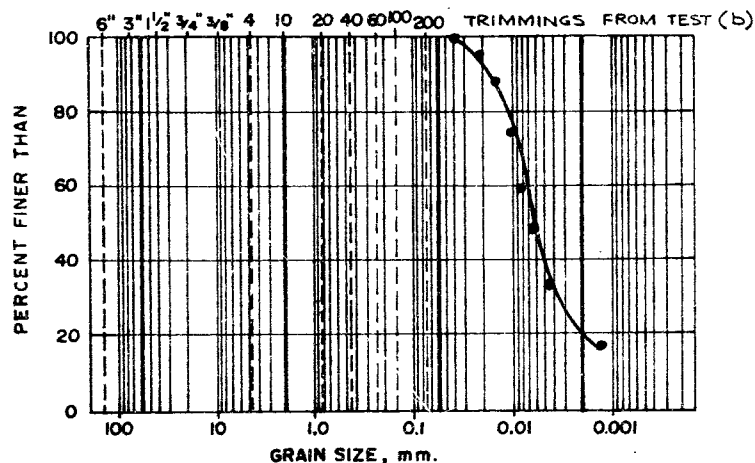
40PI-69

GECCES No.

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

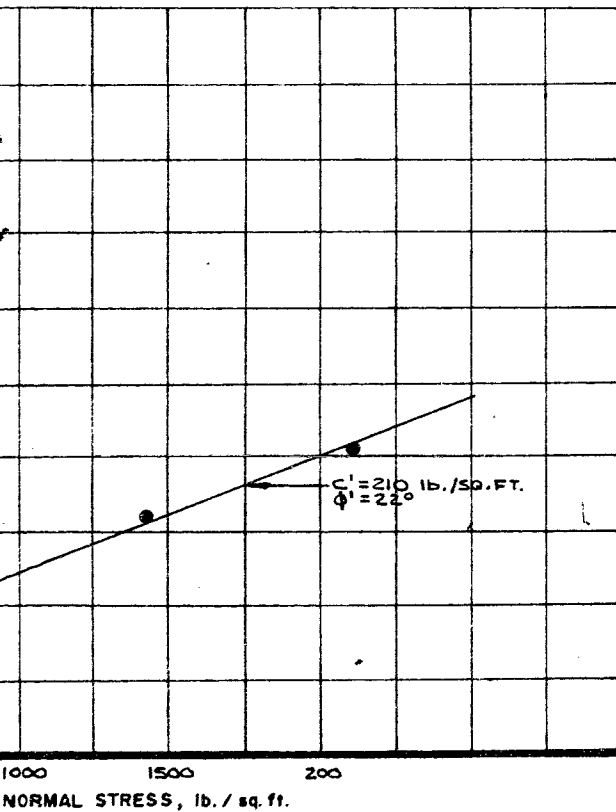
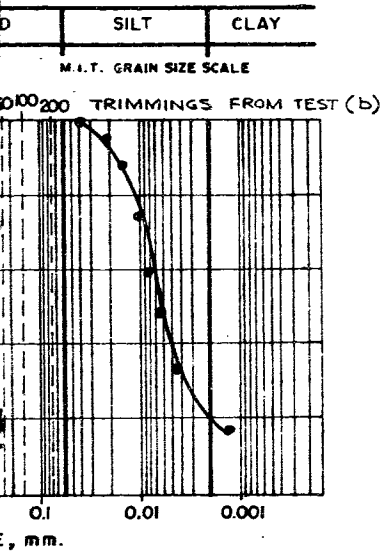
COBBLE	GRAVEL	SAND	SILT	CLAY

M.I.T. GRAIN SIZE SCALE



CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 12



a	b	c	d
•	x	Δ	

BOREHOLE NUMBER	101	101	101	
SAMPLE NUMBER	11	11	11	
SAMPLE DEPTH, INCHES RECOVERY	7-8	9-10	11-12	

SPECIMEN LENGTH, in.	2.34	2.34	2.34	
SPECIMEN HEIGHT, in.	.994	.939	.971	

TEST CONDITIONS	WATER CONTENT, BEFORE CONSOLIDATION, %	23	22	19	
	NORMAL (CONSOLIDATION) STRESS, lb./sq. ft.	720	1440	2179	
	WATER CONTENT, AFTER CONSOLIDATION, %				
	AVERAGE RATE OF STRAIN, % / hr.	.25	.25	.25	
	TIME TO FAILURE, days	2	2	2	
	WATER CONTENT, AFTER TEST, %	24	25	24	

TEST RESULTS	PEAK SHEAR STRESS, lb./sq. ft.	480	807	1030	
	RESIDUAL SHEAR STRESS, lb./sq. ft. FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	
	HORIZONTAL DISPLACEMENT, INCHES AT MAX. SHEAR STRESS	.115	.125	.120	
	SHEAR STRAIN AT RESIDUAL SHEAR STRESS, % FIRST PASS	—	—	—	
	SECOND PASS	—	—	—	
	THIRD PASS	—	—	—	

NATURAL WATER CONTENT, w , % AVG.	23	22	19	
LIQUID LIMIT, w_L	26	23	22	
PLASTIC LIMIT, w_p	17	17	14	
UNIT WEIGHT, γ_t , lb./cu. ft.	128	139	135	

REMARKS SAMPLES TRIMMED FROM 3" SHELBY TUBE
720 lb./sq. ft. SHEAR PLANE - CLAYEY SILT TO SILT
1440 lb./sq. ft. SHEAR PLANE - CLAYEY SILT TO SILT
2179 lb./sq. ft. SHEAR PLANE - CLAYEY SILT

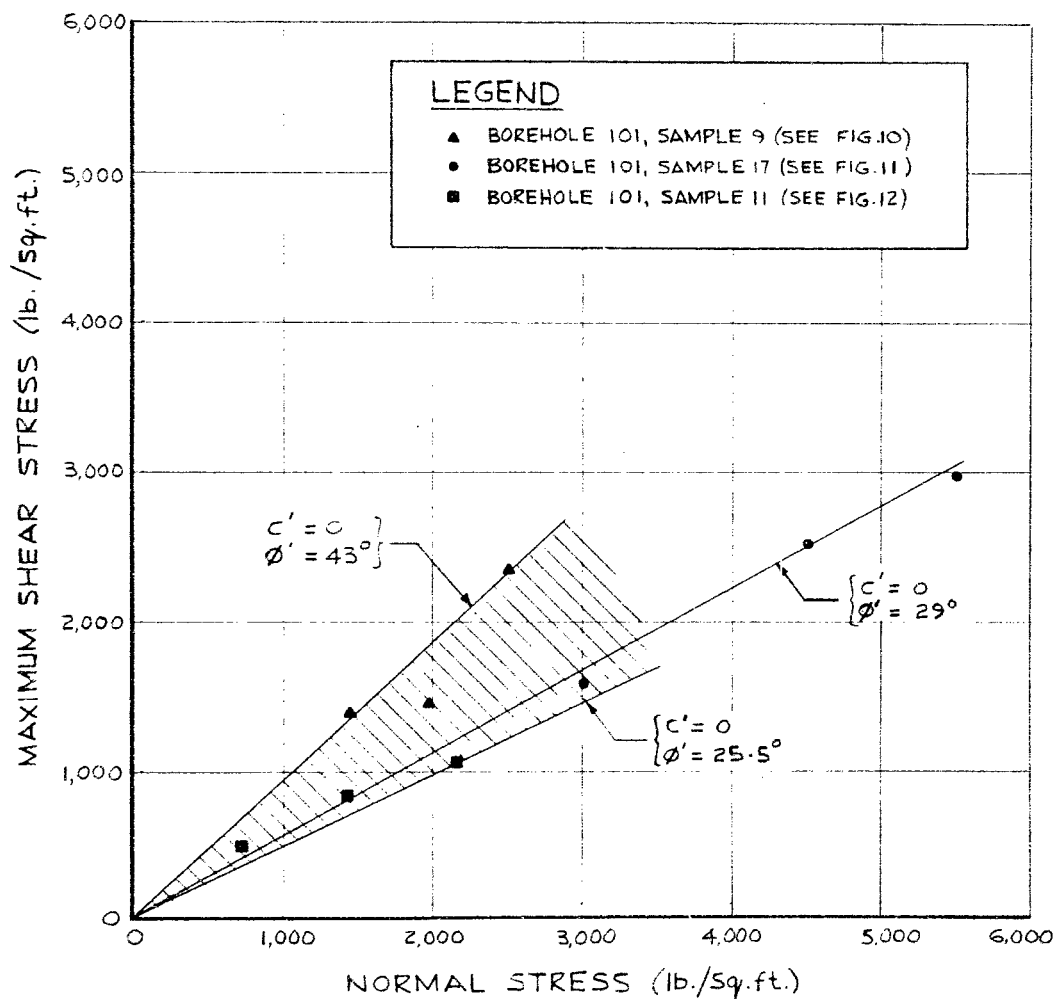
Date MAR. 3, 1975

Golder Associates

Drawn D.M.
Chkd. BG
Appd. H.S.

SUMMARY OF CONSOLIDATED DRAINED DIRECT SHEAR TESTS

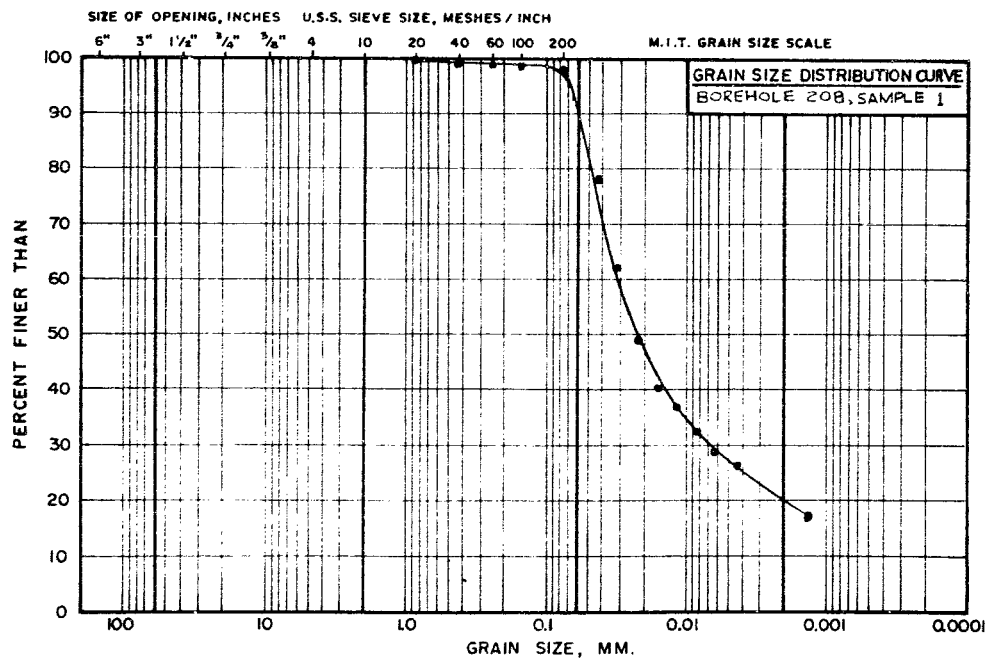
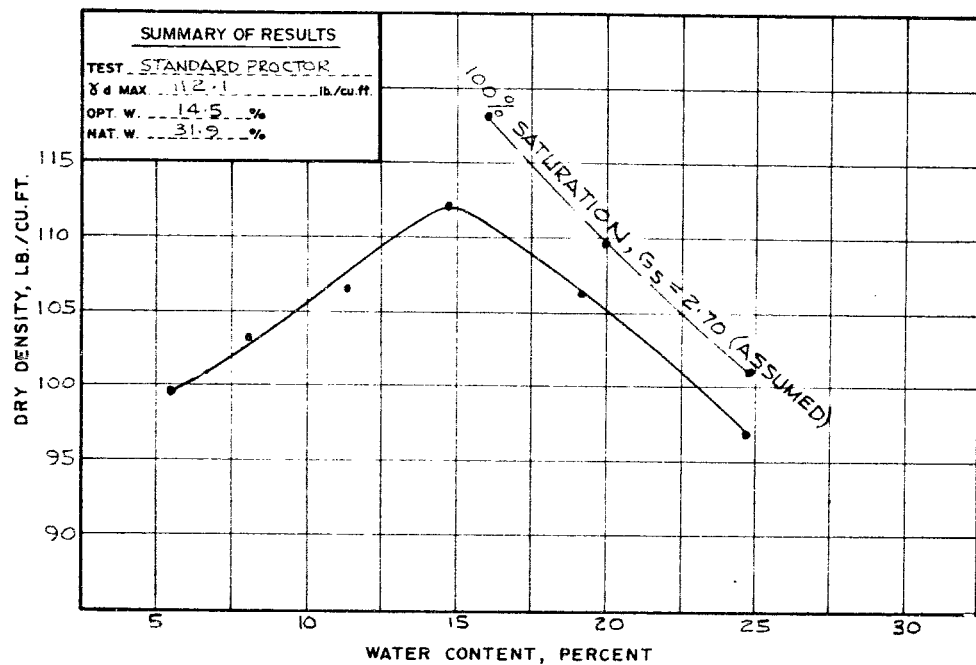
FIGURE 13



Date MAR. 3, 1975

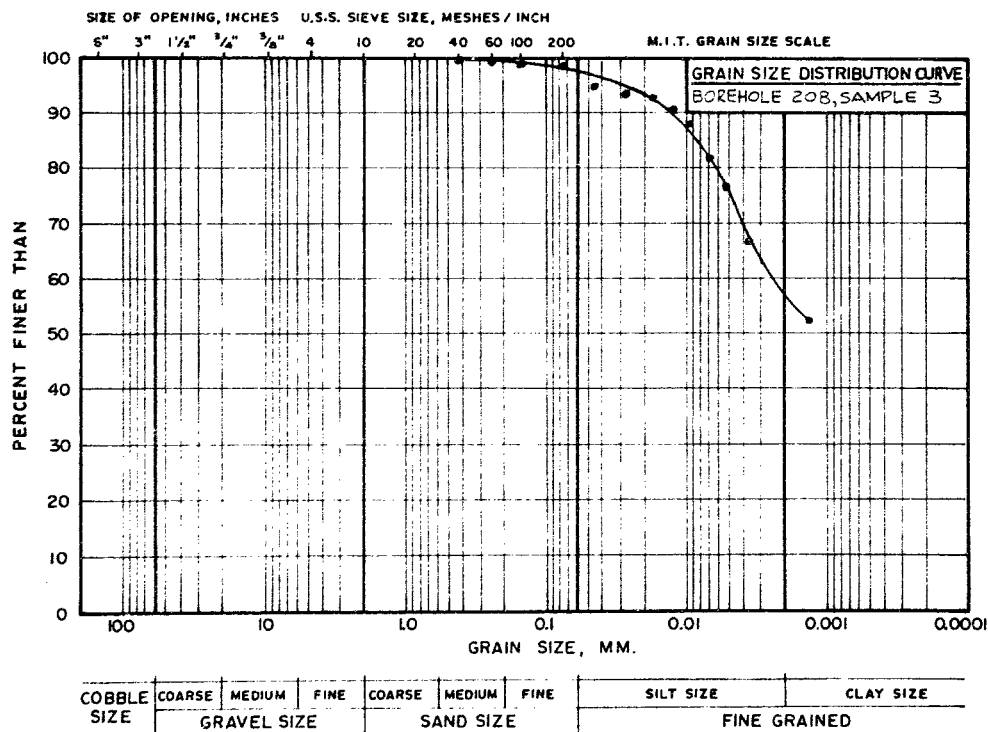
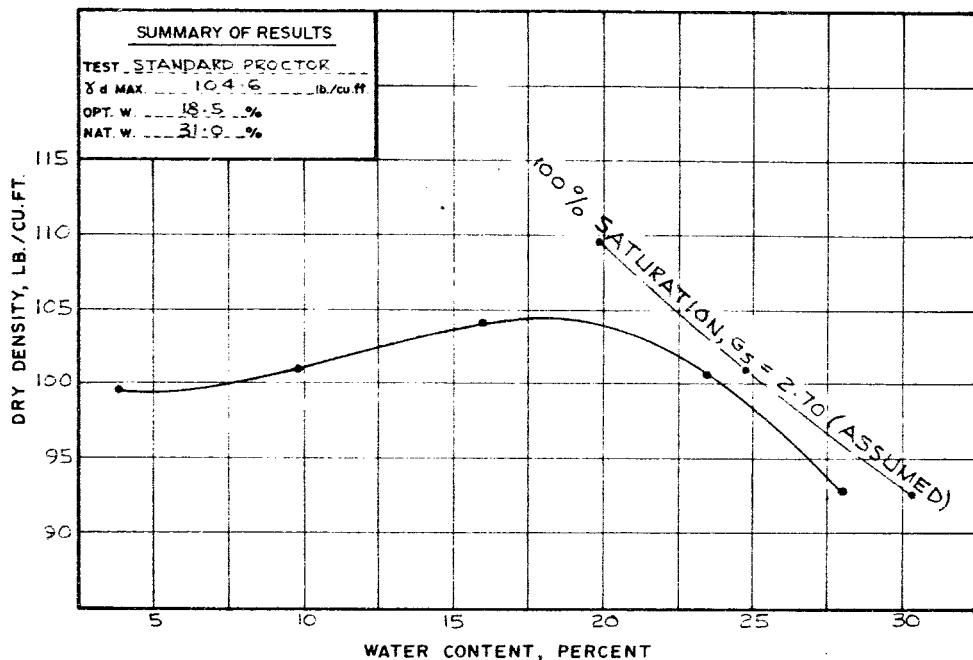
Golder Associates

