#69-F(R)-4

TONOMO LAKE

ROAD.

WAHA BRIDGE

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. G. E. French,
District Engineer,
District #13 (North Bay).

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Attention: Mr. H. Chyc,
District Municipal Engr.

Date: February 5, 1969

Our File Ref:

In reply to FEB - 6 1969

Subject:

FOUNDATION INVESTIGATION REPORT
For
Waha Bridge on Tonomo Lake Road
District No. 13 (North Bay)
W.J. 69-F(R)-4 -- W.P. (Nil)

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

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please feel free to contact our Office.

A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeP
Attach.
cc: Messrs. G. E. French (2)
K. L. Kleinsteiber
C. R. Wilmot
J. C. McAllister
E. R. Saint
B. A. Singh

Foundations Files
Gen. Files

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FOUNDATION INVESTIGATION REPORT

For

Waha Bridge on Tonomo Lake Road

District No. 13 (North Bay)

W.J. 69-F(R)-4 -- W.P. (Nil)

1. INTRODUCTION:

Following a request by Mr. J. McAllister, North Bay Regional Bridge Location Supervisor, dated January 27, 1969, the Foundation Section carried out a field investigation at the location of the above mentioned structure presently under construction by North Bay District forces.

The original structure consisted of a 10-span timber trestle bridge some 120 ft. long, crossing the narrows between Lakes Waha and Brophy. The intention of the District was to replace three spans at each end by an approach embankment, thus shortening the bridge by about 80 ft. For this purpose, the bridge superstructure was partially demolished and filling operations commenced at the North end. Profile grade for the structure is about 5 ft. above water level. As the fill was placed, it became apparent that displacement of the underlying subsoil was taking place and mud waves formed in front of and at the sides of the embankment, causing lateral displacement of the pile bents which were intended to be used in the new bridge. At this stage, when the height of the fill was about 2 ft. above water level, construction was halted.

This report contains the results of our field investigation, together with recommendations for the measures to be taken to correct the present situation.

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2. FIELD WORK:

Two borings, one at each end of the bridge and one dynamic cone penetration test at the bridge centre, were carried out during the course of the field work. These borings revealed the following stratigraphy:

At the North end of the bridge, where fill has been placed, the original organic subsoil has been displaced down to a depth of 23 ft. below water level. This fill is underlain by about 2 ft. of firm clayey silt, which is followed by granite bedrock at el. 70.00. The fill material consists of boulders, sand and gravel, and now contains large pockets of trapped organics.

At the South end of the structure the soil consists of about 8 feet of sand and gravel fill material, followed by about 8 feet of very soft peat, followed by about 7 ft. of firm clayey silt, followed by granite bedrock at el. 70.00.

At the centre of the crossing, the sounding carried out indicates about 20 ft. of very soft muck, followed by about 5 ft. of clayey silt, followed by granite bedrock at el. 70.00.

The locations and elevations of borings are shown on the attached Drawing No. 69-F(R)-4A, together with the inferred stratigraphical profile.

3. RECOMMENDATIONS:

In view of the subsoil conditions at this site, it appears that there are three possible alternative methods of achieving a reasonably economical crossing at this site. This statement is made with the understanding that a cheap supply of acceptable granular type fill exists close to the site.

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

Method 1.

For this method, a trestle bridge similar in length to the original bridge, can be constructed. Piles should be driven to bedrock. The fill which has been placed, should be excavated down to the original river bed level and all mud waves removed before driving piles.

Method 2.

The scheme in which it was proposed to construct approach embankments and build a shorter bridge may be proceeded with, but it will be necessary to displace all of the soft material for the full distance across the narrows and construct stable embankments prior to driving piles. It is believed that it will not be possible to save the existing piles, and that new piles will be necessary. It should be borne in mind, therefore, that selected fill which contains no boulders, must be dumped at locations where piles will be driven. In order to construct a stable embankment, it will be necessary to achieve side slopes of 1:1 below river bed level and 2:1 above this level. To reduce the possibility of future subsidence (due to trapped muck) and settlements, it is recommended that about 5 feet of fill be placed over the embankment as a temporary surcharge. This may be removed when the settlements cease. Displacement of soft material should be accelerated by subexcavation, if possible, and by excavation of mud waves formed during filling operations. Details relating to the fill placement and surcharging are shown on the attached Fig. 1 and Fig. 2.

