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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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J. L. RICHARDS & ASSOCIATES LIMITED

SOIL INVESTIGATION AND
SLOPE STABILITY STUDY

ROTHWELL VILLAGE PHASE #3

OTTAWA

ONTARIO.

To: Tony Sterman

Golder Associates

CONSULTING GEOTECHNICAL ENGINEERS

H. Q. GOLDER
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C. O. BRAWNER
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F. J. HEFFERNAN
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J. B. DAVIS

March 9, 1972.

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

Attention: Mr. J. B. Mangione, P. Eng.

RE: SOIL INVESTIGATION AND SLOPE STABILITY STUDY,
ROTHWELL VILLAGE PHASE #3,
OTTAWA, ONTARIO.

Dear Sirs:

Further to our recent telephone conversation, this letter covers points raised by you about our soils report.

On page 11, we state that storm water and snow melt run-off should be collected by adequate underdrainage into the proposed storm sewer. As discussed with you, proper grading of the site to catch basins connected to the storm sewer system is suitable in this case.

On page 13, we incorrectly stated that plastic pipe would be used for the waterlines as well as for the storm and sanitary sewers. We now understand that ductile iron pipe will be used for the watermains and that the performance and reliability of this type is excellent.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.


F. J. Heffernan, P. Eng.

FJH/ml
71822

Golder Associates

CONSULTING GEOTECHNICAL ENGINEERS

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REPORT

TO

J. L. RICHARDS & ASSOCIATES LIMITED

ON

SOIL INVESTIGATION AND
SLOPE STABILITY STUDY

ROTHWELL VILLAGE PHASE #3

OTTAWA

ONTARIO.

Distribution:

6 copies - J. L. Richards & Associates Limited,
Ottawa, Ontario.

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

February, 1972

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ABSTRACT

The results of a soil investigation to determine the subsoil and groundwater conditions at the site of a proposed residential development along Montreal Road and west of Ogilvie Road (Rothwell Village, Phase #3), are reported. The stability of the escarpment to the north of the property has been assessed and recommendations for site development to maintain the stability are given.

The site is underlain by a deep deposit of highly plastic grey sensitive clay. From geological information, it is known that the clay is of marine origin which is common to much of Ottawa. This grey silty clay has a stiff consistency. The upper portion of this deposit (13 to 17 feet) has been weathered to a very stiff crust of fissured brown to grey brown clay. In the escarpment area, the clay is underlain at a depth of about 95 feet by glacial till deposits of a dense to very dense nature. The groundwater level within the surface portion of the clay is at a depth of about 4 feet. The water level within the underlying till was found to be at a depth of about 85 feet.

A major landslide took place in recent geological history in the escarpment within the general area under investigation. With the lowering of the Ottawa River level, recent servicing of subdivisions at the toe of the slope, and pumping from wells, the groundwater regime in this area has been lowered and the stability of this escarpment has been improved. Employing the groundwater readings which have been taken by the National Research Council in this area over the last 4 years, we calculate the factor of safety against instability of the existing slope to be now of the order of 1.4 which is considered adequate.

Recommendations are given in the report to ensure the stability of this escarpment in the future. These recommendations include design of services to avoid a raise in the groundwater level across the site and site grading restrictions to avoid oversteepening of the slope by either filling at the crest or excavation at the toe of slope.

Due to the thickness of the clay deposit at the site, it is recommended that high rise structures be founded on raft foundations. For preliminary design, a net allowable bearing pressure of 800 lb/sq.ft. is recommended. Two and three storey structures at this site may be founded on spread footings. The allowable bearing pressure for spread footings at 5 ft. depth may be taken as 3,000 lb/sq.ft.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by J. L. Richards & Associates Ltd. to carry out a geotechnical investigation on a parcel of land on the north side of Montreal Road and west of Ogilvie Road in Ottawa, Ontario. High, medium, and low density residential development has been proposed for this site. The purpose of this geotechnical investigation was to determine the soil and groundwater conditions at the site and, based on this information:

- a) to determine the general stability of the slope along the northern edge of the proposed development,
- b) to make general recommendations concerning the geotechnical factors which should be considered in the planning of the development, and
- c) to make recommendations concerning the design and construction of foundations and services within the proposed development as well as an adjacent area to the east of the subdivision.

PROCEDURE

The field work for this investigation was carried out between December 17 and 30, 1971. Four boreholes (numbered 1 to 4 inclusive) 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 clay and till subsoils present at the site. In situ vane tests were performed at each borehole to determine the shear strength profile of the clay deposit. A piezometer was installed within the clay and two standpipes were sealed into the till stratum at depth to determine the groundwater conditions existing at the time of the investigation. 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 sections of the inferred soil stratigraphy across the site, 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 and on Figures 2 to 4.

The elevations given in this report were supplied to us by J. L. Richards & Associates Limited survey personnel. These elevations are referred to Geodetic datum.

SITE AND GEOLOGY

The site is located on an upper river terrace east of the limits of the City of Ottawa on a parcel of land fronting on Montreal Road, from 1,000 to 2,500 feet west of Ogilvie Road. Along the northern edge of the site is an

escarpment which was formed between two former river terraces and in which a large land slide took place some 1,000 years ago. The topography in the lower terrace area is generally flat and the upper terrace rises gently to the southwest. The height of the escarpment is of the order of 65 feet.

From available geological information it is known that the area is underlain by limestone of the St. Martin and Ottawa formations. This bedrock outcrops along the escarpment to the west of the site and is encountered at a shallow depth on Clancy at the base of the escarpment in the western section of the property. In general, the bedrock is overlain by glacial till followed by variable thicknesses of sensitive silty clays of marine origin which extend to the ground surface. These clays were laid down in the Champlain Sea which occupied the area following the retreat of glaciers.

Because of the extensive size of the old land slide, its proximity to urban residential development to the north, and its proximity to the National Research Council, the geotechnical conditions at the site have been investigated before.⁽¹⁾ Reports on this work are listed as references at the end of this report, and have been used in the preparation of the recommendations. At the present time, the scarp of the land slide is stable.

