

GEOCRES No. 31G5-134DIST. 9 REGION _____

W.P. No. _____

CONT. No. _____

W. O. No. 2000-11024

STR. SITE No. _____

HWY. No. 17LOCATION Hwy 17 & ORLEANS BLVD.NO OF PAGES -OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS: _____

**McCORMICK, RANKIN & ASSOCIATES
LIMITED**

consulting engineers

Copy to WP 46-78-07 file

July 3, 1980

Ministry of Transportation
And Communications
P.O. Box 4,000,
Kingston, Ontario

Attention: Mr. S. Radbone, P. Eng.,
Manager of Planning
and Design.

ENGINEERING & ROW OFFICE		
	SECTION	Initial
	PLANNING & DESIGN	
	PROPERTY	
	SURVEYS & PLANS	
	GEOTECHNICAL	
	SCHEDULE COORD	
✓	STRUCTURAL	✓
	FILE	

RE: Township of Gloucester
Orleans Boulevard
Grade Separation.
Our File: W.O. 1049-79.

Dear Sir;

As per your request, we are forwarding
one copy of the Subsurface Investigation Report
by Golder Associates.

Yours very truly,

McCORMICK, RANKIN & ASSOCIATES LIMITED



P. Turner, P. Eng.,
Area Manager.

PT/ke

Encl.

c.c. Mr. J.F. Bockstael, P. Eng.

Copy of letter & report to M. Devine.

GEOCRES No 3145-134



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

MCCORMICK, RANKIN & ASSOCIATES

SUBSURFACE INVESTIGATION

PROPOSED GRADE SEPARATION

ORLEANS BLVD. AT HIGHWAY 17

TOWNSHIP OF
GLOUCESTER

ONTARIO

Distribution:

- 6 copies - McCormick, Rankin & Associates
Ottawa, Ontario
- 2 copies - Golder Associates
Ottawa, Ontario

August, 1979

791-2083

TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	
INTRODUCTION	2
DESCRIPTION OF PROJECT	2
PROCEDURE	3
SUBSURFACE CONDITIONS	4
Surficial Materials	4
Silty Clay	5
Glacial Till	7
Bedrock	7
Groundwater Conditions	8
PROPOSED GRADE SEPARATION	8
Approach Embankments	8
Foundations	11
Abutments	12
Existing Site Services	13
Pavement	14
Construction Considerations	15
ABBREVIATIONS & SYMBOLS	
RECORDS OF BOREHOLES AND TEST PIT	
FIGURES	
1 - Key Plan	
2 - Boring Plan	
3 - Soil Profile	
4 - Summary of Geotechnical Properties	
5 to 7 - Consolidation Test Results	
8 - Grain Size Distribution Curves	
9 - Stability of Approach Embankments	
10 - Calculated Vertical Settlement Profiles	

ABSTRACT

This report gives the results of a subsurface investigation carried out at the site of the proposed grade separation of Orleans Boulevard at Highway 17 in Gloucester Township, Ontario. Based on our interpretation of the subsurface conditions encountered in the borings put down at this site, recommendations are given for the geotechnical aspects of the grade separation design.

The site is underlain by a deep deposit of fresh water and marine clays extending to some 30 to 32.5 metres below ground surface. The consistency of this clay is stiff to very stiff. The clay is underlain by some 2.5 metres of very loose to very dense glacial till followed by grey limestone bedrock. The bedrock was found to be of variable quality with a surface which slopes off somewhat from north to south.

Stability calculations indicate that the 7.6 metre high approaches could be constructed of a lighter weight (20 kilonewtons per cubic metre) sandy earth fill with side and front slopes of 2 horizontal to 1 vertical. The fill is expected to produce some 185 millimetres of settlement within the underlying clay. Construction of the approach fills some 1 to 2 years in advance of bridge construction is recommended.

It is recommended that the bridge be supported on high capacity, end bearing, concrete filled, steel pipe piles driven to the bedrock surface. Negative skin friction will act on the abutment piles as a result of the long term settlement of the approach fills. An example for 400 millimetre diameter steel pipe piles is given.

Because the existing site services will be located beneath the proposed approach fills, they will undergo significant total and differential settlements. Because of varying fill heights and service locations relative to the embankments, differential settlements are expected to produce irregularities in the grades of these lines. The effects of the total and differential settlements on structural and performance tolerances for these lines is to be investigated.

August, 1979
791-2083

INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by McCormick, Rankin and Associates to carry out a subsurface investigation at the site of a proposed grade separation in Gloucester Township, Ontario. The purpose of this investigation was to determine the soil, bedrock and ground-water conditions at the site and, based on an interpretation of the factual information obtained, to provide engineering recommendations on the geotechnical aspects of the project including:

- foundation design for the proposed structure
- stability of the approach fills
- settlement of the approach fills
- affects of the proposed construction on the existing service lines
- pavement design for the proposed roadway
- construction considerations which could influence design decisions

DESCRIPTION OF PROJECT

Plans are presently being prepared for the grade separation of Orleans Boulevard at Highway 17 in Gloucester Township, Ontario (see Key Plan, Figure 1). The proposed overpass could be of either reinforced concrete, precast concrete or steel construction with two equal spans of 30.5 metre length and a finished deck elevation of 61.0. The earth fill approaches to the bridge are to have a 5.0 per cent grade and reach a height of some 7.6 metres above the existing ground surface. The Orleans Boulevard right of way presently contains several service lines consisting of storm and sanitary sewers, watermain and gas main. These site service lines are located beneath or adjacent to the proposed approach fill areas.

From past experience in the general area it is known that the site is underlain by an extensive and thick deposit of sensitive silty clay of marine and fresh water origin followed by a mantle of glacial till and then limestone bedrock of the Ottawa formation at depths of 25 to 35 metres.

PROCEDURE

The field work for this investigation was carried out between May 5 and 17, 1979 at which time a total of 5 boreholes were put down at the site using a track mounted power auger drill machine supplied and operated by the F.E. Johnston Drilling Co. of Ottawa. Boreholes 1 to 3, put down at the proposed abutment and pier locations, were taken to total depths of about 36 to 38 metres with the bedrock proven in each of these holes by coring in BX size. Boreholes 4 and 5 were put down to depths of about 18 metres in the area of the approach fills some 40 to 50 metres back from the abutments. Standard penetration tests were carried out in the overburden soils and samples of the soils were obtained using both standard drive open and thin walled Shelby tube sampling equipment, the latter being used to obtain relatively undisturbed samples of the silty clay soil for consolidation testing. In situ vane tests were carried out to determine the shear strength profile of the clay deposit. Standpipes and piezometers were sealed into the boreholes at various levels and water levels were obtained from these devices to determine the groundwater conditions at the site at the time of the investigation. On May 17, 1979 a test pit was dug to about 4.5 metre depth using a backhoe supplied and operated by Murray R. Gray Ltd. Large block samples of clay were obtained from the test pit for more detailed laboratory testing of the clay soil. All field work was supervised by a member of our engineering staff.

A detailed log of the subsurface conditions encountered in each borehole is given on the Record of Borehole and Record of Test Pit sheets following the text of this report. The borehole and test pit locations are shown on Figure 2 and a soils profile illustrating these conditions along the proposed grade separation are shown on Figure 3.

The soil and rock core samples obtained from the borings in this investigation were brought to our laboratory for detailed examination and testing. Also, test samples were cut from the large block samples for more detailed strength testing of the silty clay. The results of the testing is given on the Record of Borehole and Test Pit sheets and on Figures 4 to 8.

The elevations given in this report are referred to a bench mark set on the turn arrow on the hydrant left of about Station 0+870. The elevation of this point was supplied to us by McCormick Rankin and Associates as 54.666, Geodetic datum.

SUBSURFACE CONDITIONS

The detailed subsurface conditions in each boring are given on the Record of Borehole sheets and are illustrated on the soil profile on Figure 3. Following is a summarized account of the conditions along the proposed grade separation alignment.

Surficial Materials

The borings generally encountered some 270 to 670 millimetres of topsoil and/or brown clayey silt material while borehole 4 encountered some 2.4 metres of firm grey-brown silty clay backfill material associated with the installation of the adjacent storm and sanitary sewers.

Silty Clay

These surficial materials were underlain by an extensive deposit of sensitive silty clay which extended to depths of about 30 to 32.5 metres at the proposed structure location. Below about elevation 40 the presence of black organic mottling within the clay deposit indicates that this portion of the deposit is of marine origin (Champlain Sea clay) while above this level the clay appears to have been reworked and redeposited in a fresh water environment. The upper 0.85 to 1.45 metres of the deposit has been weathered to form a very stiff to stiff crust of fissured grey-brown silty clay. Standard penetration (N) values of 3 to 9 blows per 0.3 metres were obtained within the crust material. Atterberg limit testing on samples of the weathered crust indicates that it is highly plastic with liquid limits of 63 and 83 and corresponding plasticity indices of 48 and 32. The moisture content of the weathered clay ranged from about 25 to 35 per cent.

Beneath this relatively thin weathered zone the colour of the clay changes to grey and the consistency decreases to stiff. The shear strength of the grey clay obtained by field vanes and laboratory tests are plotted on the Record of Borehole sheets and on Figure 4, the Summary of Geotechnical Properties. In situ vane shear strength values in the grey clay increased slightly with depth from about 60 to 75 kilopascals immediately beneath the weathered zone to about 115 to 132 kilopascals near the base of the clay deposit, indicating a stiff to very stiff consistency.

From previous experience with fresh water sensitive clays of the Ottawa area it is known that clay is anisotropic with respect to shear strength in the horizontal and vertical directions; that is, the horizontal shear strength is only a percentage of the vertical strength. A series of unconsolidated, undrained triaxial tests were carried out on 5 horizontally

oriented and 5 vertically oriented clay samples obtained from the block samples and the samples tested at a confining pressure of 35 kilopascals, approximating the in situ stress conditions. The axial strains to failure were generally between about 1 and 2 per cent and as such it is believed that the clay blocks experienced only minor sample disturbance. The average horizontal and vertical undrained shear strength values obtained from the tests were 75 and 95 kilopascals respectively corresponding to a horizontal to vertical strength ratio of 0.8. As shown on Figure 4, a relatively wide range of shear strength results was obtained from the triaxial testing on both the vertically and horizontally oriented samples. As well, the average vertical shear strength obtained from the triaxial testing was only slightly greater than that obtained from the in situ vane testing. The relatively low triaxial strengths and large range of triaxial results are thought to be largely due to the fissured and blocky structure of the clay.

