

GEOCRES No. 4053-7DIST. I REGION W.P. No. CONT. No. W. O. No. 73-F-216-MSTR. SITE No. 6-60HWY. No. Matchette RdLOCATION TURKEY CREEKNO OF PAGES -OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

73#216 M.

GEOCRES No
4053-7

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GEOCRES No.

40J3-7

GEOCRES No.

REPORT TO DIST 1
LAFONTAINE, COWIE, BURATTO
& ASSOCIATES
SUBSURFACE INVESTIGATION
PROPOSED BRIDGE OVER TURKEY CREEK
MATCHETTE ROAD
TOWNSHIP OF SANDWICH WEST, ONTARIO

STRUCTURE SITE No. 6-60

Distribution:

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Windsor, Ontario.

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Windsor, Ontario.

February, 1973.

73514

Golder Associates



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

February 13, 1973

LaFontaine, Cowie, Buratto & Associates,
Consulting Engineers,
76 University Avenue West,
WINDSOR 12, Ontario.

ATTENTION: Mr. E. O. LaFontaine, P. Eng.

RE: Subsurface Investigation
Proposed Bridge Over Turkey Creek
Matchette Road, Township of
Sandwich West, Ontario

Dear Sirs:

This letter reports the results of a subsurface investigation carried out at the site of a proposed new bridge over Turkey Creek on Matchette Road, in the Township of Sandwich West, County of Essex, Ontario. The purpose of the investigation was to determine the soil and groundwater conditions at the site, and based on this information, to provide recommendations for the foundation design of the proposed structure.

PROCEDURE

The field work for the investigation was carried out from February 5 to 7, 1973 and consisted of two boreholes put down at the locations shown on the site

plan, Figure 1. The borings were advanced to a depth of 85.5 feet below ground surface using a truck mounted power auger supplied and operated by Master Soil Investigation Ltd. The field work was supervised throughout by a member of our engineering staff.

Representative soil samples were obtained at regular 3 to 5 foot intervals of depth in both boreholes, down to 50 feet below ground surface. Standard penetration resistance tests were carried out during sampling operations using conventional 2 inch O.D. split spoon sampling equipment. A limited number of relatively undisturbed samples of the clay deposits encountered in the borings were obtained using 1-7/8 inch and 2-7/8 inch I.D. thin wall Shelby tubes. In situ vane tests were carried out at regular intervals of depth in cohesive strata in order to measure the undrained shear strength of the material. Occasional in situ vane tests and standard penetration resistance tests were carried out, and augered samples were obtained, in the boreholes below 50 foot depths.

The groundwater level in the open boreholes was observed during the drilling operations. After the completion of each borehole, a perforated standpipe was installed in order that the stabilized water level could be measured. The latest water level reading in the standpipes, taken one week following the completion of the drilling, together with the water level observations during the field work, are given on the Record of Borehole sheets.

All samples were brought to our Windsor laboratory for detailed examination and representative testing. Water content determinations were carried out on all samples together with Atterberg limit tests and grain size analyses on selected samples to confirm visual field classification of the various soil strata. The results of the field and laboratory testing are given on the Record of Borehole sheets and on Figure 2.

The locations and ground surface elevations of the boreholes were established in the field by Golder Associates. The ground surface elevation at the boreholes are referred to the top of the northwest bolt of a fire hydrant located on the east side of Matchette Road some 100 feet south of the existing bridge. The elevation of the benchmark, as given on the drawing entitled "Site Plan", sheet 1, Drawing No. LA1304, dated April 21, 1972, supplied by LaFontaine, Cowie, Buratto & Associates is 579.36. It is assumed that the benchmark elevation is referred to geodetic datum.

SITE AND GEOLOGY

The site of the proposed new bridge over Turkey Creek is located about 1000 feet south of Morton Drive on Matchette Road in the Township of Sandwich West, County of Essex, Ontario.

The existing bridge is a wooden single 45 foot span structure supported by wooden piles. Remedial works to repair flood damage to the bridge has recently been completed.

The new bridge will be a concrete structure with two spans each some 40 feet in length, and will follow the approximate alignment of the existing roadway. The proposed new grade for the roadway will involve the placement of up to a 4 foot depth of additional fill along the length of the approach embankment.