Drawn *m.y.B.*
Chkd *ELG*
Appd *1-3*



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

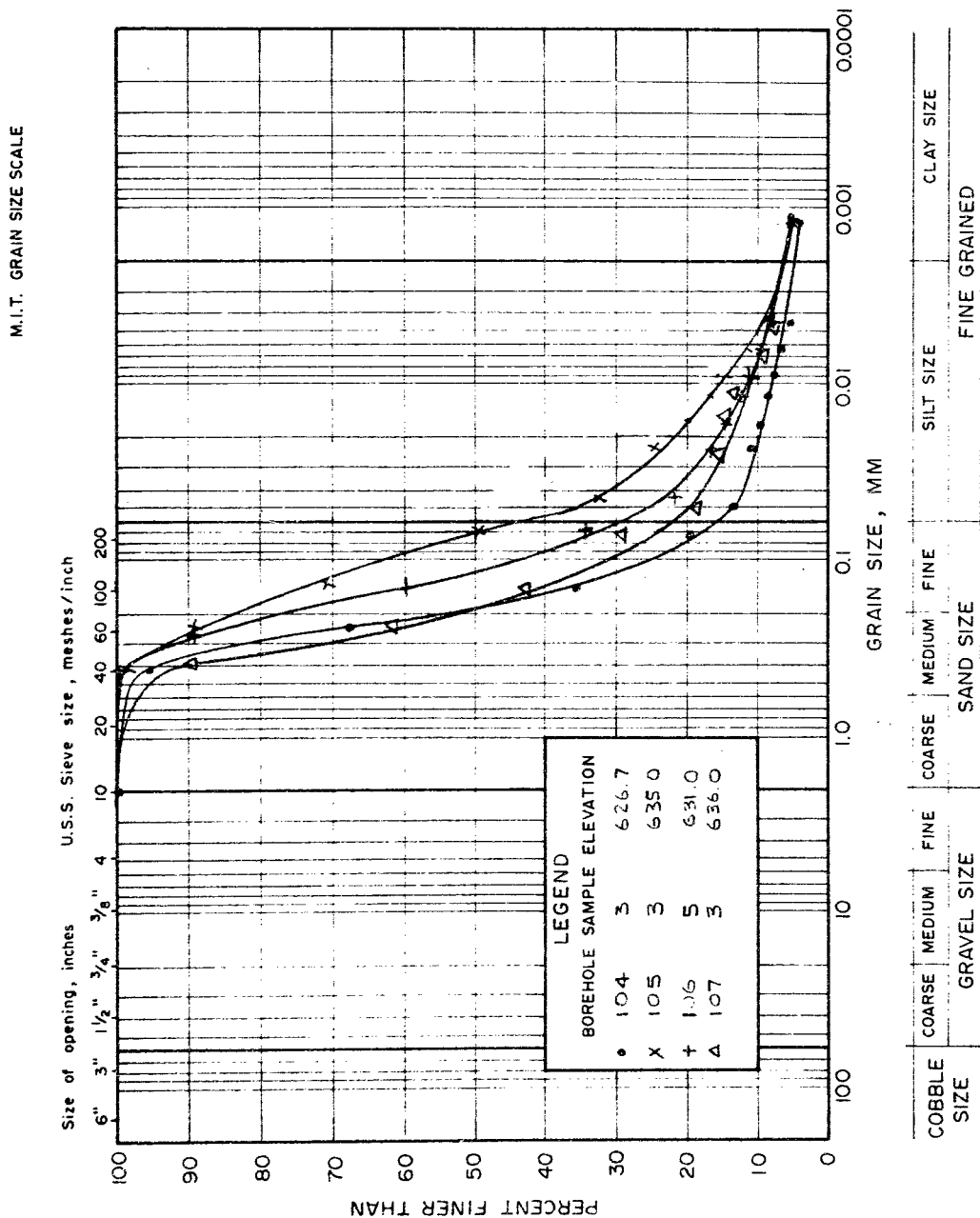
Golder Associates



Golder Associates

GRAIN SIZE DISTRIBUTION SILTY SAND

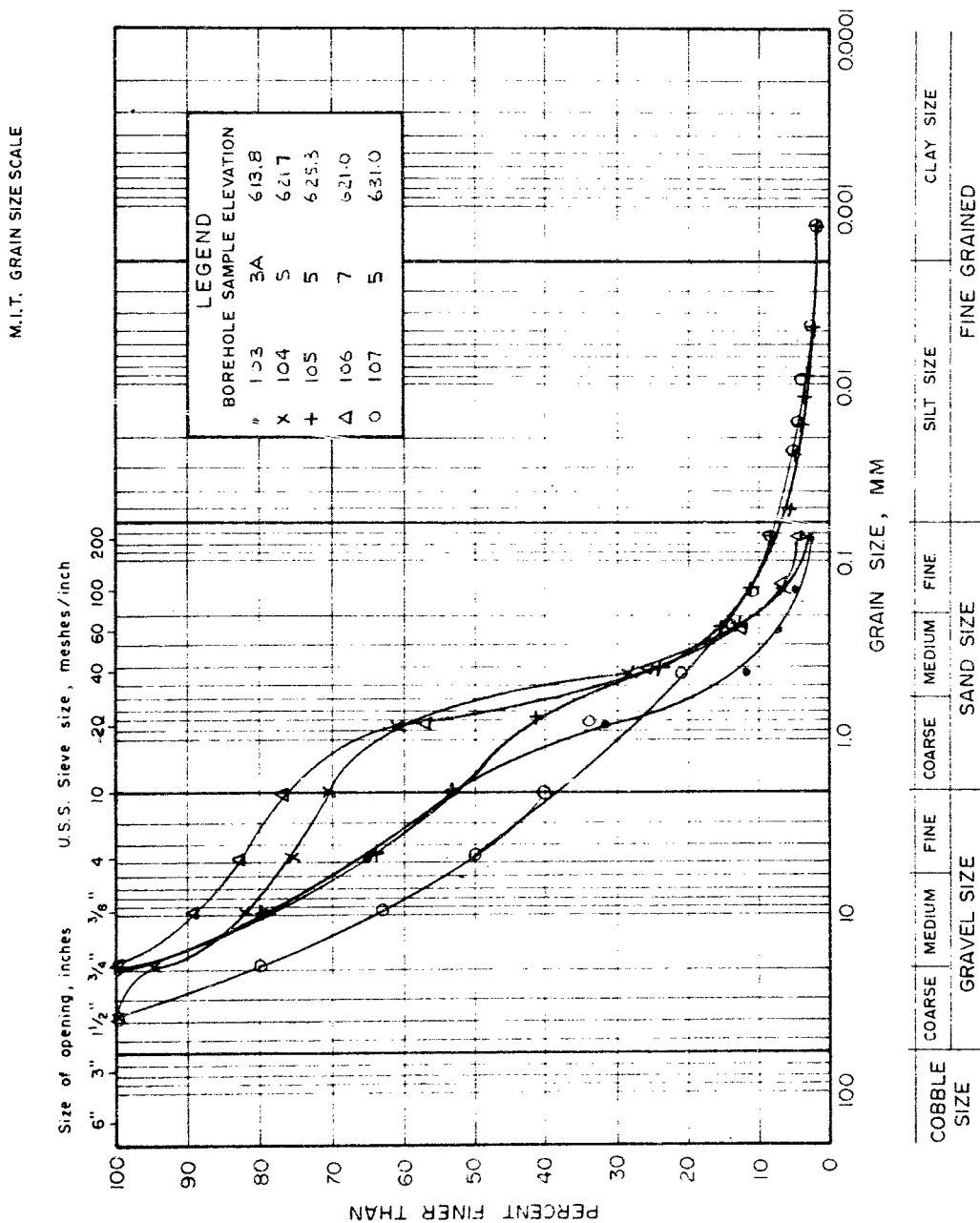
FIGURE 16



Golder Associates

GRAIN SIZE DISTRIBUTION SAND AND GRAVEL

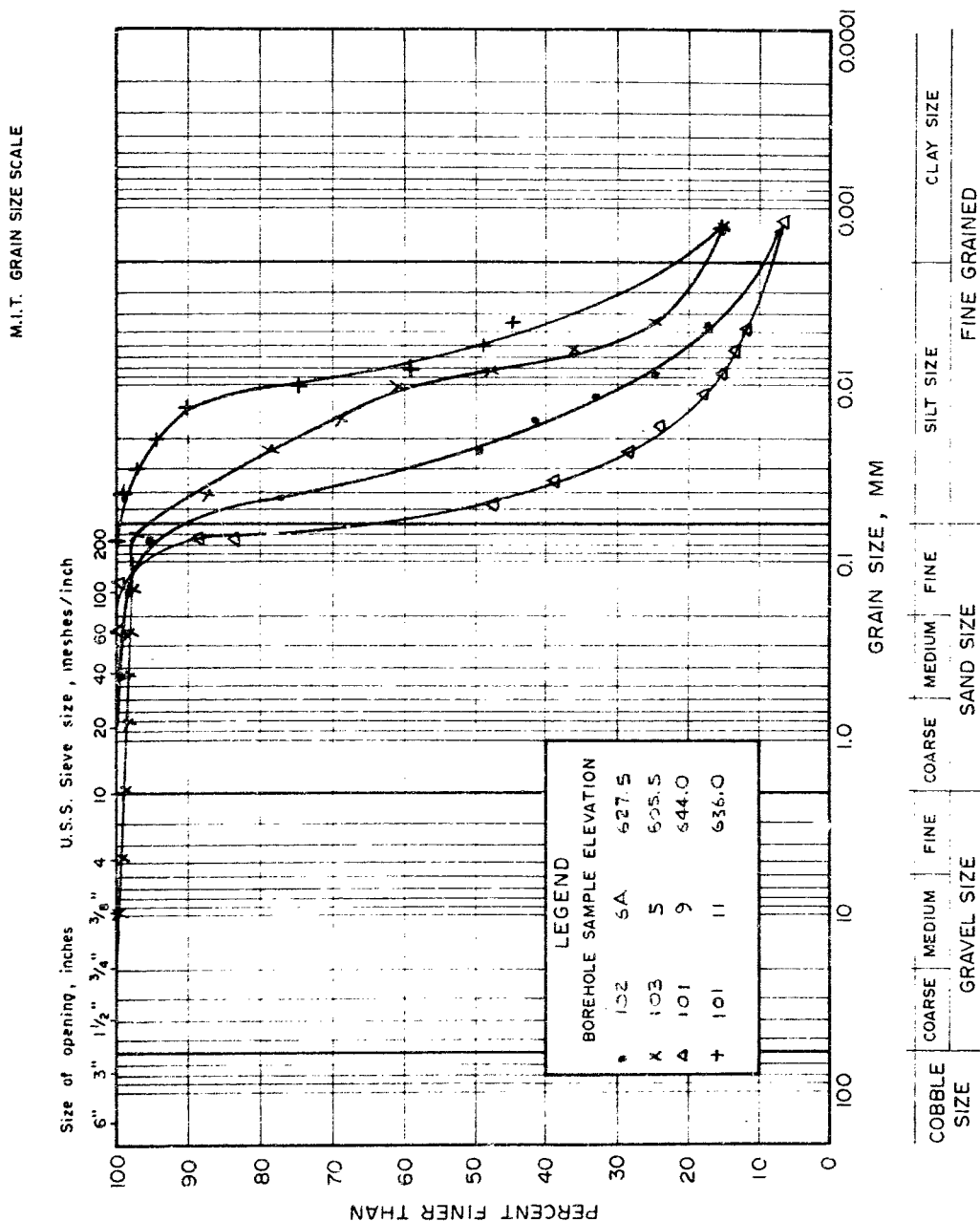
FIGURE 17



Golder Associates

GRAIN SIZE DISTRIBUTION SANDY SILT TO CLAYEY SILT

FIGURE 18

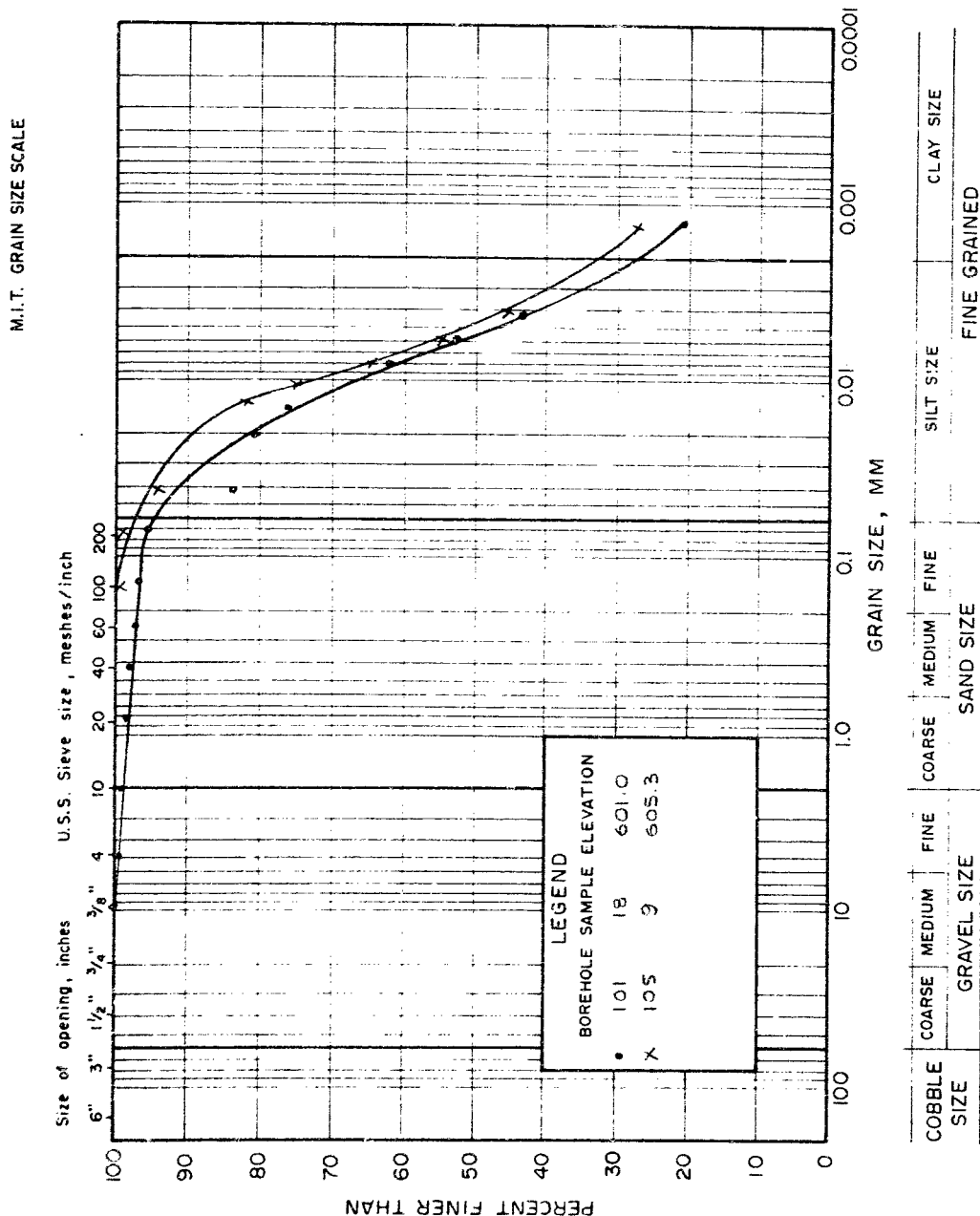


Golder Associates

PROJECT No. 741258

GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY

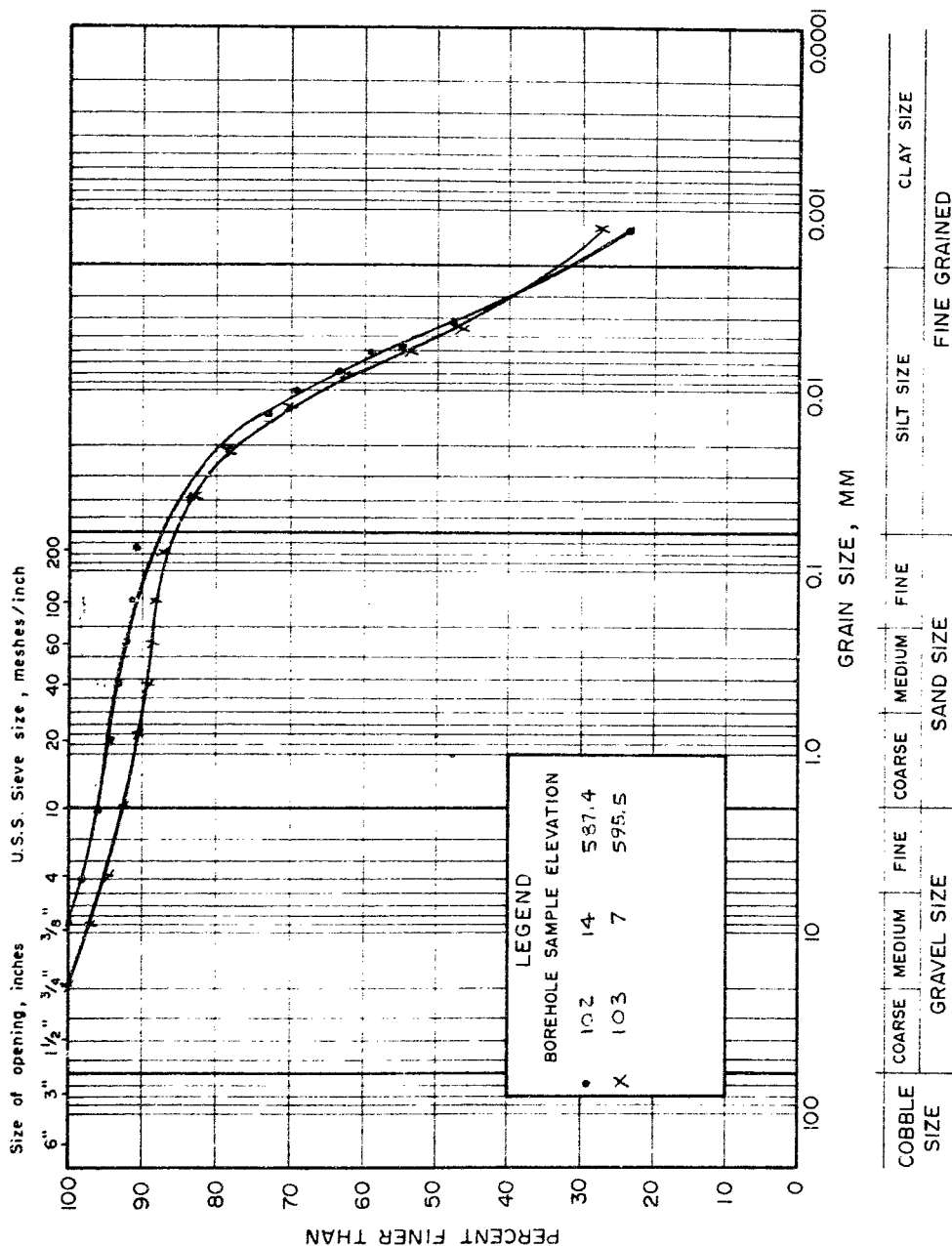
FIGURE 19



GRAIN SIZE DISTRIBUTION SILTY CLAY, TRACK 11 GRAVEL

FIGURE 20

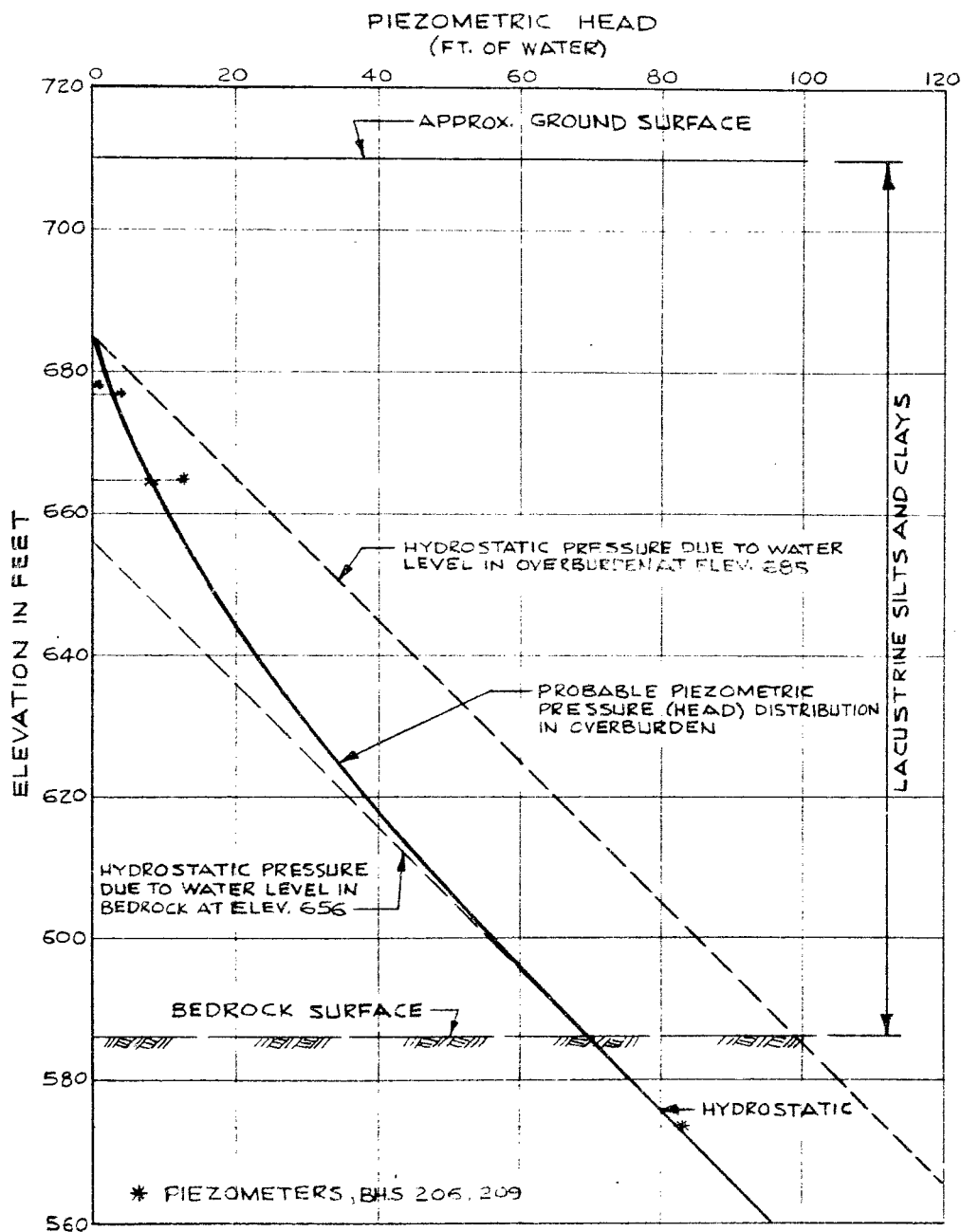
M.I.T. GRAIN SIZE SCALE



Golder Associates

