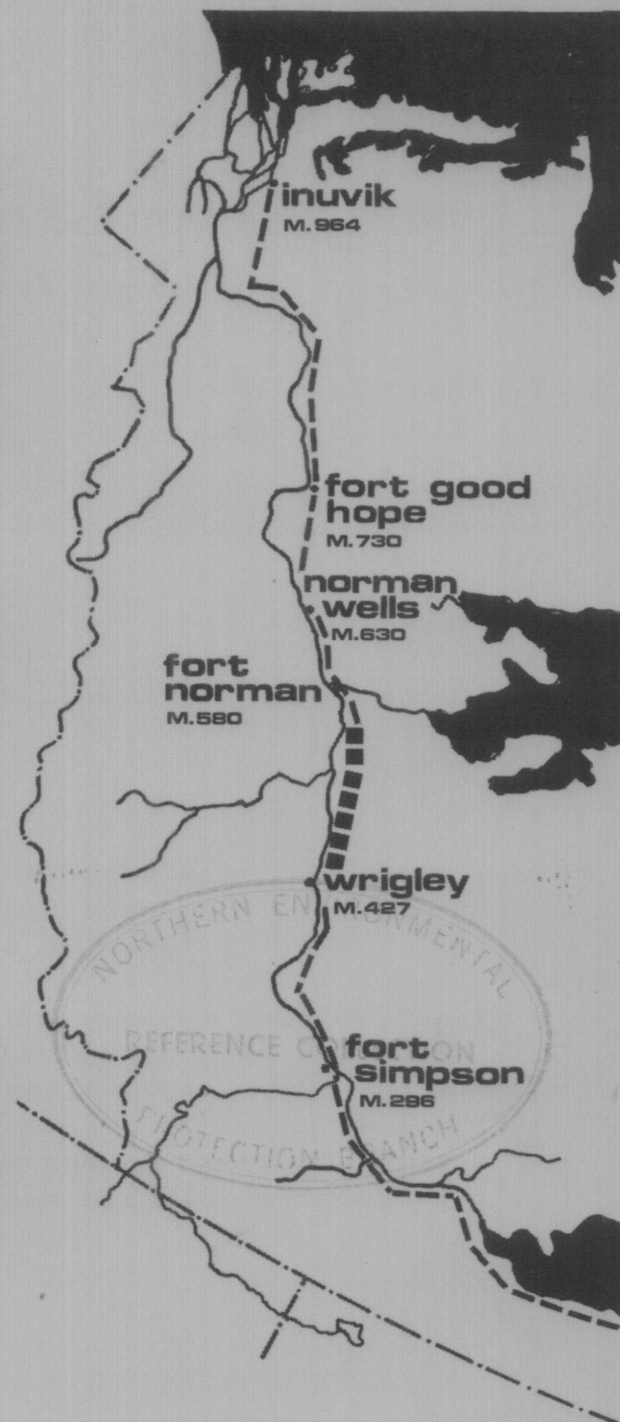

REPORT ON SUBSURFACE SOIL CONDITIONS FOR PROPOSED - STEEP CREEK BRIDGE MILE 511.8

MACKENZIE HIGHWAY

Underwood McLellan & Associates Limited

ISSUE #39

Underwood McLellan & Associates Limited

2540 KENSINGTON ROAD N.W. CALGARY, ALBERTA T2N 3S3 TELEPHONE 283-6061 TELEX 038-25565

2106-032-45
April 16, 1973

Department of Public Works
P.O. Box 488
Edmonton, Alberta

Attention: Mr. F. E. Kimball

Dear Sir:

Re: Subsurface Soil Investigation
Proposed Steep Creek Bridge
Mile 511.8 Mackenzie Highway

Attached please find our report on the subsoil and foundation conditions for the proposed Steep Creek bridge at mile 511.8 on the Mackenzie Highway in the Northwest Territories.

If you have any questions concerning the contents of this report, we would be pleased to discuss them with you at your convenience.

Yours very truly,

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DGPennell

Per: D. G. Pennell, Ph.D., P. Eng.

Enc.
DGP/fm

REPORT ON SUBSURFACE
SOIL CONDITIONS FOR PROPOSED
STEEP CREEK BRIDGE MILE 511.8
MACKENZIE HIGHWAY
NORTHWEST TERRITORIES

Prepared by

UNDERWOOD McLELLAN & ASSOCIATES LIMITED
Consulting Professional Engineers

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A. INTRODUCTION

The subsoil investigation undertaken at Steep Creek forms part of the overall geotechnical investigation which was conducted from mile 450 to 550 of the Mackenzie Highway for the Department of Public Works, Government of Canada.

It was the intention of the present investigation to determine the vertical subsoil material sequence, permafrost conditions and groundwater conditions existing which would be expected to influence the stability of the valley slope and the design and construction of the proposed bridge foundations.

The included conclusions and recommendations have been based upon exploration and sample acquisition operations conducted at 8 positions along the proposed bridge centreline in the river channel and slopes of the bridge approaches.

In the formulation of conclusions and recommendations, the consistency and composition of the subsoil materials between and in the proximity to the individual test boring positions has been assumed, but not verified.

B. FIELD AND LABORATORY INVESTIGATION

A total of eight (8) test holes has been drilled in the Steep Creek valley. In addition to the eight (8) centreline test hole logs, borrow hole test logs in the vicinity of the bridge site have been included in the Appendix. Test holes along the centreline were drilled to depths varying from 15 to 35 feet below existing site grades. The maximum hole depths were dependent upon the caving conditions which were present in the gravel strata. Borrow test holes were generally advanced to the 15 foot level.

Test borings were drilled February 2 and 3, 1973 be Kenting Big Indian Drilling of Calgary utilizing a Mayhew 1000 and Heli drill. Air recovery methods were employed in all test holes.

The locations of the individual test borings were selected and located by Underwood McLellan & Associates Limited. The approximate test boring locations are graphically illustrated on the accompanying Mosaic and Test Hole Location Plan.

All variations of soil, permafrost and groundwater conditions encountered during the test boring operations were noted and representative samples of the existing subsoils were taken. The majority of samples were disturbed, having been obtained by the air recovery method. Standard Penetration blow counts were taken with a two-inch solid penetrometer.

In the laboratory, the samples of materials derived from the drilling operations were subjected to standard laboratory classification procedures including particle size distribution and Atterberg index property determinations. Moisture contents were obtained for all samples returned to the laboratory.

Permafrost samples were classified according to N.R.C. Technical Memorandum No. 79 "Guide to the Field Description of Permafrost." The excess ice content of permafrost samples was obtained by visual inspection and by measured thicknesses of ice lenses.

Excess ice content was determined by the division of the ice thickness by the total sample thickness.

All soil samples were classified according to the Unified Soil Classification System.

The results of all laboratory and field tests are summarized in the Appendix on the Test Hole Logs.

Test holes have been designated by mileage, type and number. Example includes 511B420B, 511C285A and 511S288A. The letter symbols and numbers indicate:

511 - mileage

B, C, S - between mileage and test hole number indicate borrow, centreline and structure test holes, respectively.

420, 285, 288 - test hole numbers.

A, B - after test hole numbers indicate drilling performed by Mayhew 1000 or Heli drill, respectively.

C. SITE AND SUBSOIL CONDITIONS

The Steep Creek site is located at mile 511.8 (chainage 1085 + 00) of the Mackenzie Highway in the Northwest Territories. The Steep Creek flows into the Mackenzie River from the east having a drainage basin which extends to the Franklin Mountains.

The stream elevation at the proposed crossing is 328 (D.P.W. datum) with the estimated high water mark at elevation 351 as a result of ice jams on the Mackenzie River. The main stream channel is approximately 10 feet deep and exhibits a typical gravel bed. The stream is presently depositing silts, sands and gravels in a delta at the entrance to the Mackenzie River.

Three test holes 291A, 288A and 290A were drilled at the bridge site in the vicinity of proposed abutment and pier foundations. These three holes all indicated up to 25 feet of unfrozen saturated gravel with large boulders distributed throughout the strata. The moisture contents of the gravel were low in the range of 3% to 4% but in test hole 288A where a higher silt content exists, moisture contents reach 10%. A solid penetrometer blow count of 31 blows for 3 inches indicated the dense nature of the granular deposit, although the very large boulders can contribute to misleading inferred densities. At the time of drilling these test holes, the water table was 9 to 14 feet below the existing ground surface.

Three test holes drilled on the south slope indicated 5 to 8 feet of silty sand and silt with moisture contents from 30% to 40%. This surface strata was underlain by sand and gravel with low moisture contents of 4%, although test hole 287A had sand and gravel moisture contents of 12% where the groundwater table was present. The soils encountered on the south slopes were all unfrozen except for two feet of surface seasonal frost penetration.

Test hole 289A drilled on the north river terrace 300 feet from the bridge site indicated unfrozen sand and coarse gravel to a depth of 34 feet below grade and free water was encountered at 28 feet. Near the uplands about 1000 feet from the bridge crossing, test hole 421B indicated eight feet of interbedded unfrozen clay and sand which was underlain by dry gravel.

The deepest test hole of 35 feet did not encounter bedrock at Steep Creek but Cretaceous interbedded sandstones, shales and coal would be expected. A soil profile with plotted test hole logs of the bridge valley site is included in the Appendix.

D. CONCLUSIONS AND RECOMMENDATIONS

On the basis of the present investigation, we wish to offer the following generalized conclusions and recommendations relative to the design and construction of the proposed Steep Creek bridge foundations and approaches.

