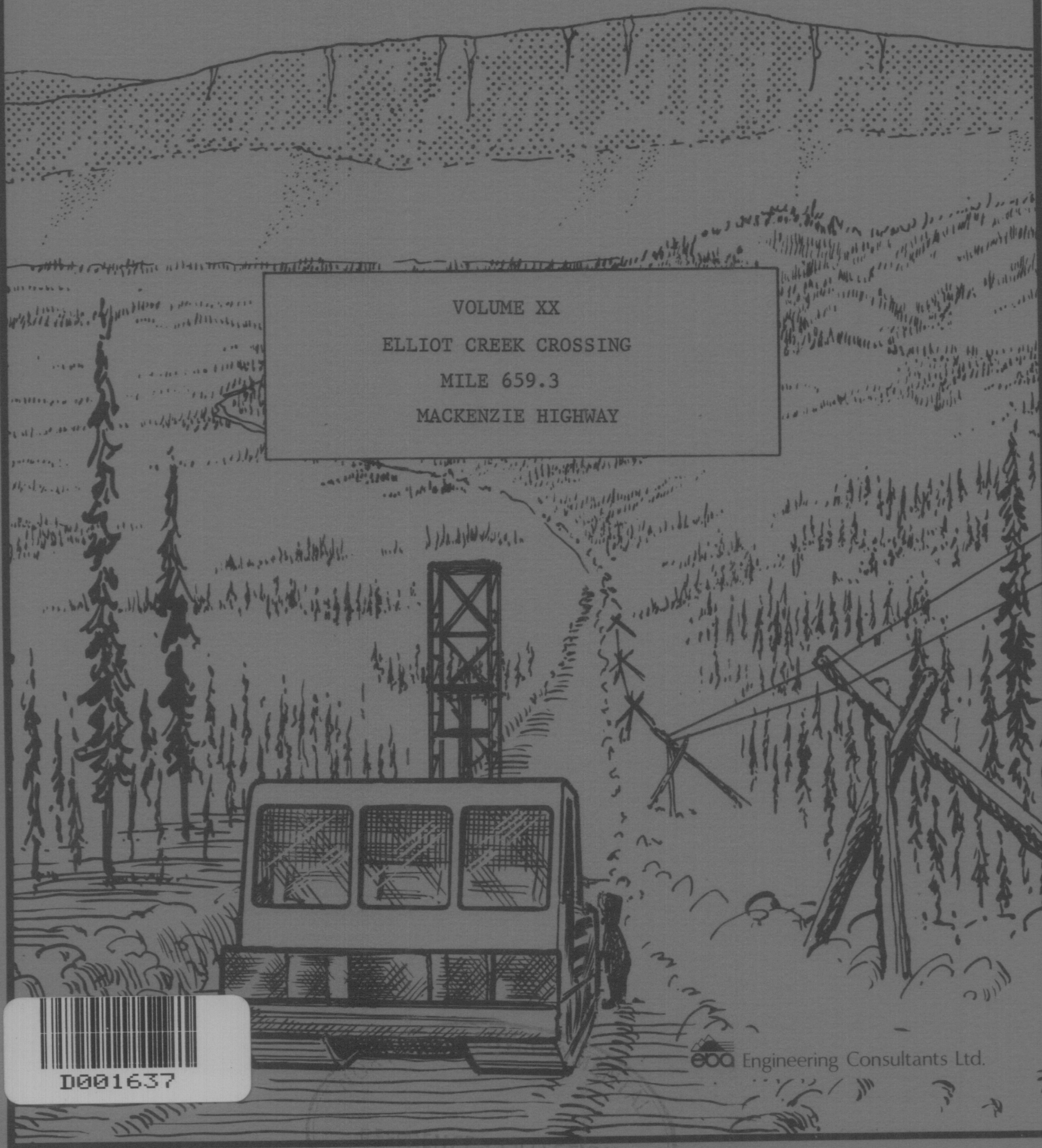
MACKENZIE HIGHWAY
GEOTECHNICAL EVALUATION



VOLUME XX
ELLIOT CREEK CROSSING
MILE 659.3
MACKENZIE HIGHWAY

eba Engineering Consultants Ltd.

D001637

COLLECTION

eba Engineering Consultants Ltd.

G. R. GILCHRIST, M.Sc., P.Eng.
vice president

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CENTRAL RECORDS
D.P.W. OF CANADA
WESTERN REGION
APR 4 1974
FILE 9305-63-309

April 3, 1974

Government of Canada
Public Works of Canada
One Thornton Court
Edmonton, Alberta

Attention: Mr. J.A. Brown, Regional Director

Gentlemen:

Subject: Geotechnical Evaluations - Oscar Creek Crossing
- Elliot Creek Crossing

It has come to our attention that an incorrect symbol was used in geotechnical reports pertaining to bridge construction activities at the Oscar and Elliot Creek Crossings. These reports are designated Volumes XXI and XX, respectively. We request that the symbol N' on line 6 paragraph 2, subsection 2.2, Laboratory Testing, be changed to read F'. Thus the corrected sentence should read (beginning on line 4):

> '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.'

We trust the required corrections do not cause any inconvenience. Should you require corrected versions of both pages, please contact our Edmonton office and we will be pleased to undertake the necessary changes.

Very truly yours,

EBA Engineering Consultants Ltd.

Garry R Gilchrist

G.R. Gilchrist, P. Eng.

GRG:lmh

F.J. Please ⊙ copy of this memo to all recipients of subject reports

THE ASSOCIATION OF
PROFESSIONAL ENGINEERS
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TABLE OF CONTENTS

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 Elliot 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 fourteen boreholes, drilled at the locations shown on Drawing No. A-2, Appendix A. Of the fourteen boreholes advanced, three were drilled as center line boreholes, in conjunction with the general route evaluation, and the remainder were located and drilled specifically to define subsurface conditions at the creek crossing.

The special boreholes consisted of Boreholes 659-S-1 to 659-S-11, inclusive. The three center line boreholes were designated Boreholes 659-C-2 to 659-C-4, inclusive. Detailed borehole logs are presented in consecutive order in Appendix B.

The center line boreholes were drilled with a Texoma Super Economatic power auger, fitted with a 12 inch diameter stub auger. All special 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 5 feet to 43 feet, and averaged 21 feet in depth. Sampling consisted of representative bag samples, obtained at depths of 2½ 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, Atterberg limits, grain size distribution, soluble sulphate concentration, and pH 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.

Frozen ground was classified according to a modification of the NRC system for describing permafrost (3). The modification was necessary because the disturbed nature of the sample 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 on grain size distribution curves, Drawings No. C-1 to C-6, inclusive, Appendix C. Drawing No. C-7, Appendix C, presents a partial summary of laboratory results.

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

III. SITE CONDITIONS

3.1 Surface Features and Geology

The proposed Mackenzie Highway crosses Elliot Creek at Mile 659.3, approximately 27 miles north-west of Norman Wells. Drawing A-1, Appendix A, is a Key Plan of the Elliot Creek area and Drawing No. A-2, Appendix A, presents a detailed Site Plan. Plate No. 1, Drawing No. A-3, Appendix A, shows the crossing from the air in June 1973.

Elliot Creek drains a relatively small area extending north-east of Mount Thomas and Mount Morrow. Part of the former Elliot Creek watershed has probably been captured by the Hanna River and Oscar Creek. This may explain the existence of a deep gully, which is presently occupied by a relatively small stream. Because of the small watershed, the summer flow in Elliot Creek is expected to be limited. However, the base flow appears to be supplemented by groundwater seepage from the mountains nearby, hence a modest base flow may be maintained throughout the year.

Aerial photographic interpretation of the surficial geology of the immediate area of Elliot Creek Crossing, is shown on Drawing No. A-2, Appendix A. The surficial materials are believed to be alluvial meander plain and outwash deposits that have been reworked to some degree by slopewash action.
A terrain legend, which describes the symbols used in the terrain analysis, is presented as Drawing No. A-2a, Appendix A.

3.2 Subsurface Conditions

Based on observations from the boreholes, a 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 ELLIOT CREEK CROSSING

MATERIAL	DESCRIPTION	APPROXIMATE DEPTH BELOW EXISTING GRADE (FT)	AVERAGE RANGE OF THICKNESS (FT)
PEAT	reddish brown, fibrous, some silt, V5%-20%	0 - 1	0 - 3
GRAVEL & SAND	fine to coarse grained, poorly graded, loose to dense, some silt and clay, medium brown, moisture content (M/C) 5% to 50% avg. 15%, NB to NF	1 - 9	2 - 13
CLAY	grey, medium plastic, silty, some sand and pebbles, M/C 18% to 34% avg. 25%, NB to V5%	9 - Depth of Penetration	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 43 feet.

2. Unfrozen clay was noted in Boreholes 659-C-2 and 659-S-3 below depths of 9 and 13 feet, respectively.

3. A silt pocket was noted in Borehole 659-S-1 between the depths of 3 feet and 5 feet.

4. Borehole 659-S-8 was terminated at a depth of 5 feet due to sloughing gravel.

5. 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

4.1 Foundation Types

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

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

4.2 Foundation Design

A major factor affecting the design of pile foundations at Elliot Creek is the noted occurrence of unfrozen zones within the subsoil. Although frozen soil was logged in the vicinity of bridge abutments, 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 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 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 and depth to a thaw stable bearing surface, 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 laboratory testing is considered necessary.

The recommended foundation types listed in Subsection 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 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 Elliot Creek Crossing, is an end bearing pile system achieving support on bedrock existing beneath the site. However, due to equipment limitations the maximum depth of drill penetration was 43 feet, with bedrock not being encountered.

Based on a review of bedrock geology of the area, it is believed that shale bedrock of Upper to Middle Devonian Age (4) may be expected at an unknown depth below the approximate abutment locations of the proposed bridge crossing. It is recommended that consideration be given to the use of steel end bearing piles for bridge foundation support. However, determination of bedrock depth and properties at 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 100± feet may be necessary.

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 prebored 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 are presently believed to be less feasible, as preboring would result in loss of lateral support, and installation without preboring to the estimated depth is anticipated to meet with high 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' 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 he 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 a correlation to the driving records.

4.2.2 Friction Piles

Based on available geotechnical information at the Elliot Creek Crossing, it is believed that a significant 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. However, the present lack of specific information, with respect to the strength of the 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 capacities 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), with a minimum nominal size of 12 inches by 12 inches, and 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 an allowable static design load of 60 kips to be used. Although preboring is not considered necessary for the installation of steel H-piles, through permanently frozen ground at this site, 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.

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 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 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 an allowable static design load of 45 kips to be used. 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.

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 sand and gravel layer is relatively thaw stable but all silty clay materials, noted below an average depth of 9 feet from existing ground surface, are thaw unstable. Consequently, significant 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 construciton 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 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 Elliot 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. However, for preliminary design purposes and an assumed depth of abutment fill of about 10 feet, it is believed that about 5 feet of thaw may take place in the natural subgrade which will contribute to negative skin friction. This estimate assumes that the fill is placed during the winter on a frozen subgrade.

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.

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	$P_s = 30d\ (X^2 + 2HX)$*
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.

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.

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 a 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 (5) and may be used for preliminary design purposes.

4.5 Subgrade Considerations on Center Line

As indicated in Table 3.2.1, the stratigraphy on center line, on both sides of Elliot Creek, is similar. A thin organic cover, averaging about 1 foot in thickness (ranging from 0 to 3 feet), was noted at several borehole locations. Generally, 2 to 13 feet of gravel and sand underlays the organic cover and overlies an unestablished depth of silty clay. Estimated visual excess ice contents are generally low, with the exception of near surface organic layers. Moisture contents are moderate and it is expected that firm 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.

Although no shear strength data for unfrozen soil at the site is available, qualitative evaluation of the shear strength of the various strata can be made from visual observations, ice content estimates, moisture content profiles and classification test results. Based on these factors, it is concluded that on thawing, medium dense, moderate to good shear strength conditions will exist in the gravel and sand layer and low to moderate shear strength will exist in the silty clay.

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

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

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 1.5 feet can be expected for winter construction, and 1.0 to 2.0 feet for summer construction. 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 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 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 depth for road grade construction on underlying frozen subgrade materials at Elliot Creek Crossing. Local fine grained materials, such as silty clay, 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. However, it is believed that gravel underlies the entire flood plain and grades into the subsurface gravel, noted in the boreholes. It is difficult to estimate the maximum depth of scour, but the presence of gravel indicates that high stream velocities occur at peak runoff and significant depths of scour may occur.

4.6 Slope Stability Considerations

No evidence of recent slope instability was detected on either valley wall, in the immediate vicinity of the proposed crossing. The slope gradient along center line ranges from about 2 to 13 degrees (about 3 to 23 percent grade). 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 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 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 grade is very granular.