ILLUSTRATION OF VARIATION OF PIEZOMETRIC HEAD
WITH DEPTH—BEHIND CREST OF SOUTH VALLEY BANK

FIGURE 21



Date MAY 9, 1975

Golder Associates

Drawn *my B*
Chkd *ALG*
Appd *ALG*

PROJECT NO. 741258

Sheet 3 of 4

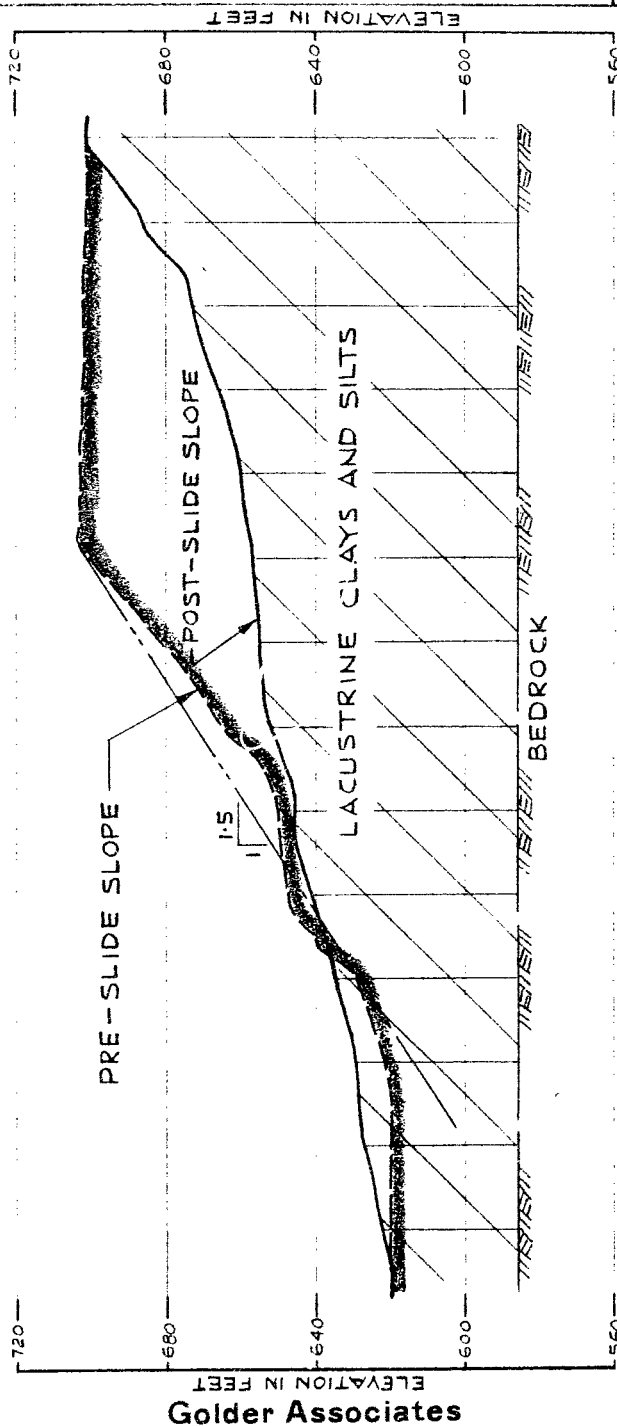
ANALYSES OF THE "1957" FAILURE PRE-SLIDE AND POST-SLIDE SLOPE GEOMETRY

FIGURE 22

REFERENCES:-

- 1- PRE-SLIDE SLOPE GEOMETRY
DRAWING NO. MUN 4201-2 "COCKSHUTT BRIDGE BRANTFORD - ONTARIO
CONTOUR PLAN AND GENERAL ELEVATION" DATED APRIL 10, 1931, REVISED SEPT. 1, 1931
- 2- POST-SLIDE SLOPE GEOMETRY
McCORMICK, RANKIN & ASSOCIATES LIMITED "COCKSHUTT BRIDGE REPLACEMENT
STUDY" PLATE 3 "SCHEME 'A' RECOMMENDED."

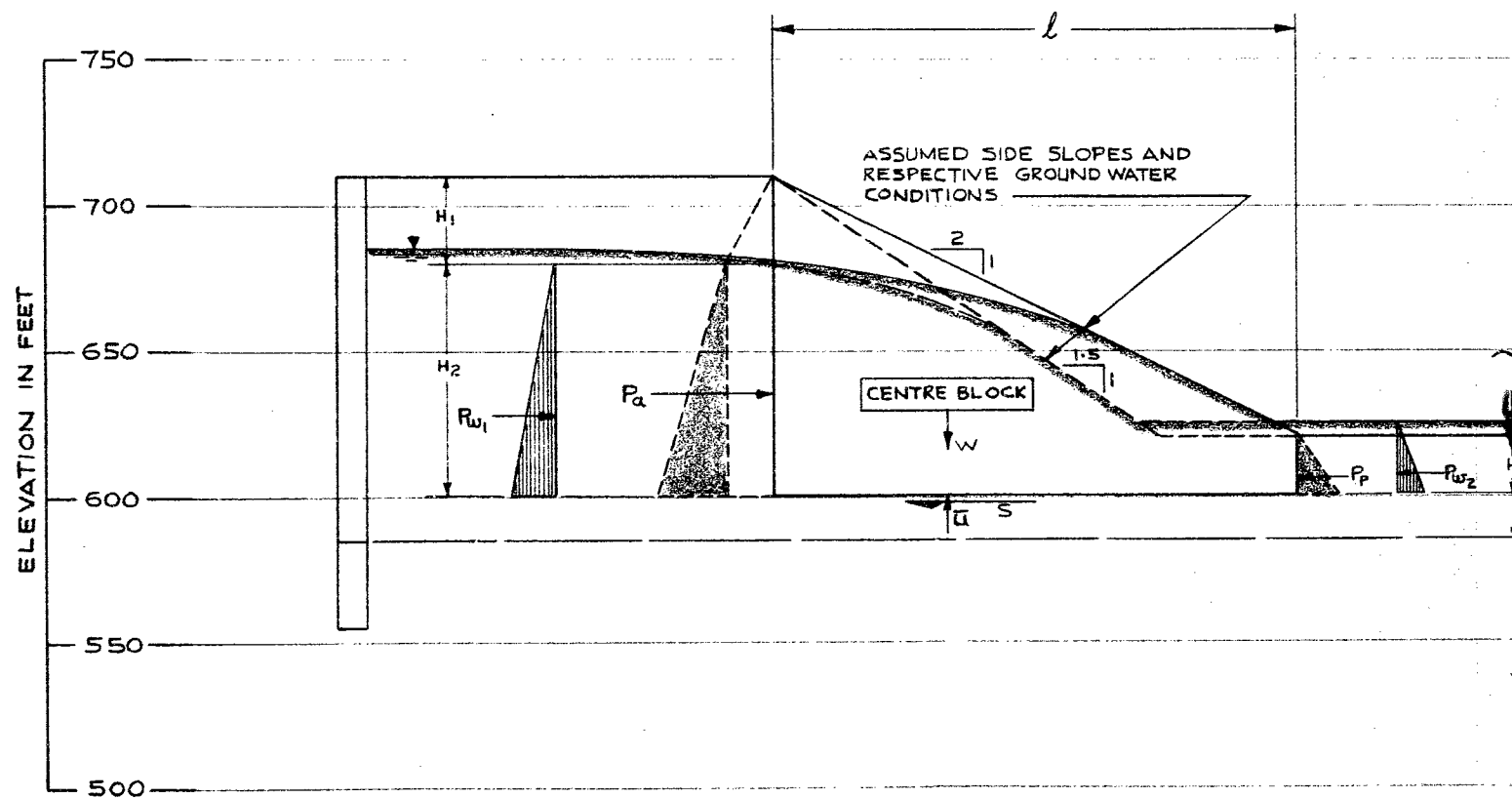
Date MAY 8, 1975



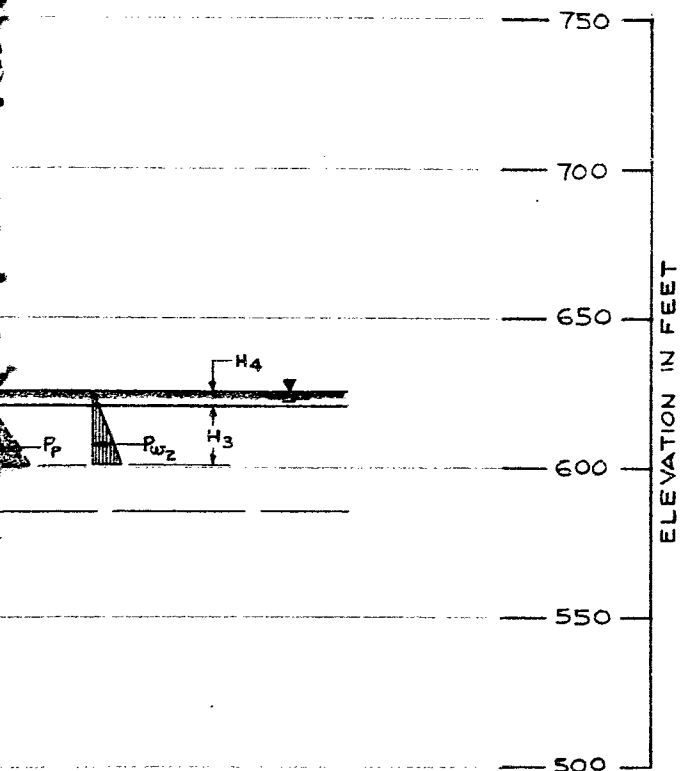
Golder Associates

SCALE: 1" TO 40'