1. Pier and Abutment Foundation Design

The gravel and sand deposits which were described in the previous section will form the bearing strata for the proposed bridge piers and abutments. A spread footing type foundation is not recommended for the piers which would bear in the upper loose saturated surface stream deposited gravels. Consequently, a driven pile foundation is recommended for all structures.

The pile types generally available would include timber, pre-cast concrete, steel H-piles and pipe piles. The timber piles are not recommended as a result of their low capacity and possible damage when driving through gravel strata. The pre-cast concrete piles are also not recommended primarily due to the difficulty in establishing ultimate pile lengths and unless the lengths can be pre-determined considerable difficulty in splicing results.

Steel H-piles or pipe piles are, therefore, recommended for consideration primarily based on high driving strength, high load capacity and relative ease in splicing. The granular strata at this site will provide adequate lateral support such that instability in the form of buckling of the piling will not be a problem.

It is recommended that all piles be driven to "refusal". "Refusal" will depend upon the energy rating of the hammer but it is commonly 15,000 ft. lb. As an initial guide, piles should be driven to blow counts of 180 blows/ft. or 15 blows/inch. The pile capacity is largely a function of the amount of energy expended in installing the pile and not just of the recorded resistances. The pile which is driven to a sustained resistance will perform better than one which is terminated the instant a given resistance is attained. Of course, the pile must not be driven until damage occurs and whenever resistance increases greatly, the driving should be terminated.

Extreme difficulty exists in establishing the insitu density and therefore, predicting the "refusal" depth in gravel strata. Steel piles driven to "refusal" can be expected to attain allowable load capacities in the range of 80 tons depending upon the cross-sectional area. "False" refusal may occur wherever extremely large boulders cannot be penetrated during driving. Although refusal may be attained while driving, fundamental refusal bearing capacity may not exist below the pile tip. Load tests would be necessary to establish allowable loads if this situation is recognized. In order to attain penetration of the steel piles around large boulders, it may be necessary to utilize vibration techniques. When driving piles through gravel strata, large open-pipe piles have often proven successful by crushing boulders with churn drilling procedures at the pipe base.

If refusal is not achieved during driving to significant depths, the capacity of the pile may be approximately based upon friction between the steel and gravel. It is recommended that the allowable capacity be based upon 800 psf over the surface area of either a H-steel or pipe pile. Consequently, a 12" diameter pipe pile driven 65 feet into the gravel would possess a capacity of approximately 80 tons.

Whenever conventional driving techniques are employed the capacity of a pile should be established in the field by several pile-driving tests. The ultimate capacity would be calculated on the basis of a dynamic pile-driving formulae such as the Hiley, Danish or Weisbach. It is recommended that the Engineering News formula not be utilized as a result of its extreme variation in factor of safety.

For piles installed by use of a vibratory hammer, there is no blow count and penetration resistance is measured by rate of penetration (inches/sec.). Therefore, conventional hammers must be utilized to "set" the pile after vibration to allow utilization of pile-driving formulae.

It is further recommended that static load tests be performed to establish more accurately the bearing capacity of a typical steel pile and the applicability of dynamic pile formulae. Data obtained from load tests on piles driven into deep granular deposits will be of assistance in designing piles throughout the Mackenzie Highway System.

The lateral resistance of piles can be established by recently developed methods based on the beam on elastic foundation method but the simple and more conventional approach is based on calculation of the Rankine passive earth pressure against the pile. Using Rankine theory the ultimate passive resistance force/ft.^{2} of a pile can be obtained by $P = \frac{1}{2} \gamma_b K_p H^2$. The submerged unit weight of the granular material may be taken as 60 pcf and the passive pressure coefficient K_p as 3.0. In order to establish the allowable lateral load a factor of safety of 2.5 should be applied to the above load. Generally, in cohesionless soils the lateral load tends to exceed the passive resistance, consequently, the above simplified approach is on the conservative side.

The relatively deep approach fills would be expected to produce some settlement of the insitu materials and subject the piles to negative skin friction but the cohesionless nature of the subsoil materials will not produce significant displacements and those which occur will be rapid.

Wherever the deep embankments are constructed of compacted granular fill as outlined and recommended in section D3, the abutments may be placed on spread footings in the fill. The bearing capacity of the spread footing type abutment foundation will depend upon the material and compaction techniques utilized in the fill. If inferior material or compaction techniques are utilized in the embankment, the abutment must be carried on piles into the insitu granular subsoils below the creek channel. Piles must be designed to carry a negative skin friction load whenever embankment fills may undergo settlement.

2. Lateral Abutment Loads

The magnitude of the pressure applied to the abutment will depend upon the characteristics of the backfill chosen for the approach fill and upon the movements that the wall undergoes during and subsequent to fill compaction. It is recommended that granular backfill from the river terraces or borrow pits be utilized for the abutment approach embankments which will be in the order of 25 feet high.

The most common procedures for analysis of the earth pressures against gravity retaining walls include the Rankine and Coulomb methods.

Generally, it is assumed that the wall rotates sufficiently to allow mobilization of the backfill shear strength which reduces the pressure to the active case, the lowest pressure that can be realized. This assumption applies reasonably accurately when the backfill consists of uncompacted granular material but pressures greater than the active occur whenever compaction of the backfill is undertaken. While the total yield at different elevations on the abutment may be adequate for mobilization of the full backfill strength, a gradual build-up of the backfill also results in a gradual yielding of the wall. Therefore, by the time the backfill is complete only a fraction of the total yield of the wall has become effective in decreasing the earth pressure. Recent studies have indicated that the actual earth pressure magnitude will exceed the active case and may approach the earth pressure at-rest. Pressures greater than the at-rest case may occur if heavy equipment is utilized against the concrete abutment.

If the Coulomb approach is utilized, the angle of wall friction between concrete and coarse sand and gravel would be 20 degrees. The angle of internal friction for compacted granular material would be 38 degrees and the dry density may be taken as 115 pcf. For a simple vertical retaining wall abutment with horizontal backfill the Coulomb active coefficient would be .24 and the at-rest coefficient based on $K_o = 1 - \sin \phi$ would be .38. These design parameters are based upon "average" gravel conditions but wherever deep abutments are proposed the actual shear strength parameters of the backfill should be established to accurately evaluate the lateral pressures. In many cases the pressure distribution behind a gravity retaining wall has been shown to be non-triangular, but as a first approximation and for simplicity, the triangular pressure diagram is recommended. It is also recommended that whenever backfill is compacted in lifts by vibratory compaction the at-rest coefficient should be utilized for calculation of the ultimate earth pressure.

Wherever potential fill settlement is anticipated a driven pile foundation has been previously recommended for the abutments which will provide significant lateral resistance but it is further recommended that some batter piles be utilized to ensure adequate lateral support against earth pressures and fill movement. Whenever batter piles are utilized for horiziontal resistance, the lateral restraint of vertical piles is neglected.

Positive drainage including horizontal perforated tile and weep holes should be provided to prevent development of hydrostatic pressures against the wall. In addition, drainage will prevent development of frost heaving forces against the abutments.

3. Bridge Approach Grades and Embankments

The centreline grades at the proposed location of the Steep Creek bridge will involve embankment depths from 3 to 25 feet in depth. These fills will generally overlie unfrozen granular materials near the stream channel and excavation of the surface insitu material is not necessary. Some settlement of the interbedded sandy silts and clays near the surface can be expected, consequently it is recommended that valley fills be placed several months before bridge construction to allow consolidation to occur. Much of the subsoil on the valley slopes consists of granular material and minimum depths of 3 foot fills may be utilized. In the area of station 1092 + 00 a 2 foot depth of muskeg is present which will be overlain by 6 feet of fill. It is recommended that this muskeg remain in place, although surface settlements of 1 foot can be expected.

It is further recommended that granular fill be compacted at all deep approach fills to 95% of

Standard Proctor density utilizing vibratory compaction. Although construction may be performed during the winter, the dry unfrozen nature of the surface granular materials will allow satisfactory compaction. It is emphasized that compaction of these deep approach fills is mandatory to ensure slope stability and prevent settlements at the bridge approaches. All fill slopes should be constructed at 2:1. Common excavation will not be available from the bridge site valley slopes as the grade line has been proposed entirely to produce fill sections. Granular embankment fill is available throughout the river terraces but its utilization as a construction material may be prohibited by land use regulation.

Suitable granular borrow material has been located at station 1065 + 00, 500 feet east of the proposed centreline. Two of the test holes drilled in the borrow pit indicated dry unfrozen gravel and sand which caused severe caving as a result of its dry state. Other suitable borrow pits were located on the uplands but haul distances are $1\frac{1}{2}$ to 2 miles from the deep bridge approach fills.

APPENDIX

EXPLANATION OF FIELD & LABORATORY TEST DATA

The field and laboratory test results as shown for a particular test boring by the Test Hole Log Data Sheet are briefly described below:-

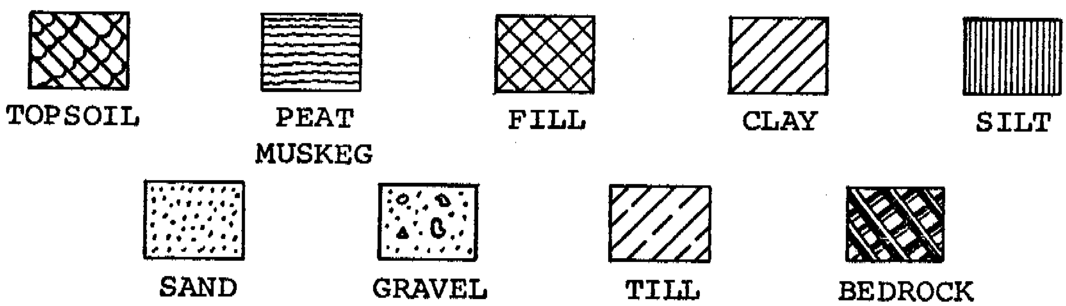
NATURAL MOISTURE CONDITIONS & ATTERBERG LIMITS

The relation between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits should be compared to the Natural Moisture Content of the subsurface soil as well as plotted on the Plasticity Chart.

