MACKENZIE HIGHWAY
GEOTECHNICAL EVALUATION
VOLUME XV
JACKFISH CREEK CROSSING MILE 721.2
MACKENZIE HIGHWAY

Submitted To:

GOVERNMENT OF CANADA
DEPARTMENT OF PUBLIC WORKS
CONTRACT NUMBER A10/73
FILE NUMBER 9305-52-307

MARCH, 1974

EBA Engineering Consultants Ltd.

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I. INTRODUCTION

In conjunction with a geotechnical engineering study carried out from Mile 725 to Mile 632 of the proposed Mackenzie Highway, several major river and stream crossings were investigated. The Jackfish Creek Crossing, whose geographic location is shown on the Key Plan, Drawing No. A-1, Appendix A, is one such site investigated in detail. Details of the investigation, site conditions, geotechnical data and recommendations pertinent to the development of the creek crossing, are reported herein.

This work was carried out for the Government of Canada, Department of Public Works, and was authorized by Contract Number A10/73, File No. 9305-52-307.

II. GEOTECHNICAL DATA AQUISITION

2.1 Field Testing

The evaluation of subsurface conditions has been based on field data obtained from five boreholes, drilled at the locations shown on the Site Plan, Drawing No. A-2, Appendix A. Of the five boreholes advanced, four were drilled as center line boreholes, in conjunction with the general route evaluation, and the fifth borehole was located and drilled specifically to define subsurface conditions at the creek crossing.

The special borehole was designated Borehole 721-S-2. The four center line boreholes were designated Boreholes 720-C-1 and 721-C-7 to 721-C-9, inclusive. Detailed borehole logs are presented in consecutive order in Appendix B.

All boreholes were drilled with a track mounted Mayhew 500 rotary drill rig, using a continuous air return circulation system. Boreholes advanced with this drill rig generally were 4-3/4 inches in diameter. Borehole penetration ranged from 18 feet to 40 feet, and averaged 22 feet in depth. Sampling consisted of representative bag samples, obtained at depths of 2 1/2, 5 and 10 feet, and at depth intervals of about 4 feet, thereafter, to the bottom of each borehole. No undisturbed soil samples were obtained at the Jackfish Creek Crossing, during this investigation.

2.2 Laboratory Testing

Laboratory testing was carried out on disturbed soil samples to determine the natural water content profile and Atterberg limits of the subsoil. The moisture content tests were undertaken in the field laboratory of EBA Engineering Consultants Ltd., while all other testing was confined to the EBA Edmonton laboratory. In addition to the laboratory testing outlined above, all samples were visually classified in both the EBA field and Edmonton laboratories. Soil classification was based on plasticity, according to the extended Unified Classification System (1)* and on textural classification according to U.S. Engineers Department (2) textural classification triangle.

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* Superscripted numbers in parentheses refer to the List of References presented at the end of this report.

Frozen ground was classified according to a modification of the NRC system for describing permafrost (3). This modification was necessary because the disturbed nature of the samples obtained did not permit full usage of the NRC system; especially in describing the form of the excess ice. The system used retains the symbols V and N for visible and non-visible ice, respectively, and the modifying symbols B and F for well bonded and poorly bonded non-visible ice, respectively. Excess ice quantities were estimated from visual observations. The results of laboratory tests are presented on the borehole logs (Appendix B), where applicable, and on the Summary of Results Table, Drawing No.C-1, Appendix C.

III. SITE CONDITIONS

The proposed Mackenzie Highway crosses Jackfish Creek at Mile 721.2, approximately 3 miles east of Fort Good Hope, N.W.T. A Key Plan of the Jackfish Creek area is presented as Drawing No. A-1, and Drawing No. A-2 Appendix A, presents a detailed Site Plan. Plate 1 of Drawing No. A-3, Appendix A, shows the crossing from the air in June, 1973,

Jackfish Creek drains a moderate sized area extending east of the Mackenzie River, south of the Hare Indian River and north of the Tsintu River basin. The size of the watershed, limited topographic relief and poorly defined creek channel suggest a low stream flow throughout the summer and fall. In the winter there is probably no flow of water in the Jackfish Creek channel.

Aerial photographic interpretation of the surficial geology of the immediate area of the Jackfish Creek Crossing, is shown on Drawing No. A-2, Appendix A. The surficial materials are believed to be mainly glacial lake basin sediments over-ridden with sand dunes on both sides of the flood plan. Peat was observed in depressions and wave modified features were noted. Ice wedge polygons were observed to exist in the immediate vicinity of the Jackfish Creek channel. A terrain legend, which describes the symbols used in the terrain analysis, is presented as Drawing No. A-2a, Appendix A.

3.1 Subsurface Conditions

Based on observations from the boreholes, an inferred stratigraphic section along center line has been compiled and is presented as Drawing No. A-4, Appendix A. The generalized center line stratigraphy noted at the site is summarized in Table 3.2.1, following.

TABLE 3.2.1
STRATIGRAPHY AT JACKFISH CREEK CROSSING

MATERIAL	DESCRIPTION	APPROXIMATE AVERAGE DEPTH BELOW EXISTING GRADE (FT)	RANGE OF THICKNESS (FT)
PEAT & ORGANIC CLAY	dark brown ,silty, frozen, NB to V50%, moisture content (M/C) up to 253%	0 - 1.3	0 - 3
SILT	low to non-plastic, brown to grey, some sand, clayey, M/C 15% to 68% (avg.35%), unfrozen to frozen, NB to V30%.	1.3 - 16.3	2 - 30
SAND, GRAVEL & CLAY TILL	silty, frozen, NB to V10%, M/C 12% to 29% (avg.21%)	Below 16.3 or Greater	Not Established

The following additional information, which may influence design or construction decisions, was also obtained during the field investigation.

1. The maximum depth of borehole penetration was 40 feet.

2. Bedrock was not encountered in any of the boreholes. Also, the depth to thaw stable soil was not confirmed within this investigation.

3. Ground ice was noted in Borehole 721-C-8 between the depths of 3 and 7.5 feet below existing grade.

4. An unfrozen zone was logged in Borehole 721-C-7 between 0.5 and 2.5 feet below existing grade.

5. Sand logged in Borehole 721-C-7, below 2.5 feet from existing grade, is believed to be of similar origin as the major silt strata.

6. The origin of the gravel and clay till noted in Boreholes 721-S-1 and 720-C-2, respectively, may be one and the same.

7. No borehole information is available in the bottom of the creek channel to indicate the type and nature of underlying subsoil materials.

IV. CONCLUSIONS AND RECOMMENDATIONS

It is recommended that consideration be given to the installation of large diameter culverts at the Jackfish Creek Crossing in combination with an earth fill, as an alternate to construction of a bridge. As this consideration is beyond the scope of this report, the following recommendations pertain solely to the development of the crossing as a bridge site.

4.1 Foundation Types

Final selection of a bridge foundation system should be determined in conjunction with economic and structural design considerations. At present, preference is given to pile foundation systems supported on bedrock or other thaw stable materials. Neither material was encountered within this investigation, hence specific foundation types cannot be recommended. However, generalized design criteria is presented for preliminary evaluation of pile foundation schemes.

