BH-190

HUNTING TECHNICAL AND EXPLORATION SERVICES LIMITED

RESOURCES AND DEVELOPMENT STUDIES

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September 15, 1958.

Mr. A. M. Toye,
Bridge Engineer,
Department of Highways - Ontario,
280 Davenport Rd.,
Toronto 2, Ontario.

Attention: Mr. S. McCombie

Dear Sir:

Re: W.P. 62-58 Snake River Bridge
Highway 17, District 9

We are forwarding herewith four (4) copies of our report entitled, "Foundation Investigation re the Proposed Crossing of Highway No. 17 at the Snake River near Cobden, Ontario.

We would like to express our appreciation of having the opportunity to perform this work for you.

Yours very truly,

HUNTING TECHNICAL AND EXPLORATION SERVICES LIMITED

J. Kilgour
for.

/ar

ASSOCIATE OF THE WORLD-WIDE HUNTING GROUP

REPORT OF FOUNDATION INVESTIGATION

FOR THE

PROPOSED CROSSING OF HIGHWAY NO. 17

AT THE

SNAKE RIVER NEAR COBDEN, ONTARIO

for the

DEPARTMENT OF HIGHWAYS - ONTARIO

by the

Engineering Division

HUNTING TECHNICAL AND EXPLORATION SERVICES LIMITED

Toronto, Ontario

August, 1958.

ORDER OF CONTENTS

Section 1.1

PURPOSE OF REPORT

1.11 General

The purpose of this report is to present the results of a subsurface soil investigation on the present site for a new bridge to replace the existing two-lane steel Snake River bridge on Highway No. 17, and to offer recommendations regarding a safe foundation for the new structure.

Section 1.2

DISCUSSION OF PROCEDURES

1.21 Location of Boreholes

The field location of the boreholes for this investigation was established by Department of Highways' surveyors. At the completion of the work each borehole was marked with a large stake denoting the hole number for future reference. The locations and elevations of top of the boreholes are shown on the plan in Appendix 1.71.

1.22 Subsurface Drilling and Sampling

A primary program, specified by the client, of 4 soil borings was carried out in the vicinity of the proposed site of the new Snake River Bridge.

One skid mounted, hydraulic head junior Longyear diamond drilling rig was used on this project. All boring and sampling operations were completed by experienced soil sampling crews under the supervision of engineering personnel experienced in soil sampling procedures.

All soil borings were performed by the standard wash boring procedure. By this method, drill casing was driven into the soil by a 350 lb. hammer to a depth determined by the boring supervisor. All the soil contained inside the casing during this operation was thoroughly washed out to the bottom of the casing. Sampling tools were then lowered to the bottom of the hole. The sample was then taken and the sampling tools removed from the hole. Additional lengths of casing were added as required and the procedure repeated.

"Undisturbed" samples were taken in 2-inch Shelby Tubes which were pushed into the soil and extracted. In order that samples with a minimum of disturbance might be obtained for laboratory consolidation tests, a hydraulically operated 3-inch Osterberg piston sampler was used. All tube samples were classified, tagged and sealed immediately upon recovery from the hole.

Undisturbed and remolded in situ vane shear tests were conducted in the cohesive soils using 2-inch and 3-inch diameter vanes. Pocket penetrometer tests were performed on all cohesive soils.

Attempts were made to obtain samples in the cohesionless or less cohesive soils by means of a 2-inch O. D. standard split spoon sampler. The standard penetration test using a 140 lb. hammer falling 30 inches was recorded for each foot of sampler penetration. When necessary, recovery of samples for identification and correlation was obtained with a side slit sampler. All samples were visually examined and classified on the site, then placed in jars and forwarded to the engineering office. Where samples obtained were representative and relatively undistrubed, apparent density tests were made on site to obtain the approximate specific weight of the material.

1.23 <u>Soil Testing</u>

Selective disturbed and "undisturbed" samples from each strata were forwarded to the laboratory as a check on the visual field classification, and for unconfined compression and other standard soil tests as required.

The results of all tests are given in the Office Logs and in the Appendices.

The laboratory tests were performed by:

Donald Inspection Limited,
340 Richmond Street West,
Toronto 1, Ontario.

Section 1.3

DISCUSSION OF SITE

1.31 Geographic Location

The proposed bridge site is located on the King's Highway No. 17 at the present crossing of the Snake River. The site is in the County of Renfrew, Township of Westmeath, Concessions I and II, Lot 11.

1.32 Site Geology

It would appear from an examination of the aerial photographs and from a brief examination of some of the literature on the area, that the deposits in the vicinity of the Snake River bridge site are predominantly glacio-lacustrine or possible glacio-marine in nature. North and northwest of the river the surficial materials appear more sandy with the exception of the portion shown as more clayey. The mantled gravel ridge has been checked in the field and is reported to be very silty at the road cut. It is probable that the mantle is silty but that gravel would be encountered at depth. The silty-clay to the west and east-southeast of the river bottom land appears less pervious and may be clayier at depth. A more pervious sandy clay occurs in the east and, in part, overlies a shallow bedrock ridge which is probably limestone. The valley bottom land along Snake River has been classed as organic soil and is predominantly swampy or marshy.

Drainage in the area is moderately good. With the exception of the Snake River bottom land, no zones of impeded drainage were noted on the photographs.

The surface geological features of the site area are shown in the air photo in Appendix 1.73.

1.33 Soil Conditions

The borings indicate that the soil stratification is fairly uniform throughout the site.

Bedr :k was found to occur at EL. 372 approximately.

The overburden soil, more or less 33 feet from bedrock to top of boreholes, consisted of three structural types in the following order:

1. Rock and gravel fill
2. Soft to stiff grey clay
3. Loose to dense sand and gravel with clay

The physical properties of the soil types and the bedrock are summarized below in order of their occurrence below the ground surface.

1. Rock and gravel fill:

This is presumably a rock-filled stratum laid around the existing abutments for protection against erosion by flood, and kept in place by wooden sheet piles having an approximate perimeter as shown in Plan, Appendix 1.71.

The depth of this fill is about 4 feet at both the abutments. Because of the presence of some large boulders, 4-inch diamond drilling had to be done to get through this stratum.

2. Soft to stiff grey clay:

This is a layer of grey glacial clay whose consistency varies with depth from stiff to soft.

The top portion, generally extending to about 6 feet in depth, is medium to stiff clay. In situ vane shear tests performed in boreholes indicate that the shear strength of this material is about 2,000 lbs/sq. foot in the vicinities of the north abutment, but dropped to about 1,000 lbs/sq. foot at the south abutment.

The underlying portion, which is about 14 feet in depth, is believed to be of the same type of clay but has a consistency varying from soft to very soft. In situ vane shear tests indicate this type of clay has a shear strength varying from 800 lbs/sq. foot in the upper regions to about 200 lbs/sq. foot in the lower regions.