SOIL CONDITIONS

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

Sensitive Silty Clay

The principal subsoil stratum at this site is the firm to very stiff grey sensitive silty clay which was found to be 97 feet thick at borehole 1 and 92.5 ft. thick at borehole 2. The upper portion of this clay, some 13 to 17 feet thick, has been weathered to a very stiff crust of fissured brown to grey brown silty clay. The water content of the weathered crust increases with depth from about 39 percent near the ground surface to about 64 percent near the base of this desiccated stratum. An Atterberg limit test indicates that this clay is highly plastic (liquid limit of about 72 and plasticity index of 49). ⁷²⁻²³
64

Below a depth of from 13 to 17 feet, the colour of the silty clay changes from grey brown to grey and the consistency decreases to stiff. In situ vane shear strength values below the very stiff crust were generally about 1,200 lb/sq.ft. in all the boreholes. At depths of 33 ft. and 46 ft. in boreholes 1 and 2 respectively, in situ shear strength values as low as 640 lb/sq.ft. were measured. The in situ shear strength values increased to 2,000 lb/sq.ft.

at depths of from 60 to 80 ft. below present ground surface. The shear strength values obtained by field vane and laboratory tests are plotted on the Record of Borehole sheets. The axial strains failure in the triaxial tests are generally about 4 percent and since strains to failure of undisturbed block samples of this clay are typically 1 to 2 percent (Townsend, Sangrey, & Walker, 1968; Mitchell, 1970) it is believed that the triaxial test results are low due to unavoidable sample disturbance.

A series of consolidated drained triaxial tests were carried out at constant mean normal stress and at low confining pressures typical of those existing within the present slope. The results of these tests are shown on Fig. 4. A curved failure envelope, typical of a cemented (or apparently over-consolidated) clay, is shown with the result that the effective stress shear strength parameters for slope stability analysis will vary depending upon the state of stress in the slope. For the preliminary stability analyses of slopes at this site, an effective angle of shearing resistance, ϕ' , of 32 degrees, and an effective cohesion intercept, c' , of about 350 lb. per sq.ft. have been used.

Atterberg limit tests indicate that the plasticity of the grey silty clay is generally high decreasing to

medium to low at depth (liquid limits of from 73 to 28 percent and plasticity indices of from 48 to 5). The moisture content of the grey clay decreases with depth from about 77 to 45. At depth the moisture content is well above the liquid limit value, typical of the sensitive clays of the Ottawa area.

Silty Sand Till

The Champlain Sea clay is underlain by a stratum of glacial till which was found to be 11 ft. thick at the location of borehole 1. As indicated by the grain size distribution curve on Fig. 3, this till is well graded. The mechanical analysis test was carried out on a sample obtained from a 1½ inch I.D. split spoon sampler and does not reflect the presence of some cobbles and boulders which exist within this stratum. Standard penetration resistances or "N" values obtained within this deposit ranged from 32 to 56 blows/ft. indicating a dense state of packing.

Sandy Silt Till

The silty sand till layer is underlain by a glacial till deposit which consists of sand with some silt, gravel, and clay and which is at least 5 ft. thick at the location of borehole 1. A mechanical analysis test was carried out on a sample of this glacial till also obtained from a 1½ inch I.D. spoon sampler. The results of this test are shown on Fig. 4 and indicate the well graded nature of this material.

The standard penetration resistances or "N" values obtained within this deposit were greater than 100 blows/ft. indicating a very dense state.

GROUNDWATER CONDITIONS

A piezometer was installed in borehole 2 and a standpipe was installed at depth in boreholes 1 and 2 to determine the groundwater conditions across the site at the time of the investigation. Details of these installations are given on the Record of Borehole sheets together with water levels obtained on January 20, 1972. The piezometer was sealed into the silty clay stratum. The water level within this stratum was found to be from 3.5 to 4.5 feet below present ground surface. In boreholes 1 and 2, a standpipe was sealed into the glacial till deposits at depth. The groundwater level within the till material was found to be 85.5 feet below present ground surface in borehole 1. In borehole 2, the standpipe became blocked during installation and reliable readings could not be obtained. The two results indicate that the clay stratum is subjected to considerable downward drainage. Groundwater readings by the National Research Council⁽²⁾ indicate a lowering of both the water level within the clay and the underlying till with time. The current readings agree in general with those obtained by the National Research Council.

PROPOSED DEVELOPMENT

General

It is understood that the proposed development for the western portion of the site is to consist of high rise apartment structures, medium density row housing, single family units, and a school. Municipal services are to be installed on both the eastern and western portions of the site. The eastern portion of the site, in addition to including a mixed residential development, will also include a proposed shopping centre. Foundations for major structures with the two proposed developments will be founded within the thick clay stratum which exists generally at the site, or on piles driven to the till deposit.

Slope Stability

An inspection of the natural escarpment which crosses the northern edge of the property indicates that the clay banks west of the old slide are generally about 65 feet high with slopes of about 20° . A large slide took place along a portion of this natural escarpment about 1,000 years ago.

Profiles of the natural slope and the scarp of the old slide are included on the inferred stratigraphic section on Fig. 1. Boreholes 1 and 2 put down some 150 to 250 feet behind the crest of the old slide indicate that stiff highly plastic clay exists from ground surface to below

the base of the clay bank.

Groundwater observations have been made by the National Research Council and during this investigation to determine the water levels that exist within the natural clay bank at this site. The results of the observations indicate that fairly high groundwater levels exist within the clay slope at the present time and also that downward drainage to the underlying till is taking place.

In order to determine the safety of the existing slopes, an effective stress stability analysis was carried out for section A-A (see Fig. 1) using the results of the consolidated drained triaxial tests and the high groundwater levels reported for the springs of 1970 and 1971. For these conditions, the factor of safety against slope movement is of the order of 1.4.

The development should be planned to maintain this adequate stability condition. The stability analysis indicates that the safety is sensitive to changes in groundwater conditions and that an increase in seepage pressures (or blockage of the present downward seepage) could lead to failure. The design and construction of the water lines and sewers should be such as to avoid any chance of leakage. Also the slopes should remain undeveloped park land and restrictions placed on any filling near the crest of the slope or excavation at the toe of the slope. The existing

foliage on the clay banks should be maintained.

Site Grading

- When an area is developed, the usual practice is to use the soil recovered from general excavation as fill over the site, resulting in an overall grade raise of about 2 feet. To prevent a reduction in the existing slope stability it is recommended that the present ground contours not be increased west of the old slide area within 350 feet of the clay bank. This covers the school site and much of the Ludgate Court portion of the subdivision. The existing grade may be lowered or remain the same within these limits.
- (1) Excavated material within 350 feet of the crest should be trucked away. Minor grade raises may be tolerated beyond this 350 foot setback.

- Any increase in loading (by filling or construction of buildings) on or near the crest of the present slope reduces the overall stability. It is therefore recommended
- (2) that the proposed school structure be located as close to the southwest corner of the allotted property as possible. Also, the present grade on the school property should not be
- (3) increased. Storm water and snow melt run-off from the paved school yard and parking areas should be collected by adequate
- (4) underdrainage into the proposed storm sewer system and
- (5) should not be allowed to seep uncontrolled towards the slope.
- (6)

It is recommended that no filling be permitted on

(7) the portion of the property designated as parkland and that no excavations near the toe of the slope in these areas be carried out. The foliage which presently covers the property designated as parkland should remain unchanged. The presence of vegetation on the slopes increases the overall stability and reduces the possibility of surficial erosion.