Piles should be driven to bedrock, in which case, the maximum allowable load for the type of pile may be assumed for design purposes.

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

Method 3.

For this method, the embankment should be constructed as outlined for Method 2, and as shown on Fig. 1 and Fig. 3. A suitably sized pipe, or several pipes, can be installed after excavating the required trench through the embankment. This method is dependent for its success on first constructing a stable fill which will not settle differentially. If differential settlements occur after construction, it is possible that failure of the pipe would take place. For this reason, it is believed that a number of small pipes would be preferable to a single large pipe. In any event, it should be borne in mind that it is extremely difficult to completely displace such a large amount of muck as exists at this site.

4. MISCELLANEOUS:

The field work was carried out from January 29 to January 31, 1969, under the supervision of Mr. P. Payer, Project Foundation Engineer. The equipment used for the subsoil investigation was owned and operated by Canadian Longyear Ltd.

The report was prepared by Mr. K. G. Selby, Supervising Foundation Engineer.

February 1969

APPENDIX I.

FORM OB-MT-126

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 69-F-4(R)

LOCATION Sta. 2 + 55; o/s 3' Lt.

ORIGINATED BY PP

W. P. ——

BORING DATE January 29 & 30, 1969

COMPILED BY PP

DATUM Temporary

BOREHOLE TYPE Washbore - NX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT / SHEAR STRENGTH P.S.F.	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
96.0	Ground Level									
0.0	Fill material		1	SS	19					El. 95.0 ▽
			2	SS	43	90				
	Mixture of boulders, gravel, sand, silt, clay and organics.		3	SS	4					
			4	SS	14					
			5	SS	48	80				
	Very loose to dense.		6	SS	4					
			7	SS	14					
70.2	Clayey Silt		8	SS	50/4"	70				
25.8	Refusal - Probable Bedrock End of Borehole					60				

FORM DB-MT-126

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 69-F-4(R) LOCATION Sta. 2 + 80; o/s 10' Lt. ORIGINATED BY PP

W.P. — BORING DATE January 30, 1969 COMPILED BY PP

DATUM Temporary BOREHOLE TYPE Cone Test Only CHECKED BY [illegible]

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 25 50 75 100 125 / SHEAR STRENGTH P.S.F.	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w — w_P w w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
95.0	Water Level									
93.7	Ground Level									
	Probably muck and organic silt.					90				
						80				
75.0										
20.0	Probably Clayey silt					70				
69.2										
25.8	Refusal End of Cone Test					60	refusal			

FORM DB-MT-126

OFFICE REPORT ON SOIL EXPLORATION

3

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 69-F-4(P)

W.P.

DATUM Temporary

LOCATION Sta. 3 + 25; o/s 2' Lt.

