Public Works
Canada

Travaux Publics
Canada

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Western Region

Region de l'Ouest

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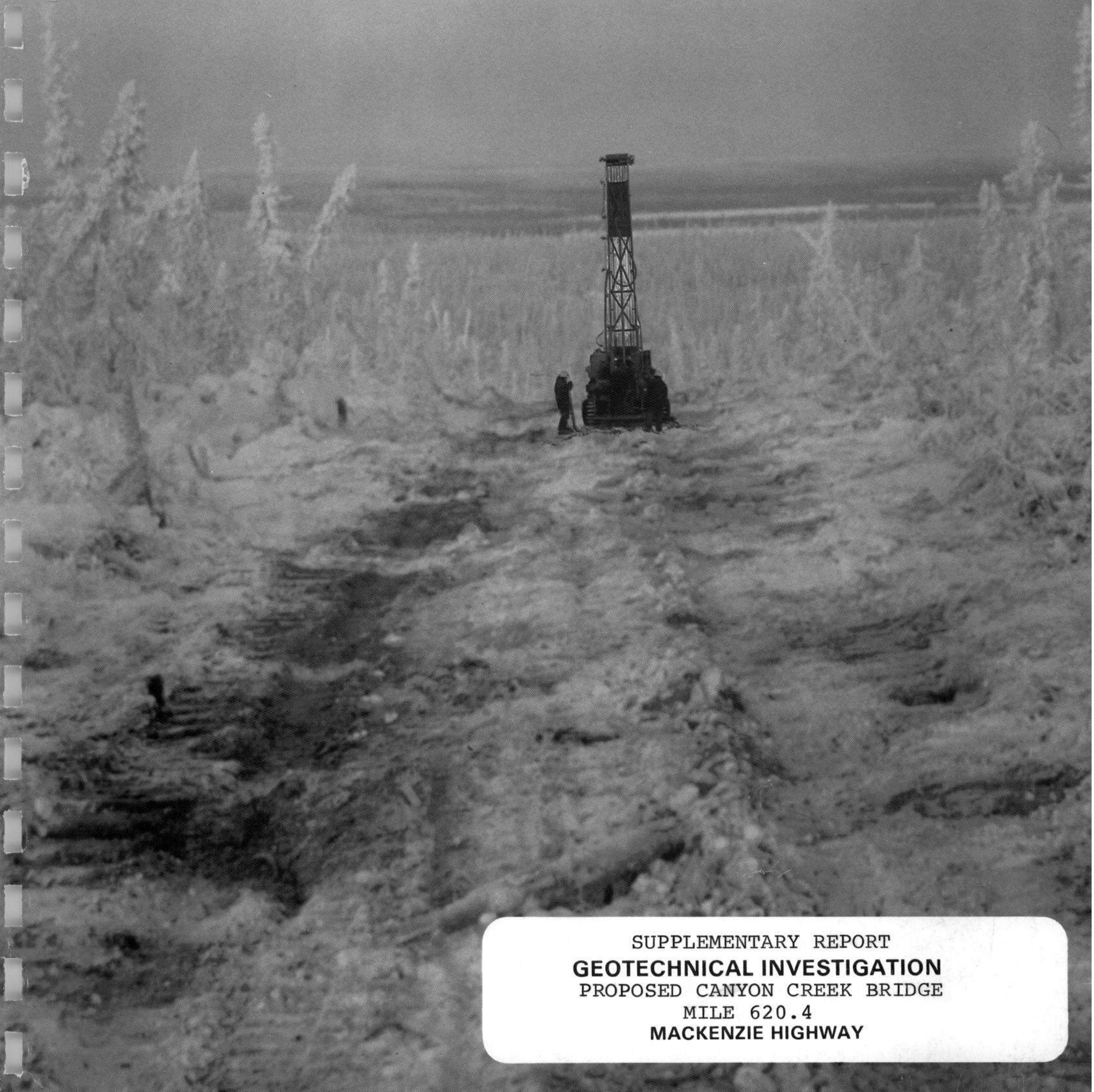

SUPPLEMENTARY REPORT

GEOTECHNICAL INVESTIGATION

PROPOSED CANYON CREEK BRIDGE
MILE 620.4
MACKENZIE HIGHWAY

PUBLIC WORKS CANADA

WESTERN REGION

SUPPLEMENTARY REPORT

FOUNDATION INVESTIGATION

PROPOSED CANYON CREEK BRIDGE

MILE 620.4, MACKENZIE HIGHWAY

Submitted By: R. D. Cook, P. Eng.
Soils Engineer
Special Services
Western Region

May 31, 1976

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APPENDICES

I PRELIMINARY

The initial foundation investigation at Canyon Creek was undertaken by R. M. Hardy & Associates during the winter of 1973, and summarized in a foundation report dated October, 1973. The text of that report has been included herein, and this report should be considered as an addendum to the original.

Three test holes were drilled by Public Works Canada at the proposed crossing site in March, 1975, to augment the initial subsoil data. A profile of the site showing boreholes from both drilling programmes and the inferred subsoil stratigraphy has been included on Drawing No. A-1 in Appendix A, and borehole logs are included in Appendix B. The proposed gradeline, and the limits of the proposed bridge, as recommended by the bridge consultants (Associated Engineering Services Ltd.) have also been shown on the profile. Piers and abutment locations for a 3-span structure are illustrated.

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II SUBSOIL CONDITIONS AND FOUNDATION RECOMMENDATIONS - R. M. HARDY & ASSOCIATES

The following 11 pages are taken from the foundation report by R. M. Hardy and summarizes subsoil conditions as inferred from boreholes by their crews, and the foundation recommendations for the bridge structure based upon that information.

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INTRODUCTION

At the request of Mr. F. E. Kimball, P.Eng., Manager of Northern Roads Program, Department of Public Works of Canada, Western Region, R. M. Hardy & Associates Ltd. undertook a geotechnical investigation along part of the proposed location of the Mackenzie Highway. This report deals only with that part of the investigation appertaining to the proposed bridge at Canyon Creek.

The location of this bridge site is shown on mocaic sheet No. 50 of the set of mosaics prepared by the Department of Public Works for the Mackenzie Highway work. The site is covered by aerial photographs No. A22773-159 and 160 (scale 1" = 1000'). In addition to the mosaics and aerial photographs, R. M. Hardy & Associates Ltd. was provided with a sketch plan profile showing the crossing. This drawing is entitled "Plan and Profile Showing Drainage Structure at Canyon Creek" and is not dated. It was used as the basis for Plate 1, Appendix A.

A report entitled "Geotechnical Investigations, Mackenzie Highway, Mile 544 to 635" has been previously submitted to the Department. The geotechnical conditions are discussed in Volume I while Volume II contains information on permafrost of a more general nature.

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We recommend that these volumes be read in conjunction with this report.