(8) A commercial area in this development has been designated for a 9 acre parcel of land which fronts on Ogilvie Road and La Verendrye Drive to the north. It is understood that a shopping centre is planned for this property and that the structures will probably be built on the western and southern portions of the property. The present ground contours indicate that the land rises to the south and that the southern boundary of the commercial area parallels the toe of an intermediate slope within the crater of the old slide. It is recommended that development of this commercial area does not include any excavation of material at or cutting into the toe of the slope along the western boundary and the western portion of the south boundary of the commercial property. Removal of material at the toe of this slope and addition of a retaining wall, for example, would reduce the overall stability.

(9) The slope at the rear of the proposed commercial property should be left with natural vegetation, and where

- (10) minor grading is necessary, should be constructed not steeper than 2.5 (horizontal) to 1 (vertical).

Installation of Services

i) Sewers and Waterlines

- (11) It is recommended that sewers and waterlines for this site should be watertight. Leakage from these sources would raise the water level in the area significantly and reduce the stability of the escarpment in the northern portion of this site.

It is understood that to guard against leakage, plastic pipe, in lengths of 40 to 50 feet, will be utilized for waterlines and for storm and sanitary sewers. The joints of this pipe would be field welded and testing would be carried out to insure the leakproof nature of these joints. Granular fill would be placed in the common excavation to above the level of the highest pipe. The presence of this granular material along a graded sewer pipe would act to some extent as a French drain and would aid in lowering the water level in the clay.

Excavations up to about 15 foot depth may be carried out in open cut with the men working within a protective box. The water inflow should be minor and should be readily handled by pumping from sumps. To avoid importing large quantities of granular fill for backfilling of trenches, consideration could be given to the use of the drier grey

brown clay for general backfill from above the pipe to subgrade level.

ii) Roads

It is understood that the roads will have curbs and gutters, connected to the storm sewer system, to remove water run-off. In general, the road grades should be chosen to maintain or improve the stability of the slopes. Roads near the crest of the slope should be in cut or at grade. Roads near the toe of the slope should be in fill or at grade.

Foundations of Structures

i) High Rise Structures

Due to the shear strength of the deep clay deposit at this site, support of high rise structures (8 to 10 storeys) cannot be accomplished with spread footings. Alternative foundation solutions are outlined below.

High rise structures could be supported on end bearing piles driven into the very dense glacial till that exists at a depth of about 90 to 100 feet below present ground surface. Due to the great depth to the till deposit, considerable expense would be involved for this foundation solution. It is recommended that a displacement pile, either pipe or precast concrete, be employed to obtain satisfactory resistance in the till stratum. In order to minimize disturbance of the sensitive clay deposit during driving of piles, it is recommended that the clay be pre-augered to below the

can it be argued?
Does it not liquefy?

elevation of the toe of the slope.

Eight to ten storey structures could also be supported on a raft foundation. Construction of a raft or mat foundation entails excavation of material to basement level and pouring a mat of concrete some 4 feet thick in the bottom of this excavation. This foundation type obtains the full bouyancy effect due to removal of the overburden weight. For a large foundation mat placed above a deep compressible clay deposit, the allowable bearing pressure is dependent upon settlement as well as upon bearing capacity considerations. From other investigations of the high rise site on the east side of Ogilvie Road, it is known that the clay is preconsolidated by about 1.3 tons/sq.ft. above existing overburden pressure. Loading of the clay deposit to stresses below the preconsolidation pressure range results in settlements occurring from recompression of the clay and settlements for this design will be relatively small. Past experience indicates that net increases of loadings imposed by rafts of less than one half of the past preconsolidation of the clay results in tolerable settlements. It is recommended in this case that the net stress increase (contact pressure less weight of overburden removed) be limited to 800 lb/sq.ft. for preliminary design purposes. In calculating the weight of soil removed from the excavation, the unit weight of the clay may be taken as 100 lb/cu.ft. In computing the load of the

structure (superstructure and foundation) which will contribute to the above net bearing pressure, only dead loads and the average sustained live loads need be considered, since transient live loads will not contribute to settlement. High rise structures on raft foundations should be set back at least 150 feet from the crest of the slope.

The total settlement at the centre of a rigid mat under this recommended loading is estimated to be of the order of 1 inch. The differential settlement between the centre of the rigid mat and the corner of the mat is expected to be of the order of 0.5 inches.

Water inflow into the foundation excavations within the clay will be minor and can be readily handled by pumping from sumps. The clay is sensitive and can be readily disturbed by construction traffic and by ponded water. It is recommended that a mud mat of lean concrete be poured on this clay as soon as each section of the foundation excavation is down to grade.

Two and Three Storey Residential Structures

Two to three storey single and multi-family structures could be founded on spread footings at this site. Such footings would be placed within the weathered crust above the relatively soft grey clay. This serves to spread the load of the footings through the very stiff grey brown clay and onto the relatively soft grey clay surface. The

allowable bearing pressure for spread footings founded,
for adequate frost protection, at about 5 foot depth may
be taken as 3,000 lb/sq.ft. It is recommended that these
residential structures be set back from the crest of the
slope by at least 75 feet.

GSW/FJH/ml
71822
February, 1972.

G. S. Webb

G. S. Webb

F. J. Heffernan

F. J. Heffernan, P. Eng.



REFERENCES

- (1) CRAWFORD, C.B. and EDEN, W.J., "Stability of Natural Slopes in Sensitive Clay", ASCE Soil Mechanics and Foundations Division, Vol. 93, No. SM4, July, 1967.
- (2) JARRETT, P.M. and EDEN, W.J., "Groundwater Flow in Eastern Ottawa, Canadian Geotechnical Journal, Vol. VII, No. 3, August, 1970.

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) 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 = $c_v t / d^2$ (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_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.