SOIL PROFILE & DESCRIPTION

Each soil strata is classified and described noting any special conditions. The unified classification system is used, and the soil profile refers to the existing ground elevation. When available the ground elevation is shown.

The soil symbols used are briefly shown below but are indicated in more detail in the Soil Classification Chart.



TESTS ON SOIL SAMPLES

Laboratory and field tests are identified by the following symbols:

QU - unconfined compressive strength usually expressed in tons per square foot. This value is used in determining the allowable bearing capacity of the soil.

γ_d - dry unit weight expressed in pounds per cubic foot. This value indicates the density or consistency of the in-situ soil.

C - Consolidation test. These test results are separately enclosed and provide information on the consolidation or settlement properties of the soil strata.

M.A. - grain size analysis. These test results are separately enclosed and indicate the gradation properties of the material tested.

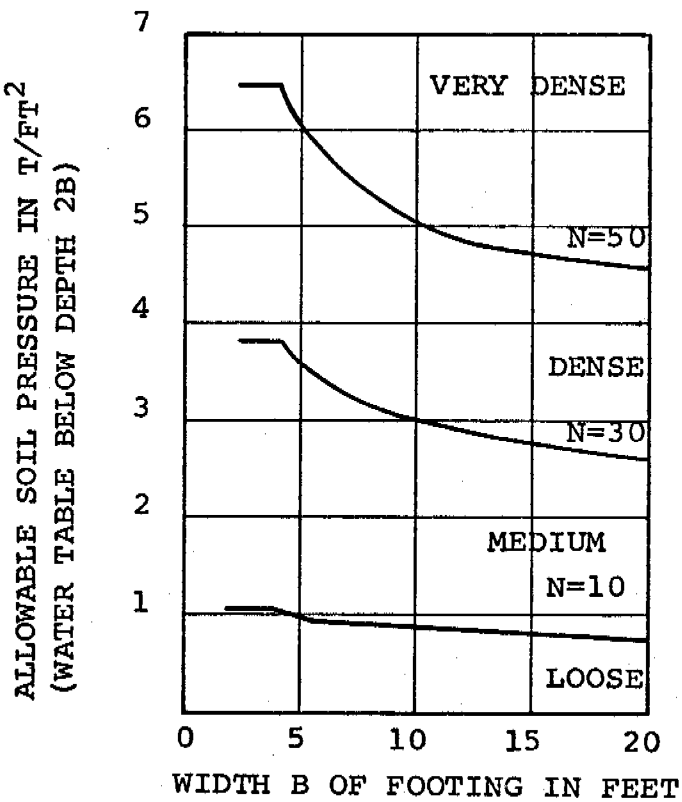
SO_4 - water soluble sulphate content is conducted primarily to determine whether sulphate resistant cement is required for the foundation structure.

N - standard penetration field test. This test is conducted in the field to determine the in-situ consistency of a soil strata. The "N" value recorded is the number of blows from a 140 lb. hammer dropped 30 inches (free fall) which are required to drive a 2" O.D. Raymond type sampler 12 inches into the soil.

The resistance and unconfined compressive strength of a cohesive soil can be related to its consistency as follows:-

N - BLOWS/Ft.	QU - T/Ft.^{2}	CONSISTENCY
2	0.25	very soft
2-4	0.25-0.50	soft
4-8	0.50-1.00	medium or firm
8-15	1.00-2.00	stiff
15-30	2.00-4.00	very stiff
30	4.00	hard

The resistance of a non-cohesive soil (sand) can be related to its consistency as follows:-



SAMPLE CONDITION AND TYPE

The depth and condition of samples are indicated by the following symbols:

UNDISTURBED DISTURBED LOST SAMPLE

SAMPLE TYPES

U	-	3" O.D. Shelby tube sample
D.S.	-	drive sample
M	-	moisture content sample
R.C.	-	rock core sample

PERCENTAGE WATER SOLUBLE SULPHATE CONCENTRATION

0	0.1	0.2	0.3	0.4	0.5	0.6	0.7%
Negligible	Positive	Considerable			Severe		
RELATIVE DEGREE OF SULPHATE ATTACK							

Negligible - Normal Portland Cement may be used.

Positive - Normal Portland Cement may be used, provided the strength of the concrete is increased up to 500 psi higher than the compressive strength which would normally be used.

Considerable - Type V cement must be used and the concrete compressive strength should be increased to 500 psi higher than the compressive strength which would normally be used.

Severe - Type V cement must be used and the concrete compressive strength should be increased from 500 to 1000 psi higher than the compressive strength which would normally be used.

GROUND WATER TABLE

The water table is indicated by the level of standing water in a test boring after equilibrium has been reached. This is generally taken 24 hours after the drilling operation. The water table is usually an inclined surface that is dynamic in nature with its highest level late in the winter or early spring gradually falling throughout the summer.

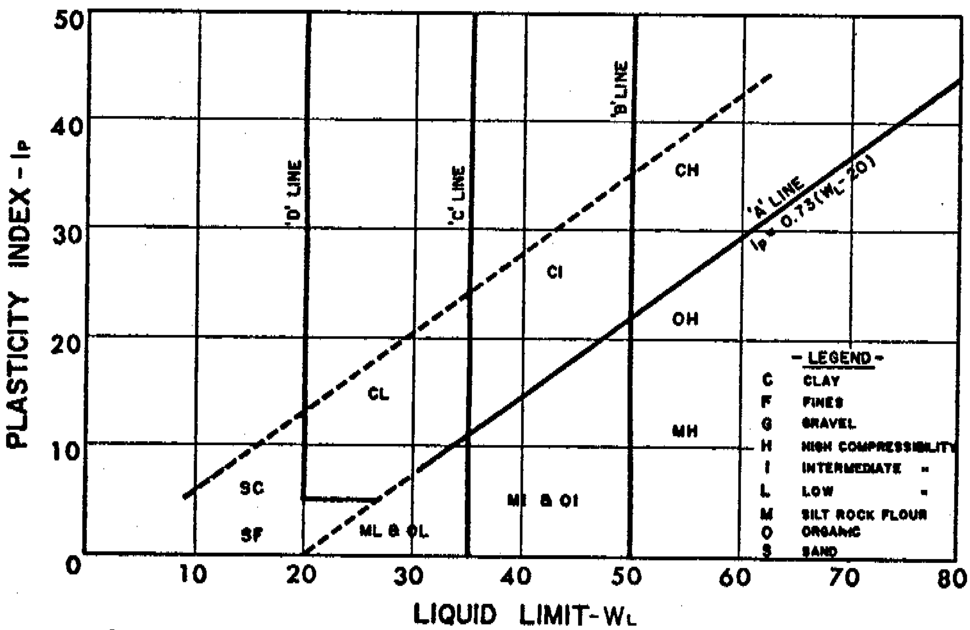
UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

SOIL CLASSIFICATION CHART

PLASTICITY CHART



FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS

NATIONAL RESEARCH COUNCIL PERMAFROST CLASSIFICATION SYSTEM

Permafrost ground ice occurs in three basic conditions including non-visible, visible (less than one inch in thickness) and clear ice.