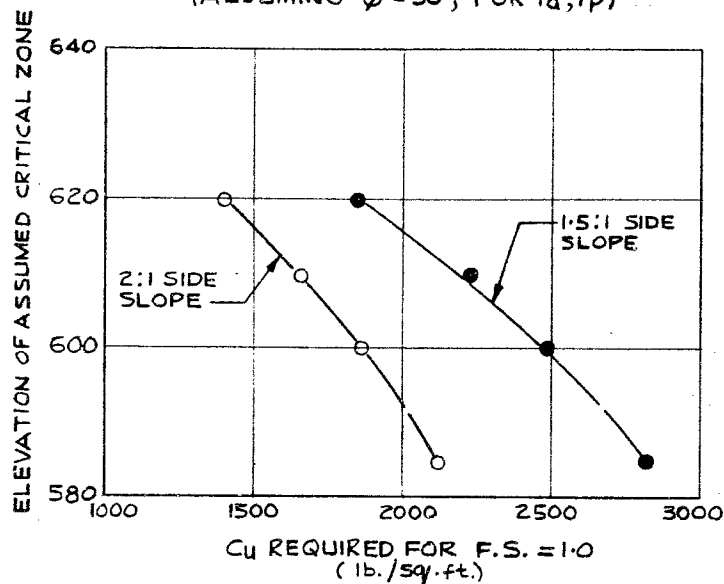
Drawn *m.v.B.*
Chkd *AG*
Appd *AG*



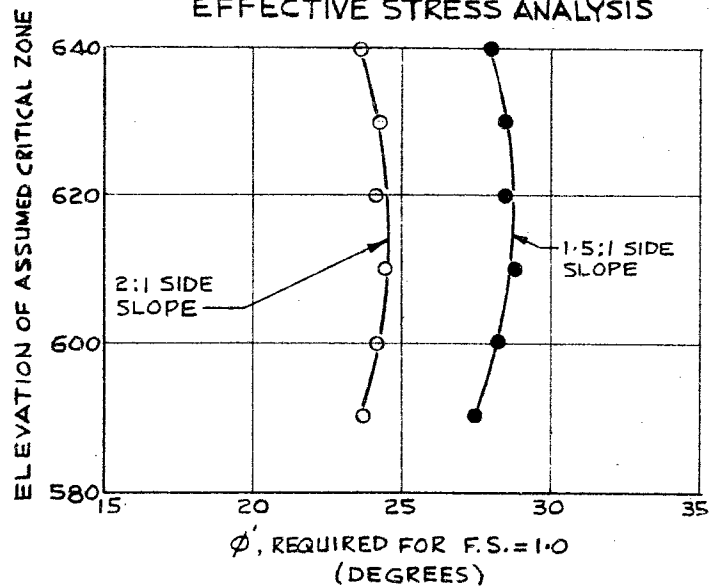
- P_a ——— ACTIVE PRESSURE
 P_p ——— PASSIVE PRESSURE
 P_{w1}, P_{w2} ——— UNBALANCED WATER PRESSURE
 W ——— WEIGHT OF CENTRE BLOCK
 U ——— UPLIFT PRESSURE
 S ——— SHEAR RESISTANCE ALONG BASE PLANE



CASE I TOTAL STRESS (UNDRAINED) ANALYSIS (ASSUMING $\phi' = 30^\circ$, FOR P_a, P_p)



CASE II EFFECTIVE STRESS ANALYSIS

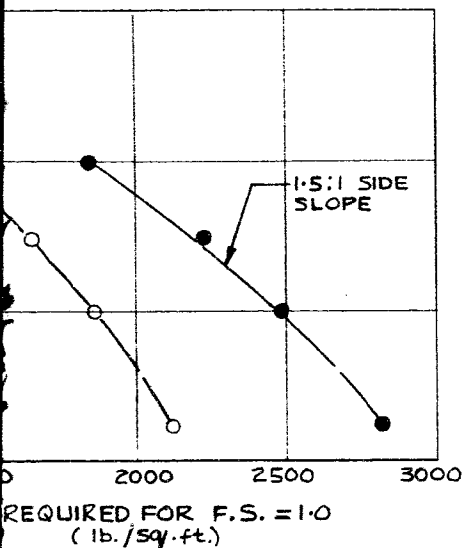


ANALYSIS OF THE '1957" FAILURE SLIDING WEDGE ANALYSIS

FIGURE 23

CASE I

STRESS (UNDRAINED) ANALYSIS
GIVING $\phi' = 30^\circ$, FOR P_a, P_p



ASSUMPTIONS IN BLOCK ANALYSIS

$$\text{ACTIVE PRESSURE } P_a = \left(\frac{1}{2} K_a \gamma' H_1^2 + K_a \gamma' H_1 H_2 + \frac{1}{2} K_a \gamma' H_2^2 \right)$$

$$\text{PASSIVE PRESSURE } P_p = \frac{1}{2} K_p \gamma' H_3^2$$

$$\text{NET DRIVING FORCE} = (P_a + P_{w1}) - (P_p + P_{w2}) = (\bar{P}_a - \bar{P}_p)$$

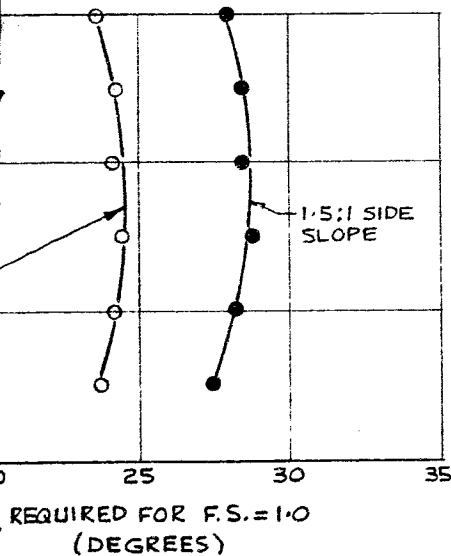
$$\text{NET RESISTING FORCE, } \bar{R} = (W - U) \tan \phi \text{ OR } \bar{R} = S = C u l.$$

FACTOR OF SAFETY

$$F.S. = \frac{\text{NET RESISTING FORCE}}{\text{NET DRIVING FORCE}} = \frac{\bar{R}}{(\bar{P}_a - \bar{P}_p)}$$

CASE II

EFFECTIVE STRESS ANALYSIS



SPECIAL NOTE

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

40 P1-69

GEOCRE No.

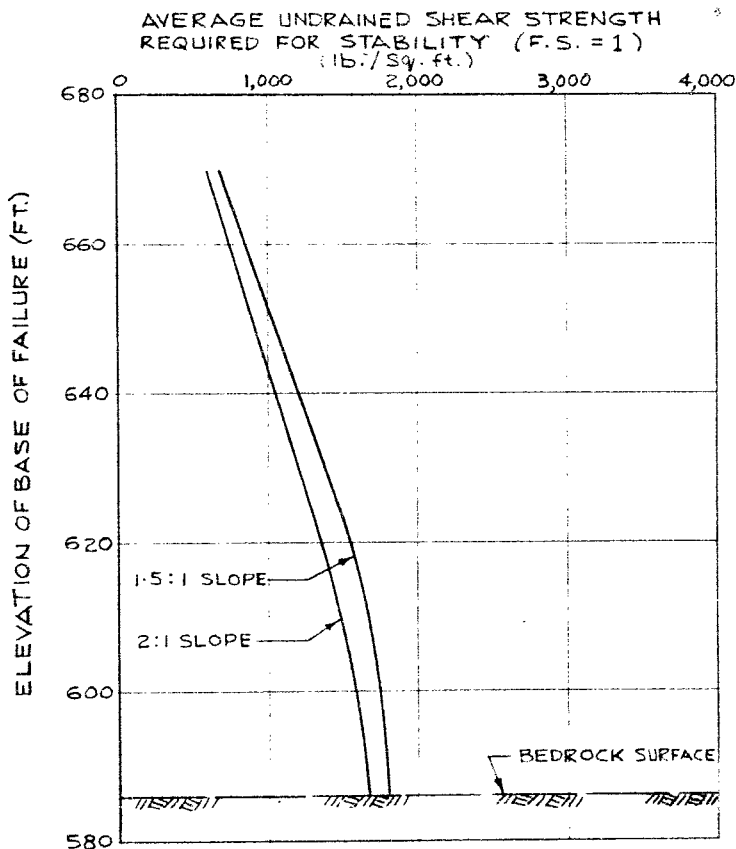
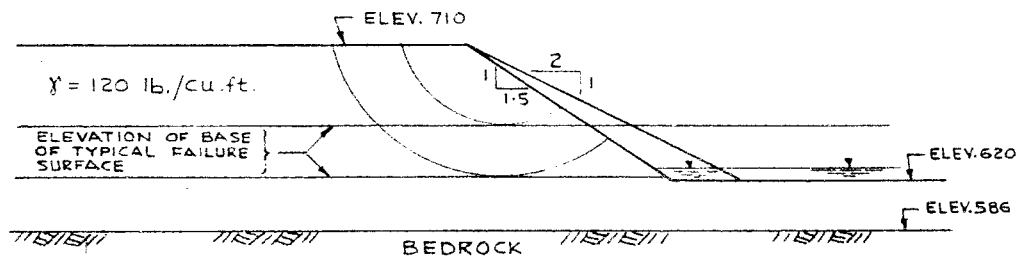
Date MAY 7, 1975

