

61-F-120

W.P. # 144-61

HWY. # 101, LINE

'A' + HAWK

CREEK

Files - 23-63-141.

Mr. A. M. Toye,
Bridge Engineer.

Materials and Research Division,
(Foundation Section)

Attention: Mr. S. McCombie.

March 12, 1962.

D.H.O. FOUNDATION INVESTIGATION
REPORT.
W.J. 61-F-120 -- W.P. 144-61.

Re: Proposed New Bridge, Hwy. #101, Line 'A'
and Hawk Creek, 1.5 Miles South of Hawk
Junction, Twp. No. 28, Range No. XXIV,
District of Algoma, District #18.

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

We believe you will find the factual data and recommendations contained therein, adequate for your future design work. If clarification, or further information is required, please feel free to contact our Office.

ACI/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
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A. G. Stermac
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FOUNDATION INVESTIGATION

For

Proposed New Bridge - Hwy. #101, Line 'A'
and Hawk Creek 1.5 Miles South of Hawk
Junction, Twp. No. 28, Range No. XXIV,
District of Algoma, District #18.

W.J. 61-F-120 -- W.P. 144-61

1. INTRODUCTION:

It is proposed to erect a new bridge, to carry Hwy. #101 Line "A" over Hawk Creek. The site of the proposed bridge is located in Twp. No. 28 District of Algoma. At this location, the chainage of Hwy. #101 Line "A" is from 59+00 to 61+00.

In order to determine the soil properties and decide on the type of foundation, an investigation was carried out by this Section. Results and the discussion of the field and laboratory investigations, as well as conclusions and recommendations for the future design work, are contained in the following paragraphs of this report.

2. DESCRIPTION OF SITE:

The area in which the structure is located is generally flat terrain. During the time of investigation the whole surrounding area was flooded to a depth of 1 to 6 feet.

3. FIELD AND LABORATORY WORK:

In order to obtain sufficient information on the type and properties of the subsoil, one sampled borehole, and two dynamic

3. FIELD AND LABORATORY WORK: (Cont'd.)...

cone penetration tests, were carried out at this site.

Split spoon samples were taken at various depth intervals. Because of the granular nature of the soil, it was not possible to obtain undisturbed samples. Samples recovered in the split spoon sampler were used to determine the following physical properties:

1. Natural Moisture Content.
2. Grain Size Distribution.

Results of these tests are summarized in Appendix I of this report.

4. SUBSOIL CONDITIONS:

4.1 General:

The stratigraphy of the soil at the site was found to be uniform. A detailed description of the soil encountered during the investigation, is shown in Appendix I of this report, and is also given in subsequent paragraph. The estimated stratigraphical profile, shown on Dwg. No. 61-F-120A is based on this information.

4.2 Silty sand to sandy silt:

All the way down the material in the borehole was more or less the same except that with depth it became slightly siltier. Also, with depth the relative density increased quite considerably. In the upper 15 to 17 ft. the material was in a very loose state, the average number of blows of the Standard

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

4.2 Silty sand to sandy silt:

Penetration Test being in the order of two. Some organic admixtures were also found in the samples. Below this depth down to where the borehole was discontinued (elev. 965.5)-----the relative density increased to very dense. Dynamic cone tests were continued to a greater depth and a practical refusal was encountered at elevations 948.0 and 947.0 respectively i.e., some 74 ft. below ground level. In test No. 1 between 60 and 70 ft. below ground surface a marked increase in penetration resistance leading to refusal at 74 ft. was observed in test No. 1 while in test No. 2 a sudden increase from 68 ft. to refusal at 73 ft. was recorded.

The investigation did not establish the nature of the deposit where practical refusal to the cone penetration was reached because it was felt that adequate bearing could be obtained at higher elevations, but it can be assumed with a great degree of accuracy that piles, if driven to this elevation, would also meet refusal.

A grain size distribution curve of the silty sand material is attached to the report under Appendix I.

5. GROUND WATER CONDITIONS:

At the time of the investigation the area was covered with 1 to 6 feet of water and no records of the ground water table could be made.

