

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division,
Admin. Bldg.

Attention: Mr. S. McCombie

FROM: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

DATE: February 26, 1968

OUR FILE REF.

IN REPLY TO

FEB 29 1968

SUBJECT:

INTERIM
FOUNDATION INVESTIGATION REPORT
For
South Nation River Crossing of
Proposed Hwy. #417 Line 'B' (Alt.)
and Hwy. #138
District No. 9 (Ottawa)
W.J. 68-F-1 -- W.P. 35-66

Attached, please find our interim foundation report for above crossings. Since the completion of all the calculations still requires some time, it was felt that an interim report be submitted to you in view of the urgency of the project.

We believe that the recommendations of our final report will not differ substantially from those of this report.

We intend to forward our final report as soon as possible.

ACS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
S. J. Markiewicz
C. R. Robertson
G. Scott
J. E. Gruspier
B. A. Singh

Foundations Files ✓
Gen. Files

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

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INTERIM
FOUNDATION INVESTIGATION REPORT
For
South Nation River Crossings of
Proposed Hwy. #417 Line 'B' (Alt.)
and Hwy. #138
District No. 9 (Ottawa)
W.J. 68-F-1 -- W.P. 35-66

1. INTRODUCTION:

A preliminary foundation investigation was requested at the above two crossing sites by Mr. G. Scott, Regional Bridge Location Engineer, Eastern Region (Kingston).

Because of the urgent nature of this job, it has been decided that a concise summary of our findings be submitted to you, prior to the completion of the detailed foundation report. It is hoped that this will enable you to go ahead with your cost analysis as to the best possible crossing.

In this interim report, therefore, a brief description of the subsoils together with the results of the stability analyses of the existing slopes and those of the suggested improvements, are given. At the end of our discussion, some comparisons are tabulated between the recently investigated crossings and the previously investigated sites of A and C. No further reference is made to Sites A-1, A-2, and B, since these sites had already been marked as inferior to Sites A and C.

2. SOIL CONDITIONS:

Some six boreholes were carried out during the recent field work, the results of which are plotted on the attached profiles. Boreholes #1, 2 and 3 were placed along the proposed crossing of Hwy. #417, Line 'B' (Alt.), and holes #4, 5 and 6 at the proposed crossing of Hwy. #138. A fine sand to silty sand layer was found to form the surficial stratum at both sites with

2. SOIL CONDITIONS: (cont'd.) ...

seams and pockets of silty clay. The depth of the predominantly granular layer was about 9 ft. at the crossing of Hwy. #417, Line 'B' (Alt.), increasing to about 17 - 21 ft. at the Hwy. #138 crossing. The relative density of the stratum ranges from very loose to compact. Underlying the silty sand, a rather thick deposit of sensitive marine silty clay was encountered with consistencies varying from soft to stiff and very stiff, generally increasing with depth. The average thickness of the silty clay deposit below the high ground may be taken to be 73 - 95 ft. The marine clay is followed by a 10 - 20 ft. thick layer of sandy gravel to gravelly sand (glacial till), which in turn, is underlain by limestone bedrock of the Trenton formation.

3. STABILITY ANALYSES OF THE SLOPES:

3.1) As may be seen on the attached profiles, the geometries of the two crossings are quite similar. The normal water level of the river (assumed as El. 500 ft.) is approximately 61 - 69 ft. lower than the general ground level. The full width of the river valley is roughly 700 - 800 ft. The west banks at both locations are naturally terraced or benched, while the east banks have continuous slopes. It is likely that the terrace has been formed by earlier failures; the overall slope is, therefore, flatter - i.e., around 5 horizontal to 1 vertical. The overall slopes of the east banks were found to be approx. 3.5 - 4 horizontal to 1 vertical.

3.2) Stability analyses in terms of total (immediate stability) and effective (long term) stresses were carried out by means of an electronic computer. Average shear strength parameters - based on laboratory and field tests - were used for the calculations. The undrained shear strength of the marine clay deposit at the proposed crossing of Hwy. #417 B (Alt.) was taken to be $C = 350 - 450$ PSF

cont'd. /3 ...

3. STABILITY ANALYSES OF THE SLOPES: (cont'd.) ...

within the upper portion of the stratum, increasing to approx. $C = 1500$ PSF with depth. At the site of the Hwy. #138 crossing the lowest average value of undrained shear strength was assumed to be $C = 750$ PSF, increasing again up to $C = 1500$ PSF with depth. The values of the effective stress parameters, used along the entire depth of the silty clay deposit were: $\phi' = 23^\circ$ and $C' = 280$ PSF.

It is to be noted that the analyses using effective stress parameters, based on consolidated undrained triaxial tests with pore pressure measurements, are not fully applicable. In stiff fissured clays, the progressive reduction in the value of C' eventually will approach zero in the failure zone. (A. W. Bishop & L. Bjerrum, 1960). Moreover, the laboratory ϕ' values of sensitive marine clays are usually grossly overestimated as was reported by Bjerrum, Bishop, Crawford, and others. A detailed study of the long-term stability of the slopes is intended to be presented in our forthcoming foundation report.

Location	F.S. (Total Stresses)	F.S. (Effective Stresses)
Hwy. #417 B (Alt.) East Bank	0.83	< 1.0
Hwy. #417 B (Alt.) West Bank	1.38	> 1.0
Hwy. #138 - East Bank	1.01	< 1.0
Hwy. #138 - West Bank	1.41	> 1.0

TABLE I

cont'd. /4 ...

3. STABILITY ANALYSES OF THE SLOPES: (cont'd.) ...

From the foregoing table, it may be seen that the stability analyses for the west banks resulted in higher factors of safety than for the east banks due to the flatter overall slopes. Safety factors of less than unity are obviously erroneous; nevertheless, slopes with safety factors near unity may very well be at the verge of failure.

3.3) The investigated crossing sites are located in an area of considerable seismic activity, as was discussed by Geocon, Ltd. No attempt was made at this early stage to incorporate the induced forces, caused by an earthquake, into the stability computations. As a very approximate guide it may be said that the safety factors will be reduced roughly 25% by an earthquake of magnitude 6 with a horizontal ground acceleration of 0.1g. It was therefore concluded that the designed slopes should have a minimum safety factor of F.S. = 1.5, in order to ensure the stability of slopes in case of an earthquake of magnitude 6.

4. RECOMMENDATIONS:

4.1) The recommended approximate slopes at both crossings are shown on the attached drawings. The lower parts of the slopes were designed with 4 horizontal to 1 vertical slopes, for a height of about 20 ft. above river level. Berm sections are utilized in order to improve stability, above which slopes of 5 horizontal to 1 vertical are suggested up to the high ground level. Stability analyses in terms of total and effective stresses, also with the assumption of a sudden drawdown of floodwater, are currently being carried out for the design sections. Although not all the results of these computations are yet available, it is felt that the designed slopes as shown on the drawings, are acceptable, necessitating bridge lengths of about 1,000 ft.

The transitions from the designed slopes to the natural slopes perpendicularly to the centre-line, should be constructed with slopes of 6 horizontal to 1 vertical.

cont'd. /5 ...

4. RECOMMENDATIONS: (cont'd.) ...

4.2) The tentative grades of Hwy's #417 and #138 will be somewhat lower than the elevation of the existing high ground. Stability analyses indicated that cuts up to about 14 ft. with 3 horizontal to 1 vertical slopes would be stable. Should deeper cuts be contemplated, they would require flatter slopes or counterbalancing benches.

4.3) The material removed from the approach slopes is entirely unacceptable for any kind of highway fills; consequently, it should be hauled away from the immediate vicinity of the proposed highways and existing river banks.

On Table #2 some factual data of the four proposed crossings are recorded for comparison purposes. It is, of course, realized that the final selection depends on several other factors; nevertheless, we hope that the tabulated figures will assist you in your decision.

The detailed foundation report of the above crossings will be submitted to you after completion of the computations.

Crossing	Height of Slope (Estimated from Bottom of River) (Ft.)	Thickness of Overburden below Water Level (Ft.)	Average Undrained Shear Strength of Marine Clay (PSF)	Estimated Length of Bridge (Ft.)	Existing Natural Slopes
Site A	East Bank - 35 West Bank - 43	112	500 - 2900	440	2:1
Site C	71	15 - 16	800 - 2000	700	3:1
Site B (Alt.)	82	45	450 - 1500	1000	4:1 & 5:1
Site Hwy. 138	85	45	750 - 1500	1000	3.5:1 & 5:1

TABLE 2

5. MISCELLANEOUS:

The field investigation, carried out during the period January 5 to January 24, 1968, was supervised by Messrs. A. Seppala and P. Payer, Project Foundation Engineers.

Equipment used was owned and operated by Canadian Longyear Ltd., and Johnston Drilling Company Ltd.

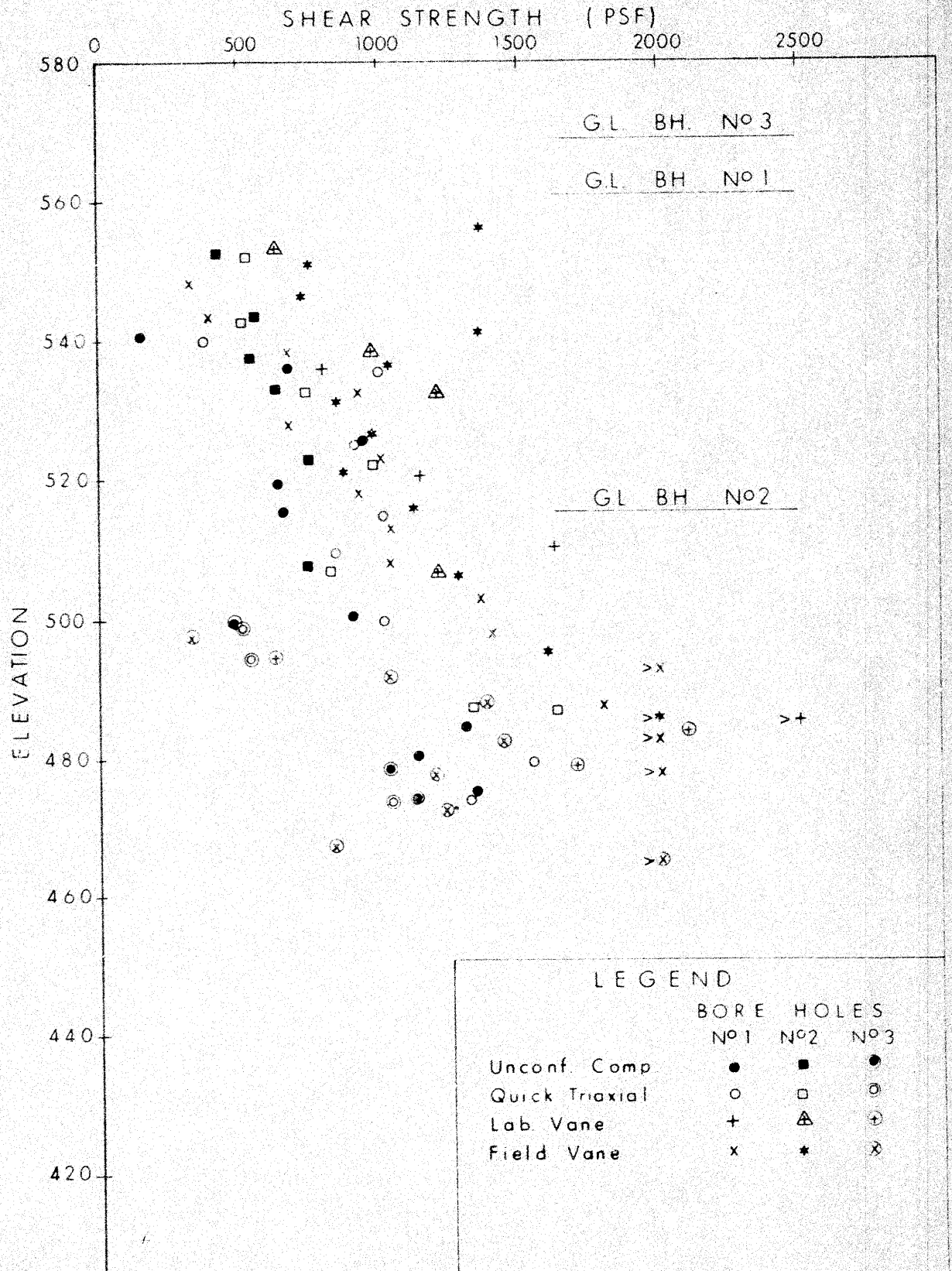
Mr. A. K. Barsvary, Senior Foundation Engineer, who was in charge of the entire project, also wrote this report.

