

file 28-14 to
Brown's Bridge
Twp. of Armstrong

Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.,
Bridge Division.

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

January 5, 1965

Brown's Bridge, (W.J. 63-F-131)
Twp. of Armstrong,
Dist. of Timiskaming (No. 14)

In connection with the questions concerning the above-mentioned structure, we herewith submit our recommendations for your consideration:

1. Q : Falsework support?

A : Because of poor subsoil conditions (org.-silt with sand - slightly plastic - soft), settlements under falsework supports cannot be discounted.

It is recommended that the design of the bridge be changed to steel, thus eliminating the necessity of falsework.

2. Q : Stability of slopes during foundation construction?

A : The excavation for the pier footings should be carried out as a braced and shored sheet pile encased excavation. Piles can be driven prior to beginning of excavation. A follower can be used, or piles have to be cut.

3. Q : Permissible depth of fill behind abutments?

A : The raising of the grade averages between one and three feet. It is felt that no stability problems should arise from this new condition.

4. Q : Can piles of existing bridge be used as falsework support?

A : No. The existing structure is much lighter than the proposed new one. The new structure would produce a heavier sustained load condition and settlements could result. How much settlement is impossible to predict.

January 5, 1965

5. Q : Is construction sequence essential?

A : No. The geometry of the ground - i.e., the slopes, remains basically unchanged. If reasonable construction procedure is used, no difficulties are expected.

AGS/MdeF

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

cc: Foundations Office
Gen. Files

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AGS/MdeF

Agtermas
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO
MEMORANDUM

Dist. 38-17.

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

FROM: Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Div.

Attn: Mr. K. L. Kleinsteinber,
Mun. Bridge Liaison Engr.

DATE: January 31, 1964

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

Proposed New Bridge over Evanturel
Creek, Township of Armstrong, Lot 9,
Con. V/VI, District of Timiskaming.
W.J. 63-F-131 -- District No. 14.
(Municipal Job)

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

We believe that you will find the factual data and
recommendations contained therein, adequate for your future
design work. If further information concerning this project
is required, please do not hesitate to contact our Office.

KYL/MdeF
Attach.

cc: Messrs. A. M. Toye (3)
J. P. Howard
J. Moffat
E. R. Saint

Foundations Office
Gen. Files

Kyho
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

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FOUNDATION INVESTIGATION REPORT

For

Proposed New Bridge over Ewanturel
Creek, Township of Armstrong, Lot 9,
Con. V/VI, District of Timiskaming.

W.J. 63-F-131 -- District #14

1. INTRODUCTION:

A memo from the Bridge Division, dated October 10, 1963, was received, requesting a foundation investigation at the site of the proposed new bridge over Ewanturel Creek in Armstrong Township, Lot 9, Con, V/VI.

A field investigation was subsequently carried out by this Section in order to determine subsoil conditions at the above site.

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

2. DESCRIPTION OF THE SITE:

The site is located approximately 2.5 miles North-West of Earlton in the Twp. of Armstrong, District of Timiskaming. At this location, the creek flows from South to North. The width of the creek at the proposed crossing is about 20 ft., and the maximum depth at the time of the investigation, was 4.0 ft. The existing banks of the creek in the vicinity of the structure location are approximately 10 ft. to 30 ft. high, having 2:1 to 3:1 natural slopes. The banks of the creek show many signs of instability in the form of slope failures. A slope failure of the natural bank some 30 ft. high, can be seen at this site North of the existing East approach embankment.

The existing structure over Ewanturel Creek is approximately 15 ft. wide and 115 ft. long which accommodates only single-lane traffic.

3. DESCRIPTION OF FIELD AND LABORATORY WORK:

Field work consisted of two sampled boreholes and one dynamic cone penetration test. The boring was carried out by means of conventional diamond drilling equipment adapted for soil sampling purposes.

Samples were recovered at required depths by means of a 2" O.D. split-spoon sampler and by a 2" I.D. Shelby tube sampler. The dimensions of the split-spoon sampler and the energy used in driving it, conform to the requirements of the Standard Penetration Test. In-situ vane tests were carried out wherever possible, in order to determine the shear strength of the cohesive deposits. A dynamic cone penetration test was carried out adjacent to B.H. #1. Driving energy to advance the 2-inch cone was 350 ft.-lbs. per blow. A Norwegian piezometer was installed near B.H. #1 in order to observe the artesian head in the bedrock. The location and tip elevation of piezometer are shown on Dwg. #63-F-131A.