Because of the granular nature of the subsoil it can be

5. GROUND WATER CONDITIONS: (Cont'd.)...

assumed that the ground water table corresponds to or is higher than the creek water level at all times.

6. DISCUSSION AND RECOMMENDATIONS:

As was described in previous paragraphs the subsoil is basically a silty sand whose relative density increases with depth from very loose to very dense. The investigation has revealed that within the upper 20 ft. of the deposit the properties are such that adequate support for spread footings could not be obtained. It is therefore suggested that the structure be founded on displacement friction piles driven some 40 ft. into the ground. Treated timber piles would be best suited for this purpose and an allowable load of 15 to 20 tons per pile could be obtained. To determine more correctly the bearing capacity a pile loading test would be required.

If much higher pile bearing capacity would be required, piles would have to be driven to refusal which is believed would be met at or a short distance below approximate elevation 947.0. Such piles provided they are steel tube piles could support a load in the order of 50 tons per pile. Again, a more precise and accurate figure could only be given after a good driving record is available and/or a pile loading test was carried out.

It is our recommendation that shorter timber piles (15 tons/pile) be used.

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

The subsoil being basically of a granular cohesionless nature and the ground water table being relatively high, dewatering during construction may present a real problem. It is suggested that prior to construction beginning the ground water elevation be established and the dewatering scheme arranged accordingly. If sheeting is used it is recommended that it be driven to for a distance below the excavation bottom equalling the height of the water above the excavation bottom. Such an arrangement will prevent the occurrence of bottom piping.

7. SUMMARY:

Timber piles (treated if not completely and permanently submerged) driven some 40 ft. into the ground and loaded with 15 tons/pile are recommended. If 20 tons/pile is desired a pile loading test would have to be conducted.

If much greater bearing capacity per pile is required steel tube piles driven to refusal some 75-80 ft. below ground level (approx. elevations 947.0-942.0) are recommended. It is assumed that a safe bearing load of 50 tons/pile could be obtained. Only after a precise pile driving record and/or pile loading test result is available a more accurate figure could be quoted.

Dewatering may present a serious problem and suggestions and recommendations contained in the body of the report should be followed.

9. MISCELLANEOUS:

The field work was carried out during the period of Dec. 11, 1961 to Dec. 21, 1961, by the Boyles Bros. Core Drill adapted for soil sampling, under the supervision of Mr. W. W. Kulmatickas.

Prepared by:
W. W. Kulmatickas,
Project Foundation Engr.

March, 1962.

Approved by:
K. G. Selby,
Sr. Project Foundation Engr.

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
	P.M.		SAMPLE ADVANCED MANUALLY
	P.H.		SAMPLE ADVANCED HYDRAULICALLY

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 INTERCEPT
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_i	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
$\bar{\sigma}$	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

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-120

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HOLE NO.	SAMP. NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETIN RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
1	S1	5.0-6.5	Very loose coarse silty sand.	31 ^{EL} 016	-	-	-	-	-	1022
	S2	10.0-11.5	Very loose coarse silty sand.	1 1011	-	-	-	-	-	
	S3	20.0-21.5	Very loose coarse silty sand.	8 1001	32.0	-	-	-	-	
	S4	30.0-31.5	Loose silty sand and sandy silt.	14 981	-	-	-	-	-	
	S5	35.0-36.5	Loose silty sand and sandy silt.	14 986	-	-	-	-	-	
	S6	45.0-46.5	Med. dense silty sand and sandy silt.	19 956	14.6	-	-	-	-	
	S7	55.0-56.5	Med. dense silty sand and sandy silt.	17 960	-	-	-	-	-	

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 144-61

BORE HOLE NO. 1

JOB 61-P-120

STATION 59+50

DATUM 1022.0

COMPILED BY H.S.