4.8 Cement Type and Corrosion Considerations

A representative sample from the crossing area was tested to determine the soluble sulphate concentration and soil acidity. The soluble sulphate concentration determined rates as considerable and the pH indicates a slightly acidic condition. Therefore, 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 pipe 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 acquisition 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 Elliot 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.

PROFESSIONAL ENGINEER G. R. GILCHRIST ALBERTA

G.R. Gilchrist, P. Eng.

PROFESSIONAL ENGINEER L. A. BALANKO ALBERTA

L.A. Balanko, P. Eng.

GRG/tmf

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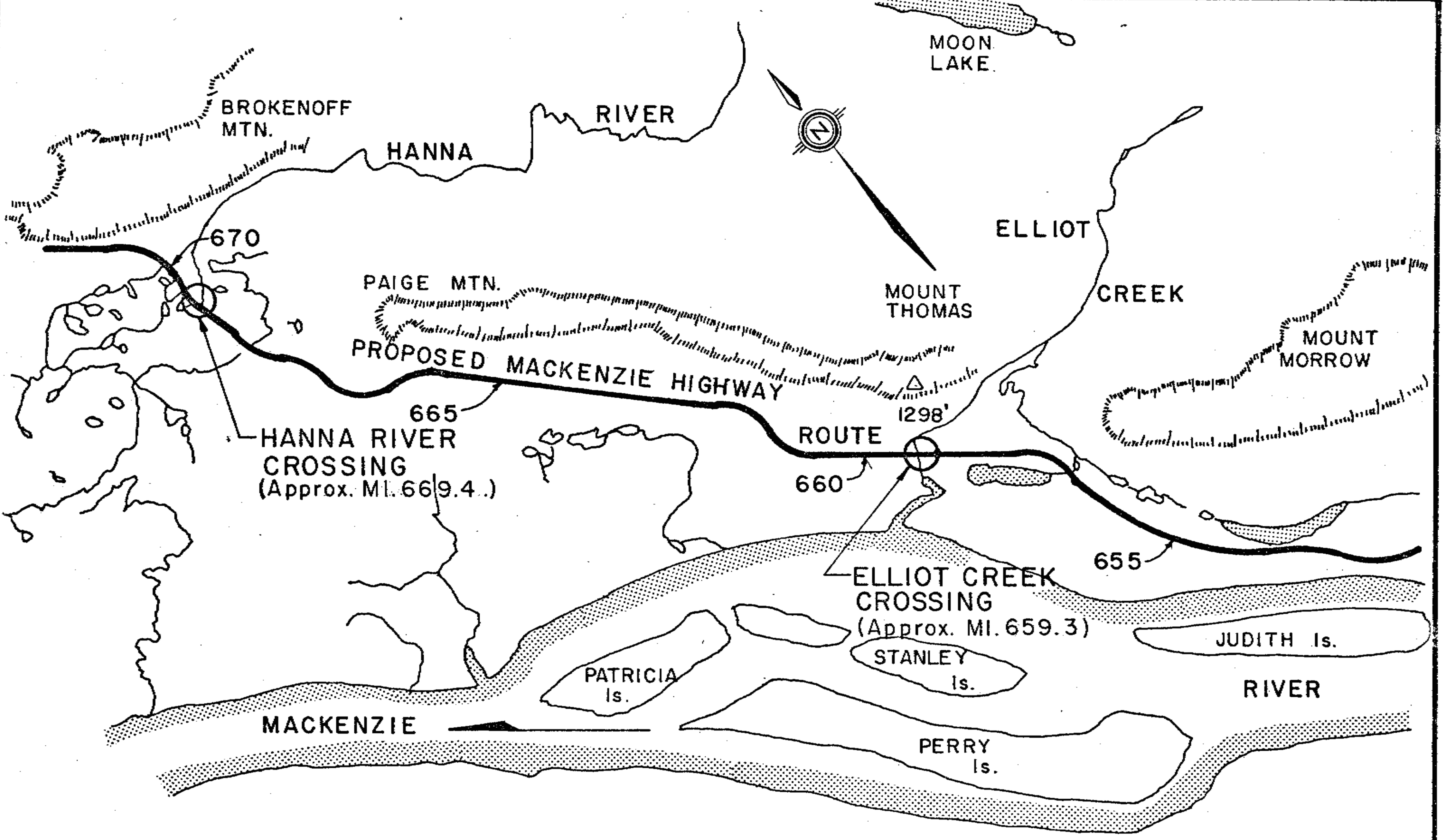
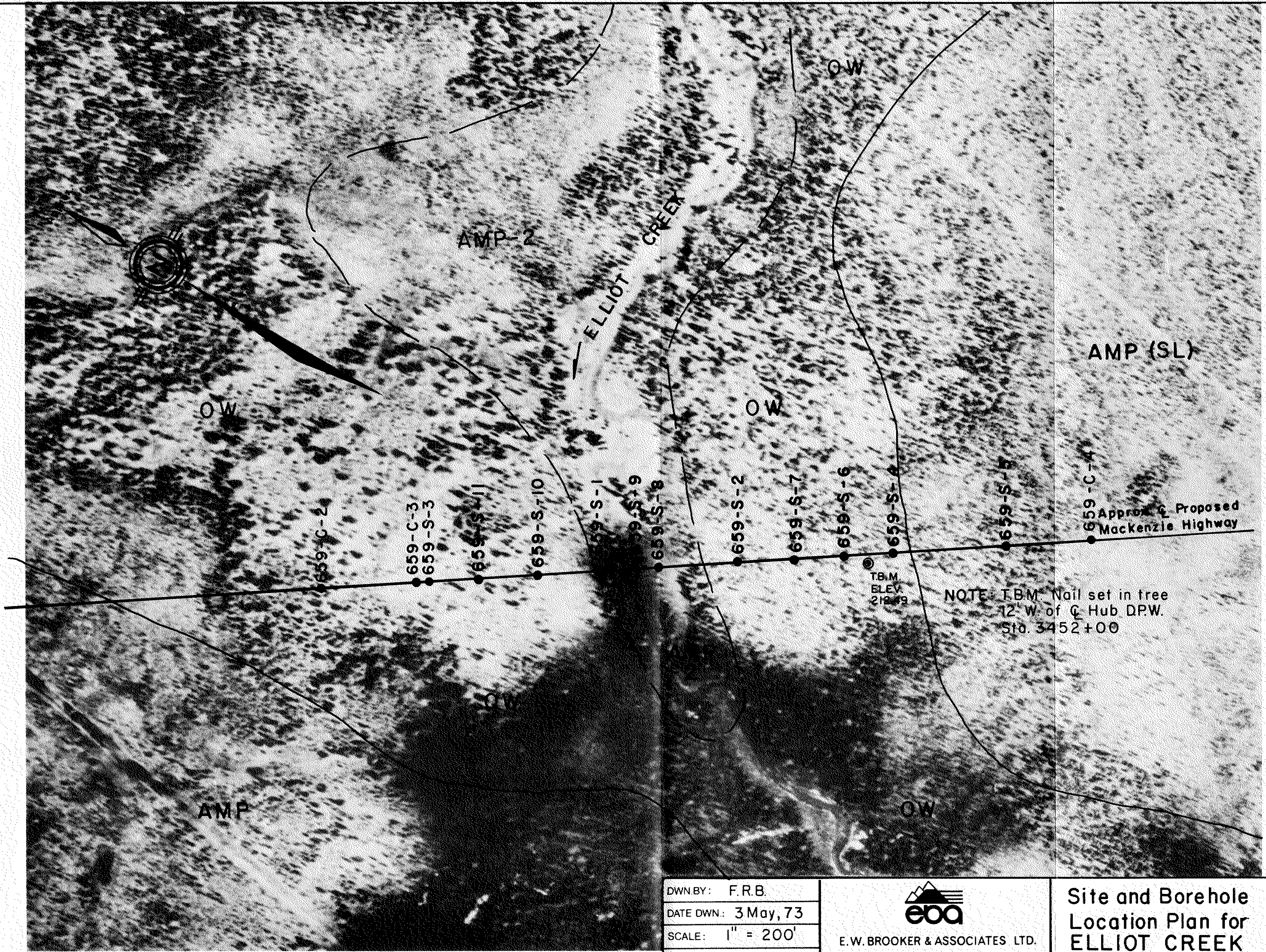


KEY PLAN

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.:	3 May, 73
SCALE:	1" = 200'
JOB No.:	E-517

E.W. BROOKER & ASSOCIATES LTD.
civil engineers

Site and Borehole Location Plan for ELLIOT CREEK Crossing

DWG.: A-2

SHT. No.:

TERRAIN LEGEND

Symbol	Terrain Type	Physiographic Features	Materials Description
AMP	Alluvial Meander Plain (Mackenzie River Meander Plain)	Flat plain often with sand dunes on it	Sands and silty sands stratified or channel deposits
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
OW	Outwash Plains or Deltas	Tabular bodies	Sand and/or gravel

Topstratum Phases (Associated with Terrain Types)

SL	Slopewash of solifluction features. Topstratum of ice-rich poorly sorted silty clay and silty sand to gravel.

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. A - 2a

eba Engineering Consultants Ltd.

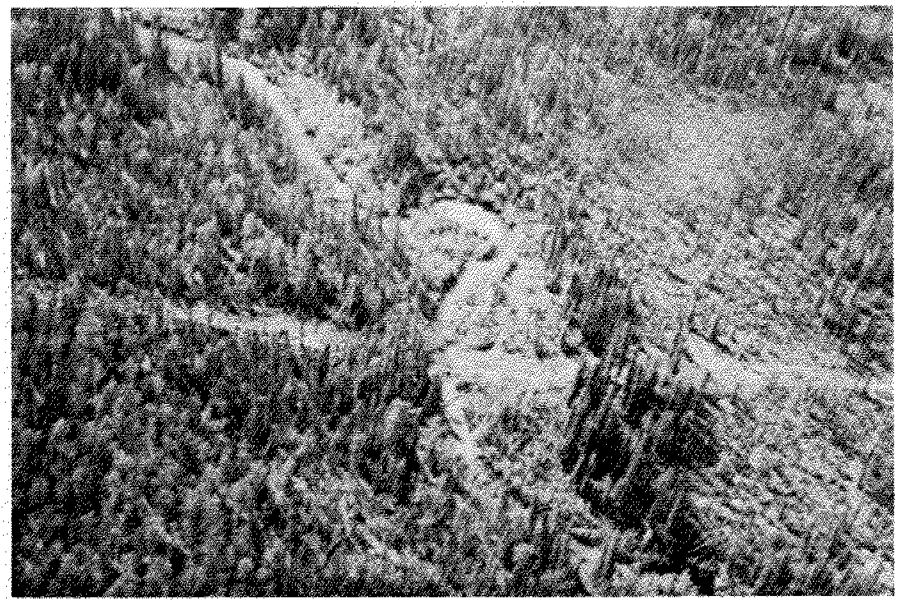
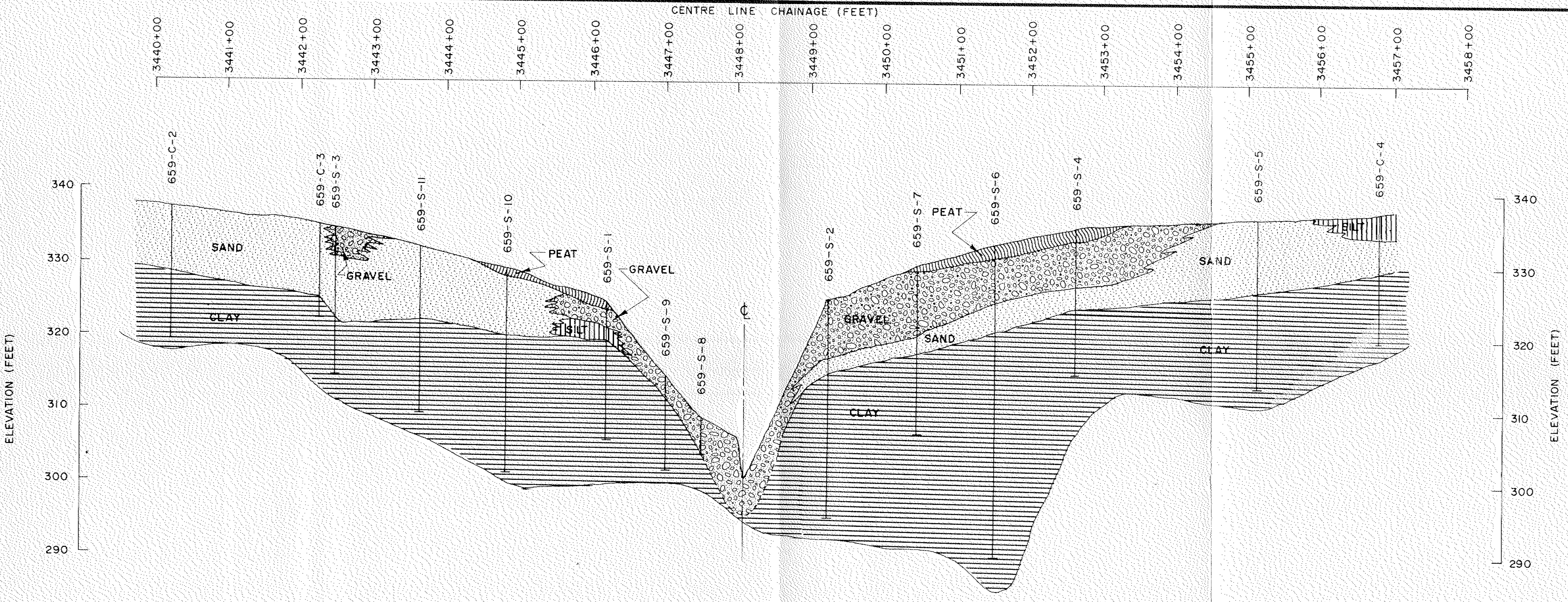


PLATE No. 1

General view of the proposed highway crossing at Elliot Creek. North is to the right of the plate. (June, 1973)

Drawing No. A - 3



NOTE: 1. STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN ASSUMED.

