

GEOCRES No. 41J-29DIST. 18 REGION W.P. No. 903-72-02/03CONT. No. W. O. No. STR. SITE No. HWY. No. 17LOCATION SAULT STE MARIE FEASIBILITY  
STUDY (FROM KENSINGTON PT RD. TO MCKNIGHT  
ROAD)No of PAGES - OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:



Ministry of  
Transportation and  
Communications

## Memorandum

To: Mr. W. L. Lees (2)  
Systems Design  
Northwestern Region  
Thunder Bay, Ontario

From: Soil Mechanics Section  
Geotechnical Office  
Downsview, Ontario

Attention:

Date: April 2, 1975

Our File Ref. W.P. 903-72-01

In Reply to

Subject:

41 J-29

GEOCRE No.

PRELIMINARY  
FOUNDATION INVESTIGATION REPORT  
FOR FEASIBILITY STUDY OF  
T.C. HWY. 17 (PROPOSED 4 LANES) FROM  
KENSINGTON POINT RD. TO McKNIGHT RD.  
DISTRICT #18 (SAULT STE. MARIE)  
W.P. 903-72-03 *ok*

Due to the urgency of this project we are forwarding to you a report containing the subsoil description and the recommendations which are based on the information from the field, transmitted by telephone. No laboratory tests have been carried out to determine the engineering properties of the subsoil. However, in our opinion, the data contained in this report will be adequate for your evaluation with regard to feasibility studies on this project. (We will be forwarding to you the preliminary data for the other sections in the very near future.)

It should be noted that the recommendations given in this report are of a preliminary nature. A complete foundation investigation will be necessary once the alignment and geometrics are finalized.

*M. Devata*

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PRELIMINARY FOUNDATION INVESTIGATION REPORT  
FOR FEASIBILITY STUDY OF  
T.C. HWY. NO. 17 (PROPOSED 4 LANES)  
FROM KENSINGTON POINT ROAD TO McKNIGHT ROAD  
DISTRICT #18 (SAULT STE. MARIE)  
W.P. 903-72-03  
\*\*\*\*\*

The Soil Mechanics Section has recently completed the field work for the feasibility study of the following:

1. Kensington Point Road Interchange
2. Structure over Desbarats River
3. McKnight Road Interchange

The field work consisted of 2 Boreholes at each site for a total of 6 Boreholes. At the Desbarats River and McKnight Road sites, the boreholes were accompanied by dynamic cone penetration tests.

In the following subsections, each site is considered individually. For each site, a brief description of subsoil conditions and our preliminary comments regarding the structure foundations and stability of embankments are given.

I. KENSINGTON POINT ROAD INTERCHANGE (BHs #3 & 4)

1. Subsoil Conditions

The subsoil, in general, consists of a deposit of silty clay to clay overlying bedrock. In Borehole #3, which was put down in the vicinity of the proposed structure for the future interchange, bedrock was proven by obtaining a BX core from 58.5 to 62.0 ft. below ground surface. In this borehole, a 5 ft. thick layer of clayey silt with sand and gravel and occasional cobbles was encountered below the topsoil. Borehole #4 was put down some

300 ft. west of Walker Creek in order to investigate the stability of the proposed embankment in this area. It was terminated at a depth of 34.5 ft. in the silty clay to clay stratum.

The in-situ vane tests indicate that in the upper 12 ft. of the silty clay to clay deposit, the undrained shear strength decreases from about 2000 p.s.f. to 500 p.s.f. Below 12 ft., the undrained shear strength gradually increases with depth to about 1000 p.s.f. at a depth of 50 from the top of this stratum.

The groundwater level at the time of investigation was 6 ft. below ground in Borehole #3 and at ground surface in Borehole #4.

## 2. Discussion and Recommendations

It is proposed to construct an interchange at the crossing of Highway 17 and Kensington Point Road. At this location Hwy. 17 will be realigned slightly to the north. The Interchange will require embankments up to 25 ft. in height.

Our recommendations for the structure foundation and the associated embankments are given in the following subsections:

### 2.1 Structure Foundations

Due to the following reasons, spread footing type foundations in the original ground are not recommended at this site:

- (i) the shear strength of the silty clay to clay deposit in the upper portion decreases very rapidly to about 500 p.s.f. at a depth of 12 ft. This would allow a boring capacity of less than 1 t.s.f., which is impractical.
- (ii) the silty clay to clay stratum is compressible and relatively deep (about 60 ft.). The footings will settle excessively in this material when subjected to loading.

