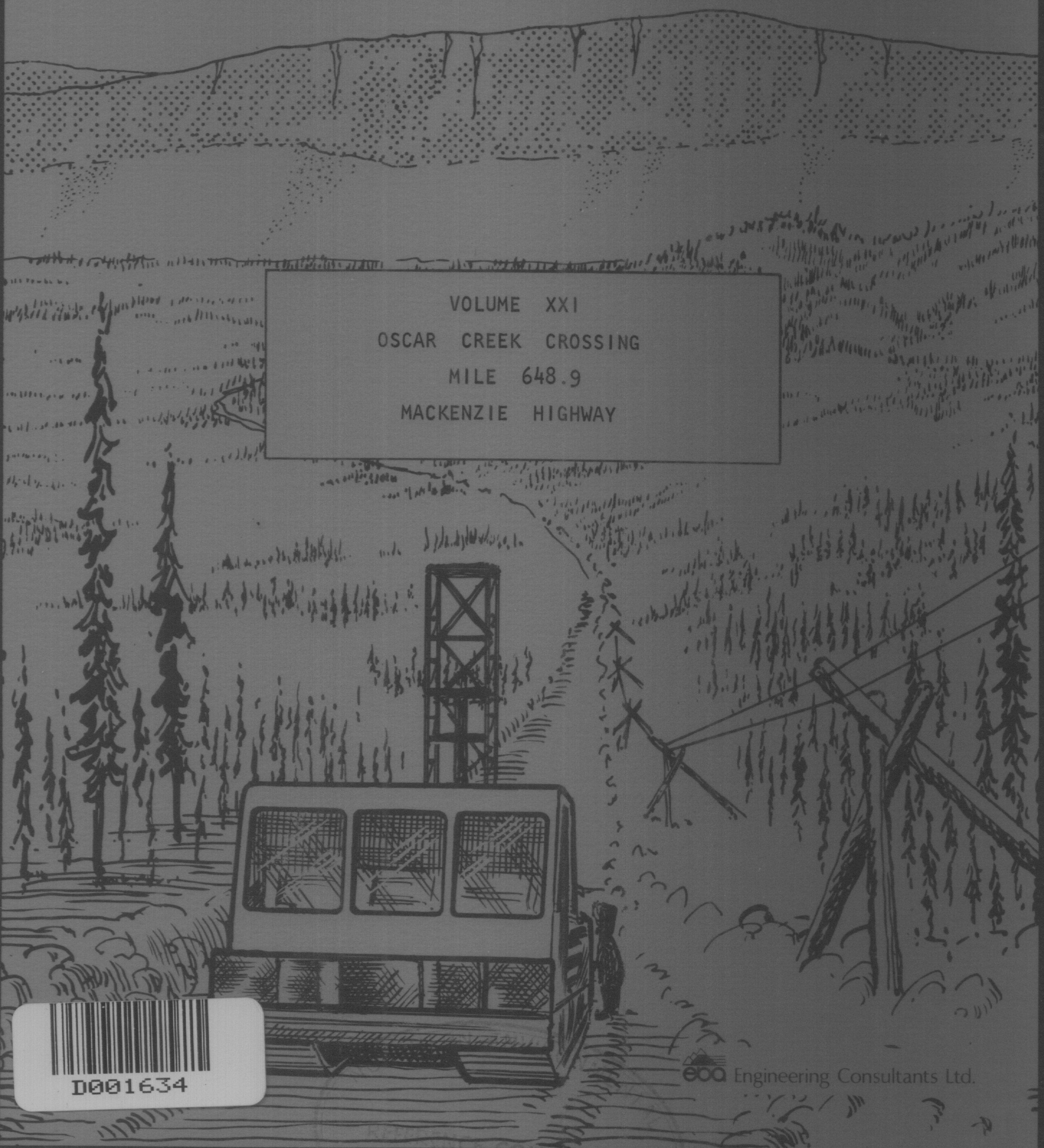
MACKENZIE HIGHWAY
GEOTECHNICAL EVALUATION



VOLUME XXI
OSCAR CREEK CROSSING
MILE 648.9
MACKENZIE HIGHWAY

eba Engineering Consultants Ltd.

