

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

Attention: Mr. S. McCombie.

22-68-15
W.P. 113-62
Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials and Research Division.

January 22, 1963

D.H.C. FOUNDATION INVESTIGATION REPORT -
Proposed New Bridge on Realigned Secondary
Hwy. #624 (Line 'B') and Blanche River,
Dist. of Timiskaming, Twp. of Evansdale,
Dist. #14 -- W.J. 62-F-120 -- W.P. 113-62.

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

We believe that the factual data and recommendations
contained therein, will prove adequate for your future design
work. Should further information be required, please do not
hesitate to contact our Office.

KYL/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
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syf/Lo
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

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FOUNDATION INVESTIGATION

For

Proposed New Bridge on Realigned
Secondary Hwy. #624 (Line 'B') and
Blanche River, Dist. of Timiskaming,
Twp. of Evanturel, District No. 14.
W.J. 62-F-120 -- W.P. 113-62

1. INTRODUCTION:

A request, to carry out a field foundation investigation, at the proposed new crossing of realigned Secondary Hwy. #624 - (Line 'B') and Blanche River, was received from the Bridge Location Section on October 12, 1962.

It is proposed to erect a new bridge to carry the realigned Secondary Hwy. #624 (Line 'B') over the Blanche River. The site of the proposed bridge is located in the District of Timiskaming, Twp. of Evanturel. At this location, the chainage of the Secondary Hwy. #624 (Line 'B') is from 16+75 to 21+35.

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 future design work, are contained in the following paragraphs of this report.

2. DESCRIPTION OF SITE:

The site of the proposed bridge is located approx. 7 miles North-east of the Town of Englehart, and approx. 1500 feet downstream of the existing bridge. The surrounding area is fairly flat, but

2. DESCRIPTION OF SITE: (cont'd.) ...

the actual section investigated is located in the old river bed of the Blanche River, which forms a ravine approx. 800 feet wide and 60 feet deep. The width of the Blanche River, at the proposed crossing, is about 170 feet, the depth of the river is about 15 feet, and gradually decreases towards the East bank of the river.

The site is in the physiographic region referred to as the "Timiskaming Clay Plain".

Photographs of the site are shown in Appendix I.

3. FIELD AND LABORATORY WORK:

In order to obtain sufficient information on the type and properties of the subsoil, a total of nine sampled boreholes, numbered 1 to 9, with accompanying dynamic cone penetration tests were carried out at this site. Conventional wash boring procedures were followed and samples of the overburden were taken at 5-foot intervals of depth. Samples were taken by 2" O.D. split-spoon, and by 3" and 2" I.D. thin-walled samplers. In-situ vane tests were performed 18" below the bottom of the sampler, immediately after the samples were removed.

Boreholes No. 2, 6 and 7 were put down through the overburden and 5 feet of bedrock core was taken in each borehole.

Boreholes No. 1 and 3 were terminated in the sand, gravel and boulder layer approx. 160 feet below ground elevations.

Boreholes No. 4, 5, 8 and 9 were advanced to a depth of 86 and 63 feet, respectively, and stopped in the varved clay stratum.

cont'd. /3 ...

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

Norwegian type piezometers were installed at various elevations in boreholes No. 1, 3, 6 and 8 for ground water observation.

The location and elevation of the boreholes, are given on Dwg. No. 62-F-120A, attached under Appendix I.

Tests were carried out in the laboratory on a selection of both disturbed and undisturbed samples to determine:

- 1) Natural Moisture Contents.
- 2) Bulk Densities.
- 3) Grain Size Distribution.
- 4) Atterberg Limits.
- 5) Undrained Shear Strengths.
- 6) Triaxial Shear Strengths.
- 7) Consolidation Curves.

Results of these laboratory 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 generally uniform. A detailed description of various soil types encountered during the investigation, is shown in Appendix I of this report, and is also given in subsequent paragraphs. The estimated stratigraphical profile, shown on Dwg. No. 62-F-120A, is based upon this information.

cont'd. /4 ...

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

4.2) Soft to Med. Stiff Silty Clay and Clayey Silt:

This layer, which extends to approx. El. 585.0 for a depth of about 19'-0" to 39'-0", was found below the topsoil. The upper portion of this layer has been subjected to oxidation, and exhibits a predominantly brownish colour.

Liquid limits for this layer varied from 39% to 68%, while plastic limits ranged from 19% to 26%. Moisture content determinations for this layer averaged about 42%. Bulk density determinations for this material gave values ranging from 106 to 109 P.C.F.

The clay percentage in this layer is 58%, silt forms 41%, and the rest of 1%, is sand.

To determine shear strength of this layer, in-situ vane and unconfined shear tests were carried out. The shear strengths obtained in the laboratory agree with the field vane tests. The shear strength of this material varies from 1040 P.S.F. to 400 P.S.F. The shear strength of this material decreases with depth.

4.3) Soft to Med. Stiff Varved Clay:

Immediately below the soft to med. stiff silty clay and clayey silt is a containing stratum of soft to med. stiff, dark grey varved clay, extending from about 110'-0" to 122'-0".

The separate layers of the varves are composed of dark grey clay of high plasticity and light grey silt. Clay layers generally range in thickness from $1\frac{1}{2}$ " to 3", and the silt layers from $\frac{1}{2}$ " to 1". The stratified layers were observed to lie in approx. plane sloping downwards 35° - 45° in a northerly direction.

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

4.3) Soft to Med. Stiff Varved Clay: (cont'd.) ...

Liquid limits for the clay layers varied from 54% to 74%, while plastic limits range from 22% to 30%. For the silt layers, liquid limits averaged about 28%, and plastic limits about 21%. Moisture content determinations for the clay layers varied from about 44% to 66%, and for the silt layers from about 24% to 32%. Bulk density determinations for the unseparated material gave values ranging from 106 P.C.F. to about 117 P.C.F. A typical plasticity chart for B.H. #3 is given in Appendix #1 of this report.

Grain size distribution curves indicate the clay layer to contain 51% to 67% particles of clay size, while the silt layers contain only 5% to 20% particles of clay size. In-situ vane and unconfined shear tests carried out in this material, showed some disagreement, particularly where the sensitivity was high. The vane results are considered to be more reliable and in view of this, it is estimated that the shear strength of this stratum varies from about 480 P.S.F. to about 1280 P.S.F. The shear strength of this material increases with depth.

Three series of consolidated undrained triaxial compression tests, with pore pressure measurements were carried out in the laboratory on samples of this material to define the effective stress parameter ϕ' and C' . The results are shown in Table No. 1.

cont'd. /6 ...

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

4.3) Soft to Med. Stiff Varved Clay: (cont'd.) ...