BORING DATE January 30 & 31, 1969

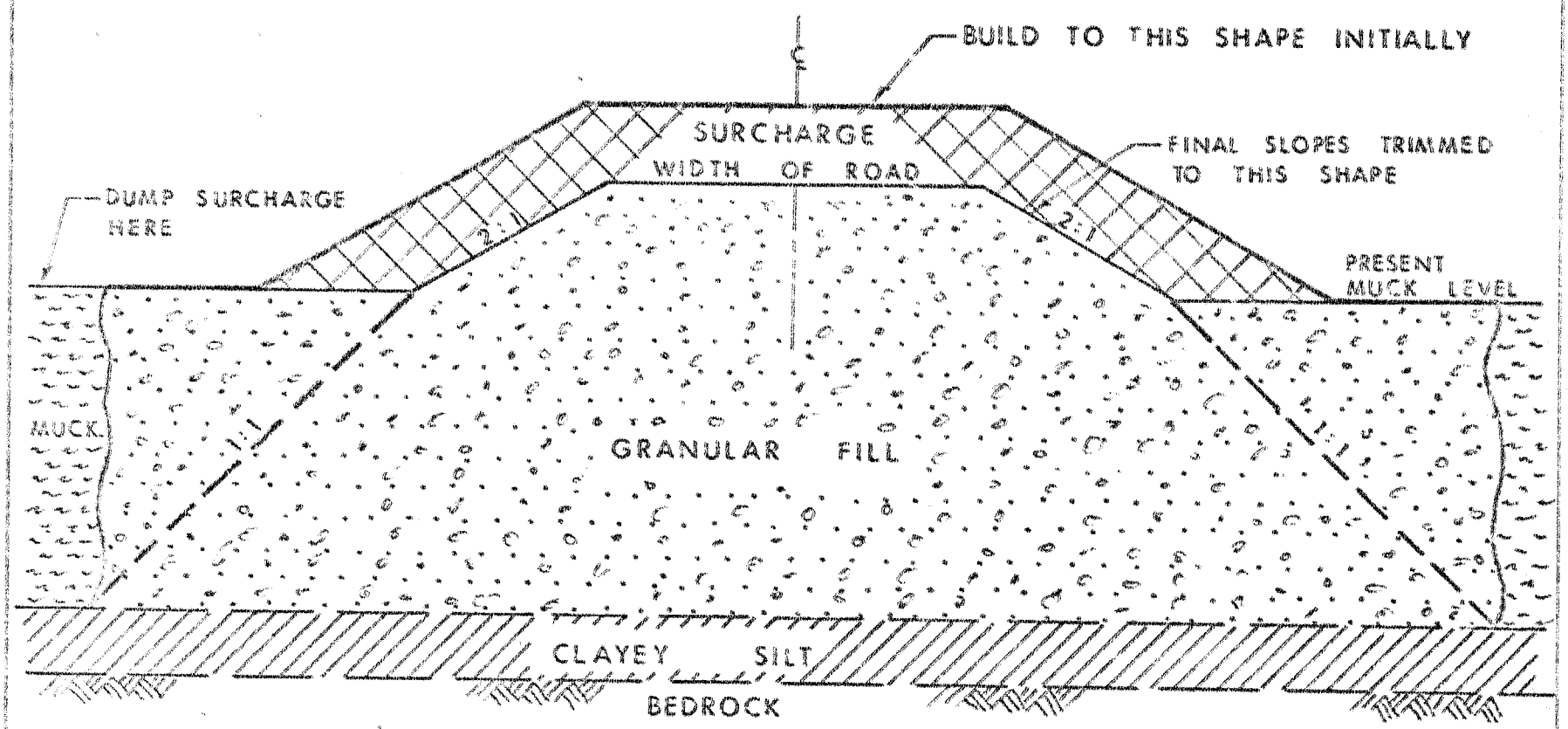
BOREHOLE TYPE Washbore - NX Casing

ORIGINATED BY PP

COMPILED BY PP

CHECKED BY

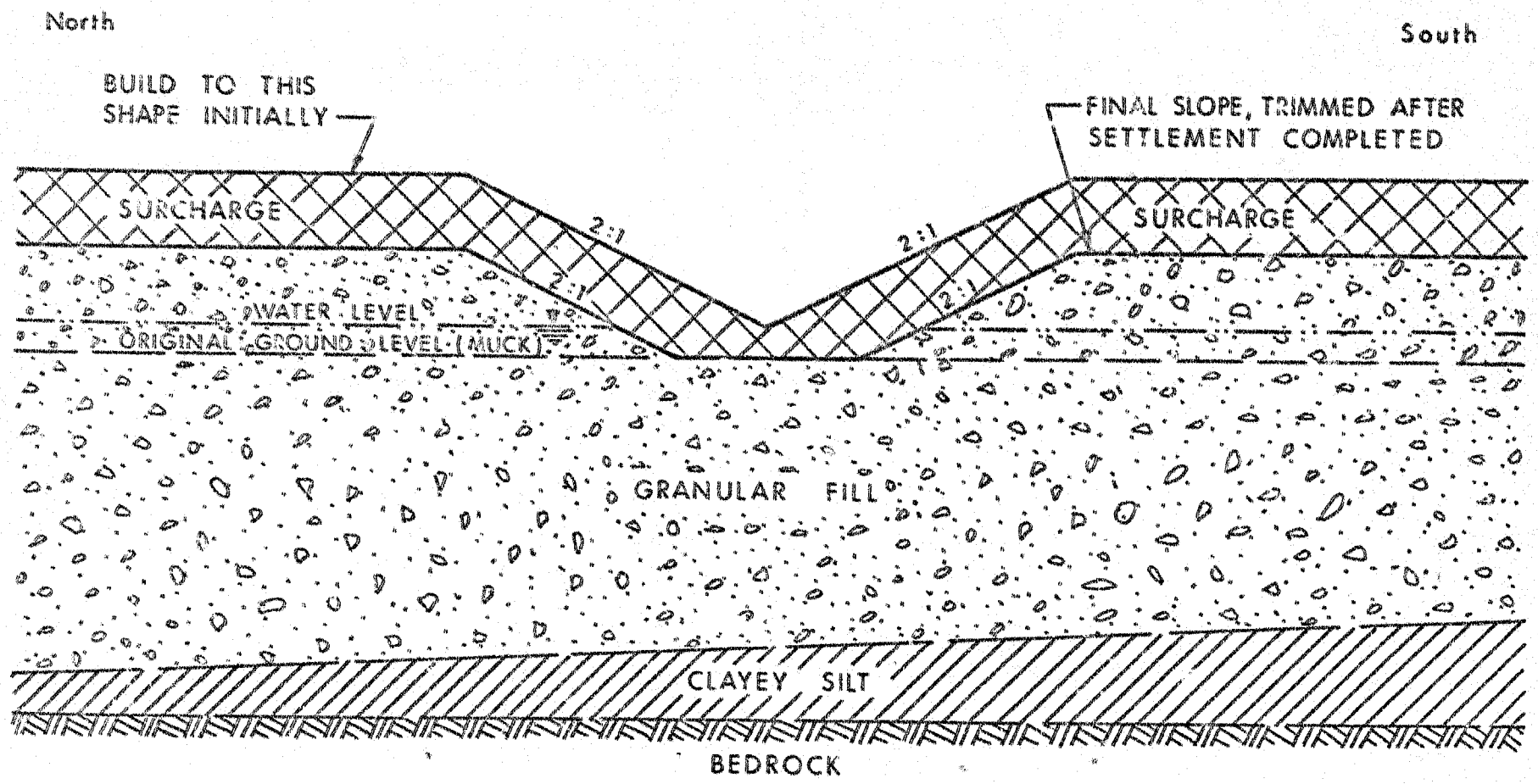
SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT / SHEAR STRENGTH P.S.F.	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
95.0	Ground Level									
0.0	Fill Material Gravelly sand with silt and clay.					90				
			1	SS	17					
87.0	Compact									
8.0	Muck and organic silt.									
			2	SS	1					
79.0	Very soft.					80				
16.0	Clayey silt with some sand and traces of gravel.		3	SS	1					
			4	SS	5					
69.0	Firm					70				
			5	SS	1					
26.0	Refusal Probable Bedrock End of Borehole									
						60				



SUGGESTED CROSS SECTION FOR FILL PLACEMENT
(METHODS 2 & 3)