4.2 Foundation Design

Major factors affecting the design of pile foundations at the Jackfish Creek Crossing are the occurrence of ground ice, a high percentage of visible ice, unfrozen soil and a high natural moisture content at or near the assumed abutment locations of the proposed bridge. Generally, the subsoil beneath the river flood plain and channel, renders pile design, based on soil adfreeze principles, hazardous. Consequently, it is considered that allowable pile bearing capacities must be determined on the basis of available end bearing support, and/or available skin friction support of existing subsoil material in the unfrozen state. In addition, the existence of permafrost is considered to preclude the use of dyanamic pile formulae as a rational approach to the determination of pile capacities. However, placement of piles through pile driving techniques will likely be the most expedient method of installation.

Because of the lack of data with respect to soil strength and depth to a frost stable bearing surface, pile designs presented herin are largely based on empirical data, and must be considered, only preliminary in nature. Confirmation of the design parameters presented herein, through additional field and/or laboratory testing, is considered necessary. It is stressed that the following recommendations are presented without knowledge of final design highway grades, geometrics, or bridge design. Consequently, the recommndations presented may require reconsideration when these factors become known.

4.2.1 End Bearing Piles

It is considered that the only positive method of foundation support, that will permit relatively high loads without excessive settlements at the Jackfish Creek Crossing, is an end bearing pile system achieving support on bedrock or other thaw stable materials existing beneath the site. However, due to equipment limitations, the maximum depth of drill penetration was 40 feet, with bedrock or other thaw stable materials not being encountered.

Based on a review of bedrock geology of the area, it is believed that Middle Ramparts shale, of Middle Devonian Age (4), may be expected at an unknown depth below the approximate abutment locations of the proposed river crossing. It is recommended that consideration be given to the use of steel end bearing piles for bridge foundation support. However, determination of bedrock or thaw stable material depth and properties, at the locations of bridge abutments and piers, is a necessary prerequisite to determination of final design pile capacity.

For preliminary design purposes, it is believed that consideration should be given to the use of closed end pipe piles to provide end bearing support in bedrock. It is recommended that piles with a minimum nominal diameter of 12 inches and a minimum weight of 65 pounds per foot be used. The design length of piles must be confirmed on the basis of additional field drilling.

Installation of pipe piles will require the use of both drilling and pile driving equipment. It is recommnded that the piles be installed in pre-bored holes having a diameter of about 95% of the pile diameter to permit a snug fit. The pile holes should be prebored at least 5 to 10 feet into the bedrock, or thaw stable materials and the piles should be driven to at least the full prebored depth. A minimum driving energy of 24,000 foot pounds is recommended. Steel H-piles are presently believed to be less feasible, as preboring would result in loss of lateral support, and installation without preboring to the anticipated depth is expected to meet with high driving resistance. Confirmation of this, however, could be achieved through the driving of test H-piles at the site.

A preliminary design load capacity of about 170 kips may be used for the foregoing recommended pipe pile section, if the piles can be driven to 'refusal'. It is considered that 'refusal' will constitute a penetration of less than 0.1 inch per blow, measured over the last foot of driving with the recommended pile driving energy. It is recommended that pile driving records be kept for all piles, for immediate review by the geotechnical consultant. A pile load test is also recommended, prior to or at the outset of pile installation, to confirm the load carrying capacity of the piles and permit correlation to the driving records.

4.2.2 Friction Piles

Based on available geotechnical information at the Jackfish Creek Crossing, it is believed that limited probability exists for the successful installation of piles at the site, achieving their load carrying capacity primarily through skin friction between pile and embedding soil. Hence, the present lack of specific information, with respect to the strength of insitu soils in an unfrozen condition, in combination with the above belief that long term satisfactory performance is a remote possibility, precludes the recommending of skin friction piles.

Confirmation of the suitablility of friction piles presentation of more detailed pile designs, and estimates of pile capacities can only be made if additional more detailed geotechnical information of subsurface deposits is obtained at the site.

4.3 Negative Skin Friction

The effect of negative skin friction, on individual piles and pile groups, will be dependent upon the occurrence and magnitude of both consolidation settlement and thaw settlement within the fill surrounding the piles and the natural subgrade soils. At the crossing site, it is considered that all peat, silt, sand, and to a lesser degree, clay till and gravel materials are thaw unstable, and consequently, significant negative skin friction effects can be anticipated in these materials if thawing occurs. Substantial skin friction effects will also be mobilized in any road grade fill surrounding piles if loss of subgrade support occurs. To limit the amount of thawing of the subgrade, the loss of subgrade support,and the magnitude of negative skin friction, fills should be placed during the winter season. In order to further limit potential negative skin friction, due to settlement of the fill itself, it is recommended that fills be placed to final grade and pre-boring and installation of piles be carried out through the fill. The maximum time period possible should be allowed between these two phases of construction.

It is extremely difficult to accurately predict the anticipated total magnitude of negative skin fricition loads on any pile or pile group that may be installed at the subject site. Negative skin friction develops due to the down drag effect of the soil around the pile as it thaws and consolidates. Table 4.3.1 presents suggested values (5) for negative skin friction in typical soils. At the Jackfish Creek Crossing, the thickness of fill placed and method of placement will significantly effect the depth and rate of thaw wherever the soil is presently frozen.

TABLE 4.3.1

NEGATIVE SKIN FRICTION OF UNFROZEN SOIL FOR PILE DESIGN

(After Woodward Lundgren And Associates, 1971) (5)

Description of Soil Categories	Design Negative Skin Friction
Clean sands and gravels with little or no silt or clay. Typically: GW, GP, SW,SP	$P_s = 30d\ (X2 + 2HX)$*
Silty or clayey sand and gravel mixtures with considerable amounts of silt and clay. Typically: GM, SM, GC, SC, SF	700 PSF
Moderately plastic to highly plastic inorganic clays. Typically: CL,CH	800 PSF
Non-plastic to slightly plastic inorganic silts and lean clays. Typically: ML, MH	350 PSF
Organic silts and clays. Typically: OL, OH	150 PSF

* Load developed on that portion of a pile embedded in granular stratum.

P_s = Load developed, lbs.

d = Diameter of pile, ft.

H = Depth of overburden to top of granular stratum, ft.

X = Length of pile embedded in granular stratum, ft.

TABLE 4.4.1

FROZEN SOIL ADFREEZE BOND STRENGTH FOR PILE DESIGN

(After Woodward Lundgren And Associates, 1971) (5)

Design Category	Applicable Criteria		Design Adfreeze Bond Stress, for Frost Heaving Soils[1] (PSF)
	Segregated Ice Condition	Water Content of Soil, %	
I -above average soil-ice condition	No visible ice, (< 1%)	15	5000
		15 - 40	4000
II -average soil-ice condition	Little visible ice, (1 - 10%)	15	4000
		15 - 40	2000
III -below average soil-ice condition	Occasional visible ice, (11 - 20%)	15	2000
		15 - 40	1500
IV -poor soil-ice condition	Some visible ice (21 - 35%)	40	1350
		15 - 40	1350
V -very poor soil ice condition	- Considerable visible ice, (> 35%)	40	900
		Any	700

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[1] Applies only for soils containing 5% or more of silt or clay size particles.