The site of the proposed structure, which will traverse the narrow flood plain of Turkey Creek, is located in the physiographic region of Southwestern Ontario known as the St. Clair Clay Plain. The clay plain was laid down during the retreat of the Wisconsin ice sheet during the latest Pleistocene era when a series of glacial lakes inundated the area. The major clay stratum, which forms the main soil deposit in the area, is considered to have a till-like structure exemplified by a random distribution of coarser particles throughout, and extends down to a thin basal sandy till stratum overlying limestone bedrock.

SOIL AND GROUNDWATER CONDITIONS

The detailed soil stratigraphy encountered in the boreholes together with the results of the laboratory testing are presented on the Record of Borehole sheets and on Figure 2. An inferred soil stratigraphic section along the centreline of the proposed bridge is given on Figure 1. A summarized account of the subsurface conditions encountered at the site is given below.

Generally, the soil conditions encountered at the borings comprise crushed stone base course, silty clay fill, and topsoil overlying a major stratum of firm to stiff silty clay extending down to a thin stratum of silty sand basal till at a depth of 81 feet. Based on the

practical refusal of the augers and standard penetration test results, it is assumed that the boreholes were terminated at the limestone bedrock surface.

The borings encountered some 8 to 9 inches of crushed stone roadway base course underlain by brown silty clay fill with organic matter extending to a depth of 5 feet and overlying an 18 inch thick stratum of black clayey topsoil which is believed to be at about the original ground surface prior to the construction of the approach embankment to the existing bridge.

Beneath the base course material, embankment fill, and topsoil, the borings encountered a major stratum of silty clay extending to a depth of about 81 feet below ground level. The upper 5 to 7 feet of the stratum consisted of brown silty clay with some sand and a trace of gravel. Two standard penetration resistance tests carried out during sampling operations in this material gave N values of 7 blows per foot indicating a generally stiff consistency.

The brown silty clay is underlain at a depth of about 13 feet by grey till-like silty clay. Typical grading curves for this material are given on Figure 2. N values within the upper 10 to 16 feet of this stratum range from 4 to 16 blows per foot. Shear strength values as measured by in situ vane tests range from about 1000 to 1600 pounds per square foot. Based on N values and vane test results the consistency of the upper portion of the grey silty clay is stiff.

Below a depth ranging from 23 to 29 feet below ground surface and extending down to elevation 546 (some

34 feet below ground level) the silty clay is only firm in consistency. N values range from 2 to 4 blows per foot and vane test results of about 900 pounds per square foot were measured.

Between elevation 546 and 499, the grey till-like silty clay possesses a stiff consistency with N values ranging from 7 to 11 blows per foot and undrained shear strengths ranging from about 1000 to 1500 pounds per square foot were measured.

The grey till-like silty clay extended to a depth of 81 feet below ground level at both boreholes and was underlain by a 4.5 foot thick stratum of grey silty sand with some gravel (basal till). Based on the practical refusal of the split spoon sampler (90 blows per 1/2 inch penetration) and grinding and refusal to advance of the augers, it is believed that bedrock was encountered at a depth of 85.5 feet below ground level (elevation 494.5).

The results of three Atterberg limit tests carried out on the grey till-like silty clay gave liquid limits ranging from 29 to 37 with an average plasticity index of 16. Natural water contents within the clay stratum generally range from 20 to 34 per cent, between the plastic and liquid limits. Liquidity indices for the grey silty clay range from 1.0 to 0.3 and these indicate that this material is generally normally consolidated to slightly over consolidated.

The water levels in the open boreholes during drilling ranged from 5 to 8 feet below ground level or some 1 to 4 feet below the water level in Turkey Creek at the time of the investigation (elevation 576). It is believed that the groundwater level in the boreholes will eventually stabilize at about the same elevation as the water level in Turkey Creek.

DISCUSSION

It is understood that the proposed structure will consist of two, 40 foot long spans and that the total foundation loading at the abutments and central pier will be 828 and 1215 kips respectively.

Spread Footings

The feasibility of using spread footing foundations to support the proposed loadings has been investigated using the detailed soil data and shear strength characteristics of the major clay stratum encountered at the site.

The allowable bearing pressure beneath a 15 by 30 foot rectangular footing at about elevation 564 would be about 2 kips per square foot. The actual average bearing pressure beneath the spread footing supporting the 1215 kip central pier foundation load would be approximately 3 kips per square foot and would therefore result in overstressing of the subsoil.

