

GEOCRES No. 31G5-94

DIST. 9 REGION

W.P. No.

CONT. No.

W.O. No. 73-11040C

STR. SITE No.

HWY. No.

LOCATION SLOPE STABILITY STUDY

BORDEN FARM SUBDIVISION

NO. OF PAGES - 1

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

73-11040C

GEOC. 31G5-94

73-110400C

3165-94

J.L. RICHARDS & ASSOCIATES LIMITED

SLOPE STABILITY STUDY

PROPOSED BORDEN FARM SUBDIVISION

OTTAWA

73-11040C

ONTARIO

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April, 1973.

73704

Golder Associates



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

April 2, 1973.

J.L. Richards & Associates Limited,
864 Lady Ellen Place,
Ottawa, Ontario.
K1Z 5M2

Attention: Mr. W.H. Kerr, P.Eng.

RE: SLOPE STABILITY STUDY
PROPOSED BORDEN FARM SUBDIVISION
OTTAWA, ONTARIO

Dear Sirs:

This letter reports the results of a slope stability study carried out at the above site. The purpose of this study was to determine the soil and groundwater conditions at the site and based on this information to determine the general stability of the slopes along Nepean Creek to the south of the site and along the banks of the Rideau River to the east of the site.

PROCEDURE

The field work for this investigation was carried out between January 26 and February 7, 1973. During this period, three boreholes were put down using a machine drill rig supplied and operated by the F.E. Johnston Drilling Co. Ltd., Ottawa. Standard drive open and thin walled Shelby tube samples were taken in the subsoil to depths ranging from about 28 to 56 ft. and cone penetration tests were carried out in each borehole to determine the shear strength profile of the silty clay deposit. Standpipes and piezometers were installed at various levels in the boreholes to determine the piezometric water levels in the subsoil. The field work was supervised throughout by a member of our engineering staff.

A detailed log of each boring is given on the Record of Borehole sheets following the text of this report. The locations of the borings, together with cross sections taken of the north bank of Nepean Creek, are shown on Fig. 1.

The samples obtained during this investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on the Record of Borehole sheets.

The ground surface elevations and locations of the boreholes were provided by J.L. Richards & Associates Limited, survey personnel. The elevations are referred to Geodetic datum.

SITE AND GEOLOGY

The site is located on a parcel of land which is bounded on the east by Highway No. 16 and on the north and south by Borden Side Road and Nepean Creek, respectively. The topography across the site is gently undulating with a fall of about 2 per cent to the south west.

From available geological information it is known that the area is underlain by dolomitic limestone of the Ottawa formation. In general, bedrock is overlain by a shallow deposit of glacial till followed by variable thicknesses of sensitive silty clays of marine origin.

SOIL CONDITIONS

The detailed soil stratigraphy encountered in each borehole is given on the Record of Borehole sheets and is illustrated on the sections on Fig. 1. Following is a summarized account of the soil conditions at the site.

Silty Clay

The principal subsoil stratum at this site is the sensitive silty clay which was encountered in all boreholes underlying a thin surficial topsoil layer, and in the area of boreholes 1 and 3 underlying 3 to 3.5 ft. of silty sand. At the locations of boreholes 1 and 2, the top of the clay deposit was found to have been weathered to a stiff to very stiff brown crust to a depth of 12 to 14 ft. At borehole 3, which was put down in the northern section of the site, the weathered crust has a firm consistency and only extends to a depth of about 6.5 ft.

Below the weathered crust the colour of the silty clay generally changes from brown to grey and the consistency decreases to soft to firm. The clay was found to contain a trace to some sand and some $\frac{1}{4}$ in. to 1 in.

thick silty sand layers. In the vicinity of boreholes 1 and 3, the silty clay is interlayered with silty sand between depths of 22 to 25 ft. and 30 to 34 ft., respectively.

The results of Atterberg limit tests carried out on the silty clay indicate that it has a liquid limit ranging from about 60 to 80 and a plastic limit of about 20, indicating a highly plastic clay. The natural water content of the weathered crust ranges from about 54 to 72 per cent. In the underlying grey clay the natural water content ranges from about 35 to 100 per cent and generally approaches or exceeds the liquid limit. The lower values were obtained in samples having a high sand content.

The shear strength profile of the silty clay stratum is shown on the Record of Borehole sheets and on Fig. 2. In boreholes 1 and 2, where the stiff crust extends to appreciable depth, the in situ shear strength is in excess of 1,000 lb/sq.ft. directly beneath the weathered crust and decreases to about 800 lb/sq.ft. with depth. In the vicinity of borehole 3, where the weathered crust is of limited extent the in situ shear strength of the grey clay is only about 500 lb/sq.ft. beneath the crust and increases to about 800 to 900 lb/sq.ft. with depth. In the grey clay, values in excess of 1,000 lb/sq.ft. are considered to have been caused by sand layers in the zone being tested.

Sand, some Silt

Borehole 2 was terminated within the silty clay at a depth of 30 ft. In borehole 1 and 3 the silty clay was found to be underlain by loose to very dense light brown to grey sand containing some silt. This sand stratum is about 9 ft. thick in the vicinity of borehole 1 and is in turn underlain by stiff grey clayey silt. Sampling was terminated within the sand in borehole 3.

Cone penetration tests were carried out below each borehole. The results of the penetration tests indicate that the soil below the depth of sampling is probably layered. Practical refusal was encountered at depths ranging from about 64 to 89 ft.

GROUNDWATER CONDITIONS

Piezometers and standpipes were installed at various levels within the overburden to measure the piezometer levels and gradients within the clay. Details of these installations are given on the Record of Borehole

sheets together with the water levels obtained on February 20, 1973. The standpipes in the underlying sand recorded a water level at about elevation 255, some 15 to 25 ft. below ground surface. Piezometers sealed in the clay indicated a downward drainage with water levels near ground surface in the shallow piezometers.

