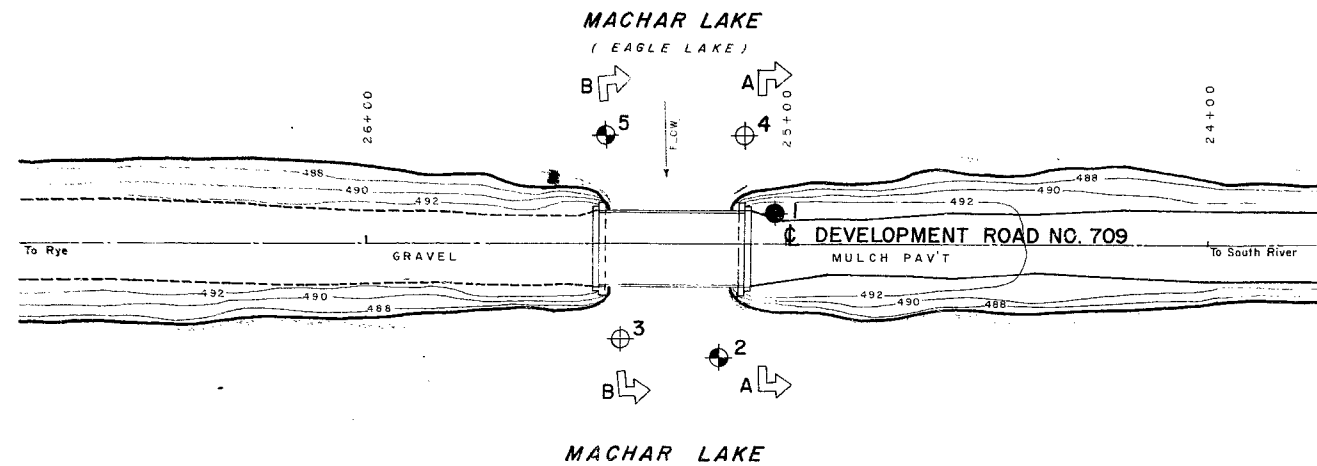


63-F-31

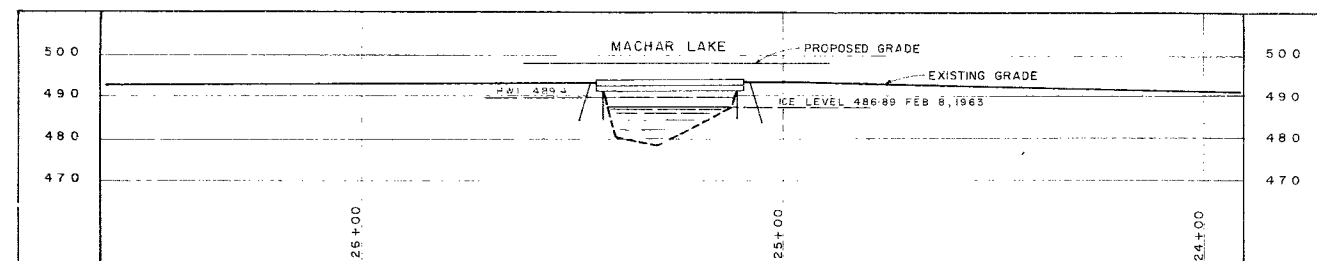
WP 702-63

DEV. RD. 709
MACHAR LAKE

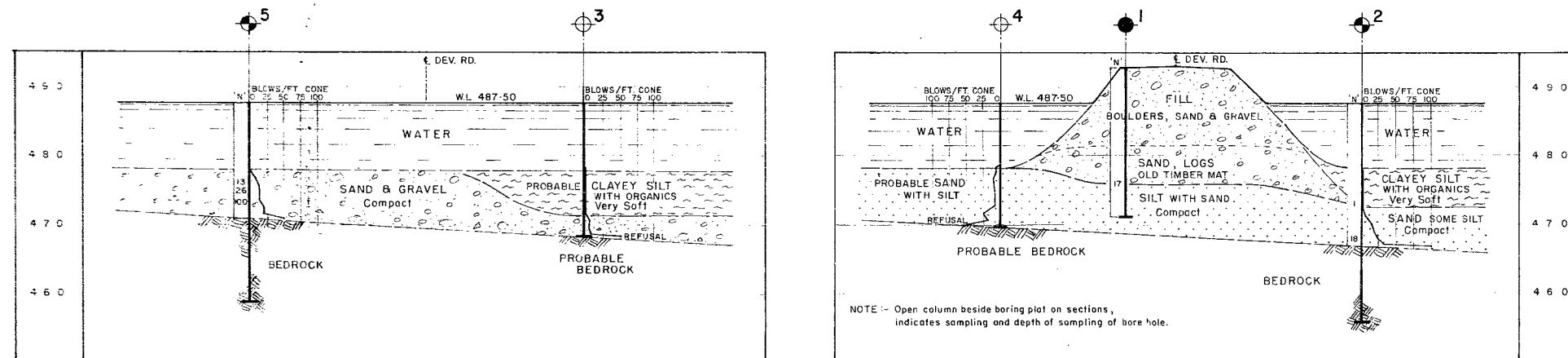


PLAN
SCALE IN FEET
0 10 20 40

NOTE
The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the NORTH BAY District Office. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.