2. ELEVATIONS AND CHAINAGES BASED ON COMBINED CANADIAN ENGINEERING SURVEYS (CES) AND EBA SURVEYS AND RELATED TO CES DATUM.

DWN BY F R B DATE DWN SCALE AS NOTED JOB No E-517	EBA Engineering Consultants Ltd.	ELLIOT CREEK CROSSING ℄ PROFILE & STRATIGRAPHY	DWG A-4 SHT No

E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT 4

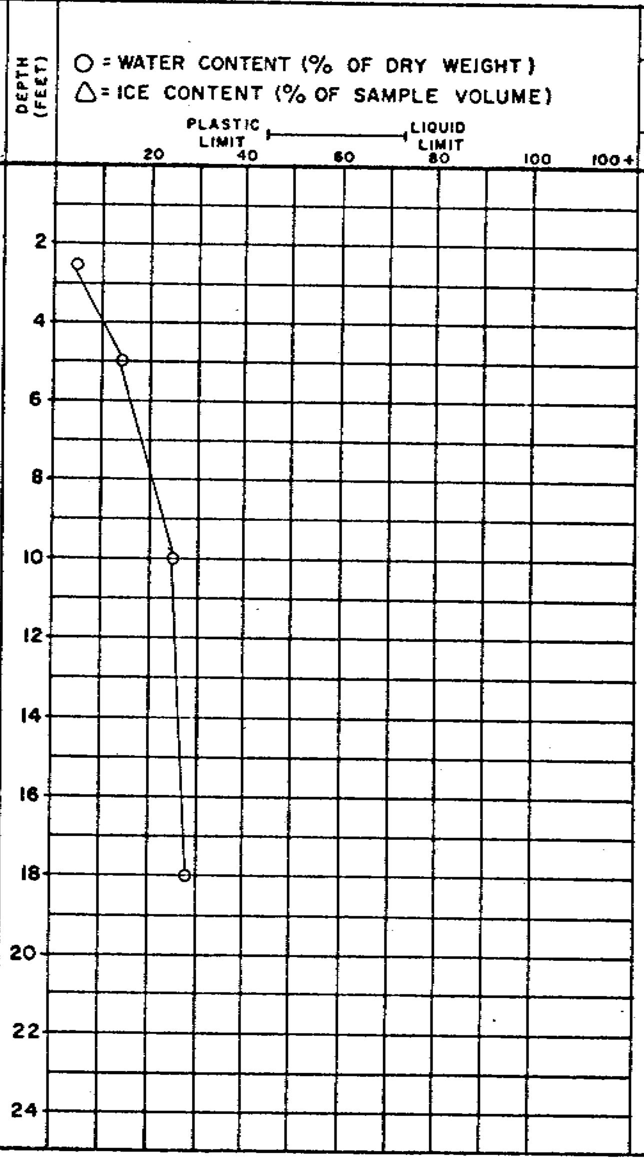
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB
CKD: GRG
FIELD ENG: NRM
TECH: JK
DATE DRILLED: 18/2/73
RIG: Texoma
AIRPHOTO NO: A22774 - 54
SURFACE DRAINAGE: Fair to South
CHAINAGE: 3440 + 25
VEGETATION: Black Spruce & Birch
OFFSET:
ELEV: 337.3'

TEST HOLE

MILE	B,C,S	NUMBER
659	C	2

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
2 4 6 8	1 2				SW	SAND - Med. Brown - Gravelly - Loose - Some silt and Clay - Well Graded	F	NF	(11)		56	33			
10 12 14 16 18	3 4				CI	CLAY - Grey - Med. Plasticity - Wet	U.	Unfrozen							
20 22 24						END OF HOLE 18'									



E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

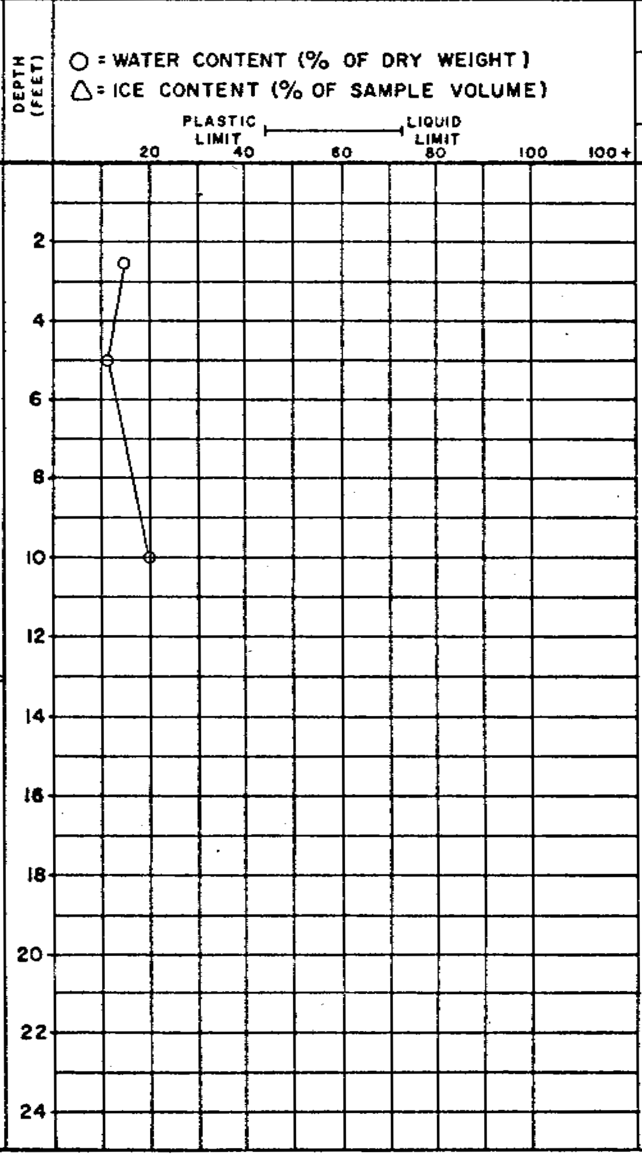
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 18/2/73	AIRPHOTO NO: A22774 - 54	CHAINAGE: 3442 + 25	OFFSET
CKD GRG	TECH: JK	RIG: Texoma	SURFACE DRAINAGE: Good to South	VEGETATION: Black Spruce & Birch	ELEV. 335.3'

TEST HOLE

MILE	B,C,S	NUMBER
559	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
0 – 10	1 2 3	▨ ▨ ▨			SP	SAND - Gravelly - Med. Brown - Dense - Poorly Graded - Trace of Silt and Clay	F	NB							
10 – 13					CI	CLAY - Grey, Silty - Med. Plasticity									
13						END OF HOLE 13'									



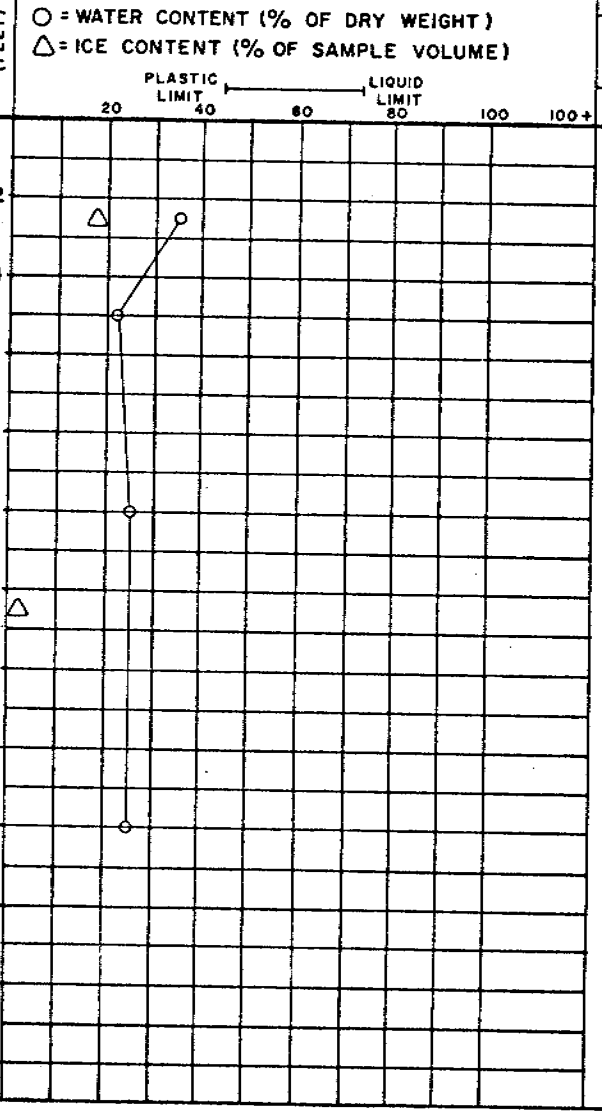
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED: 18/2/73	AIRPHOTO NO: A22774 - 54	CHAINAGE: 3456 + 80	OFFSET
CKD: GRG	TECH: JK	RIG: Texoma	SURFACE DRAINAGE: Poor	VEGETATION: Black Spruce & Birch	ELEV: 337.5'

TEST HOLE

MILE	B,C,S	NUMBER
659	C	4

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
2	1				OL to ML	SILT - Med. Brown - Organic - Low Plasticity	F	V 15-20%							
4 6	2				SP	SAND - Med. Brown - Trace of Silt - Medium - Some gravel		NB	(9)		78	13			
8 10 12 14 16 18	3 4				CI	CLAY - Grey - Silty - Med-Plasticity		NB V 0-5% NB							
20 22 24						END OF HOLE 18'									



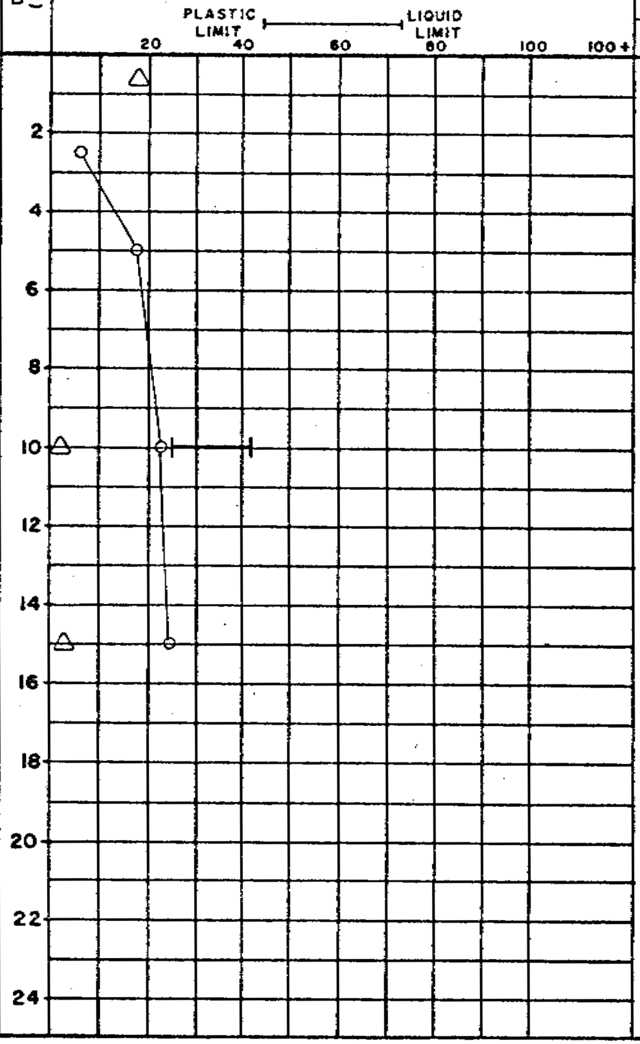
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY**

DWN: ALB
CKD LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED 24/2/73
RIG: Mayhew
AIRPHOTO NO: A22774 - 55
CHAINAGE: 3446 + 20
OFFSET
SURFACE DRAINAGE: Good to South
VEGETATION: Black Spruce & Birch
ELEV: 324.9'

TEST HOLE MVPL16		
MILE	B,C,S	NUMBER
659	S	1

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					Pt	PEAT-Reddish Brown, Fibrous		V.- 15-20%
2	1	▧			GW	GRAVEL - Sandy, Med. Brown Loose, Well Graded		NF
4	2	▧			ML to CL	SILT (TILL) - Med. Brown, Sandy, Low Plasticity		
6 8 10	3	▧			CI	CLAY - Grey - Some Silt - Med. Plasticity	F	V - 0-5%
12 14 16 18	4	▧				- A Few Isolated Pebbles		
20 22 24						END OF HOLE 19'		

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



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

REMARKS

North Bank Elliot

Loose gravel sloughing. Soil would not blow out of hole.

