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GEOCRES No. 31F-91

DIST. 9 REGION

W.P. No. 160-73-01/02

CONT. No.

W. O. No.

STR. SITE No. 29-19, 29-18

HWY. No. 62

LOCATION INDIAN RIVER BRIDGE

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

PRELIMINARY

FOUNDATION INVESTIGATION & DESIGN REPORT

SOIL MECHANICS SECTION

ENGINEERING SERVICES BRANCH
GEOTECHNICAL OFFICE

Ontario
Ministry of Transportation and Communications

PRELIMINARY
FOUNDATION INVESTIGATION & DESIGN REPORT

W.P. 160-73-01 DIST. 9 (Ottawa)

HWY. 62 STR. SITE 29-19

Indian River Bridge
1.6 Miles West of Alice

DISTRIBUTION

T.C. Kingsland (2)
R.S. Pillar
C.S. Grebski
B.J. Giroux
G.A. Wrong
S. Radbone
E.R. Saint
J.M. Childs

R. Hore

J. Anderson)
R. Forest) cover only
G. Sloan)

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DATE JUL 2 6 1976

INTRODUCTION

Grade revisions in the vicinity of the above mentioned structure is under consideration. If the existing bridge is utilized, the deck would have to be raised by some 3.5 feet. A subsurface investigation was undertaken to determine the feasibility of this proposal from the foundations aspect.

This report contains the results of the subsurface conditions carried out in the vicinity of the above mentioned structure and our recommendations pertaining to the allowable design load for the existing foundations, as well as the related approach embankments.

SITE DESCRIPTION

The site is located 1.6 miles west of Alice on Hwy. 62, Lots 5 & 6, Concs. 8 & 9, Township of Fraser, County of Renfrew.

The terrain of the area in the vicinity of the site is flat to gently rolling, and the surroundings is mainly brush covered with some tree growth. There is a gravel road just east of the structure leading to few cottages.

At the crossing, the Indian River is approximately 48 feet wide, and the water depth was found to be about 6 ft. The flow velocity of the river water is estimated to be 1 fps.

The existing structure is a 30 ft. single span standard rigid frame built in 1937 and appears to be in sound condition. From the available information (Dwgs. D2429-1, D2429-4 & correspondence), the existing structure is supported on 40' long timber piles, however, the details of the foundations as constructed are not documented.

FIELD AND LABORATORY INVESTIGATION

The field investigation consisted of two boreholes, each of which was accompanied by a dynamic cone penetration test. Boring was achieved by means of a bombardier mounted hollow-stem auger machine adapted for soil sampling purposes. Disturbed samples were obtained using a 2 inch. O.D.

split-spoon sampler driven in accordance to the specifications for the Standard Penetration Test. A couple of samples of the cohesive stratum were obtained in 3 inch. I.D. Shelby tubes which were hydraulically pushed into the soil. In situ vane tests were also carried out within this zone to determine the undrained shear strengths. Bedrock was not encountered within the depth of sampling carried out. The soil, and groundwater conditions encountered in the borings are presented on the Record of Borehole Sheets and on Dwg. 1.

At the time of report writing the E-plan was not available. Thus, we have prepared Dwg. 1 which shows the location of the boreholes in relation to the structure.

Laboratory tests were performed on selected samples to determine the following engineering properties:

Natural Moisture Contents
Grain Size Distributions
Atterberg Limits

The results of the laboratory testing are plotted on the Record of Borehole Sheets.

SUBSURFACE CONDITIONS

In general, the subsoil at this location consists of 15 to 25 ft. thick deposit of very loose to compact silty sand to sand, followed by 28 to 32 ft. thick layer of firm to stiff clayey silt to silty clay, which in turn is underlain by loose silty sand of at least 8 ft. in thickness. The borings were terminated within the lower granular deposit. Dynamic cone penetration tests were taken to depths of 100 ft. and bedrock was not encountered.

Fill material of 5 ft. in thickness was encountered in the borehole which was put down on the gravel roadway just east of the structure.

Fill Material

Fill material of about 5 ft. in thickness was encountered in the boring which was put down on the roadway. The upper portion of the fill material consists of sand, gravel and some cobbles, while the lower portion is composed of clayey silt with some sand.

Silty Sand to Sand

This deposit was encountered beneath a thin topsoil cover (6 inches) or below the fill material. The thickness of this granular material varies from 15 to 25 ft. Standard Penetration testing carried out within this material gave 'N' values ranging from 2 to 13 blows per foot which indicates that the relative density of this deposit is very loose to compact. Grain size distribution tests carried out on samples within this deposit are plotted on the Record of Borehole Sheets.

Clayey Silt to Silty Clay

This deposit which varies in thickness from 28 to 32 ft. was observed below the granular material. Within this layer, some pockets or seams of silt were also present. Standard Penetration testing within this deposit gave 'N' values ranging from 1 to 4 blows/ft. Based on the undrained shear strength values (Field Vane Tests), it is estimated that the consistency of this deposit varies from firm to stiff. The sensitivity, defined as the ratio of the undrained shear strength of the soil in an undisturbed state to that of the soil in a remoulded condition, as determined by in situ vane tests ranges from 7.3 to 22.7 (Avg. 12.3) which indicates that the cohesive stratum is highly sensitive. Due to the sensitive nature of this material, we believe that the 'N' values as obtained from the Standard Penetration Tests are not applicable in assessing the consistency of this cohesive deposit.

Typical Atterberg limits and Natural Moisture Contents are plotted on the Record of Borehole Sheets. The results indicate that the cohesive stratum is inorganic of low to intermediate plasticity and of high sensitivity with the corresponding natural moisture contents consistently higher than the liquid limit.