OVERSIZE DRAWING

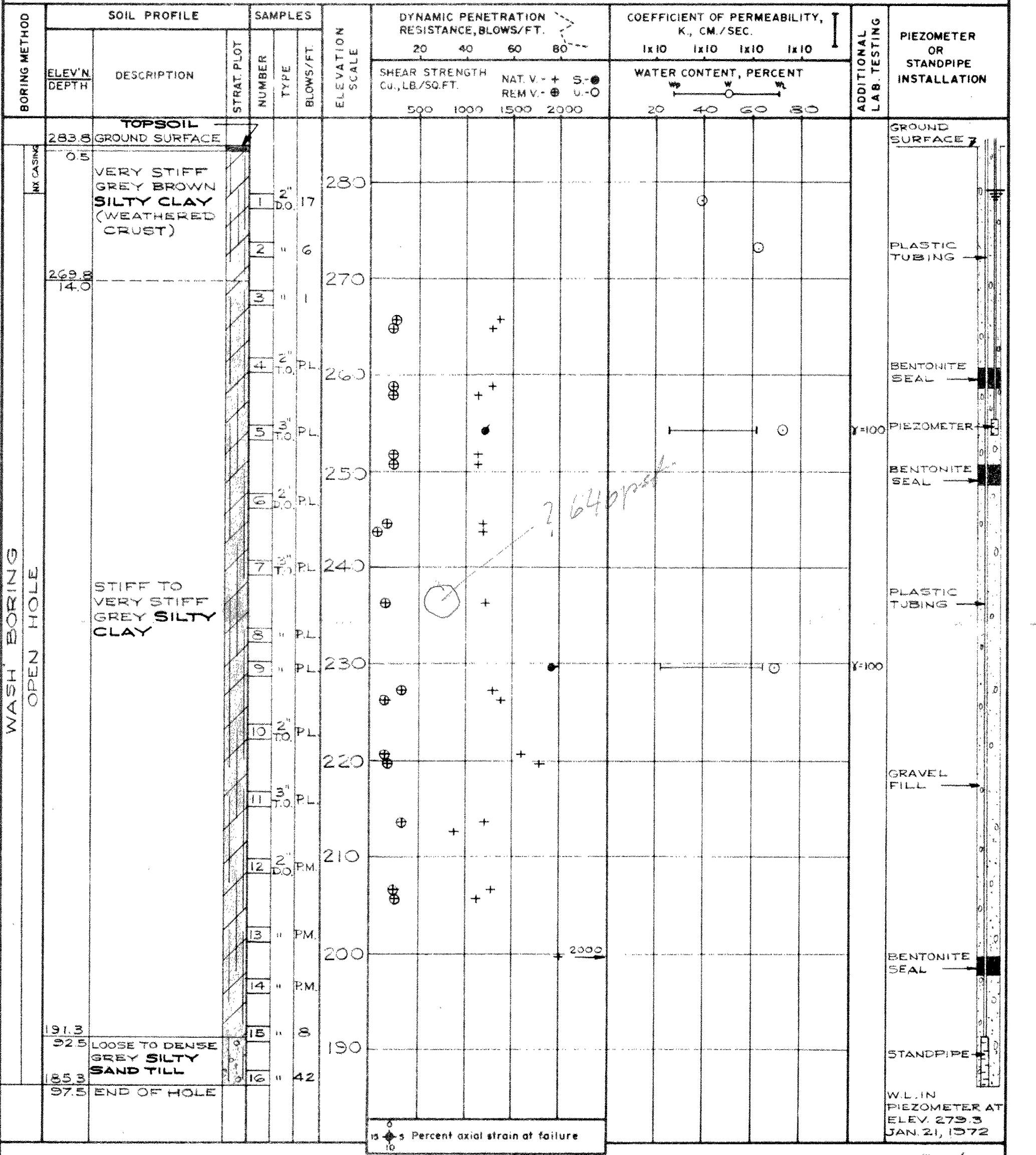
RECORD OF BOREHOLE 2

LOCATION See Figure 1

BORING DATE DECEMBER 23-28, 1971 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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



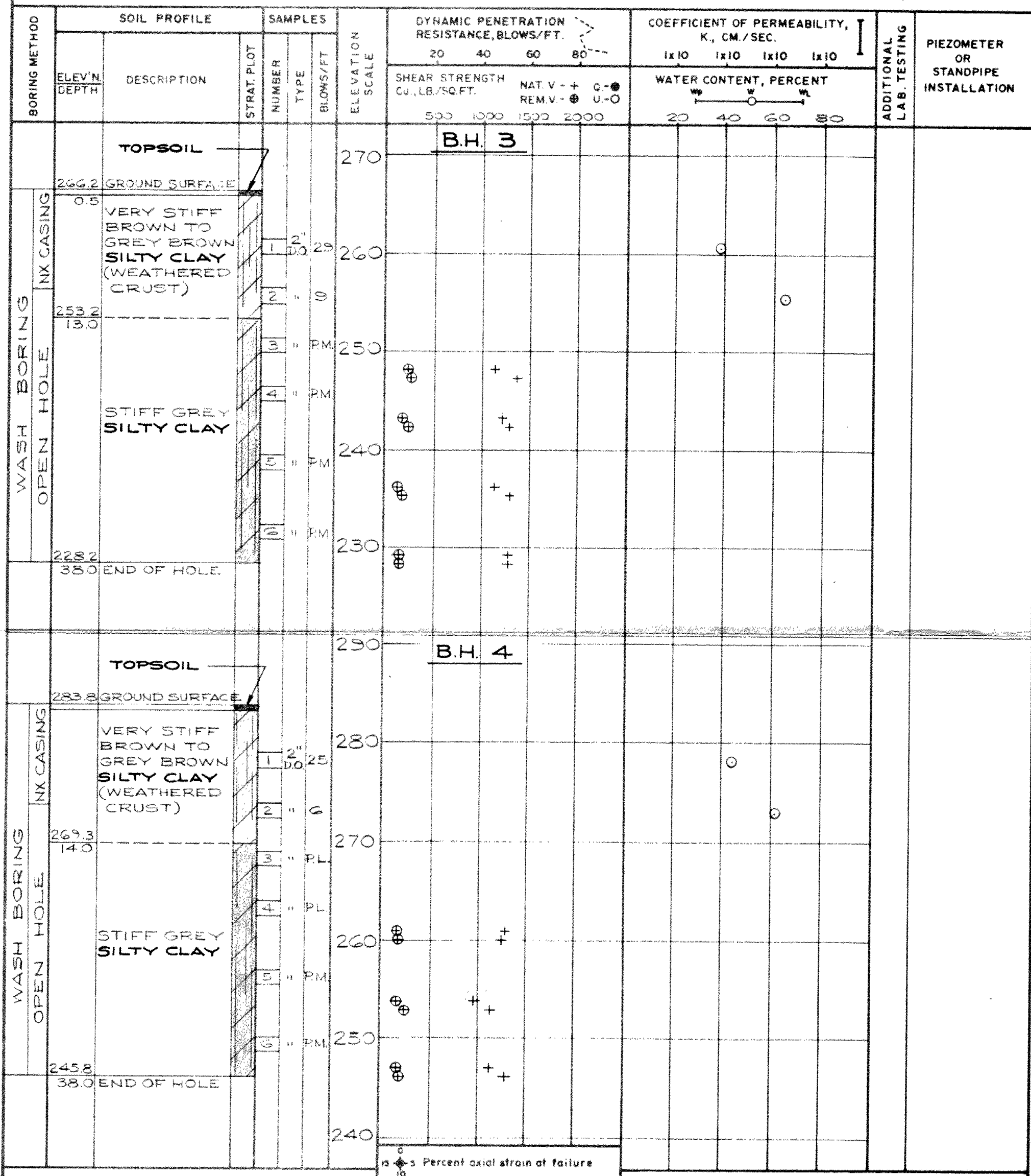
RECORD OF BOREHOLE 3 & 4

LOCATION See Figure

BORING DATE DECEMBER 28-30, 1971 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.N.
CHECKED F.J.H.