Mr. K. G. Selby, Supervising Foundation Engineer, reviewed the report.

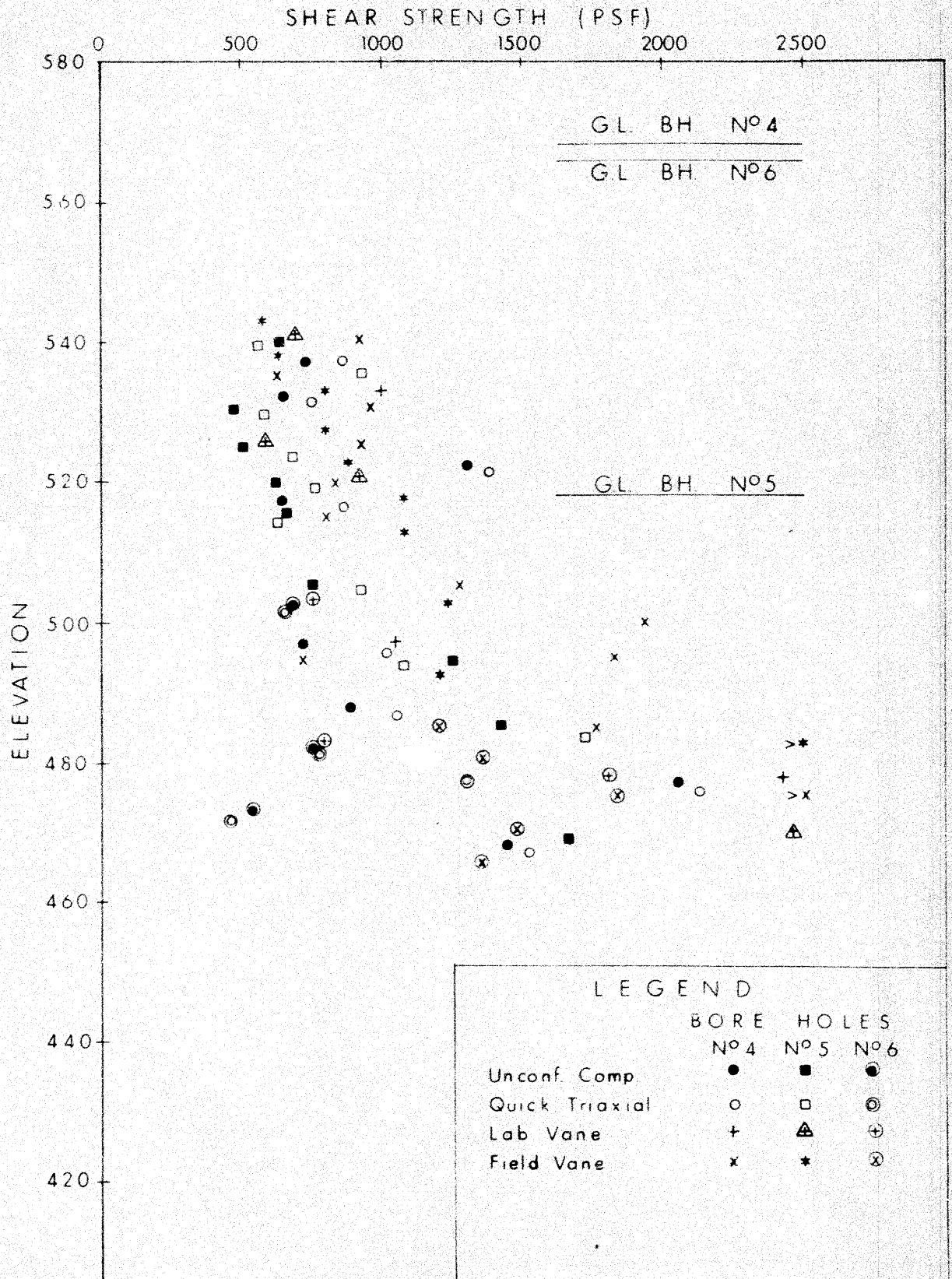
February 1968.

APPENDIX I

PROPOSED CROSSING HWY. 417 LINE 'B' (ALT.) SHEAR STRENGTH VS. ELEVATION



PROPOSED CROSSING HWY. 138 SHEAR STRENGTH VS. ELEVATION



ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' -- 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:--

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

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

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS 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
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
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

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 (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

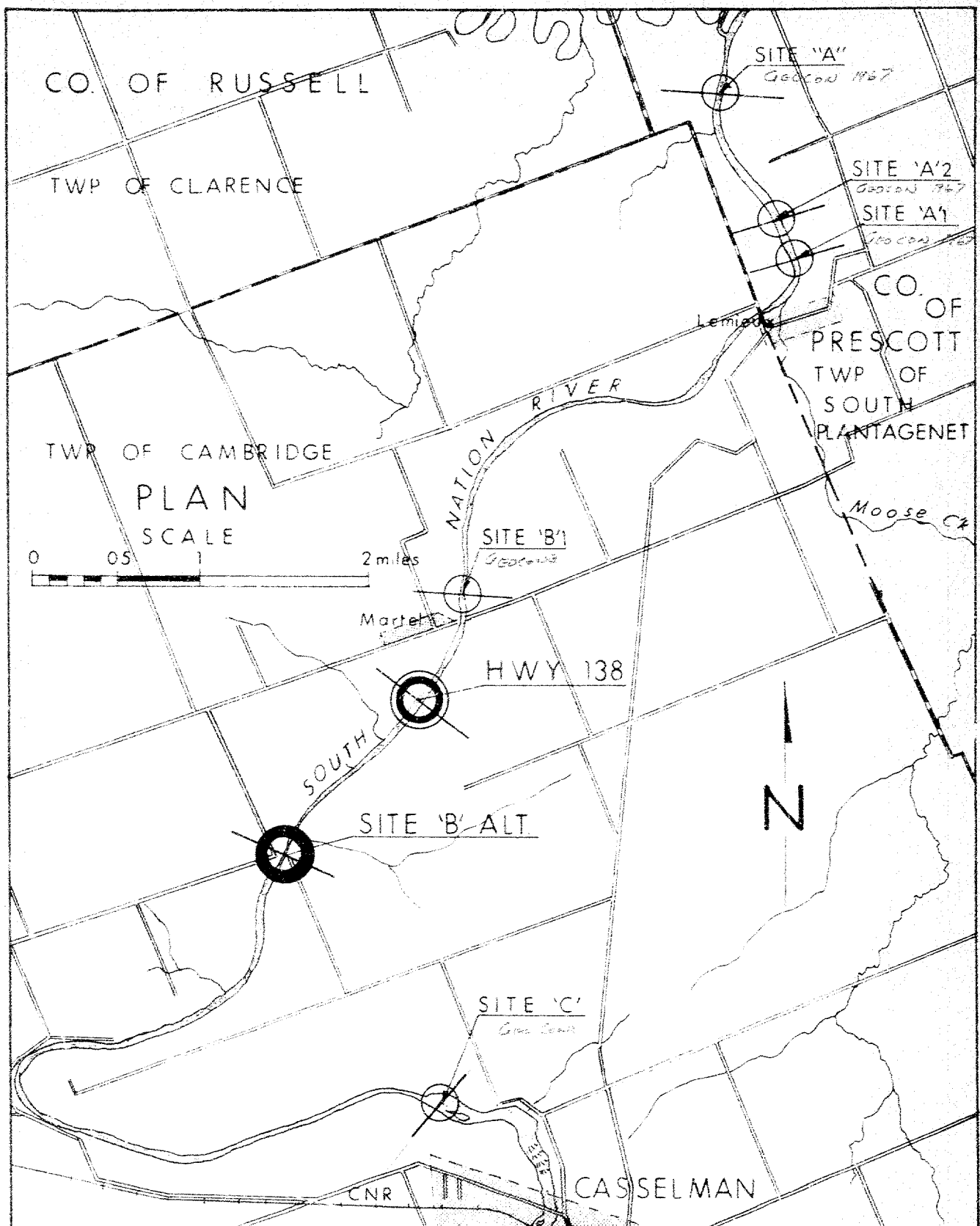
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

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



ONTARIO

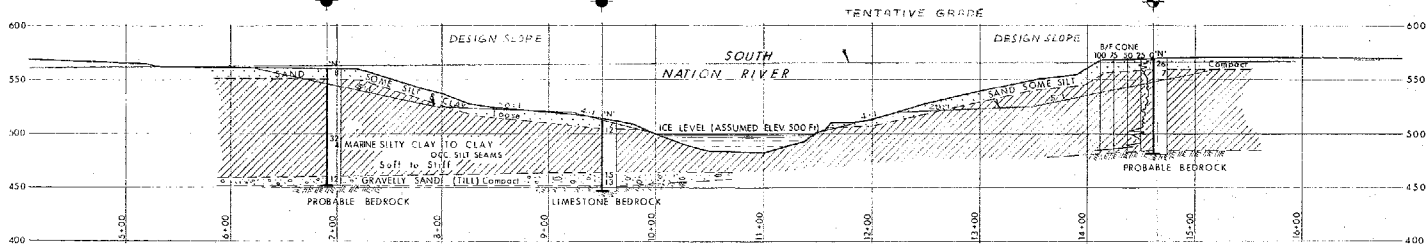
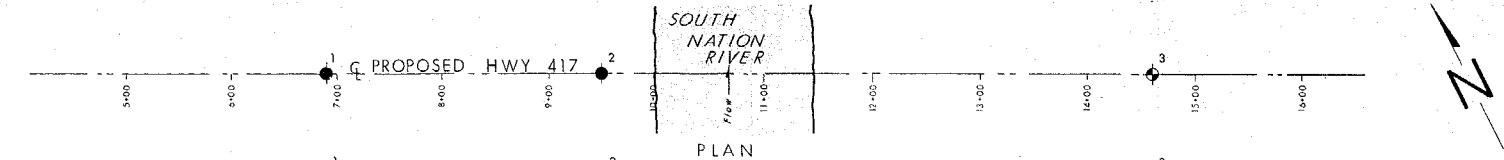
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

HWY 417 & SOUTH NATION RIVER
SITE LOCATIONS
(NORTH OF CASSELMAN)

DATE FEB. 14, 1968

APPROVED

DRAWING NO. 68-F-1A



PROFILE

SCALE: 1" = 50'



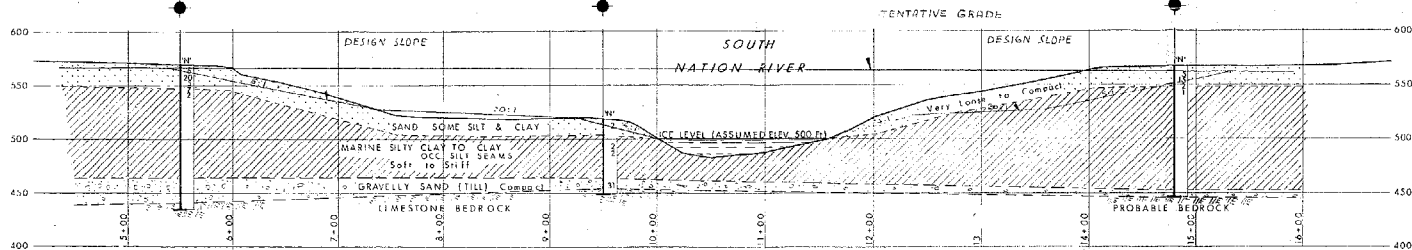
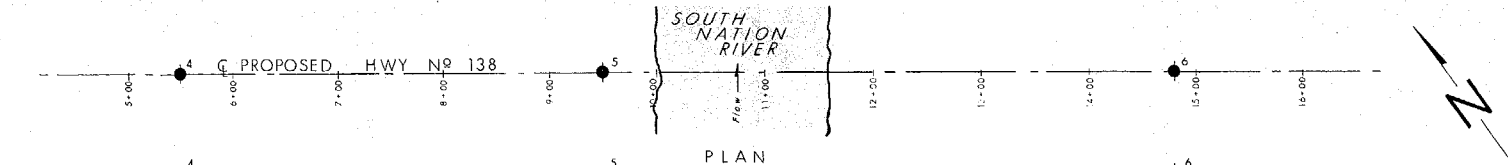
DEPARTMENT OF HIGHWAYS
ONTARIO
MATERIALS and
TESTING
DIVISION

PROPOSED HWY NO 417
SITE 'B' (ALTERNATE)

DATE FEB. 12, 1968

APPROVED *[Signature]*

DRAWING NO. 68-F-1B



PROFILE

SCALE: 1" = 50'



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PROPOSED HWY NO 138

DATE FEB. 17, 1968

APPROVED: [Signature]

DRAWING NO. 68-F-1C

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie

DATE: March 19, 1968

OUR FILE REF.