Silty Sand

This deposit was observed immediately below the cohesive stratum. The presence of a trace of gravel was evident in one of the borings. Both the boreholes were terminated within this deposit. Standard Penetration testing performed in this deposit gave 'N' values ranging from 7 to 10. Based on these results, it is estimated that the relative density of this material is loose.

GROUNDWATER CONDITIONS

Groundwater level observations were carried out in the open holes during the period of investigation and the results are shown on the Record of Borehole Sheets and on Dwg. 1. The results show that the groundwater level is similar to the creek water level which was found to be at elev. 466.5 at the time of the investigation.

DISCUSSION AND RECOMMENDATIONS

General

Grade revisions in this area would necessitate elevating the deck of the existing structure by some 3.5 feet. A subsurface investigation was carried out to determine the feasibility of this proposal for the structure from the foundations point of view.

In general, the subsoil at this location consists of 15 to 25 ft. thick deposit of very loose to compact silty sand to sand, followed by 28 to 32 ft. thick layer of firm to stiff clayey silt to silty clay, which in turn is underlain by loose silty sand. Bedrock was not encountered within the scope of the investigation carried out.

Our recommendations pertaining to the bearing capacity of the subsoil under the existing structure footings as well as the related approach embankments are presented below.

Structure Foundations

The existing structure is believed to be founded on 40 ft. long timber piles (available correspondence and Dwgs. D2429-1, D2429-4). The base of the pile cap is at about elev. 451 and the pile tips are estimated to be at elev. 411. However, the precise details of the foundations for this structure as constructed are not documented. The diameter of timber piles are not known. Assuming that the friction piles have an average diameter of 12 inches (No. 14 timber piles) and are 40 ft. long with the tip of the piles located at elev. 411, the allowable design load is 19 tons/pile.

Approach Fills

Stability problems are not anticipated for the proposed embankment height if constructed with 2 horizontal to 1 vertical slopes. The portion of the new embankment should be 'keyed' in the existing one in accordance with current MTC practices. Major settlement problems are not anticipated.

MISCELLANEOUS

The field work for this investigation was carried out during the period of June 11, 1976 to June 17, 1976 under the supervision of Mr. H. Shah, Project Engineer.

The equipment used for subsoil sampling was owned and operated by Atcost Drilling Company.

This report was written by Mr. H. Shah and was reviewed by Mr. B. Ly, Senior Engineer.

REGISTERED PROFESSIONAL ENGINEER
H. M. SHAH
PROVINCE OF ONTARIO

H. Shah
H. Shah, P. Eng.
Project Engineer

B. Ly
B. Ly, P. Eng.
Senior Engineer

BL/bp
July, 1976

APPENDIX

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

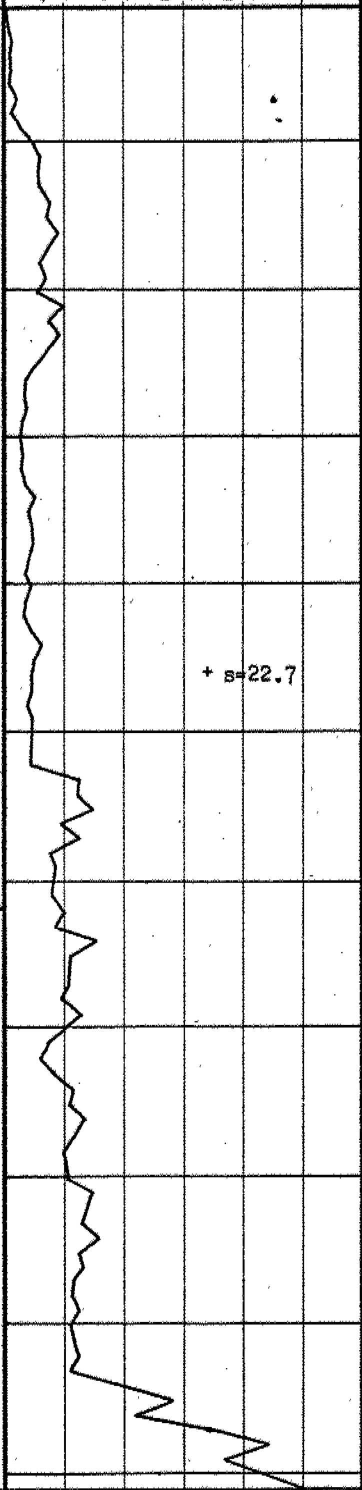
RECORD OF BOREHOLE Nº 1

WP 160-73-01	LOCATION As shown on Dwg. 1	ORIGINATED BY H.S.
DIST 9 HWY 62	BORING DATE June 11 to 14, 1976	COMPILED BY H.S.
DATUM Geodetic	BOREHOLE TYPE H.S. Augers & Cone Test	CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	REMARKS % GR SA SI CL
468.8	Ground Level						
0.0	Silty sand to sand, trace to some gravel. Very loose to compact.		1	SS	2	▽	
			2	SS	2		
			3	SS	8	460	22 74 (4)
			4	SS	7		1 74 (25)
			5	SS	13	450	13 71 (16)
444.3 / 24.5	Clayey silt to silty clay, some pockets or seams of silt - sensitive. Firm to stiff		6	SS	4		
			7	SS	3	440	
			8	SS	2		
			9	SS	2	430	
			10	SS	3	420	
416.8 / 52.0	Silty sand, Trace of gravel Loose		11	SS	7		
408.3 / 60.5	End of Borehole		12	SS	7	410	0 40 55 5
						400	
						390	
						380	
368.8 / 100.0	End of Cone Test					370	