TOPOGRAPHY

The general direction of the drainage in the area is southwesterly towards the Mackenzie River. The valley walls of Canyon Creek are very low on the southerly approach but are quite steep on the northerly side. On the southerly approach, the existing ground profile climbs 10 feet vertically in a horizontal distance of 700 feet while on the northerly approach the profile climbs 50 feet in a horizontal distance of 400 feet. The width of the creek at the water line is about 50 feet.

SOIL PROFILE

The soil profile on the southerly approach consists of glacial lake basin deposits overlying basal till. On the north approach the soils consist of basal till covered with a thin covering of slopewash. The valley walls have been classified as eroded slopes while the floor of the valley has been classified as an alluvial meander plain. Test Holes 936 and 937 were drilled on the southerly approach and Test Holes 938 and 939 were drilled on the northerly approach. The logs of the holes are in Appendix A.

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The soils in Test Holes 936 and 937 consist of gravel, sand and silt overlying the basal till. The till was encountered at depths of 12 and 10 feet respectively in these test holes. Water and ice contents are high in the top four feet of the soil profile but are fairly low, below 15 percent, below that depth. In Test Hole 937, water seepage was encountered at the 11 foot depth.

The ground was frozen for the entire depth of the test hole at Test Hole 936. In Test Hole 937 the depth of frozen ground was 8 feet while the ground below this depth was unfrozen.

The soil profile in Test Hole 938 consists of gravel and silt to a depth of 4 feet overlying the basal till. The till consists of silty clay to a depth of 28 feet below which there are alternating layers or gravel, sand and silt. Water contents in the soil at this test hole were all below 30% with some samples showing water contents of less than 10%. Ice contents were also low with only small amounts of visible ice be reported.

The soil profile in Test Hole 939 consisted of silt to a depth of two feet overlying basal till which consists of a silty clay. Water contents generally ranged between 20 and 30 percent except at the four

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foot depth where the value is 50%. A thin unfrozen layer of soil was encountered in this test hole at the 11 foot depth.

DISCUSSION AND RECOMMENDATIONS

The effect of a stream on a permafrost profile is shown on Plate 2, Appendix A. This chart shows that the thaw bulb beneath a small creek can penetrate to considerable depths so that, for bridge purposes, the presence of permafrost beneath the stream bed can be ignored. However, it should be noted that permafrost profile beneath the sides of the stream bed plunges at an extremely steep angle.

As is well known the flow of water in northern streams varies tremendously throughout the year. Very large flows can be experienced during the spring runoff so that some scour should be expected. The amount will depend on the flow of water, the constriction imposed on the stream by the bridge, and the width of piers (if any). Some erosion of the banks is also possible.

As stated above, the existing gradient on the northerly approach is very steep. We do not recommend using a cut section on this approach to reduce the gradient. We believe the side slopes in such a cut would be unstable due to the high ice contents in the silty clay. If a cut is not used the elevation of

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the bridge deck will have to be raised in order to bring the gradient on the north approach within acceptable limits.

Because of the soil and permafrost conditions in the valley walls in the approach areas to this bridge site, we do not believe it would be advisable to use concrete abutments. Also, we do not recommend piers founded on concrete spread footings in the stream bed. We therefore recommend that the abutments and any piers be supported on driven steel H-piles. It is extremely unlikely that timber piles could be driven at this site without damaging them severely. Precast concrete piles should not be used due to difficulties of transportation and also because the length of the precast piles would have to be determined in advance. Steel pipe piles are not recommended because it is doubtful that they would be able to withstand the driving stresses.

Steel H piles which are to be placed on the banks where they will not be affected by scour should be driven a minimum of 30 feet below existing grade and designed on the basis of an allowable skin friction of 800 psf (on the gross perimeter) with the top 10 feet of pile being assumed to carry no load.

Steel H piles driven in the stream bed should be driven a minimum distance of 20 feet below the bottom

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of anticipated scour and should be designed on the basis of the "Table of Penetration Resistance" following. Design parameters are summarized on Plate 3, Appendix A. Where bedrock is encountered, the piles should be driven 10 feet into bedrock.

In driving steel H piles the weight of the pile driving hammer should be at least twice the weight of the pile being driven except where a diesel hammer is used when the weight of the hammer should be at least equal to the weight of the pile. To prevent damage to the points of the pile they should be reinforced with flange plates for a distance equal to 1.5 times the size of the pile. Alternatively, the point can be reinforced with a driving shoe. Piles should be driven to practical refusal or refusal according to the following table of penetration resistances assuming that the hammer delivers an energy of 15,000 ft. pounds per blow.

TABLE OF PENETRATION RESISTANCE

Description	Inches Per Blow
refusal	.00-.05
practical refusal	.05-.25
high resistance	.25-.50
medium resistance	.50-1.25

In order to ensure that refusal has been reached, driving

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should be continued for at least 100 blows after refusal is first recorded.

Piles driven to refusal in the stream bed, as defined above, may be designed for the full structural strength of the pile section acting as a column. The design load will depend upon allowable stresses in the pile, column length and the arrangement of natural bracing. Piles driven to practical refusal, as defined above, should be designed for two-thirds of the value permitted for the pile as a structural column. Consideration should be given to using battered piles on the outside of the pile bents in order to provide lateral resistance.

If a drop hammer is used in driving the piles, care should be taken that the energy delivered to the pile is not greater than 15,000 ft. pounds per blow unless calculations show that the pile can safely take higher impact stresses. As mentioned above, bedrock may be encountered before the design depth is reached. In such a case, the piles should be driven into the bedrock a distance of 10 feet.

One of the problems facing bridges in this area is the possiblity of logs jams occurring which can cause partial or complete failure of the bridge. Log jams are only likely to occur where trees travelling down the river have a greater length than the clear

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span of the bridge. We suggest that the height of trees growing adjacent to Canyon Creek upstream of the bridge should be checked and, should it be observed that there is a possibility of large trees being washed downstream, such facts should be borne in mind by the bridge designer. If piles are used to support vertical faces of embankment fill, the lateral force against the pile can be computed by assuming the backfill to be a fluid with a density of 60 pounds per cubic foot where the backfill is not compacted.

Embankments constructed below the highest expected flood level should be protected with riprap. As suitable rock may not be available, sandbags filled with concrete may have to be used.