IN REPLY TO

MAR 22 1968

SUBJECT:

FINAL
FOUNDATION INVESTIGATION REPORT
For
South Nation River Crossings of
Proposed Hwy. #417 Line 'B' (Alt.)
and Hwy. #138
District No. 9 (Ottawa)
W.J. 68-F-1 -- W.P. 35-66

Attached, please find our final foundation report on the subsoil conditions existing at the above crossings.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
S. J. Markiewicz
C. R. Robertson
G. Scott
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Foundations Files
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A. G. Stermac
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PRINCIPAL FOUNDATION ENGINEER

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-

FINAL
FOUNDATION INVESTIGATION REPORT
For
South Nation River Crossings of
Proposed Hwy. #417 Line 'B' (Alt.)
and Hwy. #138
District No. 9 (Ottawa)
W.J. 68-F-1 -- W.P. 35-66

1. INTRODUCTION:

A foundation investigation was requested by Mr. G. Scott, Regional Bridges Location Engineer, Eastern Region (Kingston), in a memo dated December 18, 1967.

The memo called for investigations at the site of the South Nation River crossing of the proposed Hwy. #417 'B' (Alt.) and also at the contemplated crossing of proposed Hwy. #138. The purpose of the investigation was to obtain preliminary information as to the suitability of these particular sites for the crossings.

Several crossing sites had already been investigated at this general area by Geocon Ltd. of Toronto, the results of which were dealt with in their report, dated January 9, 1967. It was found by Geocon Ltd. that among the sites investigated, sites 'A' and 'C' were the most favourable ones from the soil mechanics point of view. The alignment leading to site 'A' is, however, some six miles north of Casselman, and since the service to this town is of prime importance, it would necessitate an additional crossing of the river for the extension of Hwy. #138 to the freeway.

Crossing 'C' has the desirable feature of being only 2 miles from the town, hence no additional crossing for Hwy. #138 would be required; nevertheless, the incurred property damage would be extremely high. Site 'B' (Alt.) was therefore selected, in order to eliminate high property costs, and at the same time to provide the required service to Casselman, without an additional crossing of the South Nation River.

cont'd. /2 ...

1. INTRODUCTION: (cont'd.) ...

The field investigation was carried out by this Section during January, 1953. This time of the year was not really ideal for such investigation because of the deep snow cover. On account of the snow, visual observations of existing failure areas were restricted, and the access to the crossing sites with the rather heavy machinery became a time-consuming operation.

The two lines were established and staked out, and the profiles were plotted by the staff of the Eastern Region. Elevations used were relative to the existing ice level of the river, which arbitrarily was assumed to be at El. 500 ft.

2. DESCRIPTION OF THE SITE:

The proposed crossings are some 6000 ft. distance from each other, the general area being about 3.5 miles north-west of Casselman. The ground surface of the vicinity is flat, interrupted by several gullies and ravines, all of which were formed by tributaries of the South Nation River. The normal water level of the river is approx. 61 - 69 ft. deeper than the prevailing ground level, the full width of the river valley being approx. 700 - 800 ft. The west bank is naturally terraced or benched, while the east bank has a continuous slope. The near horizontal terrace is usually inundated during the period of floods. It is likely that the terrace has been formed by earlier failures; the overall slope is therefore flatter - i.e., around 5 horizontal to 1 vertical. The overall slopes of the east banks were found to be approx. 3.5 - 4 horizontal to 1 vertical. At the toes of terraces and benches, however, much steeper partial slopes were also observed.

Near the crossings the area is mainly occupied by forests (La Rose) under the jurisdiction of the Department of Lands and Forests. The east bank of Hwy. #138 crossing is a privately owned camp site.

2. DESCRIPTION OF THE SITE: (cont'd.) ...

Physiographically, the terrain lies at the south portion of the Russell and Prescott Sand Plains. The depth of sand below ground level varies from 20 to 30 ft., thinning out to the south along the clay plain. The texture of sand also varies, being coarse towards the north, grading into fine sand and silt around the investigated crossings. The sand is underlain by stratified red and grey clay. The South Nation River cuts a valley 75 ft. deep across the plain from Casselman to Lemieux. Within the valley there are several well-preserved remnants of terraces, representing earlier stages of the river.

3. FIELD AND LABORATORY INVESTIGATIONS:

3.1) Some six boreholes, 1 dynamic cone penetration test and the installations of 7 piezometers were carried out during the field investigation. Boreholes along the crossing of Hwy. #417 B (Alt.) were numbered 1, 2 and 3. Piezometers #1, 2 and 3 were installed adjacent to these holes. Boreholes #4, 5 and 6 were placed along the crossing of proposed Hwy. #138, together with piezometers #4, 5, 6 and 7, the latter two being installed adjacent to hole #6. The depth of piezometer #7 was some 60 ft. below ground level, while the rest were placed at a depth of 30 ft. The investigation was carried out by means of two conventional diamond core drills adapted for soil sampling purposes, and one track-mounted continuous flight auger (Bombardier). Soil samples were recovered using 2-inch O.D. split-spoon samplers and 2-inch I.D. thin wall open samplers (Shelby tubes). Penetration tests were performed according to standard methods described at the end of this report. (See "Abbreviations Used in This Report".) The in-situ undrained shear strength of the cohesive deposits were measured during drilling by means of field vanes.

cont'd. /4 ...

3. FIELD AND LABORATORY INVESTIGATIONS: (cont'd.) ...

3.2) Upon arrival in the laboratory, soil specimens were subjected to laboratory testing in order to determine shear strength properties in terms of total and effective stresses, consolidation characteristics, Atterberg limits, bulk densities and grain-size distributions. The results of these tests are described in the next paragraph, and also presented graphically on the borehole sheets accompanying this report.

4. SUBSOIL CONDITIONS:

4.1) General:

Three distinct soil strata were observed to form the overburden at both proposed crossing sites. The upper layer was identified to be sandy silt to silty sand with some clay. This was followed by deposits of marine silty clay to clay, which in turn, was underlain by gravelly sand and sandy gravel (glacial till). Limestone bedrock lies beneath the till at about 45 - 50 ft. below normal river level. A detailed description of the various deposits follows.

4.2) Sandy Silt to Silty Sand:

The brown sandy silt to silty sand surficial layer was found in every borehole. The thickness of the deposit is 9 - 10 ft. at the Hwy. #417 B (Alt.) crossing site, and 17 - 21 ft. at the site of the proposed Hwy. #138 crossing. The grain-size distributions of the material indicate that the stratum is coarser towards the north, becoming almost pure silt at the south crossing. Two typical grain-size curves are attached to this report (Fig. #1). Averaging the results of these tests, we get 17% sand, 77% silt, and 6% clay size particles.

At the south crossing the soil is identified as sandy silt to silt with some clay; at the north crossing it is silty sand to sandy silt with some clay.

Standard penetration tests resulted in 'N' values between 2 and 26 blows per foot, indicating relative densities of very

cont'd. /5 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Sandy Silt to Silty Sand: (cont'd.) ...

loose to compact. The bulk density of the layer was estimated to be 115 PCF and the angle of internal friction about $\phi = 30^\circ$. Some samples exhibited a slight apparent cohesion, but it disappeared almost entirely when the samples were dried out.

4.3) Silty Clay to Clay:

Underlying the sandy silt, an extensive layer of marine silty clay to clay deposit was revealed. The thickness of this material is 75 - 95 ft. below the high ground and some 40 - 42 ft. in the river valley. The stratum is rather heterogeneous, containing layers and seams of grey and brown silty clay and clay of various dimensions. Numerous very thin seams of pure silt were also observed. The stratification is sometimes contorted, indicating wave action during sedimentations. Several fissures were also noticed within the samples. Bulk unit weights were determined from undisturbed samples, giving values ranging from 96 PCF to 124 PCF, with an average of 108 PCF. All the samples were subjected to natural moisture content and Atterberg limit tests. The large variations of the results of these tests are due to the variations of the constituent particle sizes. The brown seams appear to have higher moisture contents and plasticities than the grey portion. The lowest and highest values of natural moisture contents were 30% and 64%, averaging 52%. The obtained plasticity indices ranged between 8 and 37%, yielding a mean value of 19%. The highest, lowest and average values of Atterberg limits, moisture contents and bulk densities are listed on Table #1, as follows:

cont'd. 6 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Silty Clay to Clay: (cont'd.) ...

TEST	HIGHEST	LOWEST	AVERAGE
Plastic Limit W _p (%)	31	15	23
Liquid Limit W _L (%)	60	26	44
Moisture Content W (%)	64	30	52
Plasticity Index I _p (%)	37	8	19
Bulk Density γ _B (PCF)	124	96	108

TABLE #1

Laboratory Test Results of the Marine Silty Clay

On the attached Plasticity Chart (Fig. #2), the index properties of the samples are shown graphically, identifying the soils according to the Unified Classification System.

The undrained shear strength of the deposit was determined by field and laboratory vane, unconfined compression and unconsolidated undrained triaxial tests. The values of these tests are compiled separately for the two crossings, on Fig. #3 and #4.

cont'd. /7 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Silty Clay to Clay: (cont'd.) ...

The field and laboratory values were found to be quite comparable; however, the field vane tests are somewhat higher than the laboratory tests, as usual. The shear strength of the upper portion of the silty clay at the location of the proposed Hwy. #417 crossing is lower than at the crossing of Hwy. #138. At the former crossing, undrained shear strengths of 320 - 450 PSF were measured within the upper 10-ft. layer; the corresponding values at the latter crossing were 580 - 800 PSF. The shear strength of the material increases with depth at both locations, the ratio of undrained shear strength to the effective overburden pressure being generally constant $\frac{C_u}{P} = 0.30 - 0.45$. The average shear strength of the middle portion of the stratum may be taken to be $C_u \approx 1000$ PSF, while within the bottom portion $C_u \approx 1500$ PSF.

In studying the strain of failure of the laboratory tests, it was found to be fairly low, averaging 1.5 - 3%. Failures occur rather abruptly and the failure planes are usually well defined. In the unconfined compression tests, some specimens failed under tension, along a vertical plane, which is typical of the marine clays of the Ottawa valley (Leda clays). No such vertical splitting was observed when the samples were subjected to lateral pressures in the triaxial cell. These specimens generally develop a shear plane of about 45 - 60° to the horizontal under the applied axial load.

The sensitivity of the material - defined as the ratio of the undisturbed shear strength to the remolded strength at the same moisture content - was measured by field and laboratory vane tests. Results of these tests yielded a great deal of variations, values of sensitivity ranging between 4 and 34 with an average of 10. Based upon the results of shear tests, the consistency of the marine deposit is defined as soft to very stiff.

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Silty Clay to Clay: (cont'd.) ...

Consolidation tests were performed on a few representative samples. Tests were conducted with conventional load increments ($\frac{\Delta P}{P} = 1$), also, with controlled strain as described by Crawford (1960). The tests indicated that the deposit is preconsolidated by about 1.0 - 1.2 TSF in excess of the existing effective overburden pressure, below the high ground. At the river valley (B.H. #2), the preconsolidation pressure was found to be about 2.7 TSF in excess of the existing pressure. The difference of about 1.5 TSF between the obtained preconsolidation pressures corresponds closely to the pressure exerted by the 50 - 60 ft. of overburden removed when forming the river valley.

Consolidated undrained triaxial tests with pore pressure measurements were carried out on a few samples, using both the conventional and that of the stage loading technique. Shear strength parameters, in terms of effective stresses, were calculated to yield values of the angle of internal friction $\phi' = 23^\circ - 23.9^\circ$ and cohesion of $C' = 288 - 357$ PSF. The effective stress parameters require further scrutiny, and they will be discussed at some length under paragraph #6 (Discussion).

4.4) Gravelly Sand with Some Silt (Glacial Till):

A gravelly sand to sandy gravel (glacial till) with some silt was encountered beneath the marine clay deposit. The thickness of the grey-coloured, coarse-grained material ranges between 1 ft. and 9 ft. at the southern crossing and between 7 ft. and 24 ft. at the northern (Hwy. #138). Standard penetration 'N' values of 12 to 31 blows/ft. within this stratum indicate a material of compact to dense relative density.

cont'd. /9 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.5) Bedrock:

Bedrock was found to underlie the overburden at around 45 - 50 ft. below normal river water level. The bedrock surface was noticed to be somewhat uneven, indicating slopes or dips. Bedrock was proved by core drilling with AXT core barrels at 5 locations for about 5 - 10 ft. The rock samples were classified to be shaley limestone and limestone with shale seams of fine to medium grains.