It is recommended that the entire structure be supported on end-bearing steel piles down to bedrock, about 60 ft. below the ground level.

Perched abutments founded on spread footings placed in compacted granular fills are not recommended, because of excessive settlements of the fills. The footings placed in the fills will settle by the same amount as the fills themselves.

## 2.2 Embankments

### 2.2.1) General

The Interchange will require embankments up to 25 ft. in height. These fills will be underlain by 60 ft. of generally soft to firm silty clay to clay stratum.

The presence of this compressible and cohesive stratum requires that steps must be taken to ensure the overall stability of the fill sections, as well as limit the settlements to a tolerable magnitude.

### 2.2.2) Stability Conditions

A very preliminary analysis in terms of total stress, to determine the stability of the fill sections, indicates that:

#### A. For Granular Fill ( $\gamma = 125$ p.c.f., $\phi = 30^\circ$ )

1. Fills up to 16 ft. in height, with 2:1 slopes will be stable.
2. Fills in excess of 16 ft but less than 30 ft., in height, will require counter-balancing mid-height berms.
3. The mid-height berm will be 6 ft. long for every 1 ft. height in excess of the safe height of 16 ft.

B. For Light-Weight Fill ( $\gamma = 90$  p.c.f.,  $\phi = 30^\circ$ )

1. Fills up to 20 ft. in height with 2:1 slopes will be stable.
2. Fills in excess of 20 ft., but less than 36 ft. in height, will require counter-balancing mid-height berms.
3. The mid-height berm will be 5 ft. long for every 1 ft. height in excess of the safe height of 20 ft.

2.2.3) Settlement Considerations

The underlying cohesive subsoil is compressible and will undergo settlements due to consolidation over a long-term period, under the weight of the embankments. Settlements have been estimated, using the test results on soil in this general area, and are as follows:

<u>Height of Fill</u>	<u>Estimated Settlements</u>	
16 ft. (granular fill)	}	12 ins.      1-2 yrs.
20 ft. (light-weight fill)		24 ins.      25 yrs.
25 ft. (granular fill)		18 ins.      1-2 yrs.
		36 ins.      25 yrs.
25 ft. (lightweight fill)		15 ins.      1-2 yrs.
		30 ins.      25 yrs.

It would be advantageous to construct the embankments first and leave them in place for as long a period as possible prior to constructing the structure.

## II. DESBARATS RIVER STRUCTURE (BHs #1 & 2)

### 1. Subsoil Conditions

Borehole #1 was put down on the west side of the river and Borehole #2 on the east side of the river. The area in the immediate vicinity of the proposed structure is flat, low lying and swampy. However, about 500 ft. south east of the site, the ground gradually rises in the form of a hill about 100 ft. high.

The subsoil conditions in the two boreholes are as follows:

BH #1 (GL 580)	BH #2 (GL 580)	
0 - 1 ft.	0 - 1 ft.	Ice
1 - 4 ft.	1 - 3 ft.	Organic material (muskeg)
4 - 9 ft.	3 - 10 ft.	Organic clay, very soft
9 - 58 ft.	10 - 26 ft.	Silty clay to clay, very stiff to soft becoming stiff with depth
	26 - 30 ft.	Silty sand, some gravel and clay, compact.
58 ft.	30 ft.	Bedrock - Rhyolite.

In Borehole #2, the thickness of the silty clay to clay stratum is smaller, and the shear strength is slightly greater than in Borehole #1.

In Borehole #1, bedrock was proven by obtaining BX rock core. The bedrock was identified as sound rhyolite.

From the Borehole information, it is evident that the bedrock at the proposed structure location slopes away from the hill (i.e. from BH#2 towards BH#1). The borehole and cone penetration test data indicates that the bedrock surface is very uneven. There are numerous rock outcrops in this general area.

It is understood that the Desbarats River is about 7 feet deep.



## 2. Discussion and Recommendations

It is proposed to construct a structure to carry Hwy. 17 over the Desbarats River. The new crossing will be some 1500 ft. downstream (south west) of the existing crossing.

