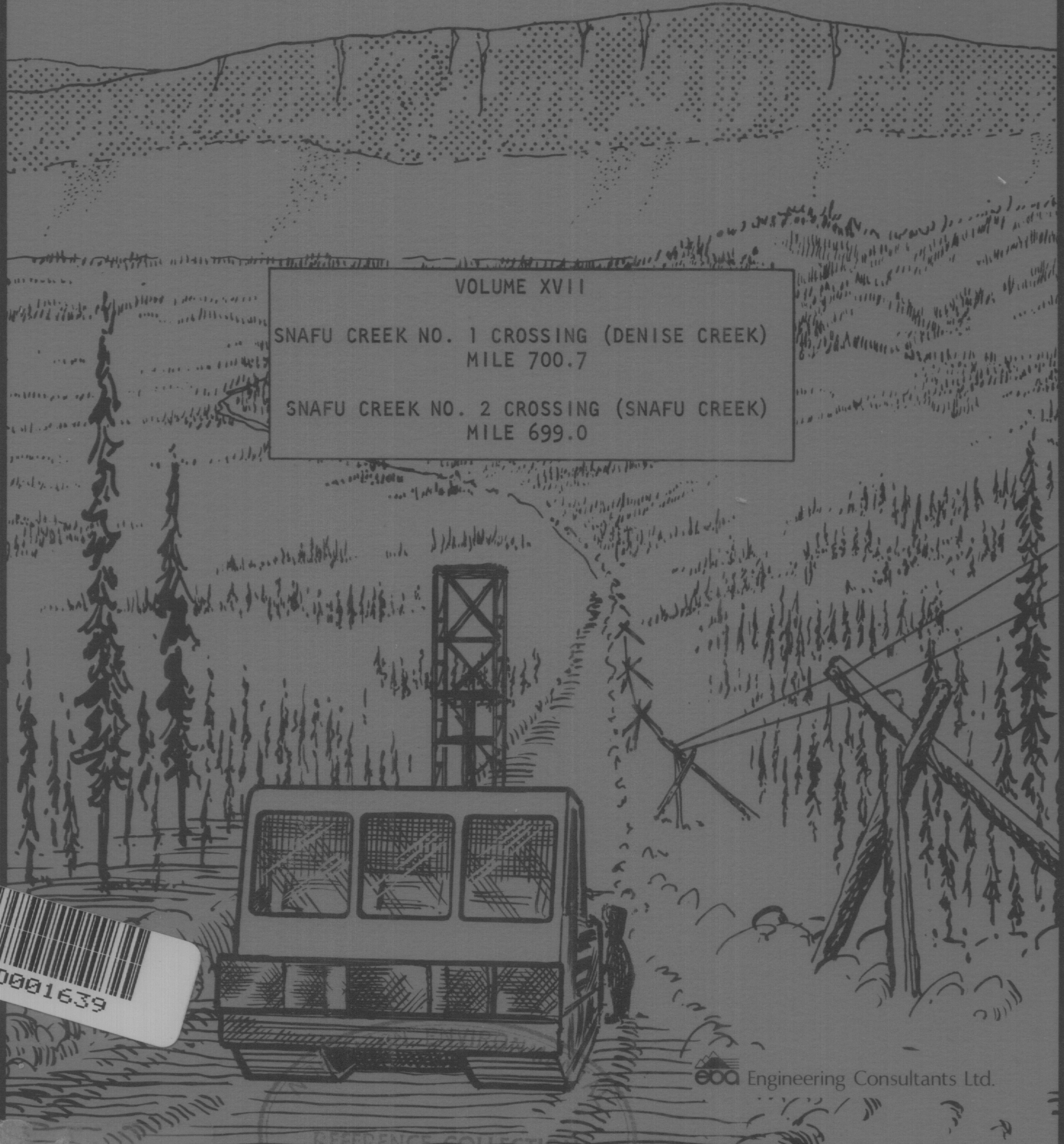
MACKENZIE HIGHWAY
GEOTECHNICAL EVALUATION



VOLUME XVII

SNAFU CREEK NO. 1 CROSSING (DENISE CREEK)
MILE 700.7

SNAFU CREEK NO. 2 CROSSING (SNAFU CREEK)
MILE 699.0

eba Engineering Consultants Ltd.

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GEOTECHNICAL EVALUATION
VOLUME XVII
SNAFU CREEK NO. 1 CROSSING (DENISE CREEK)
SNAFU CREEK NO. 2 CROSSING (SNAFU CREEK)

MACKENZIE HIGHWAY

Submitted To:

GOVERNMENT OF CANADA
DEPARTMENT OF PUBLIC WORKS
CONTRACT NUMBER A10/73
FILE NUMBER 9305-52-0-307

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. Snafu Creek No. 1 Crossing (Denise Creek) and Snafu Creek No. 2 Crossing (Snafu Creek), whose geographic locations are shown on the Key Plan, Drawing No. A-1, Appendix A, are two such sites investigated in detail. Until recently, the actual names of the creeks involved were unknown to us, hence, the crossings were arbitrarily designated as Snafu Creek No.1 Crossing and Snafu Creek No.2 Crossing. This designation system is maintained throughout this report as all drawings and plans were completed to final prior to learning of the actual creek names. Details of the investigation, site conditions, geotechnical data and recommendations pertinent to the development of the creek crossings, 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 AQUISTION

2.1 Field Testing

The evaluation of subsurface conditions has been based on field data obtained from eight boreholes, drilled at the locations shown on Drawings No. A-2 and A-3, Appendix A. Of the eight boreholes advanced, three were drilled at Snafu Creek No. 1 Crossing and the remainder at Snafu Creek No. 2 Crossing. Seven of the eight boreholes drilled were drilled as center line boreholes, in conjunction with the general route evaluation, while the remaining borehole was located and drilled specifically to define subsurface conditions at Snafu Creek No.1 Crossing.

The center line boreholes consisted of Boreholes 700-C-2 and 700-C-3, 699-C-7 to 699-C-10, inclusive and 698-C-1. The single special borehole was designated Borehole 700-S-1. Detailed borehole logs are presented in consecutive order in Appendix B.

Three center line boreholes (700-C-2, 700-C-3 and 699-C-7) were drilled with a Texoma Super Economatic power auger, fitted with a 12 inch diameter stub auger. All other center line boreholes and the special borehole 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 38 feet, and averaged 20.5 feet in depth. Sampling consisted of representative bag samples, obtained at depths of 2.5 and 5 feet, and at depth intervals of about 5 feet, thereafter, to the bottom of each borehole. Undisturbed samples were not obtained at this site.

2.2 Laboratory Testing

Laboratory testing was carried out on the 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 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 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 Drawing No. C-1, Appendix C, presents a partial summary of laboratory results.

III. SITE CONDITIONS

3.1 Surface Features and Geology

The proposed Mackenzie Highway crosses Snafu Creek No.1 Crossing and Snafu Creek No.2 Crossing at Miles 700.7 and 699.0, respectively, approximately 20 miles south-east of Fort Good Hope, NWT. Drawing A-1, Appendix A, is a Key Plan of the Snafu Lake area and Drawings No. A-2, and A-3, Appendix A, present detailed Site Plans of Snafu Creek No.1 and No. 2, Crossings, respectively. Plates No.1, and No. 2, Drawing No. A-4, Appendix A, show Crossings No.1 and No. 2, respectively, from the air, in June 1973.

The creeks under discussion drain a relatively large area extending east of Snafu Lake (Sucker Lake). The Snafu Lake watershed lies south of the Tsintu River and north of the Chick Lake Drainage basins. The terrain within the Snafu Lake watershed is relatively flat. The summer flow into Snafu Lake is expected to be relatively small and little or no flow is expected during the winter months. Water was observed at Snafu Creek No.1 Crossing in February, 1973.

Aerial photographic interpretation of the surficial geology of the immediate area of the Snafu Creek Crossings, is shown on Drawings No. A-2, and A-3, Appendix A. The surficial materials in the vicinity of Snafu Creek No.1 Crossing are believed to be ridge-and-knoll moraines, ground moraine and glacial lake basin materials with alluvial meander plain deposits along the creek channel. Ground moraine is prevalent near Snafu Creek No.2 Crossing, with lesser amounts of glacial lake basin materials. Surface materials have been modified to some degree by slopewash and wave action, as well as the depostion of peaty soil in depressions. A terrain legend, which describes the symbols used in the terrain analysis, is presented as Drawing No. A-3a, Appendix A.

3.2 Subsurface Conditions

Based on observations from the boreholes, stratigraphic sections along both center lines have been compiled and are presented as Drawings No. A-5 and A-6, Appendix A. The generalized center line stratigraphy noted at each site is summarized in Table 3.2.1, and Table 3.2.2, following.

TABLE 3.2.1
STRATIGRAPHY AT SNAFU CREEK NO.1 CROSSING, MILE 700.7

MATERIAL	DESCRIPTION	APPROXIMATE DEPTH BELOW EXISTING GRADE (FT)	RANGE OF THICKNESS (FT)
PEAT	grey to black, some silt and sand, few pebbles, NB	0 - 2	1 - 3
SILT & CLAY (TILL)	brown, low plasticity, gravelly, moist, firm, moistre content (M/C) 9% to 16% (avg. 12%) unfrozen to frozen, 0-V5%	2 - 16	4 - Greater Than 21
SHALE	silt & clay, grey, moist, low plasticity, M/C 10% to 12% (avg. 11%)	Below 16	Not Established

The following additional information, which may influence design or construction decisions, was also obtained during the field investigation at Snafu Creek No.1 Crossing.

1. The maximum depth of borehole penetration was 38 feet.

2. Unfrozen silt and clay till, and shale was noted in Boreholes 700-S-1 and 700-C-2, below depths of 10 and 6 feet, respectively.