4.5 Subgrade Considerations On Center Line

As indicated in Table 3.2.1, the stratigraphy on center line, on both sides of the creek, is similar. Generally, a peaty organic cover was noticed over a major depth of silt. Estimated visual excess ice contents are generally high near the surface, becoming lower with increasing depth. Moisture contents are high and it is expected that very soft conditions will probably exist in unfrozen soils during the summer season. A winter construction program is, therefore, advocated to limit undesirable disturbance to the sub-grade thermal regime.

Qualitative evaluation of shear strength of the silt and/or clay can be made, from visual observations, ice content estimates, moisture profiles, and classification test results. Based on these factors, it is concluded that on thawing, low shear strength will exist in the peat and silt. Moderate shear strength may be present in the clay till and gravel layers, but insufficient evidence is available to qualify this belief.

A lack of detailed information, with regard to ice contents, and a need for sophisticated testing and detailed computer analyses, make it impossible to accurately predict thaw settlement of fill on frozen materials with excess ice contents. Therefore, only qualitative estimates of thaw settlement can be made at this time. Based on visual estimates of excess ice content, it is believed that a total thaw settlement on center line (approach fill about 10 feet in thickness) of about 0.5 to 3 feet can be expected for winter construction (about 5 feet of subgrade thaw), and 1 to 7 feet for summer construction (about 15 feet of subgrade thaw). This estimate assumes thawing of subgrade soils, but does not take into account normal consolidation settlement of the unfrozen subgrade soils due to the surcharge effects of the road bed fill. In the case of peat soils, normal consolidation settlement can easily reach 50 percent of the original thickness of the deposit and can, as with thaw settlement, occur fairly rapidly.

It is considered that the conventional northern construction practice of placing fill material directly on the organic subgrade is desirable at this site. Fills for bridge approaches should be constructed with allowance being made for the occurrence of thaw subsidence, if sufficient thickness of fill is not placed to preserve the frozen subgrade. Allowance for expected subsidence can be made by either providing extra fill to compensate for the anticipated settlement, or to upgrade as subsidence occurs, or both. A 10 foot thickness of granular fill material (non-plastic) is considered to be the minimum depth for road grade construction on underlying frozen subgrade materials at the Jackfish Creek Crossing. Local fine grained materials, such as silt and sand, are not considered suitable for abutment or approach fills. The thickness of road grade material required to prevent degradation of the permafrost can only be predicted after detailed theoretical analysis, which is considered to be beyond the scope of this investigation. It is believed that fill placement should be carried out during the late winter period to minimize thermal disturbance, and possible damage to the existing ground cover and slopes by the construction equipment. Snow clearing should be carried out prior to all fill placement. Placement of the fill should be undertaken by end dumping with subsequent spreading by dozing equipment. A minimum initial lift thickness of 2 feet is suggested. Depending on construction completion schedules, placement of fills may be staged for several seasons or carried to completion as construction progresses.

It was not possible to drill through the ice into the creek bed. Therefore the extent and characteristics of the creek bed deposits could not be determined.

It was noted at the time of drilling (December, 1972 and January, 1973) that the creek was covered with ice and no water was flowing under the ice cover. It is believed that water does not flow throughout the winter in the the Jackfish Creek channel.

4.6 Slope Stability Considerations

Negligible slopes exist within the vicinity of Jackfish Creek, hence the placement of road bed fill is not expected to involve slope stability considerations. In any event, it is recommended that excessive fill thicknesses be avoided near the crest of slopes. In addition, cutting or excavating of slope material is not recommended and desired grades should be achieved solely through the placement of fill.

It is considered that rip-rap protection of the existing defined creek channel, upstream and downstream of the bridge crossing, may be necessary to protect the embankment fill at the crossing site. Bridge abutments should be set as far back from the channel banks as is practicable. Fine grained fills should not be used for subgrade construction on the flood plain as they are easily eroded.

4.7 Drainage Considerations

Approach fills will concentrate runoff water along the upslope sides of fills. Therefore, it is considered essential that significant effort and care be taken to minimize erosion on the slope parallel to the fill. Every effort should be made to preserve the vegetal lining of all designed water courses and wherever this is impossible, coarse gravel should be used as channel lining. Transverse flow breakers should be provided at frequent intervals to reduce the rate of runoff along the fill and thereby reduce the potential for erosion by running water. Spacing of flow breakers will become apparent in the field when drainage courses and gradients become accurately defined. Ponding of water adjacent to fills should be discouraged as ponded water will act as a heat source for rapid degradation of permafrost. It will also tend to reduce the shear strength of the subgrade soil and road grade fill, unless the road grade is very granular.

4.8 Cement Type and Corrosion Considerations

No representative samples from the crossing area were tested to determine the soluble sulphate concentration and soil acidity. However, it is recommended that the use of Type V Sulphate Resistant Cement be considered, for preliminary design purposes, for all concrete in contact with the soil , until further test results are available. Confirmation soil sulphate analyses can be performed prior to construction. A minimum '28 day' compressive strength of 3000 pounds per square inch is recommended for all concrete forming foundation elements.

For steel piles, extending above the groundwater level, corrosion protection may be achieved by painting or encasement with concrete. In this instance, the protective coating should extend to a minimum distance of 2 feet below final grade or minimum anticipated low water level, whichever is deeper. In the case of pipe piles, protective coating should be provided on the interior of the pipes to prevent possible corrosion. If practical, this may be achieved through filling of the piles with concrete.

4.9 Additional Studies

The consideration of utilizing large diameter culverts rather than undertaking the construction of a bridge is beleived of prime importance. In order to more accurately assess such factors as insitu shear strength, bedrock depth or depth to thaw stable materials and thaw subsidence, it is desirable to obtain additional detailed geotechnical information at the site. Such items as acquisition of representative undisturbed samples of the various soil types, refined field and laboratory tests to determine shear strength and thaw subsidence factors, and a refined theoretical analysis of these factors, constitute the additional detailed geotechnical information that is considered to be desirable.

In addition to the desirability of obtaining further detailed geotechnical information, it is recommended that consideration be given to establishment of closely superivsed and documented pile driving and pile load tests. Although preferable, these tests need not be carried out at actual bridge crossing sites, but may be carried out in areas and materials that would be representative of general foundation conditions at most of the proposed bridge sites. Such tests would provide valuable design data on which future designs of pile foundation systems could be established.

V. LIMITATIONS

The foregoing recommendations have been prepared based on our knowledge of existing conditons at Jackfish Creek and the proposed highway crossing. This knowledge has been derived from visual, physical and analytical considerations of existing soil and slope conditions, which were obtained from our field investigation. The findings and comments presented are believed to accurately reflect conditions as they are known to exist.

Due to the general nature of the study, reported herein, the findings cannot be considered to be a comprehensive assessment of slope and foundation conditions at the crossing. Should conditions be encountered other than described herein, the geotechnical consultant should be contacted so that recommendations may be evaluated in light of new findings.

Respectfully Submitted,

EBA Engineering Consultants Ltd.