OVERSIZE DRAWING

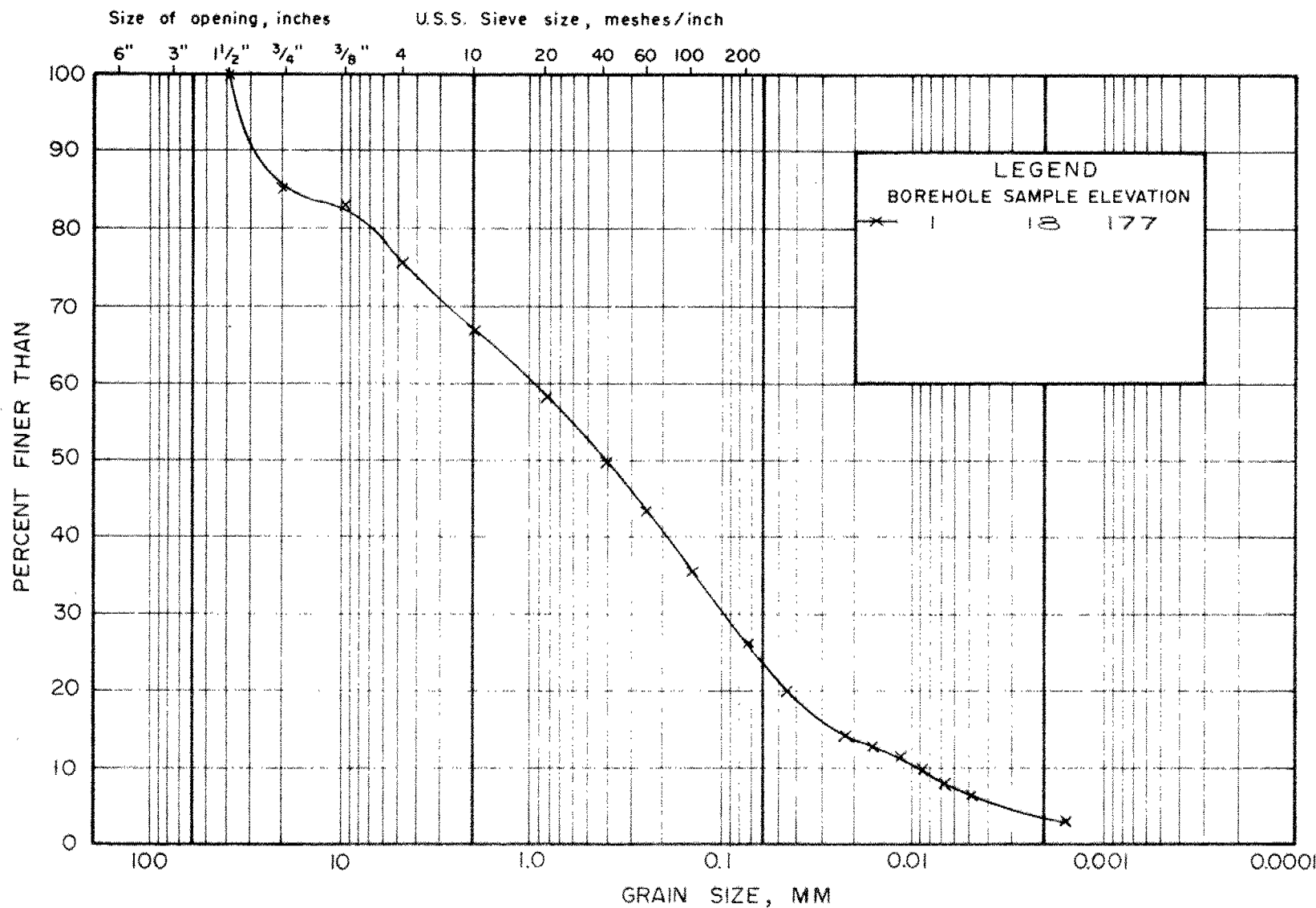
M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION

SILTY SAND/TILL

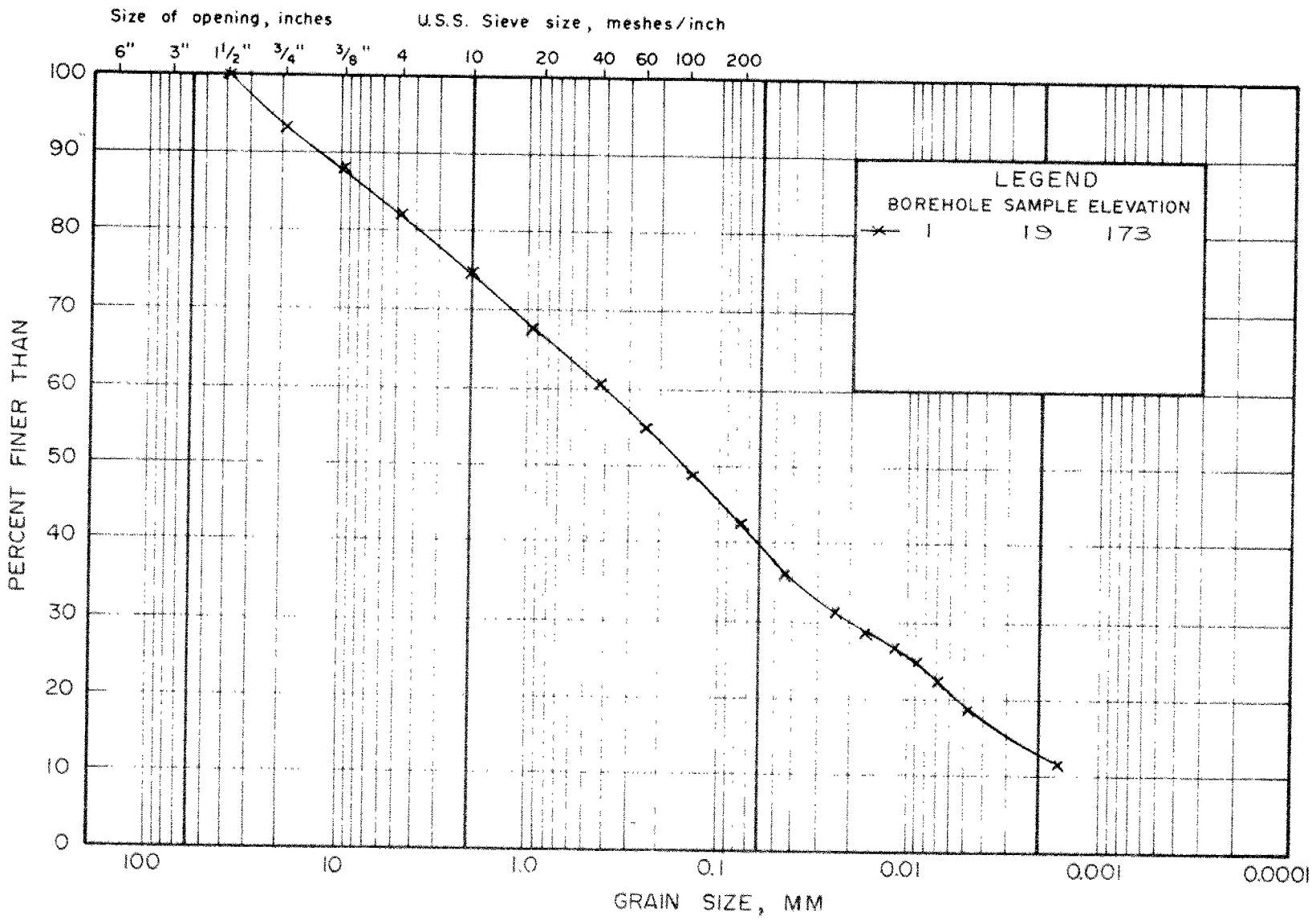
FIGURE 2

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

M.I.T. GRAIN SIZE SCALE



OVERSIZE DRAWING

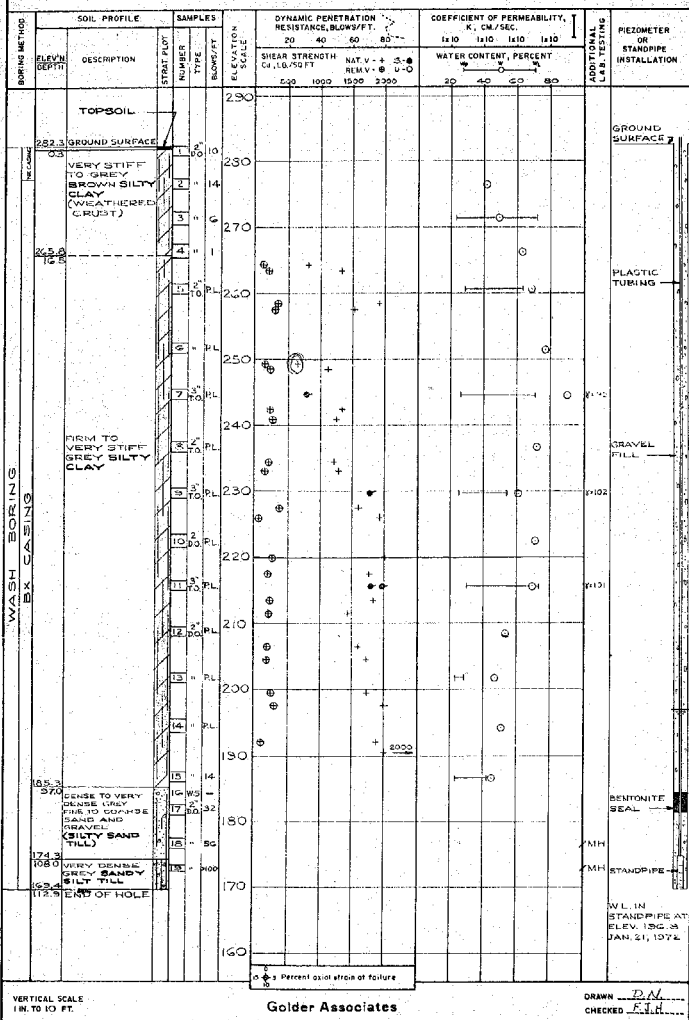
RECORD OF BOREHOLE 1

LOCATION See Figure 1

BORING DATE: DECEMBER 17-22, 1972 DATUM: GEODETIC

SAMPLER: HAMMER WEIGHT 140 LB., DROP 30 IN.