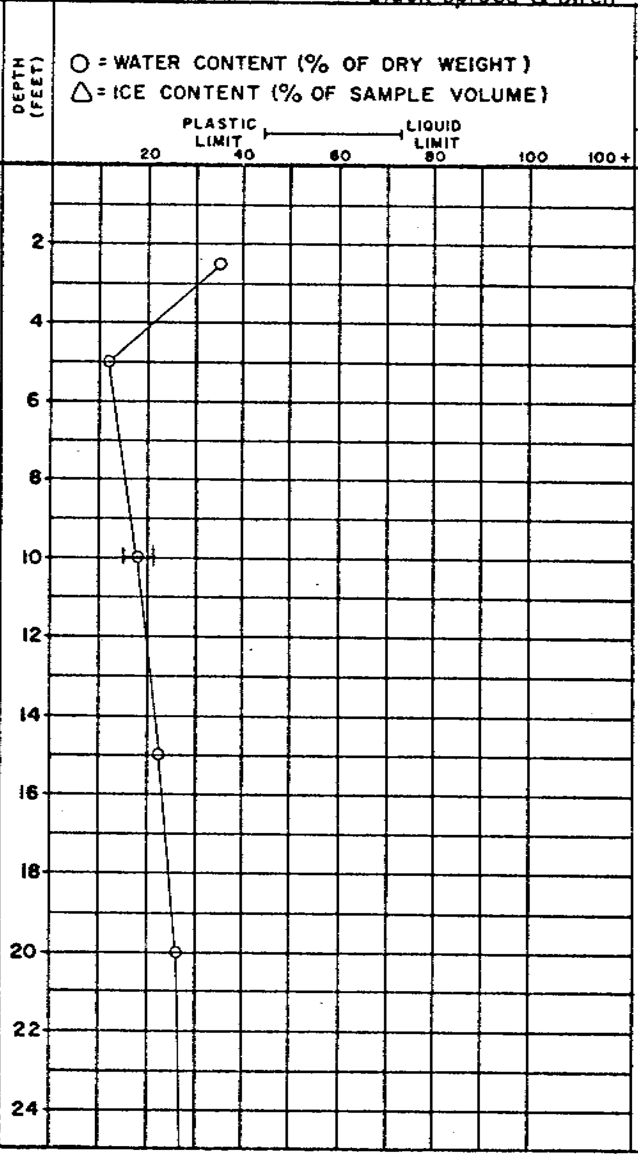
Page 1 of 2

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

DWN: ALB
CKD: LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED: 24/2/73
RIG: Mayhew
AIRPHOTO NO: A22774 - 55
SURFACE DRAINAGE: Good to North
CHAINAGE: 3449 + 22
VEGETATION: Black Spruce & Birch
OFFSET
ELEV: 325.3'

TEST HOLE		
MILE	B,C,S	NUMBER
659	S	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.)	REMARKS
2 4 6	1 2	⧅ ⧅			SP	GRAVEL - Med. Brown - Sandy - Poorly Graded - Dense - Trace of Silt and Clay									South Bank Elliot
8					SF	SAND - Med, brown gravelly & silty - Trace of Clay		NF to NB							
10	3	⧅							8	20	55	17			
12 14 16 18 20 22 24	4 5	⧅ ⧅			CI	CLAY - Grey - Some Silt - Med. Plasticity - A Few Isolated Pebbles	F	V 0-5%							



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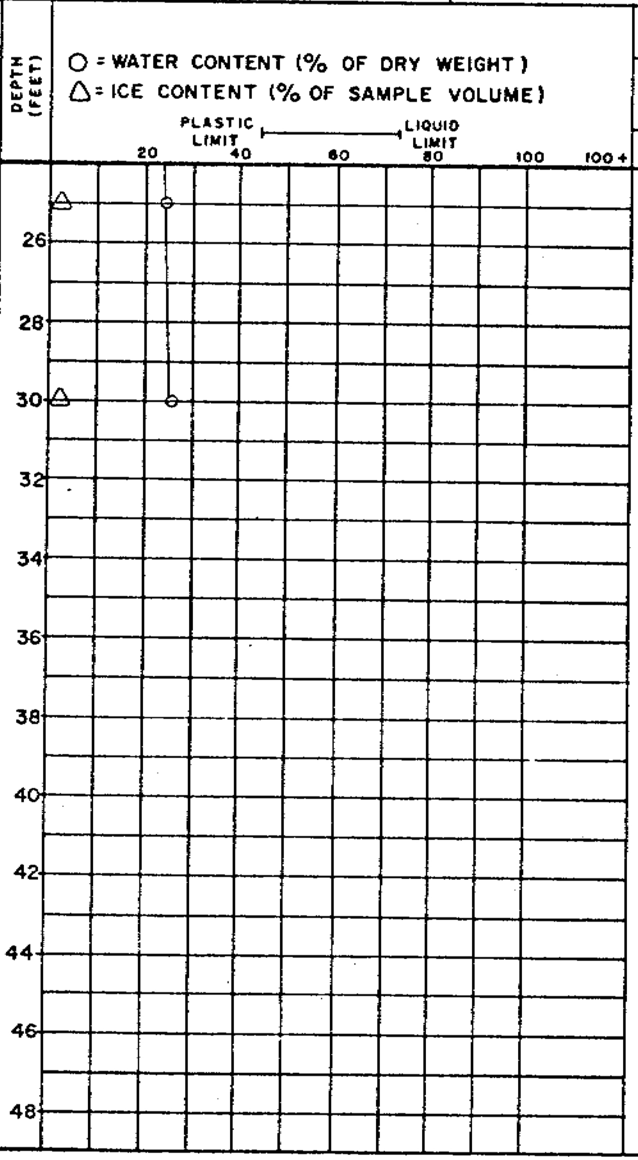
E.W. BROOKER & ASSOCIATES LTD. — DRILL HOLE REPORT — DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB
FIELD ENG: NRM
DATE DRILLED 24/2/73
AIRPHOTO NO: A22774 - 55
CHAINAGE: 3449 + 22
OFFSET
CKD LAB
TECH: JK
RIG: Mayhew
SURFACE DRAINAGE: Good to North
VEGETATION: Black Spruce & Birch
ELEV: 325.3'

TEST HOLE

MILE	B,C,S	NUMBER
659	S	2

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
26					CI	CLAY - Same as above	F	V-0-5%							
28															
30	6					END OF HOLE 30'									Sloughing in hole. Could not drill past 30'
32															
34															
36															
38															
40															
42															
44															
46															
48															



E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

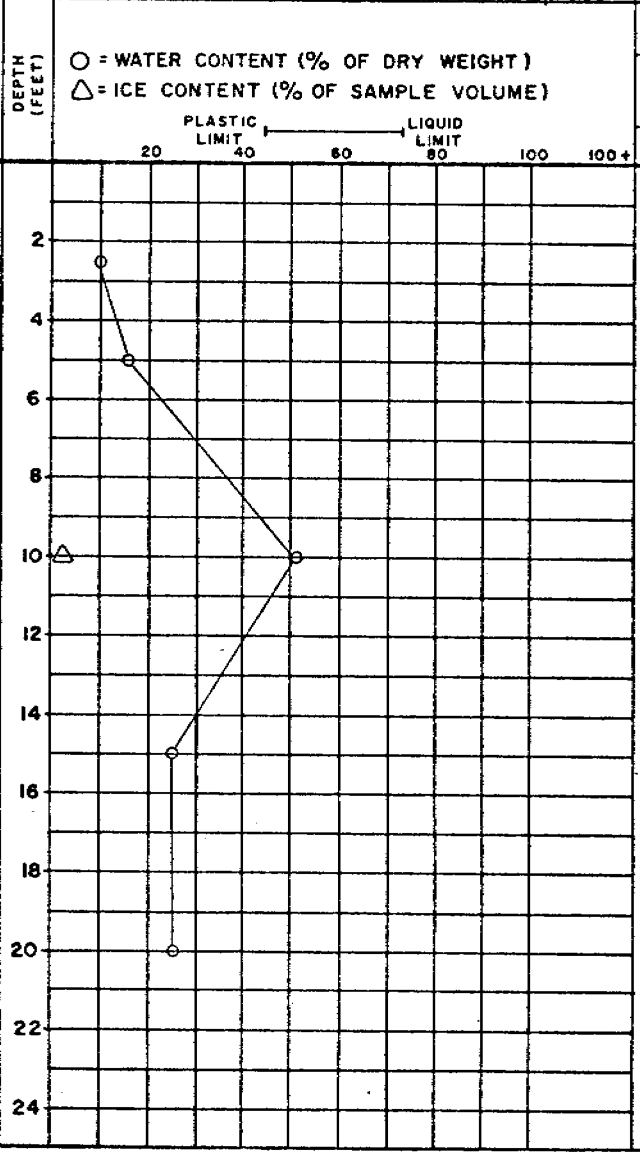
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 25/2/73	AIRPHOTO NO: A22774 - 55	CHAINAGE: 3442 + 50	OFFSET
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Good to South	VEGETATION: Birch & Black Spruce	ELEV: 334.7'

TEST HOLE

MILE	B,C,S	NUMBER
659	S	3

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
2	1				GW	GRAVEL- Med. Brown - Sandy - Loose - Well Graded		NF							Loose sand & gravel sloughing
4	2				SW to SF	SAND - Med. Brown - Silty - Gravelly, Trace of Silt - Dense	F	NB	(3)		70	27			
10	3					- Med. Brown - Some Clay - Silty - Low Plasticity		V 0-5%							
14	4				CI	CLAY - Grey - Silty - Med. Plasticity	U	Unfrozen							
20	5														
						END OF HOLE 20'									

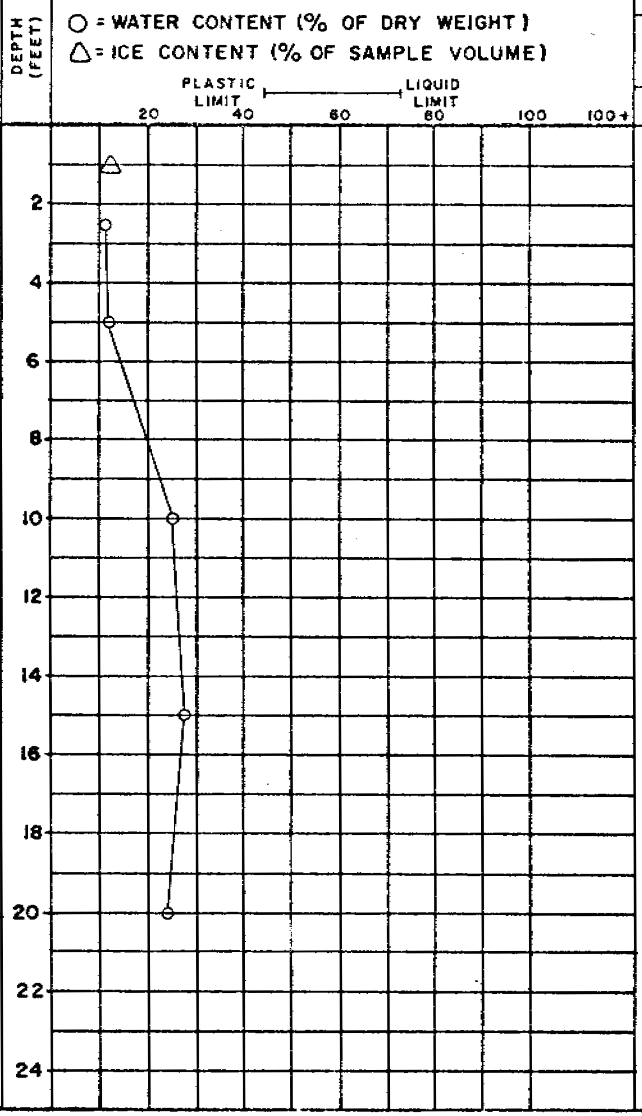


E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 3/2/73	AIRPHOTO NO: A22774 - 56	CHAINAGE: 3452 +62	OFFSET
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Good to North	VEGETATION: Black Spruce & Birch	ELEV: 335.2'