Because of this the use of spread footings to support the bridge piers and abutments is not considered feasible and it is recommended that a pile foundation be used.

Piled Foundations

It is considered that either steel H-piles or concrete filled steel pipe piles could be used to provide the necessary foundation support for the proposed structure.

Steel pipe piles could be driven closed end into the very dense sandy basal till overlying bedrock and subsequently filled with concrete. This type of pile has a relatively high load capacity and has the additional advantage that after the completion of driving, all piles can be inspected visually to check for damage, plumbness and wandering of the pile tips. The use of a closed end pipe pile will permit subsequent placing of concrete "in the dry".

A 12-3/4 inch diameter pipe pile with a wall thickness of 0.281 inch (ASTM A252, Grade 2) with a 1 inch thick plate welded at the bottom, driven to a final set of at least 20 blows per inch with a hammer developing a minimum of 40,000 foot-pounds of energy per blow, and filled with 4000 pounds per square inch concrete can be used to carry a design load of 209 kips per pile under maximum loading. Any increase in the allowable load for the size of pile, grade of steel and concrete strength listed would be based on pile load test results according to the 1970 National Building Code of Canada. To prevent undue displacement of the subsoil, it is recommended that piles be driven in pre-augered holes approximately 1 inch smaller than the pile diameter.

Alternatively, steel H-piles may be used. A 10 BP 42 steel H-pile driven to practical refusal at a

set of at least 20 blows per inch with a hammer developing 15,000 foot-pounds of energy per blow, may be used to carry a design load of 130 kips per pile under maximum loading. This recommended load capacity is based upon the 1970 National Building Code of Canada, Table 4.2.5.A using a maximum allowable axial stress of 0.3 times the yield stress of the steel. The yield strength of the steel has been taken as 35 kips per square inch.

Based on the results of the two boreholes, it is anticipated that the specified set for both types of pile will be attained at approximately elevation 494.5.

It is considered that the proposed additional 1 to 4 feet of embankment fill will not result in significant settlement of the subsoil and consequently no allowance is required for the possible negative skin friction effect on the piles.

Provided that the recommended set is achieved, settlement of the piles under the proposed foundation loadings will largely be governed by the elastic deformations of the pile and pile tips. Such settlements will be elastic in nature and no long term consolidation settlement is expected.

Sulphate Attack on Concrete

The use of Type II cement is recommended for any buried portions of the concrete structure because of the positive, though mild, danger of attack on concrete which has been experienced in the Windsor area, due to the sulphate concentration in the groundwater.

We trust that this report provides sufficient information for your design purposes. If any point requires further clarification, please do not hesitate to contact our office.

Yours truly,

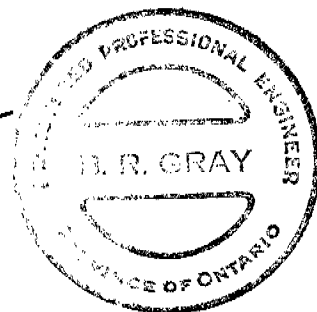
H. Q. GOLDER & ASSOCIATES LTD.,

J. F. Capps

J. F. Capps, P. Eng.

B. R. Gray

B. R. Gray, P. Eng.



BRG:cl
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LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

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 *Q* or *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

in terms of effective stress
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress
 $\tau_f = c_u + \sigma \tan \phi_u$