3. A gravel layer was noted in Borehole 700-C-2 between the depths of 3 feet and 6 feet.

4. Borehole 700-C-3 was terminated prior to encountering bedrock (end of borehole, 18 feet).

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

STRATIGRAPHY AT SNAFU CREEK NO. 2 CROSSING, MILE 699.0

MATERIAL	DESCRIPTION	APPROXIMATE DEPTH BELOW EXISTING GRADE (FT)	RANGE THICKNESS (FT)
PEAT	dark brown, fibrous, NB to V20%	0 - 1	1
CLAY & SILT (TILL)	low plasticity, medium to dark brown to grey, some sand and gravel, moisture content (M/C) 10% to 79% (avg. 22%) unfrozen to frozen, 0% - V30%	Below 1	Not Established

The following addtional information, which may influence design or construction decisions, was also obtained during the field investigation at Snafu Creek No. 2 Crossing.

1. All boreholes were drilled to a depth of 18 feet.

2. Unfrozen clay-silt till was noted at the following depths in the specified boreholes.

Borehole No.	Unfrozen Below (FT.)
699-C-8	10
699-C-9	1
699-C-10	11
698-C-1	1

3. Bedrock was not encountered in any of the boreholes prior to termination of drilling.

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

5. The average moisture content below a depth of about 10 feet from grade is low (about 11%), hence, where frozen, this zone is expected to be thaw stable.

IV. CONCLUSIONS AND RECOMMENDATIONS

4.1 Foundation Types

At present, preference is given to pile foundation systems supported on bedrock or other thaw stable materials. However, final selection of a foundation system should be determined in conjuction with economic and structural design considerations, as well as further detailed geotechnical analyses. The following foundation types are believed to be feasible for bridge structures at the site.

1. Driven steel H-piles or open end pipe piles.
2. Closed end pipe piles driven in pre-bored holes.

4.2 Foundation Design

A major factor affecting the design of pile foundations at the Snafu Crossings is the noted occurrence of unfrozen zones within the subsoil. Although frozen soil was logged in the vicinity of bridge construction, the possibility of unfrozen 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 capacitites 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 frozen zones is considered to preclude the use of dynamic 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 a lack of data, with respect to soil strength, pile designs presented herein 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 labroatory testing is considered necessary.

The recommended foundation types listed in Sub-section 4.1, may be designed in accordance with the following preliminary design parameters. However, it is stressed that the following recommendations are presented without knowledge of final design highway grades, geometrics, or bridge design. Consequently, the recommendations presented may require reconsideration when these factors become known.

4.2.1 Friction Piles

Based on available geotechnical information at the Snafu Creek Crossings, it is believed that a significant probability exists for the successful installation of piles at the sites, achieving their load carrying capacity primarily through skin friction between pile and embedding soil. However, the present lack of specific information, with respect to the strength of insitu soils in an unfrozen condition,permits only a preliminary estimate of the load carrying capacity of friction pile types.

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

The following pile design parameters may be used for preliminary design and estimating purposes, with the final design to be confirmed on the basis of field installation records and load testing.

a. Driven Steel H-Piles

As a guide to the establishment of a preliminary pile design, it is recommended that standard H-piles 70 feet in length (about 10 feet of fill assumed at abutments, and the upper 10 feet of peat and till is thaw unstable) with a minimum weight of 53 pounds per foot (CBP124), be considered for preliminary design purposes. It is believed that the suggested pile section can be driven, with an energy of 24,000 foot pounds to the full length of the pile. It is believed that piles driven to these specifications will permit allowable static design loads of 120 kips and 80 kips to be used at Snafu Creek No.1 Crossing and Snafu Creek No.2 Crossing, respectively. Although preboring through permanently frozen ground at this site, is not considered necessary for the installation of steel H-piles, it may be necessary in hard seasonally frozen ground and thick granular fills to ensure the alignment of the driven pile section. This will be particularly true if very long sections are to be driven.

It is essential that the bridge approach fill be placed to final grade, before preboring and pile driving, in order to prevent damage to the piles and to ensure working room for proper compaction of the fill. This sequence of construction will limit negative skin friction load on the piles. On site inspection and supervision of the driving of test piles, or the initial piles of the foundation system, is considered absolutely necessary in order to establish the final design bearing capacity. It is also considered essential that a pile driving record be maintained for all piles. The driving record of all piles should be reviewed by the geotechnical consultant, as is practical, to ensure the design intention is being realized.

If penetration refusal (penetration less than 0.1 inch per blow) is met, utilizing the above driving criteria, the piles may be considered to function as end bearing foundation elements, and the recommendations stated in Sub-section 4.2.2 become applicable and supersede the recommendations stated in Sub-section 4.2.1.

b. Prebored Driven Closed End Steel Pipe Piles

Closed end steel pipe piles, installed in prebored holes, may also be considered for foundation support. The prebored hole size should be 85 to 90 percent of the outside pile diameter to ensure a 'snug' fit, and should extend the full length of the intended pile penetration. It is essential that the bridge approach fill be placed to final grade before pre-boring and pile driving, in order to prevent damage to the piles and ensure working room for proper compaction of the fill. This sequence of construction will limit negative skin friction loads on the piles.

For preliminary design purposes, it is recommended that closed end pipe piles with a minimum length of 70 feet (about 10 feet of fill assumed at abutments and the upper 10 feet of peat and till is thaw unstable) and a minimum weight of 40 pounds per foot be considered. The suggested pile section should be driven, with an energy of 24,000 foot pounds to the full length of the pile. It is believed that piles driven to these specifications will permit allowable static design loads of 80 kips and 55 kips to be used at Snafu Creek No. 1 Crossing and Snafu Creek No. 2 Crossing, respectively. Driven piles must penetrate to at least the full pre-bored depth. As for steel H-piles, inspection of the driving of test piles, or the first few piles of the foundation system, is considered absolutely necessary to confirm or alter the design bearing capacity. It is also considered essential that a driving record be maintained for all piles for immediate review by the geotechnical consultant.

If penetration refusal (penetration less than 0.1 inch per blow) is met, utilizing the above driving criteria, the piles may be considered to function as end bearing foundation elements, and the recommendations stated in Sub-section 4.2.2 become applicable and supersede the recommendations stated in Sub-section 4.2.1.

4.2.2 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 Snafu Creek Crossings, is an end bearing pile system achieveing support on bedrock existing beneath the site. Bedrock (shale) was encountered at an average depth of 16 feet at Snafu Creek No.1 Crossing. However, bedrock was not encountered at Snafu Creek No.2 Crossing in any of the boreholes (maximum depth of drill penetration, 18 feet).

It is recommended that consideration be given to the use of steel end bearing piles for bridge foundation support. However, determination of bedrock properties at Snafu Creek No. 1 Crossing, and bedrock depth and properties at Snafu Creek No.2 Crossing, near the location of bridge abutments and piers, is a necessary prerequisite to the determination of a 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 the piles must be confirmed on the basis of additional field drilling. However, pile lengths of about 25 to 50 feet and between 50 to 100 feet are beleived necessary for Snafu Creek No.1 Crossing and Snafu Creek No.2 Crossing, respectively.

Installation of pipe piles will require the use of both drilling and pile driving equipment. It is recommended 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 pre-bored at least 5 to 10 feet into the bedrock 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 (12 inch by 12 inch by 53 pounds per foot, minimum) are also presently believed to be feasible as preboring would probably not be required. 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 or H-pile sections, if the piles can be driven to 'refusal' in bedrock. 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 intstallation, to confirm the load carrying capacity of the piles and permit correlation to the driving records.

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 the near surface peat and clay-silt till layers are thaw unstable, but all materials below an average depth of about 10 feet from existing ground surface, are thaw stable. Consequently, moderate negative skin friction effects can be anticipated on foundation elements within 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 interval, which is consistent with the construction schedule, should be allowed between these two phases of construction.

It is extremely difficult to accurately predict the anticipated total magnitude of negative skin friction loads on any pile or pile group that may be installed at the subject site. Negative skin friction develops due to the downdrag effect of the soil as it thaws and consolidates around the pile. Table 4.3.1 (4) presents suggested values for negative skin friction in typical soils. At the Snafu Creek Crossings the thickness of fill placed and method of placement will significantly effect the depth and rate of thaw, wherever the soil is presently frozen. However, for preliminary design purposes and an assumed depth of abutment fill of about 10 feet, it is believed that about 10 feet of thaw which will contribute to negative skin friction, may take place in the natural subgrade.

TABLE 4.3.1

NEGATIVE SKIN FRICTION OF UNFROZEN SOIL FOR PILE DESIGN

(After Woodward Lundgren And Assoicates, 1971) (4)

DESCRIPTION OF SOIL CATEGORIES	DESIGN NEGATIVE SKIN FRICTION
Clean sands and gravels with little or no silt or clay. Typically: GW, GP, SW,SP	$P_s = 30d \quad (X2 + 2HX)$*
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 a granular stratum.

Ps = 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.

4.4 Frost Heave of Piles

Frost heaving of piles can occur as the active layer freezes each winter. During the cold winter months, the surface soils freeze and bond to the pile at low temperatures. In soils containing silt and clay, this shallow surface adfreeze, if accompanied by ice lens formation, exerts a heaving force on the pile which must be resisted by the dead load on the pile, the available adfreeze bond in the permafrost, and/or pile skin friction within unfrozen soil zones in which the pile is embedded.

In order to prevent pile heave, it is necessary to check the pile design to ensure that the available resisting forces provide an adequate factor of safety against seasonal frost heaving. In general it has been found that slightly deeper pile embeddment is the most feasible means of overcoming undesirable frost heaving stresses, if they exceed the sum of the total resisting forces divided by the factor of safety. Suggested design stresses for general permafrost soils are presented in Table 4.4.1 (4) and may be used for preliminary design purposes.