TEST HOLE		
MILE	B,C,S	NUMBER
659	S	4

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
0–2					Pt	PEAT - Reddish Brown - Fibrous		V 10-15%							Elliot Creek
2–8	1 2				GW	GRAVEL - Reddish Brown - Sandy, Some Silt - Loose - Well Graded		NF	(12)		36	52			
8–11	3				SF to SP	SAND - Med. Brown - Med. Grained - Loose, Silty - Poorly Graded	F								
11–20	4 5				CI	CLAY - Grey - Silty - Med. Plasticity - Trace of Fine Sand		NB							
20						END OF HOLE 20'									



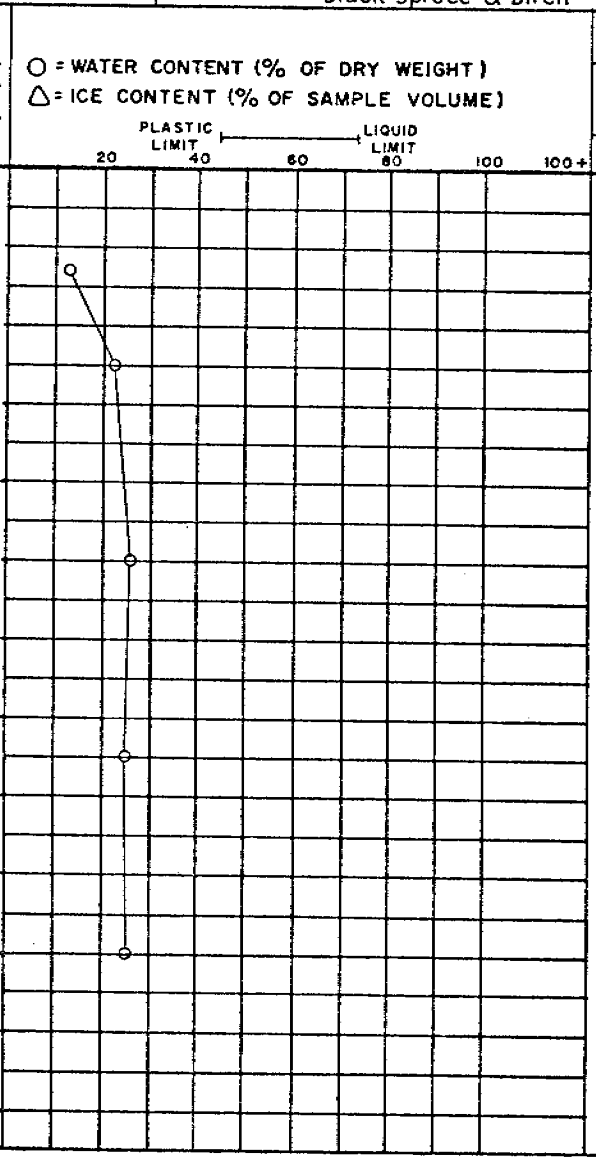
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN: ALB | FIELD ENG: NRM | DATE DRILLED: 3/2/73 | AIRPHOTO NO: A22774 - 56 | CHAINAGE: 3455 + 12 | OFFSET
CKD: LAB | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Fair to North | VEGETATION: Black Spruce & Birch | ELEV: 336.5'

TEST HOLE

MILE	B,C,S	NUMBER
659	S	5

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
0 – 10	1 2				SF	SAND - Med. Brown - Med. Grained - Silty - Dense - Gravelly from 2 - 5' - Fine		NF to NB							Elliot Creek
10 – 23	3 4 5				CI	CLAY - Grey - Silty - Med. Plasticity - With Some Fine Sand	F	NB							
23						END OF HOLE 23'									



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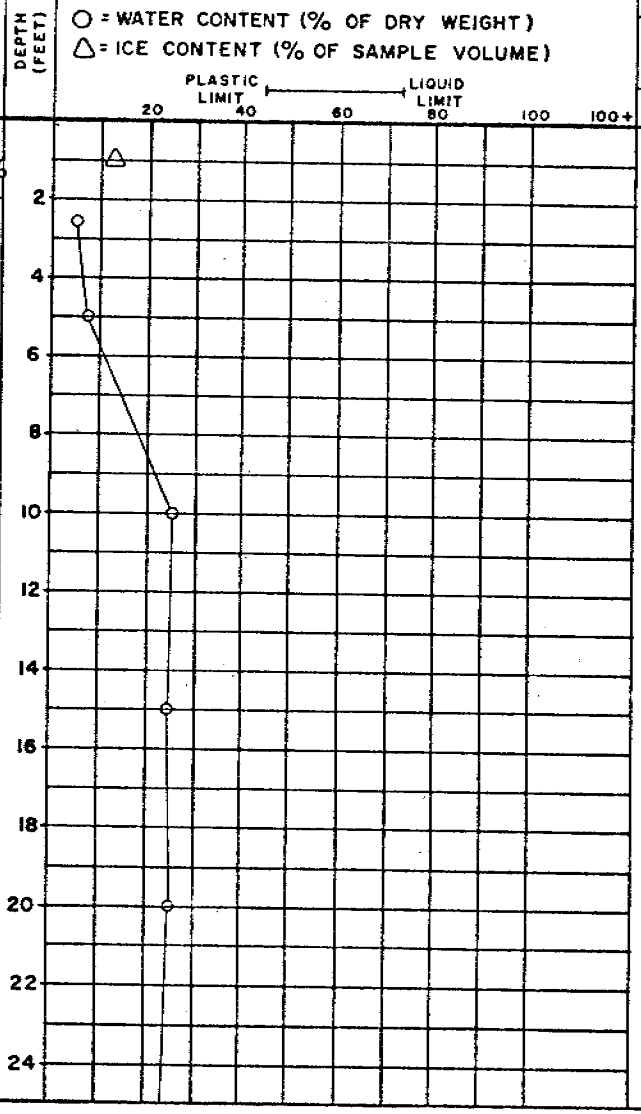
E.W. BROOKER & ASSOCIATES LTD. | DRILL HOLE REPORT | DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB | FIELD ENG: NRM | DATE DRILLED: 3/2/73 | AIRPHOTO NO: A22774 - 56 | CHAINAGE: 3451 +50 | OFFSET

CKD: LAB | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Good to North | VEGETATION: Black Spruce & Birch | ELEV: 333.0'

TEST HOLE		
MILE	B,C,S	NUMBER
659	S	6

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	PEAT - Reddish Brown - Fibrous		V - 10-15%							Elliot Creek
2 4 6	1 2	⧅ ⧅			GP	GRAVEL - Med. Brown - Sandy - Loose - Poorly Graded		NF							
8 10	3	⧅			SF	SAND - Med. Brown - Silty - Some Gravel - Fine to Med.	F		(23)		67	10			
12 14 16 18 20 22 24	4 5	⧅ ⧅			CI	CLAY - Grey SILT - Med. Plasticity		NB							



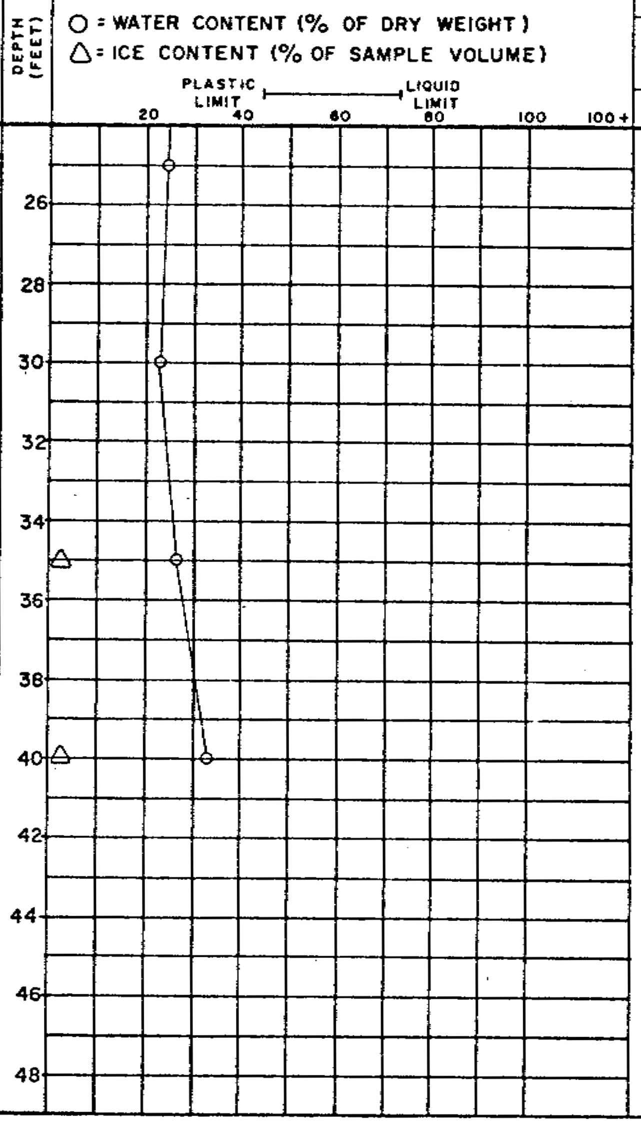
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY**

DWN: ALB	FIELD ENG: NRM	DATE DRILLED: 3/2/73	AIRPHOTO NO: A22774 - 56	CHAINAGE: 3451 + 50	OFFSET
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Good to North	VEGETATION: Black Spruce & Birch	ELEV: 333.0'

TEST HOLE

MILE	B,C,S	NUMBER
659	S	6

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
25	7	▧				CLAY SILT - Same As Above		NB
30	8	▧			CI		F	V - 0-5%
35	9	▧						
40	10	▧						
43						END OF HOLE 43'		



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

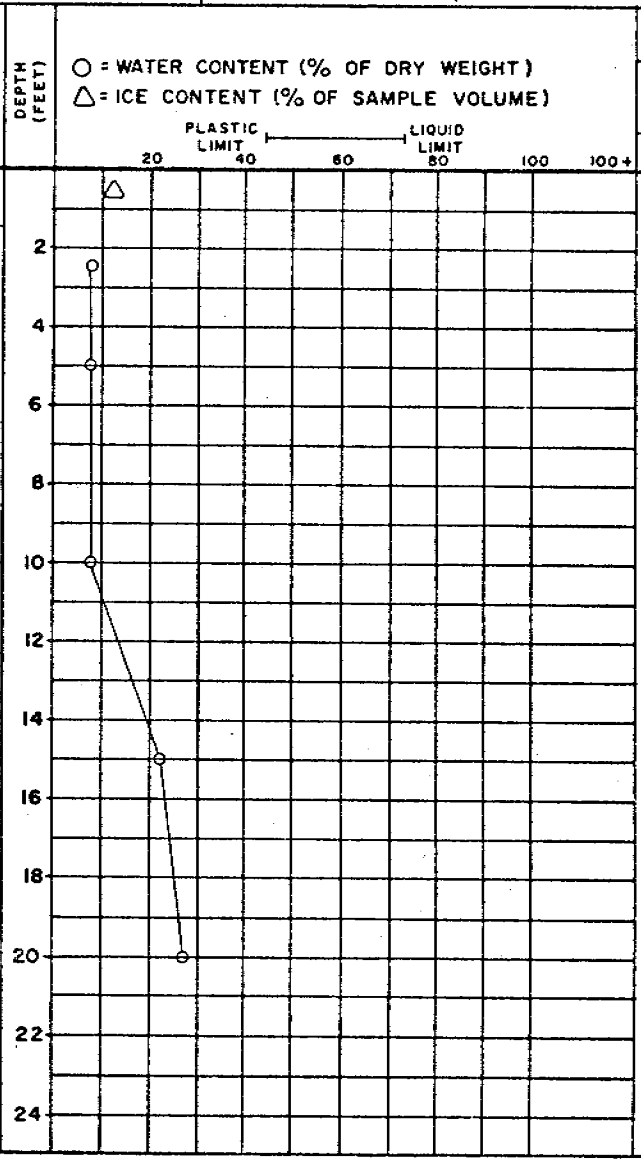
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN ALB | FIELD ENG NRM | DATE DRILLED 3/2/73 | AIRPHOTO NO: A22774 - 56 | CHAINAGE 3450 + 43 | OFFSET

CKD LAB | TECH IK | RIG Mayhew | SURFACE DRAINAGE Good to North | VEGETATION Black Spruce & Birch | ELEV 330.0'