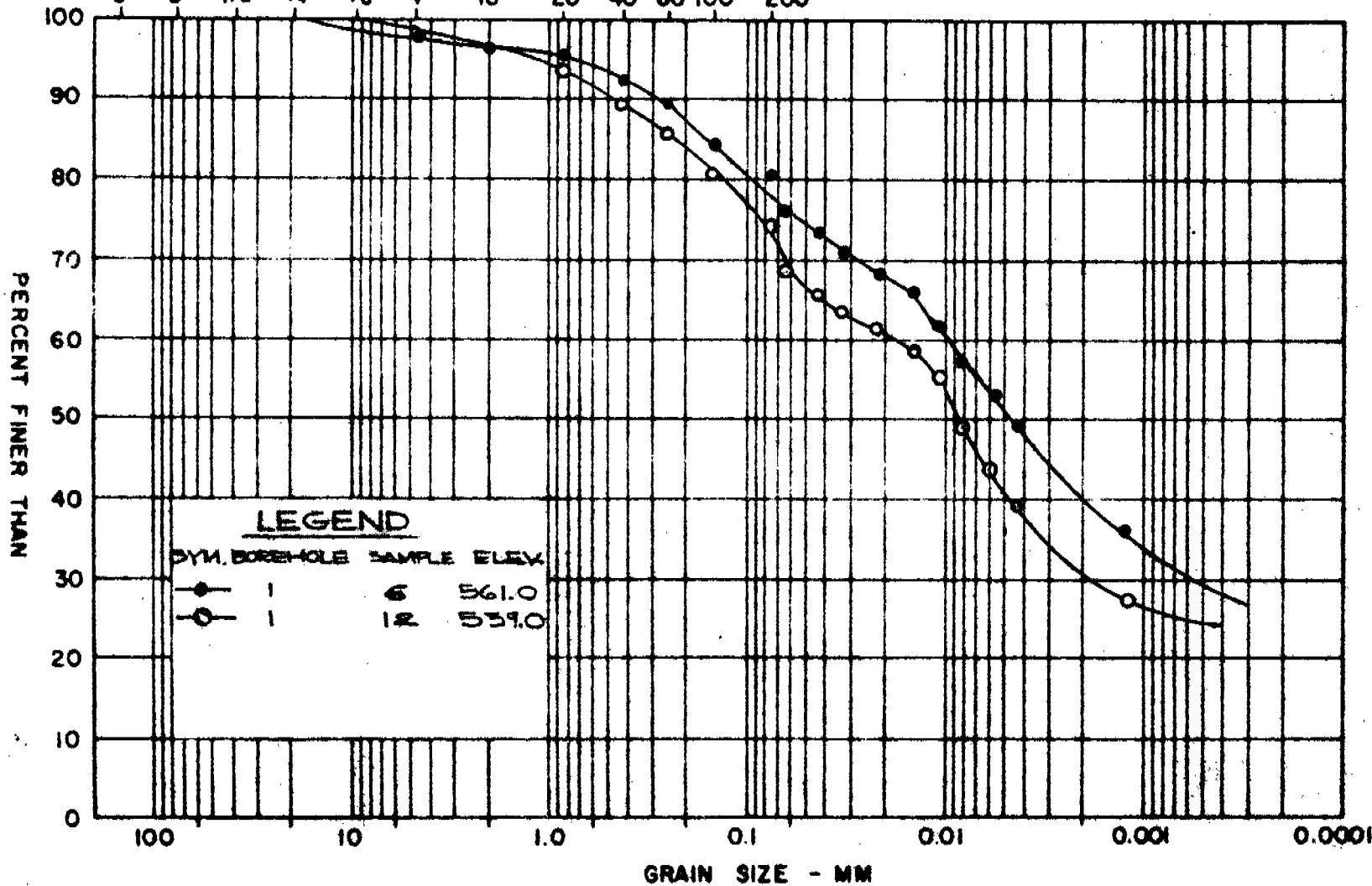
*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

OVERSIZE
DRAWING(S)

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200



GRAIN SIZE DISTRIBUTION
SILTY CLAY (TILL-LIKE)