For the very soft clay, the natural water content in the material is equal and greater than its liquid limit indicating that such soil would be very sensitive even under slight disturbance. Both field and laboratory tests gave a sensitivity of about 8 for some of the worst cases. Unless this mass of clay is sufficiently confined laterally, "clay flow" could take place under a load greater than its present overburden load.

Since the medium to stiff clay is located above the soft clay of the same type, it is believed that the upper layer has been "precompressed" by desiccation.

The void ratio of the soft clay averages about 1.4, and in volumetric measure there is about 58% of voids in the material. Thus it can be concluded that this soil is highly compressible. Since its natural water content is equal to or greater than its liquid limit, such soil can probably be classified as normally loaded clay.

3. Loose to dense sand and gravel with clay (glacial till)

This layer of soil is located immediately above bedrock.

The soil consists mainly of coarse sand and gravel, (some large stones) intermixed with clay. The material is supersaturated.

Standard penetration tests performed in this stratum gave somewhat erratic results, ranging from 6 to 36 blows per foot.

4. Bedrock

Limestone bedrock was encountered at approximately EL. 372 at the site. This is believed to be medium hard rock with slight defects and weathering along fissures and cracks, especially predominant along the surface. We estimate that such rocks would provide an allowable bearing load of about 40 tons/sq. foot.

Recovery of rock core in most boreholes is 100%, but not less than 95%, indicating the bedrock is quite solid in composition.

1.34 Water Conditions

At the time of exploration, the water table in the boreholes and the water level in the river were both found to be at EL. 403.5. Except for Borehole No. 4, boring was made in about 1.5 feet of water.

However, it was observed in Borehole No. 1 that water flowed up the hole after casings were withdrawn. This continuous flow of water might have been due to hydrostatic head created either along the more sandy and stony regions of the till layer or along the boundary regions between till and bedrock surface. There is a possibility that the flow might have come from fissures within the limestone bedrock punctured by drilling.

Laboratory tests indicate that the soft clay is supersaturated thus there should be a certain amount of free water within the material. It is probable that any gradual escape of water could result in the direction of the more porous underlying till layer.

The water levels in the river showed a maximum variation of about 5.0 feet with the high water level observed at EL. 408.7. It is difficult to estimate the effect of scouring in the clay but our opinion is that it might reach a depth around EL. 382. However, the erosion of the river banks is most likely to occur during high flood. Sheet-piles, rip-rap or other appropriate means must be provided to protect the structure from this danger.

Section 1.4

COMMENTS ON FOUNDATIONS OF STRUCTURES

1.41 General

Our understanding of the proposed bridge is that abutments are contemplated in the proximity of those of the existing structure. We have assumed that there is a raise in the grade from that of the present crossing.

The approaches to the bridge are assumed to be made up of selected fill contained and protected by wing walls where necessary.

1.42 Spread Footing Foundation

Assume the use of a spread footing foundation placed in the medium to stiff portion of the clay at EL. 398 which is about 6 feet below the present ground level. At this elevation, assuming the shear strength of the clay to be about 1,000 lbs/sq. foot, it has been estimated the bearing capacity to be about 1,800 lbs/sq. foot, factor of safety being 3.

However, if 1,800 lbs/sq. foot is applied through the footings at EL. 398, this load will create a vertical pressure of about 1,500 lbs/sq. foot on top of the soft clay at EL. 395 as estimated from Westergaard's Influence Chart for vertical pressure.

The estimated bearing load permissible on top of the soft clay at approximately EL. 395 is about 900 lbs/sq. foot. Therefore, for safe spread footing foundation at EL. 398 bearing load there should not exceed 1,100 lbs/sq. foot in order that the underlying clay will not be overstressed.

Each of the footings of the abutments is expected to settle roughly about 5.0 inches ultimately under a load of 1,100 lbs/sq. foot. This settlement analysis is based on the assumption that the soil is normally loaded clay having an average liquid limit of 49%. The net effective overburden pressure is based on locations along the boreholes on each side of the river. If footings are located behind the existing abutments, settlement is likely to be less.

1.43 Pile Foundation

(a) Steel H-Piles

Steel H-piles of section 12x12 driven to refusal into the rock should be able to provide an end bearing capacity of about 50 tons per pile. Heavier sections may be used depending on the design requirements.

Difficulties could arise in driving through the till layer immediately above bedrock, because there may be large stones or boulders. It is advisable to reinforce the penetrating end of the pile by welding plates around the point.

(b) Cast-in-place Concrete Piles

The sand and gravel till layer above bedrock could provide a good seat for bearing concrete cast-in-place piles. Cased concrete piles, cast-in-place, with compressed base section are expected to provide about 100 tons per pile.

Settlement due to such piles is often small or even negligible because of the high compaction of the immediately adjacent soil around the base or toe. This could, however, be verified by actual loading tests if necessary. If pile cap is located at EL. 397 these piles would require about 22 feet in length to reach the till layer.

1.44 Stability of Approach Fills

Our understanding of the grade of the proposed approach fills to the new bridge will be raised by about 2 feet from that of the present crossing.

For purpose of analysis we have taken the liberty of finding out the safe maximum additional fill that can be put on top of the existing crossing.

The stability of the fill on the approaches has been examined with respect to failure by sliding both in the longitudinal and transverse directions. Different cases of slide failures have been tried out. Sliding in the longitudinal direction into the river seems to be the most dangerous case, having the slip surface tangential to the top of bedrock and sliding into the river. Assuming an average cohesion of 500 lbs/sq. foot along the entire slip surface the factor of safety is about 1.30 based on an additional fill of 5 feet only.

For the raise of the 5-foot fill, there should be no danger of sliding in the transverse direction.

It is advisable to construct the abutments first before any fills on the approaches are added on.

Section 1.5

RECOMMENDATIONS

(1) Spread footing foundation for the new bridge is rejected because of the low shearing strength of the clay. In our opinion, we consider a bearing value of 1,100 lbs/sq. foot is inadequate to meet any economical design of the footings.

(2) We consider the use of a pile foundation to be the most satisfactory method to meet this type of soil conditions.

We recommend the use of steel H-piles of section 12x12 (or other available equivalent sections). Such piles driven to refusal into bedrock are expected to provide up to 50 tons per pile. Depending on design requirements heavier sections may be used. The piles should be reinforced by welding plates at the penetrating points to protect against hard driving through stony or bouldery regions of the till layer.

Battered piles should be provided to take care of any horizontal load from the structure.

(3) As an alternative, cast-in-place concrete piles formed with compressed base in the till layer may be used. If properly chosen these piles should be able to provide up to 100 tons or more per pile. Companies such as Raymond Pile Co., Franki Pile Co., Spencer, White and Prentis and others could be asked to perform the foundation work.

(4) For the approaches to the new structures, the maximum additional fill that may be safely put over the present crossing is 5 feet.