TABLE NO. I

B.H. No.	Sample No.	Sample Depth	ϕ'	C'	Remarks
1.	6	30'-0" to 32'-0"	24°	200 P.S.F.	For ($\sigma_1 - \sigma_3$) Max.
1.	8	40'-0" to 42'-0"	19°	350 P.S.F.	For ($\sigma_1 - \sigma_3$) Max.
3.	4	20'-0" to 22'-0"	23°	300 P.S.F.	For ($\sigma_1 - \sigma_3$) Max.

Consolidation tests carried out on the material from the clay layers show that the stratum is somewhat overconsolidated.

4.4) Loose Silt and Fine Sand:

This layer, approx. 15 to 20 feet thick, was found in B.H. #2, 3 and 6 underlying the stratum of varved clay. It was found to be in a very loose state with little or no penetration resistance.

It should be noted that this layer is the artesian water-bearing stratum, although artesian water pressure exists all over the investigated area, starting at approx. 5'-0" below ground elevation.

4.5) Sand, Gravel and Boulders:

In the boreholes mentioned in the previous paragraph, this layer, approx. 3 to 10 feet thick, was found immediately below the stratum of loose silt and fine sand. The overall layer is in a very dense condition with an average N' value in excess of 150 blows per foot. The boulders encountered in this layer vary in diameter from 6" to 18".

cont'd. /7 ...

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

4.6) Green Schist Bedrock:

Sound, green, schist bedrock, was encountered beneath the overburden in B.H. #2, 6 and 7. Five feet of bedrock core was taken in each borehole.

As can be seen on Dwg. #62-F-120A, the bedrock has a slight slope from East towards West.

5. GROUND WATER CONDITIONS:

Norwegian type piezometers, for water level observations, were installed in B.H. #1, 3, 6 and 8. Details of the installations are given on the Records of Boreholes and Dwg. No. 62-F-120A.

Readings taken in these piezometers over a period of 4 to 5 weeks, show that the artesian water head starts at approx. 5 feet below existing ground elevations and increases with depth, to a maximum of 19.5 p.s.i. in the layer of loose silt and fine sand, at approx. 135 - 145 feet below ground level. The distribution of excess pore pressure, assuming ground water table to be at the ground surface, is approximately linearly with depth, as shown in the Appendix.

6. DISCUSSION AND RECOMMENDATIONS:

As was described in the previous paragraphs, the subsoil is basically a soft to med. stiff silty clay and clayey silt followed by soft to med. stiff varved clay. The investigation has revealed that within the upper 20 feet of the deposit, the properties are such that adequate support for spread footings could not be

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

obtained. It is therefore suggested that the future structure be founded on piles.

The footings for the centre span will have to be founded on 'H' piles driven down to refusal. It is estimated that the 'H' piles should reach practical refusal, at or below El. 436.0 on the West bank, and El. 462.0 on the East bank, approximately. The design load will be dependent on the pile section and may be as high as 60 tons/pile in the case of 14 BP 73.

Pile caps should be founded at least 7 feet below the finished grade at approx. El. 582.0 as shown on Dwg. No. 62-F-120B.

To prevent the possibility of the artesian water from washing out particles of clay, along the vertical surface of the piles, a sand and gravel filter 3 feet thick, should be placed below the footings.

Due to the presence of artesian water condition, the excavations for the pile caps should be carried out inside steel sheet piling, which is driven at least 5 feet into the varved clay stratum, to prevent instability of excavations. The sheet piling should be adequately strutted.

To facilitate the driving of piles, it is suggested that a platform of adequate size to accommodate the pile driving equipment be formed, which later on should be filled in to form the finished grade as shown on Dwg. No. 62-F-120B.

It is further recommended that all approach spans should be approximately 40 feet in length. Pile caps, for the approach spans, should be founded on displacement friction piles. Treated timber

cont'd. /9 ...

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

piles would be best suited for this purpose; it is believed that an allowable load of 15 tons per pile could be obtained. It is strongly recommended that loading tests be carried out to establish more definitely, a suitable design load since the behaviour of friction piles in artesian water condition is not well understood. Depending on the pile load tests, the future pile driving operation may have to be controlled. The bottom of these pile caps should be 7 feet below finished grade level.

During a recent discussion with the Hydrological Section, it was established that the centre span should be approx. 260 feet long.

An effective stress analysis was carried out to assess the long-term stability of the existing river banks. Computations showed that the factor of safety of the existing slope, 2.3 horizontally to 1 vertically, is about 1.08. Examination of existing banks during the field investigation, also showed signs of instability. Therefore, it is recommended, to cut the river banks to a slope $\frac{4}{1}$ horizontally to 1 vertically, as shown on Dwg. No. 62-F-120B. The river bank slopes should be protected against surface erosion by sodding. All other slopes should be protected against surface erosion by seeding and mulching.

It is emphasized that the slopes be cut back before excavation for pier footings is carried out.

Based on the shear strength of the subsoil (750 P.S.F.) the height of the approach fills should not exceed 20 feet. Settlements of the proposed embankments will occur due to consolidation of the subsoil, under the additional weight of fill,

cont'd. /10 ...

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

but due to difficulty in estimating the probable drainage conditions and consolidation properties of the stratified subsoil, it is impossible to predict accurately, the time rate and amount of settlement. Because of the expected settlements, a flexible type of pavement is recommended. Before final paving, it would be preferable to allow a period of about 12 months to elapse.

It is further recommended, that due to the expected total and differential settlement of the fill and the foundations, the spans of the future bridge, should be simply supported.

7. SUMMARY:

1. The stratification of the soil is quite uniform. The relative consistency of the materials encountered, vary from soft to med. stiff.
2. Because of the soft consistency of the upper layers, a structure supported on piles is recommended.
3. The footings for the centre span should be founded on steel 'H' piles driven down to refusal. A design load of 60 tons per pile, may be employed in the case of 14 BF 73. The bottom of the pile caps should be at El. 582.0.
4. To prevent the possibility of the artesian water from washing out particles of clay along the vertical surface of the 'H' piles, a 3-foot thick sand and gravel filter should be placed below the pile caps.

cont'd. /11 ...

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

5. The approach spans should be approximately 40 feet in length.
6. Footings for the approach spans should be founded on displacement friction piles. Treated timber piles 40 feet in length, would be best suited for this purpose. It is believed that an allowable load of 15 tons per pile could be obtained.
7. Loading tests will have to be carried out to determine more accurately, the design load. Depending on the pile load test, the future pile driving operation may have to be controlled.
8. The bottom of the pile cap elevation should be 7 feet below the finished grade level.
9. Because of the expected settlement of piles, the spans of the future structure, should be simply supported.
10. Based on the shear strength of the subsoil (750 P.S.F.), the approach fills should not exceed 20 feet in height.
11. Due to the anticipated settlement of the fill, a flexible type of pavement is recommended. Before final paving, it would be preferable to allow a period of about 12 months to elapse.
12. The existing river banks should be cut to a slope 4 horizontally to 1 vertically, as shown on Dwg. No. 62-F-120B. It is imperative that this operation be carried out before the excavation for foundations begins.

cont'd. /12 ...