SLOPE STABILITY

From the results of this investigation and previous investigations carried out in the general area of the proposed Borden Farm Subdivision, it is known that the area is underlain by deposits of sensitive silty clay to depths of some 40 to 50 ft. Profiles were taken of the natural slopes along Nepean Creek, to the south of the site. These are shown on Fig. 1 and indicate that the sensitive clay extends to a depth of about 5 to 10 ft. below the base of the slopes.

A study of slides which have occurred along the west bank of the Rideau River adjacent to the site has shown that these slides are shallow and generally occur during the spring, when water conditions in the surficial portion of the slopes approach full saturation. The available information regarding these slides has been analysed to determine the strength characteristics active in the subsoil at the time of failure. The strength parameters obtained from analysis of the most recent slide, which occurred on the MacDonald property in April, 1972 are shown on Fig. 3 as a function of the pore pressure parameter $r_u = \frac{u}{\gamma H}$, where:

u = pore pressure, in lb/sq.ft.

γ = unit weight of soil, in lb/cu.ft.

H = height of soil above the point where 'u' is measured, in ft.

The calculated values for ϕ' (effective angle of internal friction) and c' (effective cohesion) are generally in good agreement with those obtained by the National Research Council in their analysis of the October 1965 slide in Orleans, Ontario. Effective stress parameters of $\phi' = 32^\circ$ and $c' = 250$ lb/sq.ft. were obtained by the N.R.C. for conditions of low confining pressures, such as are present in the shallow slides that occur in the Rideau River banks. These values are also in reasonably good agreement with values of ϕ' and c' calculated from available information on the Karsh property slide of 1947.

The stability of the slopes along the west bank of the Rideau River against deep seated slides, which would involve a much larger land mass, was considered. In these stability calculations the ϕ' value under higher confining pressures was reduced to 28° . The c' value was conservatively taken as 300 lb/sq.ft., the value calculated from the past surficial slides under conditions of full saturation. Due to the pronounced downward drainage of the clay stratum at this site, the r_u factor is much less severe on a deep seated potential slip surface. In this case, an r_u factor of 0.35 has been determined from the piezometer results. The calculated factor of safety against a deep seated slide is 1.3, which is just adequate. In view of this factor of safety and also the marginal surficial stability of steep river slopes, remedial measures as discussed below are required to ensure the stability of these main slopes and of the adjacent land areas.

The stability of the slopes along Nepean Creek has also been examined. A minor slope failure has taken place about 600 ft. west of Highway 16, at an outbend of the creek. Active erosion at the toe of the slope produced a slope angle of about 45° which resulted in a surficial slide. Erosion protection as discussed under "Remedial Measures" will be required to avoid the oversteepening of the Nepean Creek slopes.

The deep seated stability of the Nepean Creek slopes has been checked. On the basis of a total stress analysis, using an average shear strength of 650 lb/sq.ft. the factor of safety of Section 1 on Fig. 1 is calculated to be 1.35. For the effective stress analysis, using a ϕ' of 28° , a c' of 300 lb/sq.ft., and a r_u factor of 0.37 as calculated from the piezometer results, the factor of safety is calculated to be 1.6.

REMEDIAL MEASURES

Due to the marginal stability of the Rideau River slopes in this area, and the just adequate deep seated stability, it is recommended that remedial measures be carried out at this time to halt the active erosion at the toe of these steep slopes. It is recommended in this case that a rock berm, some 15 ft. wide and with a top elevation some 2 ft. above the high water level, be placed along the west bank of the Rideau River, from the Nepean Creek to opposite the old Borden Side Road, a distance of about 1,200 ft. This rock berm should be continued up Nepean Creek to Highway 16, an additional length of about 350 ft. This rock berm, by arresting toe erosion, should improve the surficial stability which, if left uncontrolled,

could trigger a deep seated slide.

No active undercutting of the toe of the slope has been noted along the steeper section of Nepean Creek within 400 ft. west of Highway 16. Consideration should, however, be given to installation of gabions along the north bank of the creek in this area to prevent scour and possible undercutting due to the increased flow created by runoff from future developments along this creek. Some minor undercutting of the slope has occurred approximately 600 ft. west of Highway 16 adjacent to the proposed Block H. This slope should be flattened and gabions should be installed at the toe of the slope. Gabions should also be installed at each of the outbends of the creek where active erosion is taking place.

It is recommended that the proposed high rise structures and townhouses in Blocks 'H' and 'L' be located at least 150 ft. from the crest of the slope, which would involve about a 70 ft. setback from the south property line. The present grade at the top of the creek slopes should not be increased throughout the length of the development. To ensure this, the lot and fence lines should be set back slightly from the crest of the slopes and the land at the crest of the slope and down the slopes be deeded as public land.

Since the stability of the slope is sensitive to the groundwater level, rain water and snow melt from the buildings and paved parking areas should be collected and fed into a storm sewer system and should not be allowed to seep uncontrolled towards the slope.

The foliage which presently covers the slopes along Nepean Creek should remain unchanged as it increases the overall stability and reduces the possibility of surficial erosion.

FOUNDATION AND STRUCTURES

High Rise Structure

Due to the low shear strength of the clay deposit, as shown on Fig. 2, support of high rise structures cannot be accomplished with spread footings. High rise structures could be supported on end bearing piles driven into the very dense soil at depths of about 60 to 90 ft. below present ground surface or on a raft foundation. In the case of a raft foundation, little net increase in loading would be

permissible above the soft clay, that is, the total weight of the high rise structure including the raft foundation should be approximately equal to the weight of the soil excavated for the basement level. Additional soil investigations should be carried out prior to design of foundations for each high rise structure.