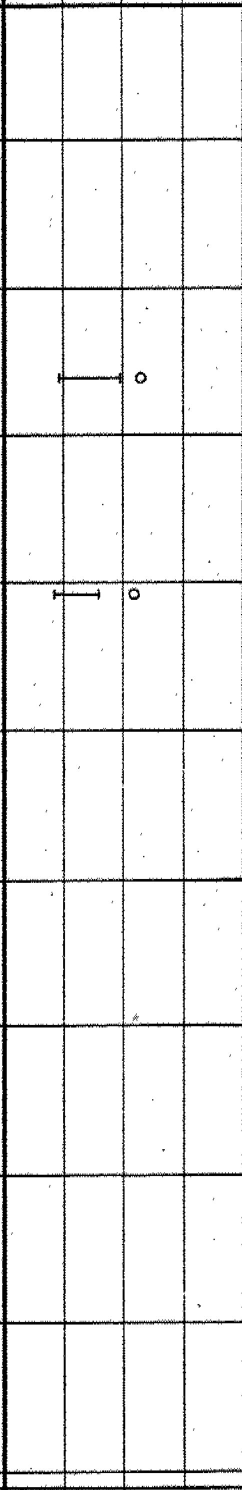
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH PSF ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000



LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W

WATER CONTENT % 20 40 60



UNIT WEIGHT γ

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 2

WP 160-73-01 LOCATION As shown on Dwg. 1 ORIGINATED BY H.S.

DIST 9 HWY 62 BORING DATE June 15 to 17, 1976 COMPILED BY H.S.

DATUM Geodetic BOREHOLE TYPE H.S. Augers & Cone test CHECKED BY

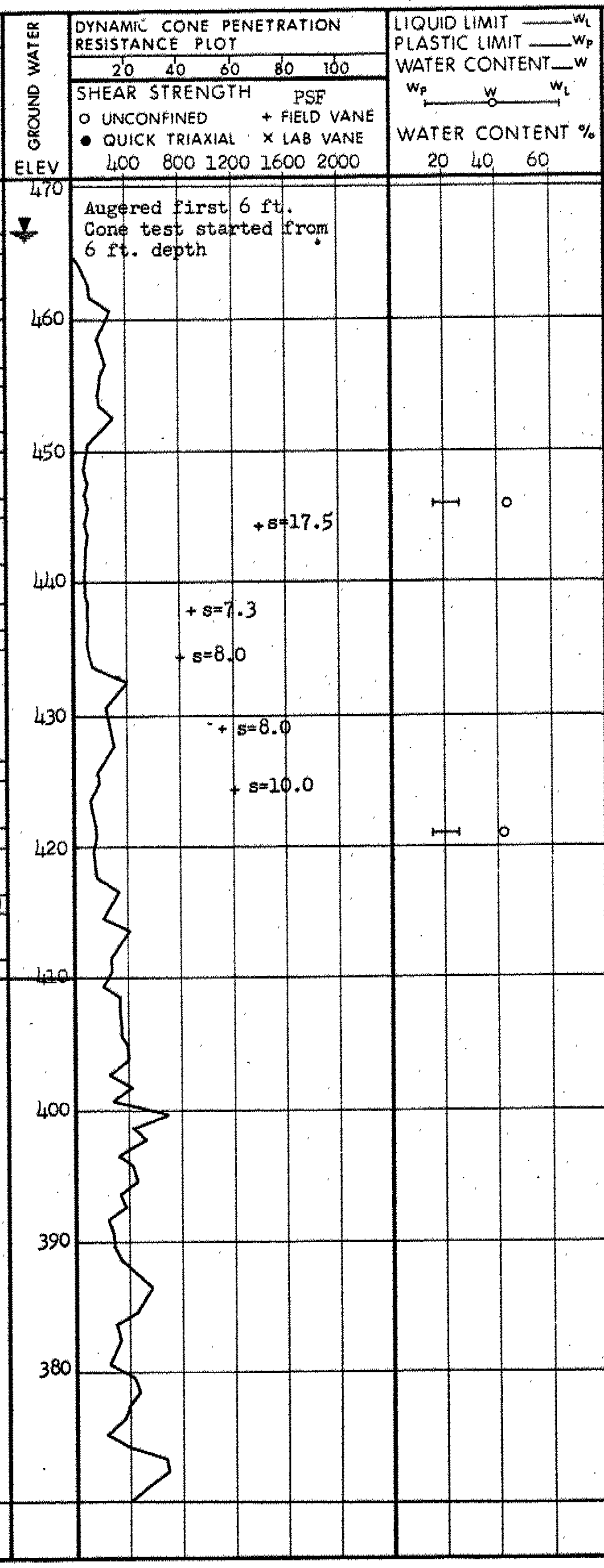
Soil Profile

ELEV / DEPTH	DESCRIPTION	REMARKS % GR SA SI CL
470.6	Ground Level	
0.0	Sand, gravel, some cobbles - FILL	
	Clayey silt, some sand	
465.3 / 5.3	Silty sand to sand Very loose to loose	0 78 (22)
450.6 / 20.0	Clayey silt to silty clay, some pockets or seams of silt - sensitive Firm to stiff	
418.6 / 52.0	Silty sand Loose	0 55 43 2
410.1 / 60.5	End of Borehole	
370.6 / 100.0	End of Cone Test	

Samples

NUMBER	TYPE	'N' VALUES	Shear strength / water content
1	SS	3	
2	SS	8	
3	SS	9	
4	SS	2	
5	SS	1	+ s=17.5
6	TW	PH	+ s=7.3
7	SS	1	+ s=8.0
			+ s=8.0
8	SS	3	+ s=10.0
9	SS	4	
10	SS	10	
11	SS	9	

GROUND WATER: ▼ at approx. elev. 466



15 ⌖ 5 (20 / 10) % STRAIN AT FAILURE

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N'= STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL. THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	c LB./SQ.FT.
VERY SOFT	0 - 250
SOFT	250 - 500
FIRM	500 - 1000
STIFF	1000 - 2000
VERY STIFF	2000 - 4000
HARD	> 4000