Golder Associates

Drawn *m.g.B.*
Chkd. *B.G.*
Appd. *mas*

ANALYSES OF "1957" FAILURE TOTAL STRESS STABILITY ANALYSIS — PUBLISHED DESIGN CHARTS

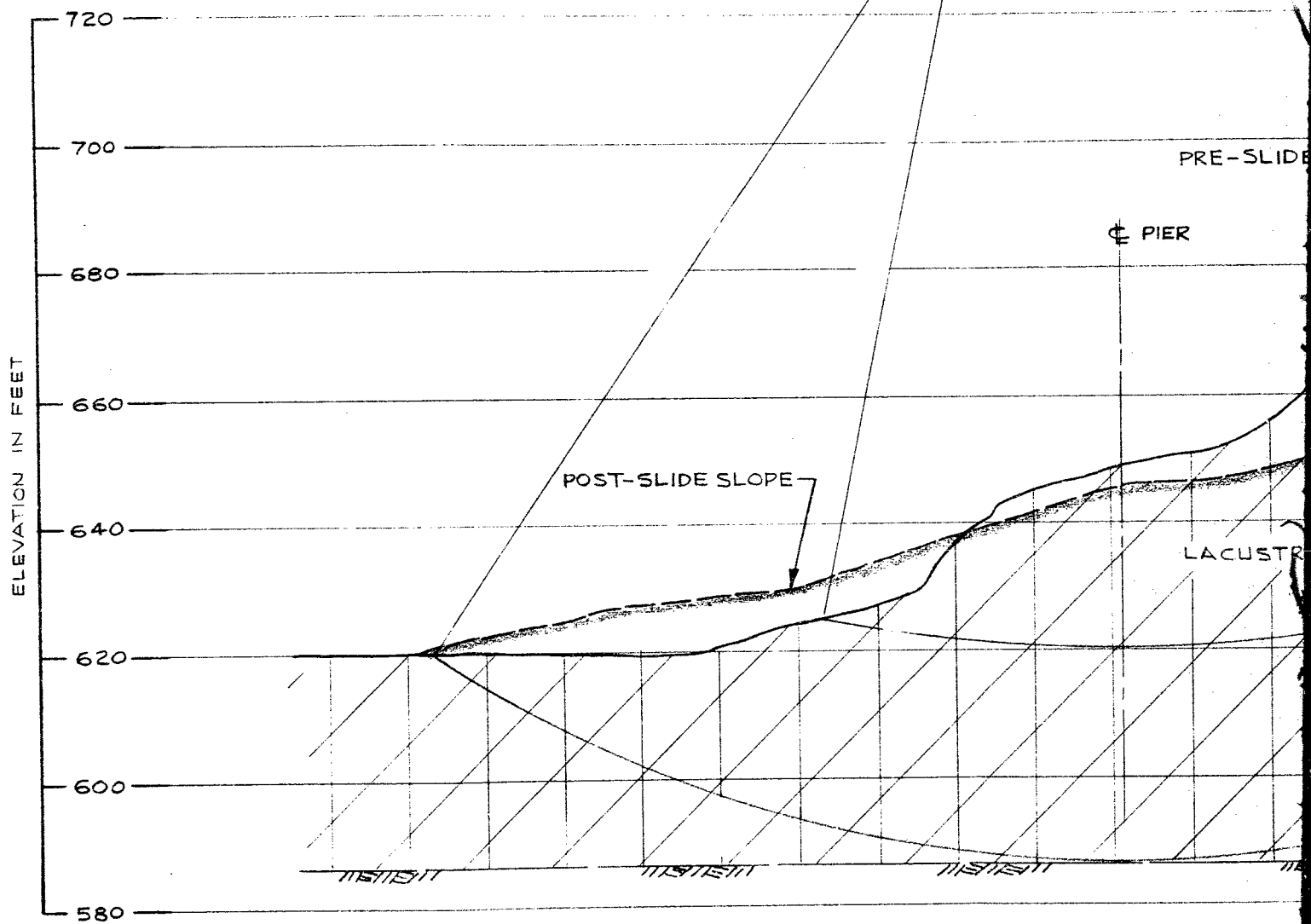
FIGURE 24



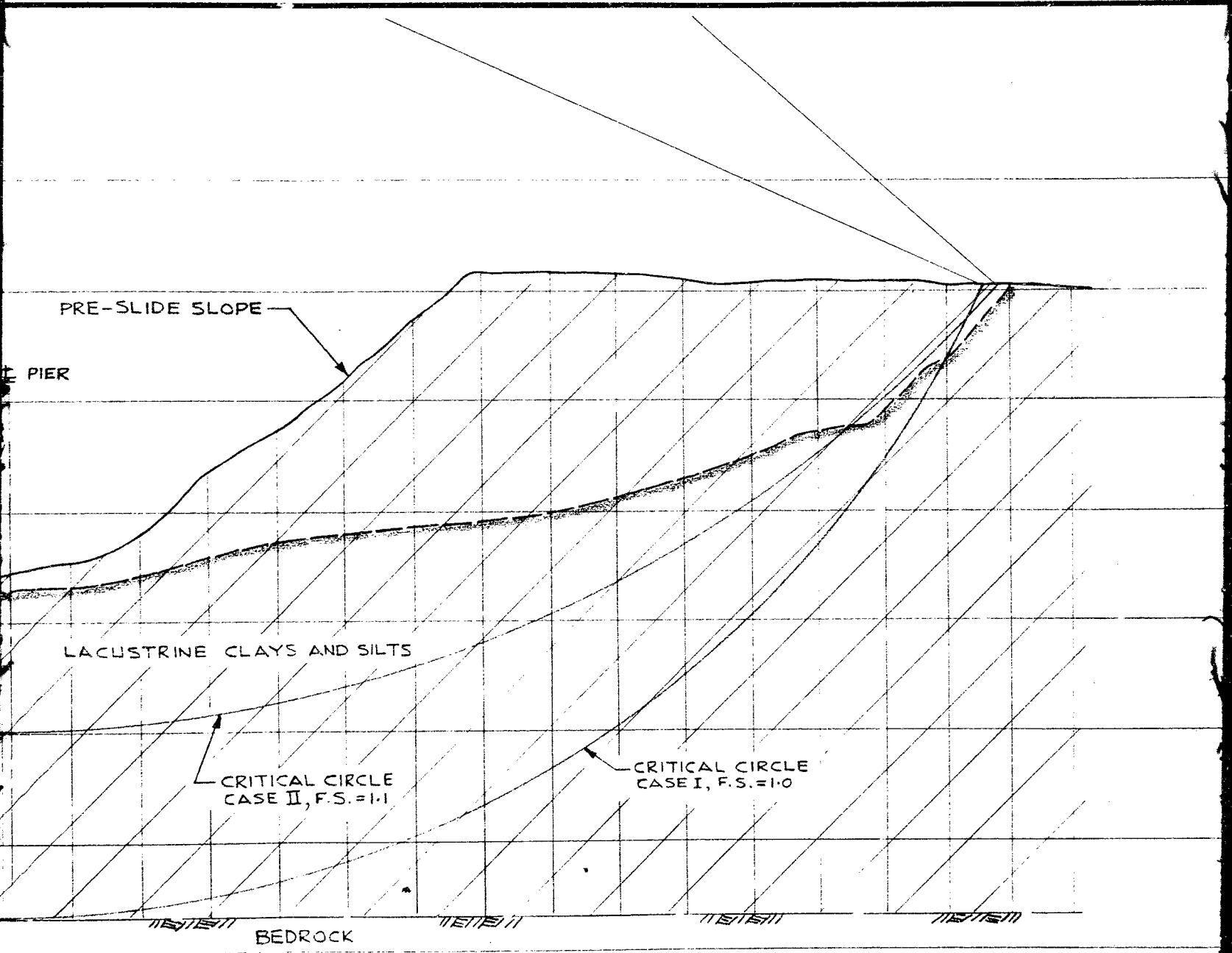
Date MAY 6, 1975

Golder Associates

Drawn *mjb*
Chkd *RG*
Appd *RG*



SCALE:



SCALE: 1" TO 20'

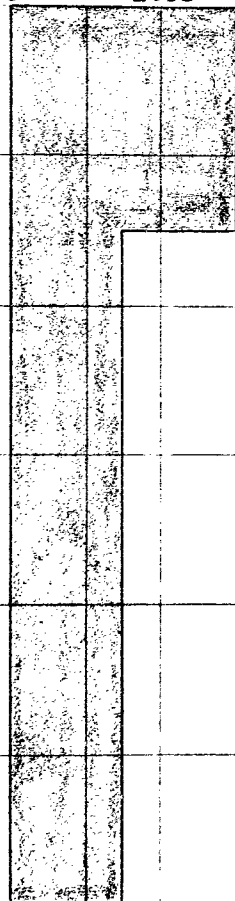
ANALYSES OF THE "1957" FAILURE TOTAL STRESS STABILITY ANALYSIS

FIGURE 25

CASE I

UNDRAINED SHEAR STRENGTH
USED FOR ANALYSIS
(lb./sq.ft.)

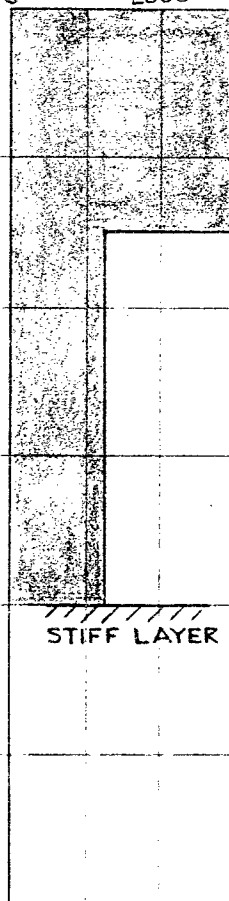
0 2000 4000



CASE II

UNDRAINED SHEAR STRENGTH
USED FOR ANALYSIS
(lb./sq.ft.)

0 2000 4000



STIFF LAYER

ELEVATION IN FEET

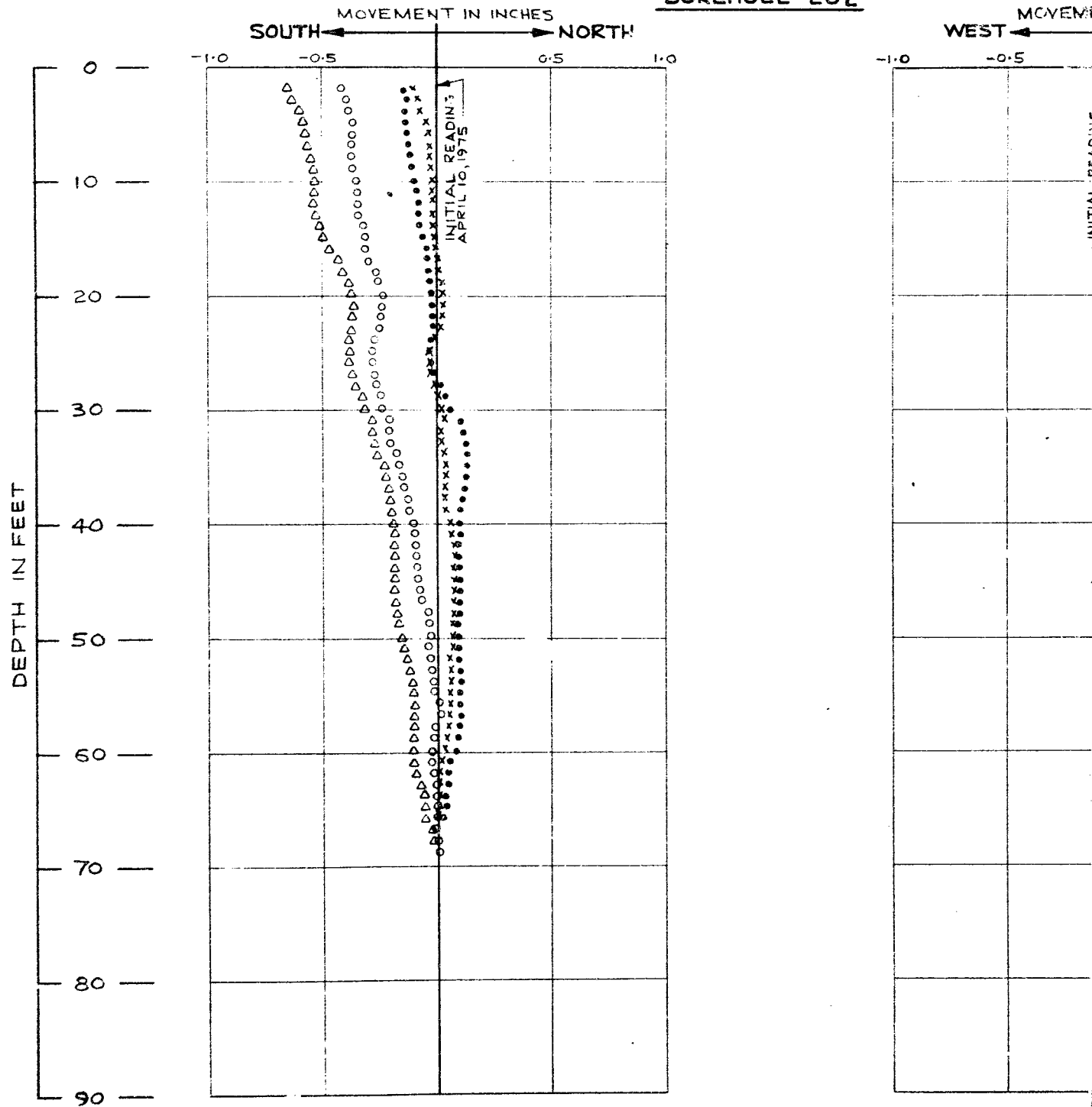
40P1-69
GEOTECH No.