4.5 Subgrade Considerations on Center Line

As indicated in Tables 3.2.1, and 3.2.2, the stratigraphy on center line, on both sides of Snafu Creek No.1 Crossing and Snafu Creek No.2 Crossing are similar. A thin organic cover averaging about 1 to 2 feet in thickness (ranging from 1 to 3 feet), was noted at all borehole locations. Generally, clay-silt till underlies the organic cover. At Snafu Creek No.2 Crossing till overlies an unestablished depth of silty shale. Estimated visual excess ice contents are generally low, with the exception of near surface organic layers and the upper 10 feet of till. Moisture contents are low and it is expected that firm to stiff conditions will probably exist in unfrozen soils during the summer season. However, a winter construction program is advocated to limit undesirable disturbance to the sub-grade thermal regime.

TABLE 4.4.1

FROZEN SOIL ADFREEZE BOND STRENGTH FOR PILE DESIGN

(After Woodward Lundgren And Associates, 1971) (4)

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.

Althoulgh no shear strength data for unfrozen soil at the site is available, qualitative evaluation of the shear strenth of the various strata can be made from visual observations, ice content estimates, moisture content profiles and classsification test results. Based on these factors, it is concluded that on thawing, moderate to good shear strength conditions will exist in the till and shale layers below a depth of about 10 feet. Shear strength in the upper 10 feet is expected to be moderate when this zone is in a thawed conditon.

A lack of detailed information, with regard to ice contents, and a need for sophisticated testing and detailed computer analyses, makes 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 total thaw settlements of average road grade fills (about 6 feet thick), of about 0.5 to 1.0 feet and 0.5 to 2.0 feet can be expected for winter construction, and 0.5 to 1.5 feet and 0.5 to 3.5 feet for summer construction; for Snafu Creek No.1 Crossing and Snafu Creek No.2 Crossing, respectively. This estimate assumes thawing of the upper 5 to 10 feet of subgrade soils, but does not take into account normal consolidation settlement of the unfrozen subgrade soils due to the surcharge effect 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 sub-grade. 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 6 foot thickness of granular fill material (non-plastic) is considered to be the minimum for road grade construction on underlying frozen subgrade materials at the crossings. Local fine grained materials, such as silty clay till, 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 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 gravel could not be determined.

4.6 Slope Stability Considerations

No evidence of recent slope instability was detected on either valley wall, in the immediate vicinity of either of the proposed crossings. The slope gradient along center line ranges from negligible up to about 21 degrees (38 percent) on the creek banks. Cursory slope stability calculations, using implied shear strength values for thawed materials and the surveyed slope configuration, indicate an adequate factor of safety with respect to slope stability. Consequently, it is believed that approach fills can be constructed on the proposed alignment in comparative safety with respect to natural slope stability. However, it is recommended that excessive fill thickness be avoided near the crest of the slopes. In addition, cutting or excavation 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 stability of approach fills. Bridge abutments should be set as far back from the present creek 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 side of fills. Therefore, it is considered essential that considerable effort and care be given to minimizing 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 materials are very granular.

4.8 Cement Type and Corrosion Considerations

No samples 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 natural soil. 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 grade or above the ground water 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

In order to more accurately assess such factors as insitu shear strength, thaw subsidence, and slope stability, it is desirable to obtain additional detailed geotechnical information at the site. Such items as acquistion of representative undisturbed samples of the various soil types, a thorough study of existing local slopes, 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 a series of closely supervised 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 the design of future pile foundation systems could be established.

V. LIMITATIONS

The foregoing recommendations have been prepared based on our knowledge of existing conditions at Snafu Creek No.1 Crossing and Snafu Creek No.2 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,

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Balanko

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GRG/tmf

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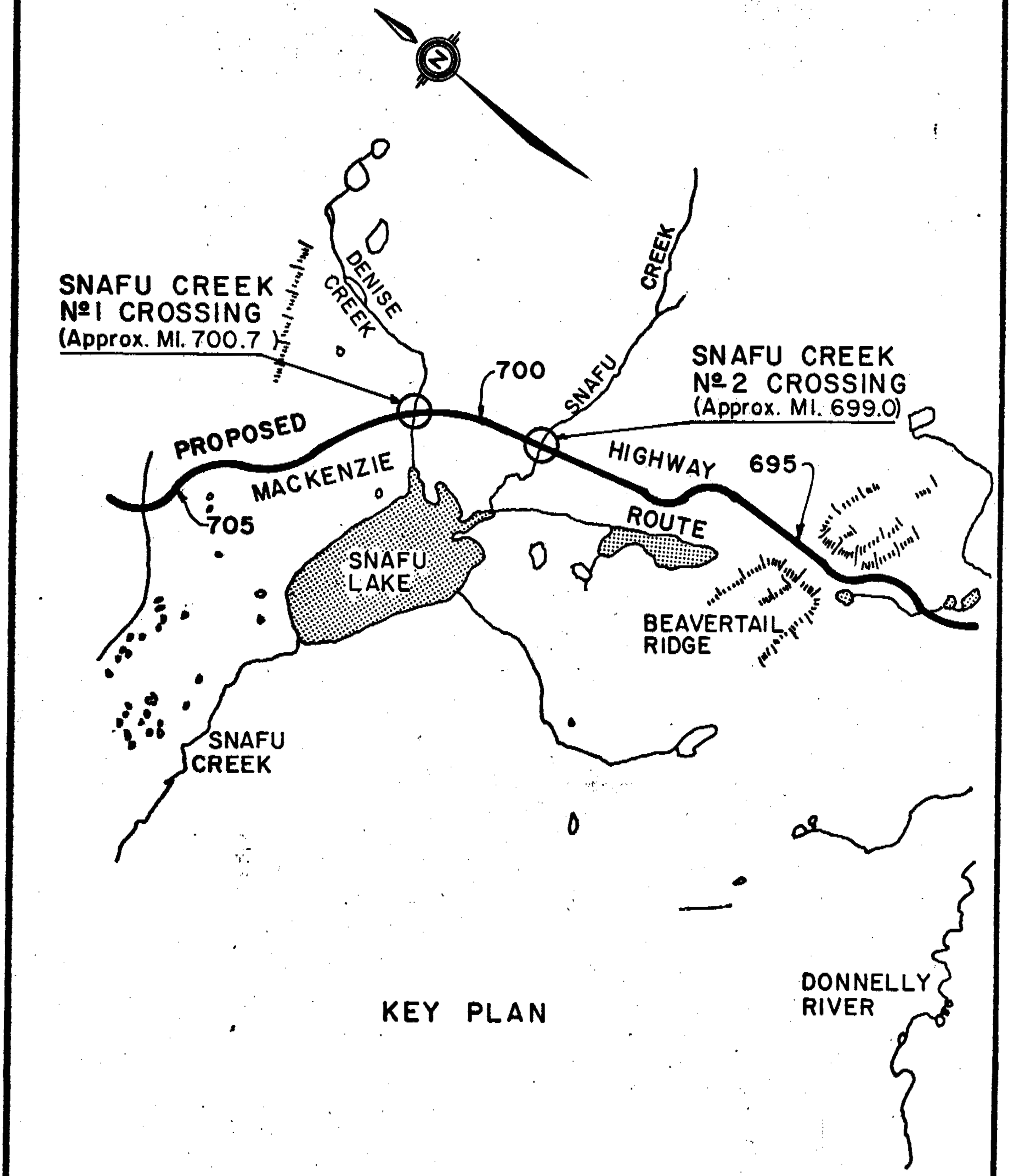
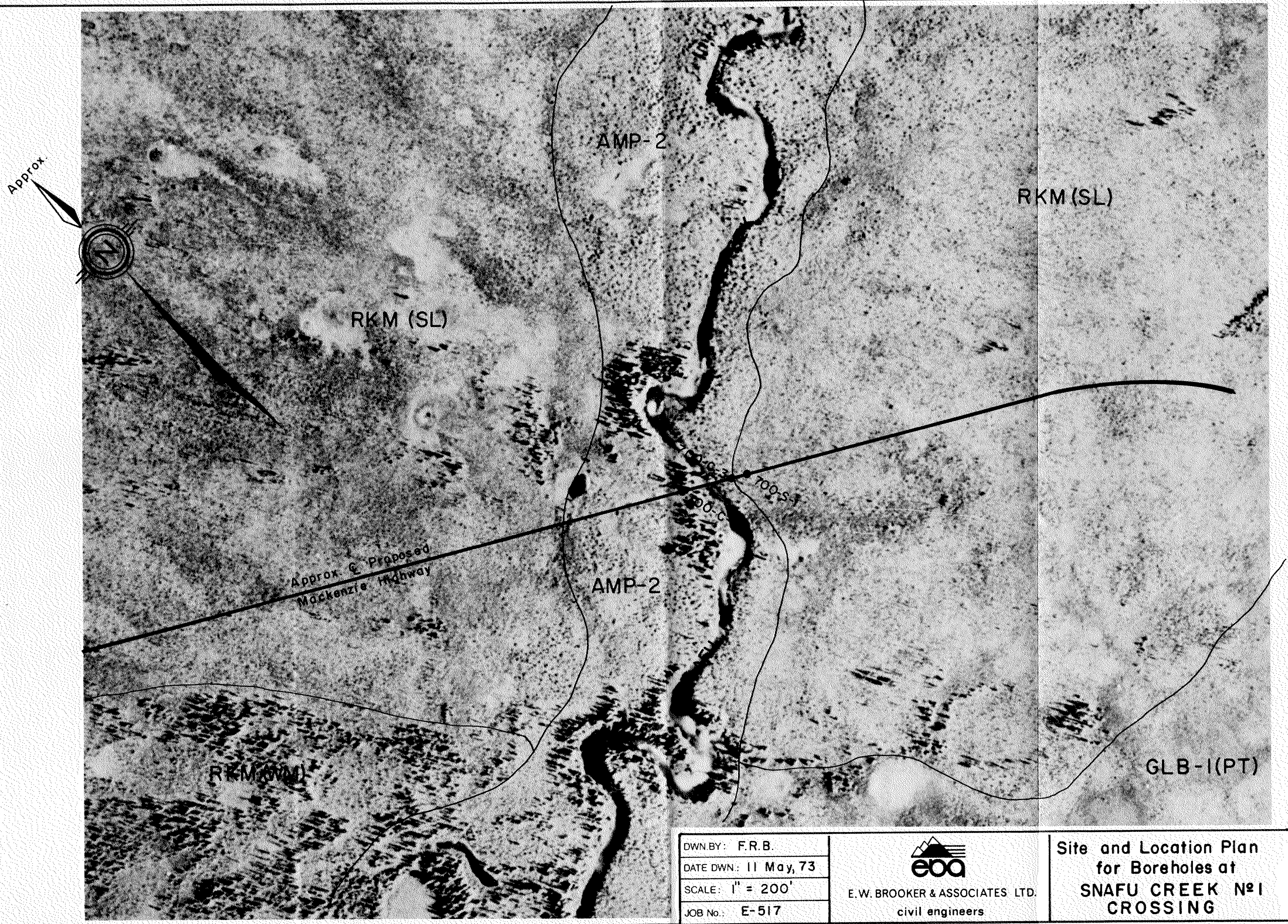