The final grade of highway is not known. There is a marina immediately downstream of the present crossing of Hwy. 17 and Desbarats River. This marina is very busy, and there are plans to expand the facility. Therefore the waters of Desbarats River south of the existing bridge at Hwy. 17, may be classified by the Federal Government as navigable channel. In this case, the height of approach embankments will be in the order of 20 ft. above the prevailing ground level. However, if the channel is not classified as navigable, then the height of approach embankments will be in the order of 10 ft.

### 2.1 Structure Foundation

The upper 9-10 ft. of subsoil consists of muskeg and very soft organic clay overlying a silty clay to clay deposit. Therefore, spread footing type foundations in original ground are not practical. Bedrock was encountered at depths varying from 26 to 58 ft. Therefore, the entire structure may be supported on end-bearing steel piles driven to bedrock. The bedrock, at places, slopes very steeply, and the piles in that area may require Oslo tips. However, this will be determined during the final investigation. A dewatering scheme will be required to construct the pile caps.

As an alternative, the entire structure may be supported on 36" diameter caissons drilled into bedrock. The caissons may be extended to act as piers. In this case, no dewatering scheme will be necessary. For estimating purposes, a design load of 200 tons/caisson may be assumed.

## 2.2 Approach Embankments

### 2.2.1) General

The final grade of Hwy. 17 may require approach embankments up to 20 ft. in height above the prevailing ground level.

Immediately below ground surface the upper 3 - 4 feet consists of muskeg followed by 5 - 7 feet of very soft organic clay with roots. The undrained shear strength of the organic clay layer, as determined from the field vane tests, is about 150 p.s.f. (Thus, it is seen that the upper 9 - 10 feet of subsoil is organic, very soft and very compressible). In order to construct embankments, it will be necessary to remove this material, and replace it with suitable granular material. This may be achieved by means of:

- ( i) excavation; or
- ( ii) displacement; or
- (iii) blasting.

The most suitable method can be decided when the final grade and extent of the deposit are known. This will require further investigation.

The following comments regarding the stability and settlements are based on the assumption that the above-mentioned soft material has been removed to its full extent underneath the embankment and replaced with suitable granular material.

### 2.2.2) Stability Considerations

A comparison of Boreholes #1 and 2 shows that subsoil conditions in Borehole #2 (on the east side of the river) are more favourable than in Borehole #1 (on the west side of the river). The shear strength in Borehole #2 is somewhat greater than in Borehole #1 and the thickness of the stratum is smaller in Borehole #2.

It is estimated that on the east side, embankments up to 20 ft. in height with 2:1 side slopes built with granular material ( $\phi = 125$ ,  $\phi = 30^\circ$ ) will be safe. However, on the west side the

safe height of embankments will be about 16 ft. The height of embankment is measured from toe of the slope to the top of the slope. The height of embankment measured in the longitudinal direction will be greater than in the transverse direction, because in the former case the height is measured from the river bed, while in the latter case, it is measured from the prevailing ground level. Embankments higher than the safe heights, will require mid-height berms to ensure their stability. For every 1 ft. height in excess of the safe height, the length of berm will be 5 ft.

If light weight aggregate ( $\gamma = 90$  p.c.f.) is used for fill material, then the safe height of fill with 2:1 slopes will be 25 ft. on the east side and 20 ft. on the west side. Beyond this height, stabilizing berms will be required. The mid-height berm will be 5 ft. long for every 1 ft. height in excess of the safe height.

### 2.2.3) Settlement Considerations

The underlying cohesive subsoil is compressible and will undergo settlements due to consolidation, over a long term period, under the weight of the embankment. Using the test results on soils in this general area, the following settlements are estimated under 20 ft. high fills:

<u>Type of Fill</u>	<u>Estimated Settlements</u>		
	<u>East Side</u>	<u>West Side</u>	<u>Time Period</u>
Granular Fill	9 - 12 ins.	12-15 ins.	1 - 2 yrs.
	18 - 24 ins.	24-30 ins.	25 yrs.
Light weight fill	7 - 9 ins.	9-12 ins.	1 - 2 yrs.
	14 - 18 ins.	18-24 ins.	25 yrs.

It would be advantageous to construct the embankments in advance and leave them in place for as long a period as possible prior to constructing the structure.