PROFILE
SCALE IN FEET
0 10 20 40

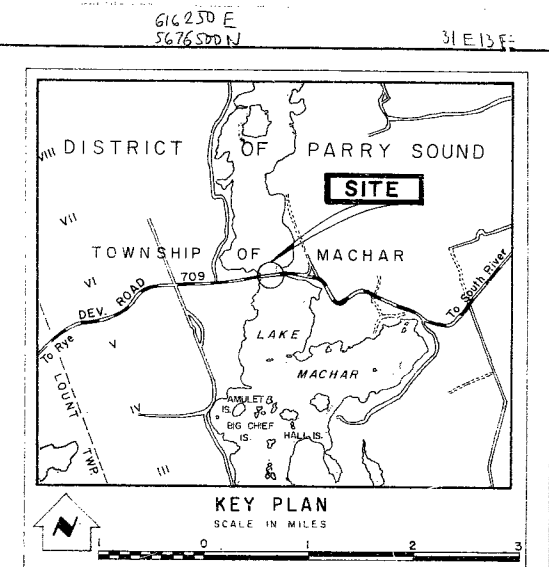


B - B

SECTIONS

A - A

SCALE IN FEET
0 5 10 20



LEGEND

- Bore Hole
- ⊕ Cone Penetration Hole
- ⊕ Bore & Cone Penetration Hole
- Water Levels established at time of field investigation, March 1963

ELEVATIONS ASSUMED

NO.	ELEVATION	STATION	OFFSET
1	493.0	25+03	7.5' RT.
2	487.5	25+16	2.7' LT.
3	487.5	25+39	2.3' LT.
4	487.5	25+10	2.6' RT.
5	487.5	25+43	2.6' RT.

NOTE

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

NO.	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION			
MACHAR LAKE			
KING'S HIGHWAY NO. DEV. ROAD NO. 709		DIST. NO. 13	
CO. PARRY SOUND		LOT 27 CON. V	
BORE HOLE LOCATIONS & SOIL STRATA			
DRAWN T.W.	CHECKED	APP. NO. 702-62	YEAR DRAWING NO.
DRAWN A.S.	CHECKED	APP. NO. 63-F-31	63-F-31A
DATE MAY 7, 1963	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie

Mr. A. G. Stermac,
Principal Foundation Engineer,
Foundation Section,
Materials & Research Division.

May 3, 1963

D.H.O. FOUNDATION INVESTIGATION REPORT --
Bridge where Development Road #709 crosses
Machar Lake, District #13, North Bay, Ont.
W.J. 63-P-31 -- W.P. 702-61

Attached, we are forwarding to you, our
detailed foundation investigation report on the subsoil
conditions existing at the above-mentioned structure site.

We believe you will find the factual data
and recommendations contained therein, adequate for your
future design work. Should you require further information,
please do not hesitate to call on our Office.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
H. McArthur
G. Martens
E. R. Saint
A. Watt

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

Foundations Office
Gen. Files ✓

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 - 4.5) Bedrock
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-

FOUNDATION INVESTIGATION

For

Bridge where Development Road #709 crosses
Machar Lake, District #13, North Bay, Ont.
W.J. 63-F-31 -- W.P. 702-63

1. INTRODUCTION:

A request dated March 19, 1963, for a foundation investigation at the site of Machar Lake Bridge where Development Road #709 crosses Machar Lake, was received from the Bridge Location Section.

A field investigation was carried out by the Foundation Section during April 1963, to determine the subsoil conditions at the site of the proposed structure.

Presented in this report, are the results of this investigation, together with the recommendations pertaining to the design of the bridge foundations and approach embankments.

2. DESCRIPTION OF SITE AND GEOLOGY:

Machar Lake lies some 10 miles west of the Town of South River. At present, Development Road #709 crosses the lake at its narrowest location by a rock fill causeway with a 30-ft. single span bridge in the middle. Geologically, the area is part of the Canadian Shield, the bedrock being of igneous origin.