A. Non-visible - N

N_f - poorly bonded or friable frozen soil

N_{bn} - well bonded soil, no excess ice

N_{be} - well bonded soil, excess ice

B. Visible - V (less than 1" thick)

V_x - individual ice crystals or inclusions

V_c - ice coatings on particles

V_r - random or irregularly oriented ice formations

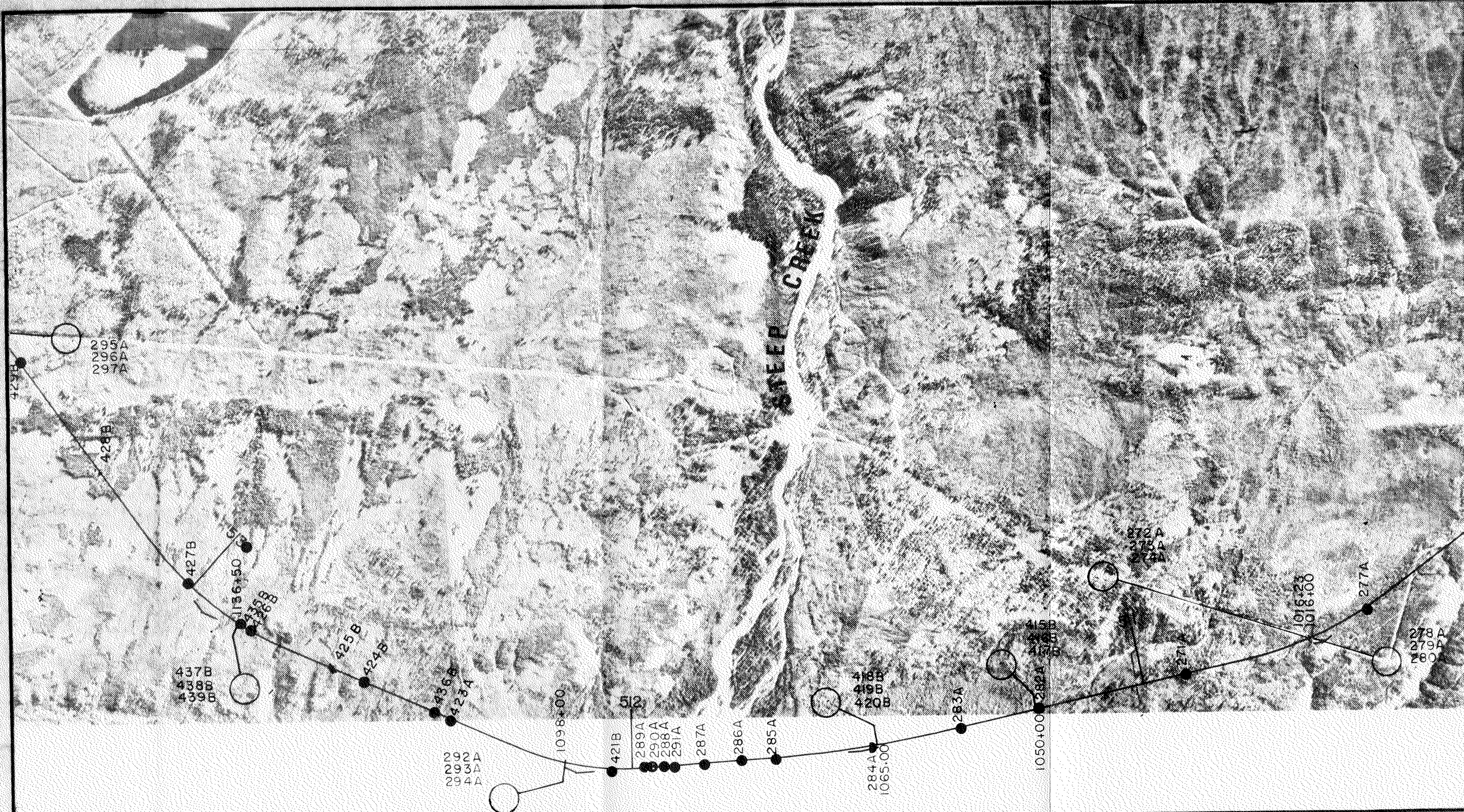
V_s - stratified or oriented ice formations

C. Visible Ice - (greater than 1" thick)

Ice - ice with soil inclusions

Ice + Soil - ice without soil inclusions.

A more complete description of this system is included in NRC publication TM 79.



Issue	Date	Remarks	No.	Revision	Date	By	Approved

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

CONSULTING ENGINEERS – TOWN PLANNERS

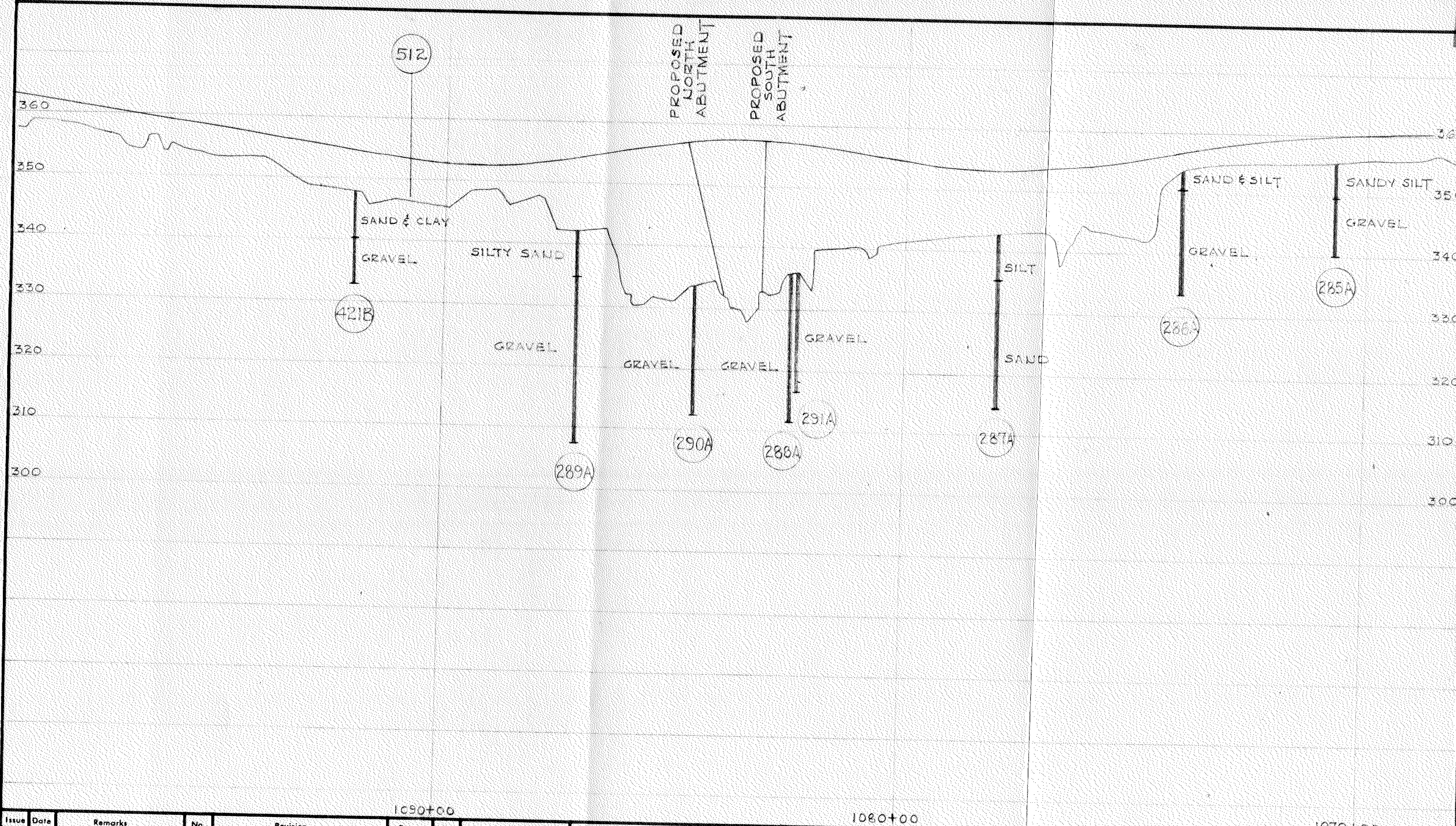
BRITISH COLUMBIA • ALBERTA • SASKATCHEWAN • MANITOBA • ONTARIO

Drawn: H·M·S	
Checked: D·G·P	
Design:	
Traced:	
Scale: 1" = 1000'	

MACKENZIE HIGHWAY

STEEP CREEK MOSAIC AND TEST HOLE LOCATION PLAN

Date: APRIL/73	
Job:	
Sheet:	
Drawing: PLATE - I	
Issue:	



Issue	Date	Remarks	No.	Revision	Date	By	Approved

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

CONSULTING ENGINEERS – TOWN PLANNERS

BRITISH COLUMBIA • ALBERTA • SASKATCHEWAN • MANITOBA • ONTARIO

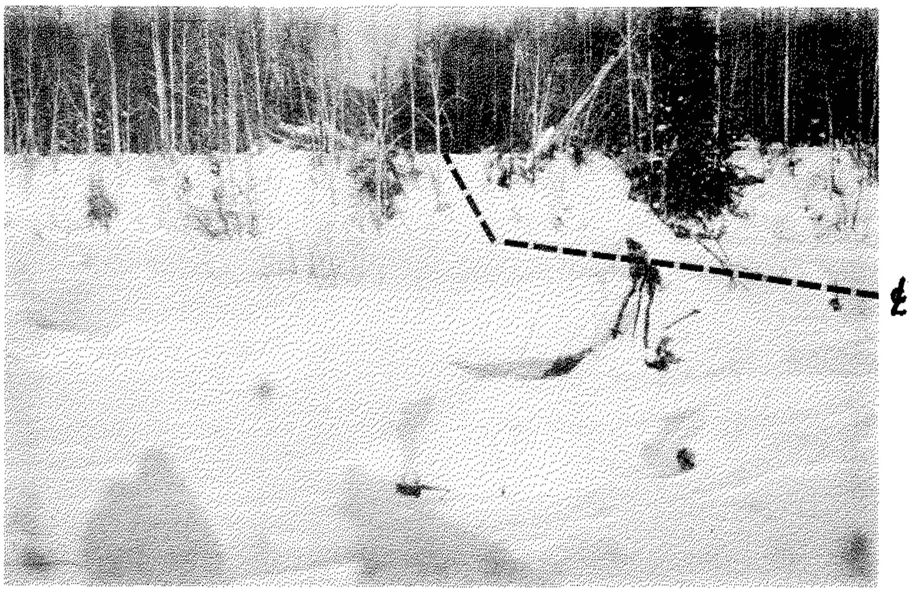
Drawn: H. M. S.
Checked: D. G. P.
Design:
Traced:
Scale: AS SHOWN