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GRG/tmf

LIST OF REFERENCES

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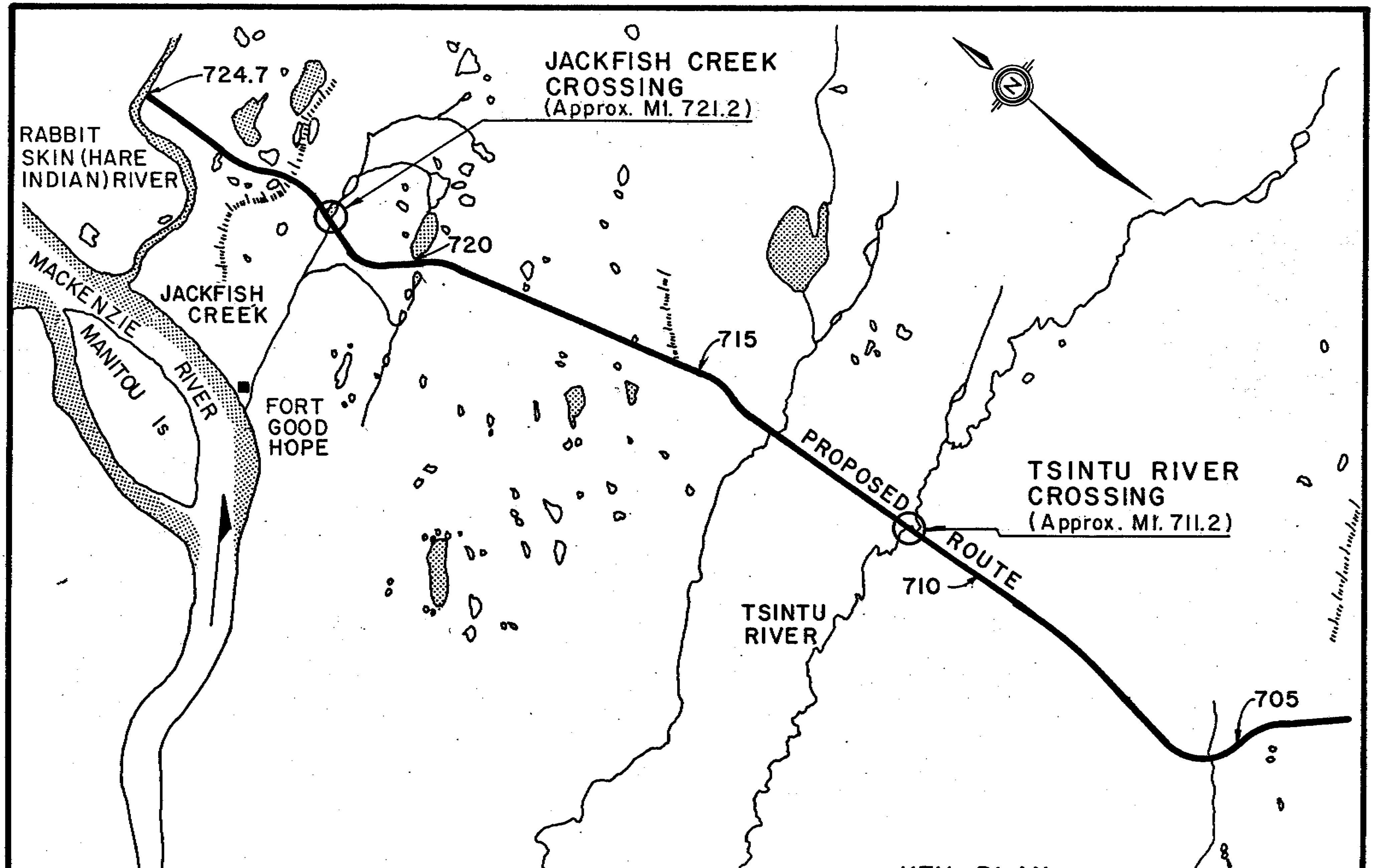
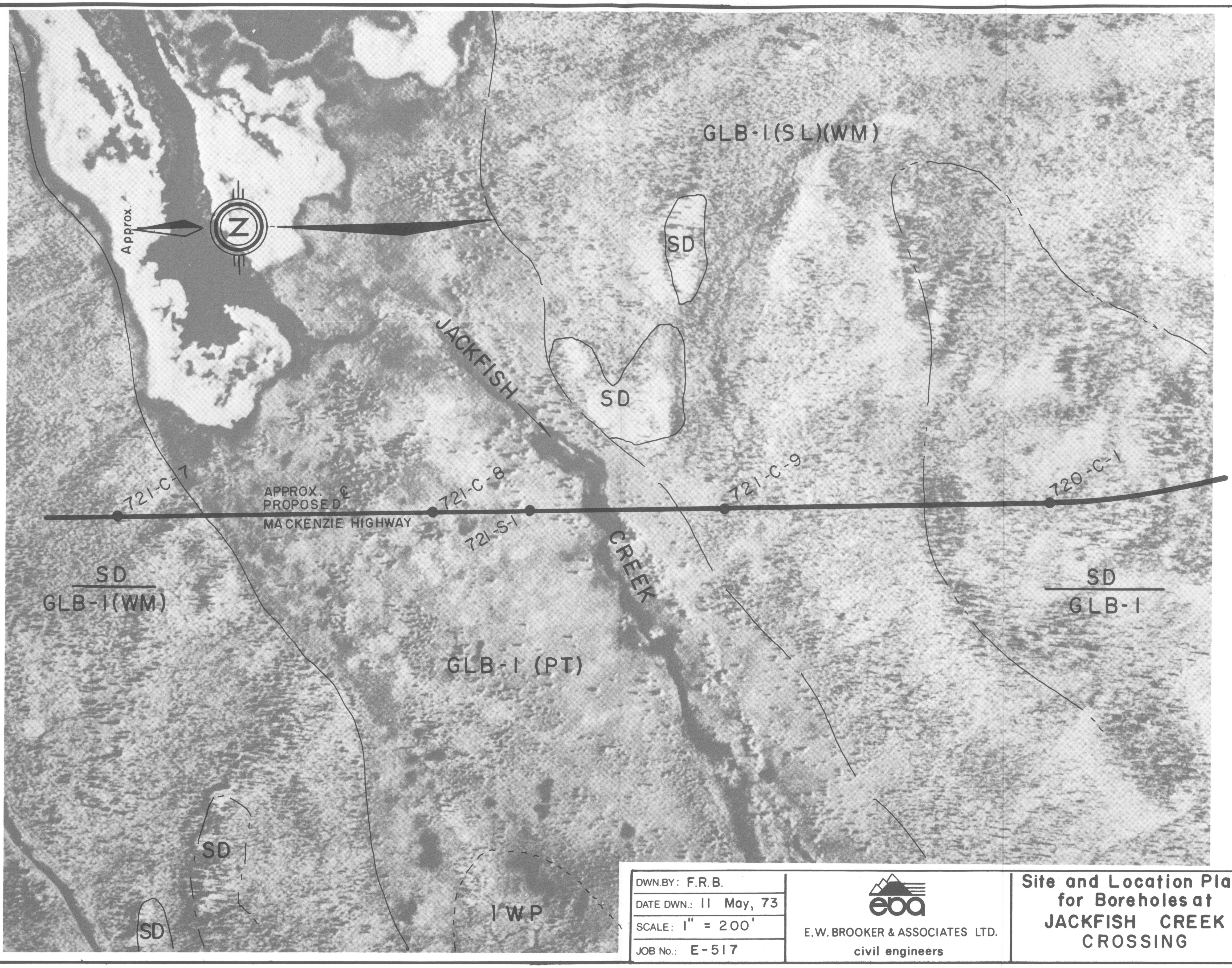