Respectfully submitted,
R. M. HARDY & ASSOCIATES LTD.,

G. McCORMICK

Per:

G. McCormick, P. Eng.

GM/jc

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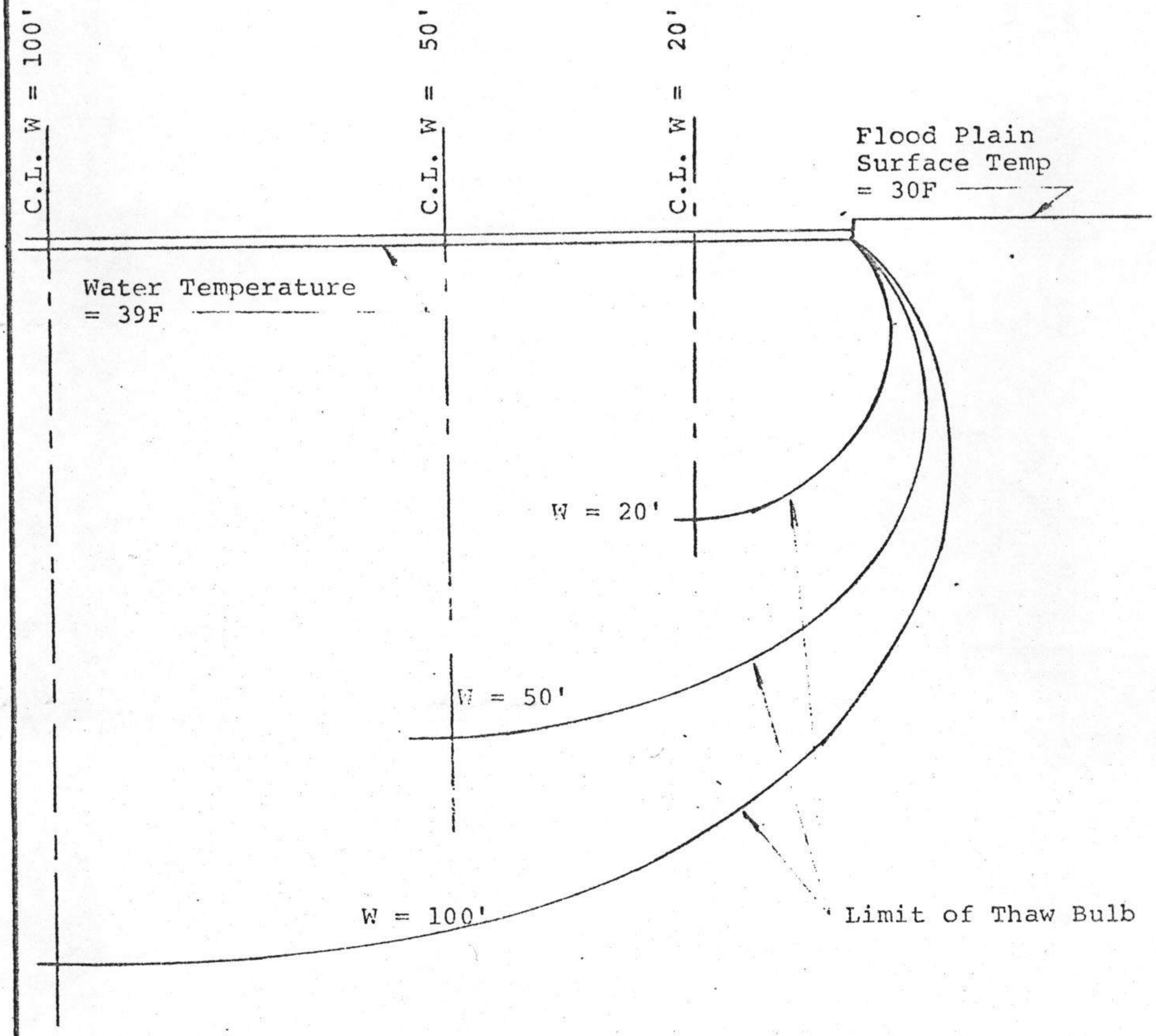
APPENDIX A

Charts
Test Hole Logs
Sections



Scale: 1" = 10'

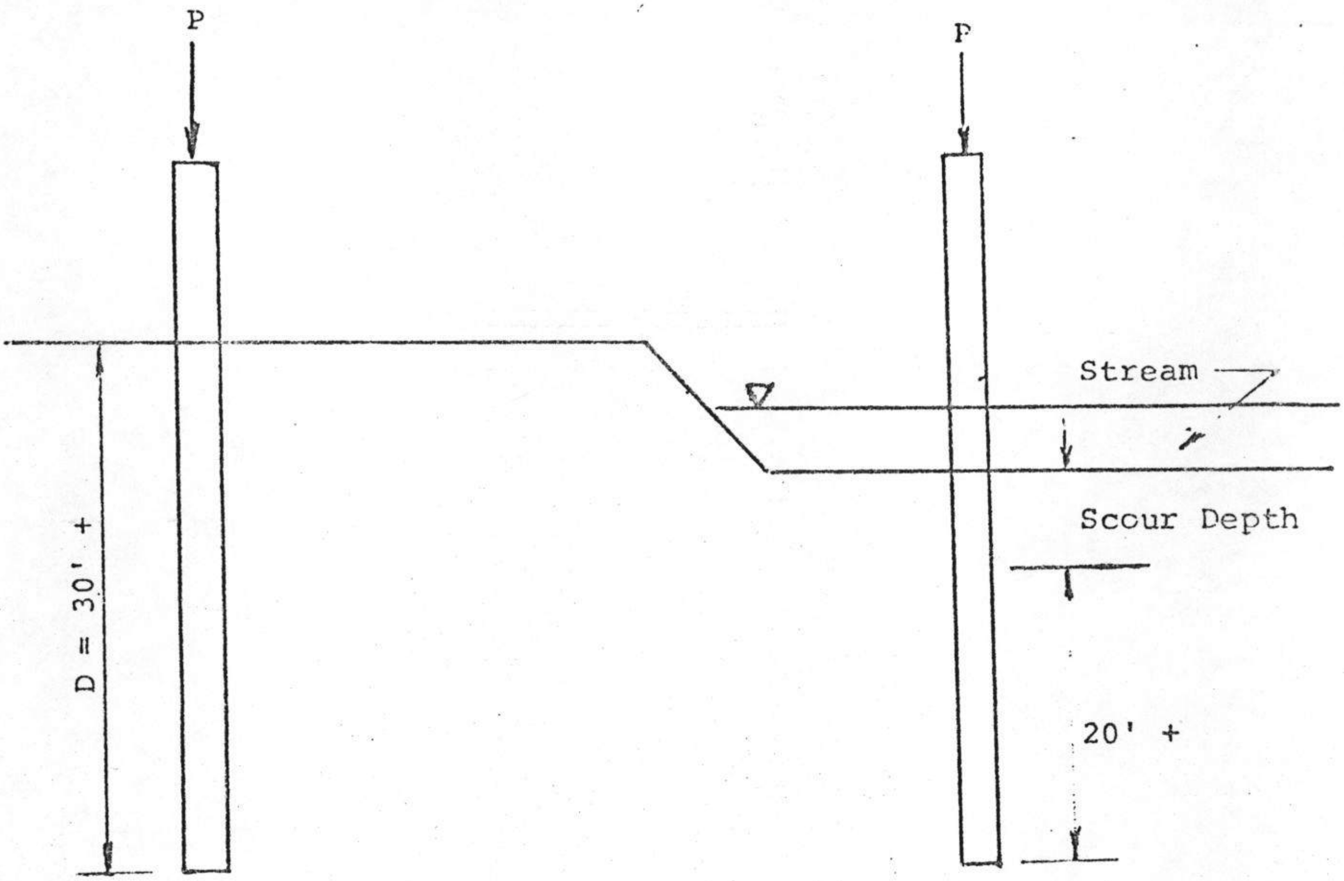
W = River Width
C.L. = Center Line

G.Mc September 14/73 E-2510

R.M.HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERING & TESTING

THAW BULBS BENEATH RIVERS
NORMAN WELLS AREA

PLATE 2



$$\text{Gross Perimeter} = \frac{4H}{12} = \frac{H}{3} \text{ ft.}$$

Piles on dry land to be designed on the basis of an allowable shaft friction over effective length of embedment of D-10 with D minimum = 30 ft.