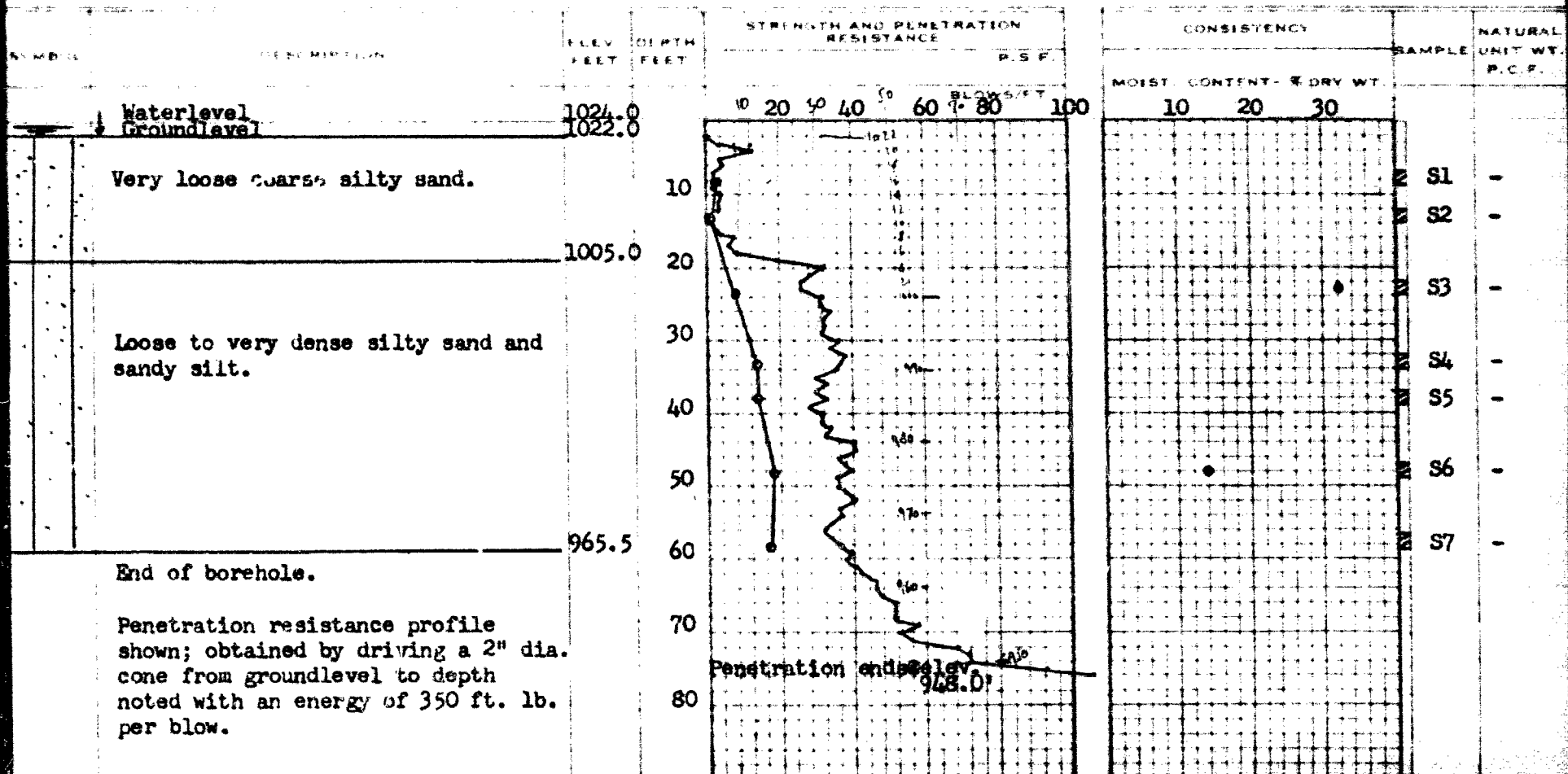
BORING DATE Dec. 11/61.