3. FIELD AND LABORATORY INVESTIGATION:

The field work consisted of three sampled boreholes and five dynamic cone penetration tests, the location of which is shown on Drawing No. 63-F-31A. Disturbed samples were obtained

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

using a 2" O.D. split-spoon sampler of which the energy used for driving conformed to the Standard Penetration Test. Bedrock was proved in B.H.'s 2 and 5, using an AXT core barrel. All samples were visually identified in the field and returned to the laboratory where further tests were carried out to determine the moisture content and particle size distribution.

4. SUBSOIL CONDITIONS:

4.1) General:

The present causeway is built of boulders. The depth of the water at the structure location is about 10 ft. and the thickness of the overburden overlying bedrock is also about 10 ft. On the south side of the bridge, a 5 to 6 ft. thick layer of very soft organic clayey silt was found overlying about 5 to 6 ft. of silty fine to coarse sand beneath which bedrock was encountered.

Boulders are strewn over the lake bottom, but four dynamic cone penetration tests were driven to absolute refusal against bedrock without encountering boulders. The N₁ casing was also driven to absolute refusal. One small boulder was encountered in B.H. 5, but it was driven ahead, or displaced by the casing until absolute refusal against the bedrock was obtained.

4.2) Existing Embankment Fill:

B.H. #1 was put down through the old fill. It consists of 11 feet of rounded boulders varying in size from 6" to 36". Beneath the boulders at this location, is a 6-foot layer of fine to medium sand containing logs some 9 inches in diameter. These may

cont'd. /3 ...

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

4.2) Existing Embankment Fill: (cont'd.) ...

be logs which sank during transportation to the mill, or may have been part of an old timber crib bridge built before the existing bridge. Another explanation is that they may have been used as a timber mat beneath the existing fill. Beneath the timbers, a stratum of silt with fine sand was found. The standard penetration resistance for this material is 17 blows/ft. indicating it to be compact.

4.3) Very Soft Organic Clayey Silt:

A layer of very soft clayey silt with some organics, was found to a depth of 15 feet below water level on the south side of the bridge. In this material, the standard split-spoon sampler was driven two feet for one blow, and the dynamic cone rods sank after one initial blow of the 140-lb. hammer, indicating a very soft consistency. This material was not found on the north side of the bridge.

4.4) Fine to Coarse Sand and Gravel:

In borehole 2 on the south side of the bridge, silty fine to coarse sand was found beneath the very soft clayey silt. A standard penetration 'N' value for this material is 18, indicating it to be compact. In borehole 5 on the north side of the bridge, fine to coarse sand with fine to coarse gravel was found to exist from lake bottom to the bedrock. A standard penetration resistance for this material is 20, indicating it to be also compact.

cont'd. /4 ...

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

4.5) Bedrock:

The bedrock was found beneath the fine to coarse sand in both boreholes. In B.H. 2, it is granite gneiss, and in B.H. 5, it is granite pegmatite turning to granite gneiss.

5. GROUND WATER CONDITIONS:

The water in Machar Lake was at elevation 487.5 at the time of the field investigation. The water level in B.H. 1 corresponds to that of the water in the lake.

6. DISCUSSION AND RECOMMENDATIONS:

It is proposed to replace the present 32-ft. span steel girder bridge at Machar Lake by a new structure that would be spanning a clear waterway of 30 ft. The grade of the causeway is to be raised by about 5 ft. to an elevation of 497.50. The existing bridge is supported on rock fill cribs placed within the approach fills.

There are a number of ways in which the supports of the new structure can be built, and all other aspects being equal or comparable, the economic factor should decide the final choice.

The new structure could, of course, be founded in the same manner as the existing one - i.e., on rock fill timber cribs bearing on bedrock. Because of the quite considerable height of the cribs, it would be advisable to place rock fill also around and in front of the cribs in order to assure more stability and safety.

cont'd./5....

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

The second proposed alternative would be to remove the present bouldery fill from the abutment locations and replace it with sandy or gravelly fill through which supporting steel H-piles could be driven down to bedrock. Such piles could be loaded with 60 - 70 tons per pile. The sandy or gravelly portion of the fill should be rip-rapped for protection against scour and erosion.

As the third alternative, it is proposed that the existing rock fill and the subsoil at the abutment locations be excavated down to bedrock and removed and a new rock fill embankment built. Spread footings for the new bridge could then be placed directly on the new fill. The new fill should be constructed of sound crushed rock. Prior to the placement of the new rock fill embankment, it should be checked to see that the bedrock surface is well exposed over the entire area where the new fill will be constructed.

All of the above-mentioned alternatives are technically sound and we believe, quite simple to execute. They do not involve any special construction procedure and should therefore, be economical.