Piles in stream bed to be driven to 20+ feet below scour depth and designed on the basis of penetration values (see text).

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERING & TESTING

MACKENZIE HIGHWAY
BRIDGE PILES
NORMAN WELLS AREA

SCALE
DATE
MADE G.Mc
CHKD.
JOB: E2510
PLATE

III EVALUATION OF ADDITIONAL BOREHOLE DATA

With reference to Drawing No. A-1 in Appendix A, the 1975 boreholes have generally confirmed and further defined the subsoil stratigraphy as inferred from the 1973 drilling.

Canyon Creek is presently adjacent to a relatively high steep valley wall on the North, with a broad, flat flood plain to the South. Subsoil deposits below the channel, and below the flood plain to the South, consist of roughly 20 feet of fluvial silts, sands and gravels, overlying glacial till. Immediately adjacent to the stream channel on the North, the fluvial deposits are largely absent and the glacial till is within 5-6' of the ground surface. Thus the 3-span structure shown on the profile would have the North abutment and, possibly the North pier, entirely on glacial till, with the South pier and abutment on flood plain deposits over glacial till.

Permafrost is present at this site although the exact limits are unknown. The thaw zone is not as simplistic as suggested by Plate 2 of the R. M. Hardy report. Three probe holes on the flood plain immediately South of the stream channel encountered water in the sands and gravels and it is considered that, as a minimum, the flood plain deposits below the bridge structure will be unfrozen. The thaw zone may extend into the till below the stream-bed, however if not, it is considered that permafrost below the channel will be relatively close to $0^{o}C$.

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On the North, permafrost was noted to a depth of at least 28' in hole #620-S-3 - below the 28' level the till was at a very low moisture content and permafrost was difficult to detect from the drill cuttings. An estimated thaw zone has been shown on Drawing No. A-1 and, although the actual depths of thaw are unknown, it is considered the shape of the thaw zone would approximate the actual site conditions.

With reference again to the 3-span structure shown on Drawing No. A-1, it is considered that the North abutment would be totally underlain by permafrost soils to a significant depth (i.e. well below piling depth); the piers would be underlain by roughly 20 feet of unfrozen sands and gravels, and probably by several feet of unfrozen till; and the South abutment would be underlain by 15-20 feet of unfrozen sands and gravels with permanently frozen till below.

IV FOUNDATION RECOMMENDATIONS

There is no field data available to indicate the relative density of the flood plain deposits or the glacial till, however the till is considered to be an excellent bearing stratum - dense, with moisture contents below the plastic limit even in the permafrost zone. Piling for the proposed structure will

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extend well into the glacial till, and the majority of load transfer will occur here. It is expected that all piling will terminate in permafrost, however, with the exception of piles at the north abutment, there will be load transfer in both frozen and unfrozen material.

It is not expected that bridge construction will produce any rapid changes in the permafrost conditions at Canyon Creek. However as the till is at a very low moisture (ice) content, and is probably close to $0^{\circ}C$, it will have a relatively low volumetric heat capacity, and the introduction of only small amounts of heat will cause degradation of the permafrost. Steel piling may introduce sufficient heat to cause slow retrogression of the permafrost, hence pile design should not be based upon tangential adfreezing, but rather upon adhesion following thaw. Since the moisture (ice) content of the till is very low, post-construction thawing around piling driven into frozen till is not expected to produce average bearing parameters that are significantly less than the bearing parameters for similar piling driven into unfrozen till, especially when thawing is relatively long-term over the length of the pile. Thus it is considered that an average shaft friction value can be applied for design of all piles, despite the variable permafrost conditions, and will permit a rational design.

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The major foundation problem at Canyon Creek is selection of a pile type and installation method which can effectively gain design penetrations through the granular flood plain deposits and into the frozen glacial till. There is a lack of construction information regarding pile driving into frozen soils, but any experiences reported with conventional driving equipment (i.e. drop hammers, diesel hammers) indicate extreme difficulty, pile damage and generally negative results. Although largely untested in permafrost, vibratory hammers would probably not be effective in the clay till. The Bodine driving equipment based upon resonance principle has apparently shown some success in permafrost soils, and by the time of highway construction in the Canyon Creek area may have proven effective for pile driving into permafrost. However for the present, the use of an impact drive hammer will be assumed.

R. M. Hardy & Associates have recommended steel H-piling for the foundation, designed on the basis of an allowable shaft friction of 800 psf., with penetration to design depths in permafrost till assumed. Both the design bearing parameter and the pile penetrations in permafrost are considered optimistic. Open-ended steel pipe piles are considered the most practical for this site as a drill can be inserted through the pile to advance a lead hole should driving progress be impeded by boulders in the

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upper flood plain deposits, or by the permafrost in the glacial till.

A heavy section - at least 10" diameter @ 40 lbs./ft., should be used to withstand hard driving which may be required to penetrate the granular stratum, and to gain adequate penetration into the till when permafrost is encountered. The hammer used should have a ram at least as heavy as the piling in order to impart relatively high energy - low velocity blows to the piling - an energy of roughly 20,000 ft. lbs. per blow is recommended.

The adhesion value of 800 psf. recommended by R. M. Hardy is considered too high for an average value over the length of a straight-sided pile in the dense till, as the effects of driving may create an annulus around the pile and the stiff till will not 'flow back' around the pile. An average value of 350 psf. is recommended for design in both frozen and unfrozen material.

Should it become practically impossible to conventionally drive the piles to the design penetration, then the piles may be advanced by a combination of pre-drilling a lead hole 1/2 to 1" in diameter less than the pile and driving, and/or drilling ahead of the pile through the central core. Pile installation should be under close inspection and driving trials may be required to ensure that pre-drilling does not exceed acceptable

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limits and pile bearing is affected. It may well be that, by the time of construction at Canyon Creek, ample experience will have been gained at other sites in installing steel piles in permafrost soils to more precisely define the method of installation. As an initial requirement it is recommended the lead hole be limited to 75% of the design pile penetration.

Detailed driving records should be obtained for all piles with particular attention to pile lengths, penetrations, marked changes in blow counts, extent of drilling required, and the number of blows for the last 10-15 feet.