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10–25% , WITH 25–40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY, ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

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 SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e VOID RATIO
n POROSITY
w WATER CONTENT
S_r DEGREE OF SATURATION
w_L LIQUID LIMIT
w_P PLASTIC LIMIT
I_P PLASTICITY INDEX
w_S SHRINKAGE LIMIT
I_L LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max} VOID RATIO IN LOOSEST STATE
e_{min} VOID RATIO IN DENSEST STATE
I_D DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
RELATIVE DENSITY D_r IS ALSO USED
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
m_v COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v COEFFICIENT OF CONSOLIDATION
C_c COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U DEGREE OF CONSOLIDATION
τ_f SHEAR STRENGTH
c' EFFECTIVE COHESION INTERCEPT
ϕ' EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u APPARENT COHESION
ϕ_u APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ COEFFICIENT OF FRICTION
S_t SENSITIVITY

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

STRESS AND STRAIN

u PORE PRESSURE
σ NORMAL STRESS
σ' NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ SHEAR STRESS
ε LINEAR STRAIN
γ SHEAR STRAIN
ν POISSON'S RATIO (μ IS ALSO USED)
E MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G MODULUS OF SHEAR DEFORMATION
K MODULUS OF COMPRESSIBILITY
η COEFFICIENT OF VISCOSITY

EARTH PRESSURE

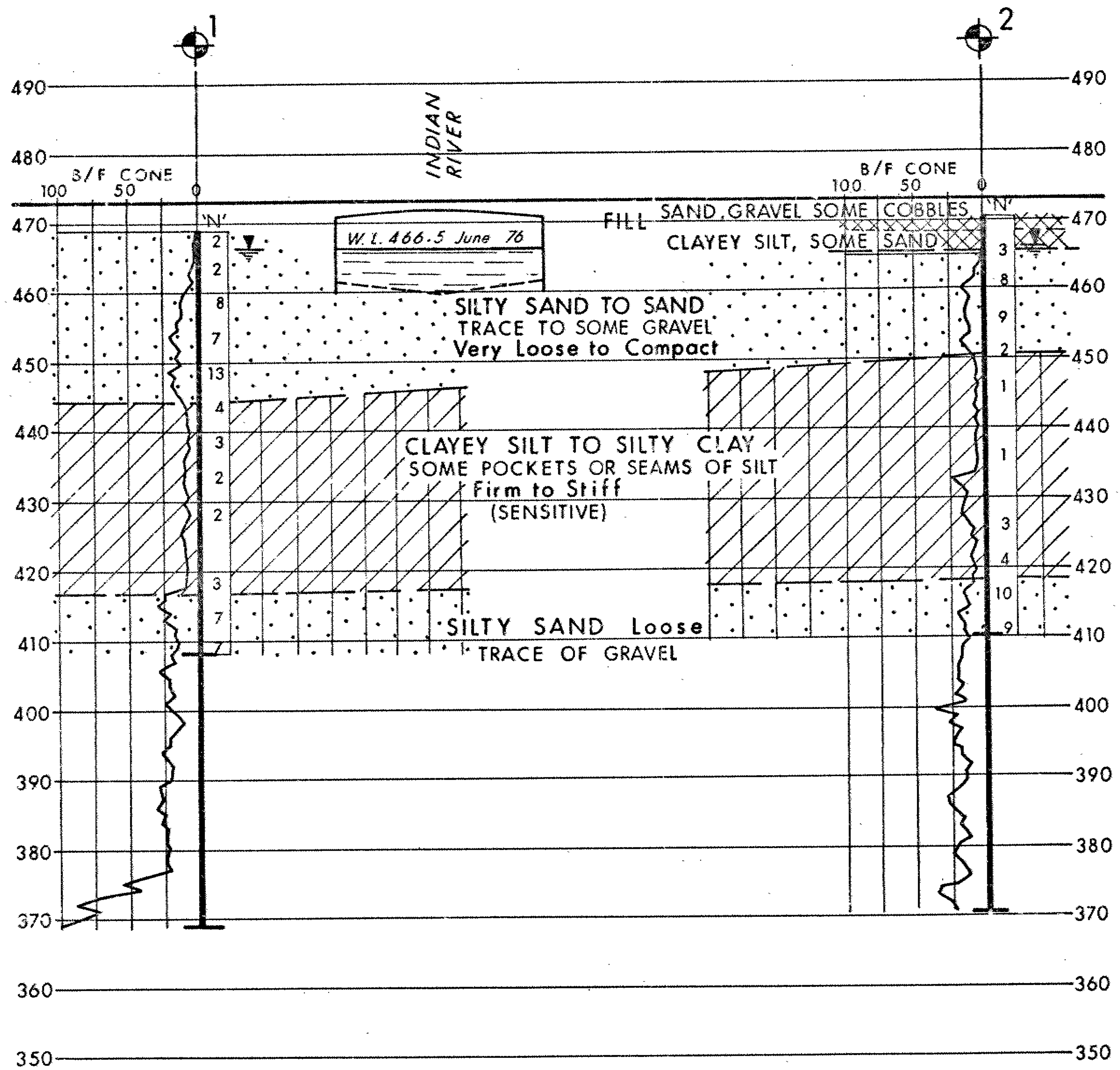
d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ ANGLE OF WALL FRICTION
K DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B BREADTH OF FOUNDATION
L LENGTH OF FOUNDATION
D DEPTH OF FOUNDATION BENEATH GROUND
N DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY. DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s MODULUS OF SUBGRADE REACTION