Geologically, the rock belongs to the Ordovician period and to the Trenton formation.

5. WATER CONDITIONS:

5.1) River Water:

At the time of the field investigation the river was frozen, the thickness of ice being 6 - 8". The ice level was assumed to be equal to the normal water level of the river (taken as El. 500 ft.) According to the information obtained from the Hydrology Section, the water level of the river rises up to 32 ft. above normal water level during the spring flood. The high water level buildup takes approx. 10 days, and after 7 - 8 days of high water, it recedes gradually. Normal water level is reached in about two months or so. A sudden drawdown of floodwater may occur occasionally in the case of a sudden break of a downstream ice jam.

5.2) Groundwater:

Piezometers were installed adjacent to the boreholes, in order to observe groundwater conditions as mentioned earlier. Due to the low permeability of the silty clay layer, it is believed that the equilibrium water level was not reached during the time of the field work. The piezometers were, therefore, left in place. It is intended to take further readings of these instruments in order to record the effect of floodwater buildup on the groundwater.

5. WATER CONDITIONS: (cont'd.) ...

5.2) Groundwater: (cont'd.) ...

The groundwater level may be taken to be between 7 and 15 ft. below existing ground level as observed in January, 1968. The excess hydrostatic head relative to the normal river level, below the high ground, is around 60 ft. (approx. 26 PSI).

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

The purpose of this investigation has been to obtain preliminary information as to the suitability of these sites for the river crossing, as compared with the previously studied sites.

The proposed design grades of Hwy. #417 and #138, shown on Drawing #68-F-1A, are somewhat lower than the prevailing high ground level. The required minimum clearance above the normal river level is 50 - 51 ft. It is also understood that the minimum length of the centre span of the bridge should be around 200 ft.

6.2) Structure Foundations:

The sensitive marine deposits - forming the main portion of the overburden - are not considered to be suitable to support a structure safely and economically on spread footings within the overburden. Footings should, therefore, be supported on piles driven to bedrock. The elevations of the bedrock surface were established at borehole locations only. Variations in the observed rock surface, warrant that the rock level be proved by additional borings at the locations of the proposed footings, after they are decided upon. For the purposes of cost estimates, the bedrock surface may be assumed to be at around El. 440 - 450 ft. The use of steel H-piles appears to be the right proposal, since the disturbance of the sensitive clay is relatively smaller during the driving of H-piles, as compared with large displacement piles. Safe pressures equal to the structural strength of the particular pile section used, are recommended for design purposes on piles driven to sound bedrock.

cont'd. /11 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.3) Approaches:

(a) The banks of the South Nation River have been known to be the subject of extensive earth movements, landslides and local failures. The instability of slopes and existing failure areas are relatively easy to detect along the banks in question.

(b) Further to the stability problems, the area is located within a region of severe earthquake history, as was reported by Geocon Ltd. (1967). No additional study of the earthquakes and their effects have been made by this Section at this time, since it was thought that by using the assumptions of Geocon Ltd., a more reliable comparison may be made as to the suitability of various sites.

As was reported, no earthquakes have caused an earthquake magnitude in excess of 6 in the area of Casselman between the year 1861 and the present time. With such a magnitude, a maximum acceleration of 100 cm./sec./sec. may be expected - i.e., an acceleration of one tenth the acceleration due to gravity (gravity = 980.66 cm./sec./sec.) Based upon this assumption, Geocon introduced an additional overturning moment to the stability analyses of slopes. This additional force of 0.1 times of the unbalanced weight acts horizontally on the centre of gravity of the unbalanced soil mass. By studying the effect of such an additional force on the stability of slopes, it was found that it reduced the factor of safety by approx. 25%. For this reason, higher factors of safety have been sought, so that banks would stay stable in the case of an earthquake as well.

Due to the visible signs of instability, it was decided that the stability of the existing slopes be studied first.

(c) Natural Slopes:

Stability analyses were carried out in terms of total (immediate stability) and effective (long-term) stresses by means

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

(c) Natural Slopes: (cont'd.) ...

of an electronic computer. Average undrained shear strength parameters were used for the total stress analyses, based upon laboratory and field tests.

The surficial sand and silt layer was assumed to be a purely frictional material with an angle of internal friction $\phi = 30^\circ$. The marine clay deposit was divided into 3 - 4 horizontal sections, each with an assigned average undrained shear parameter. The undrained shear strength of the uppermost portion of the marine clay was assumed to be $C_u = 350 - 450$ PSF, the middle portions between 750 PSF and 1000 PSF, whereas the strength of the bottom section was averaged to be $C_u = 1500$ PSF. The sandy gravel to gravelly sand (glacial till) stratum was again considered as a frictional material with $\phi = 30^\circ$.

The computations yielded values of safety factors (in terms of total stresses) as listed below (Table #2):

HWY.	SLOPE	F.S.
417 B (Alt.)	West Bank	1.38
	East Bank	0.83
138	West Bank	1.40
	East Bank	1.01

TABLE #2

cont'd. /13 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

(c) Natural Slopes: (cont'd.) ...

From the foregoing table, it may be seen that the stability analyses, in terms of total stresses for the west banks, resulted in higher factors of safety than for the east banks due to the flatter overall slopes. Safety factors of less than unity are obviously erroneous; nevertheless, those slopes having safety factors near unity, may very well be at the verge of failure. By analyzing the partial slopes, it was noted that the upper portions of the west banks at both locations with natural slopes of 3.5 horizontal to 1 vertical, yielded values of F.S. = 0.94 and 1.06; consequently, these slopes may also be considered to be rather unstable.

In reviewing the total stress analyses, it was concluded that the obtained safety factors were not high enough to ensure the stability of the natural banks, especially when assuming adverse circumstances such as an earthquake. It was therefore decided that the natural slopes be improved, and slopes similar to the existing west banks, be designed with flatter partial slopes.

The natural slopes were analyzed in terms of effective stresses as well. As mentioned earlier, the effective stress parameters obtained by means of consolidated undrained triaxial tests with pore pressure measurements, were $C' = 280 - 357$ PSF and $\phi' = 23^\circ - 23.9^\circ$. The uncertainties in using laboratory test results for the long-term stability of natural slopes, have been discussed in publications during recent years. A number of problems arise where stiff fissures, or sensitive marine clays form the slopes. In stiff fissured clays, special account has to be taken of the progressive reduction in the value of C' , which appears eventually to approach zero in the failure zone. Since the shear strains and water content change associated with

cont'd. /14 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

(c) Natural Slopes: (cont'd.) ...

failure are very localized, tests on the bulk of the soil do not reveal the decrease in C' (Bishop and Bjerrum, 1960). It is therefore recommended by some authors that the stability of natural slopes in stiff fissured clays be checked by using $C' = 0$. It is suggested that the safety factor obtained by using such a severe assumption, should still be larger than one (F.S. > 1).

The laboratory ϕ' values of sensitive marine clays are very often overestimated as reported by Bishop, Bjerrum, Henkel, Crawford, and others. The overestimation of the value of ϕ' is particularly pronounced in those cases where the initial water content is above the liquid limits, as happens to be the case with Leda clays. Due to the sensitivity to disturbance of such clays, the reconsolidation in the triaxial cell is always accompanied by a reduction of water content.

Stability analyses, in terms of effective stresses, were therefore carried out by using modified stress parameters. The angle of internal friction was assumed to be $\phi' = 20^\circ$, and the cohesion intercept was taken to be $C' = 250$ PSF. Using these values, the safety factors of the natural slopes were equal to or slightly lower than the safety factors of the total stress analyses, confirming the necessity for the design of improved banks.

(d) Design Slopes:

The recommended slopes at both crossings are shown on Drawing #68-F-1A. The lower parts of the slopes were designed with 4 horizontal to 1 vertical slopes for a height of about 20 ft. above normal river level. Berm sections are utilized in order to improve stability, above which slopes of 5 horizontal to 1 vertical are suggested up to the proposed grade. Stability analyses, in

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

(d) Design Slopes: (cont'd.) ...

terms of total and effective stresses, were carried out for the design slope, using the aforementioned modified parameters. Computations, assuming a sudden drawdown from flood level to normal river level, also assuming a complete reduction of C' , were performed. The summary of the analyses for the designed overall slope and those of the upper and lower partial slopes are tabulated on Table #3, which follows:

cont'd. /16 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

(d) Design Slopes: (cont'd.) ...

SLOPE	METHOD	ASSUMPTIONS	PARAMETERS OF MARINE CLAY	P.S.
Overall	Total Stresses	-	$\phi = 0$; $C = 450 - 1500$ PSF	1.65
	Effective Stresses	Normal River Level	$\phi' = 20^\circ$; $C' = 250$ PSF	1.73
	"	"	$\phi' = 20^\circ$; $C' = 0$ PSF	1.27
	"	Sudden Drawdown Case	$\phi' = 20^\circ$; $C' = 250$ PSF	1.58
	"	"	$\phi' = 20^\circ$; $C' = 0$ PSF	1.06
Lower Partial Slope	Total Stresses	-	$\phi = 0$; $C = 450 - 1500$ PSF	1.87
	Effective Stresses	Normal River Level	$\phi' = 20^\circ$; $C' = 250$ PSF	1.52
	"	"	$\phi' = 20^\circ$; $C' = 0$ PSF	0.84
	"	Sudden Drawdown Case	$\phi' = 20^\circ$; $C' = 250$ PSF	1.39
	"	"	$\phi' = 20^\circ$; $C' = 0$ PSF	0.59
Upper Partial Slope	Total Stresses	-	$\phi = 0$; $C = 450 - 1500$ PSF	1.34
	Effective Stresses	Normal River Level	$\phi' = 20^\circ$; $C' = 250$ PSF	1.63
	"	"	$\phi' = 20^\circ$; $C' = 0$ PSF	1.23

TABLE #3

Results of stability Analyses of the Design Slope, applicable for both sides.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

(d) Design Slopes: (cont'd.) ...

As may be seen on the table, the overall stability of the design slope appears to be satisfactory even with the assumption of a sudden drawdown and reduced C' .

The stability of the lower portion of the design slope is very adversely affected by the reduction of C' to zero. It is believed, however, that this portion can be improved by employing rip-rap over the slope and forming a small berm by the granular rip-rap material in front of the slope.

The analyses of the upper portion of the slope indicate that this portion is acceptable. The sudden drawdown case has no detrimental consequence on this portion.

By constructing the design slopes as shown on Drawing #68-F-1A, a structure of about 1000 ft. length will be necessary for the crossing.

The transitions from the design slopes to the natural slopes perpendicularly to the centre-line, should be constructed with slopes of 6 horizontal to 1 vertical maximum steepness.

The tentative grades of Hwy. #417 and #138 will be somewhat lower than the elevation of the existing high ground. Stability analyses showed that cuts up to about 14 ft. with 3 horizontal to 1 vertical slopes would be stable. Should deeper cuts be contemplated, they would require flatter slopes or counterbalancing benches.

It is recommended that granular rip-rap be placed on the approach slopes to an elevation above flood water level.

The material removed from the approach slopes is entirely unacceptable for any kind of highway fills; consequently, it should be hauled away from the immediate vicinity of the proposed highways and existing river banks.

cont'd. /18 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

On Table #4, some factual data of the four proposed crossings are recorded for comparison purposes. It is, of course, realized that the final selection depends on several other factors; nevertheless, we hope that the tabulated figures will assist you in your decision.

Crossing	Height of Slope (Estimated from Bottom of River) (Ft.)	Thickness of Overburden below Water Level (Ft.)	Average Undrained Shear Strength of Marine Clay (PSF)	Estimated Length of Bridge (Ft.)	Existing Natural Slopes
Site A	East Bank - 35 West Bank - 43	112	500 - 2900	440	2:1
Site C	71	15 - 16	800 - 2000	700	3:1
Site B (Alt.)	82	45	450 - 1500	1000	4:1 & 5:1
Site Hwy. 138	85	45	750 - 1500	1000	3.5:1 & 5:1

TABLE #4

7. SUMMARY:

The results of a soils investigation at the site of the proposed South Nation River crossing of Hwy. #417 B (Alt.) and Hwy. #138, are reported.