The surficial soil deposits on the north valley slope immediately adjacent to the north abutment are potentially thaw unstable and should be disturbed as little as possible during construction. Up-slope surface drainage should be directed away from the bridge with off-take ditches and not channeled parallel to the roadway toward the bridge. Vertical abutments to retain the embankments would be unsuitable at this site and have not been recommended by the bridge consultants, hence the north slope will be blanketed with material and permafrost degradation should not occur here following construction. Approach fills adjacent to the structure will be in the order of 15 to 25 feet in height and very little,

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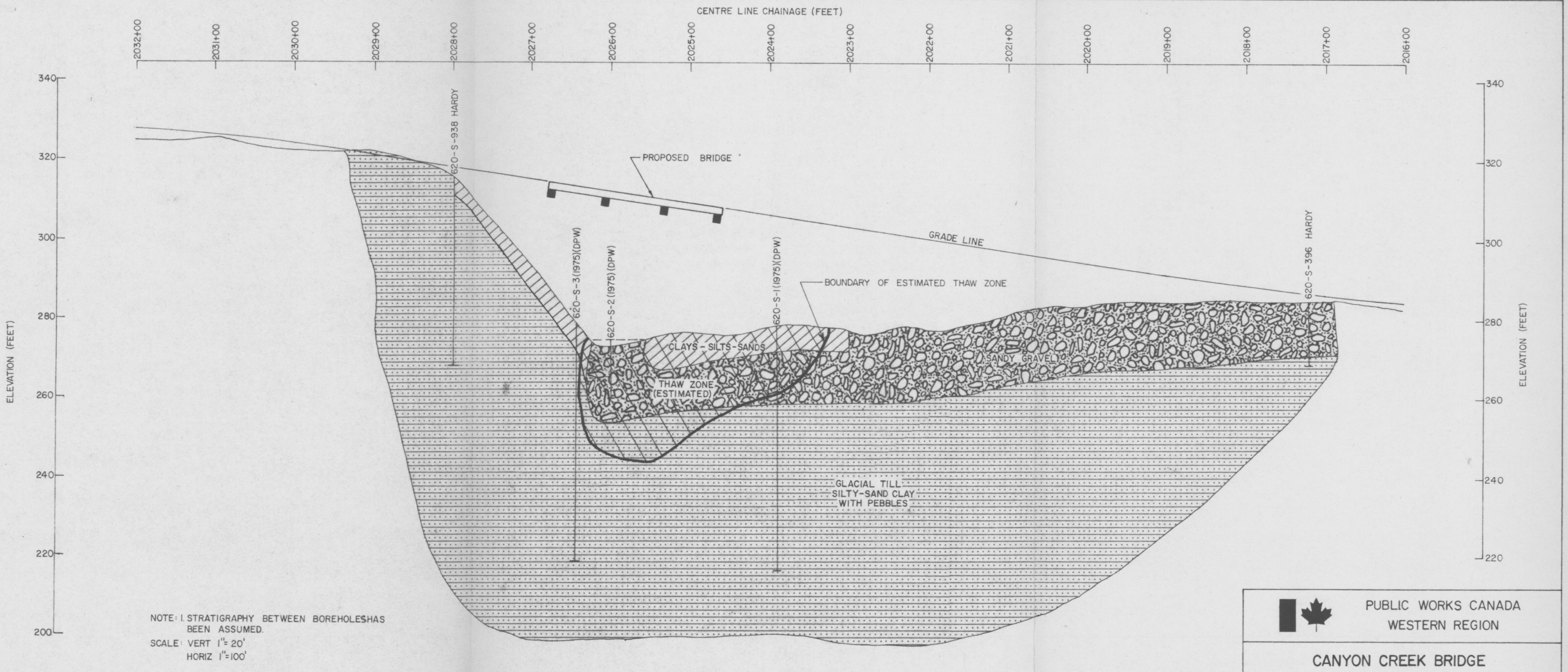
if any, post-construction embankment settlement should occur for fills placed on the north approach during the summer when the depth of thaw is near the maximum. Some minor settlement may occur in the flood plain deposits on the south, however the majority should occur during and shortly after fill placement.

Allowable lateral loads on piling may be assumed at 4 to 5 kips per pile for 1/2 inch movement, however batter piles are recommended to withstand the majority of lateral thrust on the piers or abutments. Abutment piling should be installed after fill placement and fill settlement should not cause any detrimental effects (i.e. negative friction, batter piles).

RDCook
R. D. Cook, P. Eng.
Soils Engineer
Special Services
Western Region



NOTE: 1. STRATIGRAPHY BETWEEN BOREHOLESHAS BEEN ASSUMED.

SCALE: VERT 1" = 20'
HORIZ 1" = 100'

PUBLIC WORKS CANADA
WESTERN REGION

CANYON CREEK BRIDGE
MILE 620·4
MACKENZIE HIGHWAY

DWG NO A-1

DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY	DRILL HOLE REPORT	SITE: CANYON CREEK - SOUTH BANK

FIELD ENG.

DATE DRILLED. 29/3/75

AIRPHOTO NO.

CHAINAGE.

OFFSET.

TECH. REYNOLDS

RIG. AIR

SURFACE DRAINAGE.

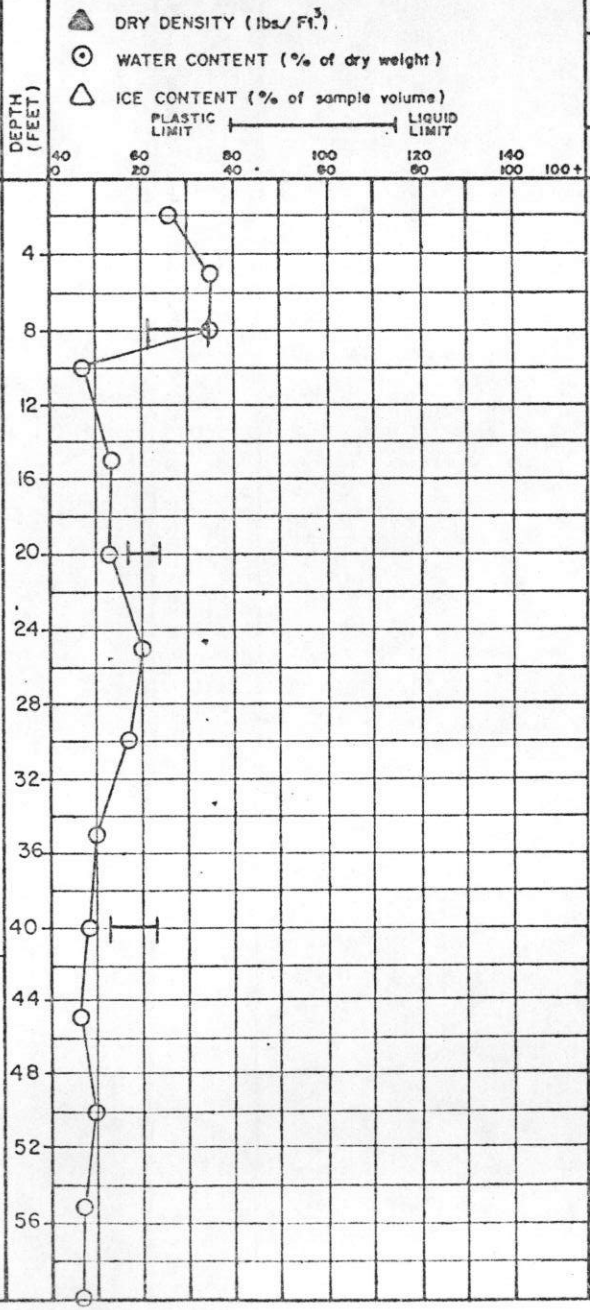
VEGETATION.