CHECKED BY W.W.K.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Q_u)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT



DEPARTMENT OF HIGHWAYS - ONTARIO

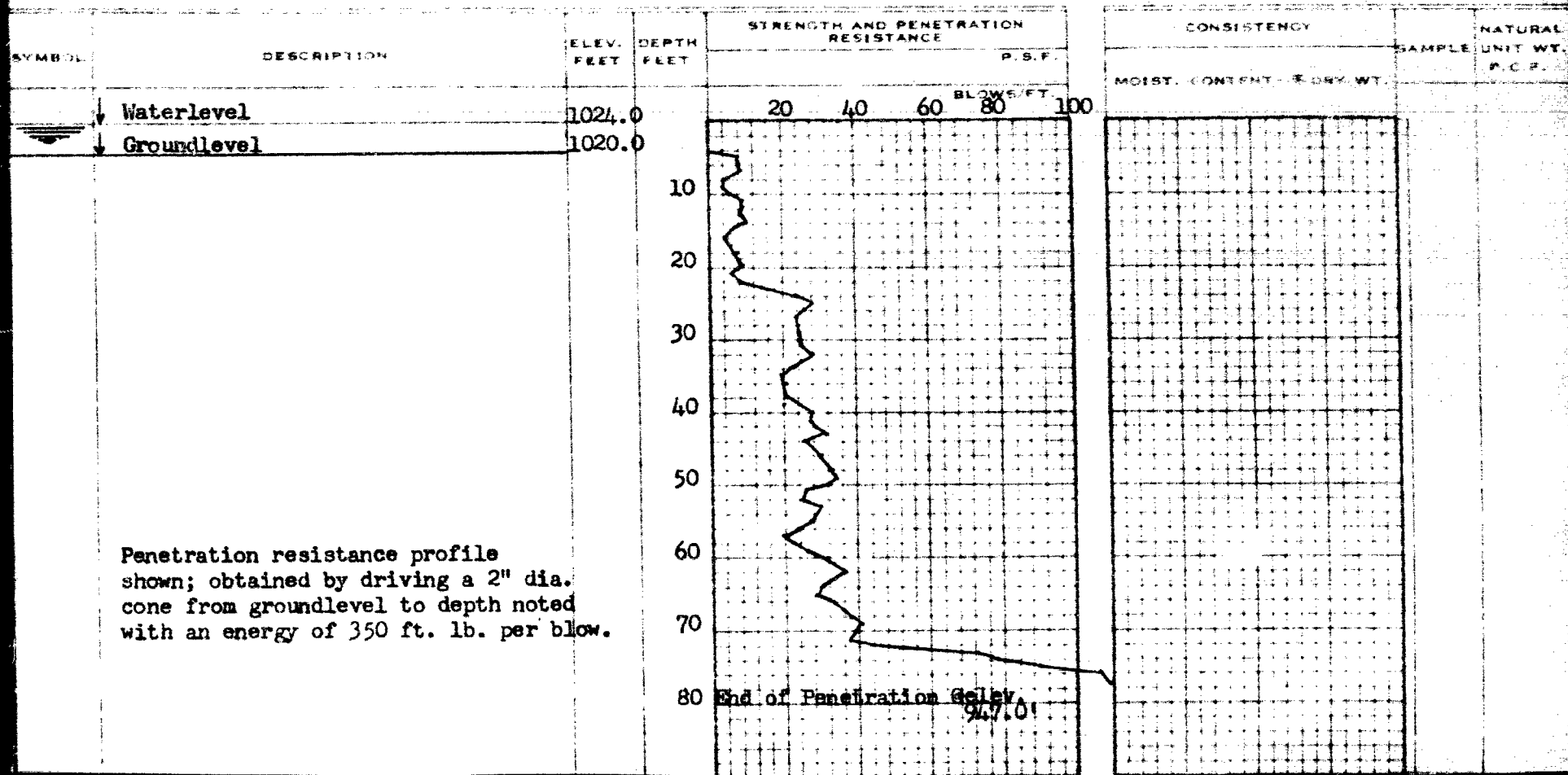
MATERIALS AND RESEARCH SECTION

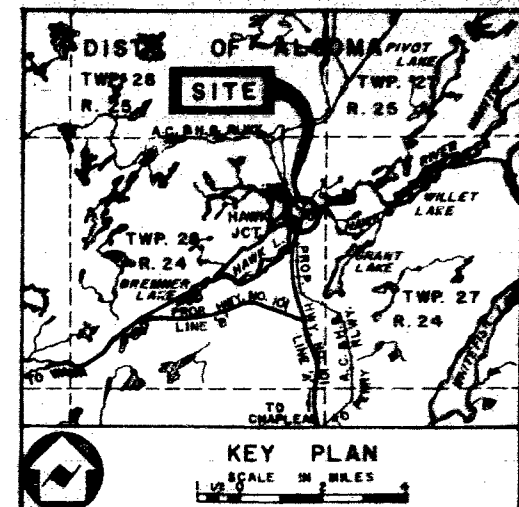
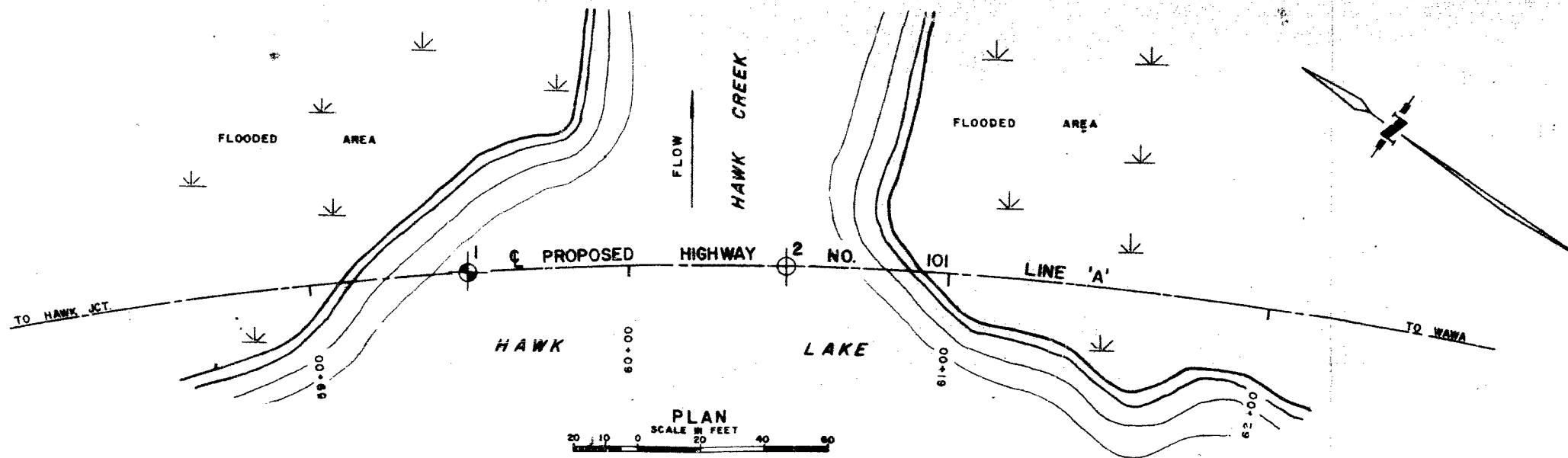
W.P. 144-61 BORE HOLE NO. 2
 JOB 61-F-120 STATION 60+50
 DATUM 1020.0 COMPILED BY H.S.
 BORING DATE Dec. 12/61. CHECKED BY W.W.K.

2" DIA SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA CONE
 2" SHELBY
 CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O
 VANE TEST (C) AND SENSITIVITY (S) +
 NATURAL MOISTURE AND LIQUIDITY INDEX X
 LIQUID LIMIT
 PLASTIC LIMIT