The overburden was found to consist of a layer of fine sand and sandy silt, underlain by a thick deposit of sensitive

cont'd. /19 ...

7. SUMMARY: (cont'd.)

marine silty clay. Some sand and gravel (glacial till) overlies the limestone bedrock, which was observed to be around 40 - 50 ft. below normal river level.

Spread footings for the proposed structures are not considered to be feasible within the overburden, hence piled foundations are recommended. It is felt that steel H-piles, driven to sound bedrock will support loads equal to the structural strength of the pile.

Stability analyses, in terms of total and effective stresses, showed that the stability of the existing natural slopes is not satisfactory. Some improvements of the banks are therefore recommended by flattening the slopes and incorporating benched sections.

The stability of the design slopes (Drawing #68-F-1A) was analyzed, using various assumptions as described under paragraph (6.3), the results of which were found to be acceptable.

By the construction of the design slopes, structures of some 1000 ft. length will be required for the crossings.

Some factual data of the two investigated crossings are compared with those of the previously studied Site A and C, on Table #4.

8. MISCELLANEOUS:

The field investigation, carried out during the period January 5 to January 24, 1968, was supervised by Messrs. A. Seppala and P. Payer, Project Foundation Engineers.

Equipment used was owned and operated by Canadian Longyear Ltd., and Johnston Drilling Company Ltd.

Mr. A. K. Barsvary, Senior Foundation Engineer, who was in charge of the entire project, also wrote this report.

Mr. K. G. Selby, Supervising Foundation Engineer, reviewed the report.

March, 1968.

APPENDIX I.

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 68-F-1

LOCATION Site "B" (alt.) Sta. 6 + 90 ±

ORIGINATED BY AKB

W.P. 35-66

BORING DATE Jan. 9 - 17, 1968

COMPILED BY AKB

DATUM Assumed

BOREHOLE TYPE Pen Drill

CHECKED BY

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — L PLASTIC LIMIT — P WATER CONTENT — W		BULK DENSITY P C F	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		SHEAR STRENGTH P.S.F. • — Q_u x — Field Vane ○ — Q + — Lab Vane		WATER CONTENT % 20 40 60			
561.1	Ground Level										
0.0	Fine sand with some silt and clay.				560						
	Loose, Brown.		1	SS	8						
551.6			2	SS	0						
9.5			3	TW	PM						
	Silty clay to clay, seams of silt.		4	TW	PM						
	Soft to stiff.		5	TW	PM						
	Grey.		6	TW	PM						
	Irregular layers of brown silty clay		7	TW	PM						
			8	TW	PM						
			9	TW	PM						
			10	TW	PM						
			11	TW	PM						
			12	TW	PM						
			13	TW	PM						
			14	SS	32						
			15	SS	4						
			16	TW	PM						
			17	TW	PM						
			18	TW	PM						
460.6											
100.5	Gravelly sand with some silt & clay.		19	SS	12						
451.6	Compact										
109.5	End of Borehole Probably Bedrock										

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 68-F-1 LOCATION Site "B" (Alt.) Sta. 9 + 50 ORIGINATED BY AKB
W.P. 35-66 BORING DATE Jan. 4, 1968 COMPILED BY AKB
DATUM Assumed BOREHOLE TYPE Washboring, NX Casing CHECKED BY AKB

SOIL PROFILE		STRAT. PLOT	SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.			WATER CONTENT %					
							• - C_u ○ - C	x - Field Vane + - Lab Vane		wp ———— wL —————					
515.5	Ground Level						1000	2000		20	40	60			
0.0	Fine sand with silt and clay.		1	SS	7	510								▼ 508.5	
506.5	Loose, Brown.														
9.0	Silty clay Soft to stiff Grey Occasional Brown seams.		2	SS	12										
			3	TW	PM	500									104
			4	TW	PM										109
			5	TW	PM	490									103
			6	TW	PM										100
			7	TW	PM	480									97
			8	TW	PM										103
			9	TW	PM	470									105
464.5			10	TW	PM										
51.0	Gravelly sand with some silt & clay.	11	SS	15	460										
	Compact														
452.5		12	SS	13											
63.0	Limestone Bedrock	13	RC		450										
447.1															
68.4	End of Borehole														
						440									
						430									

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 68-F-1

LOCATION Site "B" (Alt.) Sta. 14 + 60 @

ORIGINATED BY AKB

W P 35-66

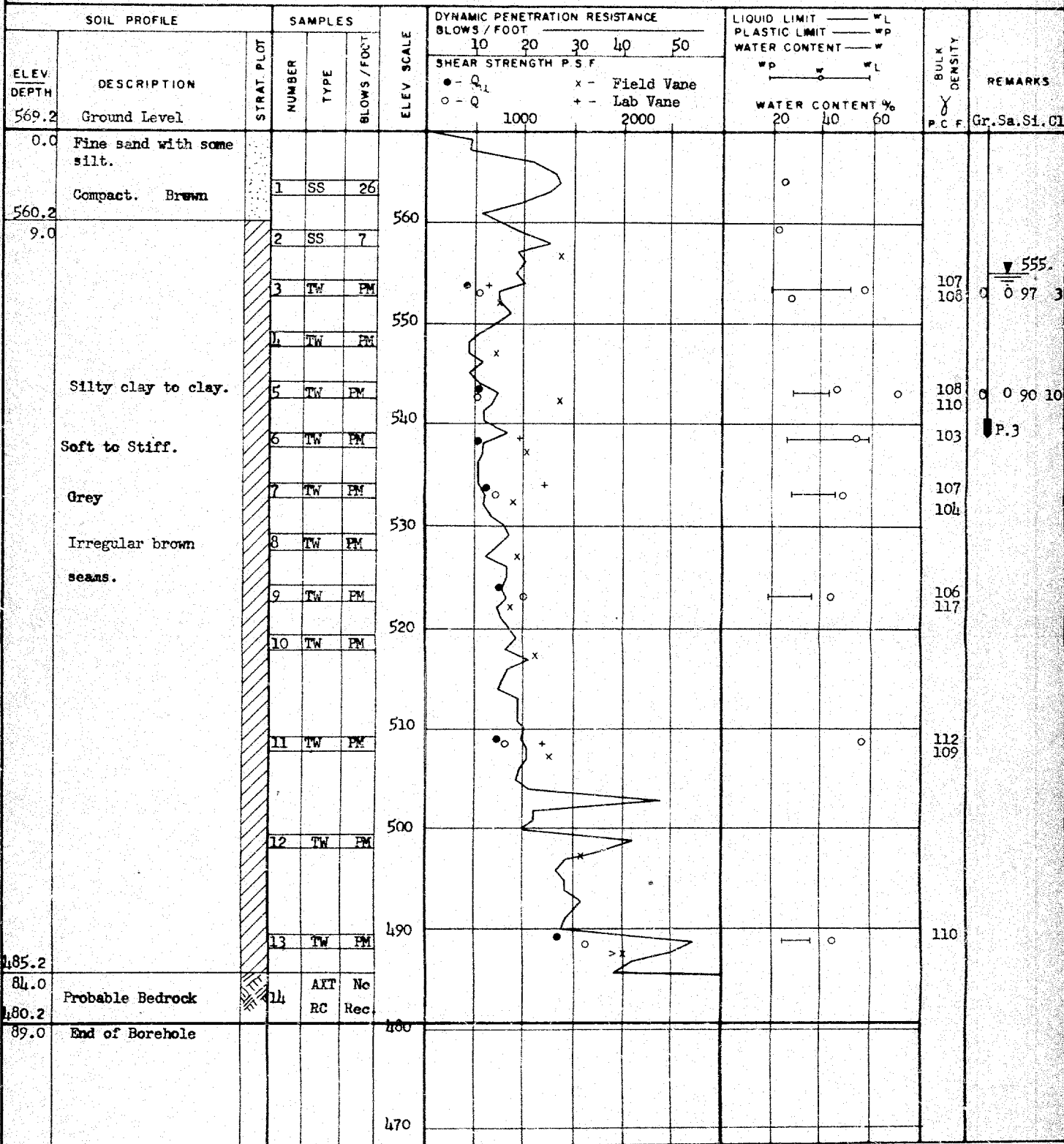
BORING DATE Jan. 11, 1968

COMPILED BY AKE

DATUM Assumed

BOREHOLE TYPE Washboring, NX Casing

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 68-F-1 LOCATION Sta. 5 + 50 @ North Crossing ORIGINATED BY AMS
W.P. 35-66 BORING DATE Jan. 18, 19, 20, 22, 23, 1968 COMPILED BY FP
DATUM Geodetic BOREHOLE TYPE Auger & Washbore - BX Casing CHECKED BY AMS

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER		TYPE	SHEAR STRENGTH P.S.F.		WATER CONTENT %		
568.5	Ground Level					1000 2000		20 40 60		
0.0	Sandy silt with seams of silty clay. Loose to compact.		1	SS	6					
			2	SS	20					
			3	SS	5					
547.5			4	SS	7					
21.0	Silty clay to clay, seams of silt. Firm to very stiff. Grey Irregular Brown Layers		5	SS	2					
			6	TW	PM					111
			7	TW	PM					112
			8	TW	PM					101
			9	TW	PM					104
			10	TW	PM					101
			11	TW	PM					102
			12	TW	PM					
			13	TW	PM					101
			14	TW	PM					102
			15	TW	PM					101
			16	AXT	Rec.					108
464.5										108
104.0	Sandy gravel with silt and clay.									111
										108
										111
										104
										105
										114
440.0										
128.5	Shaley Limestone									
435.0	Bedrock									
133.5	End of Borehole									

566.3
Jan. 19/68

P-4

111

112

101

104

101

102

109

108

108

111

104

105

114

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO.5

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOA 68-F-1

LOCATION Sta. 9 + 50 @ North Crossing

ORIGINATED BY **AMS**

W P 35-66

BORING DATE Jan. 17, 18 & 19, 1968

COMPILED BY _____ AMS

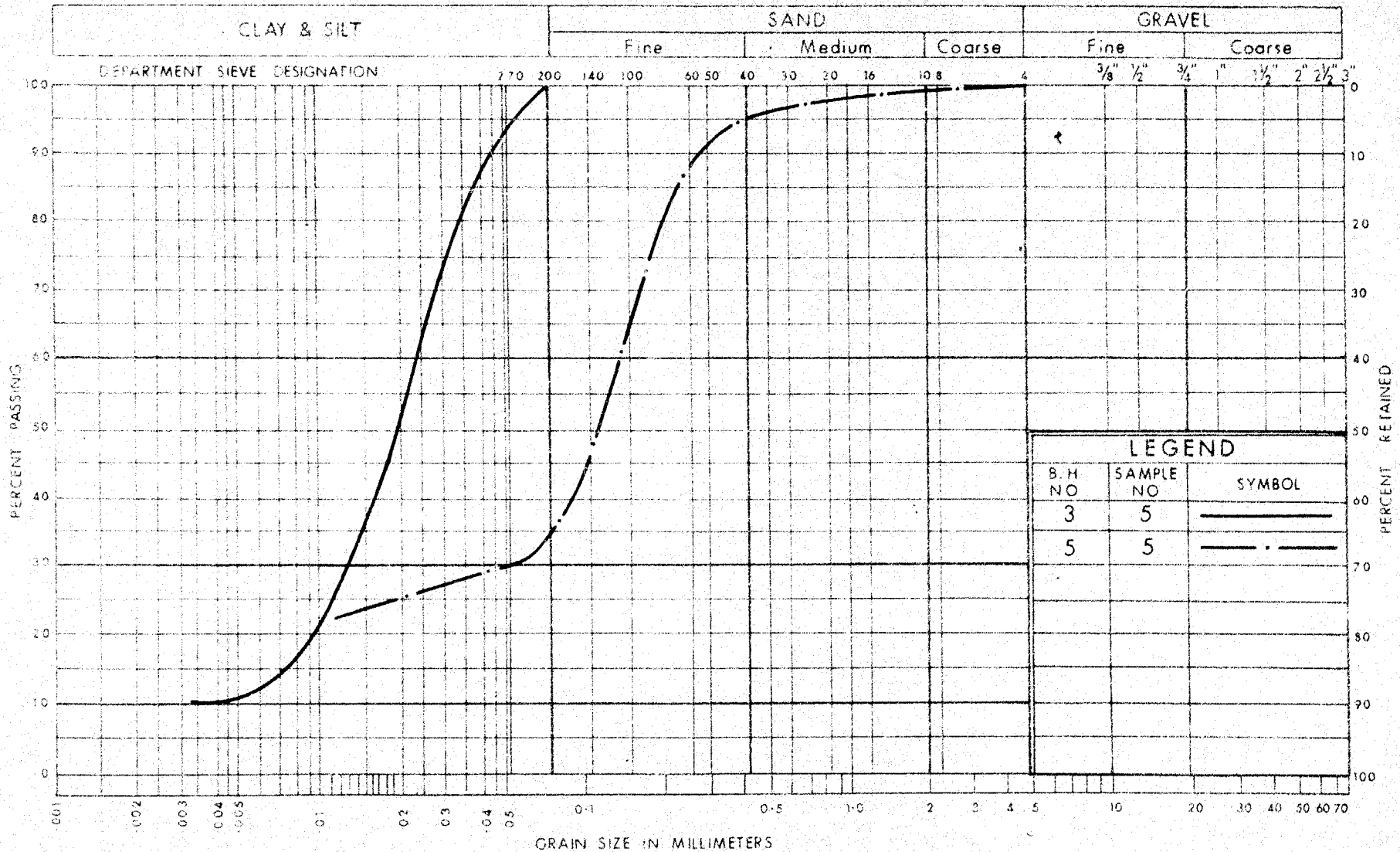
DATUM assumed

BOREHOLE TYPE Washboring, NX Casing

CHECKED BY

[illegible]