One and Two Storey Residential Structures

One and two storey single and multi-family structures could be founded on spread footings at this site. Such footings should be placed within the weathered crust above the relatively soft grey clay. This serves to spread the load of the footings through the brown clay crust and onto the surface of the underlying softer grey clay. The allowable bearing pressures for spread footings depends on the depth of the crust, which is a function of the location on the site, and on the founding depth. For preliminary design purposes, the allowable bearing pressure may be conservatively taken as 1,000 lb/sq.ft.

We trust that this report contains sufficient information for your present purposes. Should you have any questions concerning this report or if we can be of further assistance on this project, please call us.

Yours very truly,

H.Q. GOLDER & ASSOCIATES LTD.

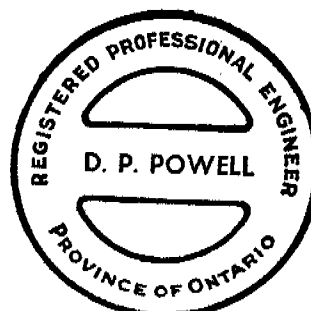


F.J. Heffernan, P.Eng.



D.P. Powell, P.Eng.

DPP/cn
73704
April, 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

<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.

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

<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_v	coefficient of consolidation
T_v	time factor = $c_v t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
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

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

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

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure 1

BORING DATE JANUARY 26 to 29, 1973

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, K, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH C _u , LB./SQ. FT.				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10			1x10
WASH BORING OPEN HOLE	274.9	GROUND SURFACE														GROUND LEVEL	
	270.9	DK BR. SILTY TOPSOIL		1	DO	7											
	270.9	LOOSE LIGHT BROWN SILTY FINE SAND, TRACE CLAY		2	DO	14											BENTONITE SEAL
	262.9	VERY STIFF REDDISH BROWN TO LIGHT BROWN SILTY CLAY OCCASIONAL 1/4" SILT LAYERS & SAND PARTINGS (WEATHERED CRUST)		3	DO	8											PIEZOMETER "B"
	262.9	FIRM TO STIFF LIGHT-BROWN TO GREY SILTY CLAY, INTERLAYERED WITH SILTY FINE SAND BETWEEN ELEV. 250 & ELEV. 253, TRACE TO SOME SAND & OCCASIONAL 1/4" TO 1/2" SAND LAYERS BELOW ELEV. 250		4	DO	2											BENTONITE SEAL
	250.0			5	TO PL												PIEZOMETER "A"
	240.0			6	DO	1											BENTONITE SEAL
	237.1			7	DO WH												PEA GRAVEL
	237.1	VERY DENSE LIGHT BROWN FINE TO MEDIUM SAND, SOME SILT		8	DO WH												STANDPIPE
	229.9	TRACE SHELLS		9	DO	52											
PENETRATION TEST	229.9	STIFF GREY CLAYEY SILT, SOME FINE SAND		10	DO	8											
	211.0	END OF PEN. TEST REFUSAL PROB. BEDROCK															
	211.0	PROBABLY VERY LOOSE TO COMPACT SOIL, BECOMING DENSE TO VERY DENSE BELOW ELEV. 212															
							100 BLOWS LAST 11"										

0
15
10
Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN P.V.

CHECKED L.P.