Atterberg limit tests indicate that the plasticity of the grey silty clay decreases from high to medium with depth with liquid limits decreasing from 83 to 42 and corresponding plasticity indices from 57 to 13. The natural water content of the grey silty clay also decreases with depth typically varying between about 55 and 70 per cent above about 18 metre depth; that is generally less than the liquid limit value. Below this depth the water content decreases to as low as about 40 per cent. The unit weight of the grey clay is about 16 kilonewtons per cubic metre.

Consolidation tests were carried out on two 75 millimetre diameter Shelby Tube samples, one each from boreholes 4 and 5, and on a clay sample taken from the large block sample. The consolidation test results are shown on Figures 5 to 7. The results indicate that the apparent past preconsolidation pressure is some 200 to 240 kilopascals in excess of the

existing overburden pressure. The compression indices, C_c , of the samples ranged from about 1.0 to 1.7 and the recompression indices, C_{cr} , as obtained from loading rebound cycles, ranged from 0.017 to 0.028.

Glacial Till

The silty clay was underlain by a deposit of glacial till which ranged in thickness from about 2.4 to 2.8 metres at the deep borehole locations. The results of a grading test carried out on a till sample are shown on Figure 8 and indicate the well graded nature of this deposit. The till consists of grey sandy silt with some gravel, trace of clay and the occasional cobble and boulder. Standard penetration testing measured N values ranging from about 2 to 54 blows per 0.3 metres indicating a very loose to very dense state of packing. The natural water content of the till was about 10 per cent.

Bedrock

Bedrock was encountered at depths ranging from about 33 metres (borehole 1) to about 35 metres (borehole 2). This corresponds to about elevation 21 at the north abutment, elevation 19.5 at the central pier and elevation 18.0 at the south abutment; that is, the bedrock surface slopes off from north to south. The bedrock consists of grey limestone with occasional thin, cemented joints and calcite filled joints and seams. In borehole 3 core recoveries of 85 to 90 per cent and R.Q.D.'s (Rock Quality Designations) of 50 and 67 per cent indicate that the upper portion of the rock has a fractured to fairly sound quality. At boreholes 1 and 2 the portion of the rock investigated is indicated to be fairly sound to sound with 100 per cent core recoveries and R.Q.D.'s of 65 to 95 per cent.

Groundwater Conditions

Standpipes and piezometers were installed at several levels within the silty clay deposit and the underlying bedrock. The details of the standpipe and piezometer installations, together with the water levels measured on July 16, 1979 are given on the Record of Borehole sheets. The water levels measured in the piezometers installed in the clay above about 17 metre depth were generally about 0.3 to 1.3 metres below the ground surface (elevation 53). In borehole 4 the water levels in the upper portion of the clay were at about 3 to 4 metre depth most probably due to a localized drawdown condition adjacent to the existing sewers. Artesian water levels to some 1 to 1.7 metres above the ground surface were encountered during drilling within the bedrock. The stabilized artesian level measured in a standpipe installed within the bedrock at borehole 1 was some 0.6 metres above ground surface.

PROPOSED GRADE SEPARATION

Approach Embankments

i) Stability

As presently proposed the overpass approaches are to be constructed to about elevation 61, some 7.6 metres above the existing ground surface. For stability considerations and also to minimize as much as possible the stresses within the underlying clay it is recommended that the approaches be constructed of a free draining, non-frost susceptible sandy earth fill having compacted in place, total unit weight not greater than 20 kilonewtons per cubic metre. Also, to facilitate compaction operations, this sandy fill should be well graded. The sandy fill should be placed in shallow lifts and each lift well compacted to at least 95 per cent of

the maximum standard Proctor dry density. For stability calculations the sand fill was assigned an angle of internal friction, ϕ , of 32 degrees.

Based on construction of an embankment as described above, stability calculations were carried out for the approach fills for both the short term (total stress) and long term (effective stress) conditions. For the total stress analysis the grey silty clay was assigned values of undrained horizontal shear strength using a horizontal to vertical shear strength ratio of 0.8 as determined from the laboratory testing and vertical shear strength values as measured by in situ vane testing. For the effective stress analysis an effective angle of internal friction, ϕ , of 28 degrees and an effective cohesion intercept of 12 kilopascals was used. The grey silty clay was assigned a unit weight value, γ_d , of 16.5 kilonewtons per cubic metre.

Stability calculations indicate that the factor of safety against instability of a 7.6 metre high embankment with 2 horizontal to 1 vertical side and front slopes would be about 1.9 in the short term. Assuming that the weight of the embankment fill could induce a pore pressure ratio, r_u , of 0.6 in the grey clay, the factor of safety by the effective stress analysis was calculated to be about 1.4. The embankment stability is illustrated in Figure 9. These factors of safety against instability are considered adequate.

ii) Settlement

The results of consolidation tests carried out on samples of the grey clay indicate that this clay has been preconsolidated by some 200 to 240 kilopascals in excess of the existing overburden pressure. Based on the sandy embankment fill material mentioned previously, the 7.6 metre high approach fill will impose a loading of 155 kilopascals onto the underlying

compressible clay deposit which is some 30 metres thick. Settlement calculations were carried out for both a 7.6 metre high transverse section and a longitudinal section along the embankment; profiles of the expected settlements are shown on Figure 10. It is calculated that a maximum settlement of about 185 millimetres will take place beneath the centre of the embankment back to a point some 65 metres behind the structure abutments. It is estimated that about 40 per cent of this total settlement will occur within the first year after construction and about 60 per cent of the total settlement after the second year. The remaining settlement would then take place at a decreasing rate over a long period of time. As such, some maintenance will be required in adjusting the approach grade adjacent to a pile supported abutment. It is recommended therefore that the approach fills be placed some 1 to 2 years in advance of the overpass construction to permit some settlement and dissipation of excess porewater pressures in the clay subsoil to take place. Some localized differential settlements due to compression of the loose clay backfill in the existing sewer trenches beneath and adjacent to the embankment areas may also be expected.

Associated with the vertical settlements described above, the embankment loading will also cause lateral displacement of the clay soil to occur. The magnitude of this lateral movement at a particular location is a function of the applied embankment loading, depth below ground surface and the thickness of the clay deposit. Measurements on instrumented fills have shown that the long term lateral displacements of the clay soil at the toe of an embankment slope is about 20 per cent of the vertical settlement below the centreline of the embankment. The magnitude of these lateral displacements may also be calculated from the settlement profiles on Figure 10.

Comparison was made with published measured settlements of the approach embankment at the Montreal Road - Highway 417 interchange¹. This involved the construction of a slightly lower embankment than the proposed fill at the Orleans Boulevard - Highway 17 site but built on a clay deposit of similar thickness and having comparable shear strength and consolidation characteristics. From private conversation it was learned that in 1974, some 17 years after the start of construction, the instrumented fill had settled some 135 millimetres. This measured settlement is in very close agreement with the settlement calculated for the Montreal Road structure. As such, it is considered that the estimated settlements at the Orleans Boulevard grade separation site should be realistic with respect to the actual performance of the embankment.

Foundations

As shown on Figure 4, the stress level induced in the upper portion of the grey clay due to the 7.6 metre high fill is only some 20 kilopascals less than the apparent past pre-consolidation pressure. In order to avoid overstressing the clay and thereby creating disproportionately larger settlements of the embankments, the allowable bearing capacity for a spread footing foundation placed within the granular fill would be very limited. In addition, placement of footings within the embankment would necessitate the use of a heavier, granular type of fill which would possibly necessitate flattening of the slopes to provide adequate stability of the embankment and contribute to additional settlement. To avoid eccentric loading of large footing pads it would also be necessary to lengthen the overpass structure.

¹Burn & Hamilton, "Settlement of an Embankment on Leda Clay", Canadian Geotechnical Journal, Volume V, No. 1, February 1968, pp. 16 to 27.

It is therefore recommended that the foundation loads from the abutments and centre pier be transferred to the underlying bedrock by means of end bearing piles. Bedrock was proven to exist at about elevations 20.8, 19.6 and 18.0 at the north abutment, central pier and south abutment locations respectively; that is, some 33 to 35 metres below the existing ground surface. Because the approach fills will be placed above a compressible clay deposit, settlement of the clay due to this increased loading will induce negative skin friction on the overpass abutment piles. In order to best accommodate the additional loading imposed by negative skin friction and maximize pile capacity with a minimum number of piles, consideration should be given to the use of high load capacity concrete filled steel pipe piles. As an example for design purposes the allowable load on a 400 millimetre diameter steel pipe pile with a 10 millimetre wall thickness driven to a final set of 24 blows for the last 25 millimetres with a hammer developing 49 kilojoules of energy per blow may be taken as 1.35 meganewtons per pile. An allowance of 0.58 meganewtons per pile should be made to accommodate negative skin friction for these abutment piles. For the central pier piles, where the grade is to remain at the existing level, it will not be necessary to allow for negative skin friction.

It has been found that the negative skin friction load can be reduced considerably by coating the piles with bitumen. Measurements on instrumented piles coated with 1 to 1.5 millimetres of 80/100 penetration bitumen have indicated that about 50 per cent of the negative skin friction was eliminated. In one specific case, this treatment reduced the negative skin friction by 90 per cent. If consideration is given to this treatment, the bitumen coating should be applied only to the depth over which negative skin friction is expected to develop, calculated to be to a depth of about

20 metres below the ground surface at this site.

There is some experience in the Eastern Ontario area with abutments tilting backwards in marine clay areas. This tilting of abutments is considered to be caused by differential settlement in the area of the abutments where relatively large settlements take place, creating lateral thrust loads on the piles. It is considered possible to avoid tilting of abutments by providing piling from the abutment which has a batter in both directions.