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

13. To protect the river bank slopes against surface erosion, they should be sodded. All other slopes should be protected against surface erosion by seeding and mulching.

14. The Hydrological Section indicates that the centre span should be approx. 260 feet long.

8. MISCELLANEOUS:

The field work, performed from October 19 to November 29, 1962, together with the preparation of this report, was undertaken by Mr. W. W. Kulmatickas. The investigation was carried out under the general supervision of Mr. K. Y. Lo, who reviewed this report.

January 1963.

APPENDIX I

FOUNDATION SECTION

CHECKED BY

[illegible]

FOUNDATION SECTION

ORIGINATED BY W.W.K.

COMPILED BY W.W.K.

CHECKED BY

[illegible]

FOUNDATION SECTION

CHECKED BY _____

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & RESEARCH DIVISION		RECORD OF BOREHOLE NO. 5										FOUNDATION SECTION	
JOB <u>62-P-120</u>		LOCATION <u>Br. 62 & Blanche River Line "B" Ch. 15/25-30'-5" Lk.</u>										ORIGINATED BY <u>S.W.K.</u>	
W.P. <u>113-62</u>		BORING DATE <u>Nov. 12, 13, 1962</u>										COMPILED BY <u>H.W.K.</u>	
DATUM <u>645.8</u>		BOREHOLE TYPE <u>Washboring HK Casing</u>										CHECKED BY _____	
SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE	BLGS / FOOT	ELEV. SCALE	BLOWS / FOOT 20 40 60 80 100	SHEAR STRENGTH P.S.F. + Field Vane	WATER CONTENT % 20 40 60					
645.8	Ground Surface												
642.7	Organic topsoil												
3.1			1 SS 11		640		+ 4						
			2 TW P				7.3 +						
	Soft silty clay and clayey silt. (Dark Grey)		3 SS P		630		+ 9.3						
			4 TW P				9						
			5 SS P		620		+ 10						
616.1			6 TW P										
31.7			7 SS P		610		+ 11						
	Soft to med. stiff varved clay. (Dark Grey)		8 TW P				9.8						
			9 SS P		600		9.2 +						
			10 TW P				11.5						
			11 SS P		590		+ 8.8						
			12 TW P				+ 7						
582.3							+ 7.3						
53.5	End of borehole.				580								
					570								
					560								
					550								
					540								
					530								
					520								
504.8	End of Penetration				510								
411.0	Probably Bedrock.				500								
					490								

RECORD OF BOREHOLE NO

JOB 62-P-12P LOCATION Box 624 & Blanche River Line "B" Ch19/95-151-1" R. ORIGINATED BY W.W.K.
W.P. 113-62 BORING DATE Nov. 19 to Nov. 26, 1962. COMPILED BY W.W.K.
BATHYM 602.5 BOREHOLE TYPE Washboring HK and RK Casing. CHECKED BY _____

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE SLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE		20	40	60	80	100	WATER CONTENT % 20 40 60				
						SHEAR STRENGTH P.S.F. + Unconfined Shear Strength + Field Vane + Lab Vane									
						250	500	750	1000	1250					
602.5	Ground Surface														P8 P9 P10 P11
0.0															
598.8	Organic Topsoil														
3.7	Med. stiff silty clay. (Brownish Grey)		1	SS	A					3.2					Elev. 592.
			2	TM	P					6					Elev. 582.
598.5															
14.0	Loose clayey silty sand		3	SS	P										
591.8	(Dark Grey)														
18.7			4	SS	P					5.3					
			5	TM	P					9					
			6	TM	P					9					
			7	SS	P					5.7					
			8	TM	P					9.9					
			9	SS	P					9.6					
			10	TM	P					8					
			11	SS	P					15					
			12	TM	P					8.8					
			13	SS	P					7.3					
			14	TM	P					9.2					
			15	SS	P					7.3					
			16	TM	P					9					
			17	SS	P					7.2					
			18	TM	P					7.2					
			19	SS	P										
			20	TM	P					6					
			21	SS	P										
492.3			22	SS	P										
490.8	Loose silt & fine sand														
111.7	Medium stiff varved clay. (Dark Grey)		23	TM	P					7.2					
			24	SS	P										
			25	TM	P					1.6					
473.5															
129.0	Alternate layers of sandy silt and silty clay		26	SS	P										
	Layers 12" to 15" thick.		27	SS	P										
463.5															
462.0	Boulders up to 12" & 15"														
460.5	Green schist														
457.0	Bedrock														
445.0	End of borehole.														

Piezometers installed in BH at elev. shown

DEPARTMENT OF HIGHWAYS, ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 62-P-120 LOCATION Rwy. 624 & Blanche River Line "B" Tr. 22400
W P 113-62 BORING DATE Nov. 19 to Nov. 26, 1962.
DATUM 627.7 BOREHOLE TYPE Neighboring HX and HX Castings.

ORIGINATED BY W.H.K.
COMPILED BY W.H.K.
CHECKED BY _____

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — % PLASTIC LIMIT — % WATER CONTENT — %			BULK DENSITY PCF	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLUG	NUMBER	TYPE		20	40	60	80	100	W _p	W _L	W		
						SHEAR STRENGTH P.S.F. • Unconfined Shear Strength • Field Vane • Lab Vane					WATER CONTENT % 20 40 60				
						250	500	750	1000	1250					
627.7	Ground Surface														
625.2	Organic topsoil														
625.2	Med. stiff silty clay.		1	SS	15			7.5							
616.7	(Brownish Grey)		2	TH	P			6.3					Clay		114.0
611.0	Soft silty clay and clayey silt.		3	SS	P			10.5					Silt		
	(Dark Grey)		4	TH	P			7					Clay		107.0
			5	SS	P			21.4					Clay		109.0
596.7			6	TH	P			5					Clay		109.0
591.0			7	SS	P			9					Clay		110.0
			8	TH	P			10.2					Silt		110.0
			9	SS	P			8.4					Clay		118.0
			10	TH	P			13.1					Silt		112.0
			11	SS	P			10.2					Clay		118.0
			12	TH	P			10.5					Silt		112.0
			13	SS	P			8					Clay		110.0
			14	TH	P			12					Silt		110.0
			15	SS	P			9					Clay		110.0
			16	TH	P			10					Silt		110.0
			17	SS	P			6.5					Clay		110.0
			18	TH	P			7.2					Silt		105.0
			19	SS	P			8.2					Clay		105.0
			20	TH	P			7.1					Silt		108.0
			21	SS	P			9.4					Clay		108.0
			22	TH	P			7.4					Silt		108.0
			23	SS	P			7					Clay		105.0
			24	TH	P			3.4					Silt		105.0
			25	SS	P			3.2					Clay		105.0
475.2	Green schist		26	TH	P			7.2					Silt		105.0
462.2	Bedrock		27	TH	P			7.2					Clay		105.0
458.8	End of borehole.		28	SS	P			7.2					Silt		105.0

[illegible]

RECORD OF BOREHOLE NO.