WATER LEVELS
TAKEN IN :
STANDPIPE AT
ELEV. 254.3
PIEZ. A. EL. 266.7
PIEZ. B. EL. 267.4
FEB. 20, 1973

RECORD OF BOREHOLE 2

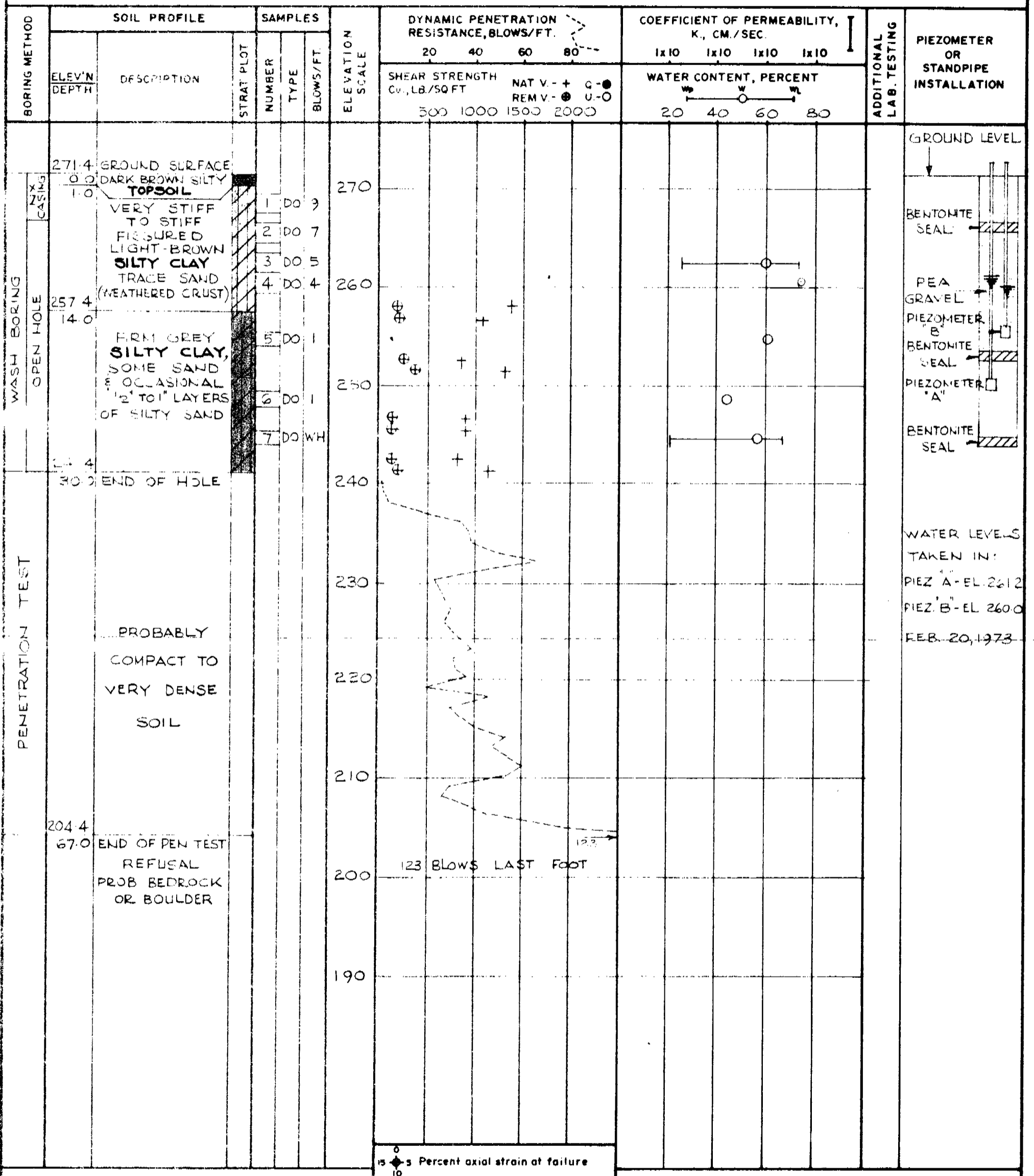
LOCATION See Figure 1

BORING DATE FEBRUARY 5, 1973

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



RECORD OF BOREHOLE 3

LOCATION See Figure 1

BORING DATE FEBRUARY 6 & 7, 1973

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN

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

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, K, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION								
	ELEV'N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20		40		60		80				1x10		1x10		1x10		1x10	
								SHEAR STRENGTH C _u , LB./SQ.FT.		NAT V - + REM V - ⊕		Q - ● U - ○		WATER CONTENT, PERCENT											
								500	1000	1500	2000					20	40	60	80						
WASH BORING BX CASING	281.8	GROUND SURFACE																							
	278.3	DARK BROWN SILTY TOPSOIL		1	DO 8		280																		
	275.5	LOOSE BROWN FINE TO MEDIUM SAND SOME SILT		2	DO 2																				
	275.5	FIRM BROWN SILTY CLAY (WEATHERED CRUST)																							
				3	DO PM		270																		
		SOFT TO FIRM GREY SILTY CLAY		4	DO WR																				
		SOME FINE TO MEDIUM SAND, OCCASIONAL 1/2"-1" SAND LAYERS		5	TO PM		260																		
		BELOW ELEV 257 INTERLAYERED WITH		6	DO PM																				
		FINE TO MEDIUM SAND BETWEEN ELEV. 248 & 252		7	DO WH		250																		
				8	DO 1																				
				9	DO 2		240																		
	WASH BORING OPEN HOLE	236.1	LOOSE TO VERY DENSE GRAY FINE TO MEDIUM SAND		10	DO 6																			
225.2		SOME SILT TRACE SHELLS		11	DO 52		230																		
56.6		END OF HOLE		12	DO 4																				
							220																		
							210																		
							200																		
							190																		
PENETRATION TEST		192.9																							
	88.9	END OF PEN. TEST REFUSAL PROB. BEDROCK																							

124 BLOWS LAST 11"

6
15 5 Percent axial strain of failure
10

GROUND LEVEL

BENTONITE SEAL

PIEZOMETER "B"

PEA GRAVEL

BENTONITE SEAL

PIEZOMETER "A"