Bedrock being only some 20 ft. below lake water level, we would suggest that the alternative of placing the abutment footings directly on bedrock, be also studied. After the bouldery fill is removed, steel sheet piling should be driven down to bedrock and the rest of the excavation - i.e., the removal of the silty fine to coarse sand should be carried out under water. Upon completion of this operation, a tremie seal should be poured within the sheeting allowing for the water to be pumped out and the subsequent construction of the abutment to be continued in the dry. A safe bearing pressure of 20 T/sq.ft. or more, can be used.

cont'd. /6 ...

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

The raising of the grade will result in the widening of the existing embankments. Except in the area where there is soft organic clayey silt, no problems should arise. It is suggested that the soft organic deposit be excavated prior to fill placement. If this proves either too costly or difficult, care should be taken that all this material be displaced and not trapped. Best results will be achieved if the fill is placed from the edge of the existing embankment and the forming mud waves removed as they are formed. Trapping of soft material could have some undesirable and detrimental consequences.

7. SUMMARY:

The present causeway approaches are built of round boulders approx. 6" to 36" in diameter. The depth of water at the structure site is about 10 ft. The thickness of overburden overlying granite bedrock is approx. 10 ft. The overburden on the north side of the structure is silty fine to coarse sand, while on the south side, a 5 - 6 ft. surface layer of very soft organic clayey silt was found.

Four alternatives for the foundations of the new structure are proposed and discussed in the bulk of the report.

It is recommended that the soft organic silty material at the location where the approaches will be widened, be excavated or displaced.

8. MISCELLANEOUS:

The field investigation was carried out in the period March 25 to April 9, 1963 by the Canadian Longyear Drilling Co. under the supervision of Mr. T. F. Widdis, who also prepared this report.

May 1963

APPENDIX I.

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_α	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

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 63-F-31 LOCATION 25403 7.5' Rt. of E. Dev. Rd. #709 ORIGINATED BY T.F.W.
W.P. 702-63 BORING DATE Mar. 27, 1963. COMPILED BY T.F.W.
DATUM Assumed BOREHOLE TYPE NX Casing Washboring CHECKED BY H.S.

SOIL PROFILE			SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	WP	WL		
493.0	Groundlevel										
0.0	Fill Boulders with sand and gravel.					490					
482.0	Fill - Sand. Logs (Old timber material)					480					
476.0	Grey compact silt with fine sand.		1	SS	17						Sa.size 14% Si.size 81% Cl.size 5%
471.0	End of borehole.					470					
22.0						460					
						450					

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 63-P-31 LOCATION 25.16, 27.0' Lt. of E. Dev. Rd. #709 ORIGINATED BY T.F.W.
W.P. 702-63 BORING DATE Apr. 3, 1963. COMPILED BY T.F.W.
DATUM Assumed BOREHOLE TYPE NX casing, Washboring. CHECKED BY H.S.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT	20	40	60	80	100		
487.5	Waterlevel					490								
0.0	Water					480								
478.0	Grey, very soft, clayey silt with organics.		1	SS	1	470								
472.5	Grey, compact sand-fine to coarse with some silt.		2	SS	18									
466.5	Bedrock					460								
455.5	End of borehole.													

Gr.size 8%
Sa.size 28%
Si.size 52%
Cl.size 12%

Gr.size 7%
Sa.size 49%
Si.size)
Cl.size) 44%

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 63-F-31 LOCATION 25+10 26.0' Rt. of E Dev. Rd. #709 ORIGINATED BY T.F.W.
W.P. 702-63 BORING DATE April 8, 1963. COMPILED BY T.F.W.
DATUM Assumed BOREHOLE TYPE Dynamic Cone Penetration Test. CHECKED BY H.S.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE										
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	BLOWS / FOOT 20 40 60 80 100						LIQUID LIMIT ——— W _L PLASTIC LIMIT ——— W _P WATER CONTENT ——— W		BULK DENSITY	REMARKS
							SHEAR STRENGTH P.S.F.						W _P W _O W _L			
													WATER CONTENT %		γ _{p.c.f.}	
487.0	Waterlevel					490										
0.0	Water						480									
478.0	Probably fine to coarse sand with silt.						470									
9.5																
469.5	Probably Bedrock.															
18.0																
						460										
						450										

JOB 63-F-31

W. P. 702-63

DATUM Assumed

LOCATION 25743 26.0' Rt. of E Dev. Rd. #709

BORING DATE Apr. 8, 1963.

BOREHOLE TYPE NX Casing Washborings.

ORIGINATED BY T.F.W.

COMPILED BY T.F.W.

CHECKED BY _____ H.S.

Sa. #1
Gr. size 17%
Sa. size 59%
Si. size } 24%
Cl. size }

Sa. #2
Gr. size 23%
Sa. size 55%
Si. size } 22%
Cl. size }

Sa. #3
Gr. size 35%
Sa. size 58%
Si. size } 7%
Cl. size }