MACKENZIE HIGHWAY

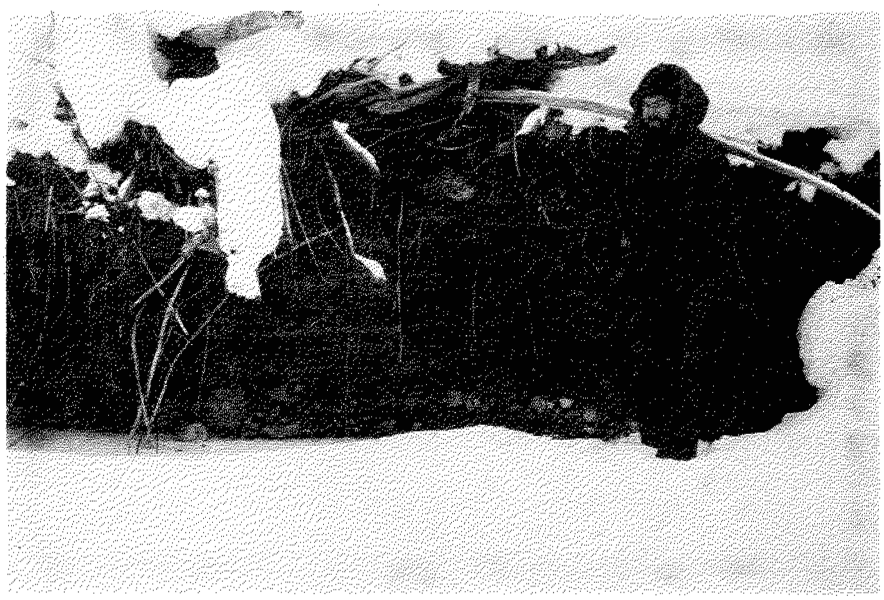
STEEP CREEK BRIDGE SITE

PROFILE & SOIL CROSS SECTION

Date: MAR./73
Job:
Sheet:
Drawing: PLATE-2
Issue:

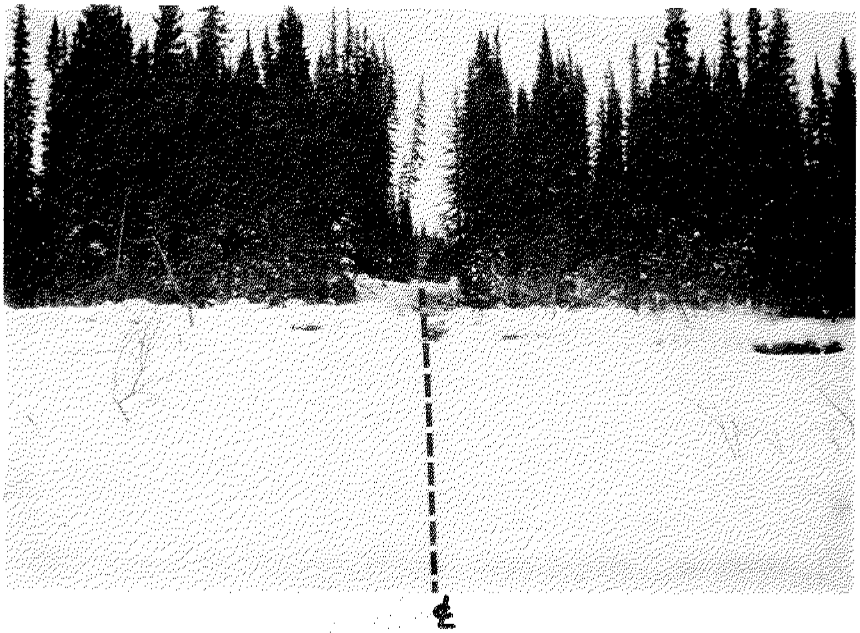


North Bank of Crossing Viewed in
a Northerly Direction

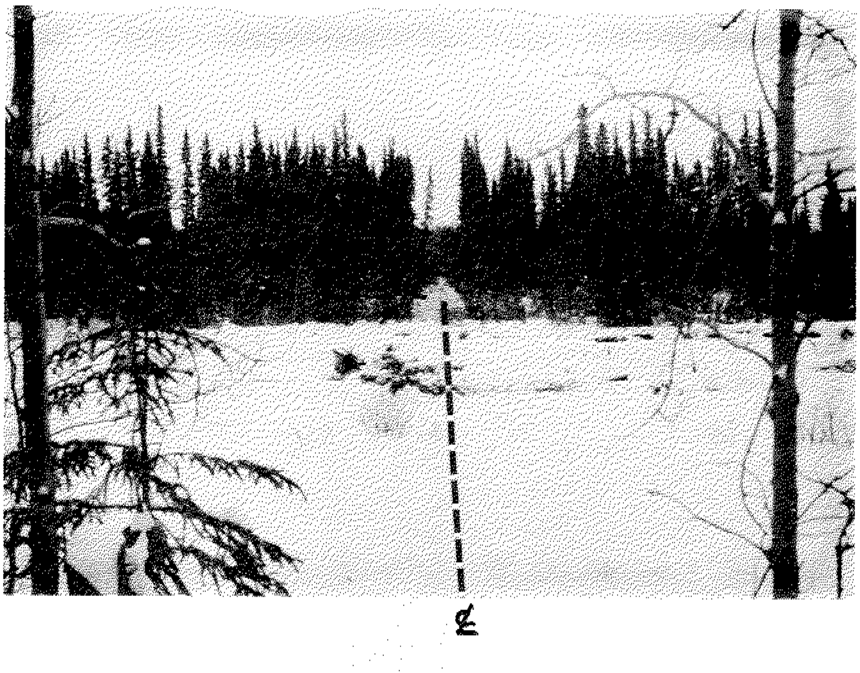


View of North Bank Exposed
400' West of Proposed Crossing
NOTE: Silty-clay Material overlaying
Boulders & Gravel at a depth of Six Feet

Plate 3



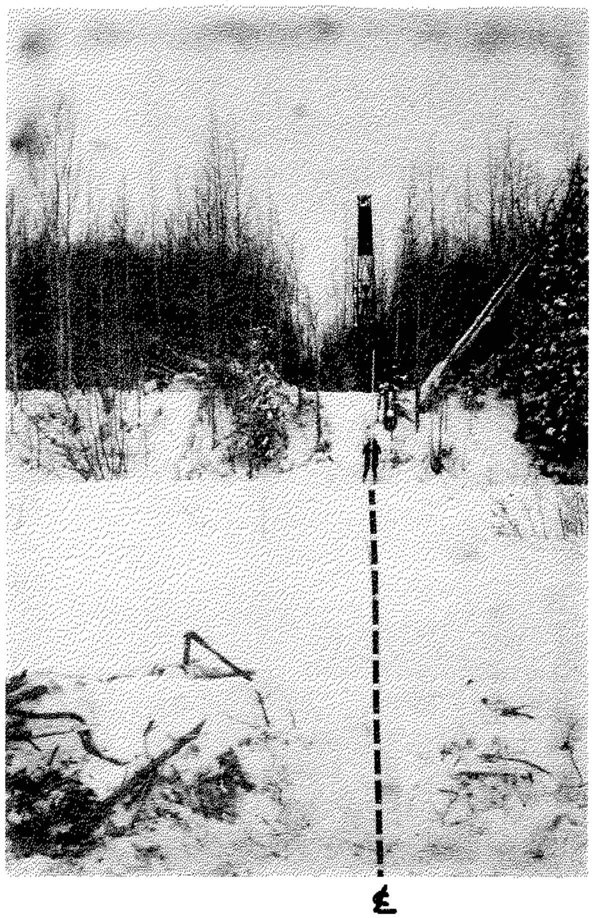
Viewing South Bank along ℄
of Crossing



Viewing South Bank of Crossing
from Top of North Bank along
℄ of Crossing

Plate 3

STEEP CREEK CROSSING
MILE 511.5

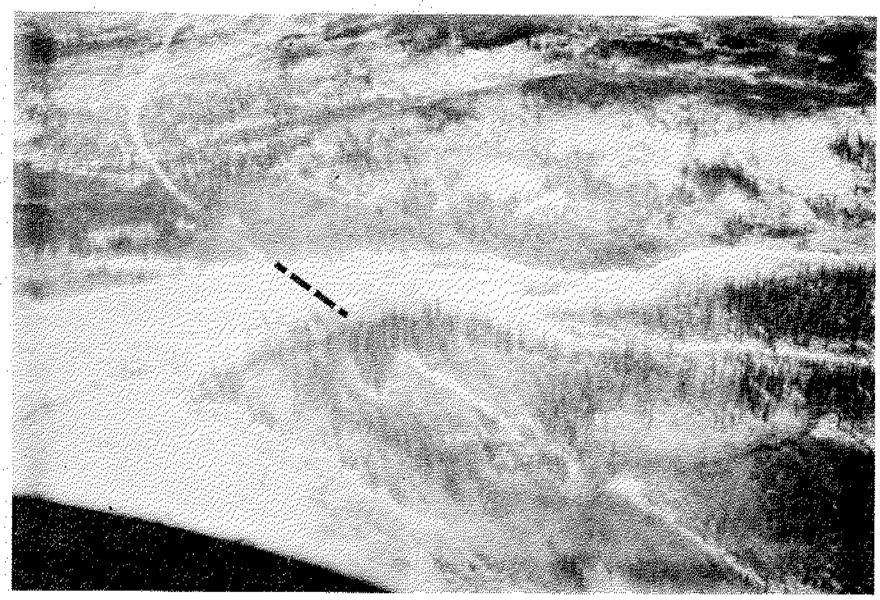


North Bank Viewed along ℄ of Crossing

Plate 3.