### III. MCKNIGHT ROAD INTERCHANGE (BH<sup>S</sup> #5 and 6)

#### 1. SUBSOIL CONDITIONS

The subsoil conditions in both boreholes are similar and consist of a deposit of silty clay to clay with occasional silt layers overlying bedrock. In the upper 10 ft. of the overburden, the undrained shear strength, as determined from the field vane tests, decreases from 1200 p.s.f. to 400 p.s.f. The higher shear strength near the ground surface may be due to desiccation or freezing in the winter. Below this level, the undrained shear strength is, in general, about 500 p.s.f. Probable bedrock was encountered at depths of 28.0 and 22.5 ft. in Boreholes 5 and 6 respectively. Bedrock was not proven but was assumed to be at a level where refusal to augering or driving a split spoon was met.

There are numerous rock outcrops in this general area.

#### 2. DISCUSSION AND RECOMMENDATIONS

##### 2.1) General

It is proposed to construct an interchange at the crossing of Hwy. 17 and McKnight Road. The structure will carry McKnight Road over Hwy. 17. The associated fills will be 30-35 ft. high.

##### 2.2) Structure Foundation

Because the shear strength of the silty clay to clay stratum is relatively low, consequently, the bearing capacity of the overburden is not adequate to support spread footing type foundations in the original ground. Therefore, it is recommended that the entire structure be supported on end-bearing steel piles driven to bedrock.

##### 2.3) Embankment Fills

###### 2.3.1) General

The proposed grade of McKnight Road is such that it will

require embankments up to 30-35 ft. in height. These fills will be underlain by 25-30 ft. of generally soft to firm silty clay to clay stratum. This requires that steps must be taken to ensure the overall stability of the fill sections as well as limit the settlements to a tolerable magnitude.

### 2.3.2) Stability Considerations

A very preliminary analysis in terms of total stress, to determine the stability of the fill sections, indicates that:

#### A. For Granular Fill ( $\gamma = 125$ p.c.f.)

1. Fills up to 16 ft. in height with 2:1 slopes, will be stable.
2. For a fill height of 30 ft., a single mid-height berm of 60 ft. will be required.
3. Fills in excess of 30 ft. in height will require multiple berms.

#### B. For Granular Fill ( $\gamma = 90$ p.c.f.)

1. Fills up to 20 ft. in height with 2:1 slopes, will be stable.
2. For a fill height of 35 ft., a single mid-height berm of 60 ft. will be required.

In order to minimize the embankment heights and consequently the berm requirements and associated settlements, consideration should be given to a longer multi-span structure over this area. It may be advantageous to minimize the berm requirements by keeping the grade of McKnight Road as low as possible.

### 2.3.3) Settlement Considerations

The underlying cohesive subsoil is compressible and will undergo settlements due to consolidation over a long-term period, under the weight of embankment. Using the test results on soils

in this general area, the following settlements are estimated:

<u>Height of Fill</u>	<u>Fill Material</u>	<u>Estimated Settlement</u>	<u>Time Period</u>
16 ft.	Granular	8 ins.	1-2 yrs.
20 ft.	Light Weight	16 ins.	25 yrs.
30 ft.	Granular	15 ins.	1-2 yrs.
		30 ins.	25 yrs.
35 ft.	Light Weight	12 ins.	1-2 yrs.
		24 ins.	25 yrs.

It would be advantageous to construct the embankments first and leave them in place for as long as possible, prior to constructing the structure.

#### IV. MISCELLANEOUS

The field work was carried out under the supervision of Messrs. H. Shah, M. MacLean and C. McKercher, Project Engineers, during the period March 11-27, 1975, using the equipment owned and operated by Master Soil Investigation Ltd.