KEY PLAN

EBA Engineering Consultants Ltd.	DATE 2 JAN. 74	SCALE 1" = 2 mi	JOB No. E - 517	DWG No. A - 1



DWN.BY: F.R.B.
DATE DWN.: 11 May, 73
SCALE: 1" = 200'
JOB No.: E-517

eba
E.W. BROOKER & ASSOCIATES LTD.
civil engineers

Site and Location Plan for Boreholes at JACKFISH CREEK CROSSING

DWG.: A – 2
SHT.No.:

TERRAIN LEGEND

Symbol	Terrain Type	Physiographic Features	Materials Description
GLB-1	Glacial Lake Basin (Better drained type).	Lowland occasionally swampy areas.	Ice-rich to medium plastic silty clay, occasionally with a trace of sand.
SD	Sand Dunes	Elongate Ridges to Barchans.	Fine to medium sand.

Surficial Features

IWP	Ice Wedge Polygons, usually in ice-rich silt.

Topstratum Phases (Associated with Terrain Types)

PT	Mixed bog and fen peats in post glacial ponded depression.
WM	Wave modified, mainly a thin sandy to gravelly washed layer over till.

Complexes are shown as combinations of two terrain types or without phases that pertain to the parent type.

Terrain Symbols are modified from Canadian Gas Arctic Study Limited Terrain Study for this area.

Drawing No. A-2a

eba Engineering Consultants Ltd.

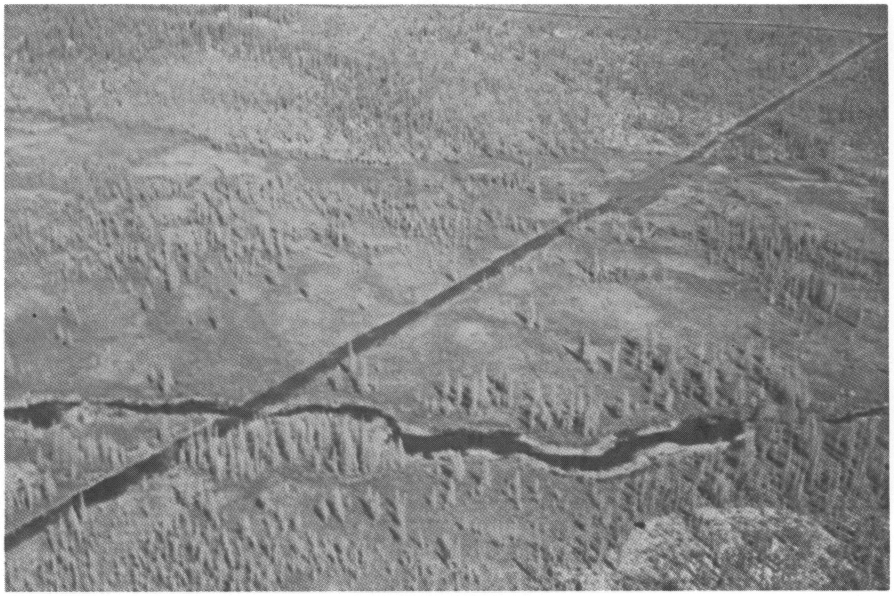
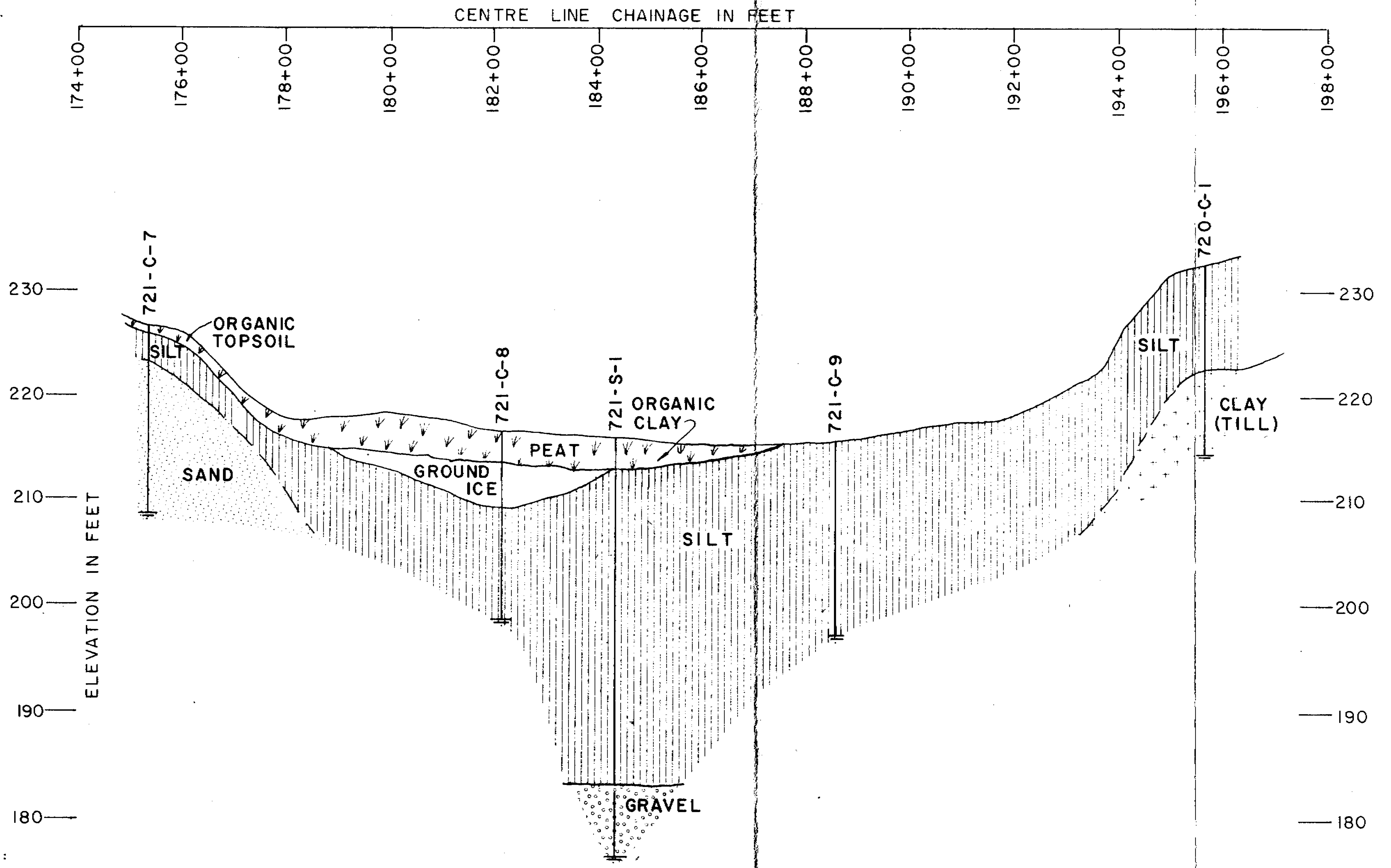