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND		
B.H. NO.	SAMPLE NO.	SYMBOL
3	5	—————
5	5	————— . —————



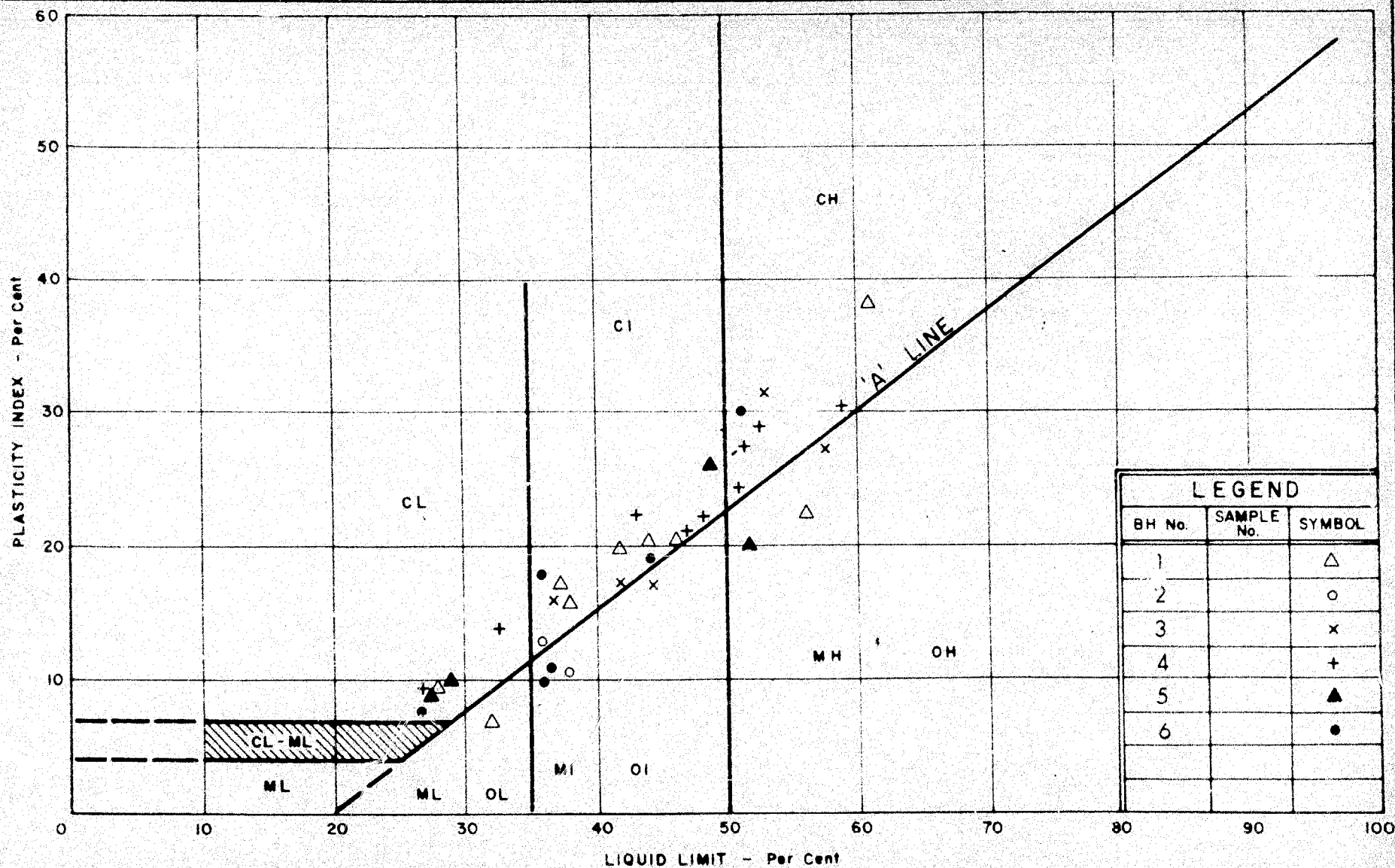
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
PROPOSED SOUTH NATION RIVER CROSSING
OF HWY NO. 417 "B" (Alt.) & HWY NO. 138

W.P. No. 35-66

JOB No. 68-F-1

FIGURE NO. 1



DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

PLASTICITY CHART
PROPOSED SOUTH NATION RIVER CROSSING
OF HWY. NO. 417 "B" (Alt.) & HWY. NO. 138

WP No. 35-66

JOB No. 68-F-1

FIGURE NO. 2

PROPOSED CROSSING HWY. 417 LINE 'B' (ALT.) SHEAR STRENGTH VS. ELEVATION

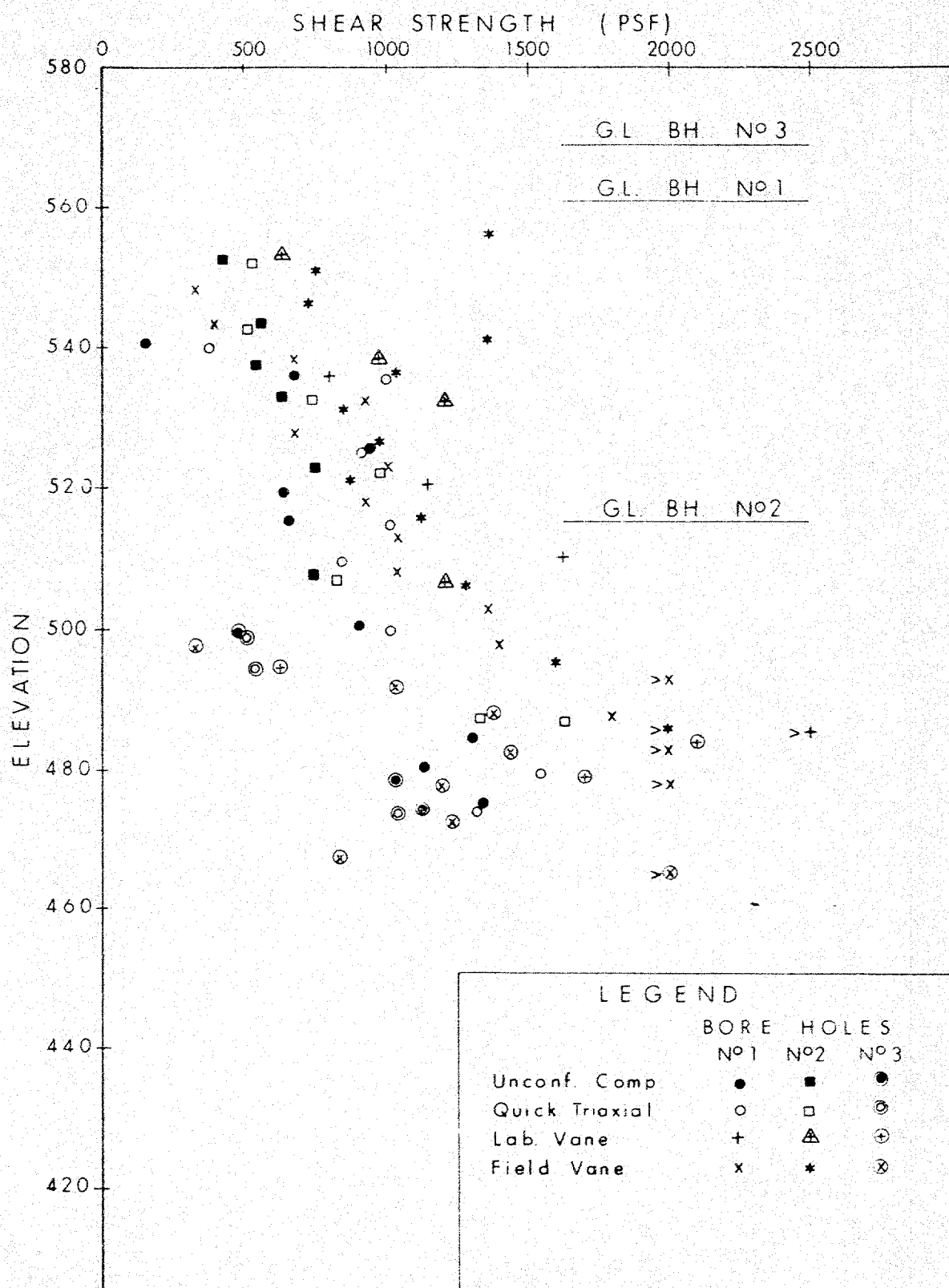


FIG. 3

PROPOSED CROSSING HWY. 138 SHEAR STRENGTH VS. ELEVATION

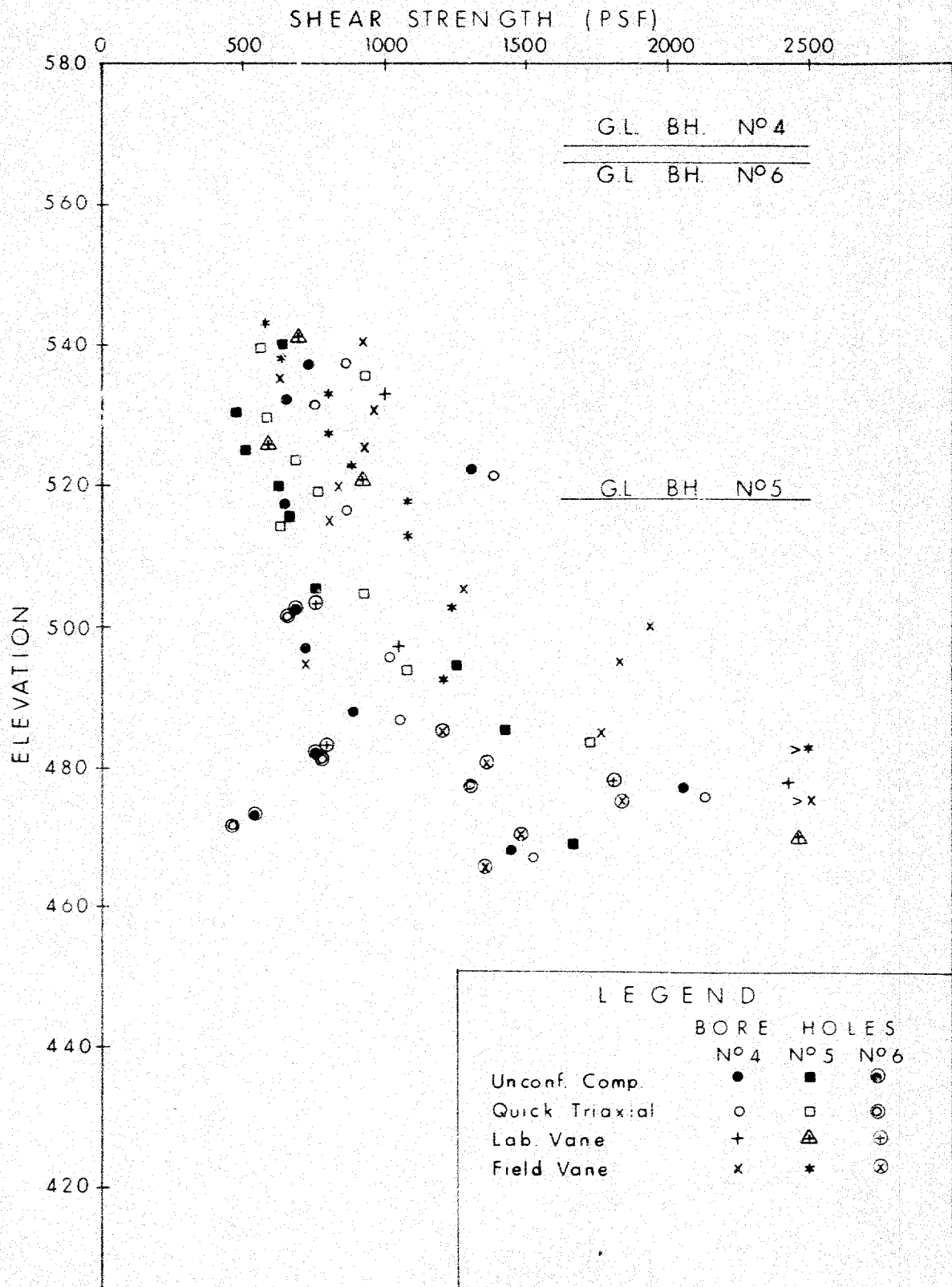


FIG. 4

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - 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 :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS 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
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
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