FOUNDATION SECTION

443 65-1420

LOCATION Hwy. 624 & Blanche River Line "B" Ch20/65-15'-0" ls.

ORIGINATED BY W.H.A.

W 2 113-62

BOARDING DATE Nov. 26, 1962

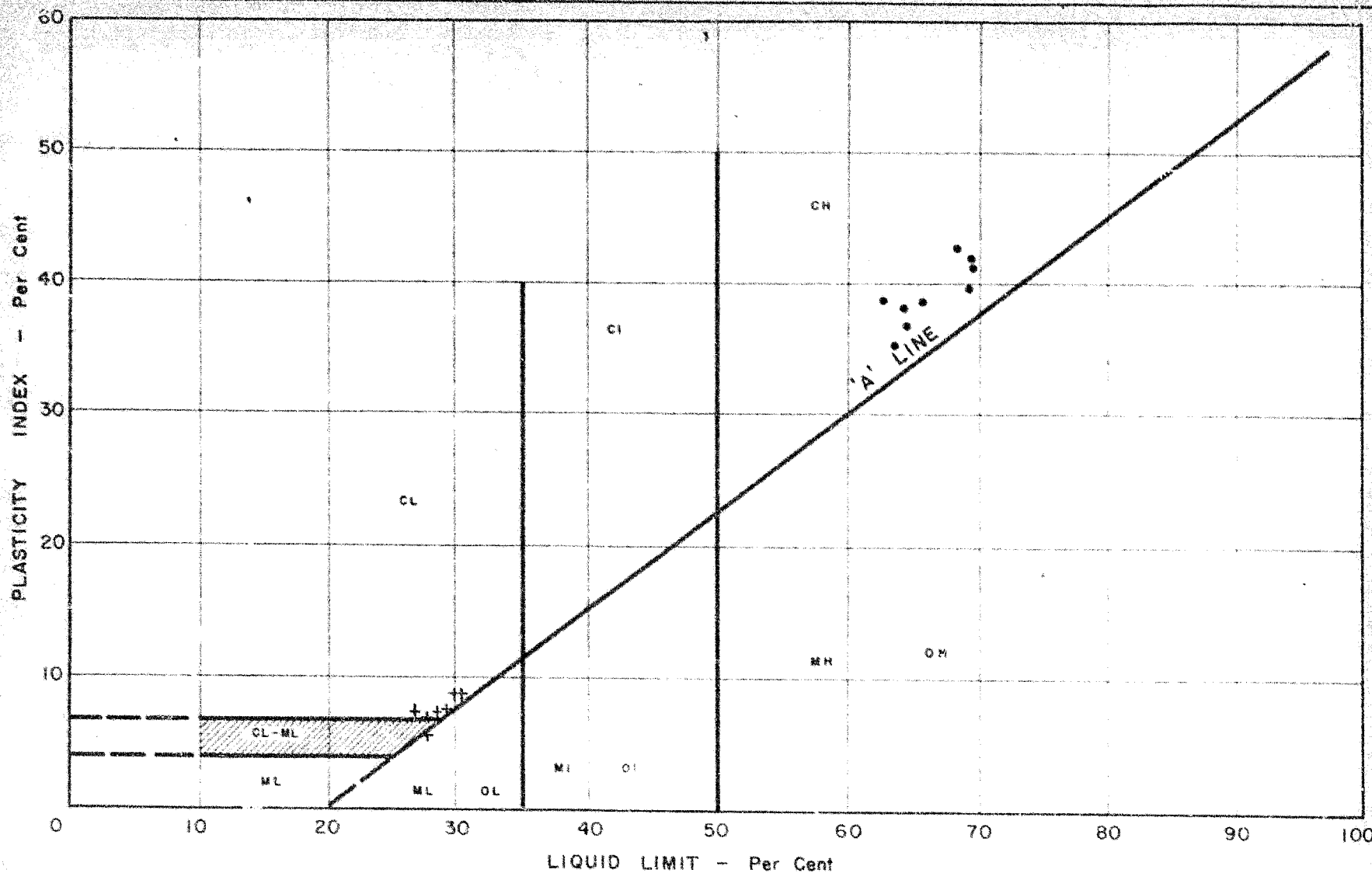
COMPILED BY W.H.K.