Date MAY 10, 1975

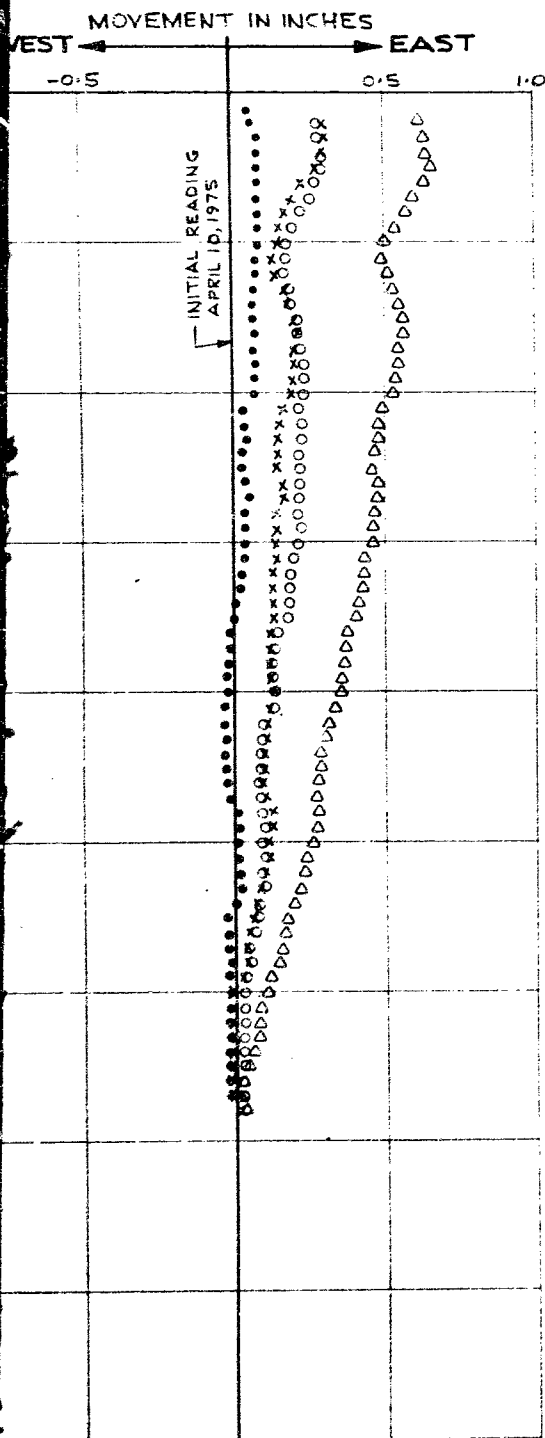
Golder Associates

Drawn *m.j.B.*
Chkd *B.S.*
Appd *B.S.*

SLOPE INDICATOR No. 1 BOREHOLE 202



SLOPE IND BOREH



0

10

20

30

40

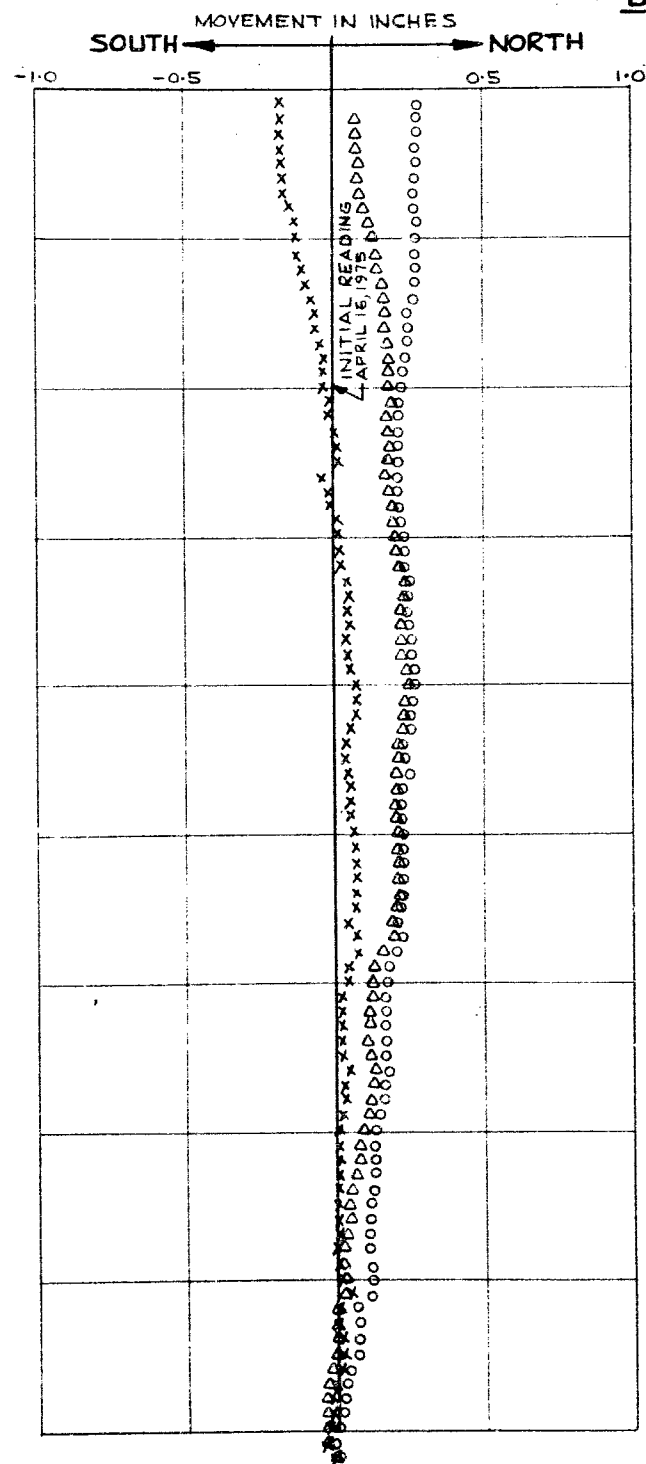
50

60

70

80

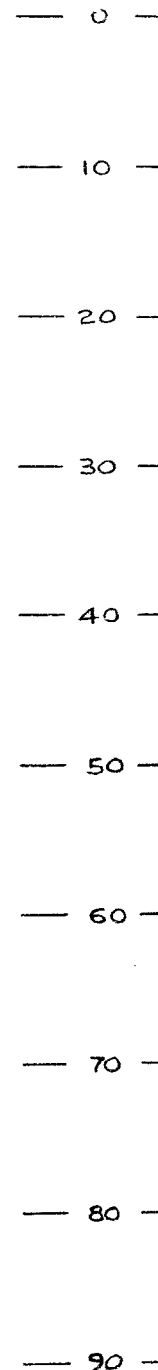
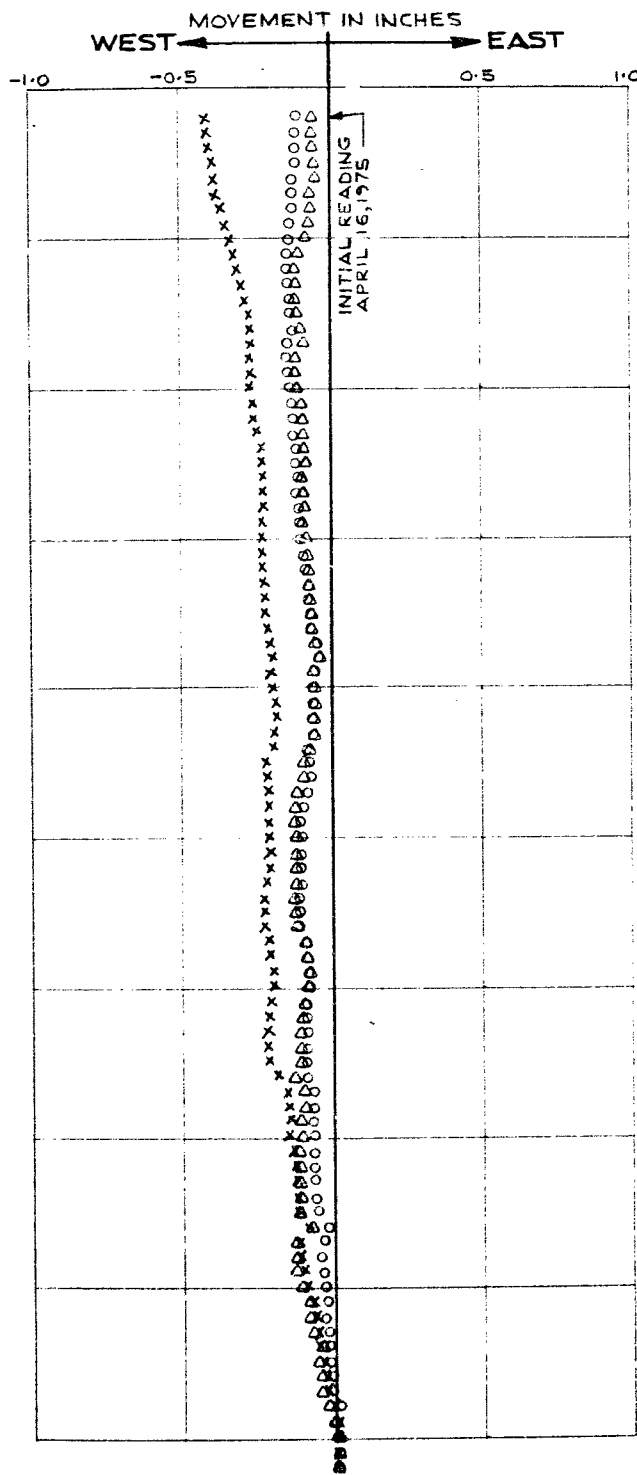
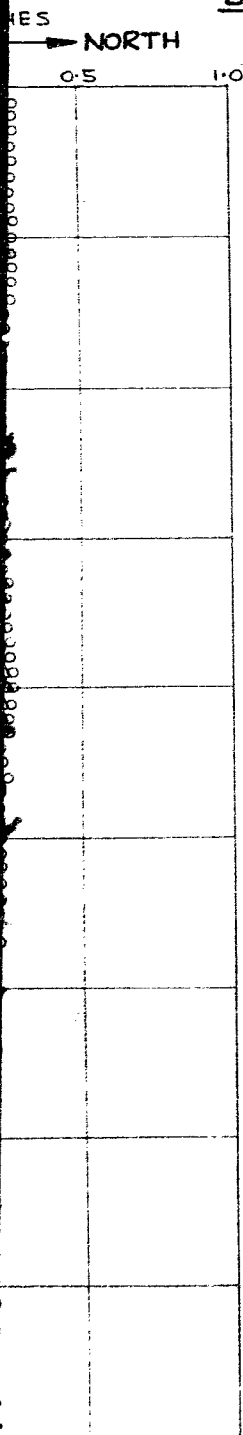
90



SLOPE INDICATOR No. 2

BOREHOLE 205

SU
SLOPE



LEGEND

- SLOPE
- X SLOPE
- o SLOPE
- △ SLOPE

NOTE:

FOR INSTALL
SHAPE OF
BOREHOLE

Date MAY 12, 1975

SUMMARY OF SLOPE INDICATOR READINGS

FIGURE 26

INCHES
→ EAST

0.5 1.0

APRIL 16, 1975

0
10
20
30
40
50
60
70
80
90

DEPTH IN FEET

LEGEND

- SLOPE INDICATOR READING, APRIL 16, 1975
- X SLOPE INDICATOR READING, APRIL 21, 1975
- o SLOPE INDICATOR READING, APRIL 30, 1975
- Δ SLOPE INDICATOR READING, MAY 7, 1975

NOTE:

FOR INSTALLATION DETAILS AND INITIAL DEFLECTED SHAPE OF CASINGS ETC. REFER TO RECORD OF BOREHOLE SHEETS.

SPECIAL NOTE

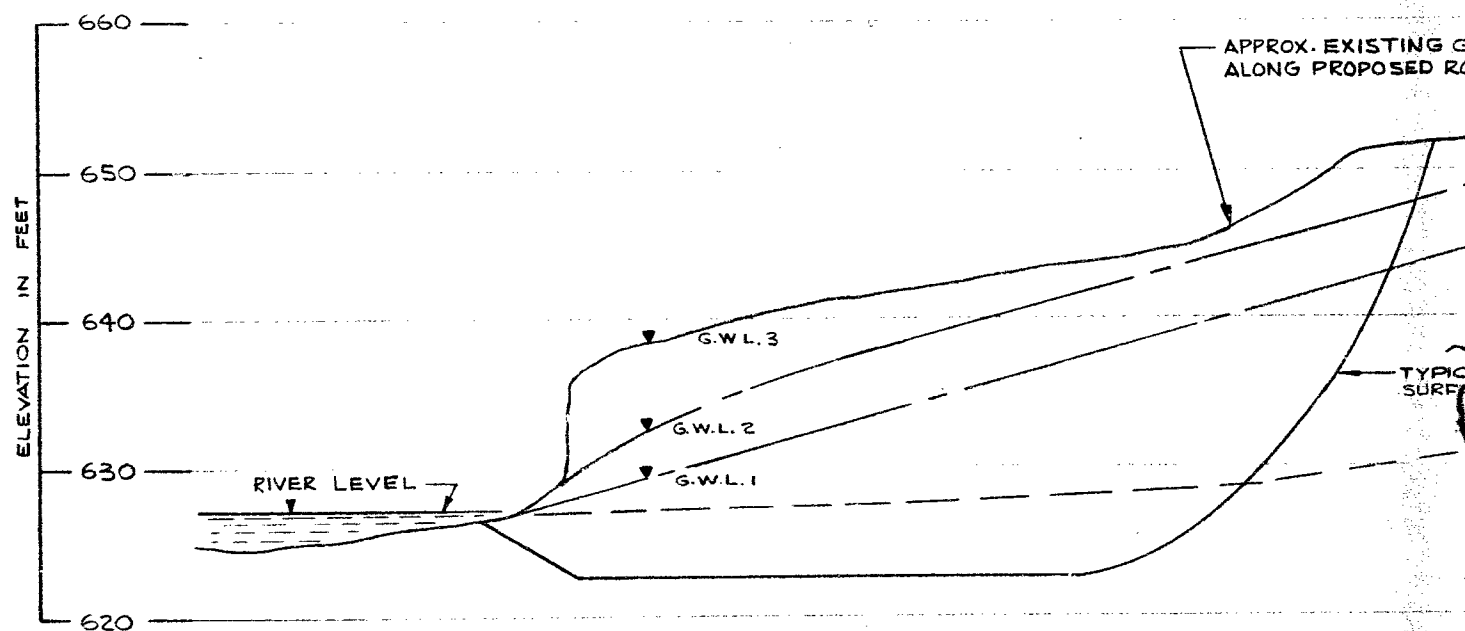
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

40P1-69

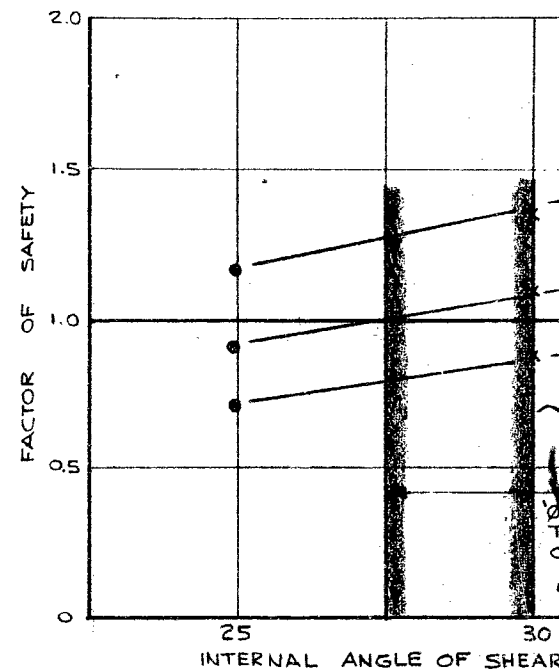
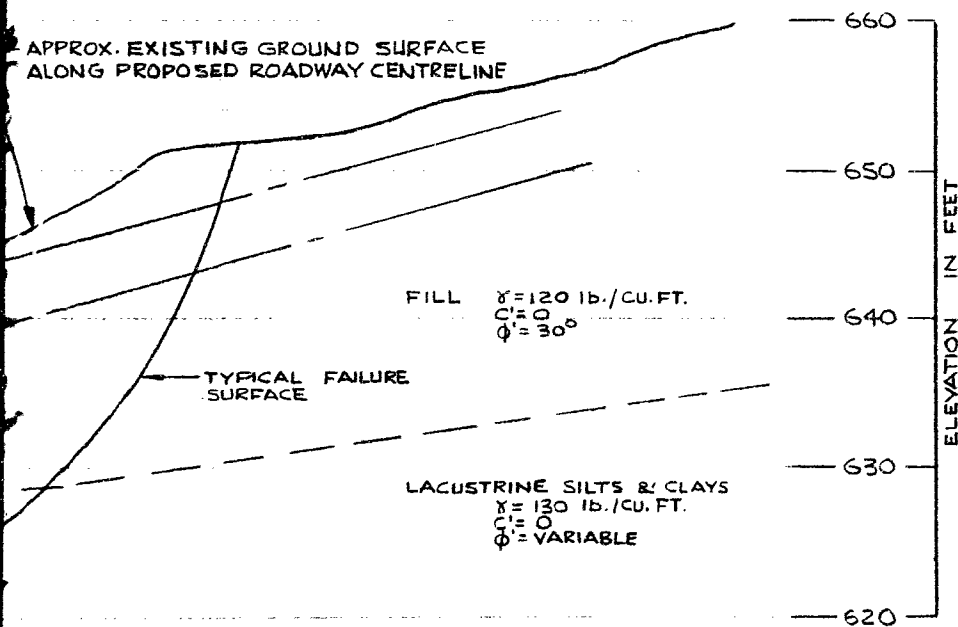
Date MAY 12, 1975

Golder Associates

Drawn *M. J. B.*
Chkd. *RG*
Appd. *[Signature]*



SCALE: 1" TO 10'

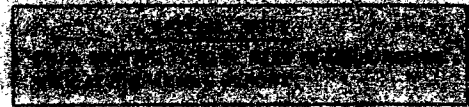
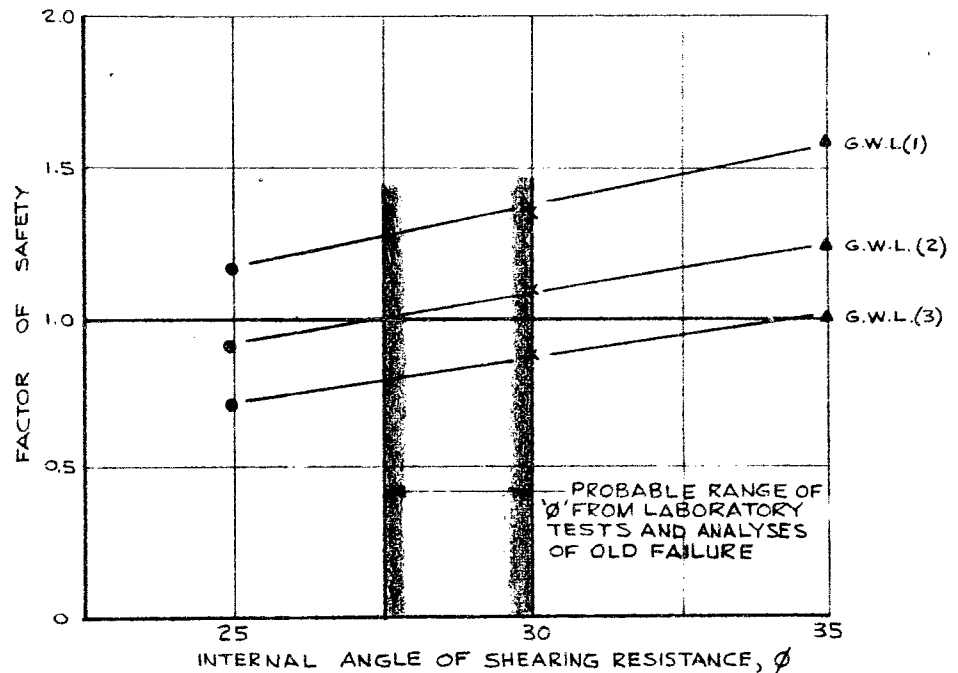
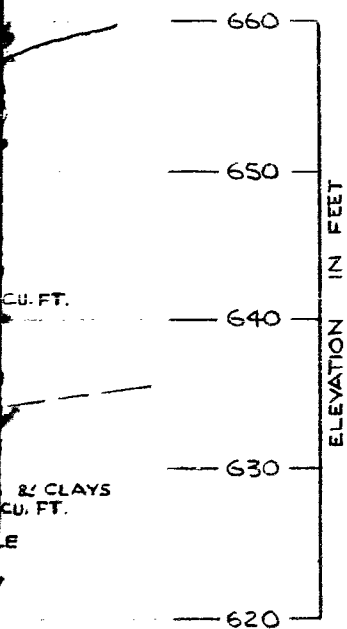


SCALE: 1" TO 10'

Date MAR 4, 1975

40P
GEO

Golder A



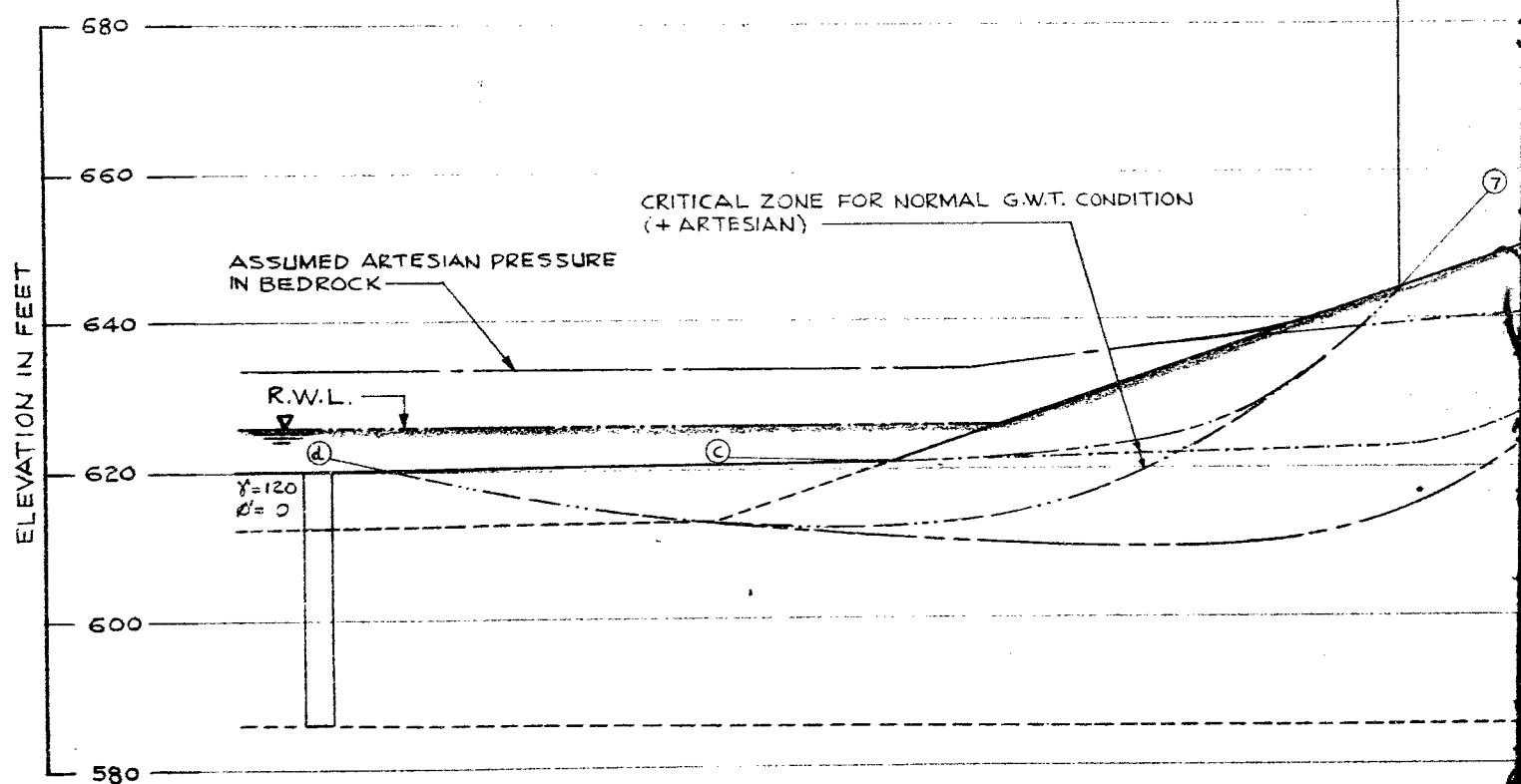
ADP1-69
GEOCRES No.