STEEP CREEK CROSSING MILE 511.5

Viewing Crossings in a Northerly
Direction



UNDERWOOD McLELLAN & ASSOCIATES LIMITED	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

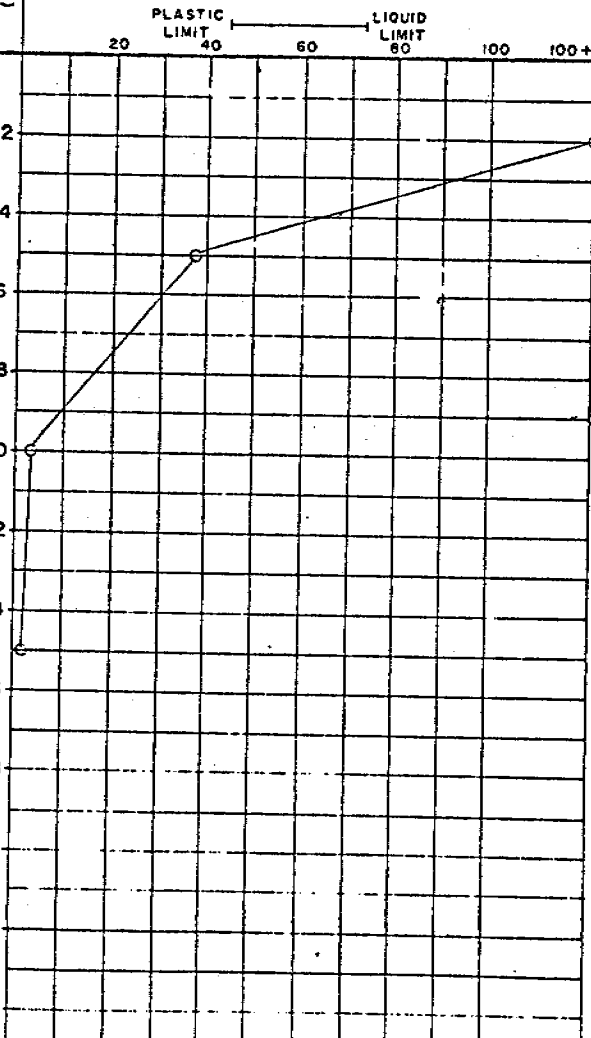
CAN R.S.	FIELD ENG G.W.M.	DATE DRILLED 2/2/73	AIRPHOTO NO:	CHAINAGE: 1070+00	OFFSET ℄
CKD	TECH	RIG: AIR	SURFACE DRAINAGE: POOR	VEGETATION: 30' SPRUCE 20' BIRCH	ELEV.

TEST HOLE

MILE	B,C,S	NUMBER
511	C	285 A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–1						ORGANIC		
1–4					Pt-ML	SILT	F	
4–8					G.M.	GRAVEL	UF	
8–14					G.W.	GRAVEL & SILTY SAND	UF	
14–15					G.W.	GRAVEL & SAND		
15						END OF HOLE		

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



GRAIN-SIZE ANALYSIS: CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS

PLATE 4

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

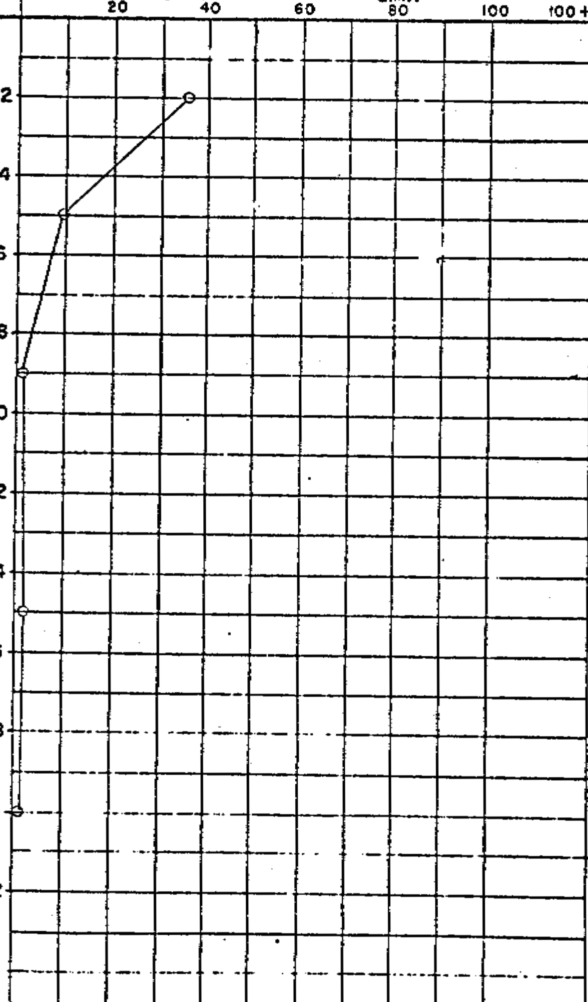
CAN R.S.	FIELD ENG B.W.M.	DATE DRILLED 2/2/73	AIRPHOTO NO:	CHAINAGE: 1074+00	OFFSET ℄
CKD	TECH	RIG AIR	SURFACE DRAINAGE GOOD N.W.	VEGETATION:	ELEV

TEST HOLE

MILE	B,C,S	NUMBER
511	C	286A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–2					S.M.	8" ORGANIC SILT SANDY	F	
2–5					G.M	SAND SILTY, GRAVEL	UF	
5–20					G.W. G.W G.W	GRAVEL & SAND	UF	
20						END HOLE @ 20'		

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT — LIQUID LIMIT



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P C F)	DRY DENSITY (P C F)	REMARKS

PLATE 5

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

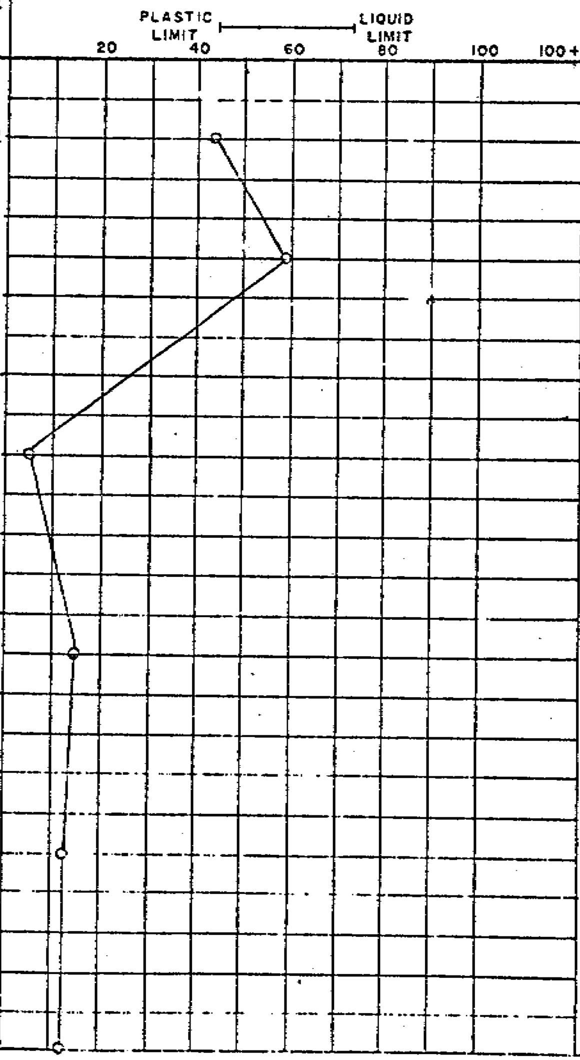
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

CAN R.J.	FIELD ENG B.V.M.	DATE DRILLED 2/2/73	AIRPHOTO NO:
CKD	TECH	RIG: AIR	SURFACE DRAINAGE: POOR
CHAINAGE: 1078+00	OFFSET: ℄	VEGETATION: 15' BIRCH	ELEV.

TEST HOLE		
MILE	B,C,S	NUMBER
511	C	287A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–1.5						ORGANIC	F	
1.5–7.5						SILT & SAND	F (to ~5)	
7.5–25						SAND, PEBBLES	UF	
						SILTY SAND & GRAVEL	UF	
25						END HOLE @ 25'		

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



GRAIN-SIZE ANALYSIS						
CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
						FREE WATER FROM 18'

PLATE 6

UNDERWOOD McLELLAN & ASSOCIATES LIMITED	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: A.E.W.	FIELD ENG S.M.	DATE DRILLED 3/2/73	AIRPHOTO NO:	CHAINAGE: 1082+50	OFFSET: ℄	TEST HOLE
CKD DGP	TECH:	RIG: AIR	SURFACE DRAINAGE: GOOD	VEGETATION: SPRUCE TO 20'	ELEV:	

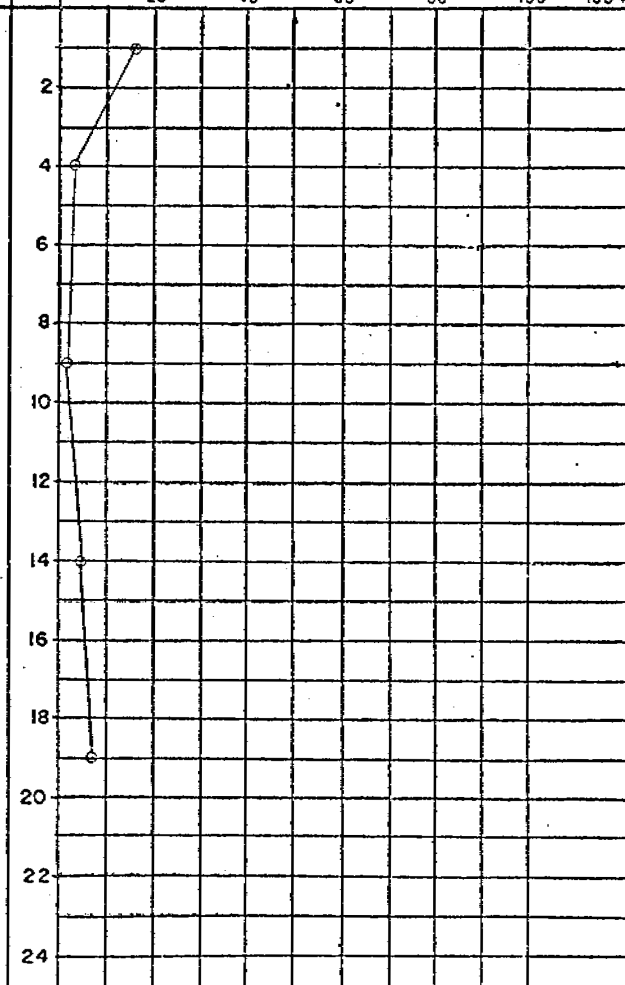
MILE	B,C,S	NUMBER
511	S	291A

REMARKS

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–2		sample			GW	GRAVEL 3" ORGANIC	UF	
4		sample						
9–10		sample						
12						▽		
14		sample						
19–20		sample			GW	SOME CLAY		
20–22						END OF HOLE		