It is advisable to have the abutments constructed first before any additional fill is put on the approaches.

(5) The embankments around the abutments should be protected against erosion by flood. This may be accomplished by using rip-rap, sheet piles or other appropriate means.

Section 1.6

PERSONNEL

The field work for this project was performed under the supervision of Mr. I. E. Thurber, B.Sc.

The airphoto interpretation of the site was provided by Mr. D. K. Erb, P. Eng.

Mr. W. W. F. Wong, P. Eng. was Project Engineer, and was responsible for the writing and completion of this report.

Mr. J. Kilgour, P. Eng. provided administrative supervision of the work and reviewed this report.

Section 1.7

APPENDICES

1.71 General Plan of Site and Subsurface Sections

1.72 Office Logs of Boreholes

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 1

JOB No. H 570/58 LOCATION SNAKE RIVER
CLIENT DEPARTMENT OF HIGHWAYS – ONTARIO
COORDINATES CH. 217 + 64·0' ; OFFSET 17·5' RT. OF ℄
ELEV (surface) 404·5 (collar) ______ Datum D.H.O.
BOREHOLE NUMBER 1
DATE (started) AUG. 12, 1958 (finished) AUG. 14, 1958
RIG No. ______ TYPE LONGYEAR JR. A

Legend: silt, clay, sand, gravel, peat, fill

x — standard penetr. 2 s.s.
Δ — vane shear
o — pocket penetrometer

SAMPLE CONDITION
— undisturbed
— disturbed but represent.
— fair
— lost

γ_f — field density
γ_d — dry density
S.S. — split spoon
S.T. — shelby tube 2"
T.W.P. — thin walled piston
D.B. — diamond bit - rock core
OST. — 3" Osterberg

C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

BORING LOG

SCALE FT	DEPTH FT	ELEV. FT	WATER OBSERVATION	LOG	DESCRIPTION
0	0	404·5			
	1·0	403·5			ROCK & GRAVEL (ROCK-FILLED STRATUM)
	3·5	401·0			MEDIUM TO STIFF GREY CLAY
10	9·5	395·0			SOFT GREY CLAY
25	23·0	381·5			LOOSE TO DENSE SAND & GRAVEL WITH CLAY (GLACIAL TILL)
	33·5	371·0			L.MESTONE BEDROCK
45	45·5	359·0			END OF BORING

FIELD TESTS

Δ SHEAR STRENGTH (TONS PER SQUARE FOOT) 1/2, 1, 1 1/2
STANDARD PENETRATION TEST x (BLOWS PER FOOT) 20, 40, 60

SAMPLES

No.	COND.	DEPTH FROM FT	DEPTH TO FT	TYPE	RECOVERY LENGTH REC / DIST DRIV %	PENETRATION RESISTANCE (BLOWS PER FOOT)
1A		7·0	8·5	OST.	90	
2		12·0	14·0	S.T.	90	
3		17·0	18·5	OST.	90	
4		22·0	23·5	S.T.	50	
5		25·5	27·0	S.S.	40	6
6		27·5	29·0	S.S.		22
7		30·0	31·5	S.S.	35	8
8				D.B.	100	
9				D.B.	100	
10				D.B.	100	

LABORATORY TESTS

ATTERBERG LIMITS wp x—o wl
⊕ — NATURAL WATER CONTENT 20% 40%

Sample	γ_f pcf	γ_d pcf	U pcf	REMARKS
2	112	75		

JOB No. H 570/58 LOCATION SNAKE RIVER
CLIENT DEPARTMENT OF HIGHWAYS — ONTARIO
COORDINATES CH. 217 + 64·0; OFFSET 17·0' L. OF ℄
ELEV. (surface) 405·0 (collar) Datum D.H.O.
BOREHOLE NUMBER 2
DATE (started) AUG. 15, 1958 (finished) AUG. 18, 1958
RIG No. TYPE LONGYEAR JR. A

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 2

silt / gravel / clay / peat / sand / fill

x — standard penetr. 2 s.s.
△ — vane shear
o — pocket penetrometer

SAMPLE CONDITION
— undisturbed
— disturbed but represent.
— fair
— lost

S.S. — split spoon
S.T. — shelby tube
T.W.P. — thin walled piston
D.B. — diamond bit — rock core
OST. — 3" Osterberg

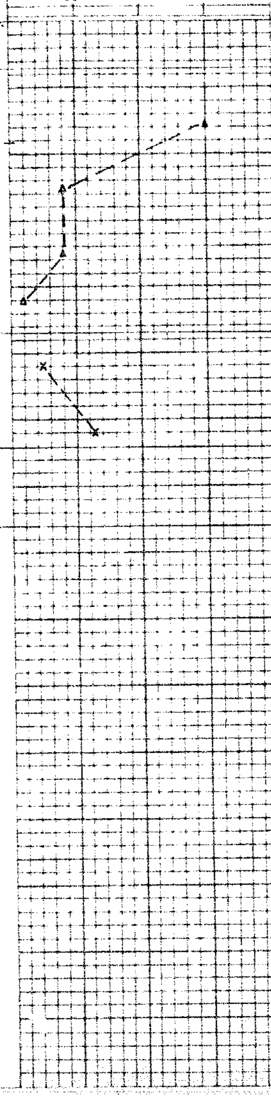
C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

BORING LOG

SCALE FT	DEPTH FT.	ELEV. FT	WATER OBSERVATION	LOG	DESCRIPTION
0	0	405·0			
	1·5	403·5			ROCK & GRAVEL
	3·5	401·5			(ROCK-FILLED STRATUM)
5					MEDIUM TO STIFF GREY CLAY
	9·0	396·0			
10, 15, 20					SOFT GREY CLAY
	23·5	381·5			
25, 30					LOOSE TO DENSE SAND & GRAVEL WITH CLAY (GLACIAL TILL)
	32·0	373·0			
35					LIMESTONE BEDROCK
	38·0	367·0			END OF BORING
40					

FIELD TESTS

△ SHEAR STRENGTH (TONS PER SQUARE FOOT) 1/2 1 1/2
STANDARD PENETRATION TEST × (BLOWS PER FOOT) 20 40 60



SAMPLES

No	COND.	DEPTH FROM FT.	DEPTH TO FT.	TYPE	RECOVERY LENGTH REC / DIST DRIV %	PENETRATION RESISTANCE (BLOWS PER FOOT)
2		5·0	6·5	S.T.	83	
3		10·0	11·5	OST.	100	
4		15·0	16·5	S.T.	100	
5		25·0	26·5	S.S.	55	9
6		30·0	31·5	S.S.		25
7		33·0	38·0	D.B.	95	