TEST HOLE		
MILE	B,C,S	NUMBER
659	S	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	PEAT - Reddish Brown - Fibrous		V - 10-15%							Elliot Creek
	1 2				GP to SP	GRAVEL- Med. Brown SAND - Loose - Poorly Graded		NF							
	3				SP	SAND - Med. Brown - Fine, Uniform - Dense	F								
	4 5				CI	CLAY - Grey - Silty - Med. Plasticity		NB							
						END OF HOLE 23'									



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

DWN: ALB | FIELD ENG: NRM | DATE DRILLED: 3/2/73 | AIRPHOTO NO: A22774 - 56 | CHAINAGE: 3447 + 50 | OFFSET
CKD: LAB | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Good to South | VEGETATION: Black Spruce & Birch | ELEV: 309.0'

TEST HOLE		
MILE	B,C,S	NUMBER
659	S	8

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

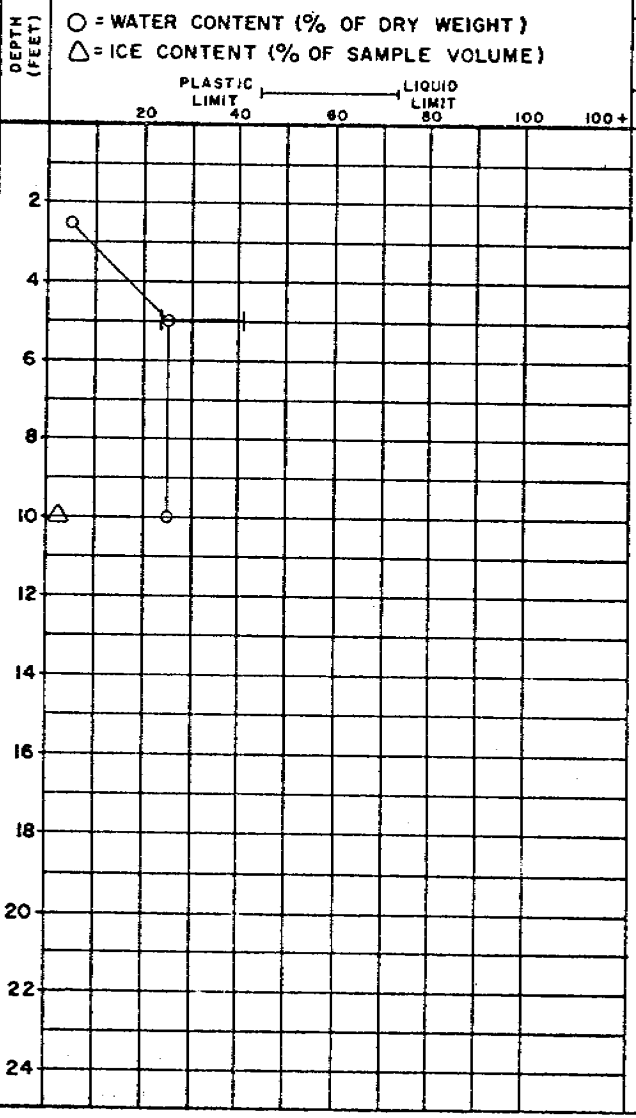
DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P C F)	DRY DENSITY (P C F)	REMARKS
2	1				GP	GRAVEL - Med. Brown - Loose - Med. to Coarse Sand - Poorly Graded	F	NF							Elliot Creek
4															
6					CI	CLAY - grey, silty, med. plastic									
8						END OF HOLE 5' (Loose gravel sloughing into hole)									
10															
12															
14															
16															
18															
20															
22															
24															

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

DWN: ALB
FIELD ENG: NRM
DATE DRILLED: 3/7/73
AIRPHOTO NO: A22774 - 56
CHAINAGE: 3447 +03
OFFSET:
CKD: LAB
TECH: JK
RIG: Mayhew
SURFACE DRAINAGE: Good to South
VEGETATION: Black Spruce & Birch
ELEV: 314.8'

TEST HOLE		
MILE	B,C,S	NUMBER
659	S	9

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0 – 3	1	▨			GP	GRAVEL- Med. Brown - Loose - Sandy - Poorly Graded		NF
3 – 9	2	▨			CI	CLAY - Grey, Silty - Med. Plasticity	F	NB
9 – 13	3	▨						V 0 - 5%
13						END OF HOLE 13'		



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

REMARKS

Elliot Creek

Page 1 of 2

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

DWN: ALB | FIELD ENG: NRM | DATE DRILLED 3/2/73 | AIRPHOTO NO: A22774 - 56 | CHAINAGE: 3444 + 83 | OFFSET

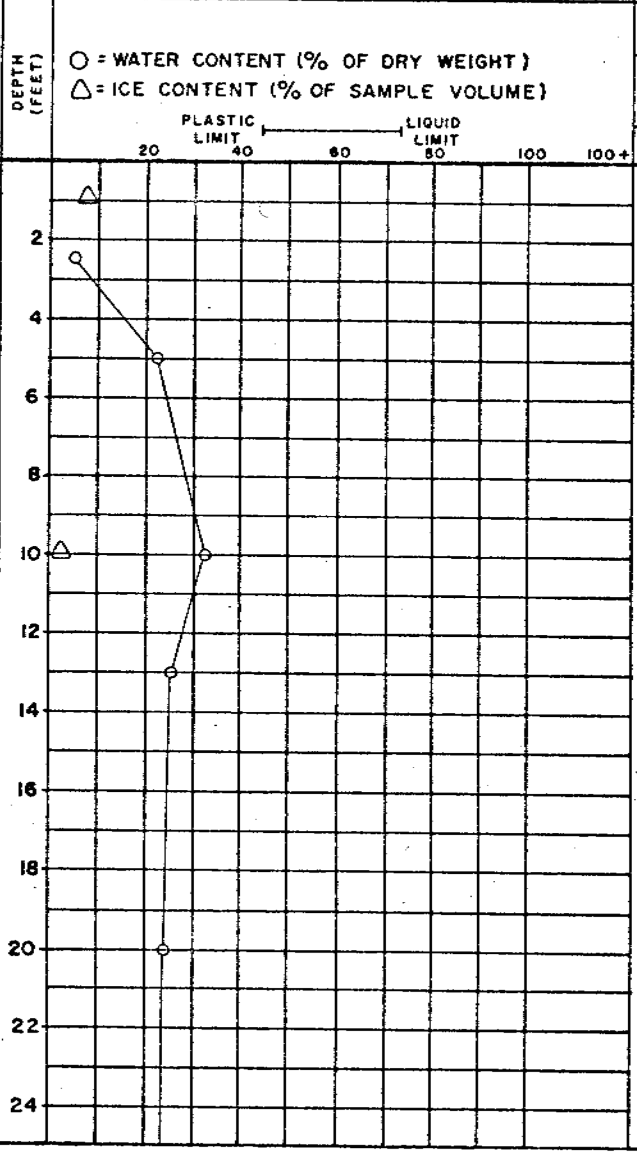
CKD LAB | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Good to South | VEGETATION: Black Spruce | ELEV 329.3'

TEST HOLE

MILE	B,C,S	NUMBER
659	S	10

REMARKS

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0 - 1					Pt	PEAT - Reddish Brown - Silty, Fibrous		V 5 - 10%
1 - 9	1 2				SP to SF	SAND - Med. Brown - Loose - Gravelly - Poorly Graded - Med. Brown - Silty - Fine - Dense - Uniform		NF NB
9 - 24	3 4 5				CI	CLAY - Grey - Silty - Med. Plasticity	F	V 0 - 5% NB



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

E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

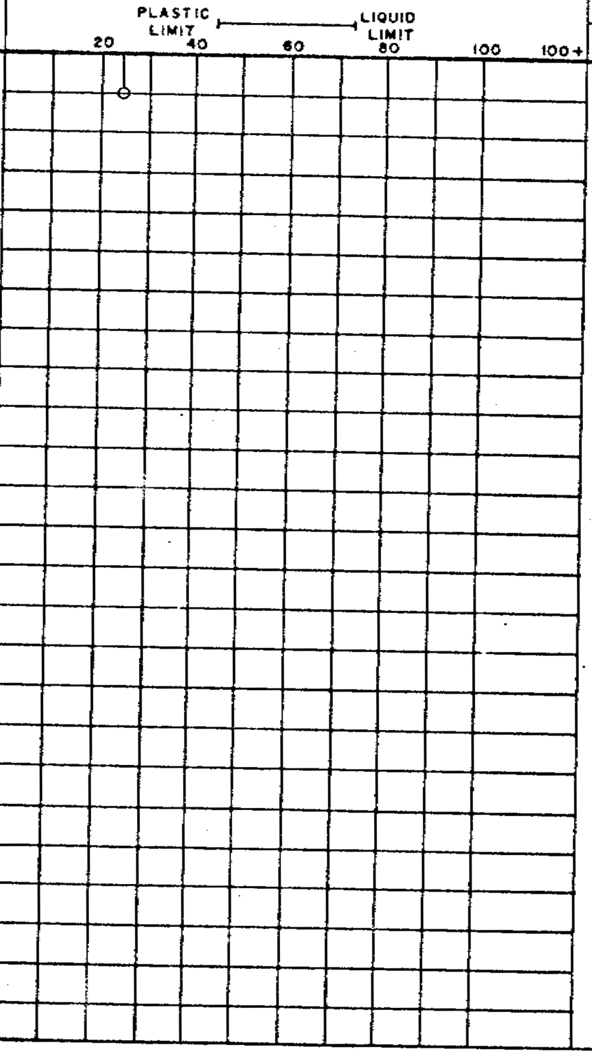
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB
CKD: LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED: 3/2/73
RIG: Mayhew
AIRPHOTO NO: A22774 - 56
CHAINAGE: 3444 + 83
OFFSET
SURFACE DRAINAGE: Good to South
VEGETATION: Black Spruce & Birch
ELEV 329.3'

TEST HOLE		
MILE	B,C,S	NUMBE
659	S	10

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
26	6				CI	CLAY - Same as above	F	NB							
28						END OF HOLE 28'									

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



E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

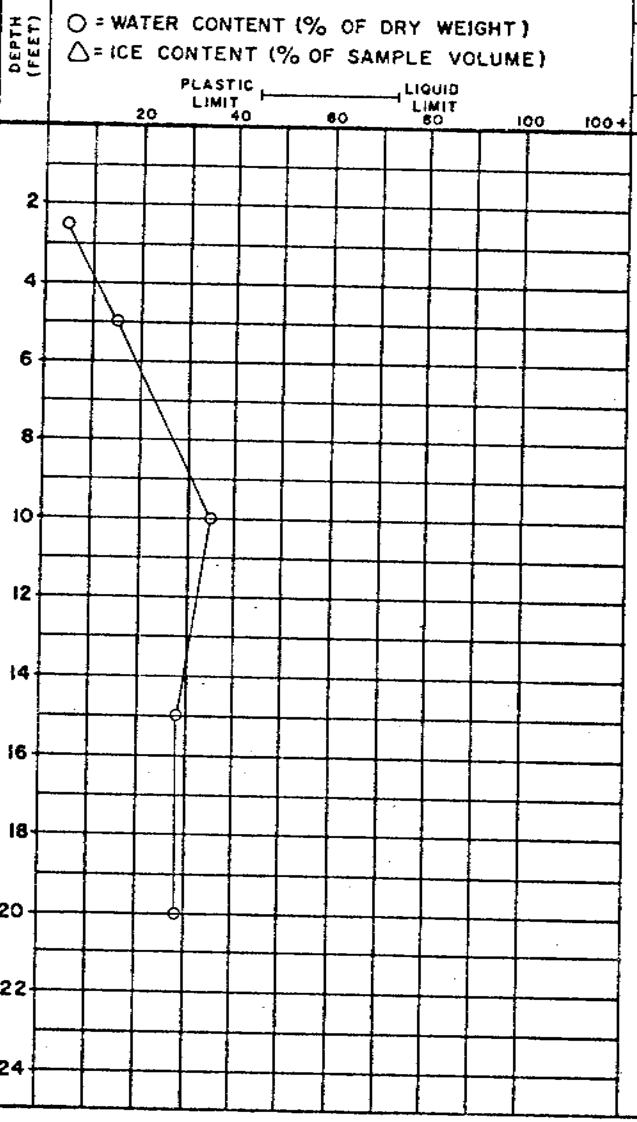
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB
CKD: LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED 5/2/73
RIG: Mayhew
AIRPHOTO NO: A22774 - 56
CHAINAGE: 3443 + 65
OFFSET
SURFACE DRAINAGE: Good to South
VEGETATION: Black Spruce & Birch
ELEV: 332.3'

TEST HOLE

MILE	B,C,S	NUMBE
659	S	11

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	REMARKS
0–5	1 2				SP to SF	SAND - Med. Brown - Gravelly - Loose - Poorly Graded		NF	Elliot Creek
5–10						- Med. Brown - Medium - Silty - Dense - Some Gravel		NF to NB	
10	3						F		
10–23	4 5				CL to CI	CLAY - Grey - Silty - Low - Med. Plasticity - Trace of Fine Sand		NB	
23						END OF HOLE 23'			