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

PLASTIC LIMIT ├──┤ LIQUID LIMIT



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)

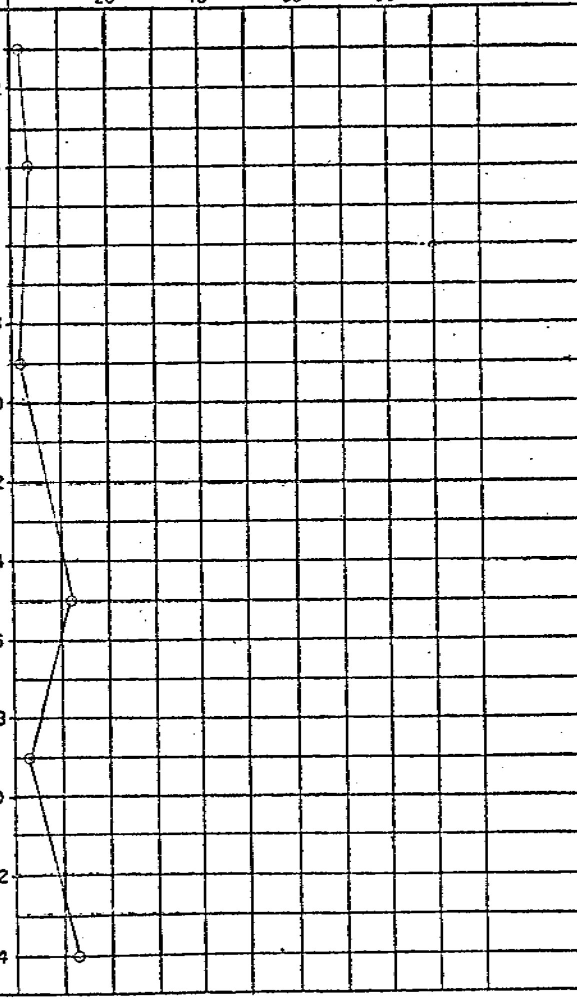
PLATE 7

UNDERWOOD McLELLAN & ASSOCIATES LIMITED	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: A.E.W	FIELD ENG: WWC	DATE DRILLED: 2/2/73	AIRPHOTO NO:	CHAINAGE: 1082+50	OFFSET: ℄	TEST HOLE
CKD: DSP	TECH:	RIG: AIR	SURFACE DRAINAGE: GOOD	VEGETATION: SPRUCE TO 30'	ELEV:	

MILE	B,C,S	NUMBER
511	S	285A

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT —— LIQUID LIMIT



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 (PCF)	DRY DENSITY (PCF)	REMARKS
0						MOSS	UF								
1–2						GRAVEL SAND, COBBLES AND BOULDERS			4		18	78			ROCK FRAGMENTS IN SAMPLE
12						SAND WITH SILT AND CLAY VERY WET									
14															WATER COMING OUT OF HOLE AT 14'
18						SAND WITH COBBLES AND BOULDERS VERY WET									
22–24						CLAY DARK BROWN LITTLE SAND END OF HOLE @ 25'									HOLE SLOUGHING

PLATE 8

UNDERWOOD McLELLAN & ASSOCIATES LIMITED	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

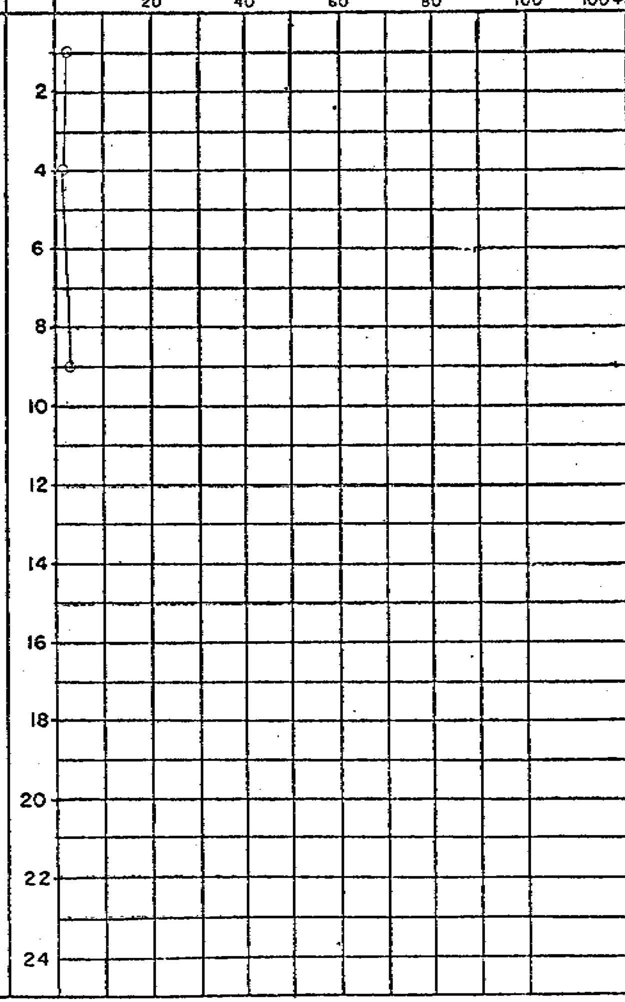
DWN: A.E.W	FIELD ENG	DATE DRILLED 3/2/73	AIRPHOTO NO:	CHAINAGE: 1084+40	OFFSET: ℄
CKD D.G.P	TECH:	RIG: AIR	SURFACE DRAINAGE: CREEK	VEGETATION:	ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
511	S	290A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–20					GW	GRAVEL	UF	
14–15		P		31 BLS (3")				
16					GW			
19						SOME CLAY		
20						END OF HOLE		

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT — LIQUID LIMIT



GRAIN-SIZE ANALYSIS: CLAY % | SILT % | SAND % | GRAVEL % | WET DENSITY (P C F) | DRY DENSITY (P C F) | REMARKS

PLATE 9

UNDERWOOD McLELLAN & ASSOCIATES LIMITED	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: A.E.W.	FIELD ENG: N.W.C.	DATE DRILLED 3/2/73	AIRPHOTO NO:	CHAINAGE: 1087+00	OFFSET: ℄
CKD D.G.P.	TECH:	RIG: AIR	SURFACE DRAINAGE: GOOD	VEGETATION: ASPEN & WILLOW	ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
511	S	289A

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 (PCF)	DRY DENSITY (PCF)	REMARKS
						MOSS									
4					ML	SAND LIGHT BROWN, SILTY FINE	F UF								SO_4 = 0.11%
8					GP	GRAVEL			17		43	40			
12						SAND COBBLES AND BOULDERS									
16						ROCK FRAGMENTS									HOLE SLOUGHING
20															
24					SP										
28						▽									
32					SW	SILT, SAND BOULDERS									
36						END OF HOLE									