BENTONITE SEAL

STANDPIPE

PEA GRAVEL

WATER LEVELS TAKEN IN :
STANDPIPE ELEV. 255.0
PIEZ. "A": EL. 261.2
PIEZ. "B": EL. 276.9
FEB. 20, 1973

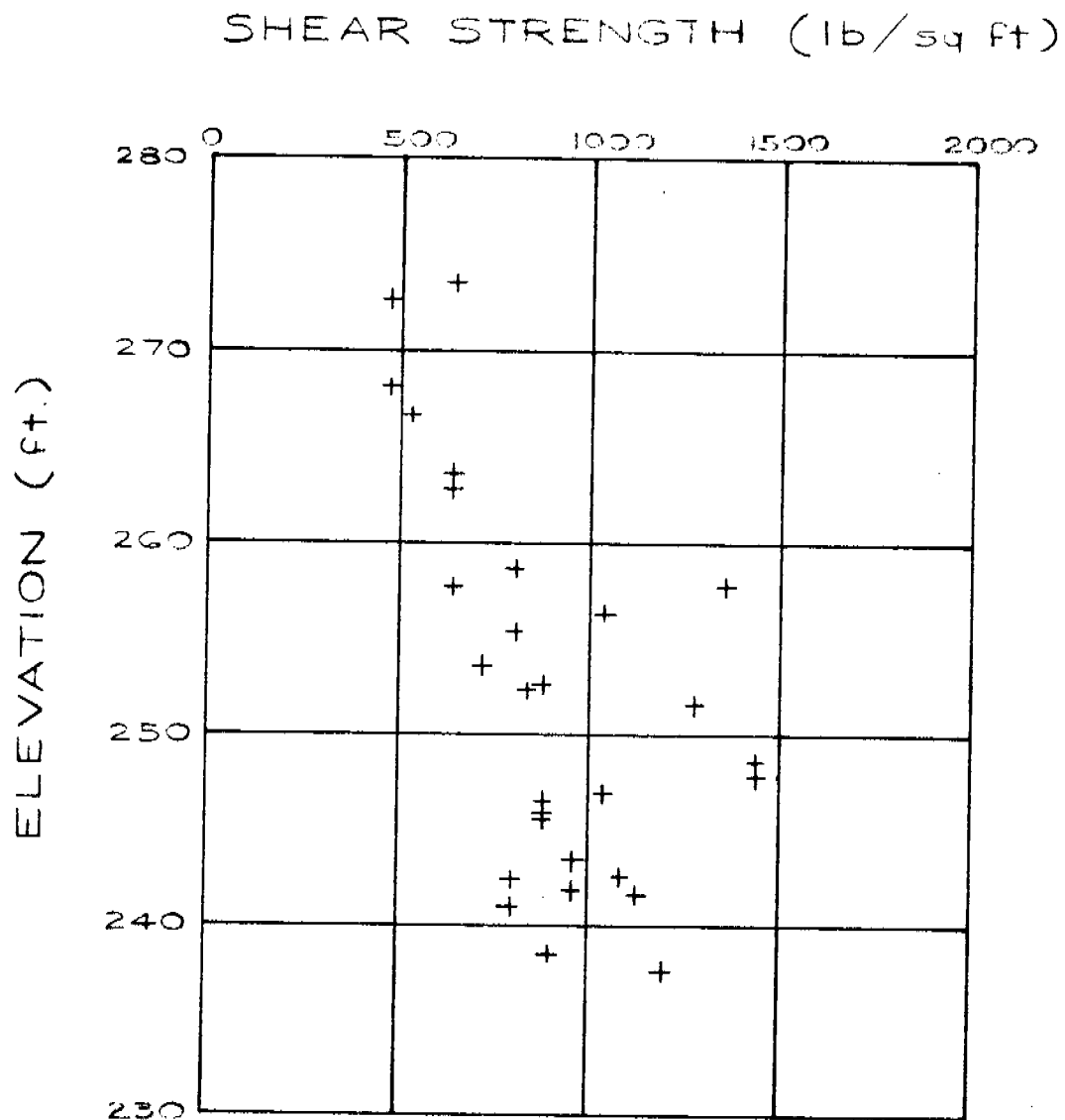
6
15 5 Percent axial strain at failure
10

WATER LEVELS TAKEN IN :
STANDPIPE ELEV. 255.0
PIEZ "A" : EL. 261.2
PIEZ. "B" : EL. 276.9
FEB. 20, 1973

OVERSIZE DRAWING(S)