The locations and elevations of all boreholes are shown on Dwg. #63-F-131A which accompanies this report. All elevations are referred to a bench mark located near the South fence line at approximately Sta. 4+09 of the existing roadway.

Samples were visually examined and identified in the field as well as in the laboratory. Tests were carried out in the laboratory on a selection of both disturbed and undisturbed samples to determine:

- i) Natural Moisture Contents
- ii) Bulk Densities
- iii) Atterberg Limits
- iv) Grain Size Distributions
- v) Undrained Shear Strengths

3. DESCRIPTION OF FIELD AND LABORATORY WORK: (cont'd.) ...

Laboratory and field test results have been summarized and are included under Appendix I of this report.

4. SUBSOIL CONDITIONS:

4.1) General:

Subsoil at the site was found to consist of about 6 - 11 ft. of organic silt with sand followed by varved clay. Immediately below this, a deposit of silty sand with gravel underlain by bedrock was observed.

The boundaries of various deposits are shown on the appended borelog sheets. The estimated stratigraphical profile of Dwg. #63-F-131A is based upon this information. From ground level downward, the different soil types are as follows:

4.2) Organic Silt with Sand:

This deposit was found to be 6.0 ft. and 11.0 ft. thick in B.H. #1 and #2, respectively. It consists of grey-coloured organic silt containing fine sand. The material has a definite organic odour. The liquid and plastic limits are in the ranges of 42%-47% and 28%-29%. The undrained shear strength, according to field and laboratory tests, is estimated to be 330 p.s.f. to 595 p.s.f. and hence, the material may be classified as soft to firm.

4.3) Varved Clay:

Underlying the organic silt, a deposit of varved clay was encountered, extending down to elev. 164.5 - 157.0. This deposit consists of alternate layers of clay of high plasticity and clayey silt. The clay layers generally range from 1/2" to 2-1/2" in thickness and are spaced 1/4" to 3/4" apart.

cont'd. /4 ...

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

4.3) Varved Clay: (cont'd.) ...

The Atterberg limits and moisture content ranges for the various layers, are tabulated below:

		<u>Clay Layers</u>	<u>Clay Silt Layers</u>
Liquid Limit	(W _L %)	60% - 64%	18% - 31%
Plastic Limit	(W _p %)	26% - 29%	12% - 22%
Moisture Content	(W %)	56% - 59%	11% - 32%

The undrained shear strength of the deposit as determined from the field vane tests and laboratory tests, was found to vary from a low of 345 p.s.f. to a maximum of 1440 p.s.f. These results indicate that the deposit is essentially soft to stiff.

4.4) Silty Sand with Gravel:

This deposit underlies the varved clay and extends down to bedrock. The depth of the layer varies from a minimum of 8.5 ft. in B.H. #1 to a maximum of 17.0 ft. in B.H. #2. The upper limit of this deposit at the East side of the creek is at approximate elev. 164.5, whereas at the West side, it is at approx. elev. 157.0.

Standard Penetration resistances or 'N' values of 13 to 89 blows per ft. were obtained in this material. From these values, it is estimated that the relative density is compact to very dense, generally increasing with depth.

4.5) Bedrock:

Sound limestone bedrock was established by drilling 5 ft. of BX core in B.H. #1 and #2. The contact with bedrock was established at elev. 147.5 and elev. 148.5 at the East and West banks of the creek, respectively.

cont'd. /5 ...

5. GROUND WATER CONDITIONS:

The water level of Ewanturel Creek at the crossing was at elev. 180.0 which corresponds to the water levels in the boreholes.

Artesian water conditions were observed in both the boreholes immediately above and within the limestone bedrock. A Norwegian piezometer was installed some 42.0 ft. below the existing ground into the bedrock. The observed artesian head from the piezometer was 10.5 ft. above the existing ground. The exact tip elevation of the piezometer and the measured artesian head are shown on the log of borehole 1 (Appendix I).

6. DISCUSSION AND RECOMMENDATIONS:

It is proposed to construct a new three-span (40' - 65' - 40' structure over the Ewanturel Creek to replace the existing one. The new centreline will be the same as the existing one, and the new profile grade will be 2 to 3 ft. higher.

Subsoil at the site mainly consists of 6 - 11 ft. of soft to firm organic silt with sand followed by 15 to 19 ft. of soft to stiff varved clay deposit. Underlying the varved clay and immediately above the bedrock is a deposit of firm to very dense silty sand with gravel. The depth to bedrock was observed to be 38.5 ft. below the ground surface.

6.1) Structure Foundations:

The subsoil generally consists of 21 to 30 ft. of soft to firm organic silt and varved clay deposits. These deposits cannot provide an adequate bearing capacity for an economical spread footing design. The new structure should, therefore, be supported on end-

cont'd. /6 ...