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

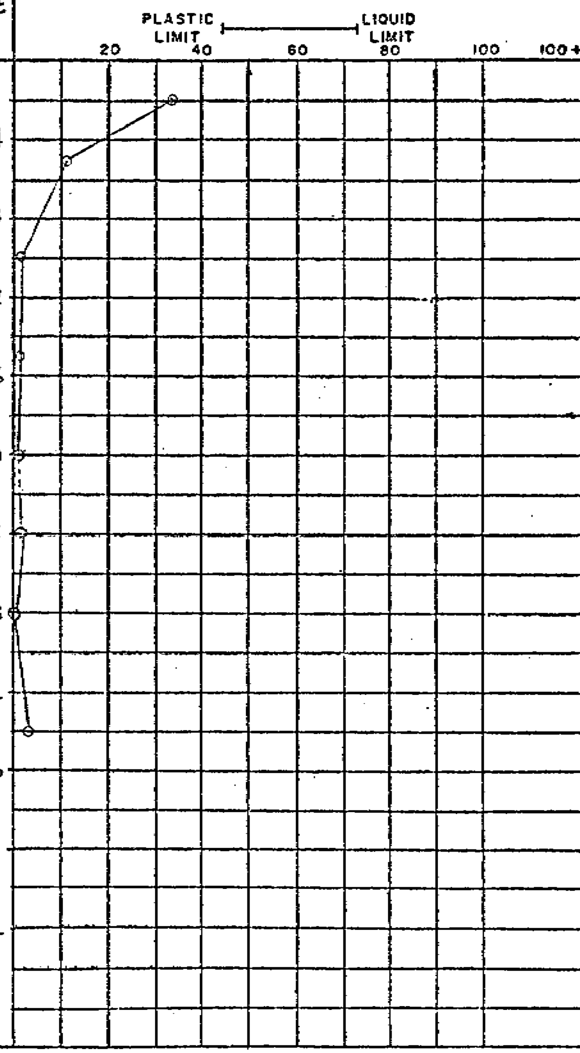


PLATE 10

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

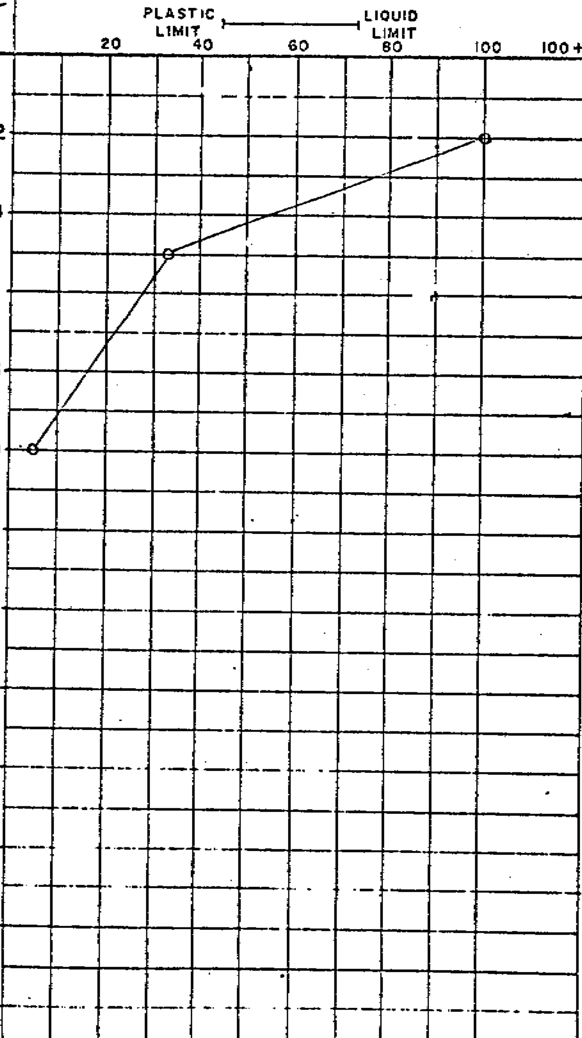
DWN R.S.	FIELD ENG B D.M.	DATE DRILLED 2/2/73	AIRPHOTO NO:	CHAINAGE: 1092+00	OFFSET ℄
CKD	TECH	RIG AIR.	SURFACE DRAINAGE POOR	VEGETATION: 20' WILLOW, ASPEN, SPRUCE	ELEV.

TEST HOLE

MILE	B,C,S	NUMBER
512	C	421 B

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–2					Pt.	MUSKEG GRAN., SOME ROOTS	F	
2–4						SAND LIGHT BROWN, MOIST	UF	
4–6		P			CL	CLAY BROWN, SOFT, MOIST		
6–8						SAND DARK BROWN, MOIST		
8–14					G.W.	GRAVEL COBBLES, SAND	UF	
						ABANDONED @ 14'		

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT — LIQUID LIMIT



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
						SLUFFING

PLATE 11

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

CAN R.S.	FIELD ENG D R.	DATE DRILLED 2/2/73	AIRPHOTO NO:	CHAINAGE: 1065+00	OFFSET 500' E of ℄
CKD	TECH	RIG AIR	SURFACE DRAINAGE GOOD - WEST	VEGETATION: 25' SPRUCE	ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
511	B	420B

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	GRAIN-SIZE ANALYSIS: CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
0–2.5					Pt	3" MOSS ORGANIC MED. BROWN	F								
2.5–5					S.P.	SAND & GRAVEL	UF								SLUFFING
5						ABANDONED @ 5'									

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

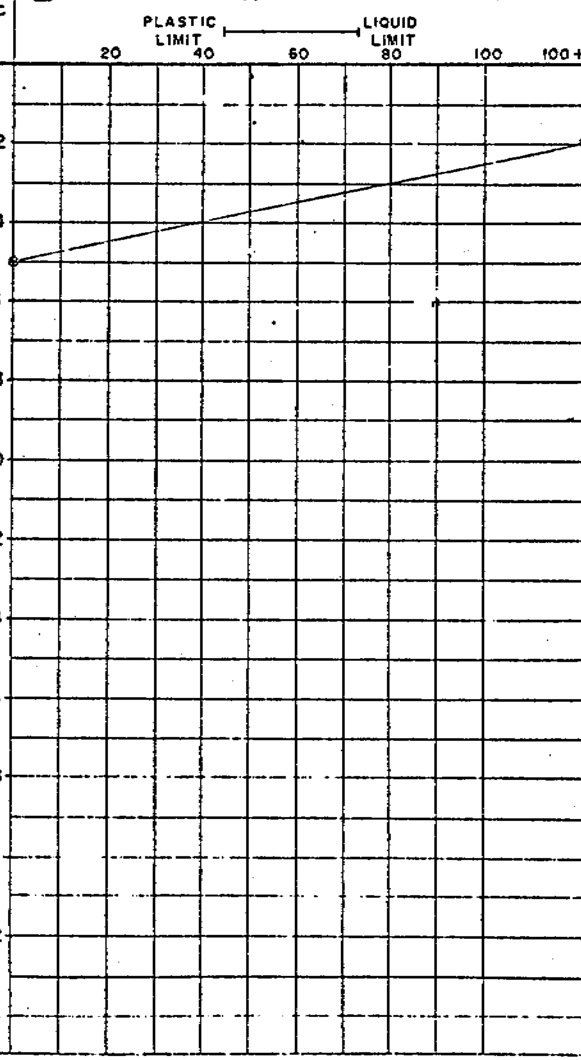


PLATE 12

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

CAN R.S.	FIELD ENG D.R.	DATE DRILLED 2/2/73	AIRPHOTO NO:	CHAINAGE: 1065+00	OFFSET 500' E of ℄
CKD	TECH	RIG AIR	SURFACE DRAINAGE GOOD - WEST	VEGETATION: 25' SPRUCE	ELEV.

TEST HOLE

MILE	B,C,S	NUMBER
511	B	419 B

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT —— LIQUID LIMIT
20 40 60 80 100 100+

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
0–1						3" MOSS SILT MED BROWN.									
1–3					S.P.	SAND SILTY	F								
3–5					G.W.	GRAVEL	UF								SLUFFING
						ABANDONED @ 5'									

PLATE 13

UNDERWOOD McLELLAN & ASSOCIATES LIMITED	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

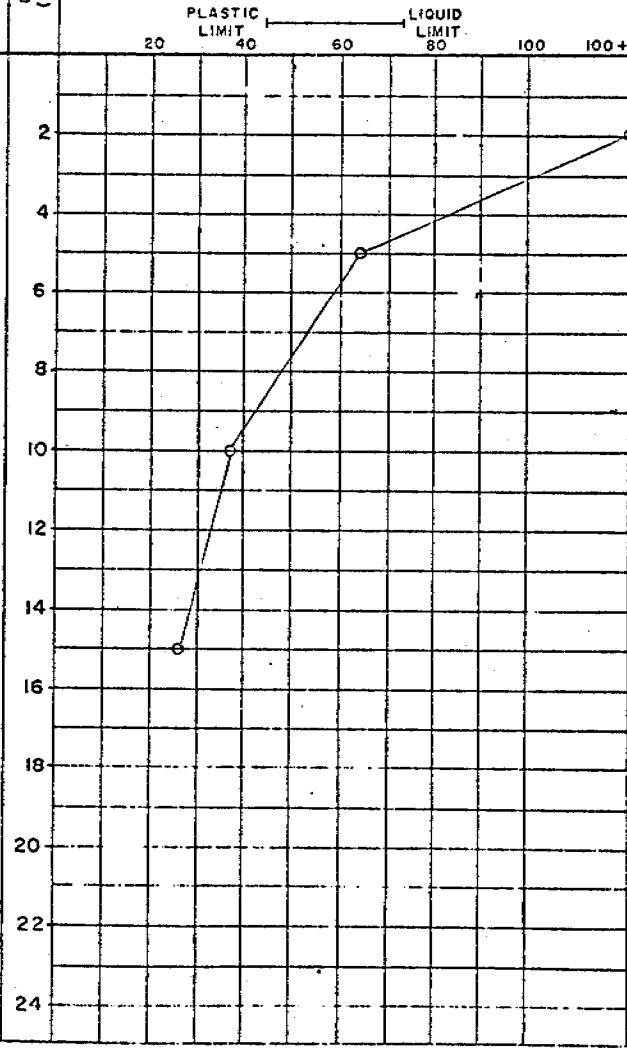
CAN R.S.	FIELD ENG D.R.	DATE DRILLED 2/2/73	AIRPHOTO NO:	CHAINAGE: 1065+00	OFFSET 500' E of ℄
CKD	TECH	RIG AIR.	SURFACE DRAINAGE G.ood-WEST	VEGETATION: 25' SPRUCE	ELEV.

TEST HOLE

MILE	B,C,S	NUMBER
511	B	418 B

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–1						3" MOSS ORGANIC	F	
1–3					Pt	ORGANIC & SILT	F	
3–6					GP&Pt	SAND & GRAVEL ORGANIC	F	
6–10.5					G.C.	GRAVEL & CLAY	UF	
10.5–12						BOULDERS	UF	
12–15					C.L.	CLAY SILTY, MED. BROWN,	UF	
15						END OF HOLE		

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT — LIQUID LIMIT



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS

PLATE 14