KEY PLAN

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

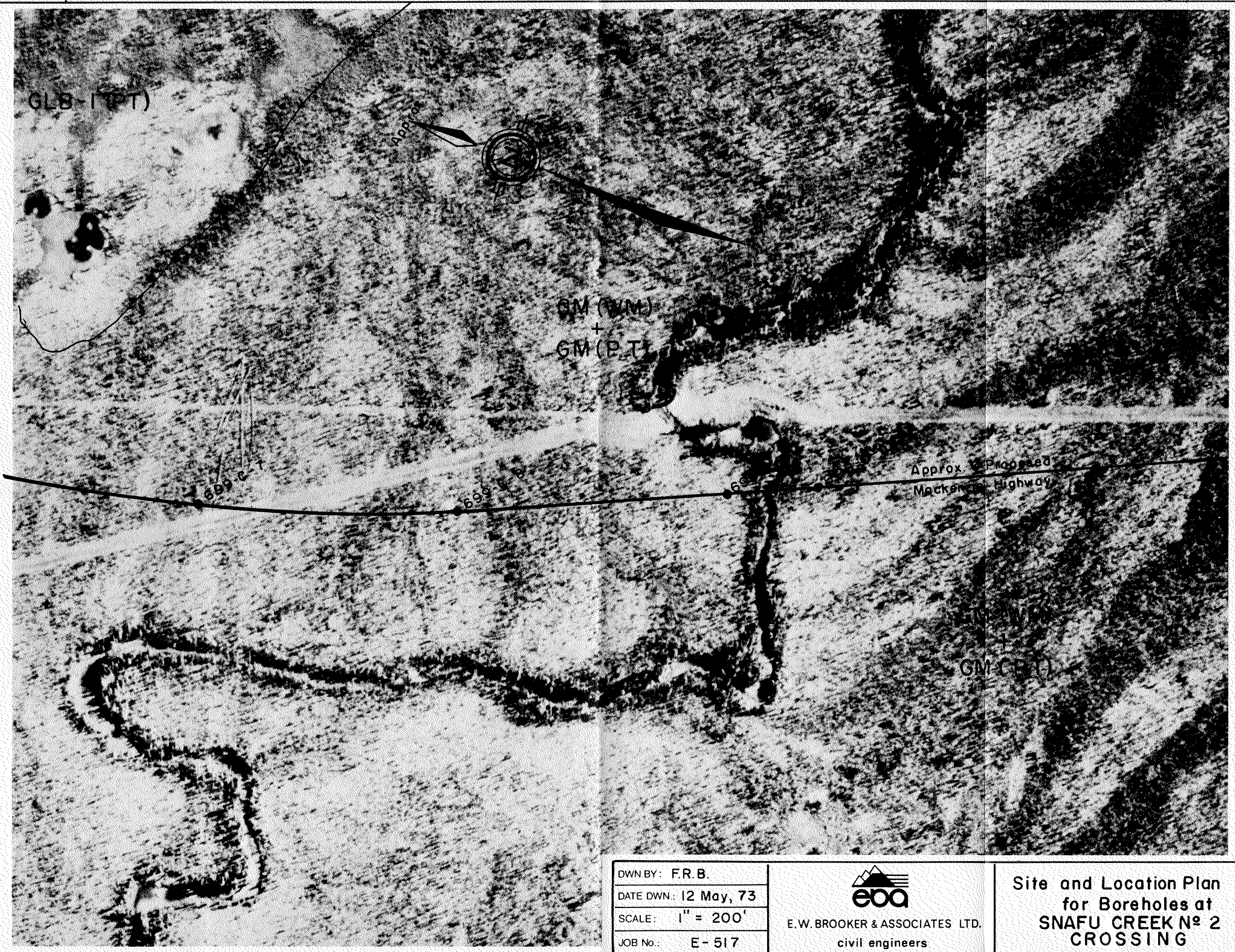


DWN.BY: F.R.B.
DATE DWN: 11 May, 73
SCALE: 1" = 200'
JOB No.: E-517

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

Site and Location Plan for Boreholes at
SNAFU CREEK Nº 1 CROSSING

DWG.: A - 2
SHT.No.:



DWN BY:	F.R.B.
DATE DWN.:	12 May, 73
SCALE:	1" = 200'
JOB No.:	E-517

E.W. BROOKER & ASSOCIATES LTD.
civil engineers

Site and Location Plan for Boreholes at SNAFU CREEK Nº 2 CROSSING

DWG.: A - 3

SHT. No.:

TERRAIN LEGEND

SYMBOL	TERRAIN TYPE	PHYSIOGRAPHIC FEATURES	MATERIALS DESCRIPTION
AMP-2	Alluvial Meander Plain (excluding the Mackenzie River Plain)	Flood plains filling bottom of the stream or river valley	Fine silt, sand or gravel as channel deposits
RKM	Ridge-and-knoll Moraine	Drumlinized till plain. Rolling large linear features	Molded basal till low plastic silty-clay till
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
GM	Ground Moraine (undifferentiated)	Flat to broad gentle slopes	Silt till to clay till usually some sand and gravel

Topstratum Phases (Associated with Terrain Types)

SL	Slopewash or solifluction features. Topstratum of ice-rich poorly sorted silty clay and silty sand to gravel
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 with 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. 3,a

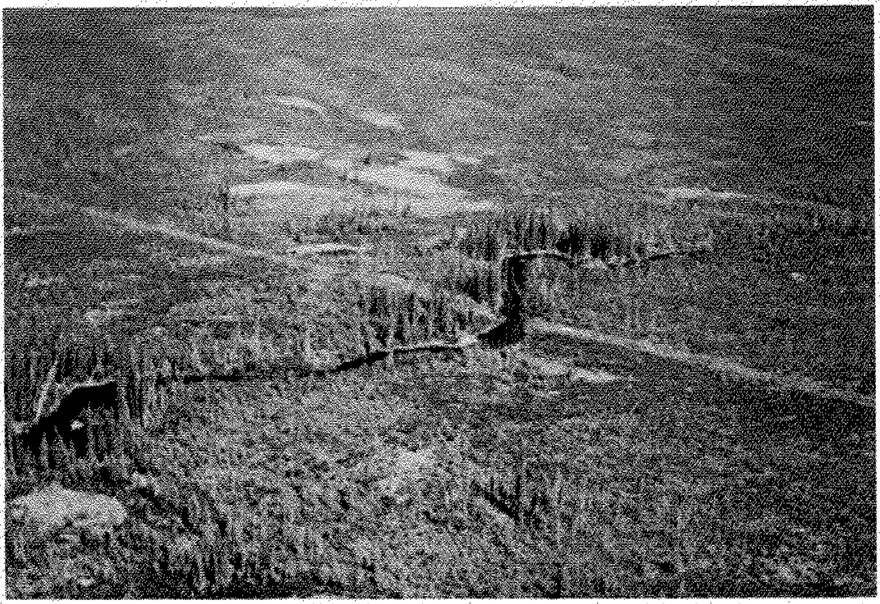


PLATE No. 1

Snafu Lake Creek No. 1
The cutline is the proposed highway route. North is to the left. (June, 1973)