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 (YOUNG'S 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_0	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

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MX KINR DEC 23/71 2.15 PM

432

OTTA 1 TO J E CALLAGHAN DIST ENGR

DOWN 5 COPIES TO:

B GIROUX ESTIMATING OFFICE

A E MCKIM CONST OFFICE

A STERMAC FOUNDATION OFFICE

B MCGAFFIGAN PROGRAM OFFICE

TORD 2 COPY TO M STOYANOFF STRUCTURAL SVCS SECTION

KINR COPIES TO:

P D BILLINGS REG DIRECTOR

R J FORREST PROGRAM OFFICE

A J PERCY FUNCTIONAL PLANNING

E R SAINT M AND T

T C KINGSLAND BRIDGE OFFICE

B MCKAY ENG AUDIT

RE WP 43-67-01 HWY 43 VILLAGE OF FINCH

WP 35-66-02 HWY 17 REG RD 8 E'LY

DUE TO TIGHT PROGRAMMING PRIORITIES THE DATES FOR REGIONAL REVIEWS
FOR THE ABOVE HAVE BEEN SWITCHED.

WP 43-67-01 B'ROOM NO. 1 KINGSTON REG OFFICES 10.30 JAN 6TH

WP 35-66-02 B'ROOM NO. 1 UNTESTON REG OFFICES BQPMEP JAN 18TH

G ICM I LAN SYSTEMS DESIGN

JM

not required - grading only
Mr Kingland advised
that there is no need
to attend this meeting.
M. D.
30th Dec/71.

DEC 17 11 2:53

3374

MX KINR DEC 17/71 2.45 PM

OTTA 4 TO J E CALLAGHAN DIST ENGR

ATT M PEVERETT

DOWN 7 COPIES TO:

G STERMAC FOUNDATIONS OFFICE

B MCGAFFIGAN PROGRAM OFFICE

A E MCKIM CONSTRUCTION OFFICE

TORD 2 COPY TO M STOYANOFF STRUCTURAL CONTROL ENGR

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E SAINT M AND I

E MCKAY ENG AUDIT

J PERCY FUNCTIONAL PLANNING

P BILLINGS REG DIRECTOR

R FORREST PROGRAM SECTION

T KINGSLAND BRIDGE SECTION

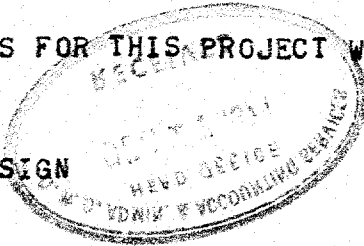
RE WP 35-66-02 HIGHWAY NO. 417 - VARS TO LIMOGES

I CONFIRM THAT THE PRE-CONTRACT REVIEW FOR THIS PROJECT WILL BE
HELD ON THURSDAY JANUARY 6TH, 1972 STARTING AT 10.30 A.M. IN
KINGSTON REGIONAL BOARDROOM NO. 1.

THE CONTRACT DOCUMENTS FOR THIS PROJECT WILL BE ISSUED BY
DECEMBER 22ND, 1971.

A E IRVING SYSTEMS DESIGN

JM



00140

1972 JAN 3 PM 1:47

MX KINR JAN 3/72 11 AM

OTTA 1 TO J E CALLAGHAN DIST ENGR

DOWN 4 COPIES TO:

B GIROUX ESTIMATING OFFICE

A E MCKIM CONST OFFICE

A STERMAC FOUNDATION OFFICE

B MCGAFFIGAN PROGRAM OFFICE

TORD 1 COPY TO M STOYANOFF STRUCTURAL SVCS SECTION

KINR COPIES TO:

P BILLINGS REG DIRECTOR

R FORREST PROGRAM SECT

A J PERCY FUNCTIONAL PLANNING

E SAINT M AND T

T KINGSLAND BRIDGE

B MCKAY ENG AUDIT

J TREW TRAFFIC SECT

RE WP 35-66-02 - HIGHWAY NO. 47 - REGIONAL ROAD 8 EASTERLY

THE REGIONAL REVIEW FOR THE ABOVE PROJECT HAS BEEN DEFERRED TO

THURSDAY JANUARY 20TH. IT WILL BE HELD IN BOARDROOM NO. 1

IN THE KINGSTON REGIONAL OFFICE STARTING AT 10.30 A.M.

A E IRVING SYSTEMS DESIGN

JM

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2043-1035
DIVISION OF HIGHWAYS
JAN 3 1972

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CABLE ADDRESS

ADRESSE TÉLÉGRAPHIQUE

PLEASE QUOTE FILE NO M43-17-13C-32

NO DE DOSSIER À RAPPELER

NATIONAL RESEARCH COUNCIL OF CANADA
CONSEIL NATIONAL DE RECHERCHES DU CANADA

DIVISION OF BUILDING RESEARCH
DIVISION DES RECHERCHES EN BATIMENT

OTTAWA 7.
K1A 0R6

24 June 1971

Slope Stability Problems -
National Capital Region

Attached to this note is a brief record of a meeting on the above topic at DBR-NRC June 3, 1971. If, in the process of summarizing the meeting, the sense of your remarks is not as you had intended, please inform the undersigned and the necessary corrections will be made.

W.J. Eden,
Geotechnical Section.

WJE/jo
Attachment

Copy for: Mr. M. Devata

RECORD OF MEETING
RE SLOPE STABILITY STUDIES IN
NATIONAL CAPITAL REGION

A meeting was convened on Thursday 3 June 1971
at 10.00 a.m. at the Geotechnical Section, Division of Building
Research, to discuss studies on slope stability of clay slopes in
the National Capital Region. Attending were:

R. J. Mitchell	-	Queen's University	
M. Devata)	Ontario Department of Highways now	
A. G. Stermac)	-	called Ontario Dept. of Transport &	
K. Torrance	-	Carleton University	Communication
J. D. Scott	-	Ottawa University	
P. J. Williams	-	Carleton University	
L. W. Gold	-	Geotechnical Section, DBR	
A. Dascal	-	Hydro Quebec	
G. C. McRostie	-	Consulting Engineer	
E. B. Fletcher	-	Carleton University	
J. S. Scott	-	Geological Survey of Canada (until noon)	
W. J. Eden	-	Geotechnical Section, DBR	
C. B. Crawford	-	Assistant Director, DBR, who acted as	
		Chairman	

The Chairman, with general agreement, proposed

the following agenda for discussion:

- (1) Regional studies which have been conducted or that
are under way
- (2) Specific studies currently under way

- (3) The problem of flowslides involving large areas
- (4) The South Nation River Landslide, 16 May 1971
- (5) A means of continuing communication for future studies

(1) Regional Studies

John Scott described the efforts of the Geological Survey of Canada in the study of the regional geology of the National Capital Region. The main objective of this study was to demonstrate mapping methods and information storage and retrieval systems. Attempts were being made to obtain all existing information. Two students were undertaking to put such records on standard forms for computer use. It was hoped the study would be completed by 1973. The study would include the surface geology, both surficial and bedrock, and the depth information available from boring logs. Seismic work would be conducted to supplement boring information where necessary.

Gordon McRostie described the study which his firm was commissioned to do for the Conservation Authorities Branch of the Ontario Department of Energy and Resources Management in 1968. The study was to indicate the problems raised by

the presence of sensitive clay soils in the Ottawa-Carleton Region. Among the problems identified was that of slope stability. Clay areas which had slopes steeper than 15° more than 20 feet high were identified on maps as areas where there were potential slope stability problems.

Peter Williams described work by graduate students in the Faculty of Geography at Carleton University. Types of mapping and mapping criteria were being examined, detailed airphoto studies are being conducted on slopes in three specific locations and the slopes along the Mud Creek ravine are being studied in detail. In addition, negotiations are under way with the City of Hull concerning mapping of undeveloped land north of the city with the view to possible zoning regulations based on slope stability considerations.

Oscar Dascal described Hydro Quebec's studies on the shores of the Ottawa River from Hawkesbury to Ottawa. Since the raising of the river level at the Carillon Dam in 1962, numerous landslides have occurred along the river banks. Several damage claims have been made against Hydro Quebec by landowners. Thus Hydro Quebec have, since 1969, done very detailed surveys of a number of sites to measure the rate of erosion and to document slope

failures. Airphotos are taken annually to assist the studies.

These studies will be continued.

(2) Analysis of Slopes

Studies by DBR had been largely reported in the literature. Groundwater observations were being continued at a number of sites in the Ottawa region.

Bob Mitchell reported on the measurements of slope movement with inclinometer tubes. These had been installed in 1969 when he was employed by DBR. Measurements were being continued. Annual movements in some slopes of $\frac{1}{2}$ centimetre had occurred.

Tony Stermac reported on studies conducted for D. H. O. at the South Nation River north of Casselman. Eight possible crossing sites had been investigated, all indicating problems with obtaining stable slopes. Slope stability improvement was considered by means of grading.

Bryan Fletcher stated that he had been asked by the City of Hull to look at two specific sites with unstable slopes. He had submitted a proposal to the City regarding a detailed study and was hopeful that the proposal would be accepted.

Ken Torrance reported he was planning to continue his studies on the geochemistry of clays which had started during his stay in Norway. He was establishing laboratory facilities and hoped to start field studies in the Ottawa area shortly. He made reference to the paper by Sangrey and Paul* where clays were typed by the Na/Ca ratio.

(3) Flowslides involving Large Areas

Gordon McRostie raised the problem of large flowslides. Current slope stability theories only dealt with the first part of an earthflow. There were no criteria to indicate that a landslide might retrogress and if so, how far. For example, none of the studies on the Nation River could take into account the large scale movement which occurred. Two extremely large landslide scars were known to exist - Alfred area and the Quyon area - in the Ottawa Valley. The existence of these large flows indicate the need for more research on landslides.

*

D. A. Sangrey and M. J. Paul. "A Regional Study of Landsliding near Ottawa". Can. Geot. Journal, Vol. 8, No. 2, pp. 315-335, May 1971

(4) South Nation River Landslide, 16 May 1971

Mr. Stermac reported on some of the activities undertaken to investigate this landslide. Bryan Fletcher was engaged to investigate a river crossing immediately upstream of the landslide. Stability studies of the crossing site were actively being pursued at the time of the landslide. Since the landslide, detailed airphoto coverage had been flown and D. H. O. (now Department of Transportation and Communication) were considering the production of detailed contour maps both before and after the landslide. Queen's University, under Bob Mitchell, had a field crew on the site, installing piezometers, conducting vane tests and sampling. DBR indicated an active interest in lending assistance to the investigation.

It was agreed that Messrs. Eden, Fletcher and Mitchell would cooperate in the preparation of a brief description of the South Nation River landslide for publication in the August issue of the Canadian Geotechnical Journal. (This has been done.)

(5) Liaison with regard to Future Studies

The problem of keeping in touch with interested agencies and individuals and of obtaining funds and other support

for future studies was discussed. Finally it was suggested that the Ottawa Geotechnical Group be approached with the view of establishing a Task Group on Landslide Studies. This Group would foster communication between interested parties in slope studies and would lend its support to worthy applications and proposals for future studies.

Meeting adjourned at 3.00 p.m.