Because the abutment piles will be end bearing, the settlement of the abutments will be equal to the elastic compression of the piles.

Abutments

The closed end abutments should be backfilled with well compacted clean sandy fill material in accordance with Ministry of Transportation and Communications (M.T.C.) Standard DD-809-C, Revision 4. Provision should be made for drainage of this backfill to prevent hydrostatic or ice pressure buildup behind the walls. With full effective drainage of the backfill the abutment walls should be designed to resist lateral earth pressures using a coefficient of lateral earth pressure at rest, K_0 , of 0.5 and a soil unit weight of 20 kilonewtons per cubic metre. Provided some movement at the top of the wall may be tolerated, the lateral earth pressure may be calculated using an active earth pressure coefficient, K_a , of 0.35.

Existing Site Services

The location of the existing site services in relation to proposed approach embankments are shown on Figure 2. These include 900 to 1,800 millimetre diameter storm and sanitary sewers, 400 millimetre diameter watermain and a

200 millimetre diameter steel gas main. The gravity flow sewers, which drain towards the north are at about 5 to 7 metre depth below existing ground surface. The water and gas mains are buried at depths of up to about 2.5 metres. Because these services are located beneath or adjacent to the edges of the proposed embankments, they will undergo some settlement corresponding to the consolidation of the clay deposit due to the embankment fill. The total settlement of the services will vary with their position relative to the embankment and the embankment height at that location. The vertical settlement may be estimated from the embankment settlement profiles shown on Figure 10. In order to account for consolidation of the clay above the pipes which will not contribute to pipe settlements, pipe settlements for the deeper services should be about 20 per cent less than the calculated embankment settlements. South of the existing Highway 17, vertical storm sewer settlements are expected to be about 40 millimetres at MH46 (Station 1+195), 75 millimetres at MH45 (Station 1+130) and increasing to a maximum of about 130 millimetres for the storm and sanitary sewers between about Stations 1+110 and 1+030. Between the approach fills the settlements should decrease to less than about 25 millimetres. As a result, an irregularity in the form of a high point below the bridge pier will be produced in the invert of the sewer lines due to this differential movement. The gas main in this area is expected to experience some 40 millimetres of vertical settlement. North of the existing highway, where the service lines are generally located along the outside portions of the embankment fill, vertical settlements should be of the order of 50 to 75 millimetres with a small pipe depression occurring where the sewers cross beneath the northern end of the embankment fill.

As pointed out previously, the silty clay and thus the service lines which are under the influence of the embankment will also experience lateral displacement. The order of magnitude of this movement may be calculated from the data provided

under Embankment Settlement and from the settlement profiles in Figure 10.

The tolerance and performance of all the service lines under the expected total and differential settlements should be assessed by the appropriate consultant prior to any embankment construction. If it is found that any or all of the existing service lines cannot accept the expected movements, it will be necessary to relocate these lines outside the zone of influence of the approach fills.

Pavement

For pavement design purposes consideration could be given to a pavement structure having a granular base equivalency of about 760 millimetres. The sandy earth embankment fill material properly compacted in place will provide an acceptable pavement subgrade. The pavement structure could possibly consist of:

- i) Conventional Design
 - 125 millimetres hot mix asphalt over
 - 230 millimetres M.T.C. Granular 'A' over
 - 380 millimetres M.T.C. Granular 'C'
- ii) Deep Strength Design
 - 230 millimetres hot mix asphalt over
 - 300 millimetres M.T.C. Granular 'A'

From our experience in the Ottawa area the Granular 'C' fill materials which are readily available often consist of uniformly (poorly) graded sands which are difficult to adequately compact in place and, if used in the above thicknesses, would produce a somewhat lower granular base equivalency than recommended. Should it prove difficult to secure a source of well graded sandy Granular 'C' fill material, provision could be made for substitution with M.T.C. Granular 'B' material having a maximum particle size of about 100 millimetres (e.g. 100 millimetre minus crushed stone fill). All pavement fills should be compacted to 100 per cent of the maximum standard Proctor dry density.

Construction Considerations

It is recommended that samples of the sandy earth fill to be used in the approach embankment construction be submitted for approval prior to any placement to ensure that it is an acceptable material for its intended use. Control on the placement and compaction of all fill materials on this project should be carried out to ensure that specifications are met from both a grading and compaction point of view. Where the approach fill thickness will be 1.2 metres or less, all topsoil and unsuitable surficial fill material should be removed from the embankment area prior to placing fill in order to avoid a weakened pavement structure.

It is recommended that the piling design and refusal criteria be checked by a geotechnical engineer prior to construction. Also, all piling operations should be inspected throughout by qualified geotechnical personnel to ensure that the piling requirements for final set, alignment, etc. are met and to ensure that the piles are not damaged during installation.

We trust that this report contains the information necessary for your design purposes. Should there be any questions concerning this report, or if we may be of further service to you on this project, please call us.



PAS:GSW:sl:rb
791-2083

Yours very truly,

GOLDER ASSOCIATES

A handwritten signature in dark ink, appearing to read "P. A. Smolkin".

P.A. Smolkin, P.Eng.

A handwritten signature in dark ink, appearing to read "G. S. Webb".

G.S. Webb, P.Eng.

LIST OF SYMBOLS

I. GENERAL

τ	= 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_c	coefficient of consolidation
T_v	time factor = $c_c t / d^2$ (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_r	sensitivity

in terms of effective stress

$$\tau_f = c' + \sigma' \tan \phi'$$

in terms of total stress

$$\tau_f = c_u + \sigma \tan \phi_u$$

*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.

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 63.5 kilogram hammer dropped 0.76 metres required to drive a 51 millimetre diameter, 60 degree cone 0.3 metres, 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 63.5 kilogram hammer dropped 0.76 metres required to drive a 51 millimetres drive open sampler 0.3 metres.

WH sampler advanced by static weight - weight, hammer
PH sampler advanced by pressure - pressure, hydraulic
PM sampler advanced by pressure - pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Relative Density	N, blows/0.3 metres
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	C _u , kilopascals
Very soft	Less than 12
Soft	12 to 24
Firm	24 to 48
Stiff	48 to 96
Very stiff	96 to 192
Hard	over 192

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 Q or R.

RECORD OF TEST PIT 1

LOCATION See Figure 2 EXCAVATION DATE MAY 17, 1979 DATUM GEODETIC

TEST PIT TYPE BACKHOE

TEST PIT SIZE 1.50 m x 3.00 m

SOIL PROFILE				ELEVATION SCALE	SHEAR STRENGTH C _u , K Pa				ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS				
ELEV _N DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER		SAMPLE TYPE	70	80	90			100			
<div><div><div>53.96</div><div>0.00</div><div>0.15</div><div>0.34</div><div>0.46</div><div>52.58</div><div>1.37</div><div>49.38</div><div>4.57</div></div><div>BROWN SILTY CLAY, SOME SAND (FILL)</div><div>GROUND SURFACE</div><div>TOPSOIL</div><div>TOPSOIL</div><div>TOPSOIL</div><div>GREY BROWN SILTY CLAY (WEATHERED CRUST)</div><div>GREY SILTY CLAY</div><div>END OF HOLE</div></div>											<div>UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS</div> <div>● HORIZONTALLY ORIENTED SAMPLES ⊗ VERTICALLY ORIENTED SAMPLES</div>			
				54										
				53										
				52										
				51										
				50	●	●	●	●	●	⊗				
				49										