PLATE No. 1

Jackfish Creek Crossing. The cutline is the proposed route for the highway. Water ponded on the cutline in the creek area could do severe damage to the permafrost. North is to the upper, right corner of the plate. Jackfish Lake Road to Fort Good Hope cuts accross the top of the plate. (June, 1973)

Drawing A-3



NOTE:

1. Stratigraphy between boreholes has been assumed.
2. Elevations and profile taken from C.E.S. profile plan of Proposed ℄ for Mackenzie Highway.
3. Scales: Horiz. 1" = 200'
   Vert. 1" = 10'

DWN BY: F.R.B.
DATE DWN: MAR./74
SCALE: As Noted
JOB No: E-517

eba Engineering Consultants Ltd.

JACKFISH CREEK CROSSING
℄ PROFILE & STRATIGRAPHY

DWG: A-4
SHT. No.:

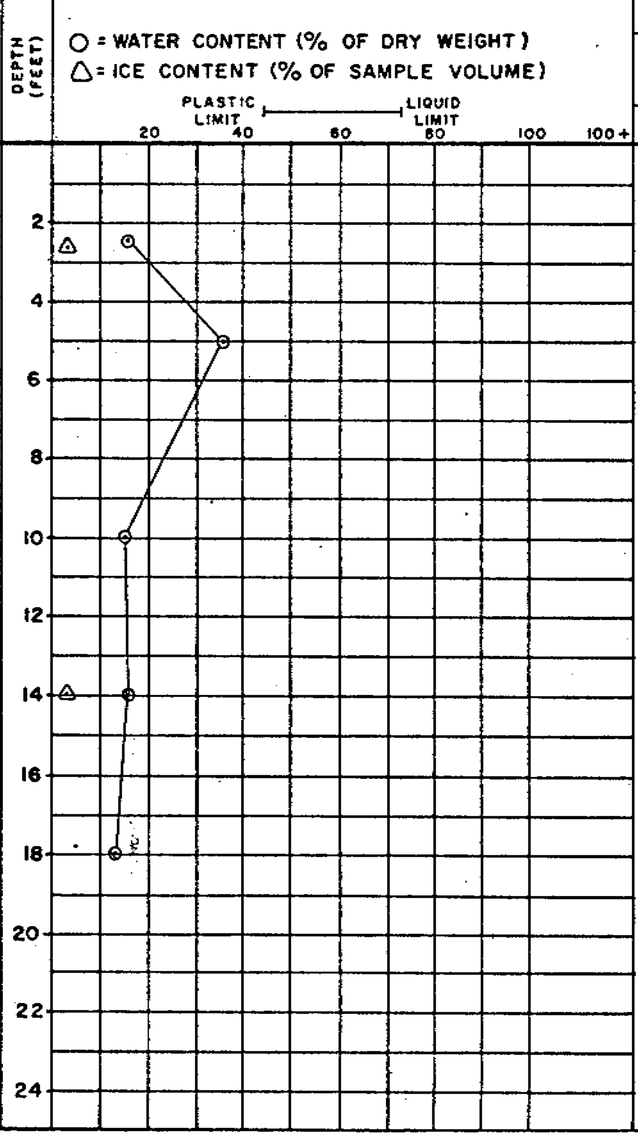
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN: ALB | FIELD ENG: LAB | DATE DRILLED 12/9/72 | AIRPHOTO NO: A22858-54 | CHAINAGE: 195 + 79 | OFFSET

CKD: GRG | TECH: TJ | RIG: Mayhew 500 | SURFACE DRAINAGE: Good To North | VEGETATION: Sparse Spruce 15-20' | ELEV: 232.9'

TEST HOLE		C35
MILE	B,C,S	NUMBER
720	C	1

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					ML or SF	SILT - Grey-Brown, Fine, Sandy or Silty Fine Sand, Non-Plastic	F	
2	1	⧄						
4	2	⧄						
6								
8								
10	3	⧄				CLAY (TILL) Grey-Brown, Silty, Gravelly		NB 3-5%
12					CL			
14	4	⧄						
16						-Ice Crystals @ 16'		
18	5	⧄				END OF HOLE 18'		
20								
22								
24								



GRAIN-SIZE ANALYSIS				WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
CLAY	SILT	SAND	GRAVEL			
%	%	%	%			
						150' N. of E.C.

E.W. BROOKER & ASSOCIATES LTD.

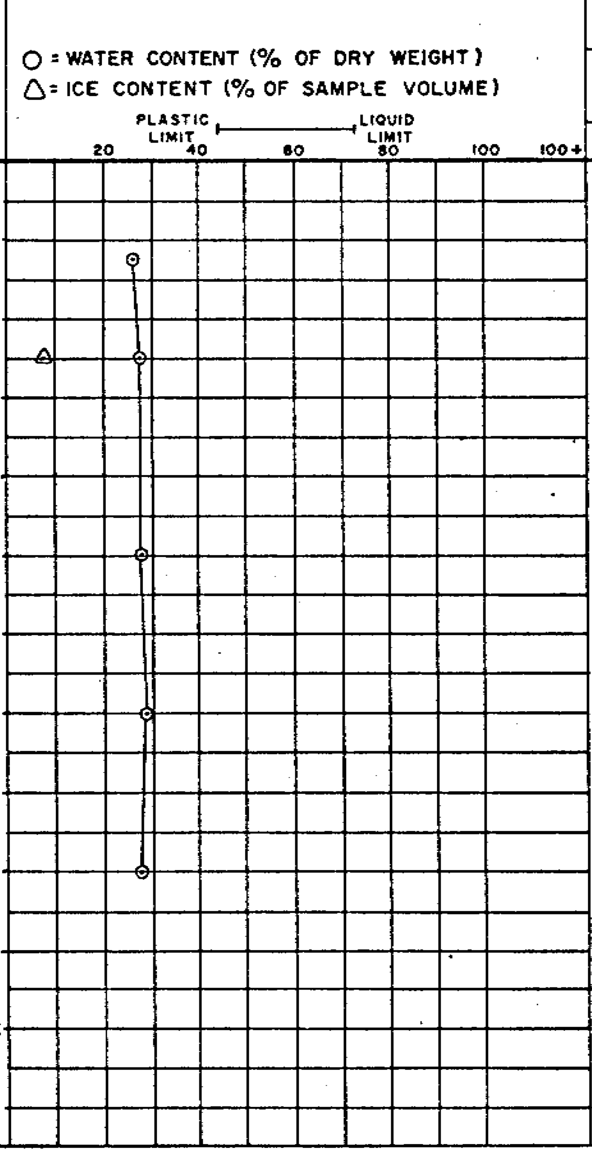
DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB
FIELD ENG: LAB
DATE DRILLED 12/9/72
AIRPHOTO NO: A22858-54
CHAINAGE: 175 + 30
OFFSET
CKD: GRG
TECH: TJ
RIG Mayhew 500
SURFACE DRAINAGE: Good To South
VEGETATION: Fairly Thick Spruce 8-15'
ELEV: 226.7'