Date MAR. 4, 1975

Golder Associates

Drawn D.M.
Chkd. BG
Appd. [Signature]

FACTOR OF SAFETY
ASSUMING NORMAL



FACTOR OF SAFETY
ASSUMING NORMAL G.W.L.

2.0

FACTOR OF SAFETY

1.0

0

ASSUMED G.W.T. DURING
FLOODING PERIOD.

ASSUMED NORMAL G.W.L.

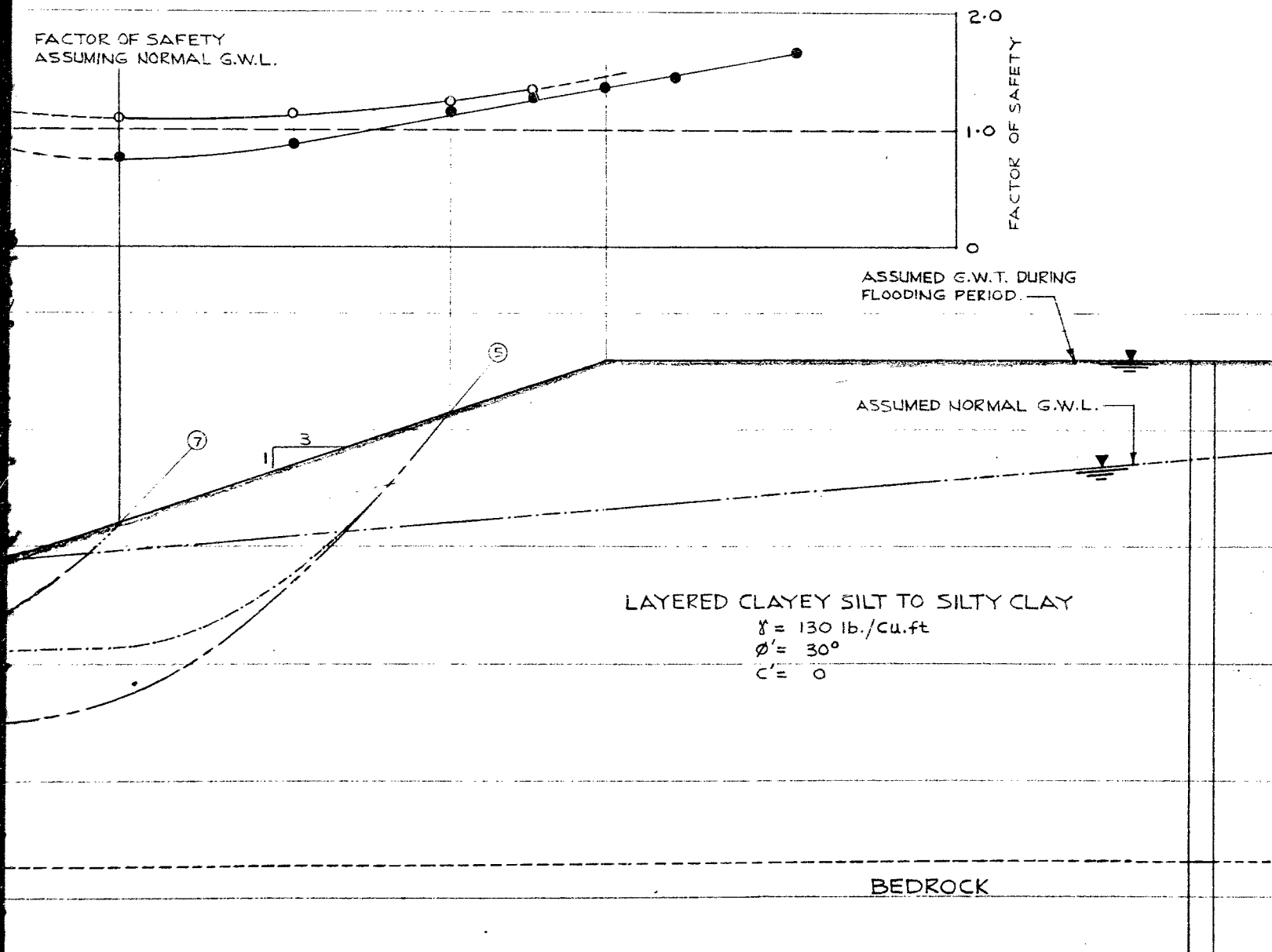
LAYERED CLAYEY SILT TO SILTY CLAY

$\gamma = 130 \text{ lb./cu.ft}$

$\phi' = 30^\circ$

$c' = 0$

BEDROCK

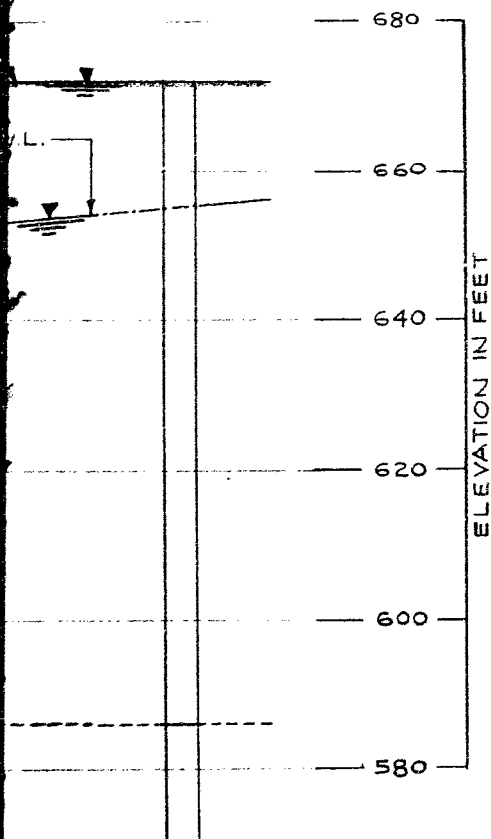


SUMMARY OF STABILITY ANALYSES TYPICAL SLOPE

FIGURE 28

LEGEND

- TRIAL SURFACES TANGENT TO ELEV. 620-624
CLAYEY SILT $\phi' = 30^\circ$, $C' = 0$, $\gamma = 130$, $\gamma' = 65$
- TRIAL SURFACES TANGENT TO ELEV. 610
CLAYEY SILT $\phi' = 30^\circ$, $C' = 0$, $\gamma = 130$, $\gamma' = 65$
SAND AND GRAVEL $\phi' = 0$, $C' = 0$, $\gamma = 120$, $\gamma' = 57$



SCALE: 1" TO 20'

40P1-69
GEOCRE No.

Date MAR. 24, 1975

Golder Associates

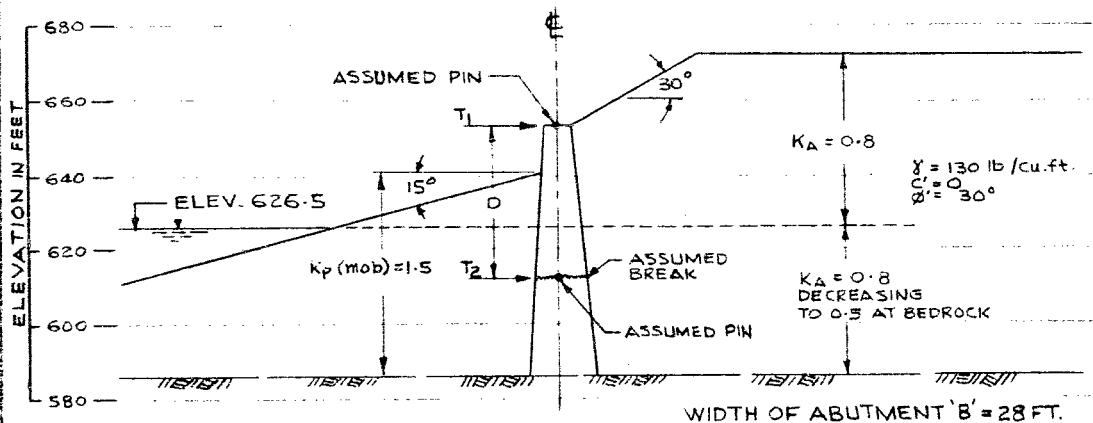
Drawn *mjb*
Chkd *BB*
Appd *[Signature]*

EARTH PRESSURES ON EXISTING SOUTH ABUTMENT

FIGURE 29

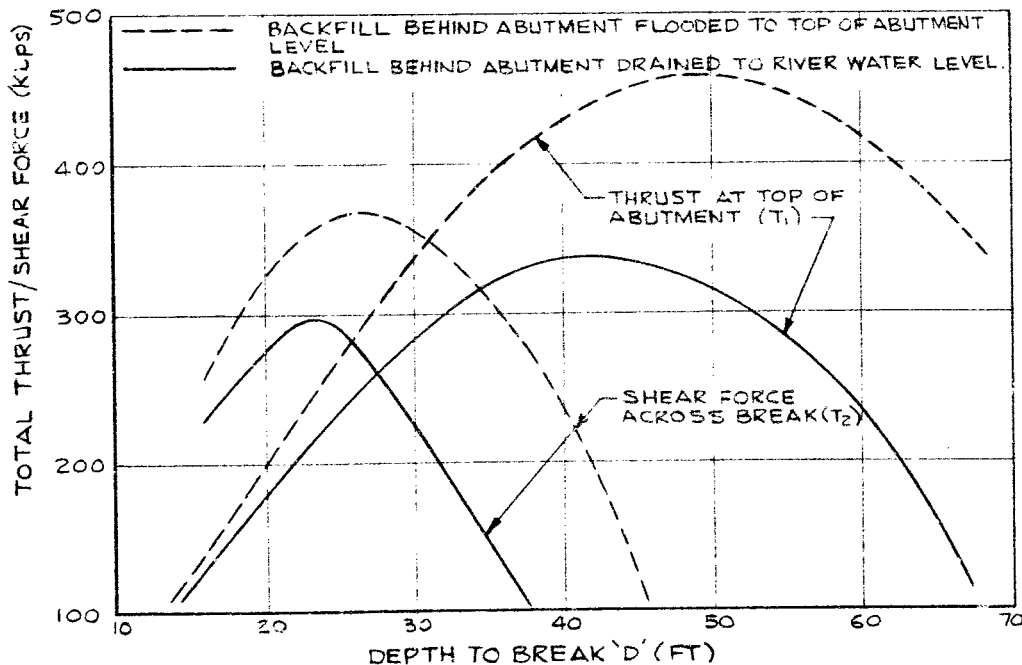
T_1 - THRUST AT TOP OF ABUTMENT DUE TO ROTATION AT BREAK.

T_2 - SHEAR FORCE AT BREAK DUE TO ROTATION AT TOP OF ABUTMENT.



KEY SECTION

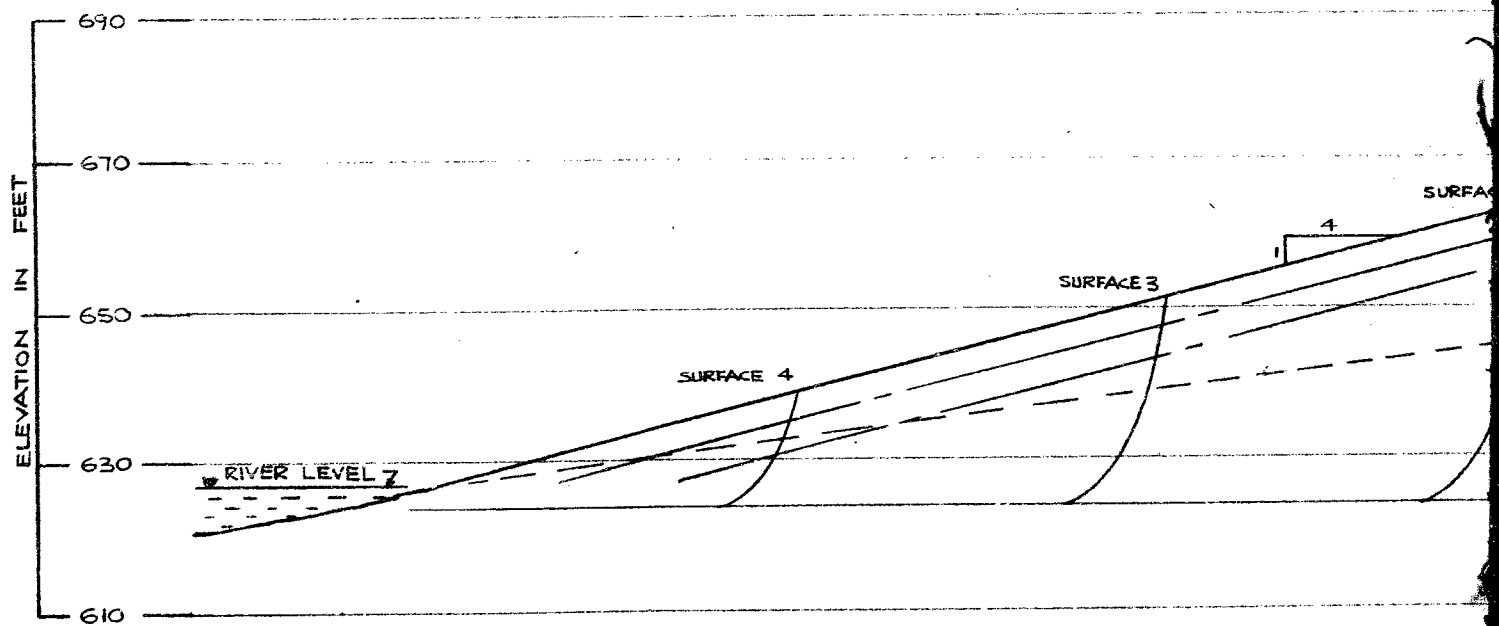
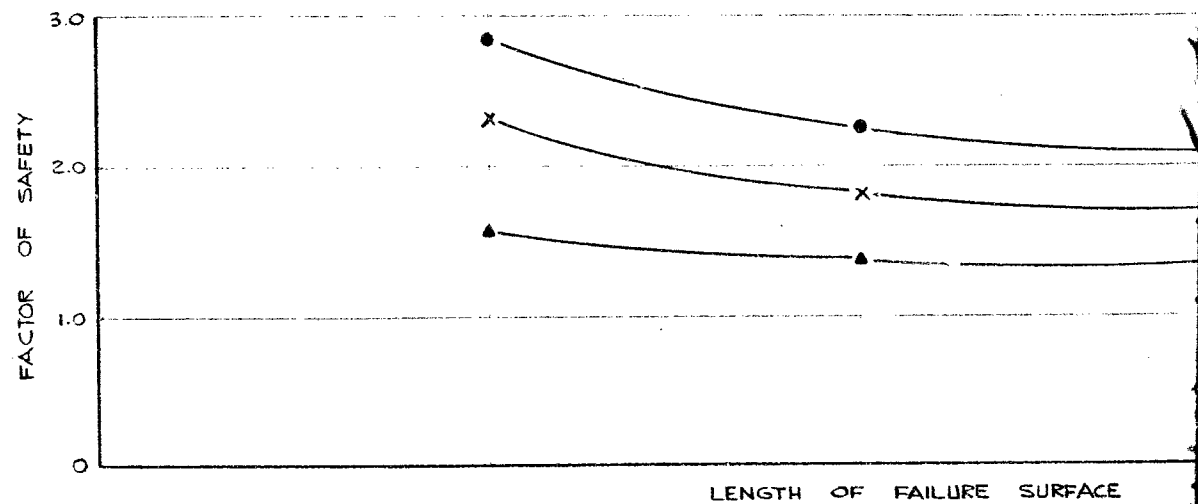
SCALE: 1" TO 40'