GRAIN-SIZE ANALYSIS

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

GRAIN SIZE DISTRIBUTION

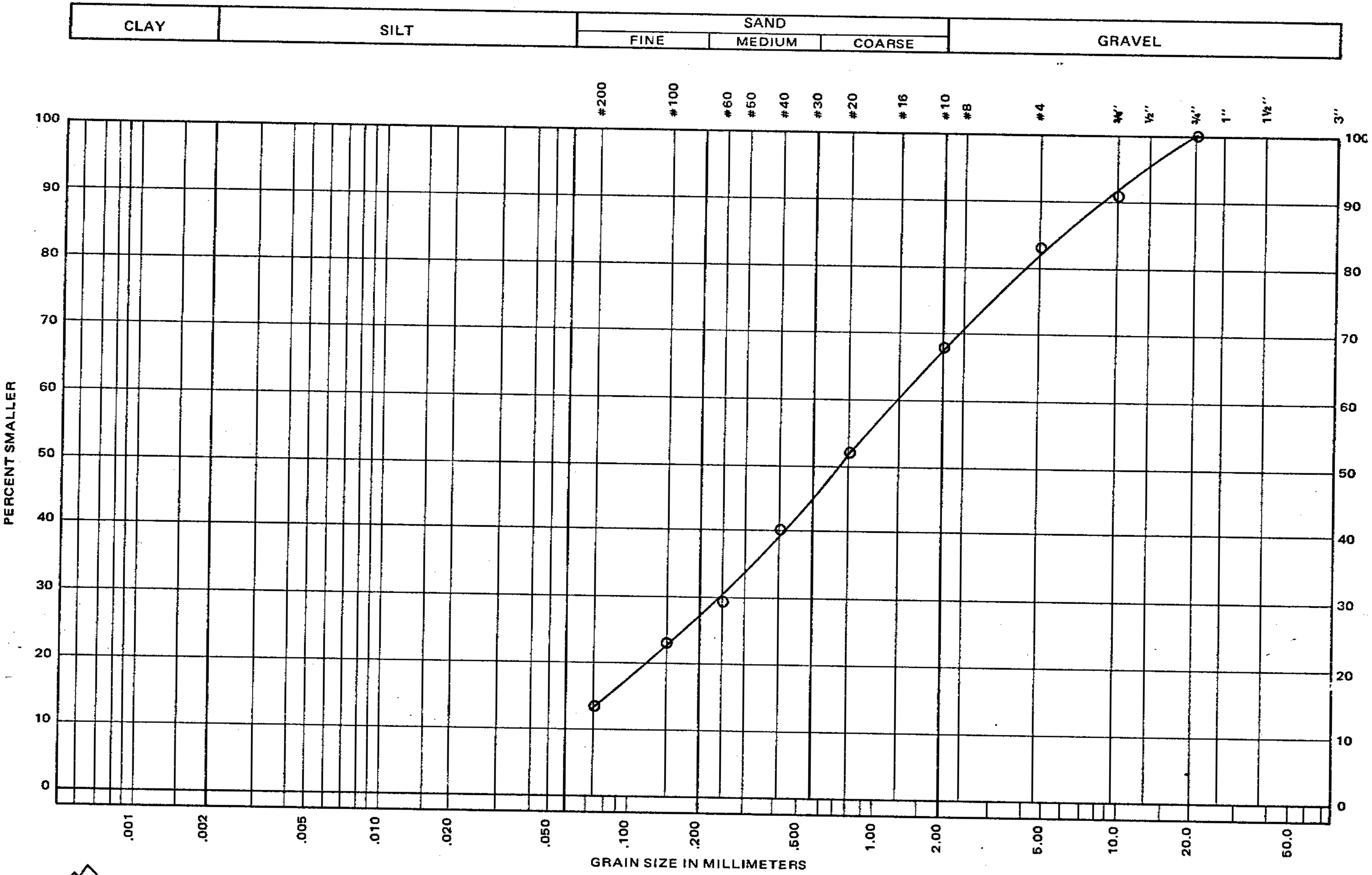


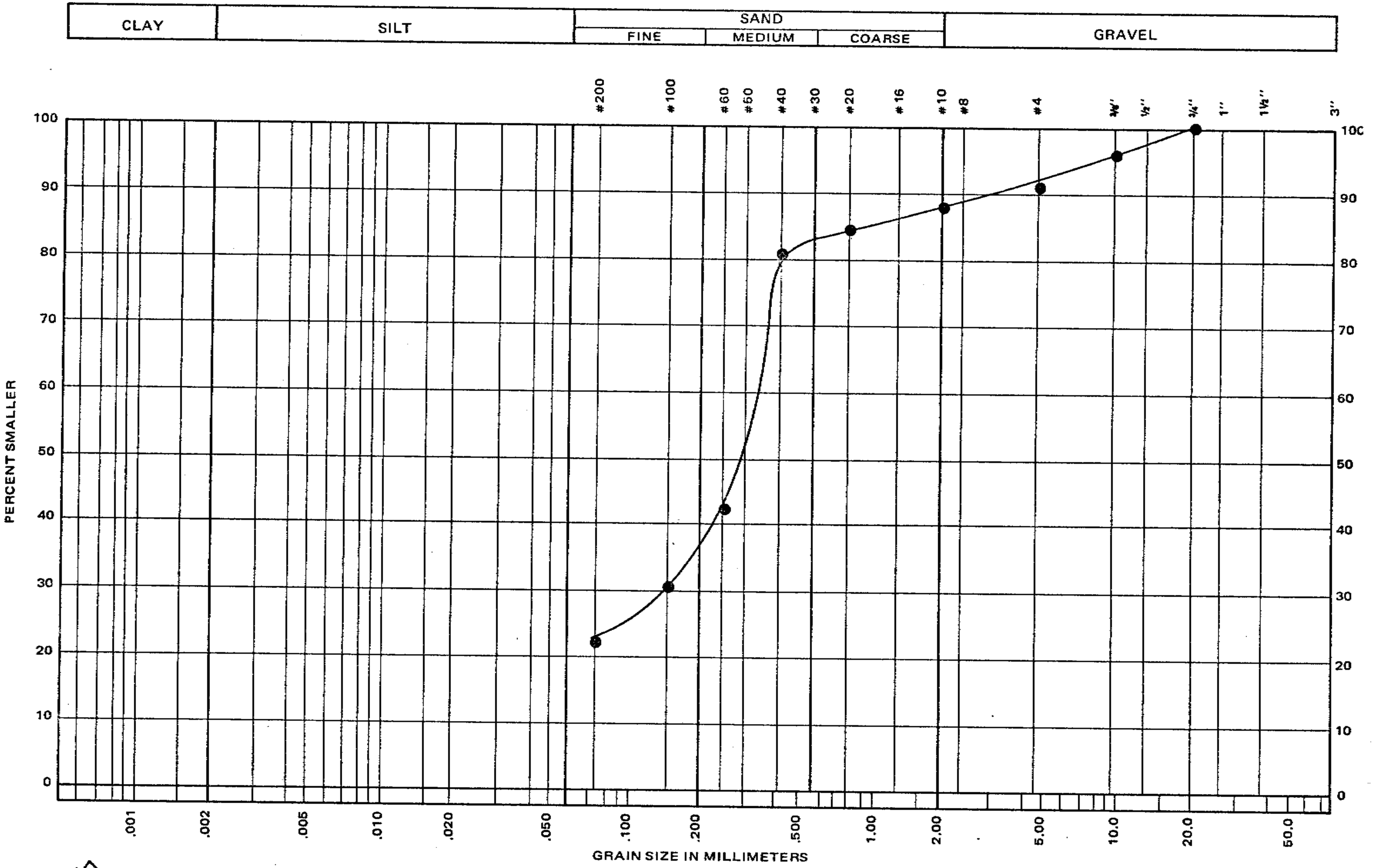
FIGURE C - 1

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION SAND, GRAVELLY, TRACE OF SILT & CLAY

PROJECT MACKENZIE HIGHWAY
JOB No. E517 DATE April 11/73
SAMPLE No. 659 - C - 2
DEPTH 5'

GRAIN SIZE DISTRIBUTION



E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION SILT, SOME CLAY

PROJECT MACKENZIE HIGHWAY
JOB No. E-517 DATE April 12.73
SAMPLE No. 659 - C - 4
DEPTH 5'

FIGURE C - 2

GRAIN SIZE DISTRIBUTION

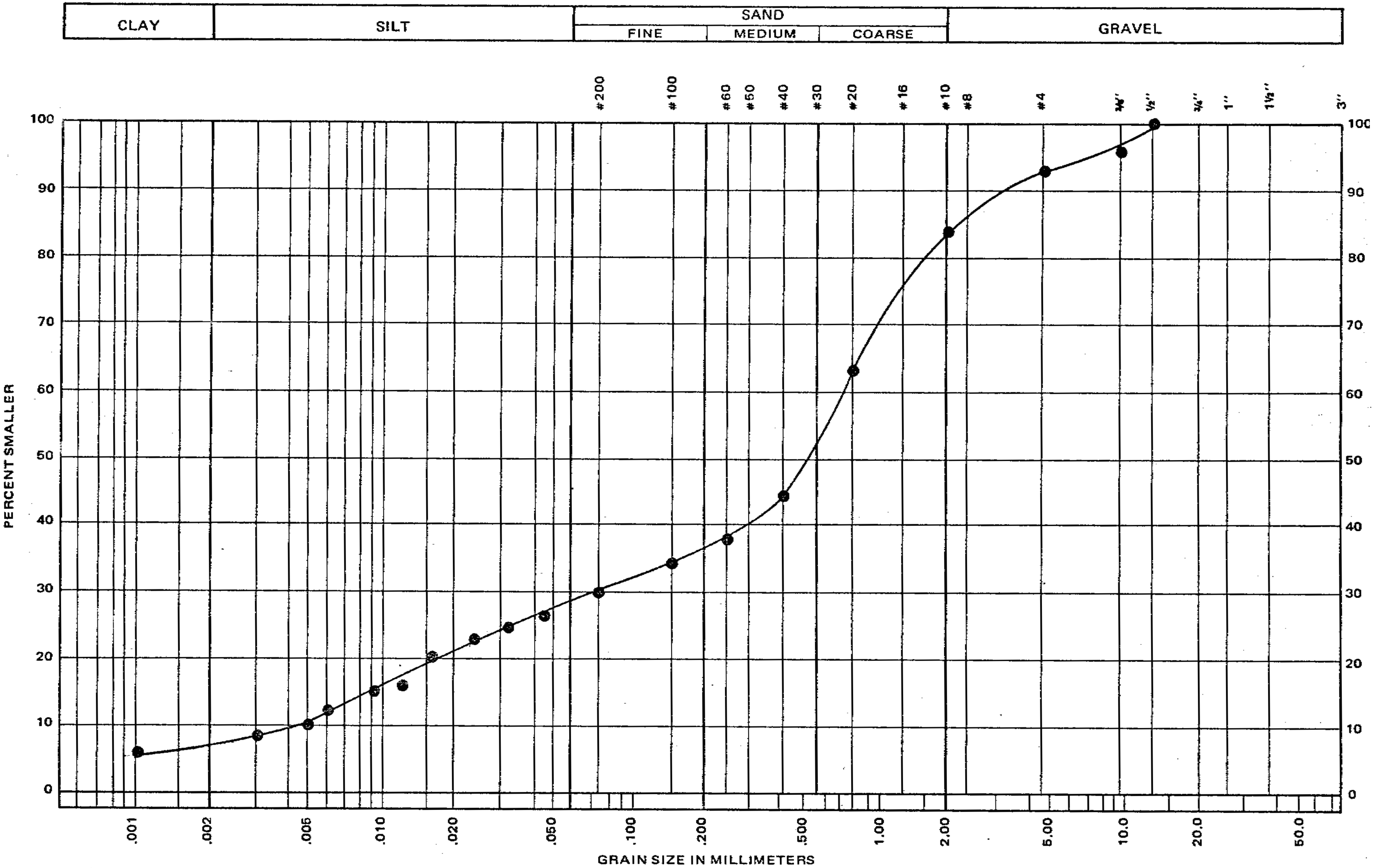


FIGURE C - 3

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION SAND, SILTY, SOME GRAVEL, TRACE OF CLAY

PROJECT MACKENZIE HIGHWAY
JOB No. E-517 DATE April 5/73
SAMPLE No. 659 - S - 2
DEPTH 10'