FIG. 1

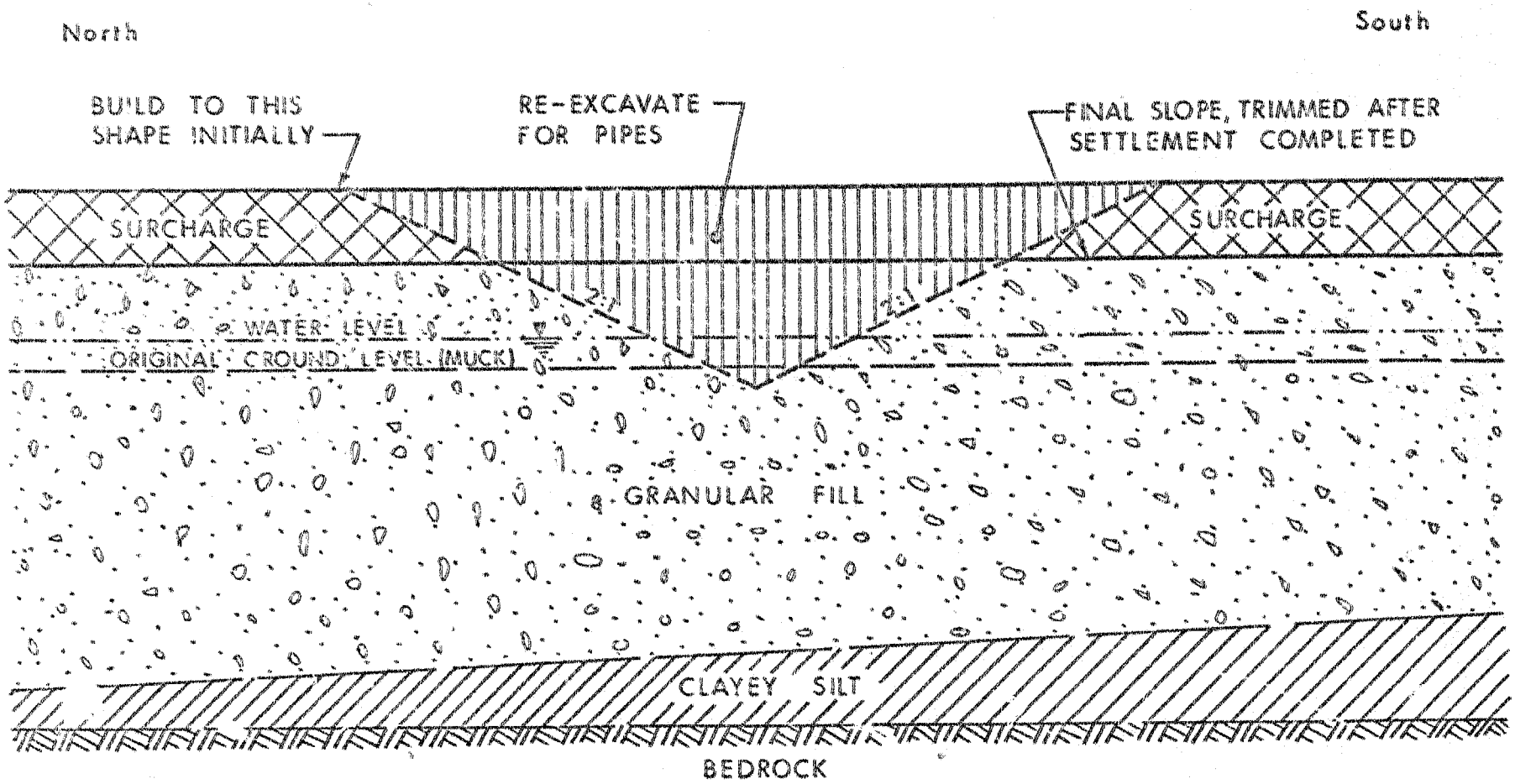
69 - F(R) - 4



SUGGESTED FILL PLACEMENT ALONG ℄
(METHOD 2)

FIG. 2

68-F(P)-4



SUGGESTED FILL PLACEMENT ALONG ℄
(METHOD 3)