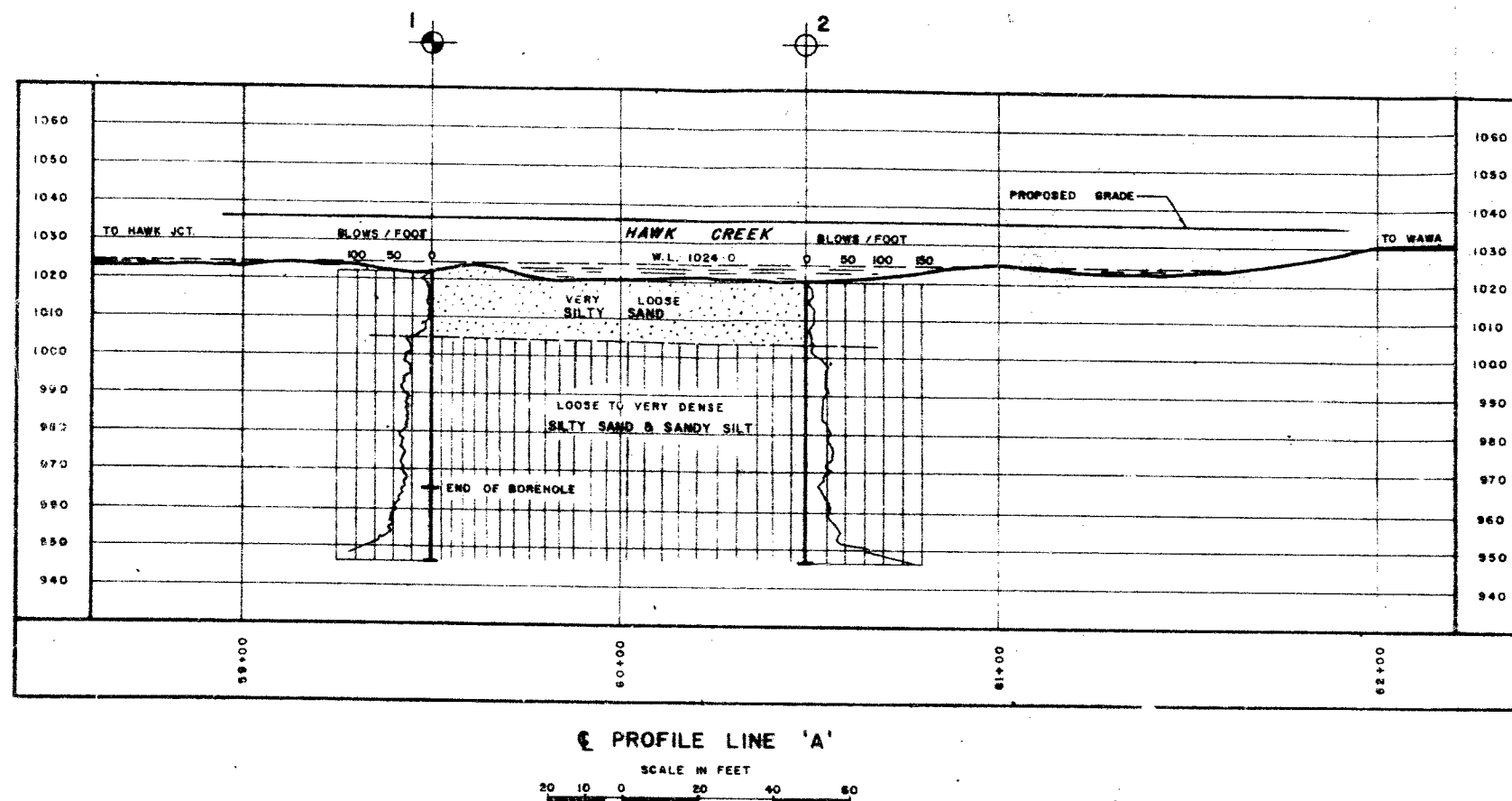




LEGEND			
	BORE & PENETRATION HOLE		
	PENETRATION HOLE (CONE)		
	BORE HOLE		
	WATER LEVELS - Established at Time of Field Investigation. DEC. 8, 1961		
HOLE	ELEVATION	STATION	OFFSET
1	1022.0	59+50	E
2	1020.0	60+50	E