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

6.1) Structure Foundations: (cont'd.) ...

bearing piles driven to bedrock. A safe design load of 70 tons may be attributed to 14 BP 73 steel H-piles driven to bedrock.

Soffets of concrete pier caps and footing bases should be formed on a 12" granular pad or a suitable working slab.

6.2) Approach Fills:

The presence of soft to firm organic silt and varved clay deposits below the approach embankments will necessitate certain measures to ensure the stability of the proposed heightening and widening of the approach fills. No attempt should be made to trim the forward slopes of the approach fills steeper than the existing ones. The sides of the embankment, however, may be constructed with a slope of 2 horizontal to 1 vertical. Precautions should be taken to protect the creek banks and approach embankments from scour action of the creek. This may be achieved by suitably placed rip-rap.

7. SUMMARY:

The site is underlain by soft to firm organic silt followed by soft to stiff varved clay. Underlying the varved clay and above the bedrock, is a deposit of compact to very dense silty sand with gravel. Depth to limestone bedrock is 38.5 ft. below ground.

Artesian water conditions were observed within and above the bedrock. A maximum head of 10.5 ft. above the ground was established from the bedrock.

The structure should be supported on end-bearing piles driven to bedrock. A design load of 70 tons may be used for 14 BP 73

cont'd. 7/ ...

7. SUMMARY: (cont'd.) ...

steel H-piles. A suitable working slab or a 12" granular pad should be provided for pier caps and footing bases.

Approach fills should be constructed as discussed under Section 6.2.

8. MISCELLANEOUS:

The field work, performed during the period October 28 to October 31, 1963, was undertaken by Mr. A. Barsvarv, Project Foundation Engineer. The investigation was carried out under the general supervision of Mr. K. Y. Lo, Supervising Foundation Engineer. The report was prepared by Mr. M. Devata, Senior Foundation Engineer.

Equipment used was owned and operated by Canadian Longyear Co. Limited.

January 1964

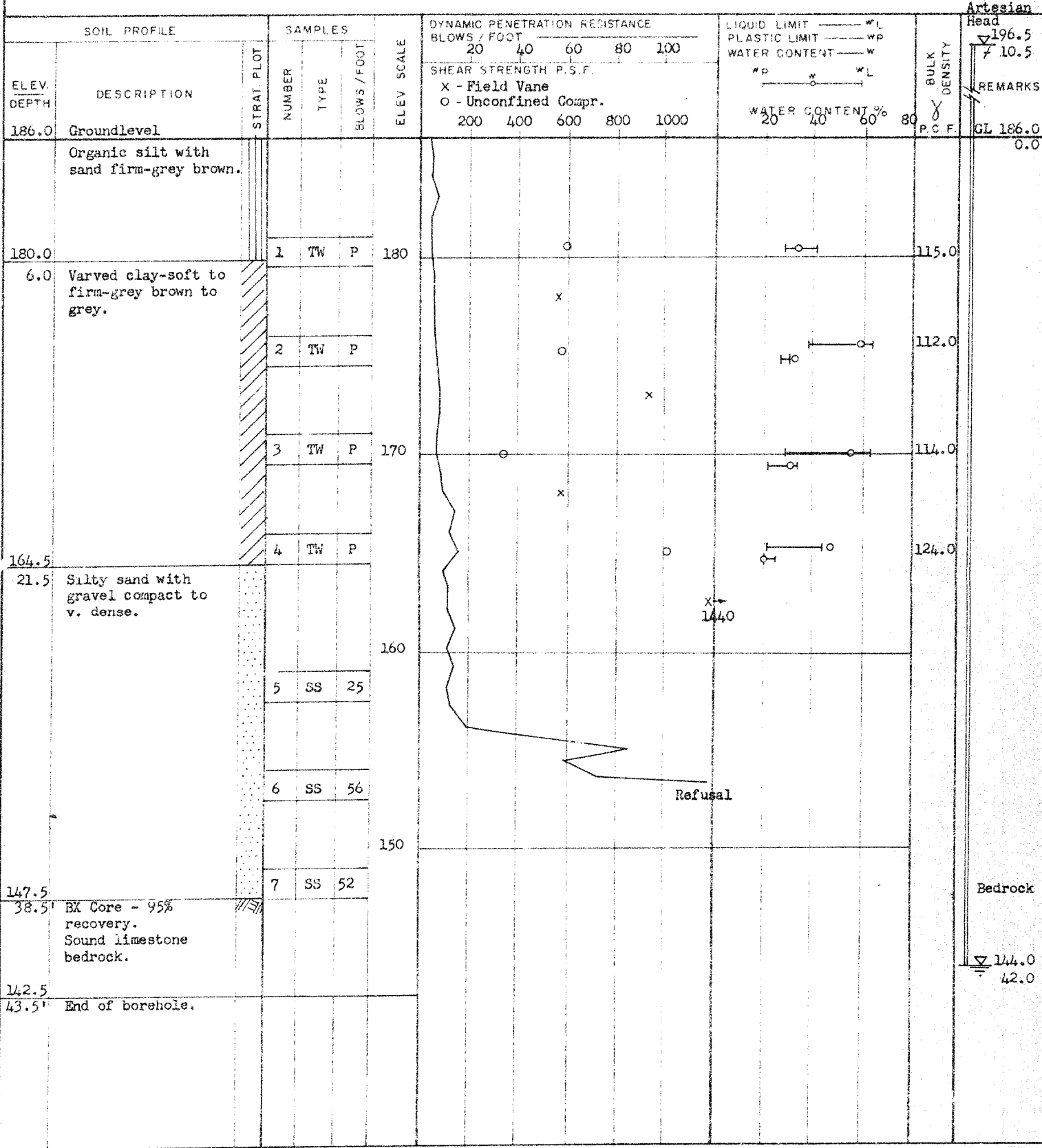
APPENDIX I.