LABORATORY TESTS

ATTERBERG LIMITS wp ×—o wl
● — NATURAL WATER CONTENT

REMARKS

HUNTING TECHNICAL AND EXPLORATION SERVICES

JOB No. H570/58 LOCATION SNAKE RIVER
CLIENT DEPARTMENT OF HIGHWAYS – ONTARIO
COORDINATES CH. 216 + 88.0 ; OFFSET 16.5' RT. OF ℄
ELEV (surface) 405.5 (collar) ______ Datum D.H.O.
BOREHOLE NUMBER 3
DATE (started) AUG. 23, 1958 (finished) AUG. 25, 1958
RIG No ______ TYPE LONGYEAR JR. A

BOREHOLE No. 3

Legend: silt; clay; sand; gravel; peat; fill

x — standard penetr. 2 s.s.
△ — vane shear
o — pocket penetrometer

SAMPLE CONDITION
— undisturbed
— disturbed but represent.
— fair
— lost

S.S. — split spoon
S.T. — shelby tube
T.W.P. — thin walled piston
D.B. — diamond bit - rock core

C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

BORING LOG

DEPTH FT	ELEV FT	WATER OBSERVATION	DESCRIPTION
0	405.5		
2.0	403.5	▽	ROCK & GRAVEL (ROCK-FILLED STRATUM)
4.0	401.5		MEDIUM TO STIFF GREY CLAY
10.0	395.5		SOFT GREY CLAY
24.5	381.0		LOOSE TO DENSE SAND & GRAVEL WITH CLAY (GLACIAL TILL)
34.0	371.5		LIMESTONE BEDROCK
44	361.5		(END OF BORING)

FIELD TESTS

SHEAR STRENGTH (TONS PER SQUARE FOOT) 1/2, 1, 1 1/2
STANDARD PENETRATION TEST (BLOWS PER FOOT) 20, 40, 60

SAMPLES

No.	COND.	DEPTH FROM FT.	DEPTH TO FT.	TYPE	RECOVERY LENGTH REC. / DIST DRIV %	PENETRATION RESISTANCE (BLOWS PER FOOT)
1	fair	12.5	14.5	S.T.	92	
2	disturbed but represent.	29.5	31.0	S.S.	67	14
3	undisturbed			D.B.	100	
4	undisturbed			D.B.	100	

LABORATORY TESTS

ATTERBERG LIMITS Wp x — o Wl
● NATURAL WATER CONTENT

REMARKS

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 4

JOB No. H 570/58 LOCATION SNAKE RIVER
CLIENT DEPARTMENT OF HIGHWAYS — ONTARIO
COORDINATES CH. 216 + 88.0 ; OFFSET 18.5' L. OF ℄
ELEV. (surface) 402.0 (collar) ______ Datum D.H.O.
BOREHOLE NUMBER 4
DATE (started) AUG. 19, 1958 (finished) AUG. 21, 1958
RIG No. ______ TYPE LONGYEAR JR. A

Legend: silt; clay; sand; gravel; peat; fill

x — standard penetr. 2 s.s.
△ — vane shear
○ — pocket penetrometer

SAMPLE CONDITION
— undisturbed
— disturbed but represent.
— fair
— lost

γ_f — field density
γ_d — dry density
SS — split spoon
ST. — shelby tube
T.W.P. — thin walled piston
D.B. — diamond bit
OST. — 3" Osterberg

C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

BORING LOG

Depth (FT)	Elev. (FT)	Description
0	403.5	WATER LEVEL
1.5	402.0	GROUND LEVEL
5.5	398.0	ROCK & GRAVEL (ROCK-FILLED STRATUM)
11.5	392.0	MEDIUM GREY CLAY
27.0	376.5	SOFT GREY CLAY
32.5	371.0	LOOSE TO DENSE SAND & GRAVEL WITH CLAY (GLACIAL TILL)
47.5	356.0	LIMESTONE BEDROCK
		END OF BORING

FIELD TESTS — SAMPLES / LABORATORY TESTS

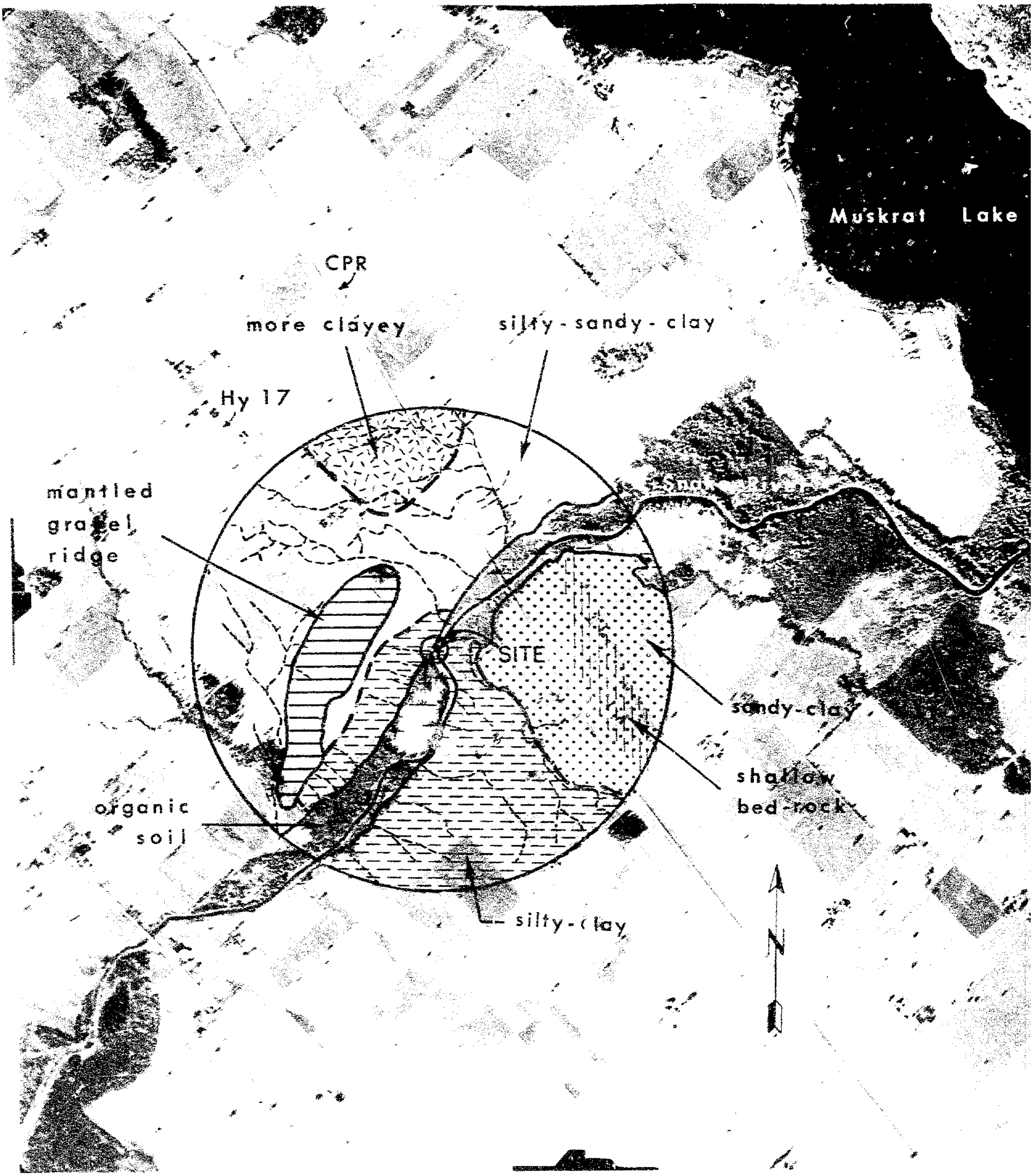
No.	Depth From (FT)	Depth To (FT)	Type	Recovery (%)	Penetration Resistance (blows per foot)	γ_f (pcf)	γ_d (pcf)	Remarks
1	6.5	8.0	SS	67	12			
2	11.5	13.0	OST.	88		106	66	
3	16.5	18.0	OST.	100				
4	23.5	25.0	S.T.	44				
5	27.0	28.5	SS	100	9			
6	31.5	32.5	SS	50	36			
7	32.5	37.5	D.B.	100				
8	37.5	42.5	D.B.	100				
9	42.5	47.5	D.B.	100				