SHEAR STRENGTH VERSUS ELEVATION

FIGURE 2

LEGEND

+ IN SITU VANE SHEAR TEST

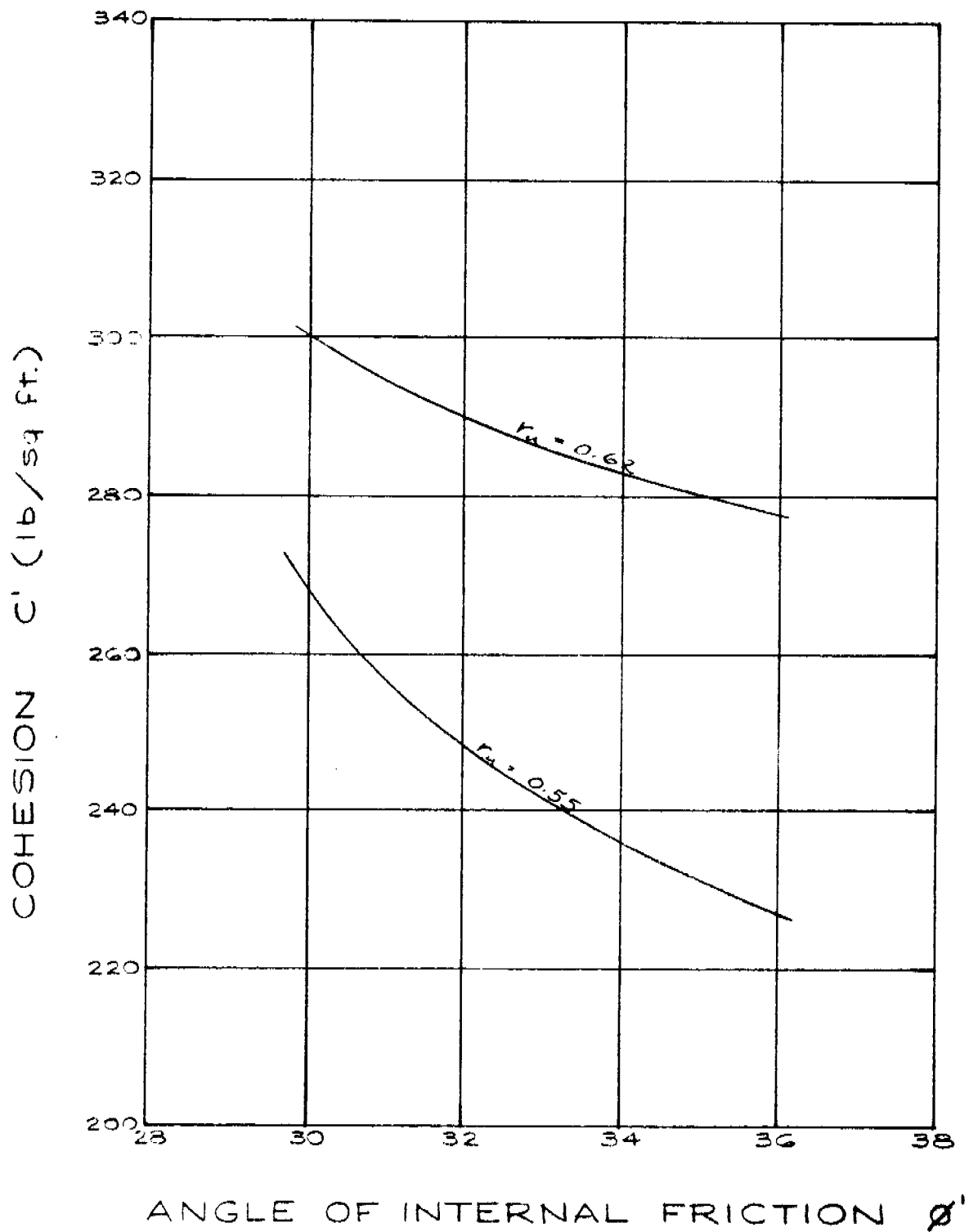
Date MARCH 21, 1973

Golder Associates

Drawn D.N.
Chkd. J.P.
Appd. J.P.

COHESION VERSUS ANGLE OF
INTERNAL FRICTION FOR F.S. = 1
MACDONALD SLIDE (APRIL, 1972)

FIGURE 3



Date MARCH 8, 1973

Golder Associates

Drawn D.N.
Chkd. [Signature]
Appd. [Signature]



Ontario

ACTION REQUEST

7840-1037 (2)

DATE

TO

FROM

Bob Lingwood

TELEPHONE NO.

~~553-4444~~ 5-5344

☐ - PLEASE CALL

☐ - WISHES APPOINTMENT

☐ - RETURNED YOUR CALL

☐ - WILL CALL BACK

☐ - NOTE AND FILE

☐ - PROVIDE MORE DETAILS

☐ - PLEASE ANSWER

☐ - NOTE AND FORWARD

☐ - FOR YOUR INFORMATION

☐ - DRAFT REPLY FOR MY SIGNATURE

☐ - NOTE AND RETURN

☐ - FOR YOUR APPROVAL

☐ - INVESTIGATE AND REPORT

☐ - NOTE AND SEE ME

☐ - FOR YOUR SIGNATURE

☐ - TAKE APPROPRIATE ACTION

☐ - RETURN WITH COMMENTS

☐ - PER YOUR REQUEST

☐ - _____

COMMENTS:

*re: Borden Farms
Dev. Ottawa*

CALL TAKEN BY:

TIME

Discussed this letter with Bob
Loywood of the Ministry of Natural
Resources and concluded that our
Comments of June 6th 1973 letter
covers their queries of their letter
dated June 15th 1973. It was
further agreed that they ~~will~~ should
incorporate ~~the~~ remedial measures
suggested in ~~Goldner~~ Goldner's Report
and no reply for their letter is
needed.

M. Devata
27th June/73.



Ontario

Ministry of
Natural
Resources

Land Use Co-ordination Branch
Whitney Block, Parliament Bldgs.
Toronto, Ontario

DIVISION OF LANDS

965-5344

Our file number

Your file number

June 15, 1973

Mr. A. G. Stermac,
Principal Foundations Engineer,
Ministry of Transportation and
Communication,
Design Services Branch,
1201 Wilson Ave.,
Downsview, Ontario.
M3M 1J8

Dear Mr. Stermac:

RE: T-20769, City of Ottawa, Borden Farm, W.O. 73 - 11040

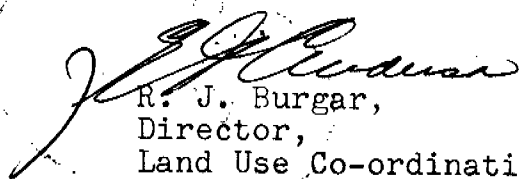
Further to your letter of June 6, 1973, it appears there may have been some confusion regarding the area surveyed.

The area shown in red on the small plan is already draft-approved. The area up for approval in principle lies to the south of the red area. Though we are, naturally, concerned about the already approved area, we are most concerned about the latter, southerly area. The questions we put were thus directed towards it.

Would you please advise us if this knowledge would affect your analysis and recommendation, specifically relating to any potential hazard in the southerly area.

We regret the confusion we caused and trust this added review is not burdensome. If you have any questions regarding the above, please do not hesitate to contact us.

Yours very truly,



R. J. Bugar,
Director,
Land Use Co-ordination Branch

WDD/af

mde



Design Services Branch,
1201 Wilson Avenue,
Downsview, Ontario.
M3M 1J8

Telephone: 248-3282.

June 6, 1973.

Mr. W. D. Dolman,
Development Plan Analyst,
Land Use Co-ordination Branch,
Ministry of Natural Resources,
Whitney Block, Parliament Bldgs.,
Toronto, Ontario.

Dear Mr. Dolman:

Re: *T-20769, City of Ottawa and Township
of Nepean Borden Farm W.O. 73-11040*

Further to your letter of May 24, 1973, we have reviewed the Foundation Report (No. 73704, dated April 1973) prepared by Golder Associates together with the information provided by you and submit the following comments.

The borings carried out by Golder Associates were confined to the areas (Blocks 'H', 'L' and 'M') which are outside the area outlined in red on the plan enclosed with your letter. It is, therefore, extremely difficult for us to comment on the two questions raised in your letter with regard to the possible instability hazards.

In any case, it is advisable to incorporate the remedial measures suggested by Golder Associates in their foundation report, since the future development in this area may likely increase the flow in the Nepean Creek.