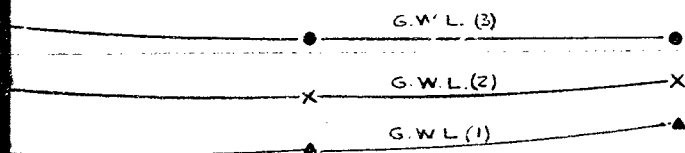
Date MAY 10, 1975

Golder Associates

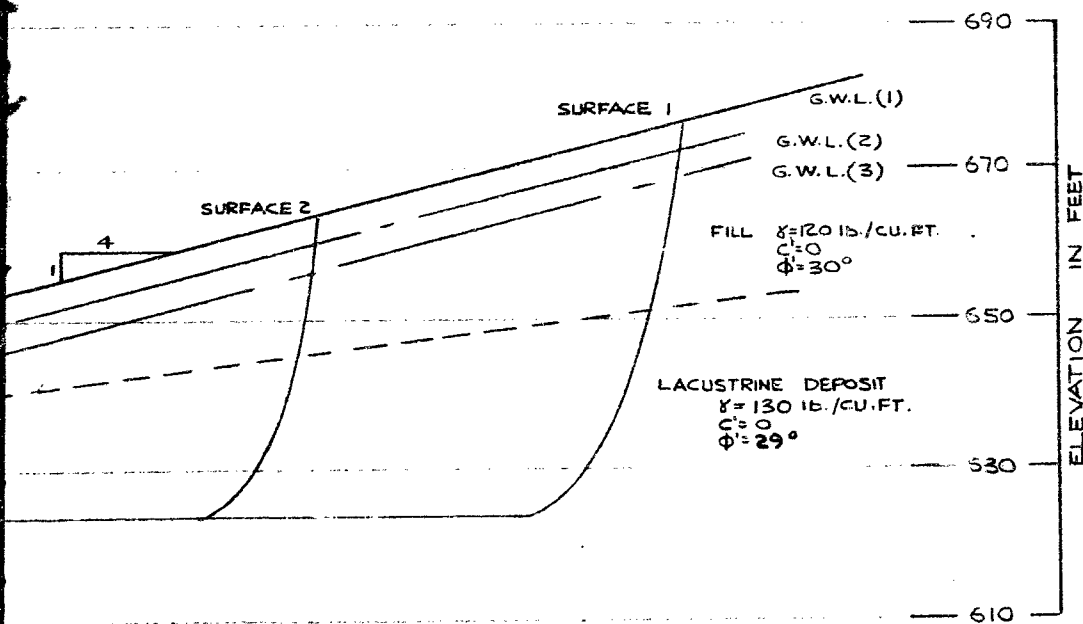
Drawn *m.j.b.*
Chkd *RG*
Appd



SUMMARY OF STABILITY PROPOSED SLOPE



FAILURE SURFACE



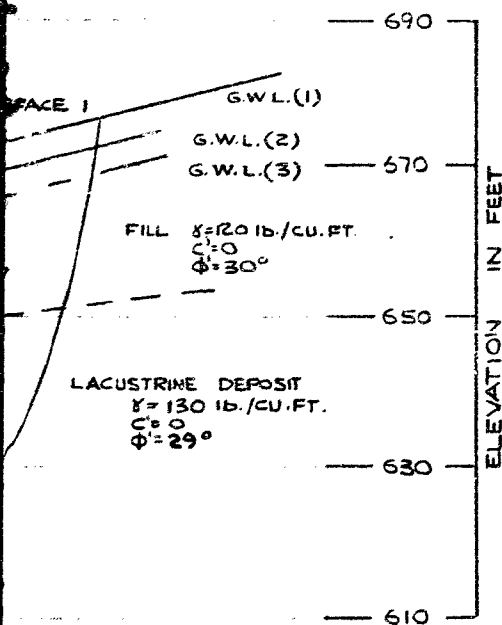
SCALE

Date MAR 4, 1975

Golder A

SUMMARY OF STABILITY ANALYSIS PROPOSED SLOPE

FIGURE 30



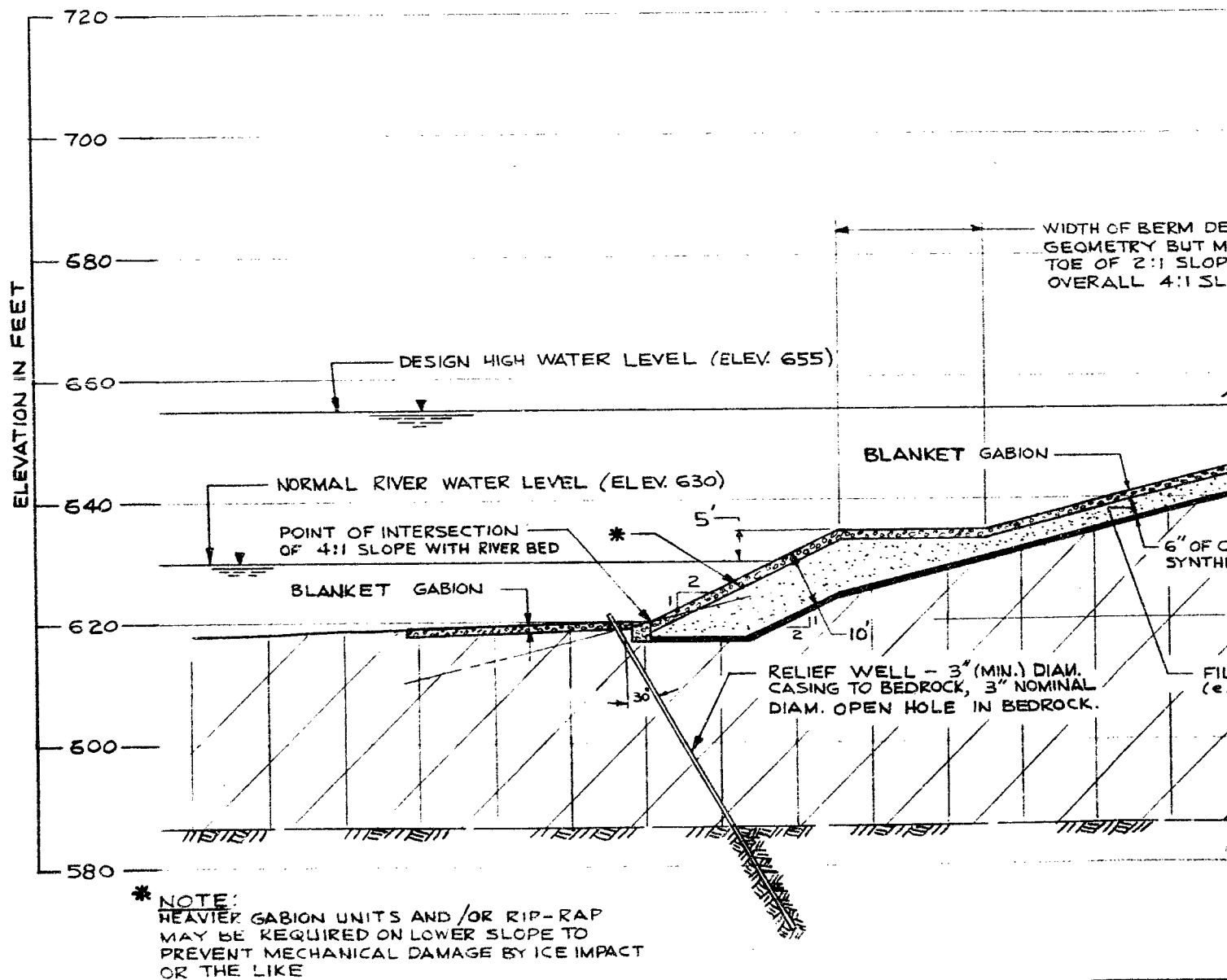
SCALE: 1" TO 20'

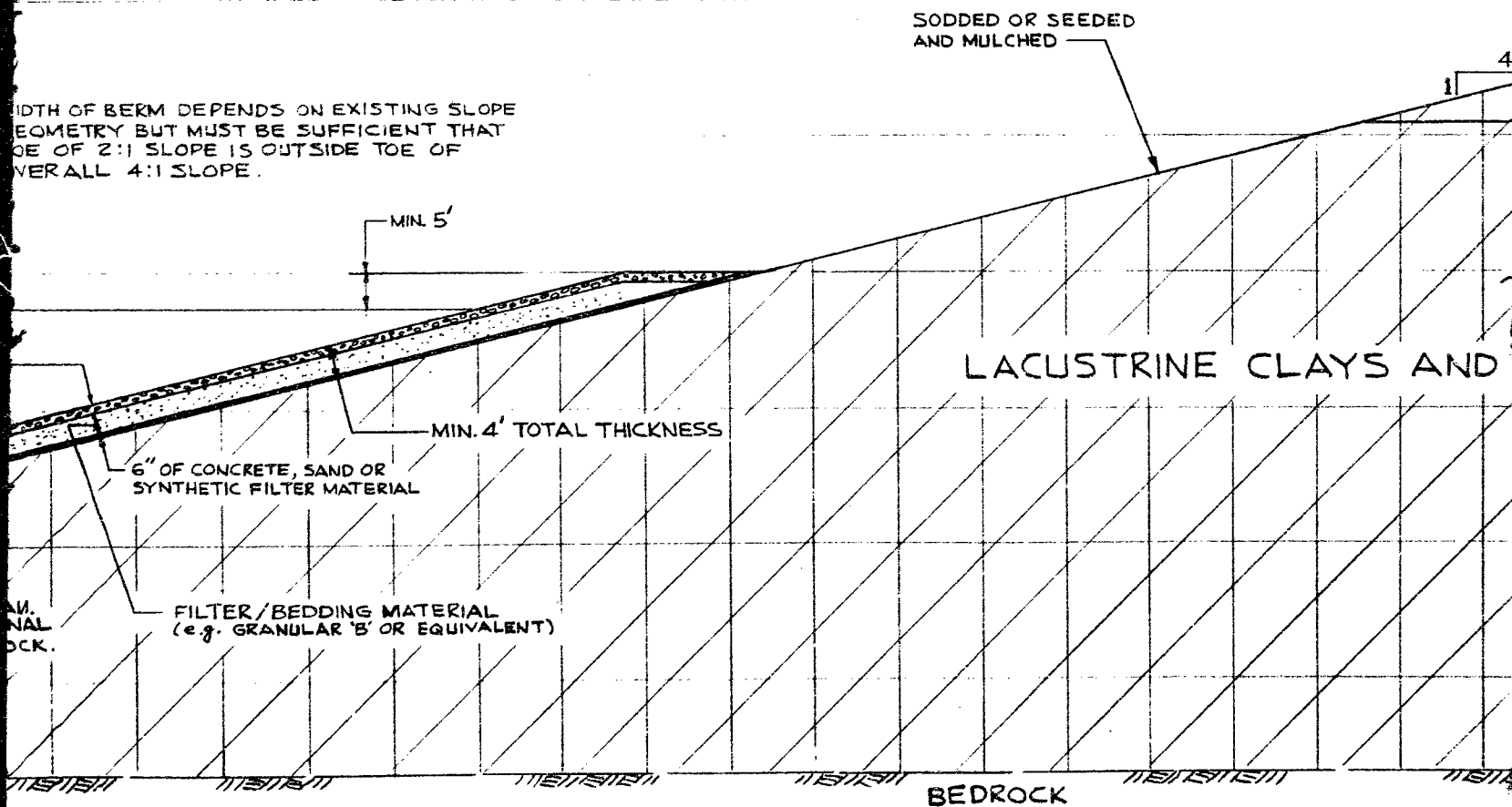
HOPI-69
GEOCRES No.

Date MAR 4, 1975

Golder Associates

Drawn D.M.
Chkd. RG
Appd. jm



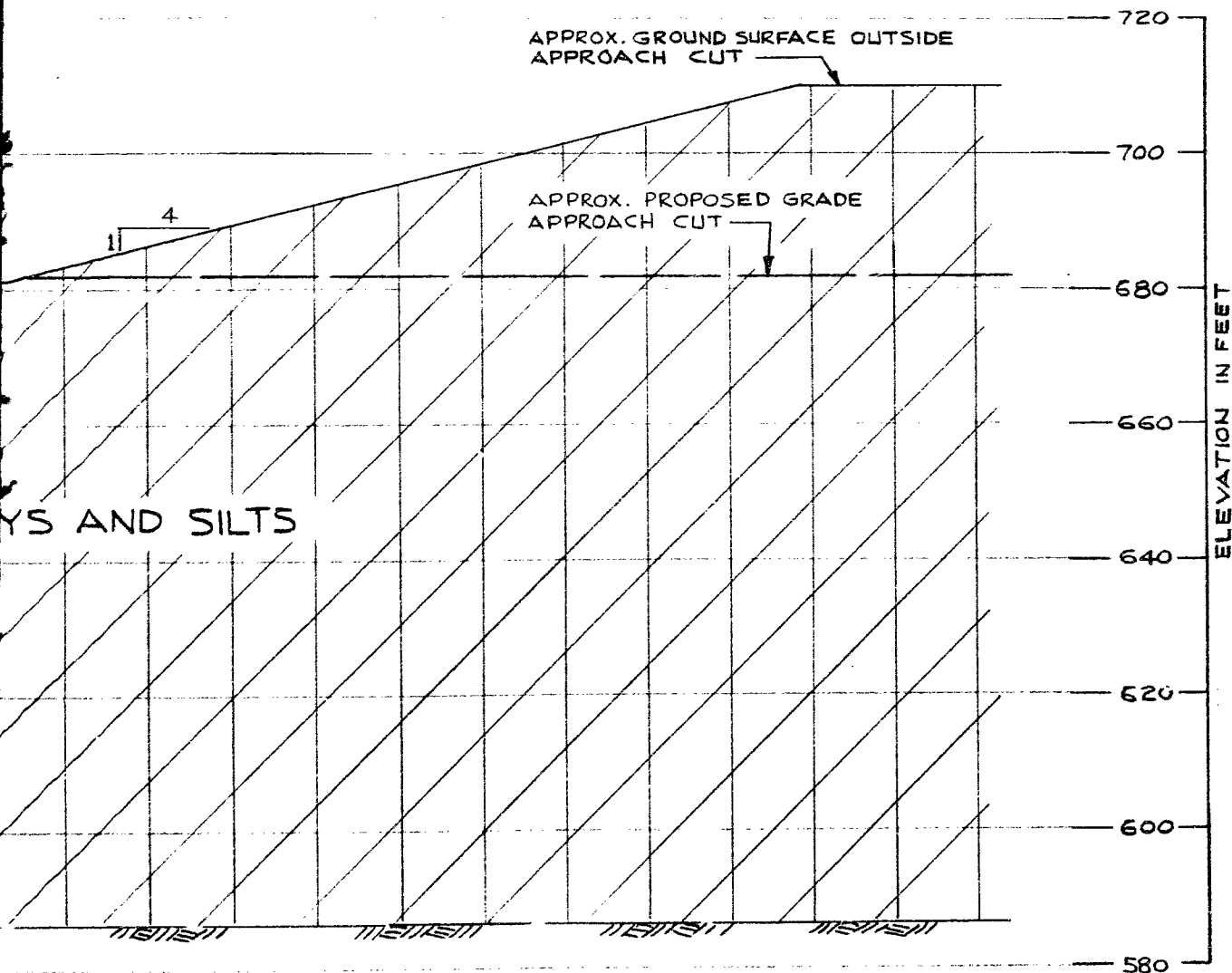


SCALE: 1" TO 20'

ILLUSTRATION OF PROPOSED SLOPE PROTECTION

FIGURE 31

40P1-69
GEOCRE No.



Date MAY 8, 1975

Golder Associates

Drawn *M. J. B.*
Chkd. *H. G.*
Appd. *J.*