GRAIN SIZE DISTRIBUTION

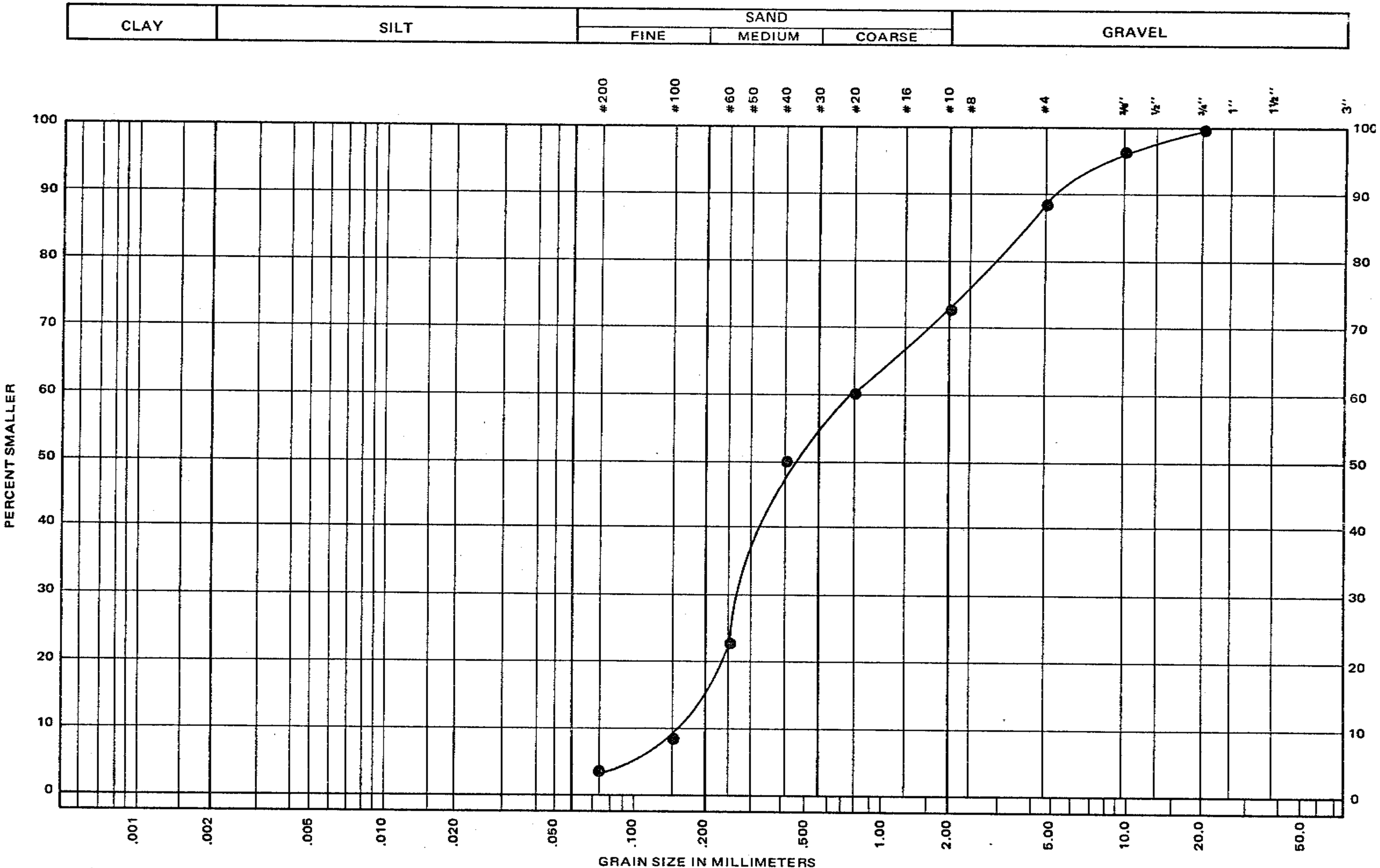


FIGURE C - 4

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION SAND, GRAVELLY, TRACE OF SILT

PROJECT MACKENZIE HIGHWAY
JOB No. E-517 DATE April 12/73
SAMPLE No. 659 - S - 3 MVPL
DEPTH 5'

GRAIN SIZE DISTRIBUTION

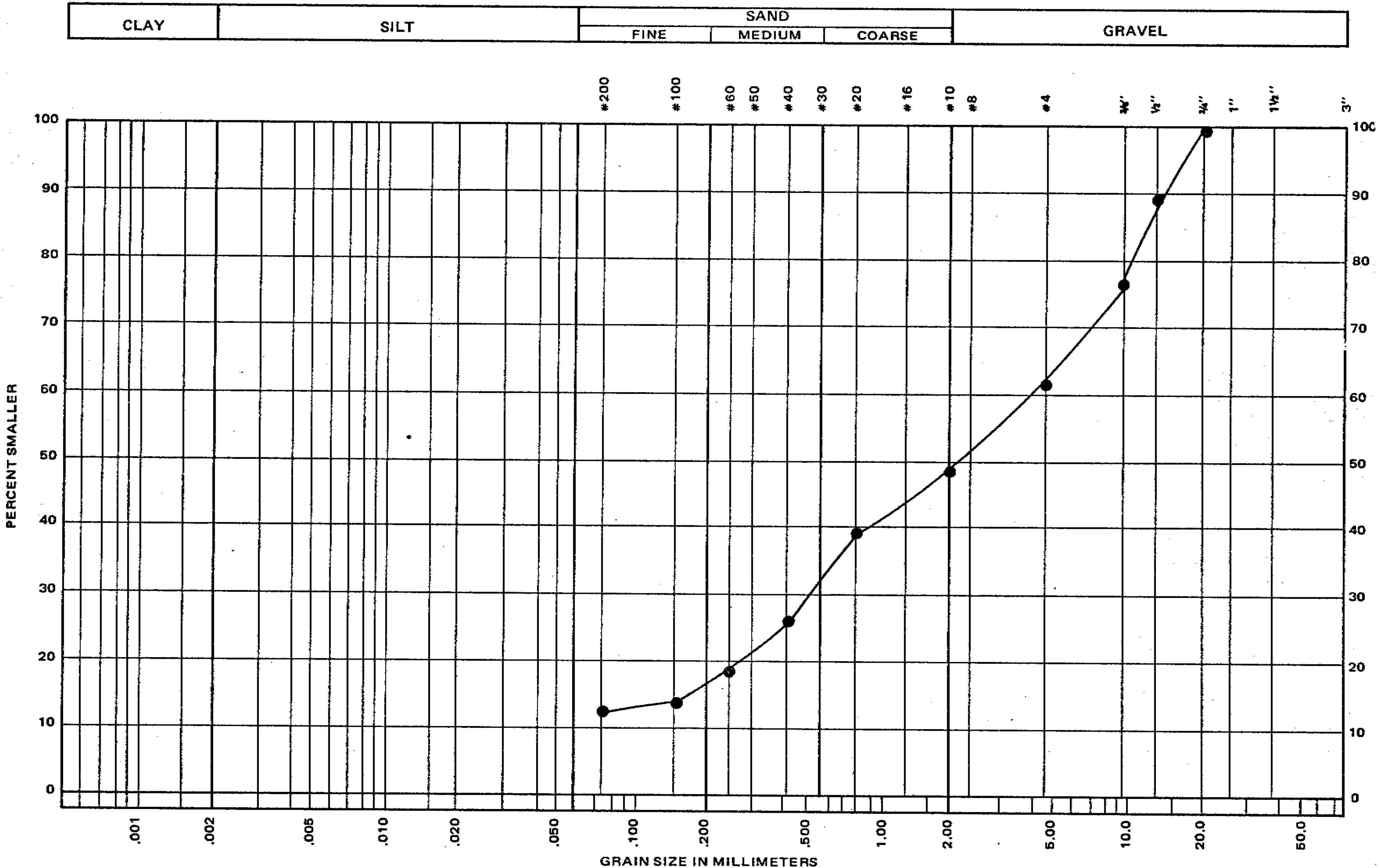


FIGURE C - 5

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION GRAVEL, SANDY, SOME SILT

PROJECT MACKENAIE HIGHWAY

JOB No. E-517 DATE April 9/73

SAMPLE No. 659 - S - 4

DEPTH 5'

GRAIN SIZE DISTRIBUTION

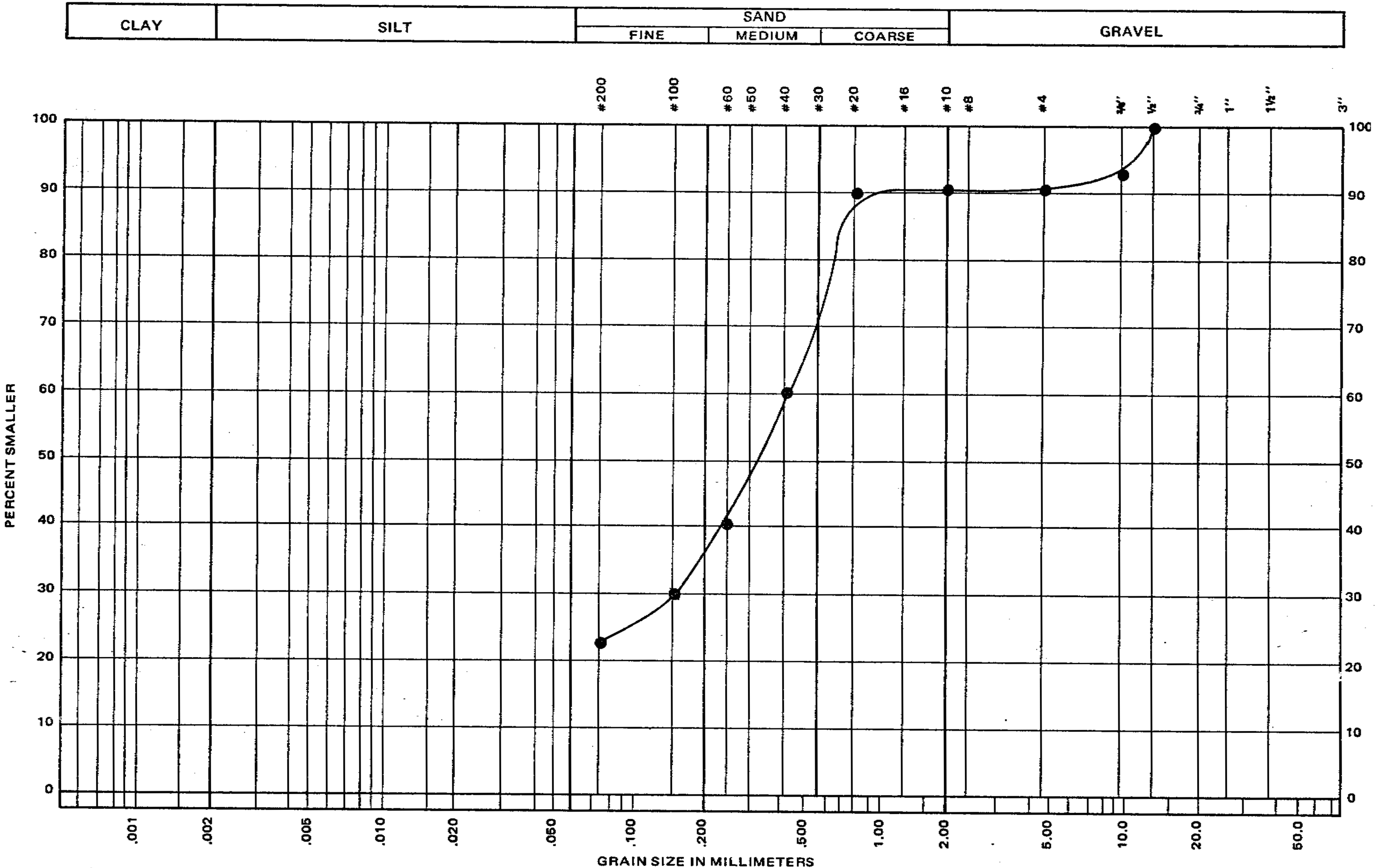


FIGURE C - 6

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION SAND, SILTY, TRACE OF GRAVEL

PROJECT MACKENZIE HIGHWAY
JOB No. E-517 DATE April 12/73
SAMPLE No. 659 - S - 6
DEPTH 10'

SUMMARY OF TEST RESULTS

JOB No. E - 517

ELLIOT CREEK CROSSING

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		
659-C-2	5					(1	1)	56	33	SW	
659-C-4	5						9	78	13	SP	
659-S-1	10	22.0	40.7	23.7	17.0					CI	
659-S-2	10	18.5	20.5	17.8	2.7	8	20	55	17	SF	
659-S-3	5					(3)		70	27	SW	
659-S-4	5					(1	2)	36	52	GW	
659-S-6	10					(2	3)	67	10	SF	
659-S-9	5	25.0	40.2	24.2	16.0					CI	Solube Sulphates 0.43%
											pH 6.5

DWG. No. C - 7

EBA ENGINEERING CONSULTANTS LTD.