Should you have any queries with regard to this project, please contact this Office.

Yours truly,



A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

MD/ao



Ontario

Ministry of
Natural
Resources

DIVISION OF LANDS

Land Use Co-ordination Branch
Whitney Block, Parliament Bldgs.
Toronto, Ontario

965-5344

Our file number

Your file number

May 24, 1973.

Mr. A. G. Stermac,
Foundations Office,
Ministry of Transportation and Communications,
West Building,
Downsview, Ontario.

73-11-040 (x)

Dear Mr. Stermac:

Re: T-20769, City of Ottawa and Township of Nepean
Borden Farm

Further to our telephone conversation, would you please review the attached 'Slope Stability Study' for the above plan by Golders Associates.

In the course of your analysis, could you please if at all possible answer the following questions in addition to your normal response:

1. Is there a potential instability hazard at the present time on the site outlined in red on the copy of the plan attached to the inside cover of the Golders report or on the site immediately south of it?
2. Would the total development proposed add in any way to any instability hazard if the remedial works recommended for the Rideau River were not done?

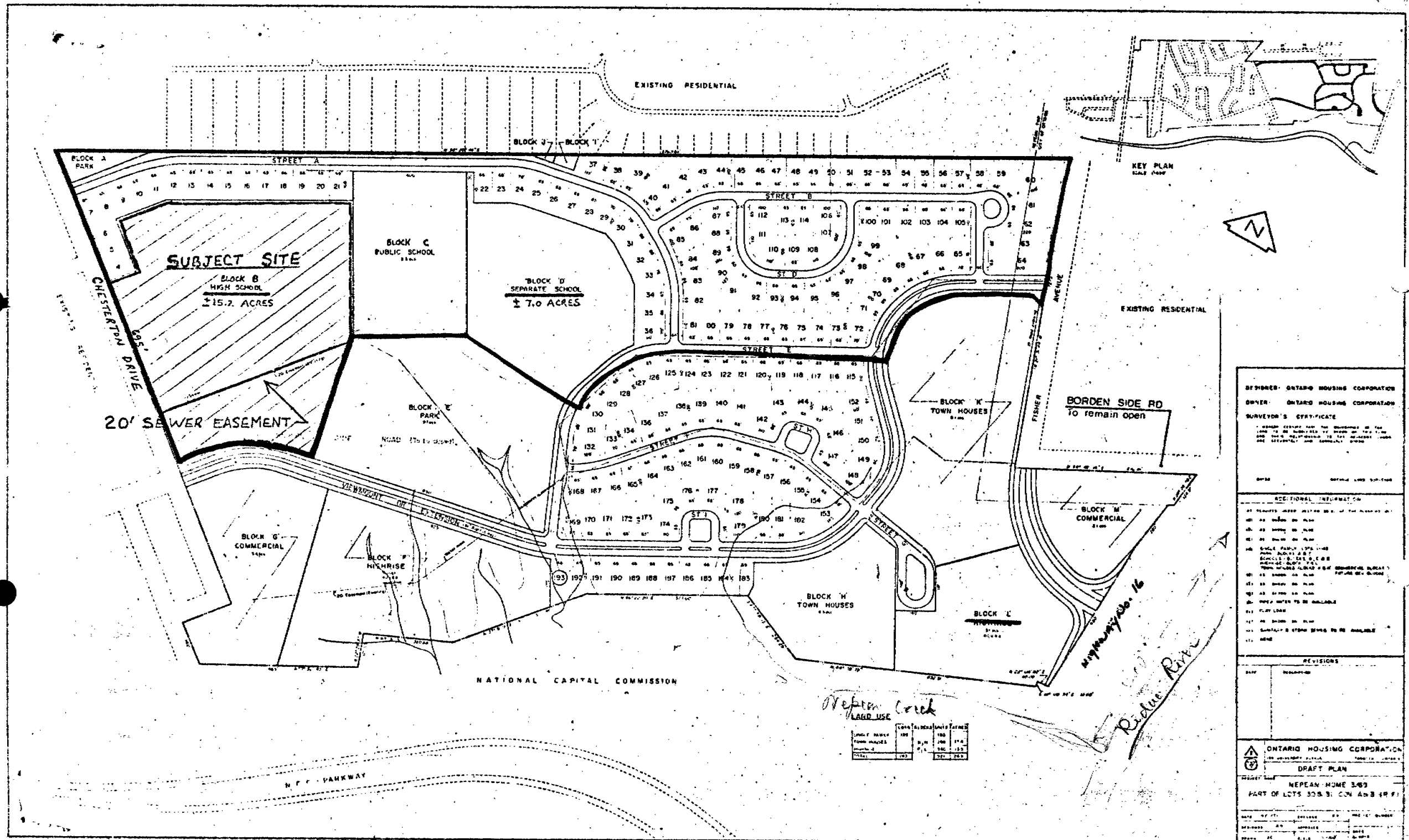
If you have any questions, or if the above cannot be answered from the information provided, would you please contact me.

Yours very truly,

W. D. Dolman,
Development Plan Analyst,
Land Use Co-ordination Branch.

Encl.