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

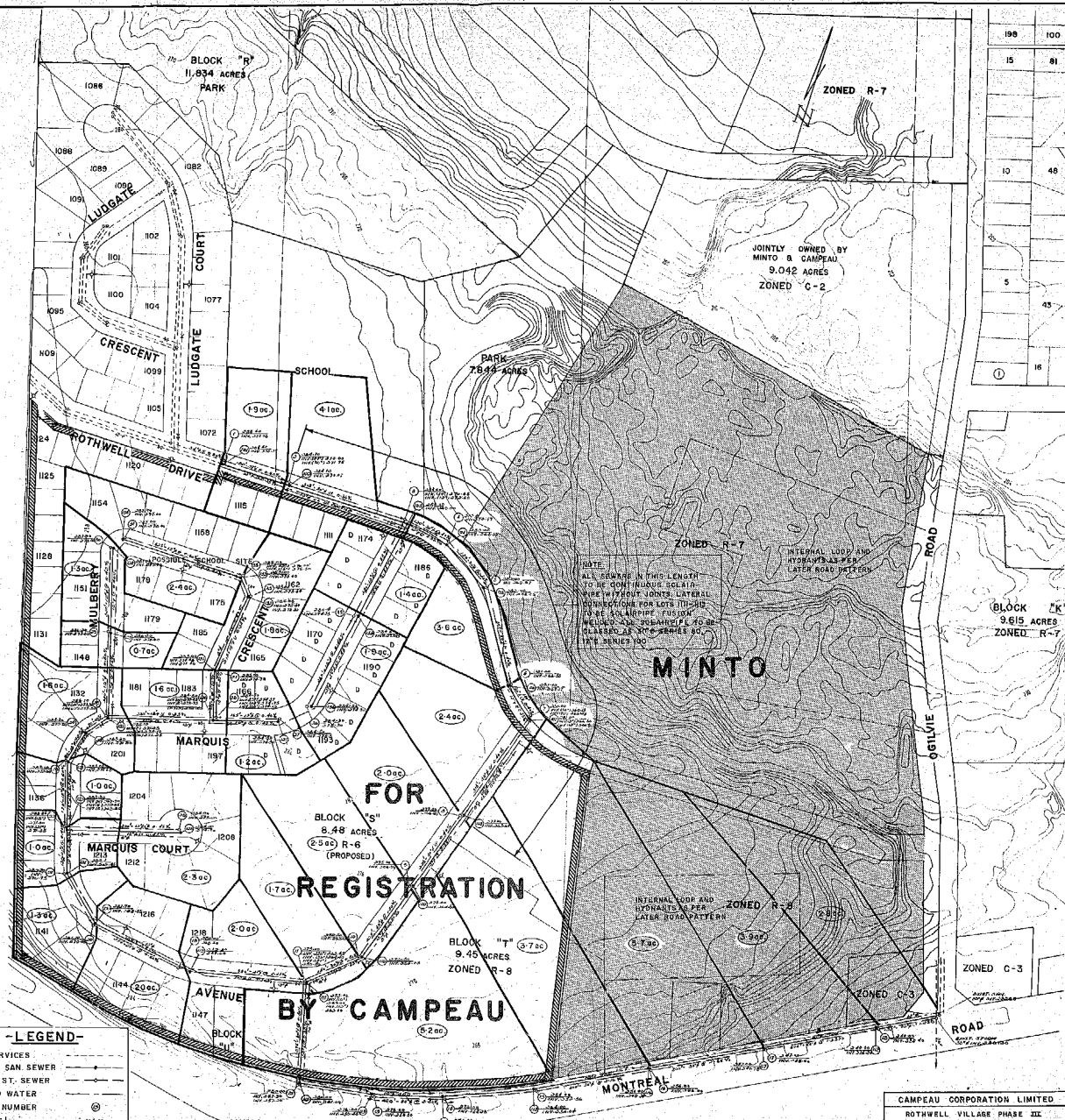


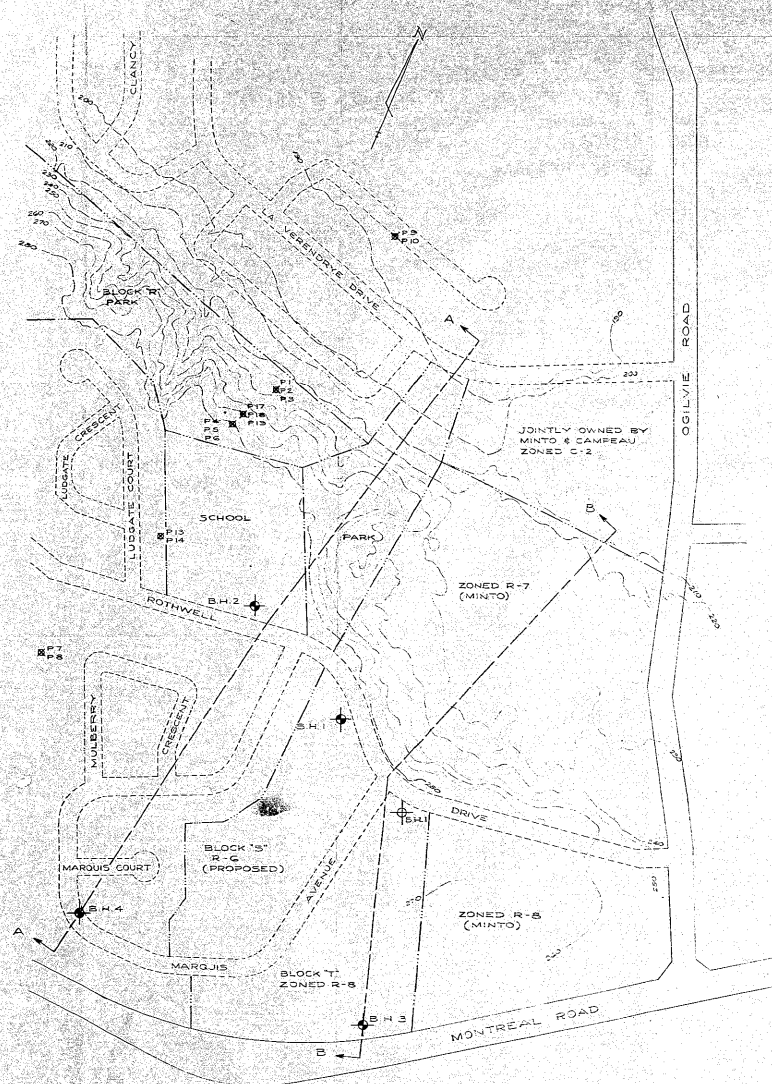
-LEGEND-	
EXIST. SERVICES	---
PROPOSED SAN SEWER	---
PROPOSED ST. SEWER	---
PROPOSED WATER	---
MANHOLE NUMBER	⊙
GROUND E.L.	⊕
INVERT	⊖
DRAINAGE AREA	(Area)

CAMPEAU HOLDINGS
 D (DUAL) - 19 x 7 x 13 People
 S (SINGLE) - 9 x 6 x 31 People
 R-6 (63/Acre) - 190 x 63 x 693 People
 R-5 (100/Acre) - 8 x 5 x 100-180 People
 TOTAL POPULATION 4297 People
 ALL LOTS NOT MARKED-D ARE SINGLES

MINTO HOLDINGS
 R-7 - 1200 People
 R-8 - 2200 People
 TOTAL POPULATION 4240 People

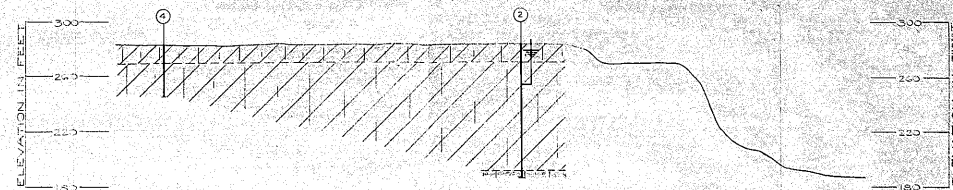
CAMPEAU CORPORATION LIMITED			
ROTHWELL VILLAGE PHASE III			
PRELIMINARY			
SANITARY, STORM & WATER SYSTEMS			
J.L. RICHARDS & ASSOCIATES, LTD.			
CONSULTING ENGINEERS & PLANNERS			
UTTERA WILSON	SCALE	APPRO. JAN. 1971	FILE NO.
DATE	SCALE	APPRO. JAN. 1971	FILE NO.
MAY, 1971	1" = 100'	SCALE 5/16"	20-17471





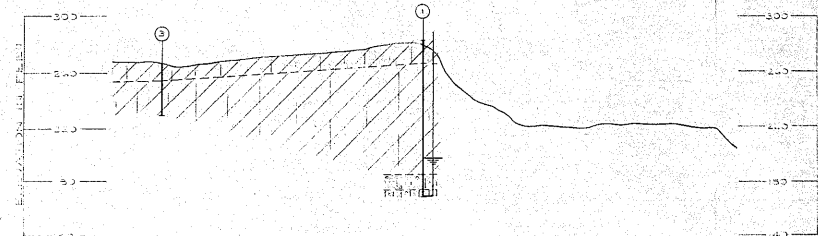
PLAN

SCALE 1" TO 200'



SECTION A-A

SCALE: HOR. 1" TO 200'
VER. 1" TO 40'



SECTION E-E

SCALE: HOR. 1" TO 200'
VER. 1" TO 40'