TEST HOLE		C32
MILE	B,C,S	NUMBER
721	C	7

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					Pt	Organic Topsoil	F	NB
2	1				ML	SILT - Med. Brown, Fine, Sandy, Non-Plastic	U	Unfrozen
4	2				SF	SAND - Dk. Brown, Fine, Silty	F	V 5-10%
10	3				SP	- Fine To Med. Grained		
14	4							
18	5				SP	- Dk. Brown, Med. Grained Sand, Trace Of Coal		
						END OF HOLE 18'		



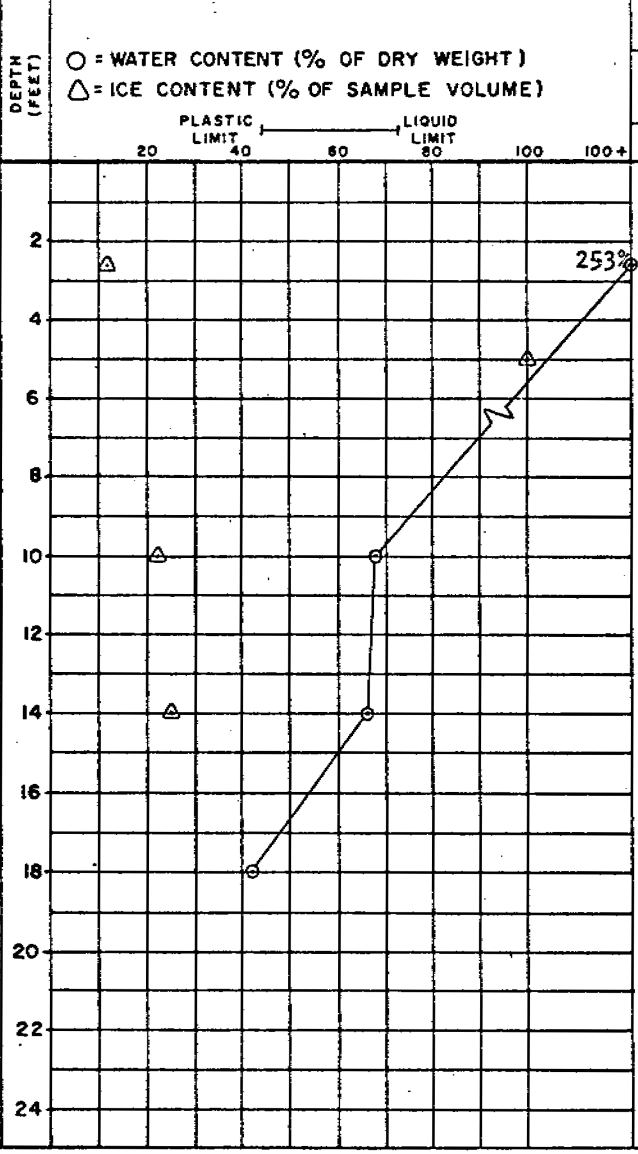
GRAIN-SIZE ANALYSIS				WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
CLAY	SILT	SAND	GRAVEL			
%	%	%	%			
						300' N. of First Camp Setup.

E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY**

DWN: ALB	FIELD ENG: LAB	DATE DRILLED 12/9/72	AIRPHOTO NO: A22858-54	CHAINAGE: 182 + 17	OFFSET
CKD: GRG	TECH: TJ	RIG: Mayhew 500	SURFACE DRAINAGE: Good To North-West	VEGETATION: Very Sparse Tamarack	ELEV: 216.5'

TEST HOLE		C33
MILE	B,C,S	NUMBER
721	C	8

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
2	1				Pt	PEAT - Frozen, Clayey & Silty, Organic, Dark Brown	F	V 10-15%
4	2					GROUND ICE - Trace of Soil		Ice
6								
8						SILT - Med. Brown, Clayey, Some Sand, NonPlastic		V 20-25%
10	3				ML			
12						- Grey, Clayey, Sandy, Low Plastic		V 20-30%
14	4							
16						- (Ice Lenses 16'-18')		
18	5					END OF HOLE 18'		
20								
22								
24								



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

REMARKS

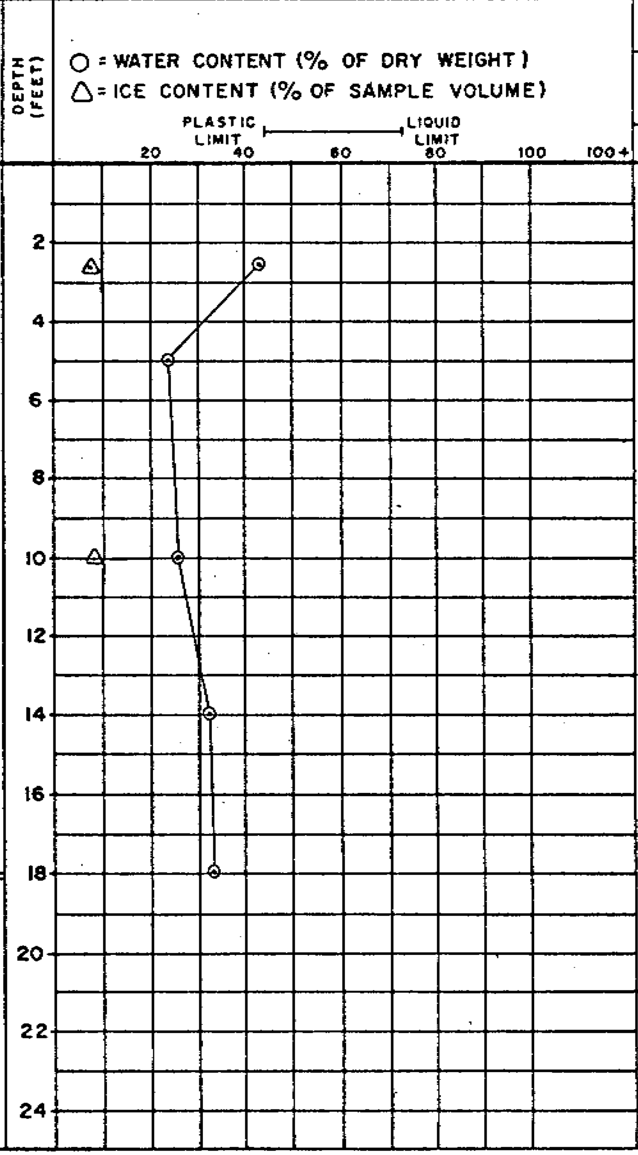
300' S. of First Camp Setup. In Former Lakebed.