THE ASSOCIATION OF
PROFESSIONAL ENGINEERS
OF ALBERTA
PERMIT NUMBER
P 245
E B A ENGINEERING
CONSULTANTS LTD.

MACKENZIE HIGHWAY
GEOTECHNICAL EVALUATION
VOLUME XXI
OSCAR CREEK CROSSING
MACKENZIE HIGHWAY

Submitted To:

GOVERNMENT OF CANADA
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E.W. Brooker & Associates 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, a number of major river and stream crossings were investigated. The Oscar 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 eight boreholes, drilled at the locations shown on Drawing No. A-2, Appendix A. Of the eight boreholes advanced, one was drilled as a center line borehole, in conjunction with the general route evaluation, and the remainder were located and drilled specifically to define subsurface conditions at the crossing.

The special boreholes consisted of Boreholes 650-S-2 to 650-S-8, inclusive. The single center line borehole was Borehole 650-C-7. Detailed borehole logs are presented in consecutive order in Appendix B.

All boreholes were drilled with a track mounted Mayhew 500 rotary drill rig, using a continuous air return circulation system. Boreholes advanced with this drill rig generally were 4-3/4 inches in diameter. Borehole penetration ranged from 18 feet to 58 feet, and averaged 34 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 organic content 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 the excess ice. The system used retains the symbols V and N for visible and non-visible ice, respectively, and the modifying symbols B and F for well bonded and poorly bonded non-visible ice, respectively. Excess ice quantities were estimated from visual observations. The results of laboratory tests are presented on the borehole logs (Appendix B), where applicable, and on grain size distribution curves, Drawings No. C-1 to C-5, inclusive, Appendix C. Drawing No. C-6 presents a summary of laboratory results.

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* Superscripted numbers 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 Oscar Creek at mile 648.9, approximately 19 miles north-west of Norman Wells. Drawing A-1, Appendix A, is a Key Plan of the Oscar Creek area and Drawing No. A-2, Appendix A, presents a detailed Site Plan. Plates 1 and 2, of Drawing No. A-3, Appendix A, show the crossing from the air in June 1973, and Plates 3 and 4, of Drawing A-4, Appendix A, show the ground conditions at the crossing in October, 1972.

Oscar Creek drains a large area extending north-east of Discovery Ridge and south-eastward past Norman Wells. The large watershed of Oscar Creek results in a substantial stream flow throughout the summer and fall. In the winter there is probably a continuous flow of water under the ice. The gravel bottom of Oscar Creek is considered to be suitable as a spawning area for several species of fish (4).

3.2 Subsurface Conditions

Based on observations from the boreholes, inferred stratigraphic sections along center line and on a single cross line have been compiled and are presented as Drawings No. A-5 and A-6, respectively, 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 OSCAR CREEK CROSSING

Material	Description	Approximate Depth Below Existing Grade (ft)	Average Range of Thickness (ft)
PEAT	black, fibrous, some sand, V-5%-15%	0	0 - 2
SAND	medium to fine grained, dense, some silt layers, medium brown, moisture content (M/C) = 12% - 55%, NB	0 - 2	4 - 8 (north bank) Generally 2' on south bank
GRAVEL	sandy, dense, well to poorly graded, silty layers, M/C = 8% - 12%, NB to NF	4 - 10 (north bank) 2 - 10 (south bank)	7 - 9 (north bank) 8 - 12 (south bank)
SILT	fine grained, sandy to clayey, grey, low plasticity, M/C = 22% - 38%, NB to V-5%	10 - 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 58 feet.
2. Partially unfrozen material was observed below the 25 foot depth in Borehole 650-C-7.
3. A sand layer 3 feet thick was noted at the base of the gravel in Borehole 650-S-5.