BAT356 650.0

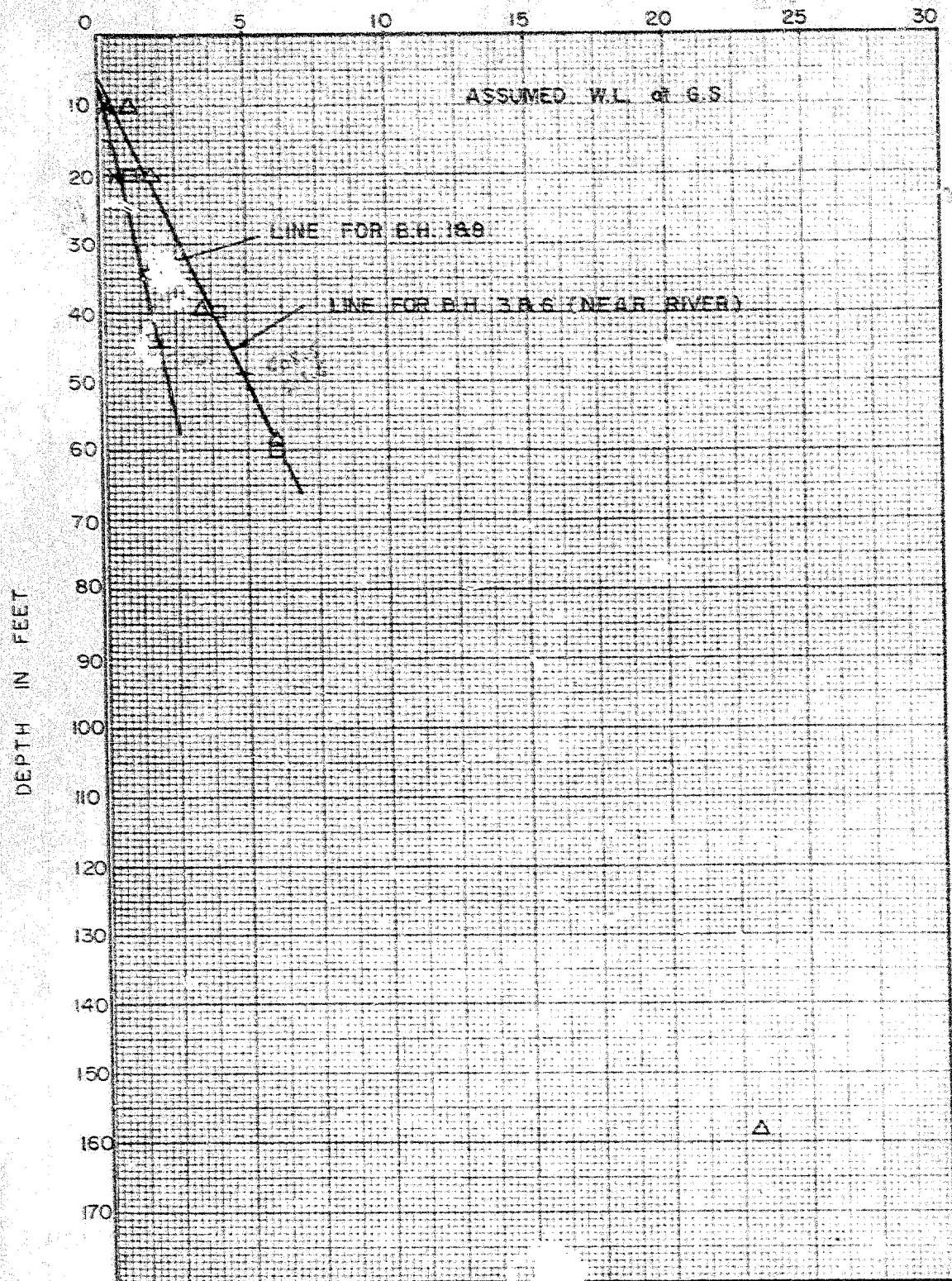
BOREHOLE TYPE Washboring HX Casing

CHECKED BY

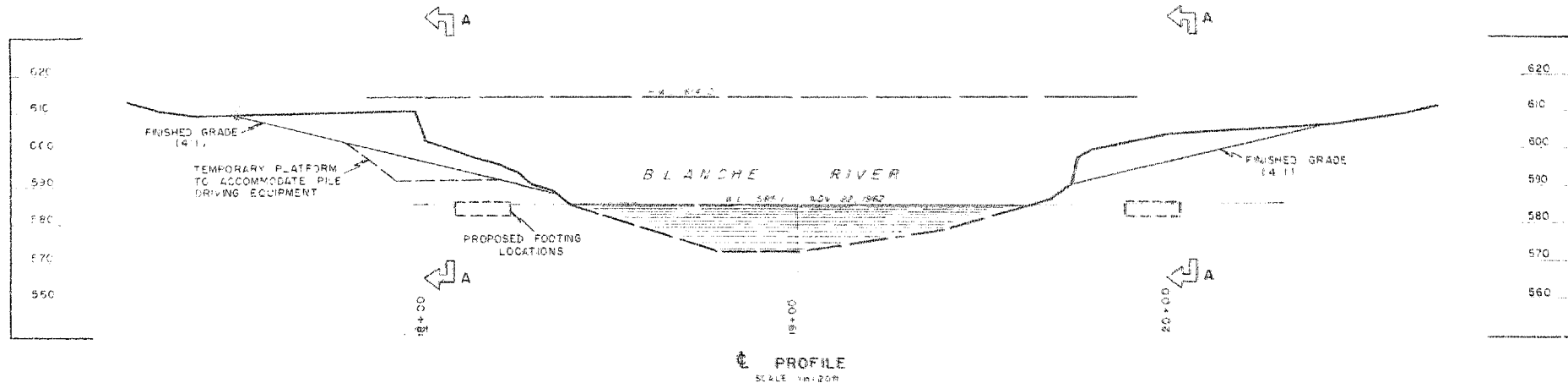
[illegible]



EXCESS PRESSURE (P.S.I.)



- + B.H. NO. 1
- B.H. NO. 3
- △ B.H. NO. 6
- x B.H. NO. 8



TYPICAL SECTION A-A

SCALE 1 in = 20 ft

DESIGNED BY K. L. MATICKAS	DEPARTMENT OF HIGHWAYS - ONTARIO	SCALE AS SHOWN
DRAWN F. CLARK	MATERIALS & RESEARCH SECTION	CONT. NO. --
CHECKED [Signature]		JOB NO. 62-F-120
APPROVED [Signature]		DWG NO. 62-F-120
DATE JAN. 22, 1963		

BLANCHE RIVER
FOOTING LOCATIONS

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 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

D.H.O. FOUNDATION INVESTIGATION
Blanche River and Sec. Hwy. 624
Line 'B' Revision
W.J. 62-F-120 - W.P. 113-62
District 14



Photo No. 1

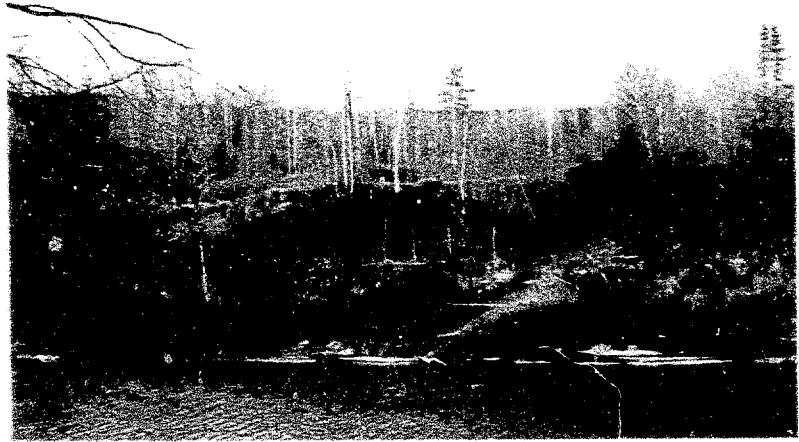
West Bank



Photo No. 2

West Bank (farther back)

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D.H.O. FOUNDATION INVESTIGATION
Blanche River and Sec. Hwy. 624
Line 'B' Revision
W.J. 62-F-120 - W.P. 113-62
District 14



Photo No. 3

East Bank

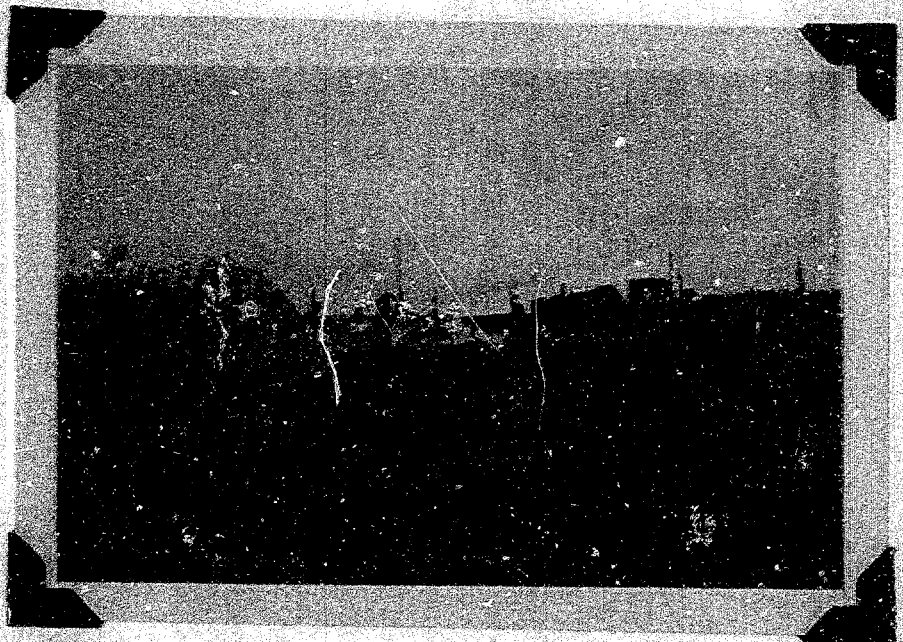


Photo No. 4

East Bank (farther back)