*A. Prakash*

A. PRAKASH, P. Eng.  
Senior Engineer

AP/sah

A P P E N D I X

## 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 :-

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

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

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	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
$\tau_f$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

## GENERAL

$\pi$	$= 3.1416$
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
$\sigma$	NORMAL STRESS
$\sigma'$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

## EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	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

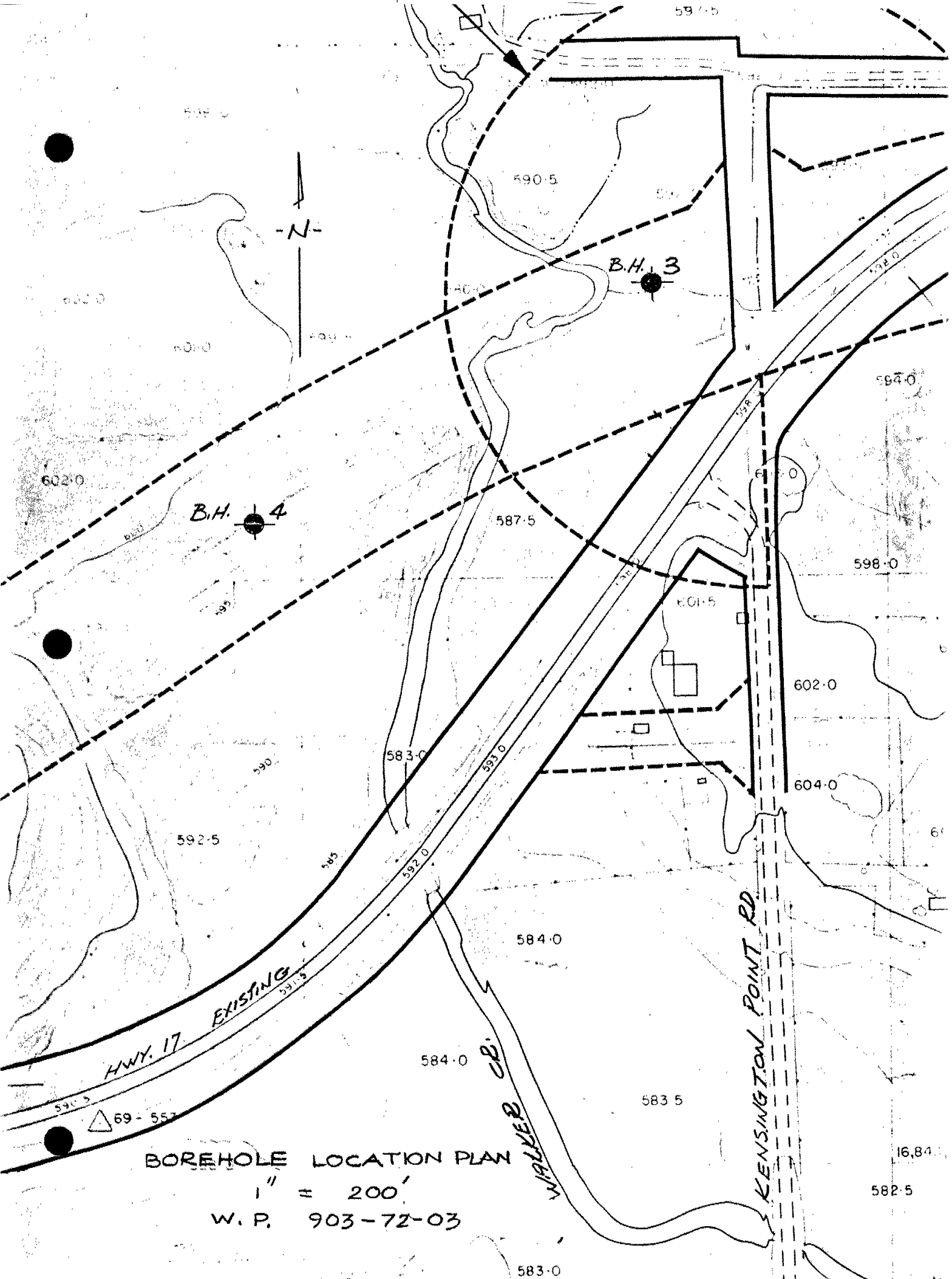
## FOUNDATIONS

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
$\beta$	ANGLE OF SLOPE TO HORIZONTAL

KENSINGTON POINT ROAD INTERCHANGE



BOREHOLE LOCATION PLAN

1" = 200'