ELEV.

TEST HOLE #1 (1975)

MILE 620

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–3'					SM	Sand - Silty	F	Vc-Vx
3'–9'					Cl	Clay - Silty Sandy Medium Plastic		
9'–					GP	Gravel - Sandy	UF	
					CL	Clay - Silty Sandy Pebbles Low Plastic	F	Nbn
					CL	Clay - Silty Sandy Pebbles Low Plastic		
							F?	



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT
-	40	-59	1	Wet
-	71	-29	0	Moist
-	76	-24	0	Wet
-	8	-20	72	Sat.
-	14	-43	43	Sat.
-	71	-28	1	Damp
-	92	- 7	1	Wet
-	85	-13	2	Moist
-	58	-36	6	Damp
-	52	-39	9	Damp
-	37	-48	15	Damp
-	73	-27	0	Damp
-	74	-25	1	Damp
-	75	-23	2	Damp

REMARKS

BOTTOM OF HOLE - 60'

D3·14·5·74 a

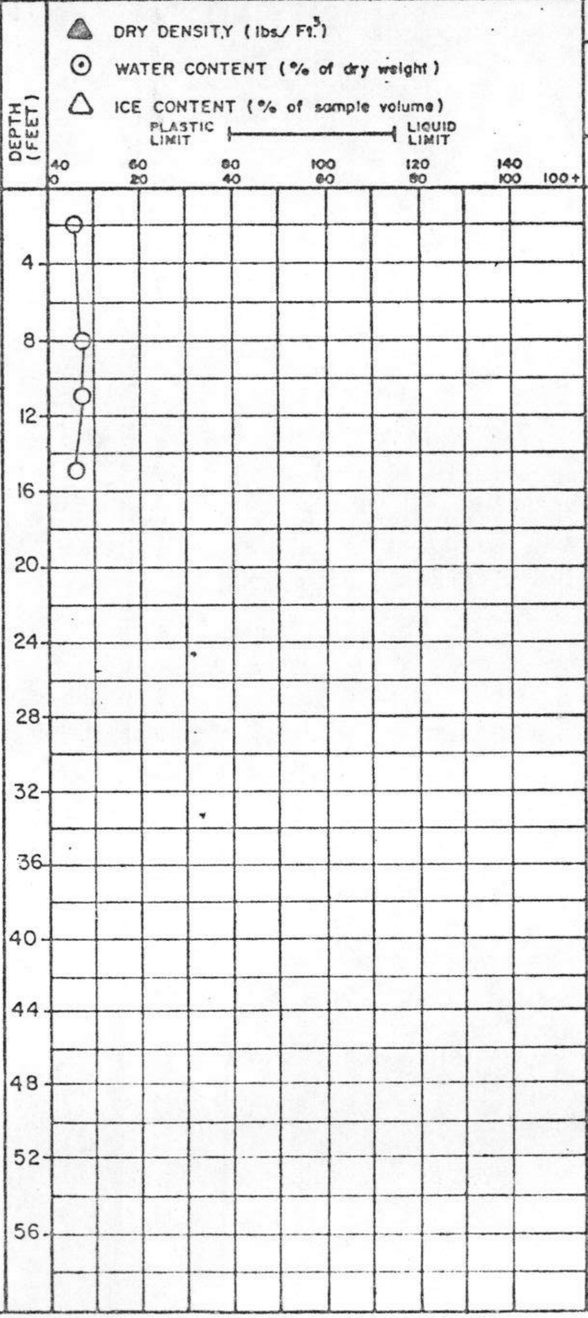
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: CANYON CREEK - MID-STREAM

FIELD ENG.
DATE DRILLED. 29/3/75
AIRPHOTO NO.
CHAINAGE.
OFFSET.
TECH. REYNOLDS
RIG. AIR
SURFACE DRAINAGE.
VEGETATION.
ELEV.
TEST HOLE #2 (1975)
MILE 620
REMARKS

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
						Ice		Ice
4					GW	Gravel - Sandy	F	V_x
8								
12						15'		
16						Bottom of Hole - 15'		
20								
24								
28								
32								
36								
40								
44								
48								
52								
56								



CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT
-	3 -	44	53	Wet
-	1 -	49	50	Wet
-	2 -	36	62	Wet
-	28 -	27	45	Wet

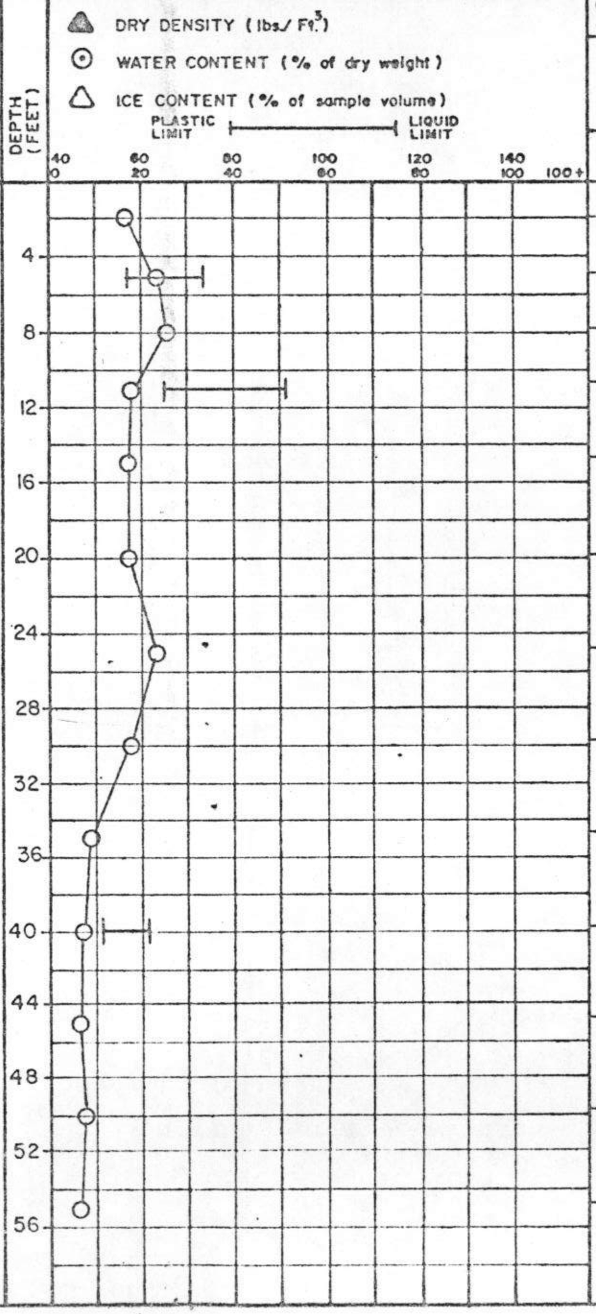
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: CANYON CREEK - NORTH BANK