SLOPES

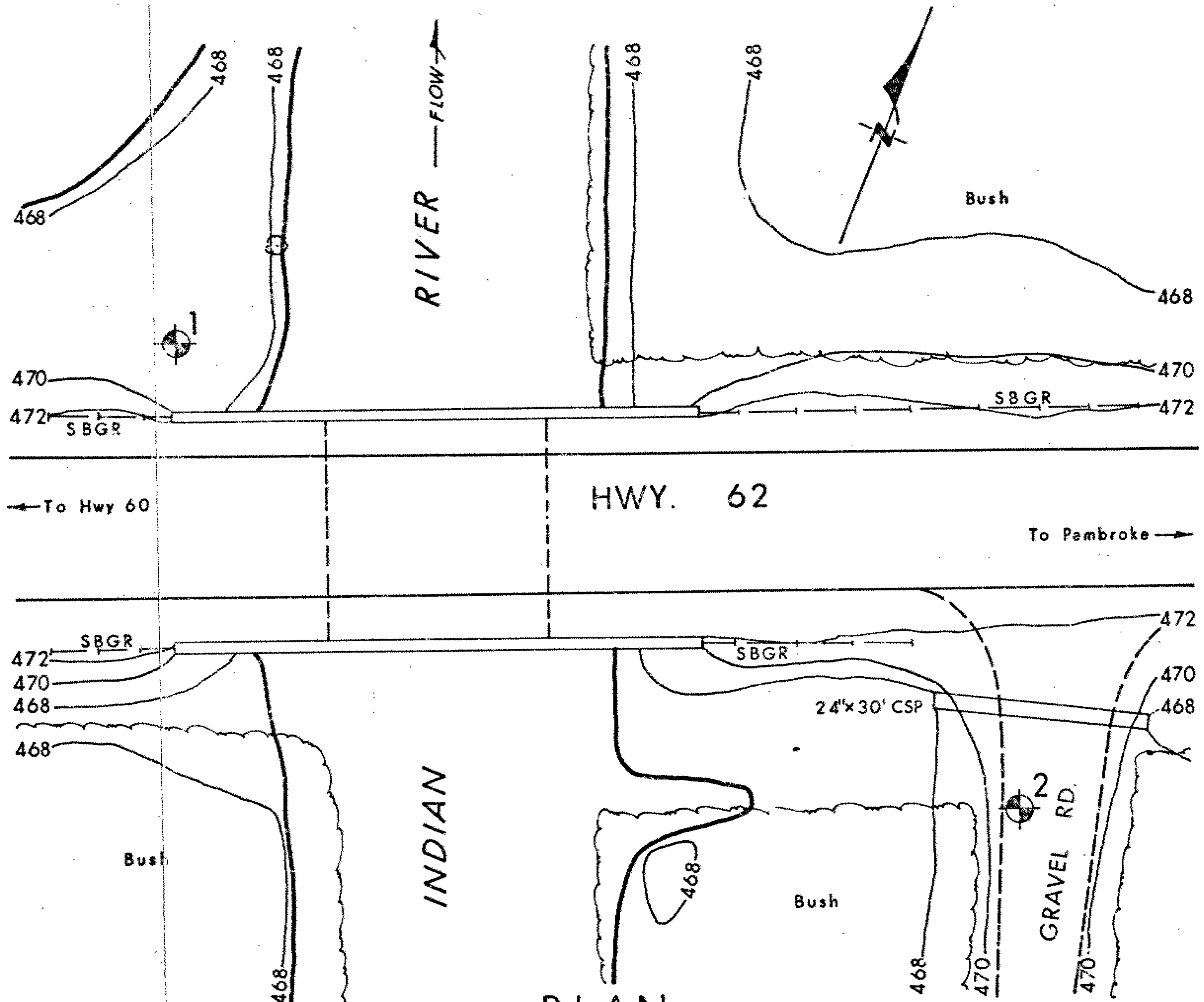
H VERTICAL HEIGHT OF SLOPE
D DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β ANGLE OF SLOPE TO HORIZONTAL



PROFILE HWY 62

SCALE

20 10 0 20 40 FT.



PLAN

SCALE

20 10 0 20 40 FT

REGISTERED PROFESSIONAL ENGINEER
H. M. SHAH
PROVINCE OF ONTARIO

REF No E-5258-1, Dec 1973

Ministry of Transportation and Communications Ontario	PRELIMINARY INVESTIGATION INDIAN RIVER & HWY. 62		
ENGINEERING SERVICES BRANCH	CO. RENFREW CON 8 & 9	TWP. FRASER LOTS 5 & 6	DIST. 9 SITE No 29-19
DATE July 20, 1976	WP 160-73-01		Dwg No 1

PRELIMINARY

FOUNDATION INVESTIGATION & DESIGN REPORT

SOIL MECHANICS SECTION

ENGINEERING SERVICES BRANCH
GEOTECHNICAL OFFICE

Ministry of Transportation and Communications

Ontario

PRELIMINARY

FOUNDATION INVESTIGATION & DESIGN REPORT

W.P. 160-73-02 DIST. 9 (Ottawa)

HWY. 62 STR. SITE 29-18

Indian River Bridge
1.9 Miles West of Alice

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R. Forest) cover only
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Files

GEOCRES DATE JUL 2 6 1976

INTRODUCTION

It is proposed to raise the grade of Hwy. 62 in the vicinity of the above mentioned location. In order to accomodate the proposed grade, the existing structure if utilized would necessitate the deck to be raised by some 1.5 feet. A subsurface investigation was carried out to determine the feasibility of this proposal from the foundation aspects.

This report contains the results of the subsurface investigation carried out in the vicinity of the above mentioned structure and our recommendations pertaining to the bearing capacity of the subsoil under the existing structure footings, as well as the related approach embankments.