VERTICAL SCALE
1:50

Golder Associates

DRAWN - DN
CHECKED JPS

KEY PLAN

FIGURE 1



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

SCALE 3 MILES TO 1 INCH

Date **AUG. 14, 1979**
Project **79L-2083**

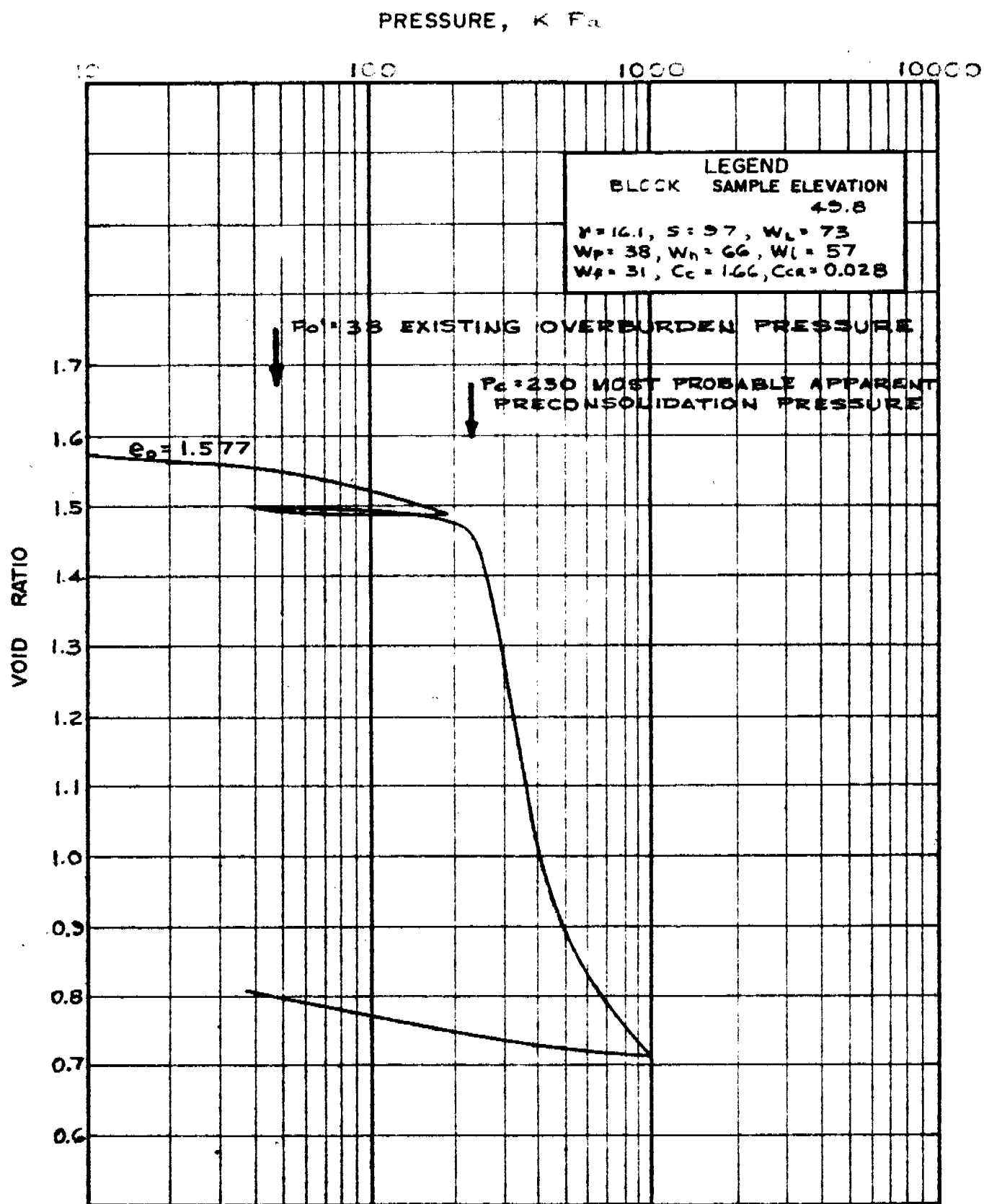
Golder Associates

Drawn **DN**
Chkd **HS**

OVERSIZE DRAWING(S)