D.H.O. FOUNDATION INVESTIGATION
Blanche River and Sec. Hwy. 624
Line 'B' Revision
W.F. 62-7-120 - W.P. 113-62
District 14

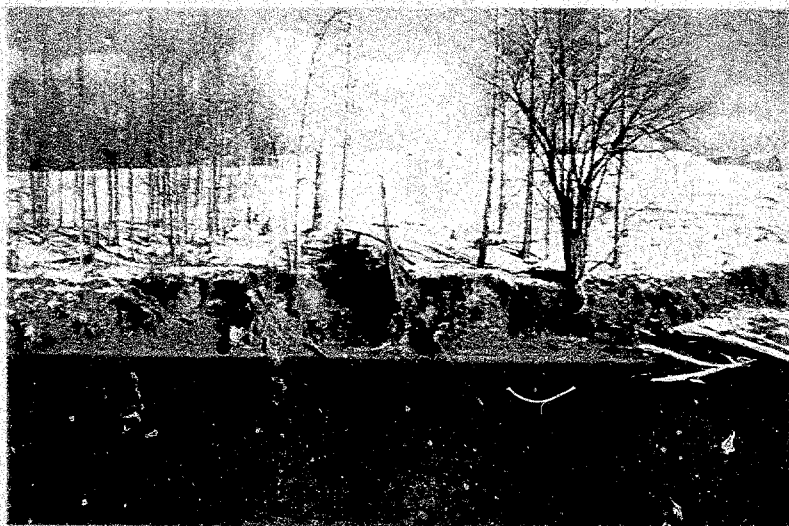


Photo No. 3

East Bank



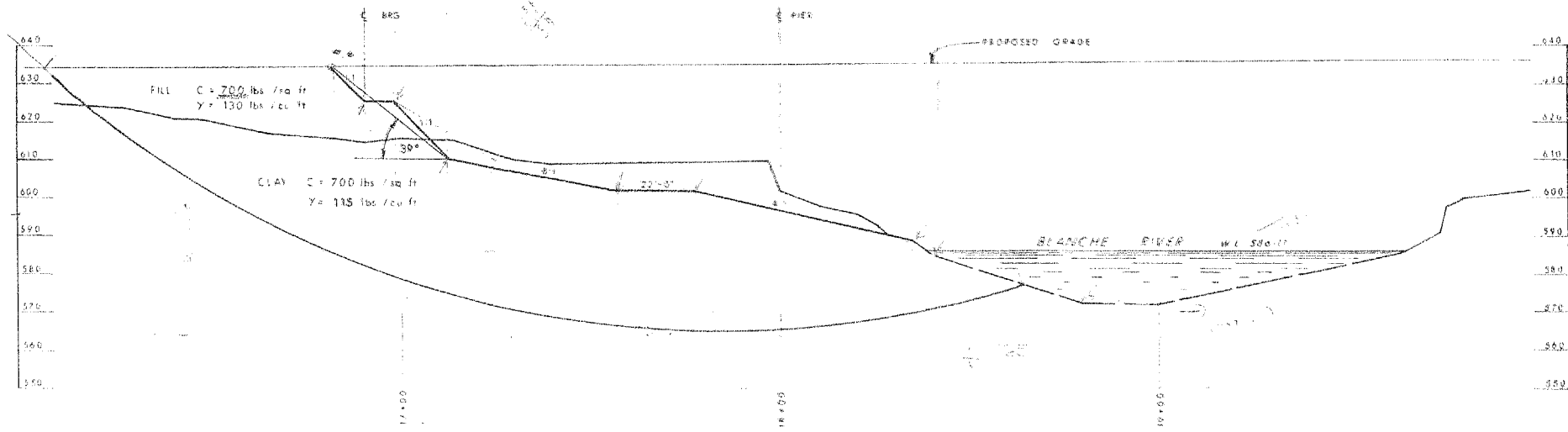
Photo No. 1

East Bank (farther back)

R=2690

60.290

RADIUS = 2690' FS = 1.032



TOTAL STRESS ANALYSIS
BLANCHE RIVER & SEC. HIGHWAY NO. 624

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

Mr. A. G. Sternac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
February 5, 1961.

Attention: Mr. J. McCombie.

D.E.C. FOUNDATION INVESTIGATION REPORT -
Proposed New Bridge on Realigned Secondary
Hwy. #624 (Line 'B') and Blanche River,
Dist. of Tisdalehaming, Twp. of Avanturel,
Dist. #14 -- S.J. 62-7-120 -- S.P. 113-62.

Attached, are Site Plan and Graph which supersede
those contained in the appendix of the above report recently
forwarded to you.

Would you, therefore, kindly remove the existing
Site Plan and Graph from your copy(s) of the report and insert
the replacements.

Thank you.

KIL/maef
Attach. (2)

cc: Messrs. A. M. Toye (2)
H. A. Fregaskes
H. D. McMillan
H. McArthur
H. C. Chapman
A. E. Saint
T. J. Kovich
J. Roy
J. S. Grunprier
S. Moran
A. Watt


K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:

A. G. Sternac,
PRINCIPAL FOUNDATION ENGR.

Foundations Office
G.B. Giles.

62-F-1
09144

1963 JAN 31 PM 4 14 2

DOWN LISK 6 JAN 31/63 4:30P

A G STERNAC PRINCIPAL FOUNDATION ENGINEER

ATTENTION: K Y LO SUPERVISING FOUNDATION ENGINEER

RE: FOUNDATION INVESTIGATION BLANCHE RIVER SEC HWY 624 WP 113-60

DISTRICT 14 JOB 62F-120

KEY PLAN IS INCORRECT. SEC HWY 624 RUNS DUE SOUTH FROM PRESENT BRIDGE
AND NOT WEST AND SOUTH TO ENGLEHART. SITE CIRCLED IS PRESENT
STRUCTURE AND NOT PROPOSED SITE ON REVISED LINE "B". ERROR IN KEY
PLAN ONLY.

R S CHAPMAN DISTRICT ENGINEER

DP

Corrected *Kylw*

BA 1578A

MEMORANDUM

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

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

Attention: Mr. S. McCombie.

DATE: February 5, 1963.