FIELD ENG.	DATE DRILLED. 29/3/75	AIRPHOTO NO.	CHAINAGE.
OFFSET.			
TECH. REYNOLDS	RIG. AIR	SURFACE DRAINAGE.	VEGETATION. SPRUCE & BIRCH
ELEV.			
TEST HOLE	#3 (1975)	MILE	620
REMARKS			

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
4					Gc	Clay - Sand - Gravel Mix		
						5'		
8						Clay - Silty Sandy Gravelly		
12					Cl CH	Medium-High Plastic Pebbles	F	Frozen to at Least 28' ·Nbn ·Vx
16								
20					CL	Low Plastic		
24								
28								
32						Clay Silty Sandy		
36						Pebbles Gravelly		
40					CL	Low Plastic		
44								
48								
52								
56								



GRAIN-SIZE ANALYSIS

CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT
24	-	52	24	Sat.
80	-	18	2	Moist
60	-	27	13	Wet
33	-	27	40	Wet
73	-	18	9	Moist
72	-	22	6	Moist
86	-	12	2	Wet
94	-	6	0	Damp
61	-	37	2	Damp
48	-	21	11	Damp
41	-	42	17	Damp
55	-	41	4	Damp
48	-	40	12	Damp

Bottom of Hole - 60'

DS·14·5·74 a

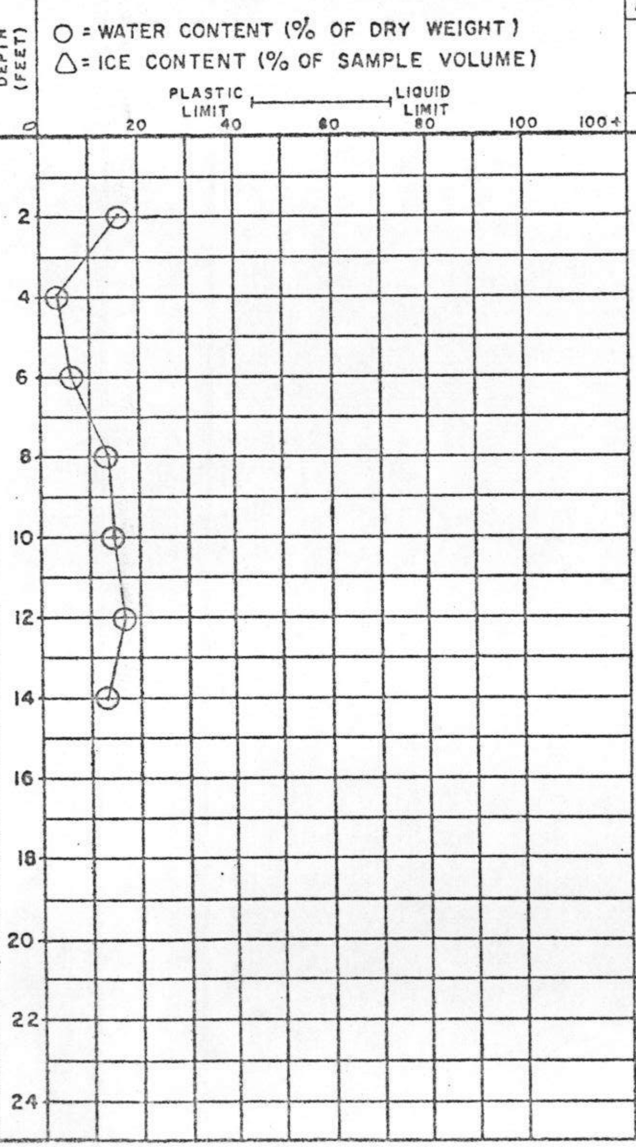
R.M. HARDY AND ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN: CK
FIELD ENG: KD
DATE DRILLED: 12/3/73
AIRPHOTO NO: A22934-143
CHAINAGE: 2017+25
OFFSET:
CKD:
TECH: M.J.
RIG: FAILING
SURFACE DRAINAGE: GOOD
VEGETATION: SEE REMARKS
ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
620	C	936

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
0						(MOSS-ORGANIC SILTY CLAY COVER).									
					GM	GRAVEL; SANDY, SILTY, NON PLASTIC BROWN	F	Vx 10%							MODERATELY DENSE SPRUCE 25-50' HIGH 12"ϕ MAX OCCASIONAL BIRCH
4'						FINER		Nbn							
6'						COARSER									
8'						VERY SILTY, FINE SANDY									
10'					SW-SM	SAND; COARSE, FINE GRAVELLY, VERY FINE GRAIN SAND									
12'					CL	CLAY (TILL) SILTY, SANDY, LOW PLASTIC GREY, PEBBLES									
15'						END OF HOLE									



R.M. HARDY AND ASSOCIATES LTD.

DRILL HOLE REPORT

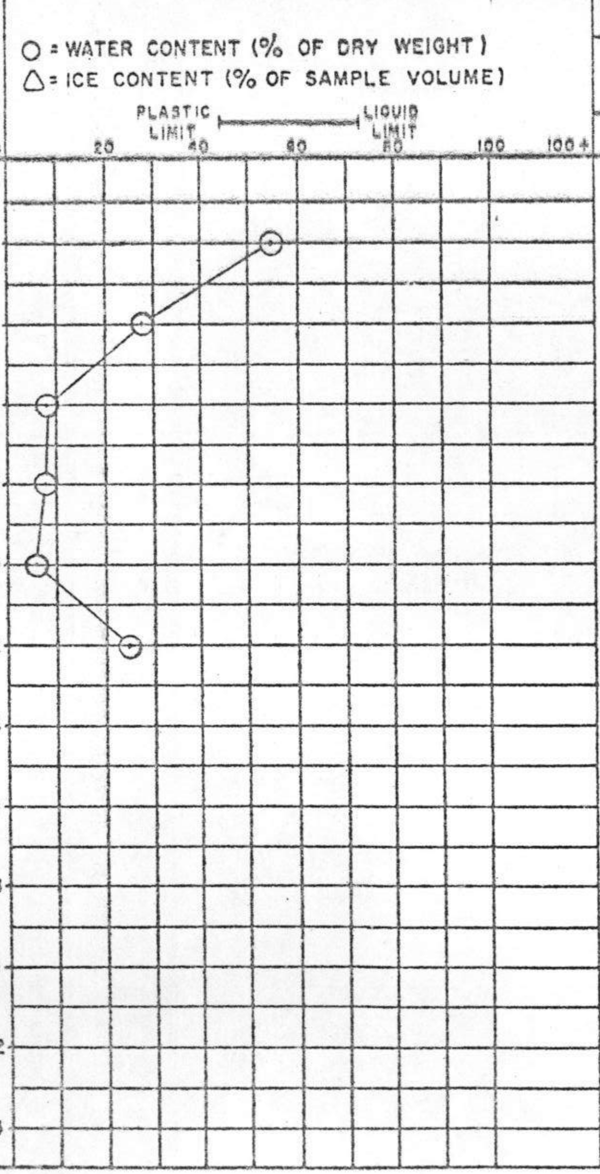
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: CEJ	FIELD ENG: BD	DATE DRILLED: 12/3/73	AIRPHOTO NO: A22934-143	CHAINAGE: 2024+80	OFFSET:	
CKD:	TECH: MD	RIG: FAILING	SURFACE DRAINAGE: GOOD	VEGETATION: SEE REMARKS	ELEV:	