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

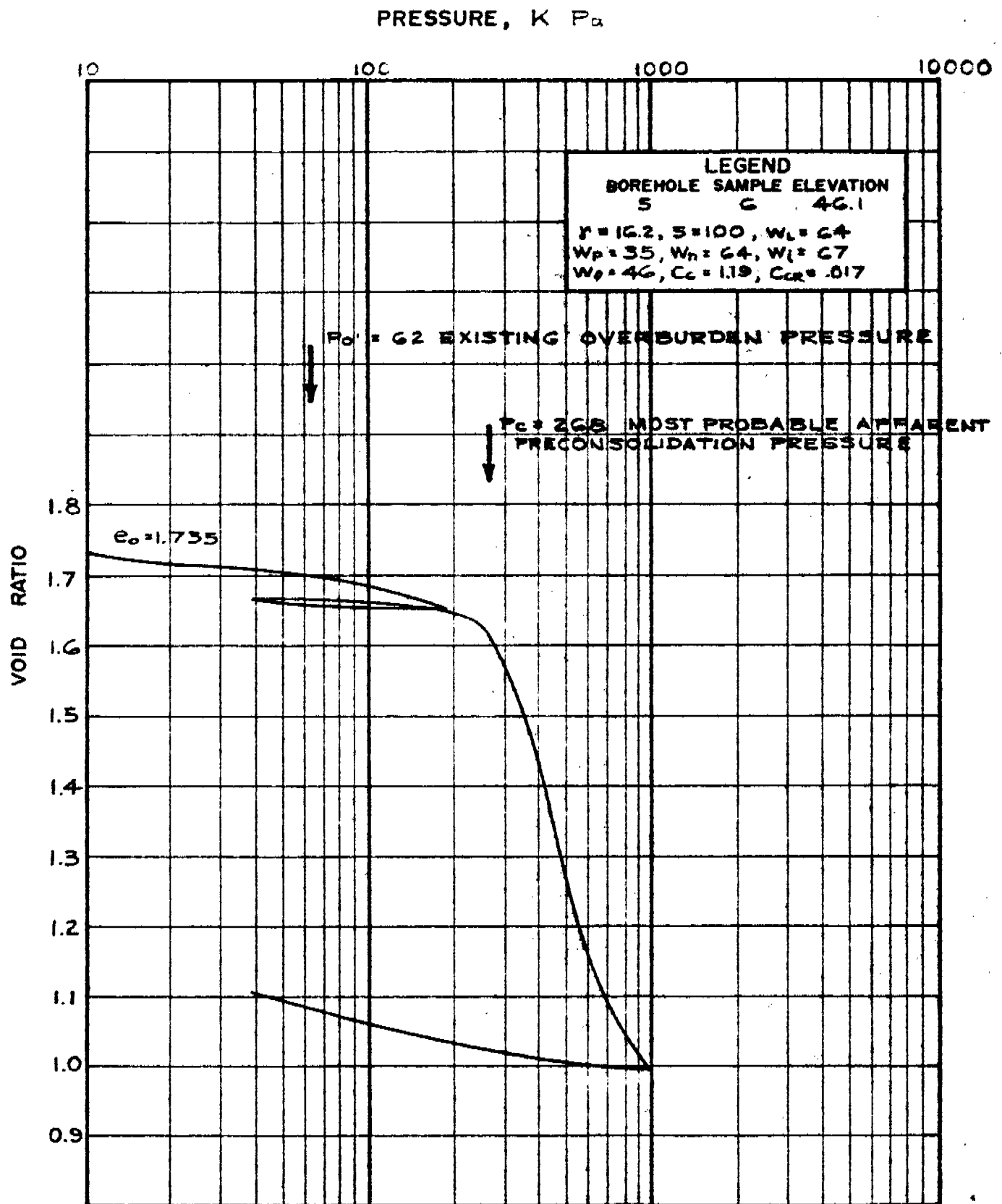
FIGURE 5



Golder Associates

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

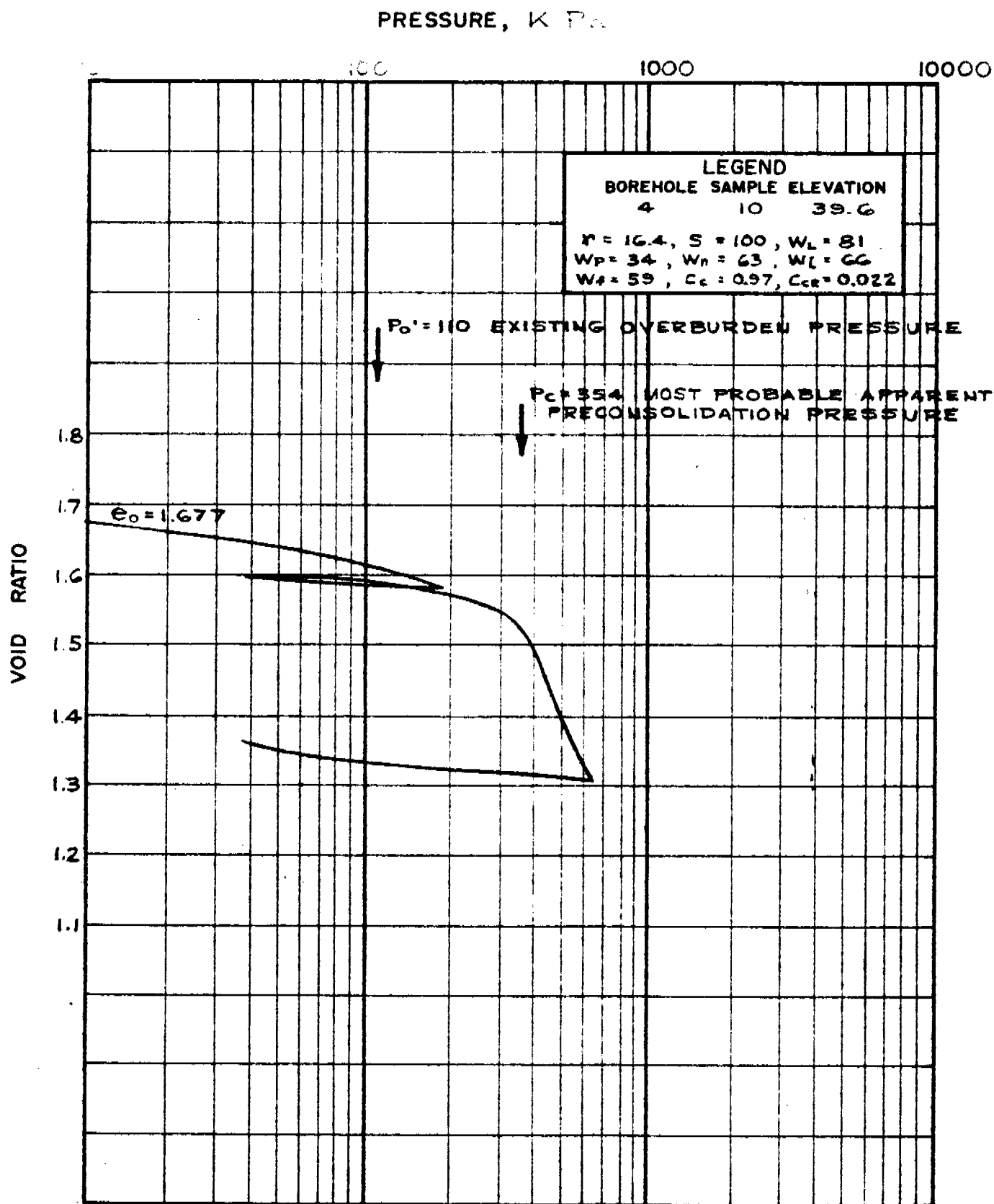
FIGURE 6



Golder Associates

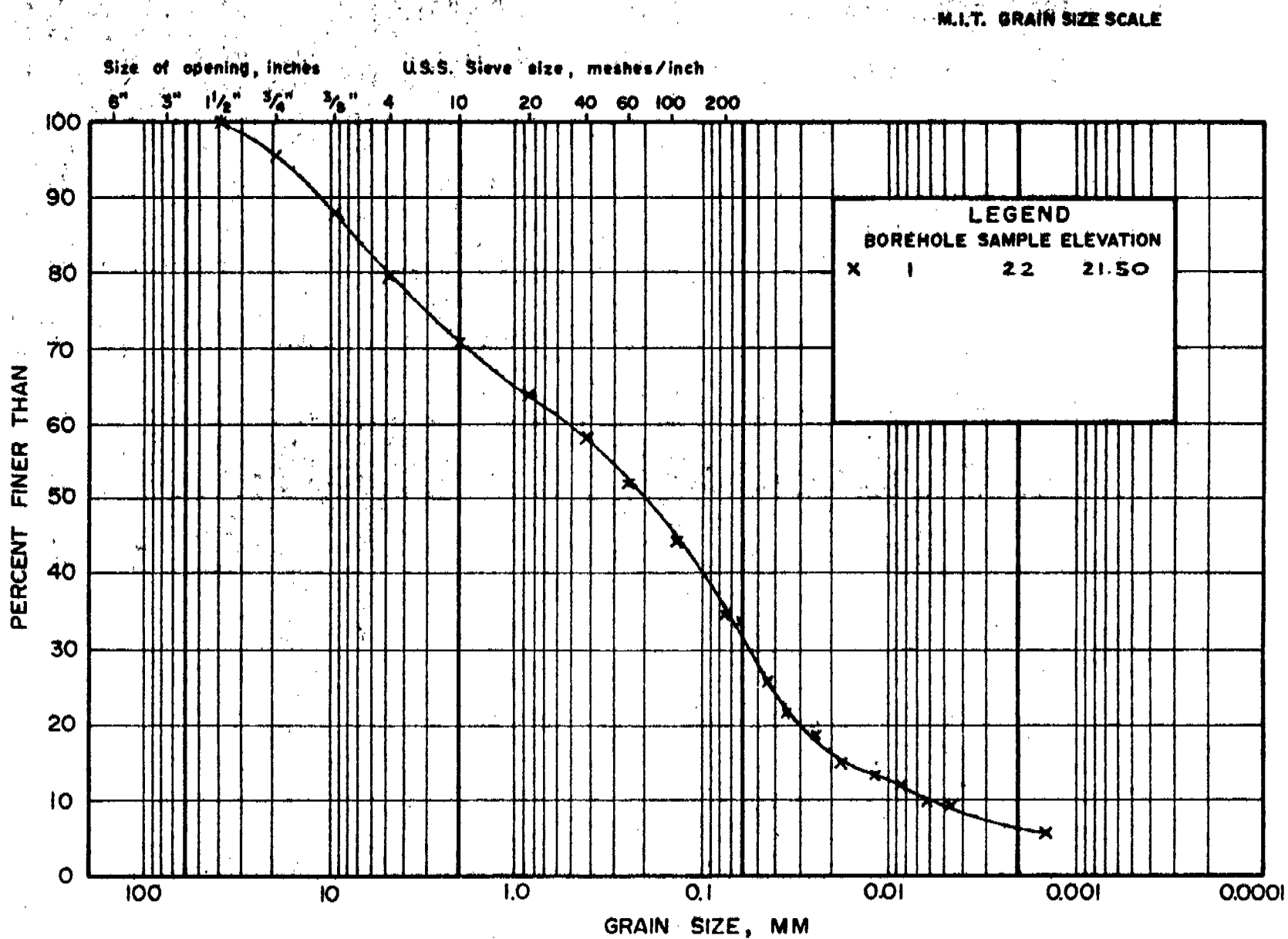
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 7



Golder Associates

Golder Associates



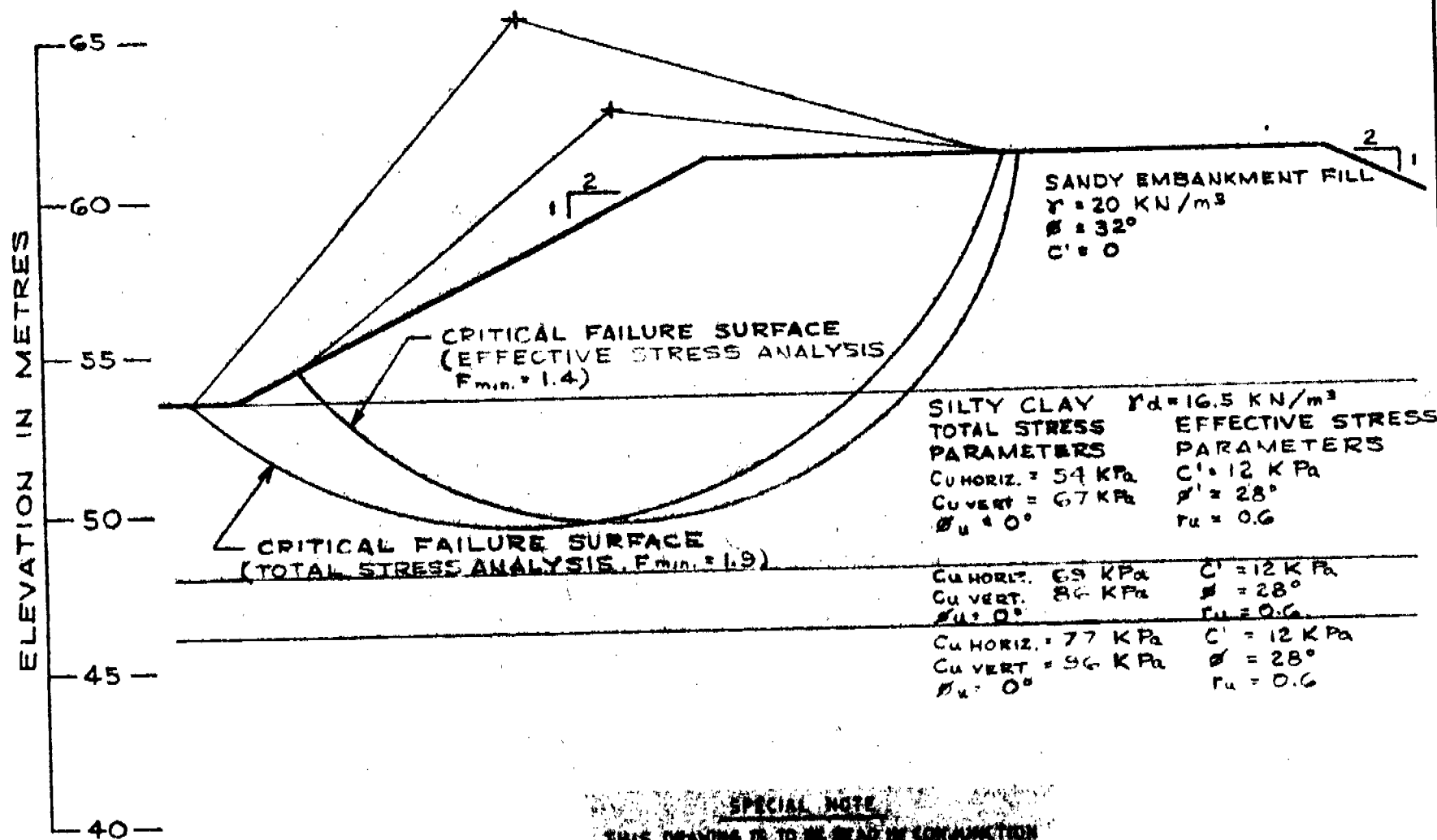
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