Shear strength (tons per square foot) 1/2, 1 1/2; Standard penetration test x (blows per foot) 20, 40, 60.
Atterberg limits wp x — o wl; ● — natural water content 30% 60%.

1.73 Air Photo of Site

SNAKE RIVER · BRIDGE SITE

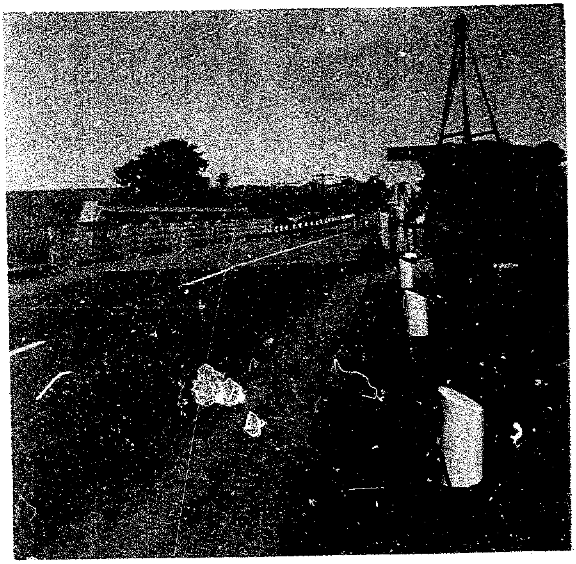
Scale - 1" = 1320'



SNAKE RIVER - BRIDGE SITE

Scale 1" = 1320'

1.74 Photos of Site and Existing Bridge



General view of existing bridge and site looking north.

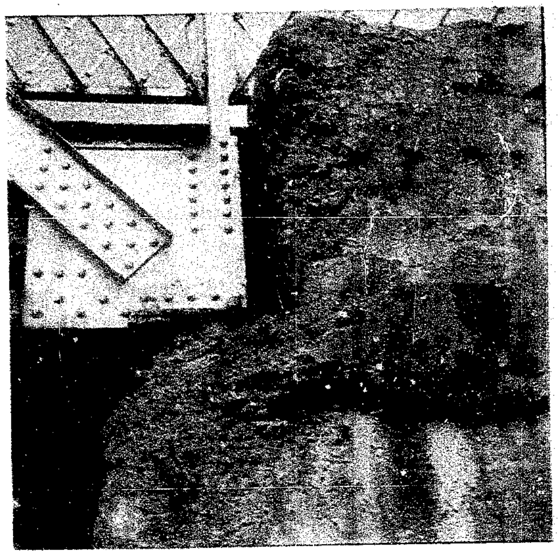
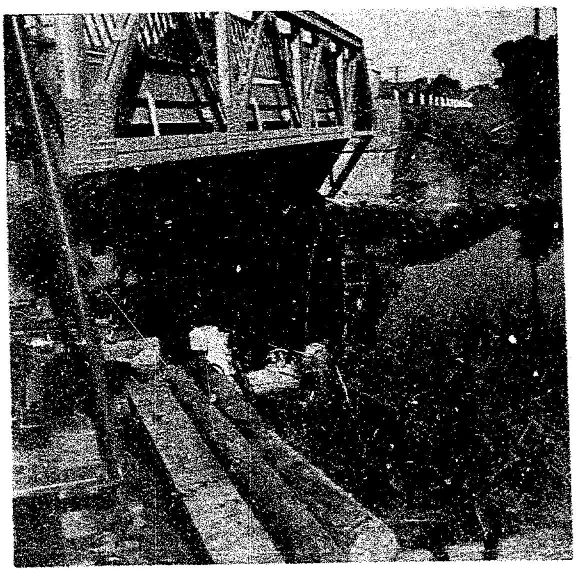
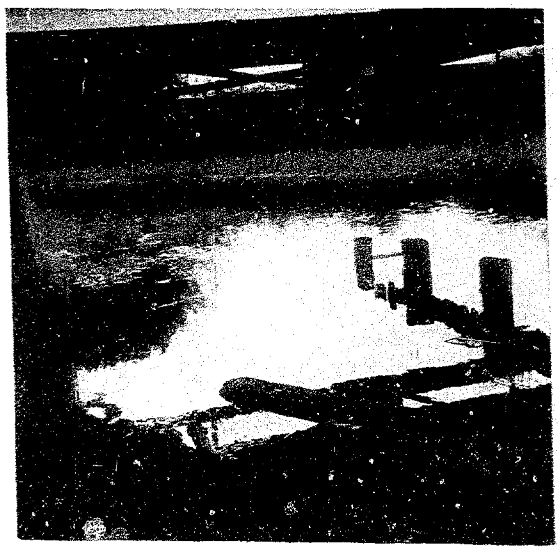


Photo of left corner of existing south abutment showing deterioration of concrete around bearing plate and anchor bolts.