TEST HOLE

MILE	B,C,S	NUMBER
620	S	937

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION (EST. %)	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)
0					ML	SILT; SANDY, CLAYEY, LOW PLASTIC ORGANICS, BROWN	F							
2					ML	2' SILT; TRACE OF SAND, PEBBLES BROWN		Vx 10%						
4					GM	4' GRAVEL; SANDY (COARSE), SILTY BROWN		Nbn						
6						6' LESS SILT								
8						WET-UNFROZEN 8'	UF	NO ICE						
10					SC	10' SAND (TILL); VERY SILTY, TRACE OF PLASTICITY, DK BROWN		WATER SEEPAGE						
12						12 END OF HOLE								



REMARKS

MODERATELY DENSE SPRUCE
25-50' HIGH
12"ϕ MAX

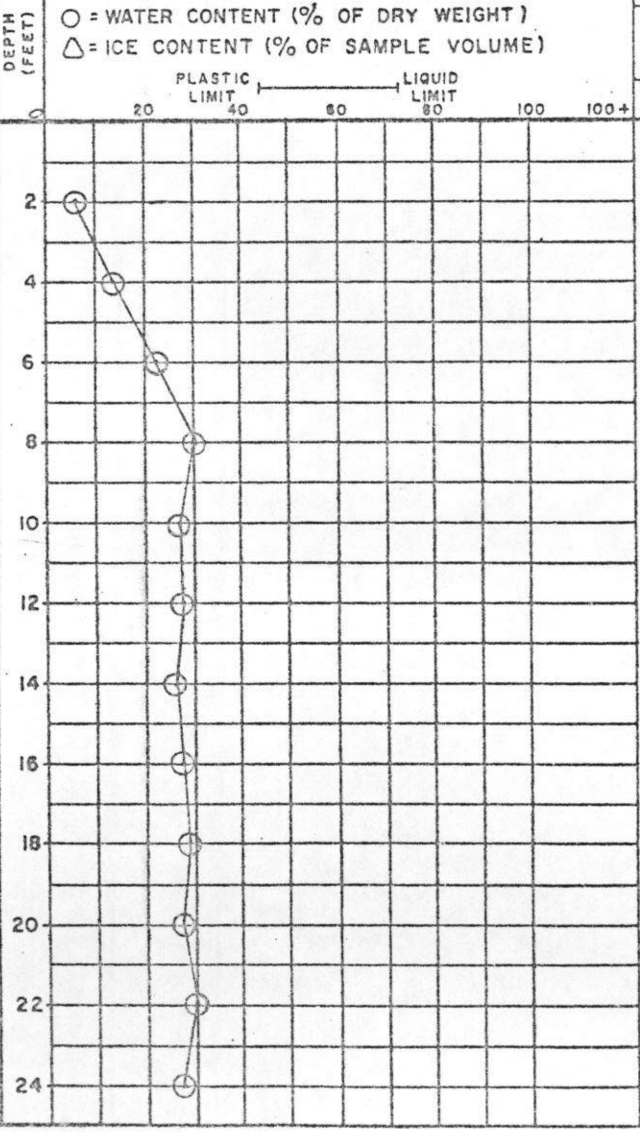
R.M. HARDY AND ASSOCIATES LTD. | DRILL HOLE REPORT | DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: Ch | FIELD ENG: R.D | DATE DRILLED: 12/3/73 | AIRPHOTO NO: A22934-143 | CHAINAGE: 2027 + 90 | OFFSET:
CKD: [illegible] | TECH: DR | RIG: MAYHEW 1000 | SURFACE DRAINAGE: GOOD | VEGETATION: SEE REMARKS | ELEV:

TEST HOLE
SHEET 1 OF 2

MILE	B,C,S	NUMBER
620	S	938

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION (EST. %)
0					GW	GRAVEL (FINE); WELL GRADED SANDY ROOTS	F	Nf
2								
3½'					ML	SILT; SANDY LOW PLASTIC BROWN		Nbn
4'					CI	CLAY (TILL); SILTY, MED PLASTIC CALCAREOUS, PEBBLES BROWN		Vx 5%
6'					CH	HIGH PLASTIC		
8								
10								
12								
14								
16								
18'						DARK GREY		
20								
22								
24								



GRAIN-SIZE ANALYSIS

CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)

REMARKS

MED DENSE
SPRUCE
6-25' HIGH
8" ϕ MAX

R. M. HARDY AND ASSOCIATES LTD.

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

WN:	FIELD ENG BD	DATE DRILLED 12/3/72	AIRPHOTO NO: A22934-143	CHAINAGE:
KD	TECH D.R	RIG MAYHEW 1000	SURFACE DRAINAGE GOOD	OFFSET:
			VEGETATION: SEE REMARKS	ELEV:

TEST HOLE
SHEET 2 OF 2

MILE	B,C,S	NUMBER
	S	938

(FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
26					CH	CLAY (TILL) SILTY, HIGH PLASTIC PEBBLES, GREY CALCAREOUS, STRATIFIED 28'		Vx 5%							MODERATELY DENSE SPRUCE 6-25' HIGH 8"ϕ MAX
28					GM	GRAVEL (TILL); SANDY, FINE, SILTY . GREY STRATIFIED, CLAYEY									
32					SM	SAND; SILTY, ANGULAR AND SUBROUNDED									
35					ML SM	SILT ; SANDY, NON PLASTIC, GREY 38									
38					ML	SILT (TILL); SANDY CLAY (TILL) INCLUSIONS GREY 40'									
40					ML	SILT; FINE SAND, CLAYEY, LOW PLASTIC GREY. 48'		Nbn							
48						END OF HOLE									
50															

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)