4. The main gravel layer appears as sand and gravel in Borehole 650-S-6.

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

The following foundation types are believed to be feasible for a bridge structure at the subject site. At present, preference has not been given to any of the suggested systems, and final selection of a foundation system should be determined in conjunction with economic and structural design considerations.

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

4.2 Foundation Design

A major factor affecting the design of pile foundations at Oscar Creek is the noted occurrence of an unfrozen zone within the subsoil, at the assumed north abutment location of the proposed bridge (BH 650-C-7). Generally the subsoils were noted to be frozen near the creek. However, the noted unfrozen zone near a potential bridge abutment location and 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 have to be employed.

Because of a lack of data, with respect to soil strength and depth to a 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 Oscar 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 58 feet, with bedrock not being encountered.

Based on a review of bedrock geology of the area, it is believed that shale and/or sandstone bedrock of Upper to Middle Devonian Age (5) may be expected to be at a depth of about 100 feet, below existing grade at the approximate abutment locations of the proposed bridge crossing. As this depth of support is readily reached with steel piling, 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 locations of bridge abutments and piers is a necessary prerequisite to the determination of a final design pile capacity. It is believed that this could be readily achieved through the use of drilling equipment available locally in the Norman Wells area.

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. Although the design length of the piles must be confirmed on the basis of additional field drilling, it is believed that a pile length of about 100 feet below existing grade, should be considered for preliminary design and estimating purposes.

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 is anticipated to meet with high resistance. Confirmation of this, however, could be achieved through the driving of test H-piles to bedrock.

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 the geotechnical consultant. A pile load test is also recommended prior to, or at the outset of, pile installation to confirm the load capacity of the piles and permit a correlation to the driving records.

4.2.2 Friction Piles

Based on available geotechnical information at the Oscar 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. In this regard, it is believed that this information could be readily obtained through the use of drilling equipment locally available in Norman Wells.

Alternatively, the following preliminary 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 fill be placed to final grade, before the piles are driven or pre-bored, in order to prevent damage to the pile and to ensure proper compaction of the fill, which will limit negative skin friction loads on the pile. 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 has been realized.

b. Prebored Driven Closed End Steel Pipe Piles

Steel pipe piles, installed in prebored holes, may also be used 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 fill be placed to final grade before the piles are driven or pre-bored in order to prevent damage to the pile and ensure proper compaction of the fill, which will limit negative skin friction loads on the pile.

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.

c. Pre-bored Driven Timber Piles

For preliminary design purposes, it is recommended that No. 12 Douglas Fir timber piles, with a minimum length of 70 feet (about 10 feet of fill assumed at abutments), be considered for foundation support. The piles should be pressure treated with creosote and should have a minimum creosote retention of 12 pounds per cubic foot.

It is essential that the fill be placed to final grade before the piles are driven or pre-bored to prevent damage to the pile and to ensure proper compaction of the fill, which will limit negative skin friction loads on the pile. Prebored holes should have a maximum diameter equivalent to the pile tip diameter to permit a snug fit, and should extend for the full depth of anticipated pile penetration. If excessive driving resistance is encountered, overboring of approximately the upper half of the hole may be required. Details of the overbore should be determined in the field, on the basis of interpretation of driving records obtained from test piles.

If the suggested pile section can be driven, with an energy of 24,000 foot pounds, to the full pile length, it is believed that an allowable static design load of 30 kips may be used. Piles must penetrate to at least the full pre-bored depth.

Field inspection by qualified personnel is considered necessary during the preboring and driving of test piles or the first few foundation piles, to establish or alter the preliminary design capacity and method of installation recommended herein. A driving record should be kept, for all piles installed, 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 soil. At the crossing site, it is considered that all subgrade materials are thaw unstable and consequently significant negative skin friction effects can be anticipated in these materials if thawing is allowed and settlement 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 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.