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Kingston

KINR DOWN 2 APRIL 24/68 10.54 A VR

J L FORSTER REG FUNCT PLANNING ENGR

COPY TO S MCCOMBIE BRIDGE PLANNING ENGR DOWN

RE SOUTH NATION RIVER CROSSING LINE B (ALTERNATE) AND

HIGHWAY 138 DISTRICT 9 OTTAWA

THE REPORT FOR THE ABOVE CROSSING WAS COMPLETED AND SENT OUT
ON MARCH 22/68 FOUR PERSONS IN KINGSTON RECEIVED IT S J MARKIEWISZ
C R ROBERTSON G SCOTT AND J D GRUSPIER WE ARE NOW ARRANGING FOR
A COPY TO BE SENT TO YOU BY MR S MCCOMBIE IN THE MEANTIME WOULD
PLEASE USE ONE OF THE COPIES PRESENTLY AVAILABLE IN THE REGION.

A G STERMAC PRINCIPAL FOUND ENGR MATLS AND TESTG DIV

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer
Materials and Testing Division,
Downsview.

FROM: Functional Planning Division,
Kingston.

DATE: April 22, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

South Nation River Crossing, Line 'B' (Alternate) and
Highway 138, District 9 - Ottawa

68-F-1

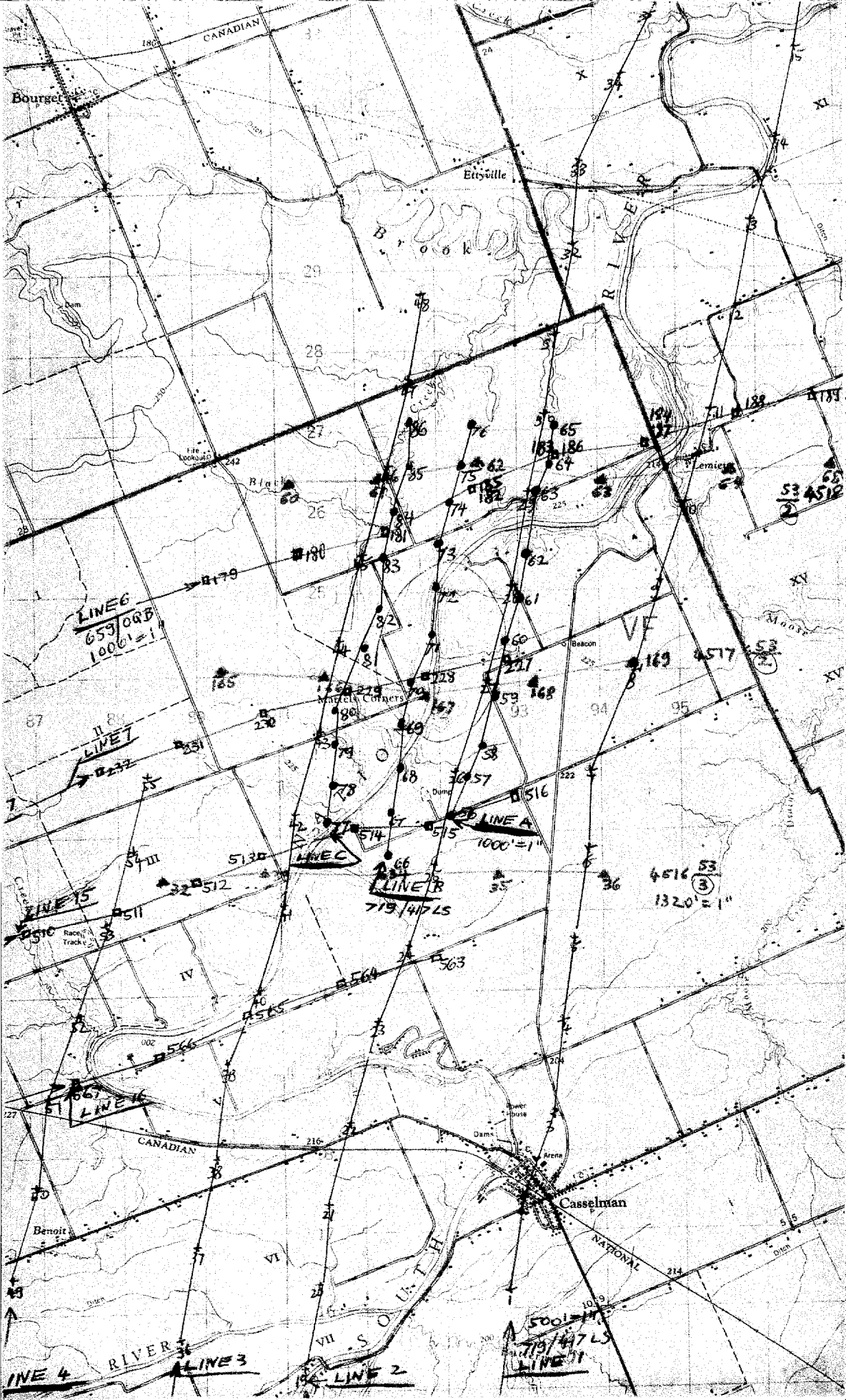
I understand that you will soon be completing your foundation report for the above crossings. I would appreciate it if you would add my name to your mailing list since the report is mainly for the use of Functional Planning.

As you understand it is most urgent that we receive a copy of this report as soon as possible so we may finalize the alignment of Highway 417 and prepare our Head Office presentation for this proposed freeway.

J. L. Forster
J. L. Forster,
Regional Functional Planning Engineer

MJM/cam

c. c. G. Scott
S. McCombie

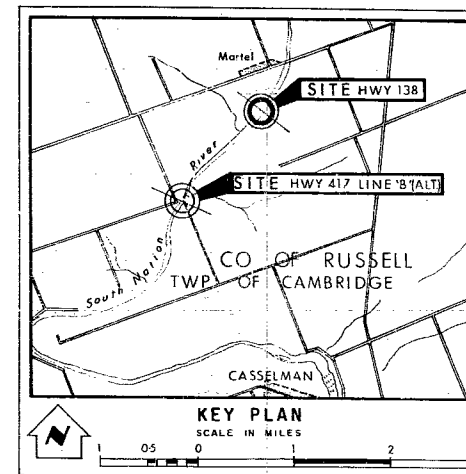
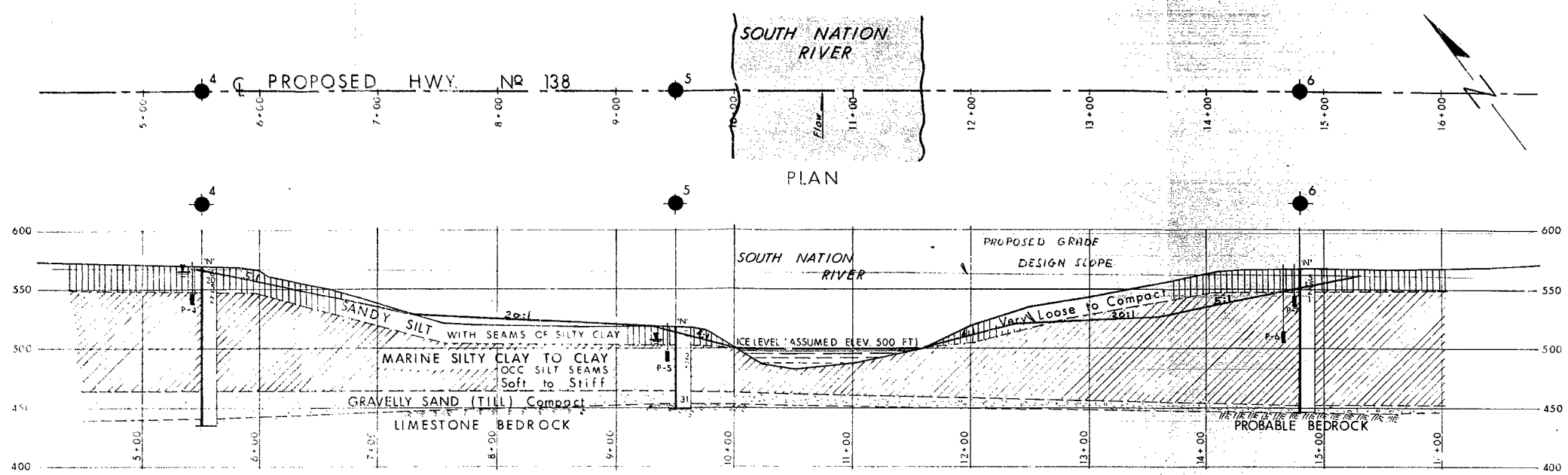


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W.P. #35-66

HWY #4178/38

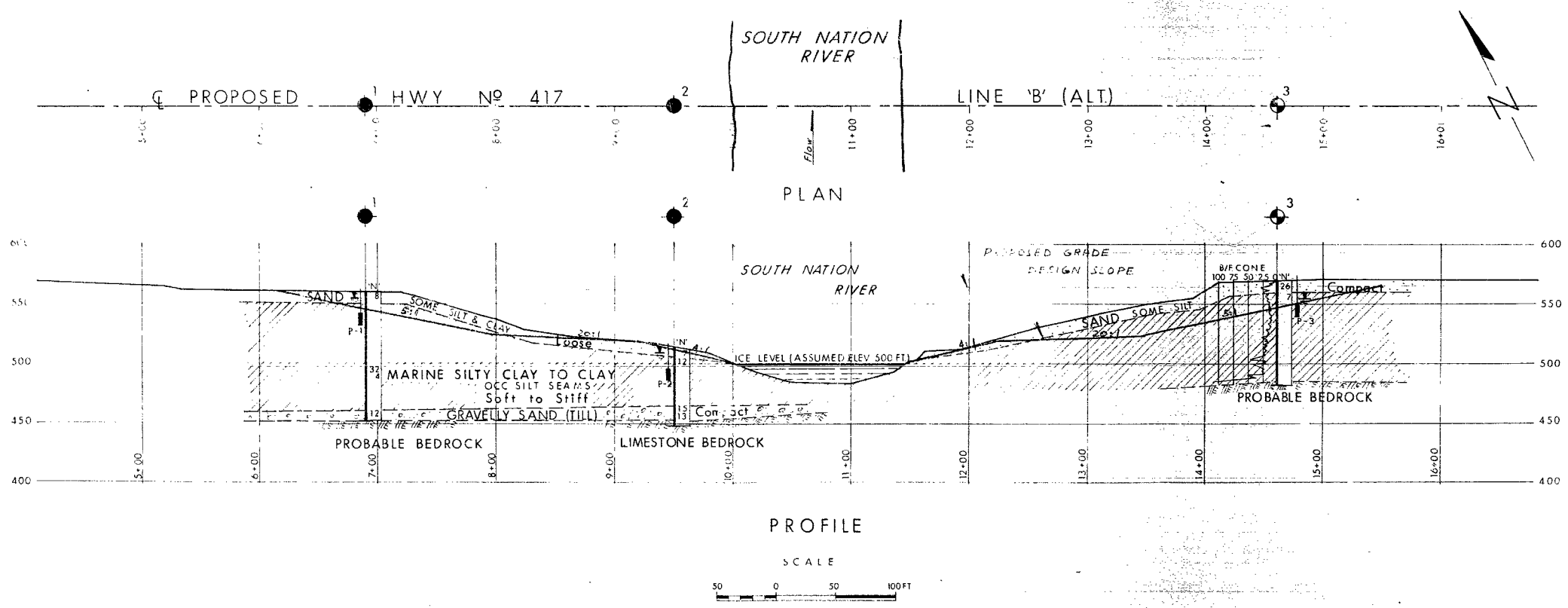
SOUTH NATION
RIVER



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation, JAN. 1968		
	Piezometer		

NO.	ELEVATION	STATION	OFFSET
1	561.1	6+90	Q HWY 417
2	515.5	9+50	Q HWY 417
3	569.2	14+00	Q HWY 417
4	568.5	5+50	Q HWY 138
5	518.4	9+50	Q HWY 138
6	566.1	14+80	Q HWY 138

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



NO.	FOR	DATE

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & TESTING DIVISION - FOUNDATION SECTION			
SOUTH NATION RIVER			
KING'S HIGHWAY NO. 138 & 417 LINE 'B' (ALT) DIST. NO. 9			
CO. ROUSSEL			
TWP. CAMBRIDGE	LOT	CON.	
BORE HOLE LOCATIONS & SOIL STRATA			
SUBM'D. A.B.	CHECKED	W.P. NO. 35-66	M.S.T. DRAWING NO.
DRAWN a.b.	CHECKED	JOB NO. 68-F-1	68-F-1A
DATE M.A.R. 12, 1968	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>	CONT. NO.		