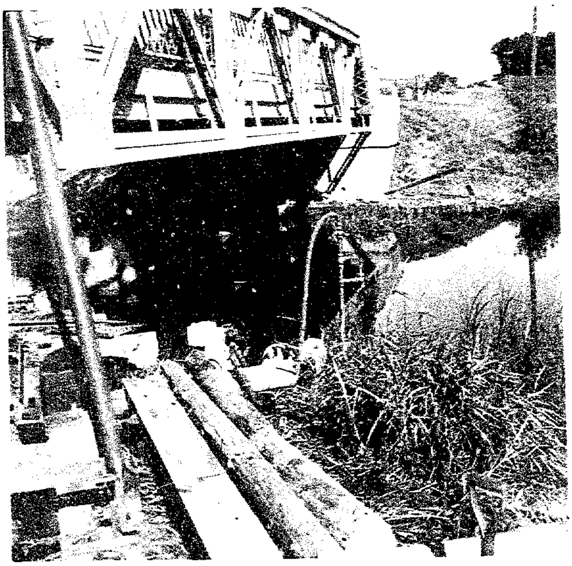
View of existing north abutment from location of Borehole No. 3 looking north.



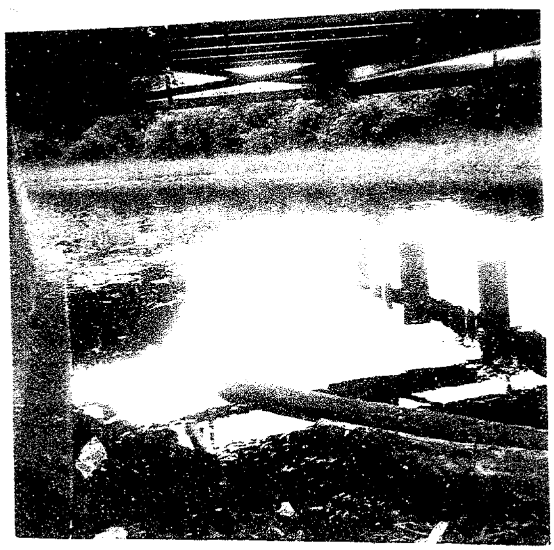
View of site at face of existing south abutment looking west.

General view of existing bridge and site looking north.

Photo of left corner of existing south abutment showing deterioration of concrete around bearing plate and anchor bolts.

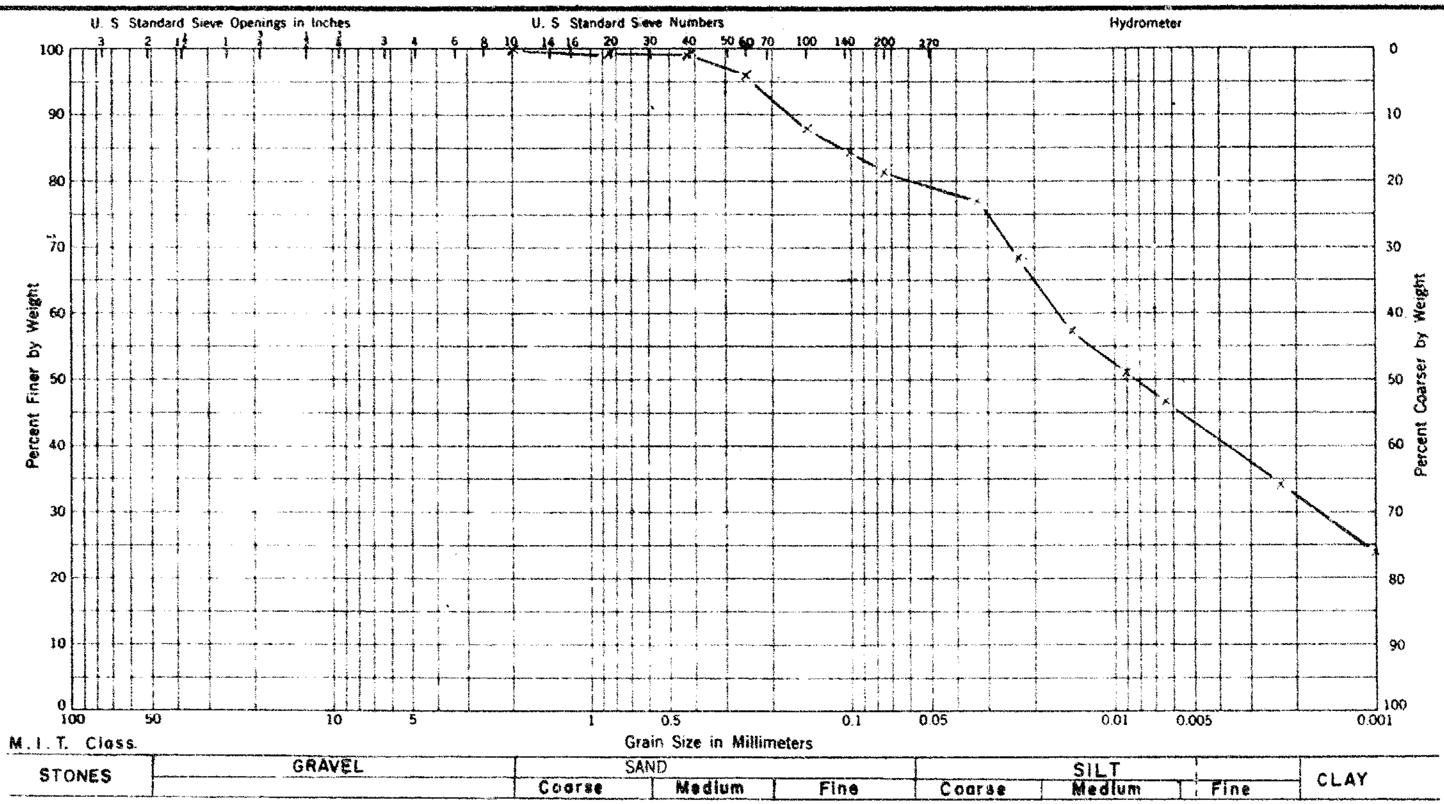


View of existing north abutment from location of Borehole No. 3 looking north.



View of site at face of existing south abutment looking west.