SITE DESCRIPTION

The site is located 1.9 miles west of Alice on Hwy. 62, Lots 7 & 8, Concs. 8 & 9, Township of Fraser, County of Renfrew.

At the crossing, the Indian River is approximately 30 feet wide, and the water depth was found to be up to 4.5 ft. The flow velocity of the river water is estimated to be 1 fps.

The relief of the area in the vicinity of the site is generally rolling, and the environs is mainly brush covered with some tree growth. The area on the southwest side of the structure is utilized both for residential and recreational purposes. Rock outcrops are visible about 800 feet northwest of the bridge site.

The existing structure is a 30 ft. single span standard rigid frame bridge built in 1937 and appears to be in sound condition. It is believed to be supported on spread footings at about elev. 461 (Dwg.D2428-1 and available correspondence), however, the details of the footings as constructed are not documented.

FIELD AND LABORATORY INVESTIGATION

The field investigation consisted of two boreholes, each of which was accompanied by a dynamic cone penetration test. Boring was achieved by means of a bombardier mounted hollow-stem auger machine adapted for soil sampling purposes. Disturbed samples were obtained using a 2 inch. O.D. split-spoon sampler driven in accordance to the specifications for the Standard Penetration Test. Bedrock was not encountered within the depth of sampling carried out. The soil and groundwater conditions encountered in the borings are presented on the Record of Borehole Sheets and on Dwg 1.

At the time of report writings, the E plan was not available. Thus, we have prepared Dwg. 1 which shows the location of the boreholes in relation to the structure.

Laboratory tests were performed on selected samples to determine their grain size distributions, and the results are plotted on the Record of Borehole Sheets.

SUBSURFACE CONDITIONS

Beneath a thin mantle (6") of topsoil, is a deposit of sand with trace to some gravel. Both the borings were terminated within this material, and they were sampled up to depths of 36 to 41 feet below the ground level. Standard Penetration Tests carried out in this deposit gave 'N' values ranging from 2 to 20 blows/ft. Difficulties were encountered in sampling since water saturated sand backed up inside the augers. This disturbance leads to alteration of the relative density, and thus 'N' values are not reliable. However,the results of cone penetration tests are more reliable, and it is estimated that the relative density of this material is very loose to compact. The dynamic cone tests were taken down to depths of 42 ft. and 72 ft., and bedrock was not encountered within the scope of the field work carried out. The results of the typical grain size distribution tests are plotted on the Record of Borehole Sheets.

GROUNDWATER CONDITIONS

Groundwater level observations were carried out in the open holes during the period of investigation and the results are shown on the Record of Borehole Sheets and on Dwg. 1. At the time of the investigation, the groundwater level was at elev. 470.2 which was almost similar to the creek water level (470.1).

DISCUSSIONS AND RECOMMENDATIONS

General

In conjunction with the proposed increase of Hwy. 62 grade, in the vicinity of the Indian River, the deck of the existing structure would have to be elevated by some 1.5 feet. A subsurface investigation was carried out to determine the feasibility of this proposal for the structure from the foundations point of view.

The subsoil consists of at least 41 feet thick deposit of very loose to compact sand with trace to some gravel. Bedrock was not encountered within the depth of sampling undertaken.

Our recommendations pertaining to the bearing capacity of the subsoil under the existing structure footings, as well as the related approach embankments are presented below.

Structure Foundations

The existing structure was proposed to be founded on spread footings at about elev. 461 (Dwg.D2428-1 and available correspondence), and was built in 1937. However, as discussed previously, details of the founding levels for the footings as constructed are not documented.

Due to the loose to compact nature of the subsoil, spread footings founded within this material can support an allowable design load of 2 t.s.f.

Approach Fills

No stability problems are anticipated for the proposed embankment height if constructed with 2 horizontal to 1 vertical slopes. The portion of the new embankment should be 'keyed' into the existing embankment in accordance with current MTC practices. No major settlement problems are anticipated.

MISCELLANEOUS

The field work for this investigation was carried out during the period of June 10, 1976 to June 18, 1976 under the supervision of Mr. H. Shah, Project Engineer.

The equipment used for subsoil sampling was owned and operated by Atcost Drilling Company. This report was written by Mr. H. Shah and was reviewed by Mr. B. Ly, Senior Engineer.

H. Shah
H. Shah, P. Eng.
Project Engineer

LICENSED PROFESSIONAL ENGINEER
H. M. SHAH
PROVINCE OF ONTARIO

B. Ly
B. Ly, P. Eng.
Senior Engineer

BL/bp
July, 1976

APPENDIX

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 1

WP 160-73-02 | LOCATION As shown on Dwg. 1 | ORIGINATED BY H.S.

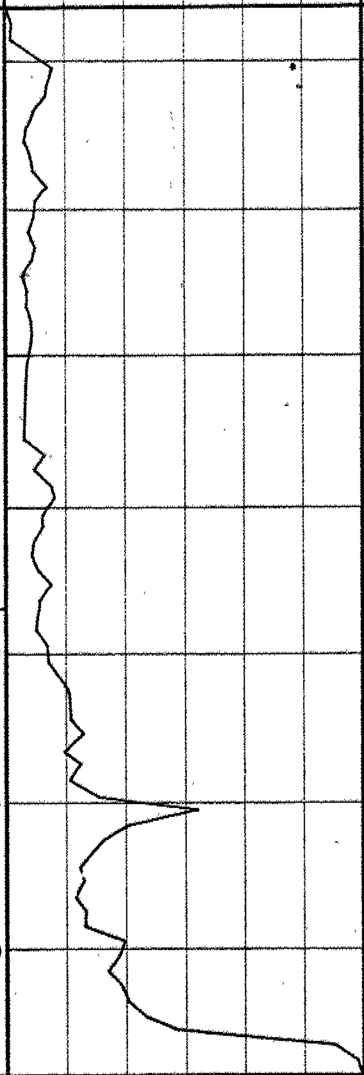
DIST 9 HWY 62 | BORING DATE June 10, 1976 | COMPILED BY H.S.