**GRAIN SIZE DISTRIBUTION
GLACIAL TILL**

FIGURE 8

STABILITY OF APPROACH EMBANKMENT

FIGURE 9



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

SCALE 1:200

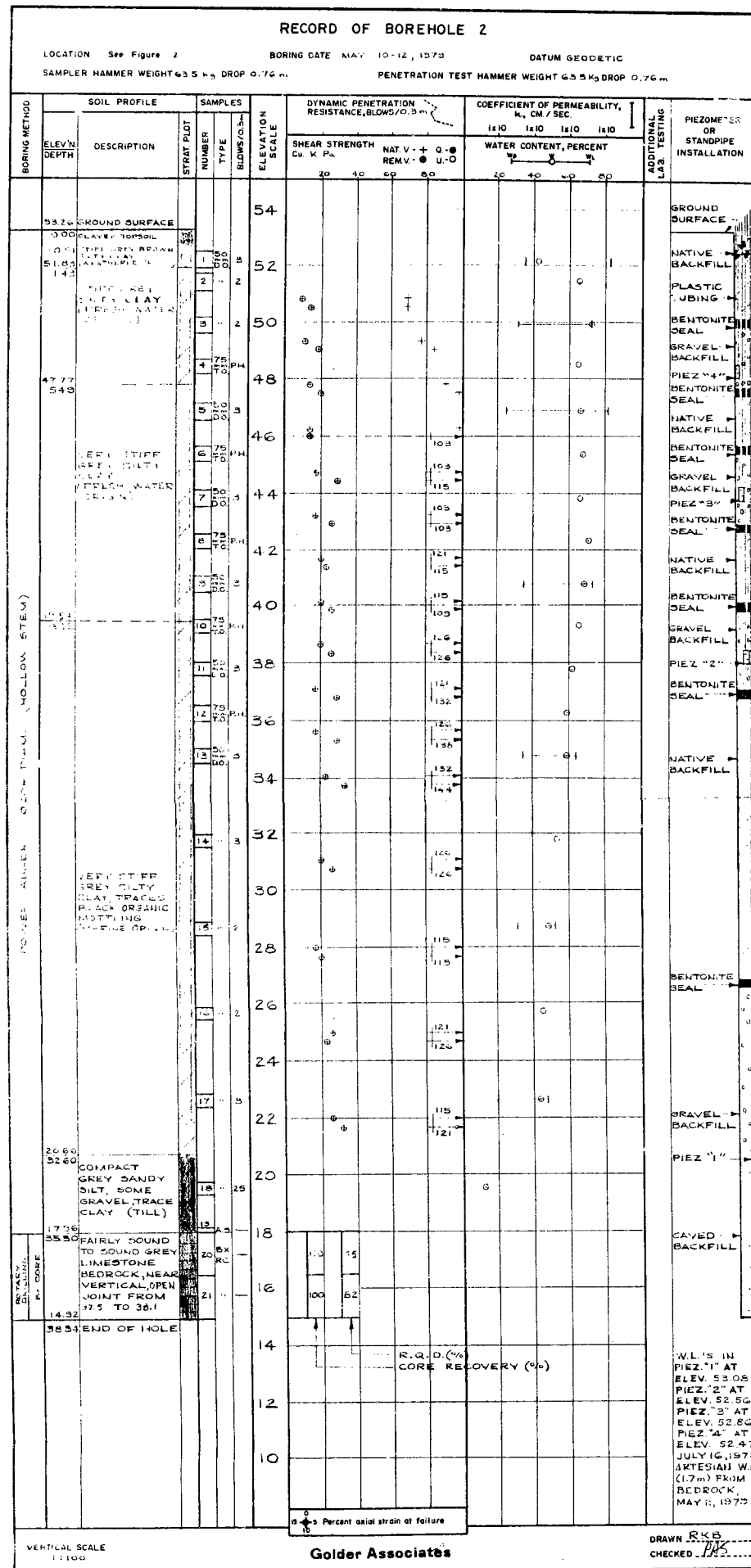
Date AUG. 13, 1979
 Project 791-2083

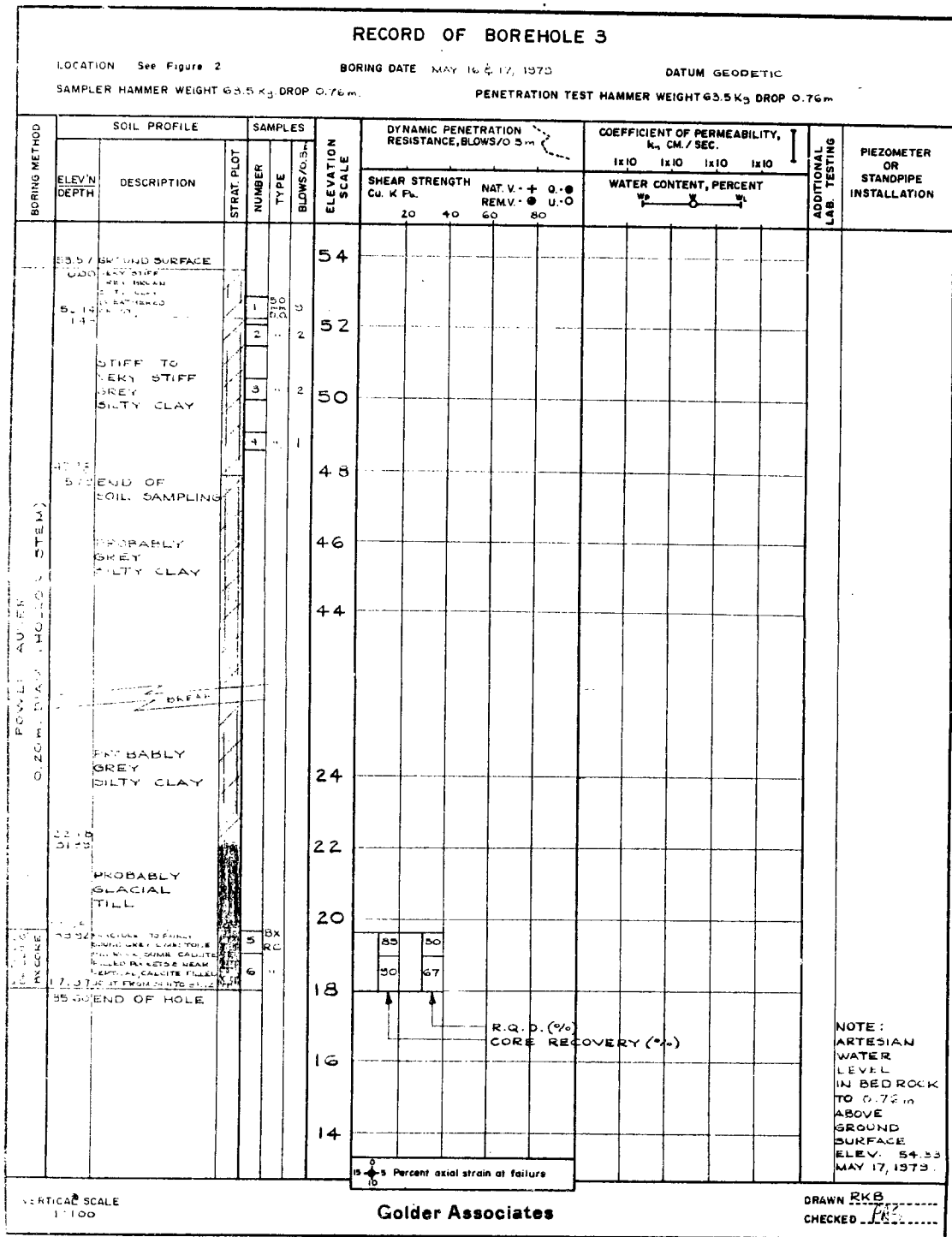
Golder Associates

Drawn
 Chd.

OVERSIZE DRAWING(S)

RECORD OF BOREHOLE 1														
LOCATION See Figure 2			BORING DATE MAY 7-3, 1979			DATUM GEODETTIC								
SAMPLER HAMMER WEIGHT 63.5 K _g DROP 0.76m			PENETRATION TEST HAMMER WEIGHT 63.5 K _g DROP 0.76m											
BORING METHOD	SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				COEFFICIENT OF PERMEABILITY, K _v , CM./SEC.				PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV./N DEPTH	DESCRIPTION	STRAT. PLAT. NUMBER	TYPE		SHEAR STRENGTH Cu, K Pa.	NAT. V. - + 0 - 8 REM. V. - 0 - 10	1x10	1x10	1x10	1x10	WATER CONTENT, PERCENT		
POWER AUGER (HOLLOW STEM)	53.73	GROUND SURFACE			54									GROUND SURFACE
	0.00													
	0.34	VERY STIFF GREY BROWN SILTY CLAY (WEATHERED GRAY)	1	50	5									
	56.42		2	50	52									NATIVE BACKFILL
	1.57		3	50	50									BENTONITE SEAL
		STIFF GREY SILTY CLAY (FRESH WATER ORIGIN)	4	75	48									CRUSHED STONE
			5	50	48									PIEZ #3
	46.73		6	75	46									BENTONITE SEAL
	7.01	VERY STIFF GREY SILTY CLAY (FRESH WATER ORIGIN)	7	75	44									NATIVE BACKFILL
			8	75	42									BENTONITE SEAL
	42.12		9	75	40									CRUSHED STONE
	10.47		10	75	38									PIEZ #2
			11	50	36									BENTONITE SEAL
			12	75	34									NATIVE BACKFILL
			13	50	32									BENTONITE SEAL
			14	75	30									CRUSHED STONE
			15	50	28									PIEZ #1
			16	75	26									BENTONITE SEAL
			17	50	24									NATIVE BACKFILL
			18	75	22									PLASTIC TUBING
			19	50	20									
			20	75	18									
			21	50	16									
			22	75	14									
		23	50	12										
		24	75	10										
		25	50	8										
		26	75	6										
		27	50	4										
		28	75	2										
		29	50	0										
		30	75											
		31	50											
		32	75											
		33	50											
		34	75											
		35	50											
		36	75											
		37	50											
		38	75											
		39	50											
		40	75											
		41	50											
		42	75											
		43	50											
		44	75											
		45	50											
		46	75											
		47	50											
		48	75											
		49	50											
		50	75											
		51	50											
		52	75											
		53	50											
		54	75											
		55	50											
		56	75											
		57	50											
		58	75											
		59	50											
		60	75											
		61	50											
		62	75											
		63	50											
		64	75											
		65	50											
		66	75											
		67	50											
		68	75											
		69	50											
		70	75											
		71	50											
		72	75											
		73	50											
		74	75											
		75	50											
		76	75											
		77	50											
		78	75											
		79	50											
		80	75											
		81	50											
		82	75											
		83	50											
		84	75											
		85	50											
		86	75											
		87	50											
		88	75											
		89	50											
		90	75											
		91	50											
		92	75											
		93	50											
		94	75											
		95	50											
		96	75											
		97	50											
		98	75											
		99	50											
		100	75											
		101	50											
		102	75											
		103	50											
		104	75											
		105	50											
		106	75											
		107	50											
		108	75											
		109	50											
		110	75											
		111	50											
		112	75											
		113	50											
		114	75											
		115	50											
		116	75											
		117	50											
		118	75											
		119	50											
		120	75											
		121	50											
		122	75											
		123	50											
		124	75											
		125	50											
		126	75											
		127	50											
		128	75											
		129	50											
		130	75											
		131	50											
		132	75											
		133	50											
		134	75											
		135	50											
		136	75											
		137	50											