E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: LAB	DATE DRILLED 12/9/72	AIRPHOTO NO: A22858-54	CHAINAGE: 188 + 59	OFFSET		
CKD: GRG	TECH: TJ	RIG: Mayhew 500	SURFACE DRAINAGE: Good To . North	VEGETATION: Sparse Tamarack & Spruce	ELEV: 215.7'		

TEST HOLE		C34
MILE	B,C,S	NUMBE
721	C	9

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 (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
						SILT	F								Just South of Former Lakebed.
2	1					- Lt. Brown, Fine, Sandy, Trace of Clay, Non-Plastic									
4	2														
6					ML	- Grey,		NB -V 5-10%							
10	3														
14	4														
18	5					END OF HOLE 18'									



E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB
CKD: GRG
FIELD ENG: NRM
TECH: DY
DATE DRILLED: 1/21/73
RIG: Mayhew 500
AIRPHOTO NO:
SURFACE DRAINAGE: Well Drained
CHAINAGE: 184 + 36
VEGETATION: Willow
OFFSET
ELEV: 215.9'

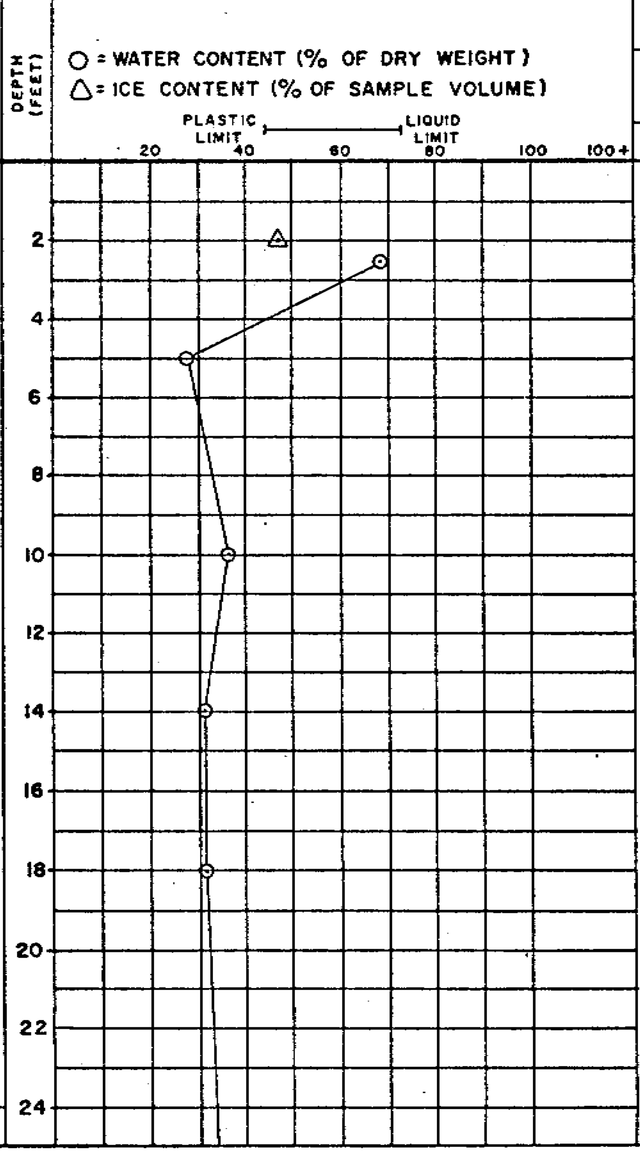
TEST HOLE MVPL4

MILE	B,C,S	NUMBER
721	S	1

REMARKS

Sheet 1 of 2

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					OH	CLAY - Organic	F	V 45 - 50%
2	1	⧅						
4					ML	SILT - Med. Brown, Sandy		NB
	2	⧅						
6								
8								
10	3	⧅						
12					ML	- Grey, Sandy, Fine Grained		
14	4	⧅						
16								
18	5	⧅						
20								
22								
24								



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

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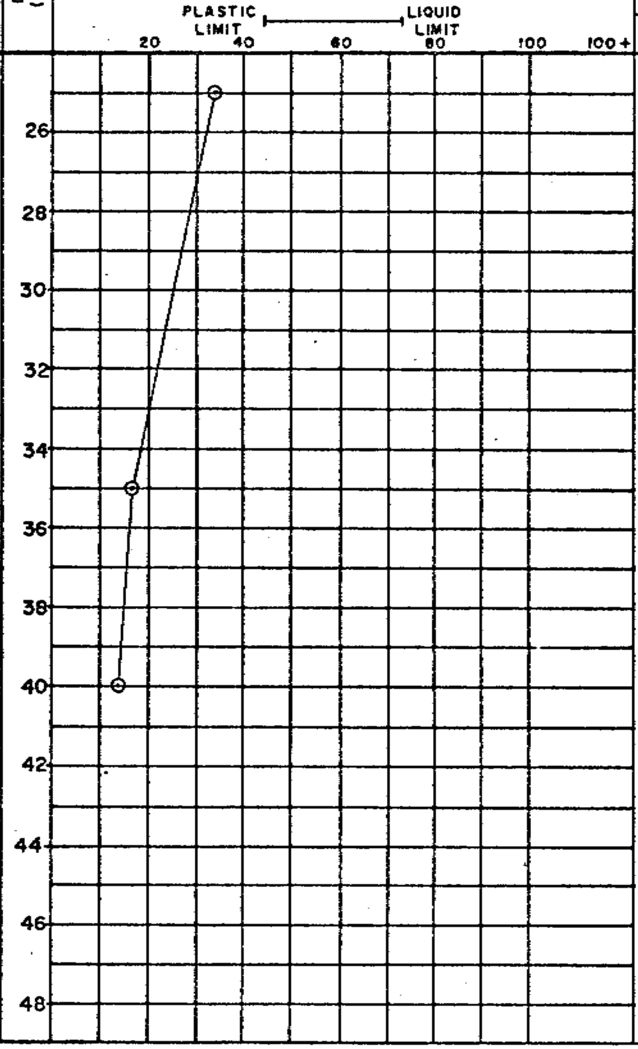
DWN: ALB	FIELD ENG: NRM	DATE DRILLED: 1/21/73	AIRPHOTO NO:	CHAINAGE: 184 + 36
CKD GRG	TECH: DY	RIG: Mayhew 500	SURFACE DRAINAGE: Well Drained	VEGETATION: Willow
OFFSET	ELEV: 215.9'			

TEST HOLE MVPL4

MILE	B,C,S	NUMBER
721	S	1

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
	6	◪			ML	SILT - Grey, Sandy, Fine Grained	F	NB							Sheet 2 of 2
					GW	GRAVEL - Grey, Sandy									
	7	◪													
40	8	◪													
						END OF HOLE 40'									

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



SUMMARY OF TEST RESULTS

JOB No. E - 517

JACKFISH CREEK CROSSING

MILE 721.2

BORE HOLE	DEPTH	NATURAL WATER CONTENT	Atterberg Limits			MECHANICAL ANALYSIS (M.I.T. CLASSIFICATION)				SOIL CLASSIFICATION (UNIFIED)	REMARKS
			W_L	W_P	PI	% CLAY	% SILT	% SAND	% GRAVEL		
	feet	%	%	%	%						
721-C-8	10										Non-plastic

DWG. No. C-1

EBA ENGINEERING CONSULTANTS LTD.