FIG. 3

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 :-

CONSISTENCY	'N' BLOWS / FT	c LB. / SQ FT	DENSENESS	'N' BLOWS / FT.
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 INTERCEPT — IN TERMS OF EFFECTIVE STRESS
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION — IN TERMS OF TOTAL STRESS
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF σ
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF σ TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

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

EARTH PRESSURE

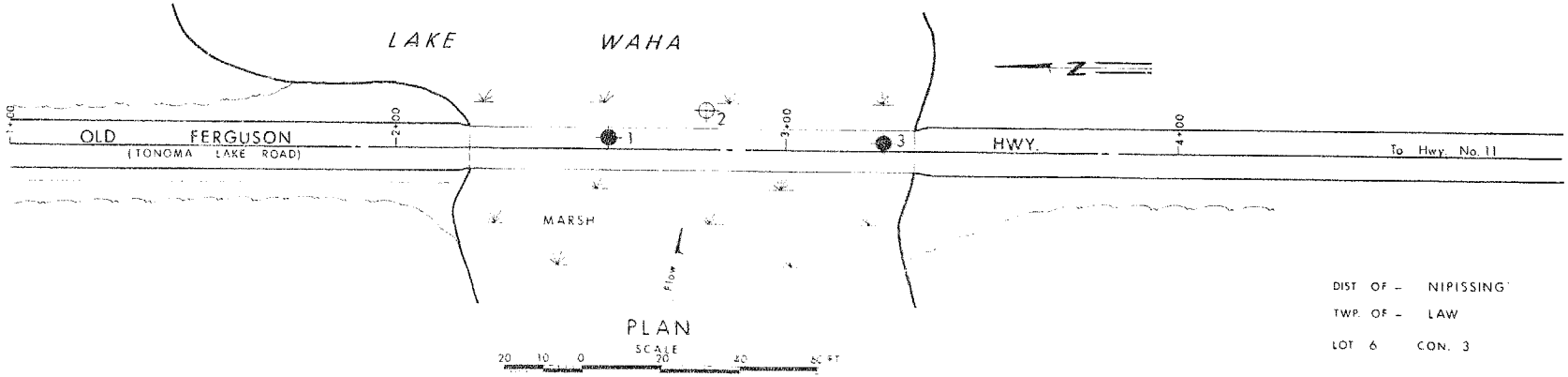
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

SLOPES

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



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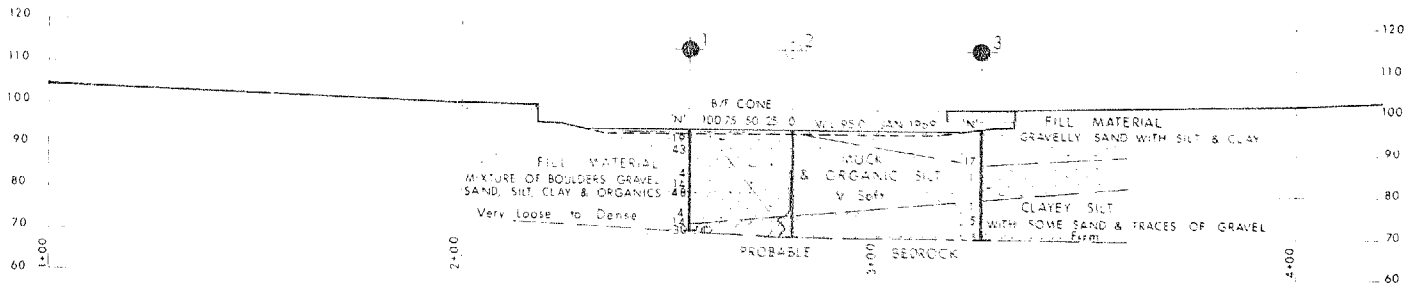

℄ PROFILE - OLD FERGUSON HWY.

SCALE
20 10 0 20 40 60 FT

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

WAHA BRIDGE
ON TONOMA LAKE ROAD

DATE FEB. 5 1969 | APPROVED | DRAWING NO.