RECORD OF BOREHOLE 4																
LOCATION See Figure 2			BORING DATE MAY 14 & 15, 1979			DATUM GEODETTIC										
SAMPLER HAMMER WEIGHT 63.5 Kg DROP 0.76m			PENETRATION TEST HAMMER WEIGHT 63.5 Kg DROP 0.76m													
BORING METHOD	SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				COEFFICIENT OF PERMEABILITY, k_v , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEVATION DEPTH	DESCRIPTION	STRAT. PLAT	NUMBER		TYPE	SHEAR STRENGTH Cu, K. Pa.	NAT. V. + REM. V. -	Q. - U. -	1x10	1x10	1x10	1x10			
POWER - CABLE CABLE DATA (SEE W. 100)	51.00	GROUND SURFACE													GROUND SURFACE	
	50.27	FIRM GREY BROWN SILTY CLAY, TRACE BROWN ORGANIC MATERIAL (FILL)		1	PH											SURFACE SEAL
	51.1			2	PH											NATIVE BACKFILL
	50.44			3	PH											BENTONITE SEAL
	49.16	STIFF GREY SILTY CLAY (FRESH-WATER ORIGIN)		4	PH											GRAVEL BACKFILL
	48.95			5	PH											PIEZ "3"
	48.16			6	PH											BENTONITE SEAL
	47.46			7	PH											NATIVE BACKFILL
	46.75			8	PH											BENTONITE SEAL
	46.05			9	PH											GRAVEL BACKFILL
	45.35			10	PH											PIEZ "2"
	44.65	VERY STIFF GREY SILTY CLAY (FRESH-WATER ORIGIN)		11	PH											BENTONITE SEAL
43.95			12	PH											NATIVE BACKFILL	
43.25			13	PH											PLASTIC TUBING	
42.55			14	PH											BENTONITE SEAL	
41.85			15	PH											GRAVEL BACKFILL	
41.15			16	PH											PIEZ "1"	
40.45			17	PH												
39.75			18	PH												
39.05			19	PH												
38.35			20	PH												
37.65			21	PH												
36.95			22	PH												
36.25			23	PH												
35.55			24	PH												
34.85			25	PH												
34.15			26	PH												
33.45			27	PH												
32.75			28	PH												
32.05			29	PH												
31.35			30	PH												
30.65			31	PH												
29.95			32	PH												
29.25			33	PH												
28.55			34	PH												
27.85			35	PH												
27.15			36	PH												
26.45			37	PH												
25.75			38	PH												
25.05			39	PH												
24.35			40	PH												
23.65			41	PH												
22.95			42	PH												
22.25			43	PH												
21.55			44	PH												
20.85			45	PH												
20.15			46	PH												
19.45			47	PH												
18.75			48	PH												
18.05			49	PH												
17.35			50	PH												
16.65			51	PH												
15.95			52	PH												
15.25			53	PH												
14.55			54	PH												
13.85			55	PH												
13.15			56	PH												
12.45			57	PH												
11.75			58	PH												
11.05			59	PH												
10.35			60	PH												
9.65			61	PH												
8.95			62	PH												
8.25			63	PH												
7.55			64	PH												
6.85			65	PH												
6.15			66	PH												
5.45			67	PH												
4.75			68	PH												
4.05			69	PH												
3.35			70	PH												
2.65			71	PH												
1.95			72	PH												
1.25			73	PH												
0.55			74	PH												
0.25			75	PH												
0.00			76	PH												

VERTICAL SCALE 1:100

5 Percent axial strain at failure

W.L. IN PIEZ "1" AT ELEV. 52.10
PIEZ. 2" AT ELEV. 50.43
PIEZ. 3" AT ELEV. 49.84
JULY 16, 1979

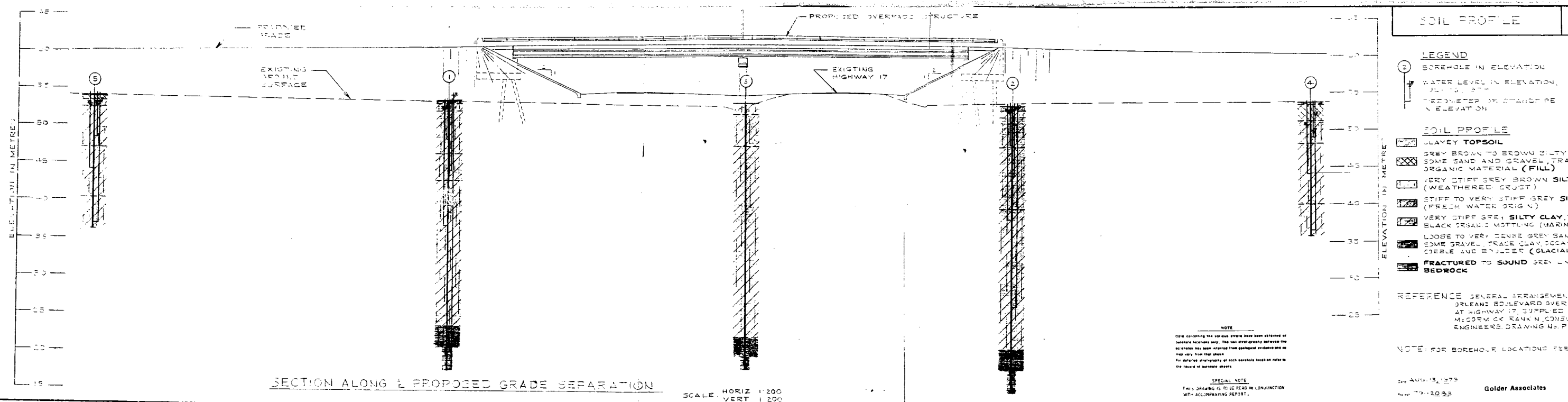
DRAWN RKB
CHECKED JAS

Golder Associates

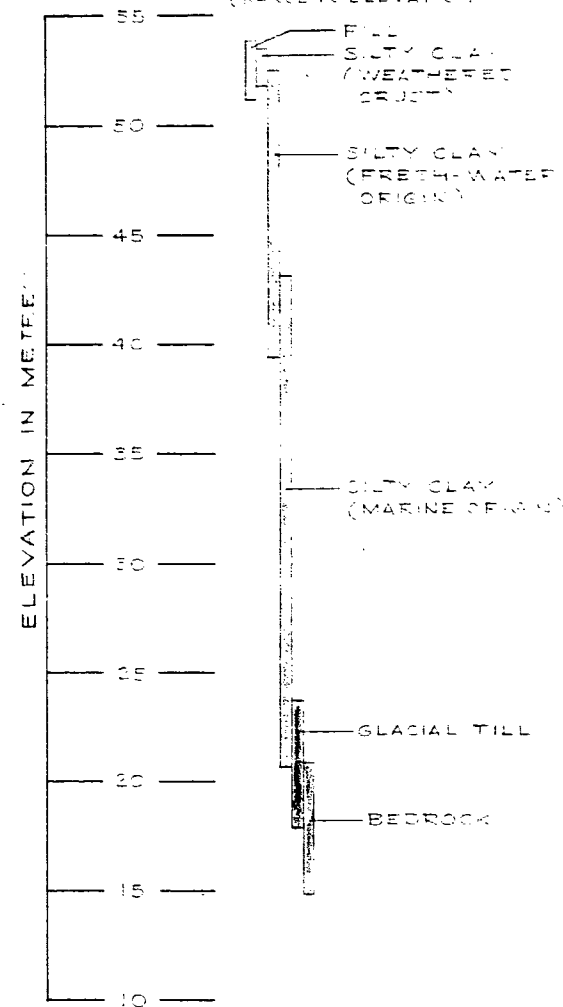
RECORD OF BOREHOLE 5															
LOCATION See Figure 2			BORING DATE MAY 15 & 16, 1979			DATUM GEODETIC									
SAMPLER HAMMER WEIGHT 45.5 kg DROP 0.76 m			PENETRATION TEST HAMMER WEIGHT 63.5 kg DROP 0.76 m												
BORING METHOD	SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3 m				COEFFICIENT OF PERMEABILITY, k_v , CM/SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEVATION DEPTH	DESCRIPTION	STRAT. PLT. NUMBER	TYPE		SHEAR STRENGTH CU: K Pa	NAT. V + REM V -	Q - U - O	1x10	1x10	1x10	1x10			
POCKET AUGER 0.76 m DIA. (HOLLOW STEEL)	54.0	1.5 m SANDY SILT CLAY SANDY SILT CLAY GRAVEL FILL	1	50	54										GROUND SURFACE
	52.0	2.0 m SANDY SILT CLAY SANDY SILT CLAY GRAVEL FILL	2	50	52										SURFACE SEAL
	50.0	3.0 m STIFF GREEN SILTY CLAY FRESH WATER	3	50	50										NATIVE BACKFILL
	48.0	4.0 m STIFF GREEN SILTY CLAY FRESH WATER	4	50	48										PLASTIC TUBING
	46.0	5.0 m STIFF GREEN SILTY CLAY FRESH WATER	5	50	46										BENTONITE SEAL
	44.0	6.0 m STIFF GREEN SILTY CLAY FRESH WATER	6	50	44										PIEZ. 1"
	42.0	7.0 m STIFF GREEN SILTY CLAY FRESH WATER	7	50	42										GRAVEL BACKFILL
	40.0	8.0 m STIFF GREEN SILTY CLAY FRESH WATER	8	50	40										BENTONITE SEAL
	38.0	9.0 m STIFF GREEN SILTY CLAY FRESH WATER	9	50	38										NATIVE BACKFILL
	36.0	10.0 m STIFF GREEN SILTY CLAY FRESH WATER	10	50	36										BENTONITE SEAL
	34.0	11.0 m STIFF GREEN SILTY CLAY FRESH WATER	11	50	34										PIEZ. 2"
	32.0	12.0 m STIFF GREEN SILTY CLAY FRESH WATER	12	50	32										GRAVEL BACKFILL
	13.0 m STIFF GREEN SILTY CLAY FRESH WATER	13	50											W.L. IN PIEZ. 1" AT ELEV. 53.20	
	14.0 m STIFF GREEN SILTY CLAY FRESH WATER	14	50											PIEZ. 2" AT ELEV. 52.89	
	15.0 m STIFF GREEN SILTY CLAY FRESH WATER	15	50											PIEZ. 3" AT ELEV. 52.80	
	16.0 m STIFF GREEN SILTY CLAY FRESH WATER	16	50											JULY 16, 1979	
	17.0 m STIFF GREEN SILTY CLAY FRESH WATER	17	50												
	18.0 m STIFF GREEN SILTY CLAY FRESH WATER	18	50												
	19.0 m STIFF GREEN SILTY CLAY FRESH WATER	19	50												
	20.0 m STIFF GREEN SILTY CLAY FRESH WATER	20	50												
	21.0 m STIFF GREEN SILTY CLAY FRESH WATER	21	50												
	22.0 m STIFF GREEN SILTY CLAY FRESH WATER	22	50												
	23.0 m STIFF GREEN SILTY CLAY FRESH WATER	23	50												
	24.0 m STIFF GREEN SILTY CLAY FRESH WATER	24	50												
	25.0 m STIFF GREEN SILTY CLAY FRESH WATER	25	50												
	26.0 m STIFF GREEN SILTY CLAY FRESH WATER	26	50												
	27.0 m STIFF GREEN SILTY CLAY FRESH WATER	27	50												
	28.0 m STIFF GREEN SILTY CLAY FRESH WATER	28	50												
	29.0 m STIFF GREEN SILTY CLAY FRESH WATER	29	50												
	30.0 m STIFF GREEN SILTY CLAY FRESH WATER	30	50												
	31.0 m STIFF GREEN SILTY CLAY FRESH WATER	31	50												
	32.0 m STIFF GREEN SILTY CLAY FRESH WATER	32	50												
	33.0 m STIFF GREEN SILTY CLAY FRESH WATER	33	50												
	34.0 m STIFF GREEN SILTY CLAY FRESH WATER	34	50												
	35.0 m STIFF GREEN SILTY CLAY FRESH WATER	35	50												
	36.0 m STIFF GREEN SILTY CLAY FRESH WATER	36	50												
	37.0 m STIFF GREEN SILTY CLAY FRESH WATER	37	50												
	38.0 m STIFF GREEN SILTY CLAY FRESH WATER	38	50												
	39.0 m STIFF GREEN SILTY CLAY FRESH WATER	39	50												
	40.0 m STIFF GREEN SILTY CLAY FRESH WATER	40	50												
	41.0 m STIFF GREEN SILTY CLAY FRESH WATER	41	50												
	42.0 m STIFF GREEN SILTY CLAY FRESH WATER	42	50												
	43.0 m STIFF GREEN SILTY CLAY FRESH WATER	43	50												
	44.0 m STIFF GREEN SILTY CLAY FRESH WATER	44	50												
	45.0 m STIFF GREEN SILTY CLAY FRESH WATER	45	50												
	46.0 m STIFF GREEN SILTY CLAY FRESH WATER	46	50												
	47.0 m STIFF GREEN SILTY CLAY FRESH WATER	47	50												
	48.0 m STIFF GREEN SILTY CLAY FRESH WATER	48	50												
	49.0 m STIFF GREEN SILTY CLAY FRESH WATER	49	50												
	50.0 m STIFF GREEN SILTY CLAY FRESH WATER	50	50												