FIGURE 2

RECORD OF BOREHOLE 1											
LOCATION See Figure 1			BORING DATE FEB. 5-6, 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		WATER CONTENT, PERCENT	PIEZOMETER OR STANDPIPE INSTALLATION
	DEPTH	DESCRIPTION	STRAT. LOT NUMBER	TYPE		20 40 60 80	NAT. V. - + C - REM V. - + U -	1x10 1x10 1x10 1x10	10 20 30 40		
POWER AUGER (UNCASED)	580.0	GROUND SURFACE			580						GROUND SURFACE
	578.0	CRUSHED STONE			578						SKY
	575.0	STIFF BROWN SILTY CLAY WITH ORGANIC MATTER	1	12	575						FEB. 6/73
	573.0	BLACK SILTY CLAY WITH ORGANIC MATTER	2	12	573						
	570.0	LOOSE BROWN SANDY CLAY WITH BLACK ORGANIC MATTER	3	4	570						
	567.0	STIFF BROWN SILTY CLAY SOME SAND, TR. GRAVEL, OCC. FRATTING FINE SAND	4	7	567						
	565.0		5	4	565						
	560.0	STIFF GREY SILTY CLAY SOME SAND, TR. GRAVEL (TILL-LIKE)	6	6	560						PIEZOMETER
	555.0		7	5	555						
	550.0	FIRM GREY SILTY CLAY SOME SAND, TR. GRAVEL (TILL-LIKE)	8	2	550						PLASTIC TUBING
	545.0		9	16	545						
	540.0		10	7	540						MH
	535.0		11	8	535						
	530.0	STIFF GREY SILTY CLAY SOME SAND, TR. GRAVEL (TILL-LIKE)	12	10	530						GRANULAR MATERIAL
	525.0		13	10	525						PERFORATED STANDPIPE
	520.0		14	10	520						
515.0		15	10	515						CAVED MATERIAL	
510.0		16	10	510							
505.0		17	10	505							
500.0	DENSE GREY SILTY SAND, SOME GRAVEL (BASAL TILL)	18	10	500							
495.0		19	10	495							
490.0	REFUSAL TO AUGER	20	10	490							
485.0	ASSUMED BEDROCK	21	10	485							
480.0		22	10	480							
475.0		23	10	475							
470.0		24	10	470							
465.0		25	10	465							
460.0		26	10	460							
455.0		27	10	455							
450.0		28	10	450							
445.0		29	10	445							
440.0		30	10	440							
435.0		31	10	435							
430.0		32	10	430							
425.0		33	10	425							
420.0		34	10	420							
415.0		35	10	415							
410.0		36	10	410							
405.0		37	10	405							
400.0		38	10	400							
395.0		39	10	395							
390.0		40	10	390							
385.0		41	10	385							
380.0		42	10	380							
375.0		43	10	375							
370.0		44	10	370							
365.0		45	10	365							
360.0		46	10	360							
355.0		47	10	355							
350.0		48	10	350							
345.0		49	10	345							
340.0		50	10	340							
335.0		51	10	335							
330.0		52	10	330							
325.0		53	10	325							
320.0		54	10	320							
315.0		55	10	315							
310.0		56	10	310							
305.0		57	10	305							
300.0		58	10	300							
295.0		59	10	295							
290.0		60	10	290							
285.0		61	10	285							
280.0		62	10	280							
275.0		63	10	275							
270.0		64	10	270							
265.0		65	10	265							
260.0		66	10	260							
255.0		67	10	255							
250.0		68	10	250							
245.0		69	10	245							
240.0		70	10	240							
235.0		71	10	235							
230.0		72	10	230							
225.0		73	10	225							
220.0		74	10	220							
215.0		75	10	215							
210.0		76	10	210							
205.0		77	10	205							
200.0		78	10	200							
195.0		79	10	195							
190.0		80	10	190							
185.0		81	10	185							
180.0		82	10	180							
175.0		83	10	175							
170.0		84	10	170							
165.0		85	10	165							
160.0		86	10	160							
155.0		87	10	155							
150.0		88	10	150							
145.0		89	10	145							
140.0		90	10	140							
135.0		91	10	135							
130.0		92	10	130							
125.0		93	10	125							
120.0		94	10	120							
115.0		95	10	115							
110.0		96	10	110							
105.0		97	10	105							
100.0		98	10	100							
95.0		99	10	95							
90.0		100	10	90							
85.0		101	10	85							
80.0		102	10	80							
75.0		103	10	75							
70.0		104	10	70							
65.0		105	10	65							
60.0		106	10	60							
55.0		107	10	55							
50.0		108	10	50							
45.0		109	10	45							
40.0		110	10	40							
35.0		111	10	35							
30.0		112	10	30							
25.0		113	10	25							
20.0		114	10	20							
15.0		115	10	15							
10.0		116	10	10							
5.0		117	10	5							
0.0		118	10	0							

VERTICAL SCALE
1 IN. TO 5 FT.

Golden Associates

DRAWN *ALP*
CHECKED *RRS*

RECORD OF BOREHOLE 2

LOCATION See Figure 1

BOIRING DATE FEB. 7, 1973

DATUM GEORETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

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

[illegible]