W.P. 903-72-03

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

W.P. 903-72-03 LOCATION HWY. 17 & KENSINGTON POINT ROAD (as shown on plan) ORIGINATED BY HS  
DIST. 18 HWY. 17 BORING DATE MARCH 15-17, 1975 COMPILED BY VK  
DATUM GEODETIC BOREHOLE TYPE AUGER & SAMPLE WITH C.M.E. 55 H.S. CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$		UNIT WEIGHT $\gamma$	REMARKS  % GR. SA. SI. CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		SHEAR STRENGTH					WATER CONTENT %			
							20 40 60 80 100					$w_p$ $w$ $w_L$			
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 400 800 1200 1600 2000									
590.0	Ground Level														
0.5	Clayey Silt with Sand and Gravel Occ. cobbles		1	AS	-										
			2	SS	7										
5.5	Stiff		3	SS	2	580									
	Firm		4	SS	1										
	Soft		5	TW	PH	570									
			6	SS	1										
	Silty		7	SS	1	560									
	Clay		8	SS	1										
	Firm		9	TW	PH	550									
	Clay		10	VS	-										
			11	VS	-	540									
	Stiff		12	SS	1										
531.5			13	WS	-										
58.5	Bedrock		14	BX	100% Rec	530									
528.0	Sound			RC											
62.0	End of Borehole														

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

## RECORD OF BOREHOLE NO 4

W.P. 903-72-03 LOCATION HWY. 17 KENSINGTON POINT RD. (AS SHOWN ON PLAN) ORIGINATED BY H.S.  
 DIST. 18 HWY. 17 BORING DATE March 18, 1975 COMPILED BY VK'  
 DATUM GEODETIC BOREHOLE TYPE AUGER & SAMPLE WITH C.M.E. 55 (H.S.) CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			W. GROUND WATER L. ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_p$ WATER CONTENT ——— $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20    40    60    80    100					$w_p$ $w$ $w_L$				
							SHEAR STRENGTH					WATER CONTENT %				
							○ UNCONFINED                      + FIELD VANE ● QUICK TRIAXIAL                  x LAB VANE									
						400	800	1200	1600	2000					% GR SA. SI. C	
597.0	GROUND LEVEL															
TOP SOIL																
1.0	V. Stiff Stiff Firm Silty Clay to Clay Soft Firm		1	SS	11											
			2	SS	3											
			3	TW	PH											
			4	SS	1/18"											
			5	SS	1/18"											
			6	SS	1/18"											
			7	SS	1/18"											
			8	TW	PH											
562.5																
34.5	End of Borehole					560										

STRUCTURE OVER DESBARATS RIVER



## RECORD OF BOREHOLE No 1

W.P. 903-72-03 LOCATION HWY. 17 & DESBARATS RIVER (as shown on plan) ORIGINATED BY MM  
 DIST. 18 HWY. 17 BORING DATE MARCH 11-13, 1975 COMPILED BY AP  
 DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER, Bx CASING, Bx CORE & CONE CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		SHEAR STRENGTH					WATER CONTENT %				
							O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					$w_p$ — $w$ — $w_L$				
						10	20	30	40	50						
						400	800	1200	1600	2000						
580±	Ground Level															
579	Ice															
1.0	Muskeg		1	SS	1											
576																
4.0	Organic Clay		2	SS	0											
571	With Roots Very Soft															
9.0	Stiff to Very Stiff		3	SS	3	570										
	Firm		4	TW	PH											
	Soft		5	SS	0	560										
	Firm		6	TW	PH											
	Stiff		7	SS	1	550										
	Silty Clay to Clay															
			8	TW	PH											
			9	SS	3	540										
			10	SS	5	530										
522																
58.0	Bedrock Sound		11	BX RC	75%	520										
			12	BX RC	100%											
511.2			13	RC	100%											
68.8	End of Borehole					510										
Note: The original borehole was terminated at a depth of 53.5 ft., when bedrock was encountered. The hole was blocked by the broken field vane. Rock coring was done in a separate hole, about 8 ft. east of the original borehole.																

Note: The original borehole was terminated at a depth of 53.5 ft., when bedrock was encountered. The hole was blocked by the broken field vane. Rock coring was done in a separate hole, about 8 ft. east of the original borehole.