1.75 Soil Classification Charts



		Classification
20%	SAND	SILT WITH CLAY, SOME SAND
47%	SILT	(CLAY)
33%	CLAY	

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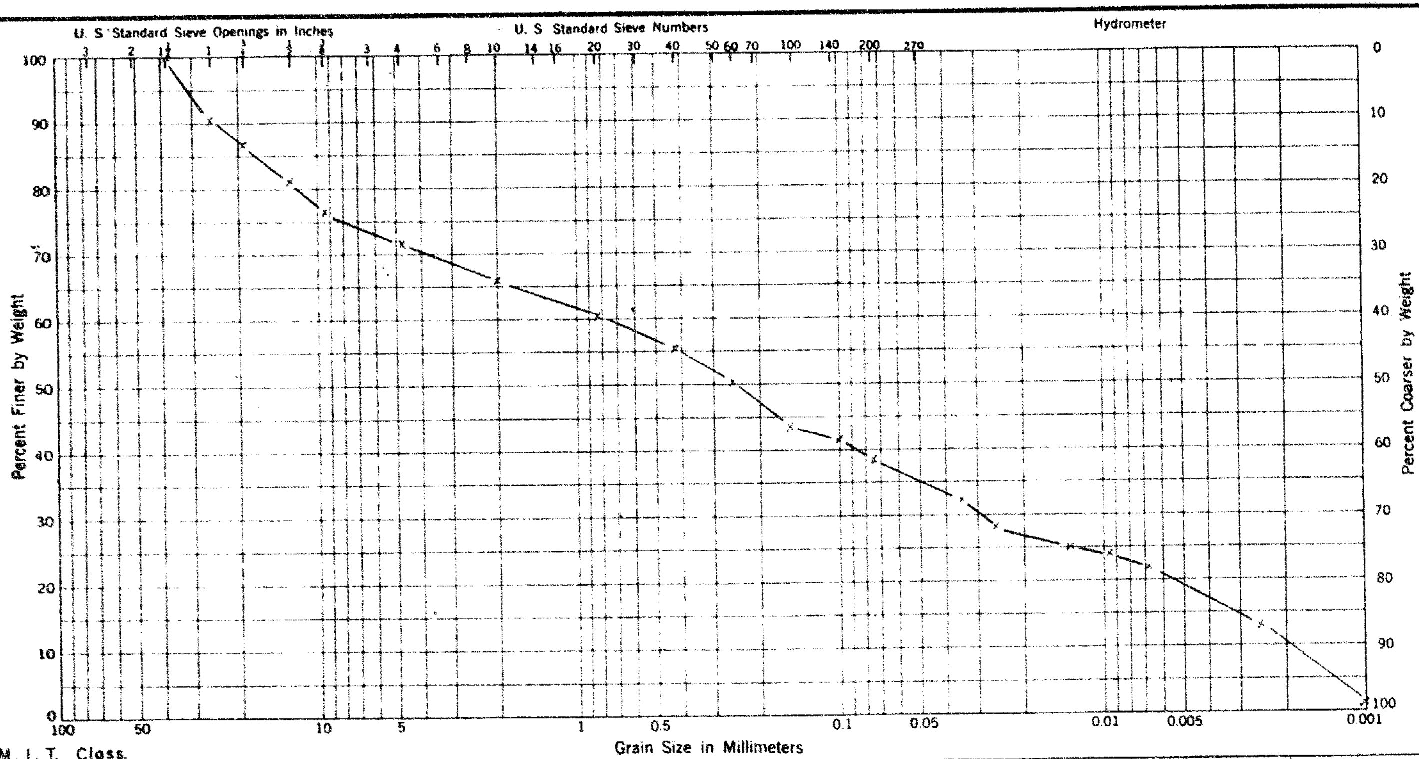
PROPOSED BRIDGE SITE

SNAKE RIVER - KING'S HWY. 17

SOIL MECHANICAL ANALYSIS

BOREHOLE 1. Depth 12' to 14'



M.I.T. Class.								
STONES	GRAVEL	SAND			SILT			CLAY
		Coarse	Medium	Fine	Coarse	Medium	Fine	

		Classification
34 %	GRAVEL	GRAVEL AND SAND
30 %	SAND	SOME SILT AND CLAY
25 %	SILT	
11 %	CLAY	

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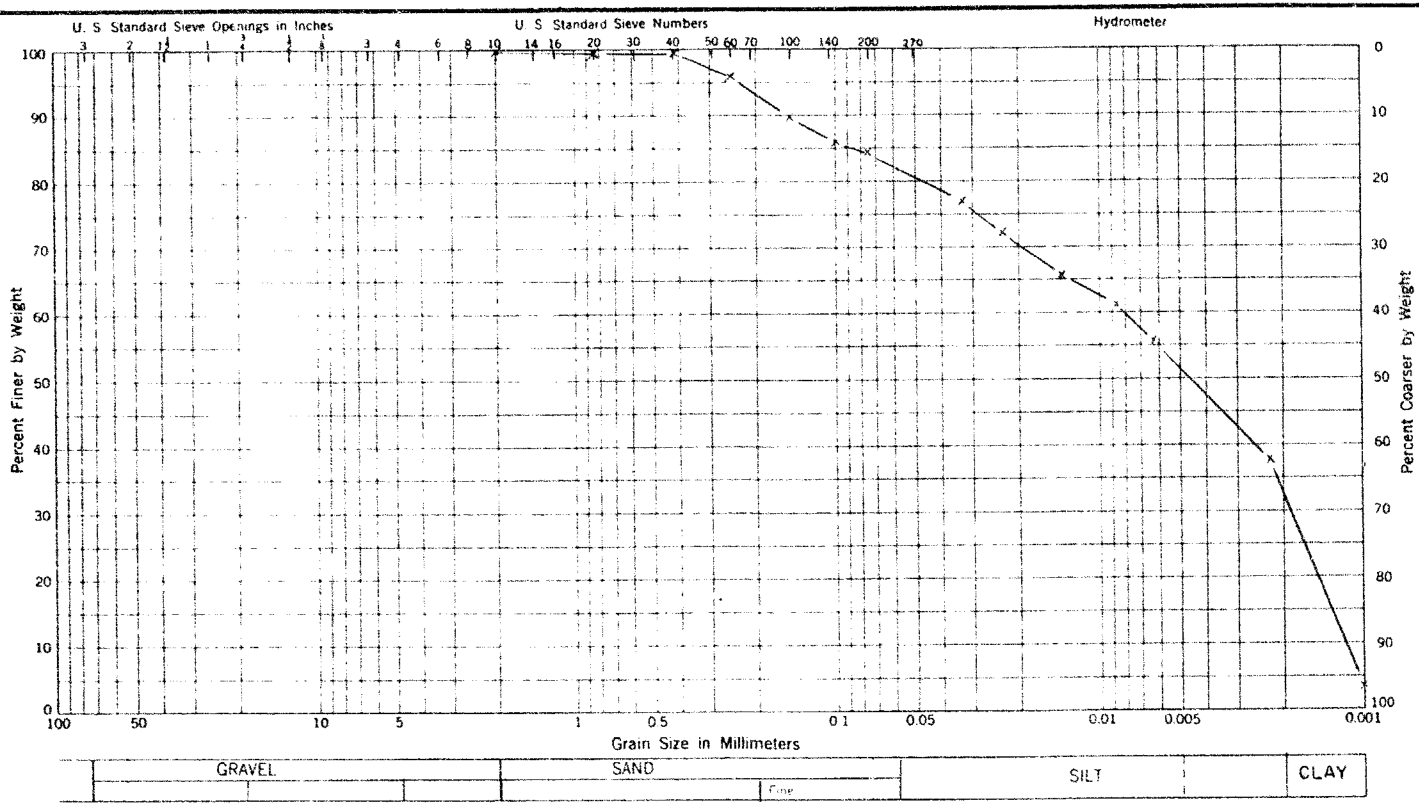
PROPOSED BRIDGE SITE