OUR FILE REF.

IN REPLY TO

SUBJECT:

D.H.O. FOUNDATION INVESTIGATION REPORT -
Proposed New Bridge on Realigned Secondary
Hwy. #624 (Line 'B') and Blanche River,
Dist. of Timiskaming, Twp. of Evanturel,
Dist. #14 -- W.J. 62-F-120 -- W.P. 113-62.

Attached, are Site Plan and Graph which supersede those contained in the Appendix of the above report recently forwarded to you.

Would you, therefore, kindly remove the existing Site Plan and Graph from your copy(s) of the report and insert the replacements.

Thank you.

KYL/MdeF
Attach. (2)

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
H. McArthur
R. S. Chapman
E. R. Saint
T. J. Kovich
J. Roy
J. E. Gruspier
F. Norman
A. Watt

KYL
K. I. Lo,
SUPERVISING FOUNDATION ENGR.
For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

Foundations Office
Gen. Files.

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

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

Attention: Mr. S. McCombie

June 17, 1963

Proposed New Bridge Location on Realigned
Secondary Hwy. No. 624 and Blanche River,
District of Timiskaming, Twp. of Evanturel,
District No. 14 -- W.P. 113-62

The Foundation Section has carried out the investigation for Line 'B' of the above-mentioned highway where it crosses Blanche River. The results, together with the conclusions and necessary recommendations, are contained in the Report W.J. 62-F-120 which was submitted to you on January 22, 1963.

The investigation showed the subsoil conditions to be rather unfavourable and, therefore, the problems to be expected, quite difficult. Consequently, the cost of the structure would become high.

Before final decisions are reached, it was suggested that an investigation be carried out to prove whether a more favourable crossing could be found in the immediate vicinity. We felt that the best and most expedient way was to carry out a geophysical survey and, therefore, made all the necessary arrangements for it.

During the month of May, a geophysical investigation was carried out by the crew under the supervision of Mr. Alex Szenasi of the Soil Section, and a report containing the findings and conclusions was prepared and submitted on June 11, 1963. The geophysical investigation confirmed the belief that the subsoil conditions in the area are basically uniform and no appreciable variations exist.

The geophysical investigation was carried out at 500-ft. intervals to a distance of 3,900 ft. to the north and south of the existing bridge, respectively. Line 'B' that was investigated by the Foundation Section, is approx. 1,500 ft. south (downstream). The geophysical investigations were carried to a distance of 400 ft. on both river banks.

The overburden was found to be essentially of the same properties as on Line 'B', and also, the depth to bedrock does not essentially differ throughout the seismically surveyed area from the findings of the drill investigation on Line 'B'.

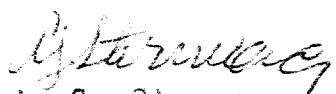
cont'd. /2 ...

June 17, 1963

It can be finally concluded that due to the uniformity of the subsoil conditions, the choice of the crossing location should be dependent on other factors rather than foundation requirements.

The geophysical report will be filed in our office. Should there be any need for more information, we will be pleased to supply any part of it, or submit the report in its entirety.

AGS/MdeF


A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. M. Toye
H. A. Tregaskes
H. D. McMillan
H. McArthur
R. S. Chapman
E. R. Saint
A. Watt

Foundations Office
Gen. Files

Mr. C. S. Greboki,
Bridge Design Engineer,
Bridges Division,
Admin. Bldg.

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

January 20, 1967

31M - 25
GEOCRE No.

Blanche River Bridge,
Geo. Hwy. No. 624,
Twp. of Ewanturel,
N.P. 113-62,
District 14 (New Liskeard)

62-F-120

The investigation for the above structure was carried out during the latter part of 1962. A pile loading test was carried out one year later. During the investigation, an artesian water condition was encountered. Apparently, no attempt was made after the completion of the investigation, to seal the water-producing boreholes.

In the latter part of 1966, a complaint was received by the District, that a well in the neighbourhood had dried out because of the loss of water through one of the boreholes.

A very serious attempt was made to seal the water-producing hole, but with no success. The flow of water was stopped at the borehole location, but water appeared at another location - this time, some 30 feet from the toe of the approach embankment.

There is no doubt that this represents a rather serious condition. Whether the condition should be considered critical is yet impossible to say. The adverse weather conditions make the assessment of the problem at this stage, impractical. It is, however, believed that measures to intercept this water and thus control it, can be conceived and implemented. This operation would, in all probability, eliminate the present problem. Before this problem is resolved and taken care of, no construction can be started.

It has been agreed upon with the District representatives that the problem will be dealt with sometime in the spring of 1967 when the ground and weather conditions will permit such an operation.

cont'd. /2 ...

January 20, 1957

As far as the bridge construction itself is concerned, we would recommend that the piers be completed before any construction of the abutments is started. This sequence is recommended because the pier construction (i.e., pier pile driving and footing construction) represents a more delicate operation and it is reasoned that if some damage does occur, it should be attempted to keep it at a minimum. We would also recommend that in the "Special Provisions" a paragraph be inserted which would forewarn the contractor that a sequence of pile driving may be requested by the Engineer at the time of construction, which he would have to follow.

It would also be essential to specify that no material can be placed on the slopes at any time, and that the shape of the slope cannot be altered without specific written permission by the Engineer. This would serve as a precaution that the conditions of stability of the banks not be unfavourably changed during construction.

It is also suggested that the contractor submit for approval, his scheme of shoring and protecting the pier foundation excavations. He should also indicate the area where the excavated material is to be placed.

From the previous correspondence regarding the stability of the river banks, you may gather that the margin of safety is not too great, and we would therefore like to point out the need for adhering strictly to the recommendations. The benefit of close supervision cannot be overemphasized in this particular instance.

ACS/adeP

A. G. Sternac

A. G. Sternac

PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Files

Gen. Files

31M - 25

GEOCRE5 No.

BA1579
file

To be included with 31M-25
31M-25

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division,
Admin. Bldg.

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

DATE: November 20, 1968

31M-25

GEOCRE No.

OUR FILE REF.

IN REPLY TO

SUBJECT:

Blanche River Bridge at Englehart
Hwy. #624, District #14 (New Liskeard)
W.P. 113-62, W.J. 62-F-120 & 67-F-65

PAGE 17

The above mentioned site was visited by the writer on November 19, 1968. We would like to draw your attention to the following points relating to the design:

(1) Rip-rap is provided for protection of the river banks and river bed for a width of about 20 ft. each side of bridge centre-line. We are of the opinion that this should be extended to at least 60 ft. each side of bridge centre-line, since it is essential that no part of the river bank or bed be eroded, which can affect the stability of the approach embankments.

(2) Drainage from the north side of the west approach embankment can, at present, discharge into an area behind the west pier. This situation can be rectified easily by constructing a shallow ditch to route the drainage water to discharge into the river at a point to the north of the structure and protected slopes. The Project Supervisor, Mr. H. Shepphard, has advised us that the District are aware of this situation and intend to carry out the necessary measures.