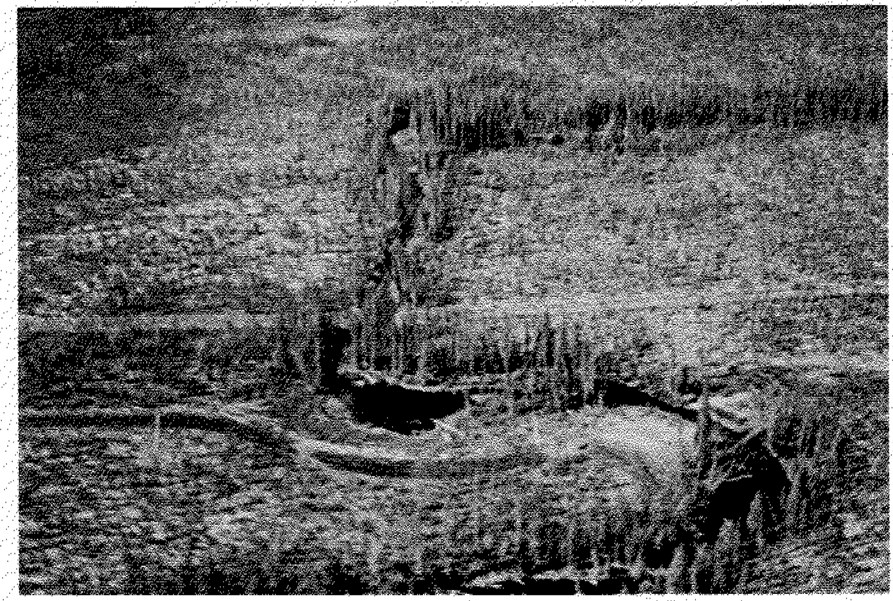
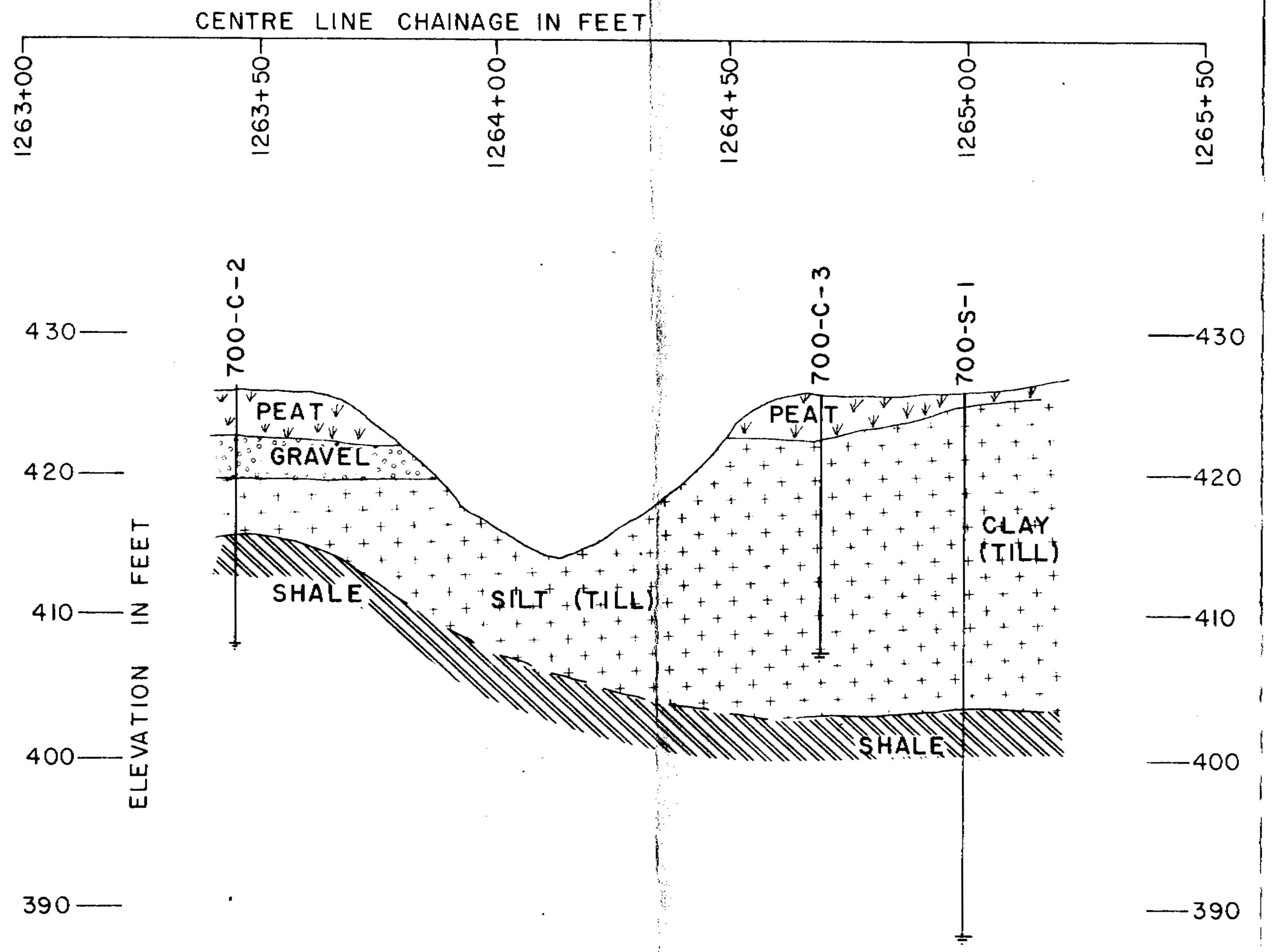


PLATE No. 2

Snafu Lake Creek No. 2
The upper cutline in the plate is the proposed route for the highway. North is to the right of the plate. (June, 1973)

Drawing No. A-4

E.W. Brooker & Associates Ltd.



NOTE:

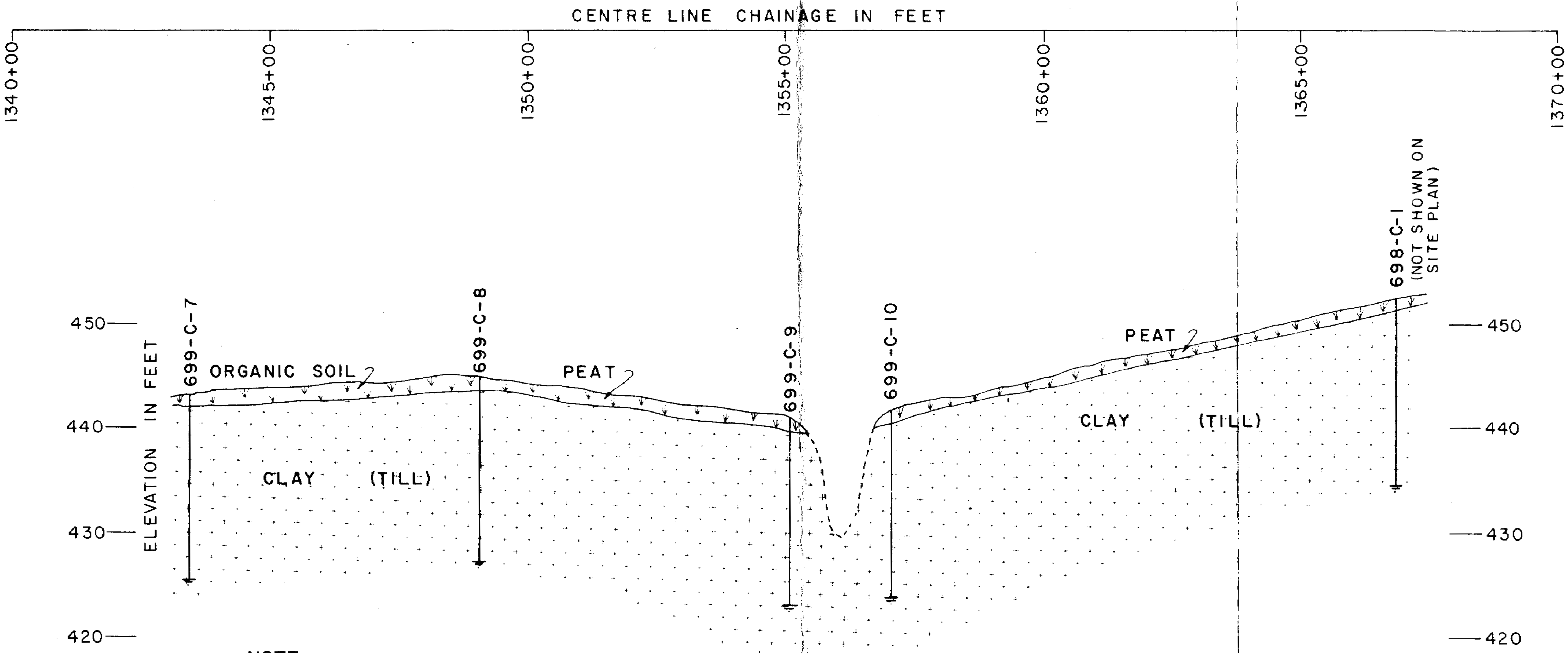
1. Stratigraphy between boreholes has been assumed.
2. Elevations and profile taken from C.E.S. profile plan of Proposed ℄ for Mackenzie Highway.
3. Scales: Horiz. 1" = 30'
   Vert. 1" = 10'

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

eba Engineering Consultants Ltd.

SNAFU CREEK Nº1 CROSSING
℄ PROFILE &
STRATIGRAPHY
(DENISE CREEK)

DWG: A-5
SHT. No.:



NOTE:

1. Stratigraphy between boreholes has been assumed.
2. Elevations and profiles 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:
SCALE: As Noted
JOB No. E-517

eba Engineering Consultants Ltd.