DATUM Geodetic | BOREHOLE TYPE H.S. Augers & Cone Test | CHECKED BY CP.

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	REMARKS % GR SA SI CL
473.5	Ground level						
0.0	~~Topsoil~~		1	SS	13		
			2	SS	20	▼ 470	
	Sand, trace to some gravel		3	SS	12		
			4	SS	8		
			5	SS	10	460	
	Very loose to Compact		6	SS	8		17 80 (3)
			7	SS	7		
			8	SS	2		
						450	
			9	SS	10		
			10	SS	12		
						440	
			11	SS	8		
433.0			12	SS	5		0 91 (9)
40.5	End of Borehole					430	
						420	
						410	
401.5 / 72.0	End of Cone Test						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE



LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W

W_P —— W —— W_L WATER CONTENT %

UNIT WEIGHT γ

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

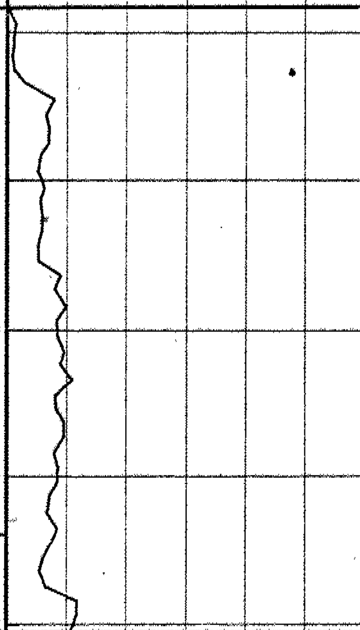
RECORD OF BOREHOLE № 2

WP 160-73-02 LOCATION As shown on Dwg. 1 ORIGINATED BY H.S.

DIST 9 HWY 62 BORING DATE June 18, 1976 COMPILED BY H.S.

DATUM Geodetic BOREHOLE TYPE H.S. Augers & Cone Test CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
471.6	Ground Level									
0.0	Topsoil					▼ 470				
	Sand, trace to some gravel		1	SS	5					
	Loose to Compact		2	SS	6	460				10 85 (5)
			3	SS	16					26 68 (6)
			4	SS	19	450				
			5	SS	18					
			6	SS	19	440				9 84 (7)
436.1			7	SS	19					
35.5	End of Borehole									
429.6						430				
42.0	End of Cone Test									



20
15 5 % STRAIN AT FAILURE
10

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N'= STANDARD PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	c LB./SQ. FT.
VERY SOFT	0 - 250
SOFT	250 - 500
FIRM	500 - 1000
STIFF	1000 - 2000
VERY STIFF	2000 - 4000
HARD	> 4000

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

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	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
w_S	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
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
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\,\Delta\sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
	(c', ϕ') IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
	(c_u, ϕ_u) IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3{\cdot}1416$
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

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

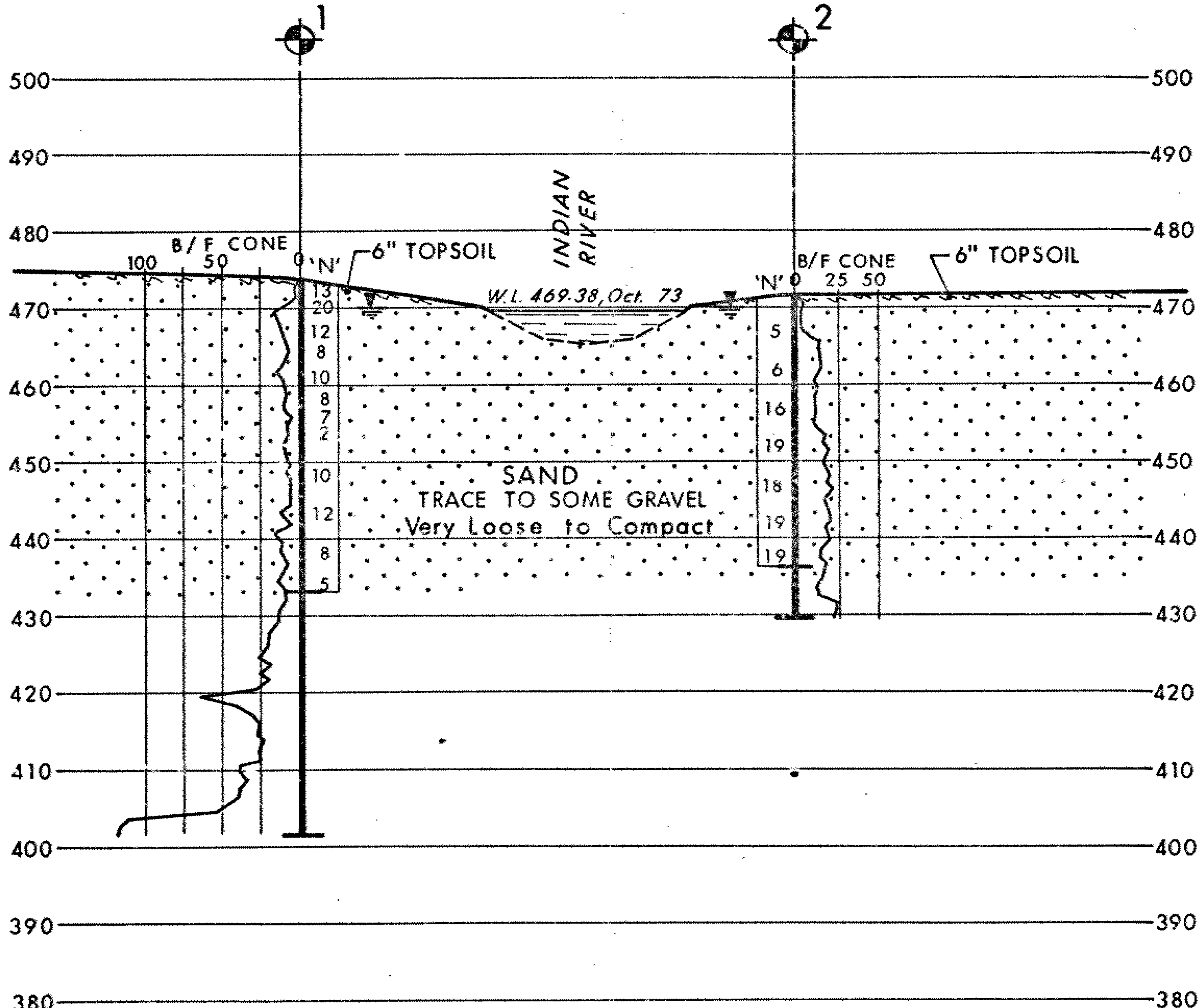
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY. DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