In both of the above cases, we believe that the recommended changes be effected prior to the spring run-off. Rip-rap is at present still being placed, and it would be a simple matter to extend these operations.

KGS/MdeP

cc: Messrs. B. R. Davis
J. McAllister
J. D. Harris
H. A. Tregaskes
D.A.O. White
H. McArthur
E. R. Saint

Foundations Files
Gen. Files

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

D.H.O.
TORONTO
RECEIVED
NOV 21 1968
BRIDGE
OFFICE

31M-25

cc: Bridge Office (3rd copy)

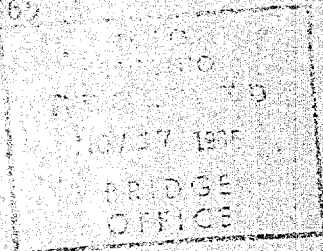
SITE 47-25

CO. of TIMINSKAMING
EVANTUREL TWP.
CON. VI Lot 6Mr. B. H. Davis,
Bridge Engineer,
Bridge Division.Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.Attention: Mr. C. Grobski,
Bridge Design Engr.

November 16, 1965



-- Blanche River Bridge --
3 Miles North of Sac. Hwy. No. 569,
Sec. Hwy. 624, - District No. 14.



In the course of all of our discussions, we have pointed out that the subsoils at the above crossing are of rather poor quality and that the stability of the natural banks is quite precarious. This fact was also mentioned in our original investigation report, and flattening of the slopes was recommended.

In our memo of February 18, 1965, to your Office, we again elaborated on the stability of the slopes and once more mentioned the very low factor of safety obtained from calculations of the so-called "end of construction case".

Because of the fact that, according to our calculations, the construction stage is the most critical one, care should be taken to assure that proper construction practices are applied.

It was agreed that the grading - i.e., the flattening of the slopes and the building of the approach embankments - is to be carried out as a separate contract, one year ahead of the construction of the bridge. This should serve the purpose of increasing the stability of the slopes by the time the bridge is to be built.

The approach fills should be built to their full height. The part where the abutment footings are to be located, should also be built up. At the time of the bridge construction, this part of the fill will have to be excavated and the material hauled away and placed in an area where it cannot influence the stability of the banks. This area is to be approved by the Engineer.

It is also drawn to your attention that we consider the protection of the toes of the slopes as essential. Numerous slides are constantly occurring along the river banks, and we feel that one of the reasons is erosion of the material at the water level. Flattening of the slopes will naturally decrease the susceptibility of the material to erosion, but we would still recommend that some additional measures be considered.

cont'd. /2 ...

31M25

November 16, 1965

The construction of the pier footings will require the excavation of material on the slopes, and since this may change the stability, it is essential that:

(1) The excavations be carried out within sheet piles and that the excavated material be hauled at least 200 ft. away and dumped in an area where it can have no detrimental influence on the stability of the slopes. The choice of this area should be subject to the approval of the Engineer.

(2) The sheeted excavation be adequately braced in order to prevent any possible movement of the soil around the excavation.

(3) The Contractor be not allowed to either excavate or place any material within the area of the grading contract without the specific permission of the Engineer.

(4) All access roads within the area of the grading contract be approved by the Engineer, and that no deviations from such approved plans be tolerated.

Attached to this memo, please find a drawing showing the final slope - i.e., the final shape of the west bank. We have introduced only a minor change to the cross section, as shown on your drawing No. D-5320-1, general layout. This drawing supersedes our previous drawing (62-F-120) which was attached to our memo of February 18, 1965.

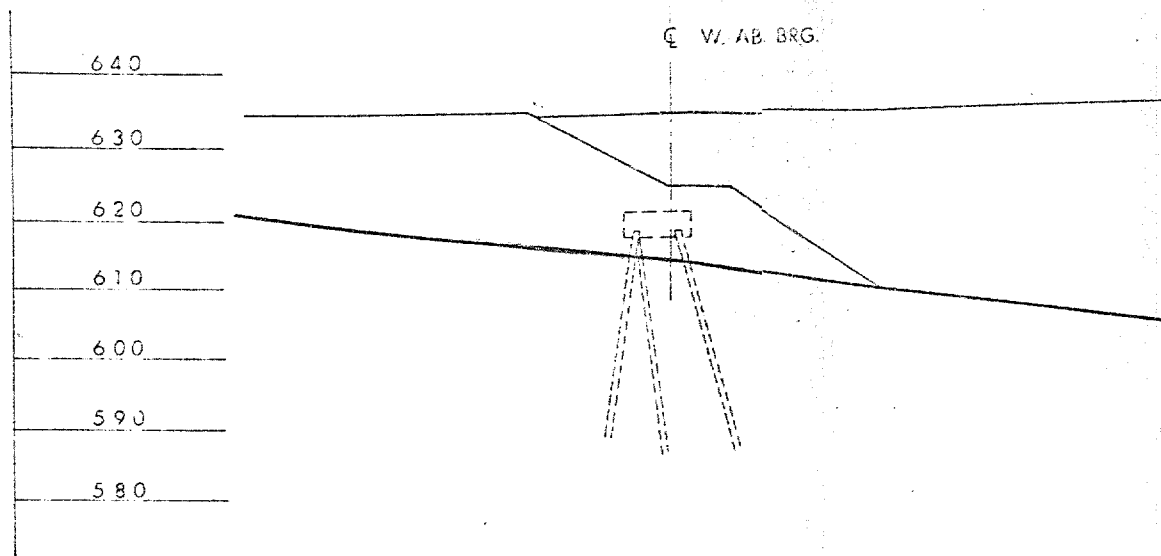
We believe that this memo contains all of the provisions that we presently consider as essential. However, there may still be some that should be included, either on the drawings or in the Special Provisions. If you have any suggestions, we would be very interested to hear about them.

AGS/MdeP
Attach.

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

cc: H. W. Astor
G. M. Sincere



C PROFILE - V
SCALE 1"

NOTE

Soil excavations to be 50 ft. wide at the bottom and to have 4:1 lateral slope.

WEST PIER

BLANCHE RIVER

W.L. 586.11 AUG. 1962

WEST BANK
in. = 20 ft



ONTARIO

DEPARTMENT OF HIGHWAYS

MATERIALS and
TESTING
DIVISION

SEC. HWY. 634

BLANCHE RIVER
WEST BANK - FINAL

DATE 16 NOV. 1965

APPROVED

DRAWING NO.

BLANCHE RIVER

W.L. 586-11 AUG. 1962



ONTARIO

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

SEC. HWY. 634

BLANCHE RIVER
WEST BANK - FINAL GRADE

DATE 16 NOV. 1965

APPROVED

DRAWING NO. 62-F-120 C

62-F-120

W.P. # 113-62

REALIGNED

SEC. HWY. #624

§ BLANCHE R.