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

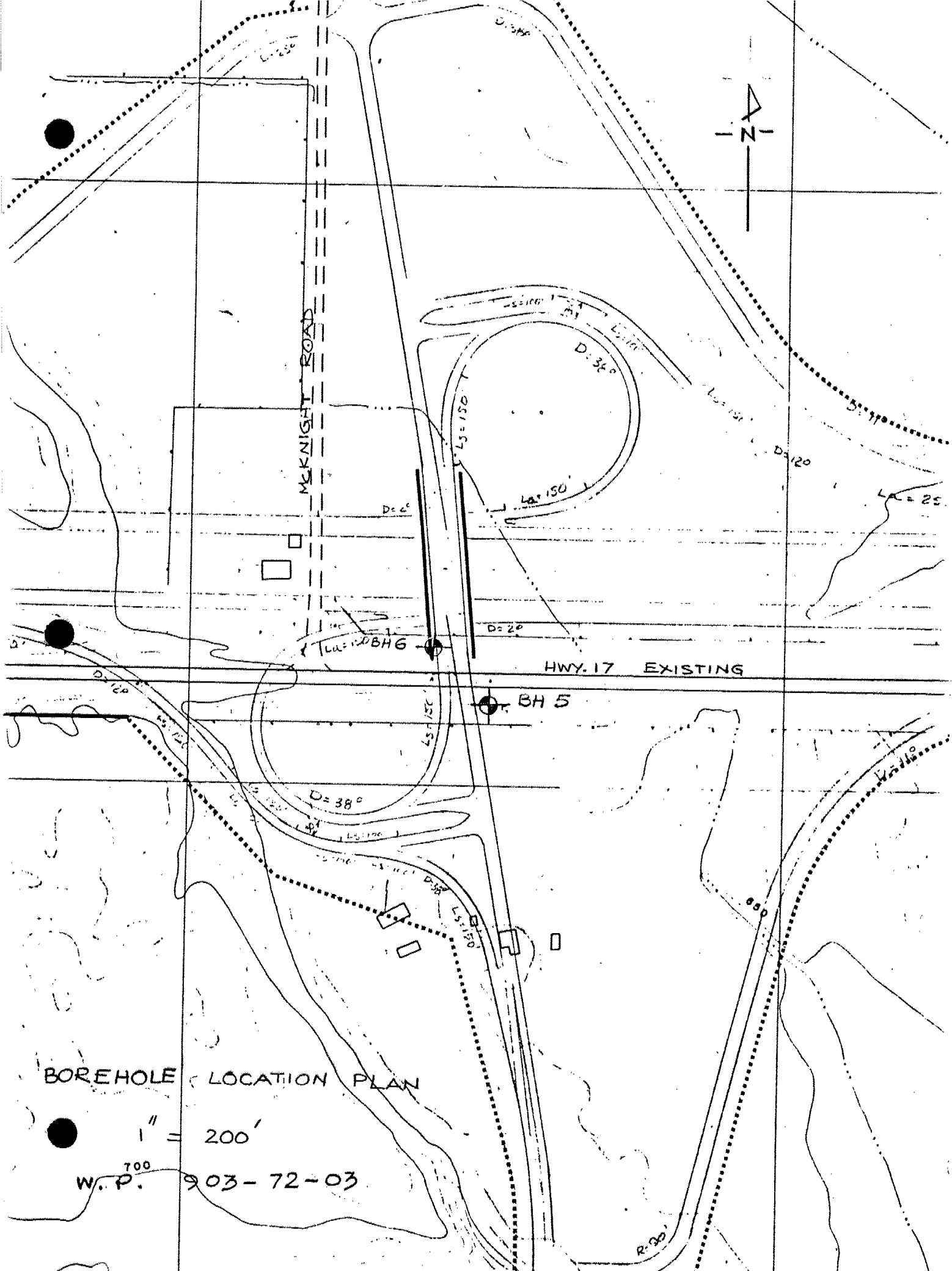
RECORD OF BOREHOLE NO 2

W.P. 903-72-03 LOCATION HWY. 17 & DESBARATS RIVER (as shown on plan) ORIGINATED BY MM  
DIST. 18 HWY. 17 BORING DATE MARCH 14-15, 1975 COMPILED BY AP  
DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER & CONE CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS  % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		SHEAR STRENGTH					WATER CONTENT %				
							O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					$w_p$ — $w$ — $w_L$				
						10	20	30	40	50	400 800 1200 1600 2000					
580±	Ground Level															
579	Ice		1	SS	3											
577.0	Muskeg		1A	TW	PM											
3.0	Organic Clay With Roots Very Soft		2	TW	PM											
570			3	SS	4											
10.0	V. Stiff															
	Silty Stiff															
	Clay Firm		4	TW	PH											
	to															
	Clay Stiff		5	SS	4											
554			6	TW	PH											
26.0	Silty Sand, Some Gravel & Clay		7	SS	17											
550	Compact															
30.0	End of Borehole Refusal Probable Bedrock															
Note: Dynamic Cone Penetration Test was carried out about 40 ft. north of the borehole.																