SNAFU CREEK Nº 2 CROSSING
℄ PROFILE &
STRATIGRAPHY
(SNAFU CREEK)

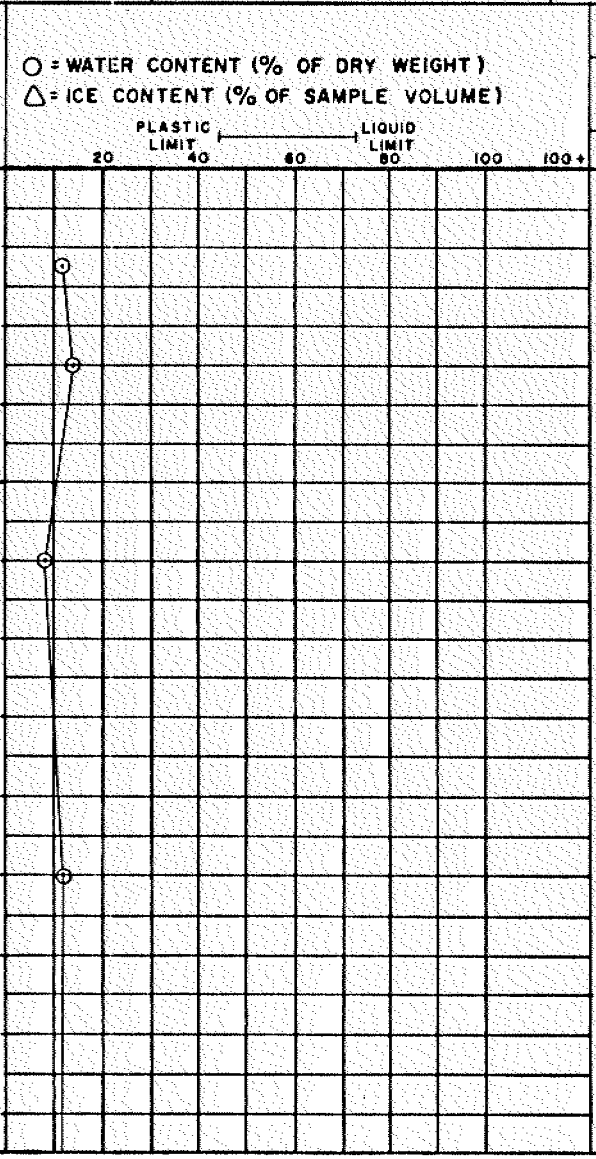
DWG. A-6
SHT. No.

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

DWN: BL	FIELD ENG: LAB	DATE DRILLED: 29/1/73	AIRPHOTO NO: A-22965-102	CHAINAGE: 1265 + 00	OFFSET		
CKD: NRM	TECH: DF	RIG: Mayhew	SURFACE DRAINAGE: Good	VEGETATION: Burnt Out Black Spruce	ELEV: 426.0'		

TEST HOLE	MVPL 7	
MILE	B,C,S	NUMBER
700	S	1

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0					Pt	PEAT - Organic Soil, Dark Brown	F	
2	1				CL	CLAY (TILL) - Silty, Gravelly, Medium Brown, Low Plastic		NB
4	2							
10	3					- Grey, Gravel Particles, Low Plastic	U	Unfrozen
18	4							
22					CL	SHALE - Silt & Clay, Grey, Very Soft		
24								



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

Page 2 of 2

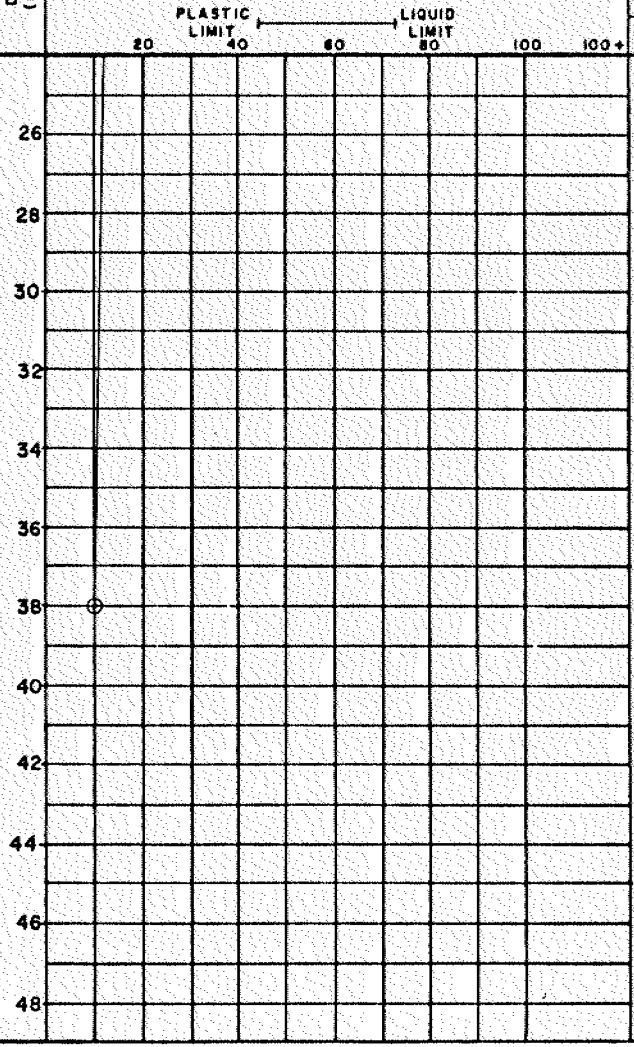
E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: BL	FIELD ENG: LAB	DATE DRILLED 29/1/73	AIRPHOTO NO: A-22965-102	CHAINAGE: 1265 + 00	OFFSET
CKD: NRM	TECH: DF	RIG: Mayhew 500	SURFACE DRAINAGE: Good	VEGETATION: Burned Black Spruce	ELEV: 426.0'

TEST HOLE MVPL 7		
MILE	B,C,S	NUMBE
700	S	1

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
						SHALE - Grey, Silt & Clay Shale, Very Soft		
26								
28					CL		U	Unfrozen
30								
32								
34								
36								
	5							
38						END OF HOLE 38'		
40								
42								
44								
46								
48								

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



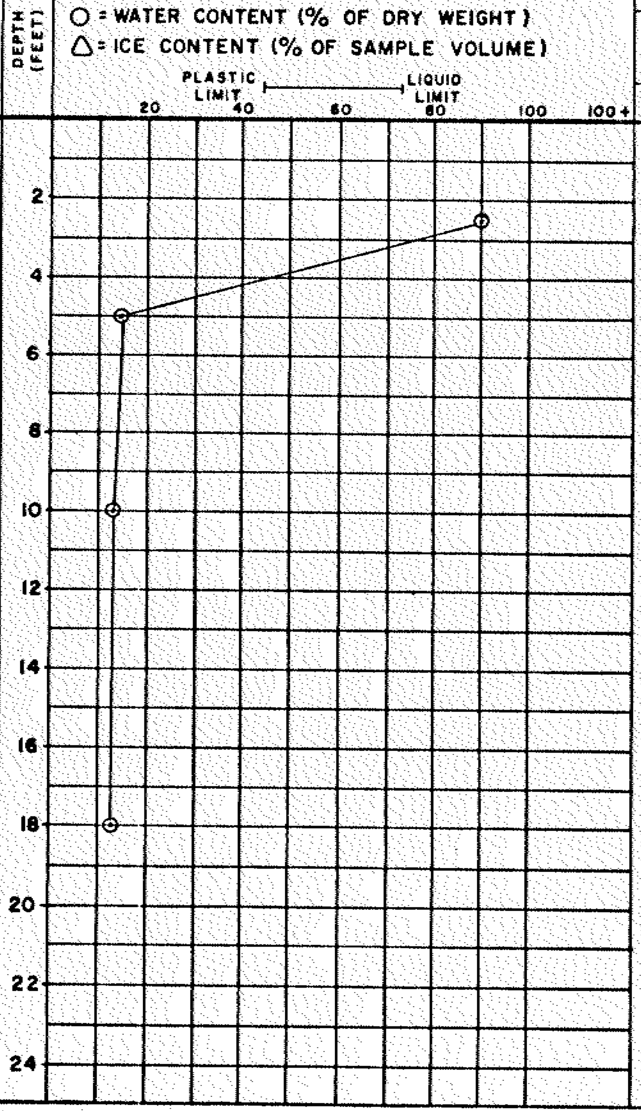
GRAIN-SIZE ANALYSIS				WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
CLAY	SILT	SAND	GRAVEL			
%	%	%	%			

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

DWN: ALB | FIELD ENG: LAB | DATE DRILLED 24/1/73 | AIRPHOTO NO: A22965-124 | CHAINAGE: 1263 + 46 | OFFSET
CKD: NRM | TECH: HC | RIG: Texoma | SURFACE DRAINAGE: Good | VEGETATION: Black Spruce | ELEV: 426.0'

TEST HOLE 203		
MILE	B,C,S	NUMBE
700	C	2

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)
2	1				Pt	PEAT - Organic, Black, Some Silt & Sand @ 2½'								
4	2				GP	GRAVEL - Some Silt & Sand, Med. Brown, Poorly Graded	F	NB						
6														
8					CL	SILT (TILL) Med. Brown, Gravelly Clayey, Moist, Firm, Low Plasticity - Water Slicks @ 8'	U	Unfrozen						
10	3													
12					CL	SHALE - Grey, Silty Shale, Moist, Hard								
14														
16														
18	4					END OF HOLE 18'								
20														
22														
24														