SNAKE RIVER - KING'S HWY. 17

SOIL MECHANICAL ANALYSIS

BOREHOLE 1. Depth 30' to 31·5'



		Classification
18 %	SAND	SILT WITH CLAY, SOME SAND
48 %	SILT	(CLAY)
34 %	CLAY	

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PROPOSED BRIDGE SITE

SNAKE RIVER - KING'S HWY. 17

SOIL MECHANICAL ANALYSIS

BOREHOLE 4 Depth 15' to 16·5'

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1450 O'Connor Drive Toronto, Ontario

SOIL TYPES

The following system was used in classifying the various soils by name:

Percentage	Term
100% – 50%	Predominant Material
50% – 40%	And
40% – 25%	With
25% – 10%	Some
10% – 0%	Trace

Example:

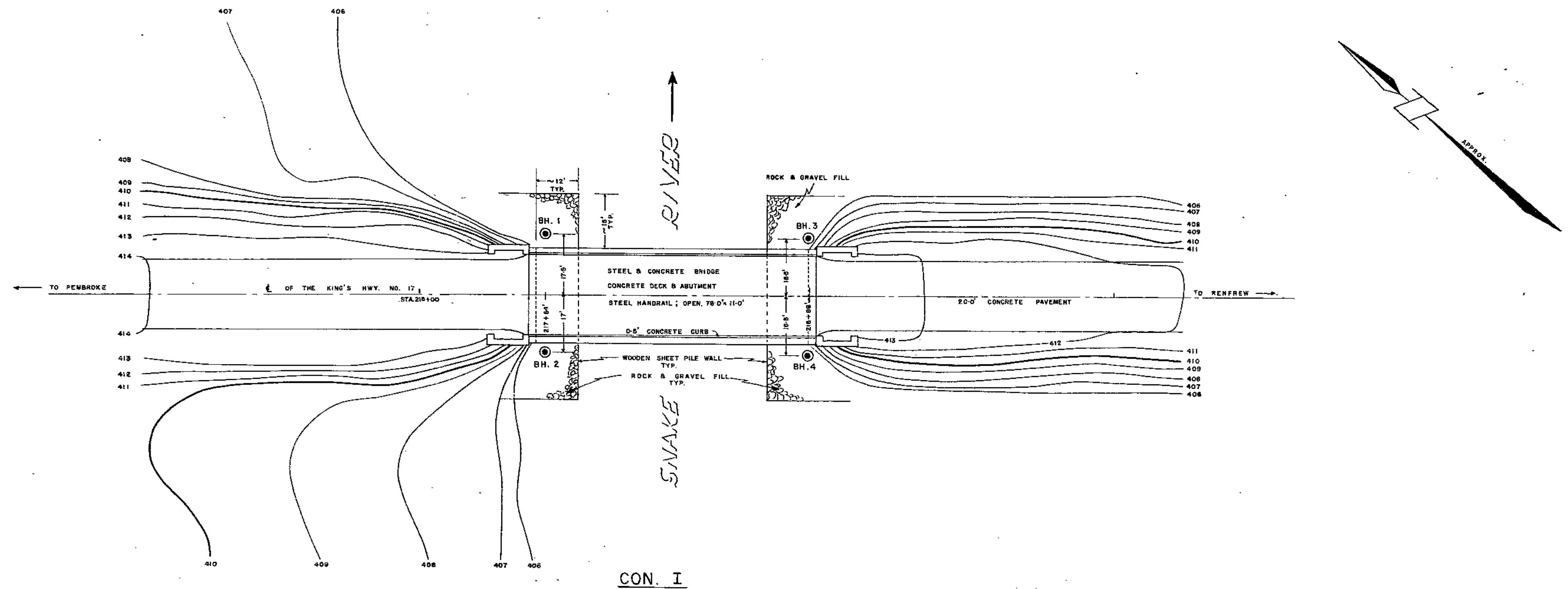
Medium dense	grey	silt	with	fine sand
(Penet. resist.) or relative density	(colour)	(pred. type)	(25%-40%)	(other type)

Unless believed to have a significant effect on the soil characteristics the minor soil types (i.e. traces) present are disregarded in the name used on the boring log and cross-sections. The complete classification is given with the gradation analysis.

In all cases the strength characteristics (e.g. penetration resistance) is quoted first, followed by the colour and finally the descriptive name based on the mechanical analysis.

#58-F-220-C
W.P. 62-58
HWY. #17
SNAKE RIVER
BRIDGE

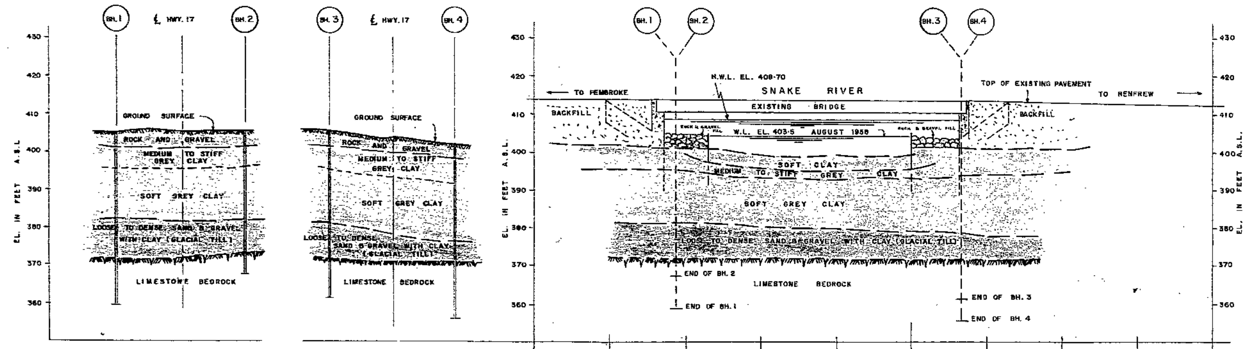
COUNTY OF RENFREW
TOWNSHIP OF WESTMEATH
CON. II LOT 11



CON. I

PLAN

SCALE : 1" = 20'



SOIL PROFILE ALONG BOREHOLES 1 & 2

SOIL PROFILE ALONG BOREHOLES 3 & 4

PROBABLE SOIL PROFILE ALONG CENTRE LINE OF HIGHWAY No. 17

SCALE — Hor. & Ver. 1 in. = 20 ft.

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TORONTO

DEPARTMENT OF HIGHWAYS - ONTARIO

LOCATION OF BOREHOLES
AND
SUBSURFACE SOIL PROFILES
FOR
PROPOSED CROSSING AT SNAKE RIVER
AND
THE KING'S HIGHWAY No. 17

BRIDGE SITE

SCALE - 1 in. = 20 ft.	DRAWN BY - C.I.B.	DATE - AUG. 1958.

Reference — PLAN E 3419-1