Note: Dynamic Cone Penetration Test was carried out about 40 ft. north of the borehole.

McKNIGHT ROAD INTERCHANGE



McKNIGHT ROAD

HWY. 17 EXISTING

BH 6

BH 5

BOREHOLE LOCATION PLAN

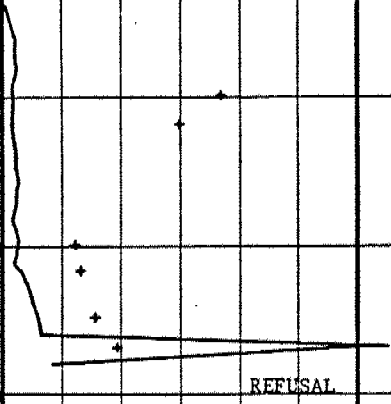
1" = 200'

700  
W.P. 903-72-03

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

## RECORD OF BOREHOLE NO 5

W.P. 903-72-03 LOCATION HWY. 17 & MCKNIGHT ROAD (AS SHOWN ON PLAN) ORIGINATED BY C. McK.  
 DIST. 18 HWY. 17 BORING DATE March 27, 1975 COMPILED BY C. McK.  
 DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER & CONE TEST CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS  % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100					$w_p$ $w$ $w_L$				
							SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					WATER CONTENT %				
547.±	GROUND LEVEL						400	800	1200	1600	2000					
655.0	TOP SOIL		1	SS	3											
2.0	Stiff to Firm  Silty Clay to Clay Occ. Silt Layers  Firm		2	SS	11											
			3	TW	PH											
			4	SS	1/18"											
			5	TW	PH											
			6	SS	1/18"											
			7	TW	PH											
629.0																
28.0	End of Borehole Refusal Probably Bedrock															

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 6

W.P. 903-72-03

LOCATION HWY. 17 & McKNIGHT ROAD (as shown on plan)

ORIGINATED BY C.M.

DIST. 18 HWY. 17

BORING DATE MARCH 27, 1975

COMPILED BY C.M.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGER & CONE TEST

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			UNIT WEIGHT $\gamma$	REMARKS % GR. S.A.S.	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 400 800 1200 1600 2000					WATER CONTENT %
658.0	Ground Level																
656.0	Top Soil		1	SS	5/18"												
2.0	Silty Clay to Clay	Stiff to Firm Soft	2	SS	10												
			3	TW	PH												
			4	SS	2/18"												
			5	TW	PH												
			6	SS	3												
635.5	End of Borehole																
22.5	Refusal																
	Probable Bedrock																

W. Lees  
Manager, Planning & Design

Materials & Testing  
Northwestern Region

July 9, 1975

WP 903-72-02 -- Hwy 17  
Bar River to Hwy 548



Our investigations of the proposed cuts at the WBL approaches to Shewfelt Creek indicate that the excavation material will consist of wet clay and silt. This material will be very difficult to excavate and will be unsuitable for fill purposes. A very substantial pavement design (42" of granular) would be required through these cuts.

It is recommended that a bridge be considered across the Shewfelt Creek valley in order to raise the gradeline sufficiently to eliminate the cuts. The following advantages would be realized:

1. The gradeline would be visually more attractive.
2. The gradeline would be improved geometrically.
3. The difficulty of excavating the wet cut material would be avoided.
4. The construction would be less dependent on weather conditions and season of construction.
5. The cost of placing 6" diameter pipe subdrains on both sides of the roadway through the cuts would be avoided.
6. The structure length could be set to avoid fills in excess of 7' in height at each end. Fill settlements would therefore be minimal.

/mle  
c.c. W. Neillipovitz  
M. Devata  
G. French  
B. McKenna  
G. Wong

*H. A. Meyer*  
H. A. Meyer  
Project Soils Engineer  
for: R. Morgenroth  
Regional Materials Engineer