REMARKS

N. Side of Snafu
Creek No. 1 Crossi

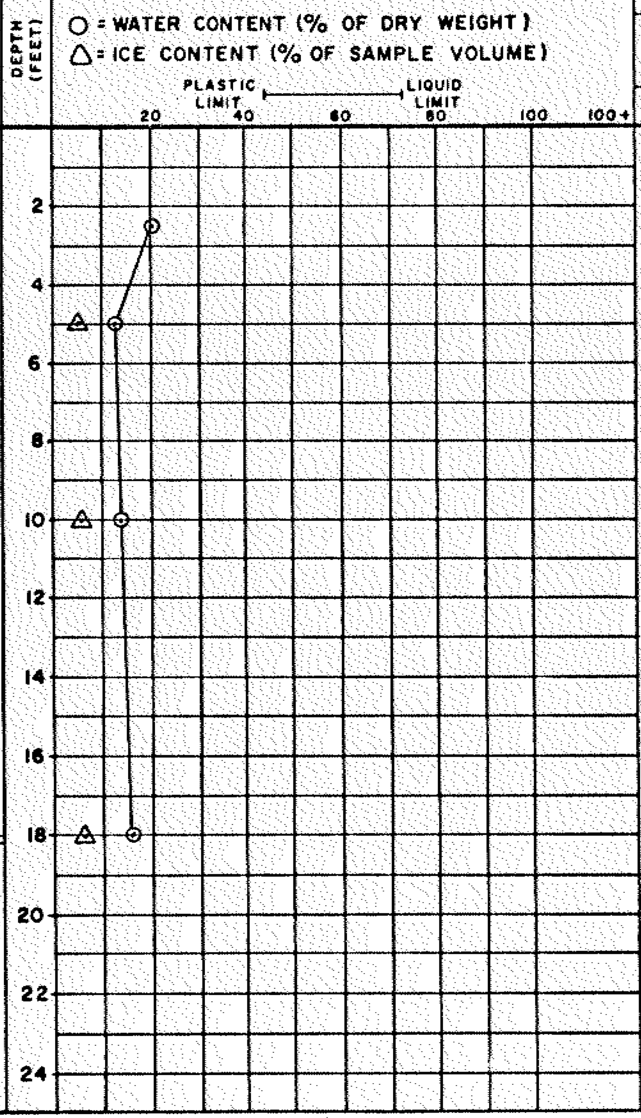
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY**

DWN: ALB | FIELD ENG: LAB | DATE DRILLED 24/1/73 | AIRPHOTO NO: A22965-124 | CHAINAGE: 1264 + 70 | OFFSET

CKD: NRM | TECH: HC | RIG: Texoma | SURFACE DRAINAGE: Good | VEGETATION: Black Spruce | ELEV: 426.0'

TEST HOLE 202		
MILE	B,C,S	NUMBER
700	C	3

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
					Pt	PEAT - Organic Peat, Silt, Sand, Rocks	F	NB							South Side of Snafu Creek No. 1 Crossing
2	1														
4						SILT (TILL) Med. Brown, Gravelly, Low Plasticity, Moist, Firm									
	2				CL										
6															
8															
10	3					- Grey, Silt Till, Low Plasticity, Rocky to 12'		V 5%							
12					CL										
14															
16															
18	4					END OF HOLE 18'									
20															
22															
24															



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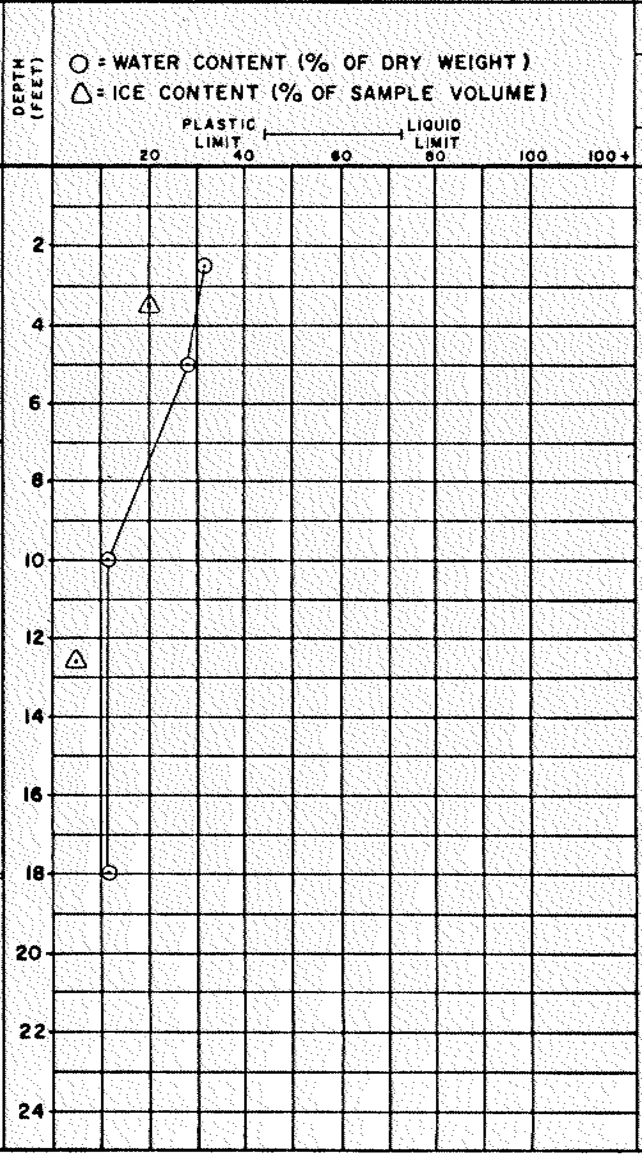
DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: BL
FIELD ENG: LAB
DATE DRILLED 25/1/73
AIRPHOTO NO: A-22965-104
CHAINAGE: 1343 + 43
OFFSET
CKD NRM
TECH HC
RIG: Texoma
SURFACE DRAINAGE: Poor
VEGETATION: Black Spruce
ELEV: 443.8'

TEST HOLE 208		
MILE	B,C,S	NUMBER
699	C	7

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
					Pt	ORGANIC SOIL									
2	1					CLAY (TILL) - Silty, Medium Brown									20' E of CNT Line.
4	2				CL	- Darker Brown With Depth		V 20%							
6						- Low Plasticity	F								
8															
10	3					- Gravelly @ 9 1/2' - 11'	-								
12															
14					CL	CLAY (TILL) - Some Clay - Brownish Grey - Grey @ 16' - Low Plasticity		V 5%							
16															
18	4					END OF HOLE @ 18'									
20															
22															
24															



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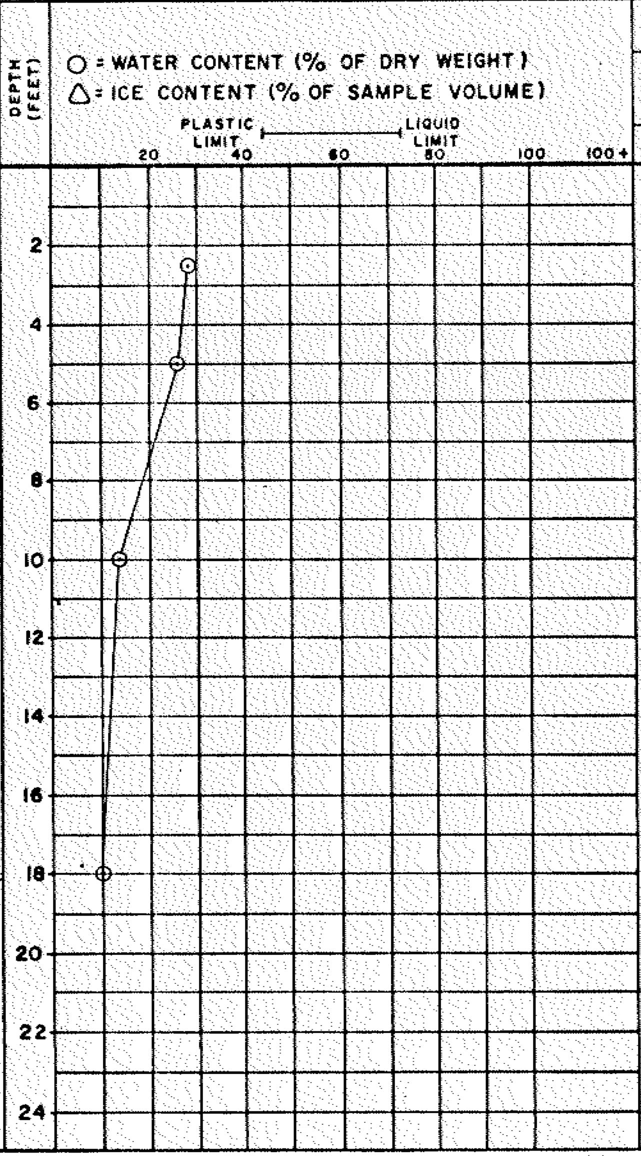
MACKENZIE HIGHWAY

DWN: BL	FIELD ENG: LAB	DATE DRILLED 30/1/73	AIRPHOTO NO: A-22965-104	CHAINAGE: 1349 + 10	OFFSET		TEST HOLE 207
CKD: NRM	TECH: DF	RIG: Mayhew	SURFACE DRAINAGE: Good	VEGETATION: Black Spruce	ELEV 444.6'		

MILE	B,C,S	NUMBER
699	C	8

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
0 – 1					Pt	PEAT - Organic Soil, Dark Brown, Fibrous	F								
1 – 14					CI	CLAY (TILL) - Silty, Medium Brown, Medium Plasticity, Some Coarse Sand To Gravel		NB							
10 – 18							U	Unfrozen							
14 – 18					CL	- Grey, Low Plasticity, Gravel									
18						END OF HOLE 18'									