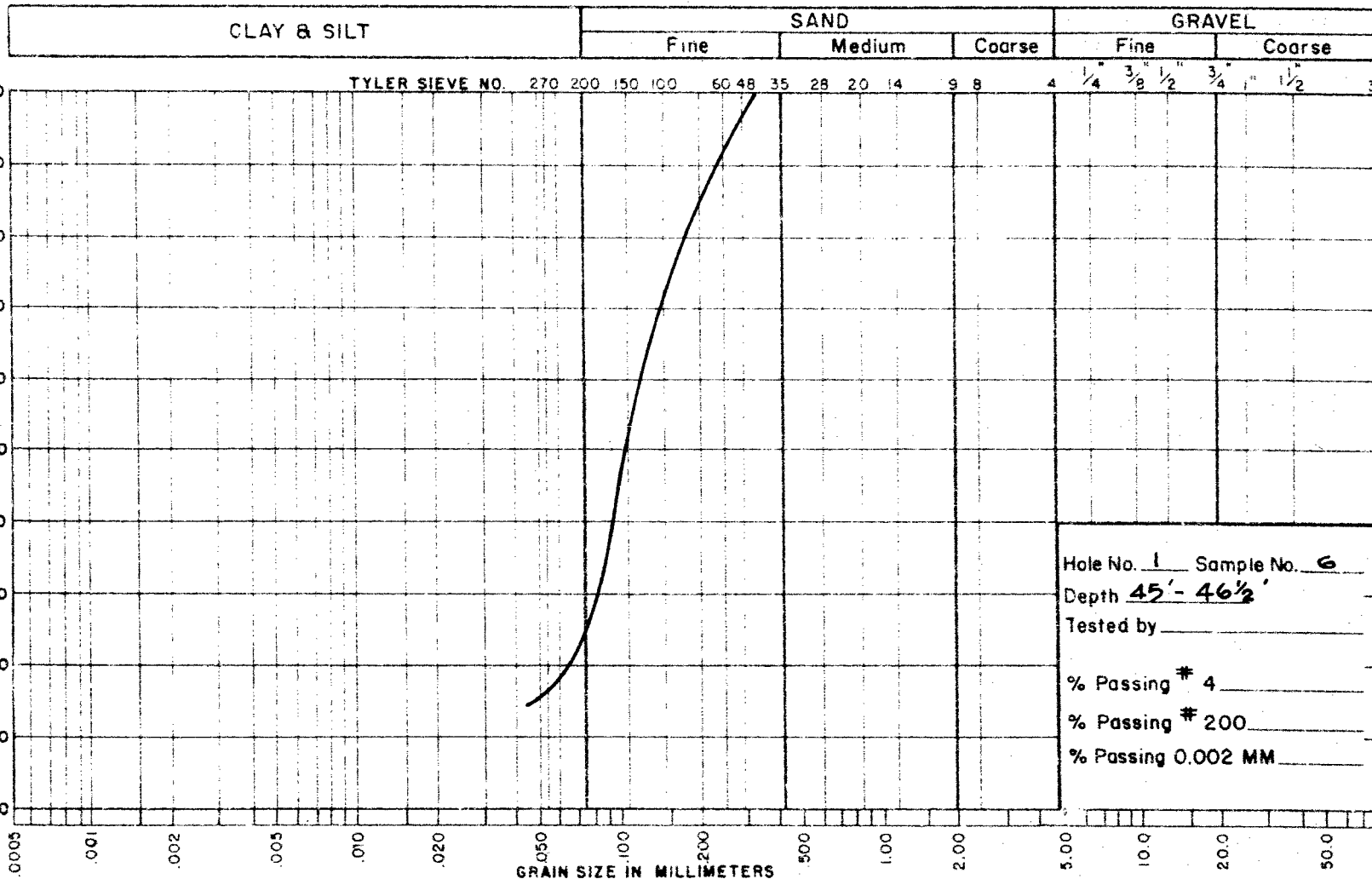
— NOTE —

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.



DEPARTMENT OF HIGHWAYS - ONTARIO		
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION		
HAWK CREEK AND PROPOSED HIGHWAY NO. 101 LINE 'A'		
ORIGINATED W. KULBICKAS	DISTRICT NO. 18	DATE FEB. 8, 1962
DRAWN F. CLARE	W.P. NO. 144-81	JOB NO. 61-F-120
CHECKED <i>[Signature]</i>	CONTRACT NO.	REVISION NO.
APPROVED <i>[Signature]</i>		61-F-120A

UNIFIED SOIL CLASSIFICATION SYSTEM



NOTES _____

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
GRAIN SIZE DISTRIBUTION

Job No. 61-F-120 W.P. No. 144-61

Location _____