RECORD OF BOREHOLE NO. 1

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

FOUNDATION SECTION

JOB 63-F-131 LOCATION Sta. 3+00; 15' Rt. of E ORIGINATED BY A.B.
W.P. Municipal BORING DATE Oct. 1-29, 1963. COMPILED BY M.D.
DATUM Assumed BOREHOLE TYPE Washboring, NX Casing. CHECKED BY M.D.



RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 63-F-131 LOCATION Sta. 2+20; 10' Lt. of E ORIGINATED BY A.B.
W.P. Municipal BORING DATE Oct. 30, 1963. COMPILED BY M.D.
DATUM Assumed BOREHOLE TYPE Washboring, NX Casing. CHECKED BY M.D.

[illegible]

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

Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.,
Bridge Division.

Foundation Section,
Materials & Research Div.,
Room 107, Lab. Bldg.

Attn: Mr. G.C.E. Burkhardt

April 21, 1964

Prop. New Bridge over Evanturel Creek,
Twp. of Armstrong, Lot 9, Con. V/VI,
District of Timiskaming, District 14.
W.J. 63-F-131 -- (Mun. Job)

The report on the foundation investigation for the proposed bridge at the above site was submitted on January 31, 1964. The recommendations concerning the stability of the forward slopes of the approach fills were based on a tentative proposal of a three-span bridge - (40' - 65' - 40'). It is now intended to construct a bridge with spans 30' - 40' - 30'. The stability conditions were re-examined and verbal recommendations were made which are confirmed below:

The stability conditions become more critical with the shorter spans, especially at the west abutment where an additional fill of 5 ft. is necessary. The factor of safety is very close to unity. In order to cope with this stringent condition, the following construction procedure should be followed:

The approach fills and the forward slopes should be constructed to the final grade and a period of three months should be allowed to elapse. Rip-rap should then be placed before construction for the foundations begins.

If the above construction procedure cannot be met, it is then necessary to revert to longer spans.

We believe that the foregoing recommendations will suffice for your design requirements. Should there be any further queries in connection with this project, please feel free to contact our Office.

KYL/MdeF

cc: Foundations Office
Gen. Files

KYL
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Eng.
Lab. Bldg. Room 107.

FROM: G. C. E. Burkhardt

DATE: October 10, 1963.

OUR FILE REF.

IN REPLY TO

SUBJECT: Township of Armstrong
Brown Bridge over Ewantural Creek
Lot 9, Con. V/VI
District of Timiskaming
Structure Site No. 60-64


Recoverable
0012-1

Enclosed please find one (1) copy of the Preliminary Plan for above mentioned structure.

The District Municipal Engineer from the District of New Liskeard requests, that a foundation investigation be carried out at this site by the Foundation Section.

We have not received the contract number for this project at the present time. We will forward to you the necessary information as soon as we are notified by the District.

GCEB/es


G. C. E. Burkhardt,
for K. L. Kleinsteinber,
Mun. Bridge Liaison Engineer.

cc. T. Kovich

#63-F-131

W.P. MUNIC.

NEW BRIDGE

OVER EVANTUREL

CREEK