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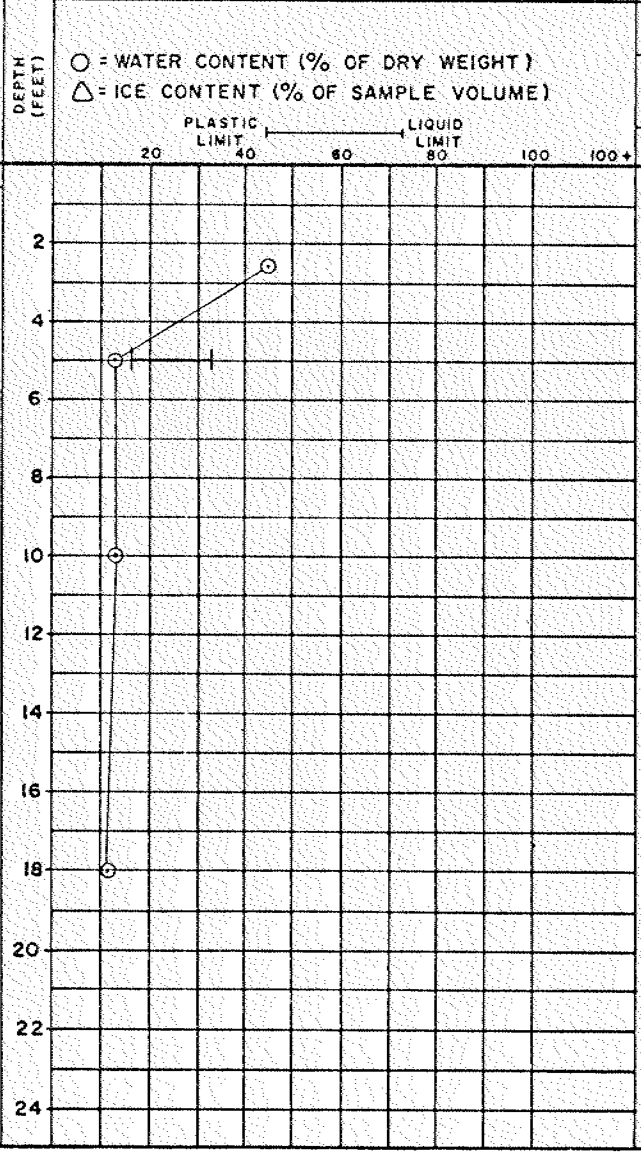
MACKENZIE HIGHWAY

DWN BL	FIELD ENG	LAB	DATE DRILLED 30/1/73	AIRPHOTO NO: A-22965-104	CHAINAGE 1355 + 05	OFFSET		TEST HOLE 206	
CKD NRM	TECH	DF	RIG Mayhew	SURFACE DRAINAGE Good	VEGETATION: Black Spruce	ELEV 440.8'			

MILE	B,C,S	NUMBER
699	C	9

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					Pt	PEAT - Organic Soil, Dark Brown	F	NB
						CLAY (TILL) - Silty, Medium Brown, Low Plasticity, Very Gravelly, Some Sand	U	
2	1	◸						
4	2	◸			CI			
8								Unfrozen
10	3	◸				- Grey, Low Plasticity, Gravelly		
13					CL			
18	4	◸						
						END OF HOLE 18'		



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

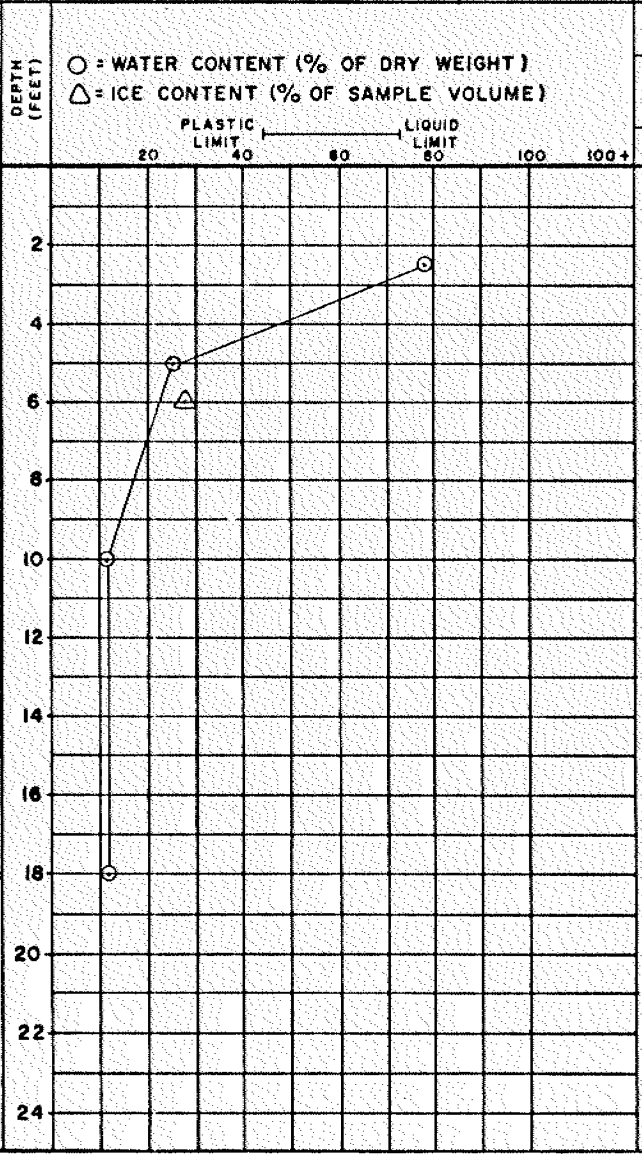
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY**

DWN: BL	FIELD ENG: LAB	DATE DRILLED 30/1/73	AIRPHOTO NO: A-22965-104	CHAINAGE: 1357 + 10	OFFSET
CKD: NRM	TECH: DF	RIG: Mayhew	SURFACE DRAINAGE: Good	VEGETATION: Black Spruce	ELEV: 442.0'

TEST HOLE 205

MILE	B,C,S	NUMBER
699	C	10

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					Pt	PEAT - Organic Soil, Dark Brown	F	NB
2	1					CLAY (TILL) - Silty, Medium Brown, Low Plasticity, Gravelly to Sandy		V 25-30%
4	2				CL			
6								
8								
10	3							
12						- Greyish Brown, Low Plasticity, Gravelly	U	
14								Unfrozen
16						- Grey, Low Plastic, Gravelly		
18	4					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 %			

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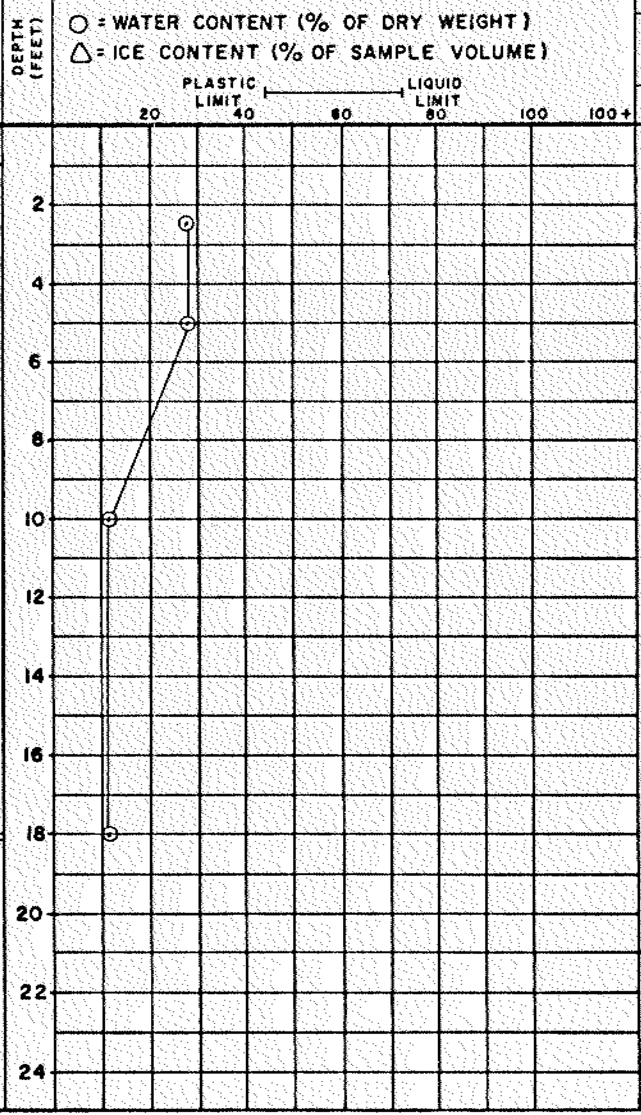
MACKENZIE HIGHWAY

DWN: BL | FIELD ENG: LAB | DATE DRILLED: 30/1/73 | AIRPHOTO NO: A-22965-104 | CHAINAGE: 1367 + 00 | OFFSET

CKD: NRM | TECH: DF | RIG: Mayhew | SURFACE DRAINAGE: Good | VEGETATION: Black Spruce | ELEV 452.7

TEST HOLE 220		
MILE	B,C,S	NUMBE
698	C	1

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					Pt	PEAT - Organic Soil, Dark Brown	F	NB
2	1				CL to CI	SILT (TILL) - Clayey, Medium Brown, Low to Medium Plasticity, Gravel Particles, Traces of Sand	U	
4	2							
10	3				CL	CLAY (TILL) - Gravelly, Grey		Unfrozen
14						- Grey, Silty, Low Plasticity, Gravel Particles		
18	4							
						END OF HOLE 18'		



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

SUMMARY OF TEST RESULTS

SNAFU CREEK CROSSINGS

JOB No. E-517

BORE HOLE	DEPTH	NATURAL WATER CONTENT	Atterberg Limits			MECHANICAL ANALYSIS (M.I.T. CLASSIFICATION)				SOIL CLASSIFICATION (UNIFIED)	REMARKS
			W_L	W_P	PI						
	feet	%	%	%	%	% CLAY	% SILT	% SAND	% GRAVEL		
699-C-9	5	12.9	32.5	17.1	15.4					CL	SO_4 - 0.125%

DWG. No. C-1

EBA ENGINEERING CONSULTANTS LTD.