STRATIGRAPHY

- VERY STIFF BROWN TO GREY BROWN SILTY CLAY (WEATHERED CRUST)
- FIRM TO VERY STIFF GREY SILTY CLAY
- LOOSE TO VERY DENSE GREY FINE TO COARSE SAND AND GRAVEL (SILTY SAND TILL)
- VERY DENSE GREY SANDY SILT TILL

LEGEND

- BOREHOLE IN PLAN BY H.J. GOLDER AND ASSOCIATES
- BOREHOLE IN PLAN BY NATIONAL RESEARCH COUNCIL
- PIEZOMETER BOREHOLE AND PIEZOMETER NUMBER IN PLAN BY NATIONAL RESEARCH COUNCIL
- BOREHOLE IN ELEVATION BY H.J. GOLDER AND ASSOCIATES
- WATER LEVEL IN ELEVATION JAN. 1972
- BOUNDARY LINE BETWEEN ZONING AREAS

REFERENCE: SITE DEVELOPMENT PLAN OF ROTHWELL VILLAGE, PHASE III, SUPPLIED BY J. R. RICHMOND AND ASSOCIATES LTD.

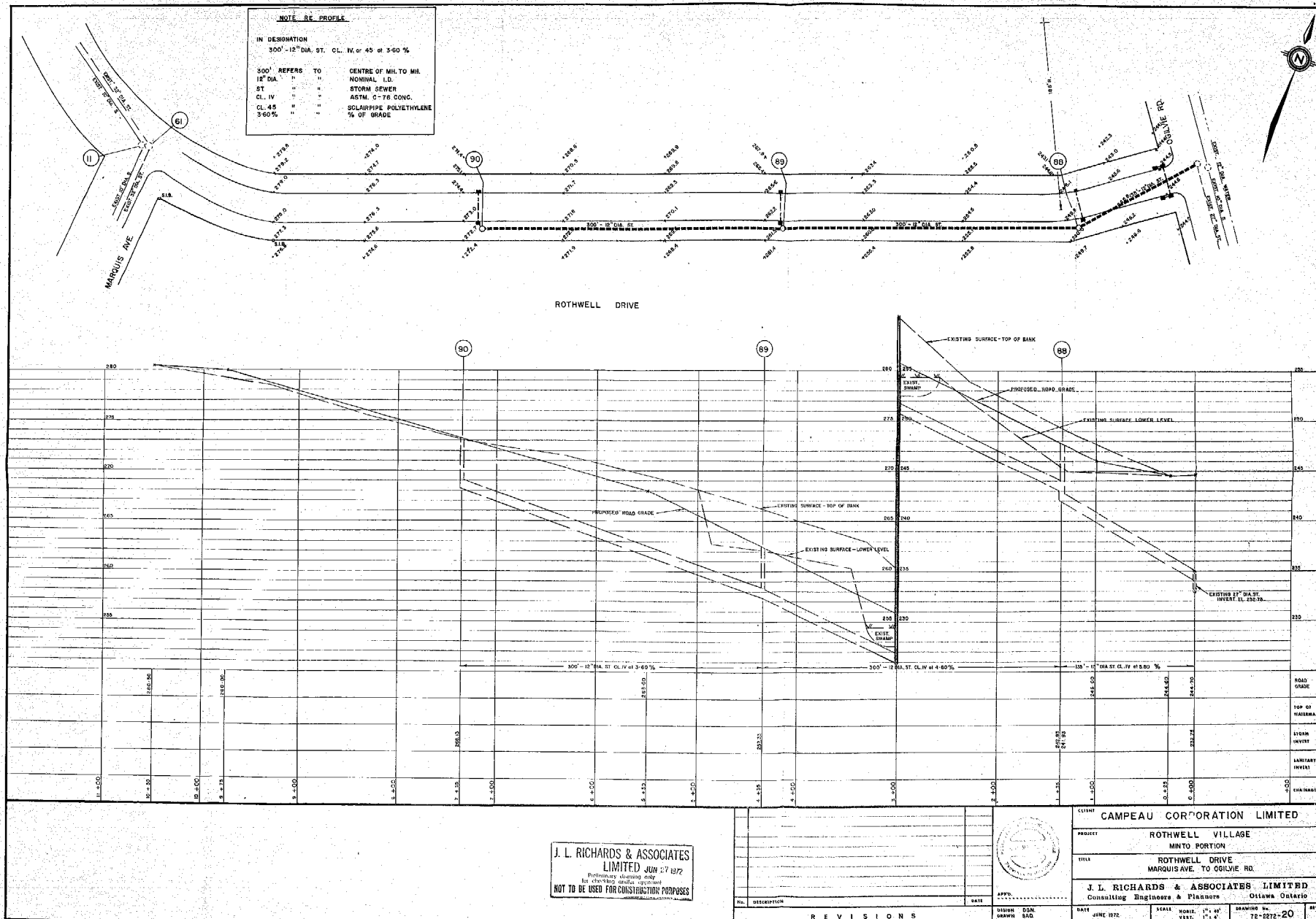
SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

NOTE
Data concerning the various charts have been obtained at various intervals and the soil stratigraphy between the boreholes has been inferred from geological maps and is not true. For this reason, the accuracy of the charts is not guaranteed.

Date: 12/1/72

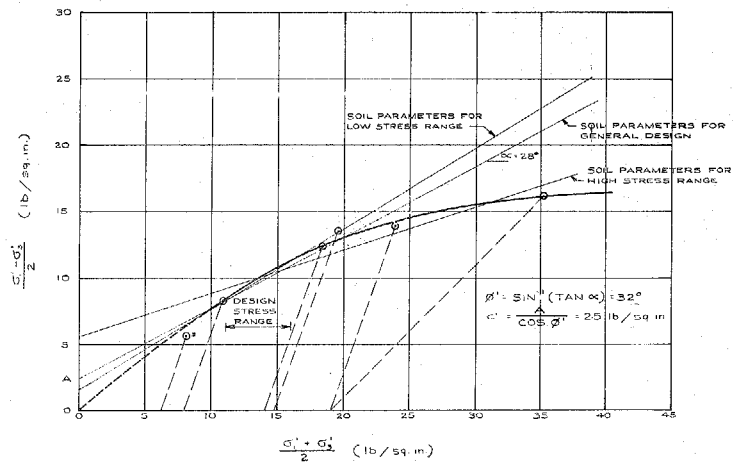
Golder Associates

Drawn: J.H.
Check: J.H.
Appr.: J.H.



DRAINED TRIAXIAL RESULTS

FIGURE 4



* POSSIBLE SAMPLE DISTURBANCE

Date FEB. 23, 1972

Golder Associates

 Drawn *D.M.*
 Chkd *SSW*
 Appd *JJS*