Golder Associates

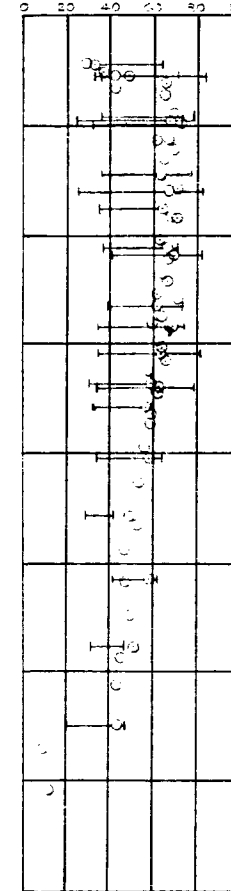
DRAWN BY J. D. B.
CHECKED BY J. D. B.



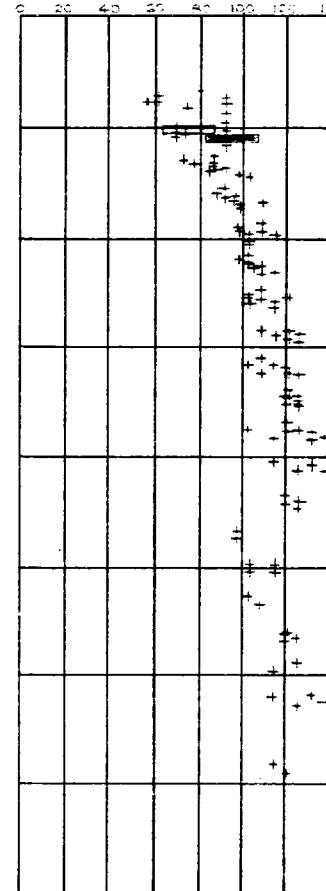
EMPIRICAL SOIL PROFILE (RANGE IN ELEVATION)



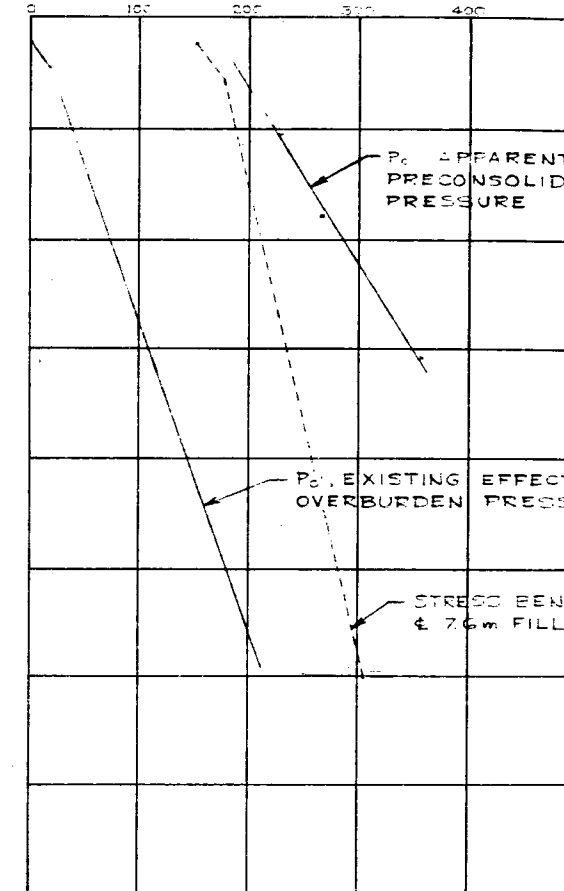
WATER CONTENT %



SHEAR STRENGTH, KPa



PRESSURE, KPa



SUMMARY OF GEOTECHNICAL PROPERTIES

FIGURE 4

LEGEND

- W_p = PLASTIC LIMIT
- W_L = LIQUID LIMIT
- W = NATURAL
- + IN SITU FIELD VANE TEST
- UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULT
- RANGE FOR HORIZONTALLY ORIENTED SAMPLES
- RANGE FOR VERTICALLY ORIENTED SAMPLES

Date: AUG 13, 1979

Project: 791-2053

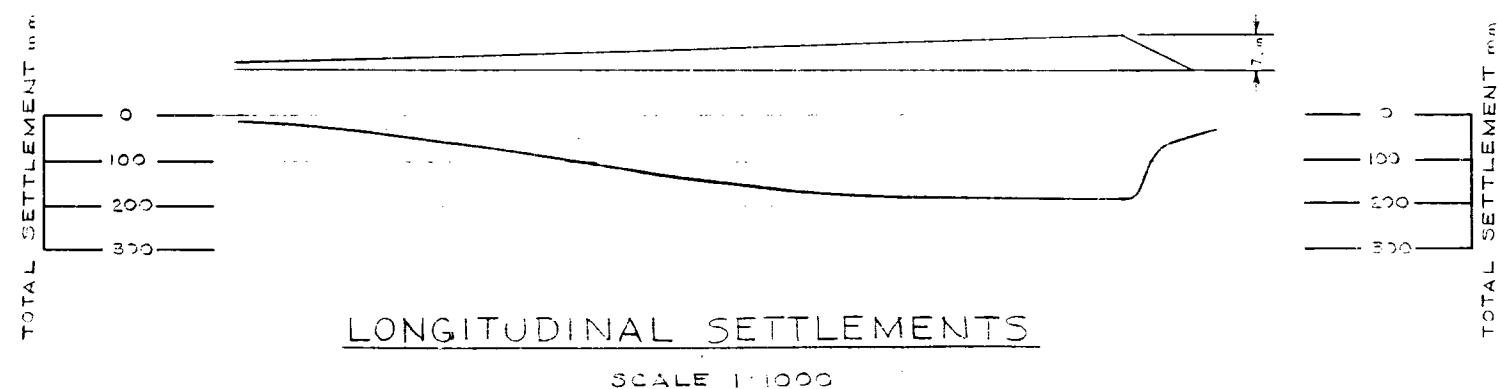
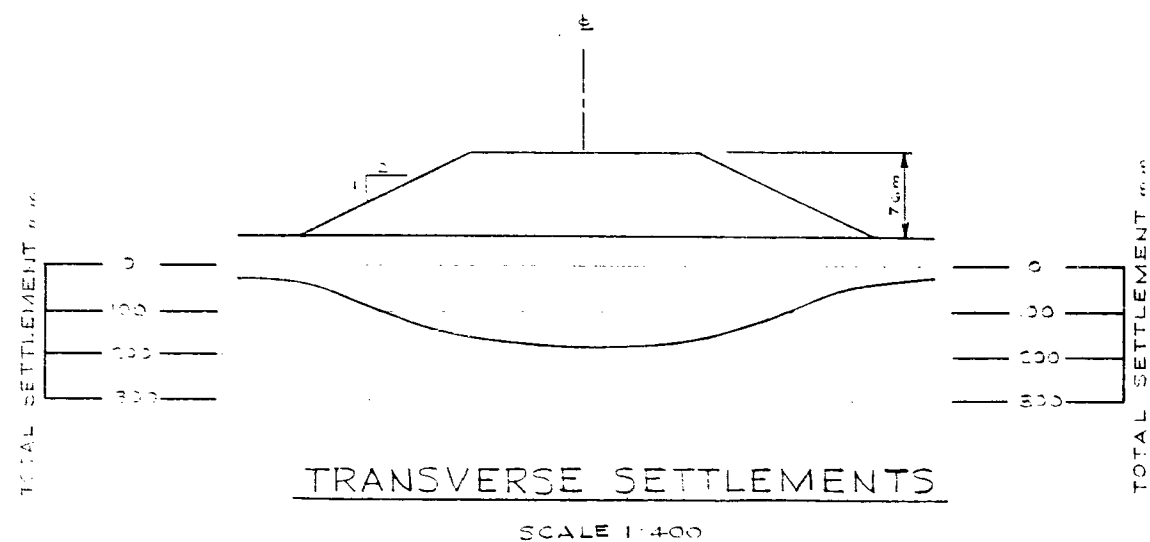
Golder Associates

Drawn by: J. J. J.

Checked by: J. J. J.

CALCULATED VERTICAL
SETTLEMENT PROFILES

FIGURE 10



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

Date: 11/11/2011
Project: 11111111

Golder Associates

Drawn: [Signature]
Chkd: [Signature]