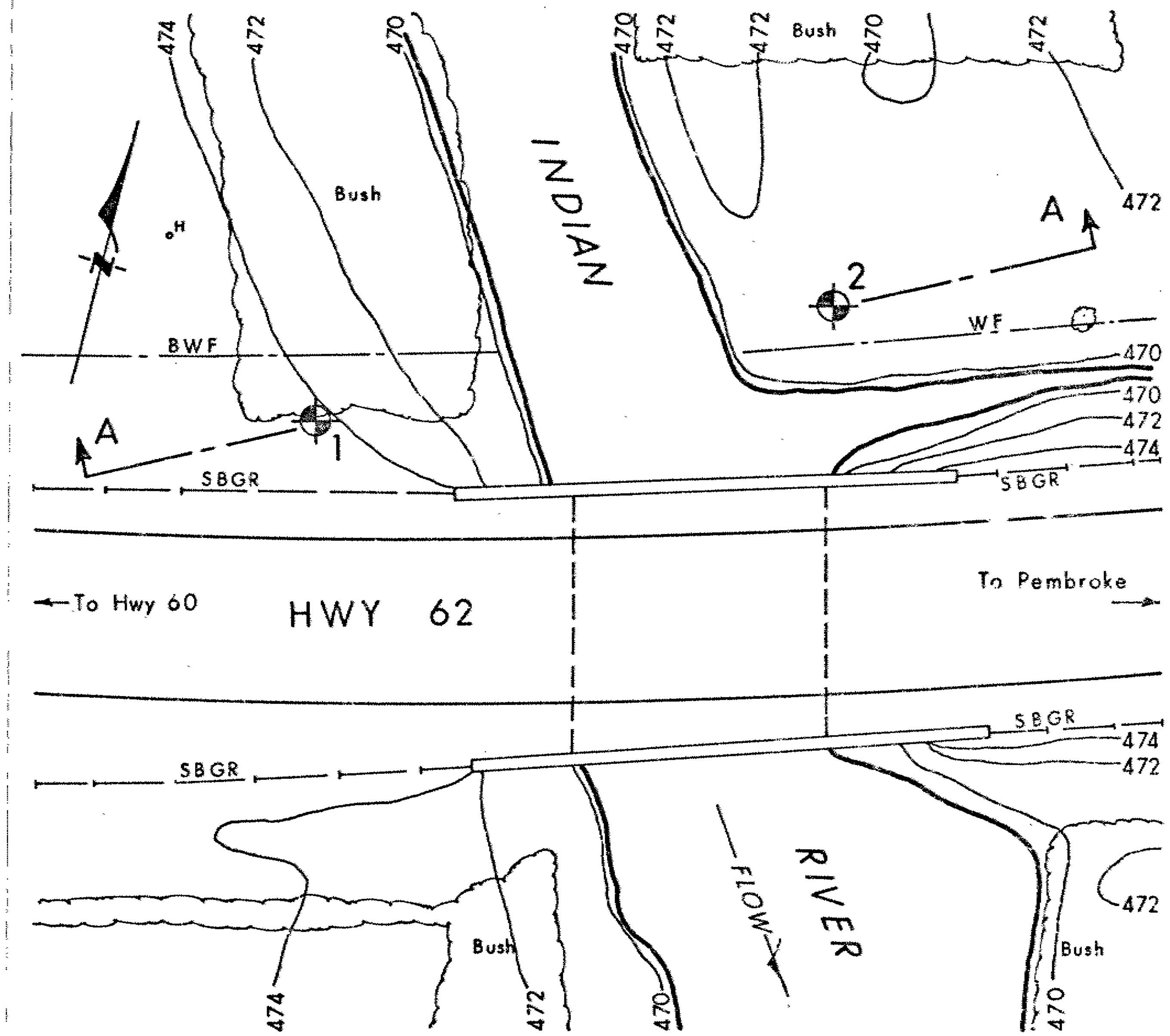
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



SECTION A-A

SCALE

20 10 0 20 40FT.



PLAN

SCALE

20 10 0 20 40FT

REGISTERED PROFESSIONAL ENGINEER
H. M. SHAH
PROVINCE OF ONTARIO

REF No E-5257-1, Dec 1973

Ministry of Transportation and Communications Ontario ENGINEERING SERVICES BRANCH	PRELIMINARY INVESTIGATION INDIAN RIVER & HWY. 62		
	CO. RENFREW CON 8 & 9	TWP FRASER LOTS 7 & 8	DIST. 9 SITE No 29-18
DATE July 20, 1976	WP 160-73-02		Dwg No 1