SCHEDULE "A"

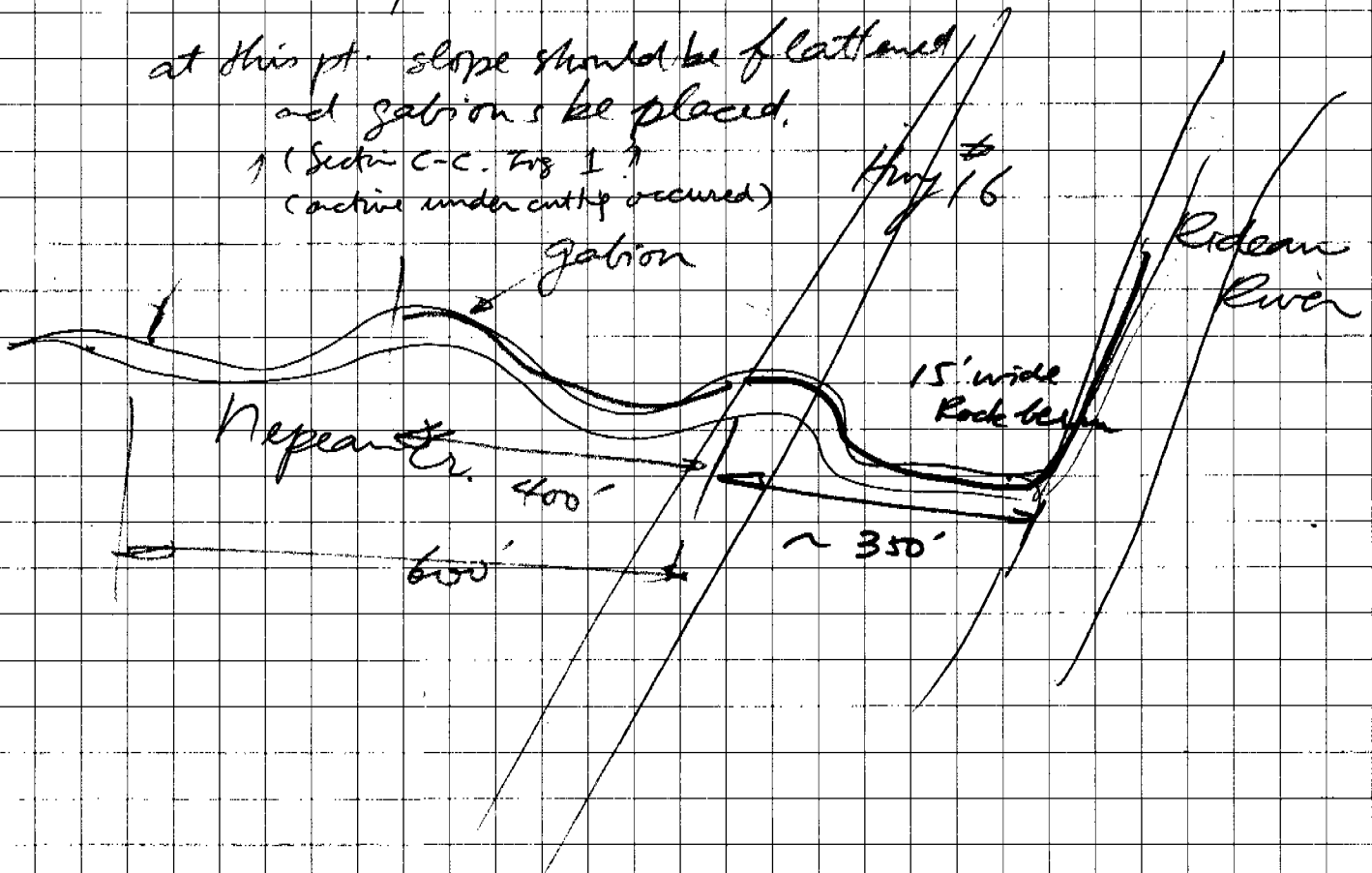
③ Recommendation:

at west Bank of Rideau River, 15-foot-wide rock berm installed at a level 2' above the H.W.L.,

also at north bank of Nepean Creek from Hwy 16 to Rideau River

at this pt. slope should be flattened and gabions be placed.

↑ (Section C-C. Fig 1)
(active under cutting occurred)



- ① Back calculations of a surficial slide
(MacDonald property April 1972) at west bank
of Rideau River (which is approx 400 ft east
of the subject site)

using $r_u = 0.55$ $r_u = 0.62$ $F_{\text{of } S.} = 1$, \therefore Result shown = Fig 3,
Result:
(1965 slide in
ex leaves out
by N.E.C.) $c' = 250$ psf $\phi' = 32^\circ$ \therefore seems to agree with

conclusion:

- 1) at low confining pressure: (surficial slide)

$$c' = 250 \text{ psf}$$

$$\phi' = 32^\circ$$

$$r_u = 0.55 - 0.62$$

- 2) at higher confining pressure (deep-seated)

$$c' = 300 \text{ psf}$$

$$\phi' = 28^\circ$$

$$r_u = 0.37 \text{ (as determined from piezometeric readings)}$$

may be used in analyzing the
stability of north bank of Nepean
Creek.

② Nepean Creek

total stress: $C_u = 650 \text{ psf}$

$FOS = 1.35$

Effective Stress: $C' = 300 \text{ psf}$
 $\phi' = 28^\circ$
 $F_u = 0.87$

$FOS = 1.6$

Typical slope: $2:1$
 $H = 30 \text{ ft}$

REQUESTS FOR
SERVICES OF THE FOUNDATIONS OFFICEW.O. 73-11040(X)W.P. NO — nil — CONT. NO — SITE NO —LOCATION: PARCEL OF LAND WHICH IS BOUNDED ON THE EAST
BY HWY[#] 16 & ON THE NORTH & SOUTH BY BORDEN SIDE RD.
& NEPPAN CREEKSERVICES REQUESTED: REVIEW OF GOLDER'S REPORT ON SLOPE
STABILITY STUDY - PROP. BORDEN FARM SUBDIVISION
OTTAWAREQUESTED BY: MR. DELMAN OF MINISTRY OF NATURAL
RESOURCEDATE OF REQUEST: JUNE 6/73