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 (6) for negative skin friction in typical soils. At the Oscar Creek site 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

TABLE 4.3.1

NEGATIVE SKIN FRICTION OF UNFROZEN SOIL FOR PILE DESIGN

(After Woodward Lundgren And Associates, 1971) (6)

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

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* 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.

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 skin friction of unfrozen soil zones in which the pile is embedded.

In order to prevent pile heave, it is necessary to check the 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 undesireable 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 (6) 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 the creek is similar. Generally, 4 to 8 feet of sand on the north bank, and 2 feet of sand on the south bank, overlies 7 to 12 feet of relatively

TABLE 4.4.1

FROZEN SOIL ADFREEZE BOND STRENGTH FOR PILE DESIGN

(After Woodward Lundgren And Associates, 1971) (6)

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.

dense gravel. The gravel overlies sandy clayey silt which extends to the maximum depth of drilling. Estimated visual excess ice contents are generally low. Moisture contents, however, are high and it is expected that soft conditions will probably exist in unfrozen soils during the summer season. A winter construction program is, therefore, considered to be the most desireable.

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, loose, low shear strength conditions will exist in the sand; moderate shear strength can be expected in the gravel; and, to a depth of 25 feet, low shear strength will exist in the silt. Generally below 25 feet the moisture content in the silt is low enough that the silt is expected to have a moderate shear strength, on thawing.

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 1.5 to 2.5 feet for summer construction. This estimate assumes thawing of subgrade soils, but does not take into account normal consolidation settlement of the unfrozen subgrade soils due to the surcharge effects of the road bed fill. In the case of peat soils, normal consolidation settlement can easily reach 50 percent of the original thickness of the deposit; and can, as with thaw settlement, occur fairly rapidly.

It is considered that the conventional northern construction practice of placing fill material directly on the organic subgrade is desireable 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 material. 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 Oscar Creek. Local fine grained materials such as silts and clays 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 detrimental disturbance to the existing ground cover and slopes by the construction equipment. Snow clearing should be carried out prior to all fill placement. Placement of the fill should be carried out through end dumping with subsequent spreading by dozing equipment. Minimum initial lift thicknesses of 2 feet are suggested. Depending on construction completion schedules, construction 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 impossible to estimate the maximum depth of scour, but the presence of the gravel indicates that high stream velocities occur at peak runoff and significant depths of scour may occur.

It was noted at the time of drilling (first week of March, 1973) that the creek bed was covered with ice but there was water flowing under the ice cover. It is probable that water continues to flow throughout the winter. Therefore, a potential icing problem may develop with respect to winter construction and maintenance.

4.6 Slope Stability Considerations

The only noted evidence of recent slope instability is a river steepened bank, approximately 1300 feet upstream from the proposed crossing. The estimated angle of repose, for the material exposed at this slide, is approximately 26 degrees.

As indicated in Subsection 3.1, the slopes of the meander plain on either side of the crossing are less than 1 degree and the slopes of the banks are 14 and 11 degrees for the north and south banks, respectively. These slopes are significantly less than the angle of repose in the noted unstable section of slope. Therefore, these slopes are expected to be stable at the creek crossing.

It is considered that rip-rap protection of the existing defined channel, upstream and downstream of the bridge crossing, may be necessary to protect the overall stability of the crossing site. Bridge abutments should be set as far back from the channel banks as is practicable. Fine grained fills should not be used for subgrade construction on the flood plain as they may be washed out if the river floods.

4.7 Drainage Considerations

Approach fills will concentrate runoff water along the upslope sides 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

Representative samples from the crossing area were not available for testing to determine soluble sulphate concentrations and soil acidity. However, based on the site conditions and samples tested from the area, it is considered that undesireable soluble sulphate concentrations are possible. 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 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 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 would be desireable 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 desireable.

In addition to the desireability 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 future designs of pile foundation systems could be established.

V. LIMITATIONS

The foregoing recommendations have been prepared based on our knowledge of existing conditions at Oscar 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.

However, due to the general nature of the study, 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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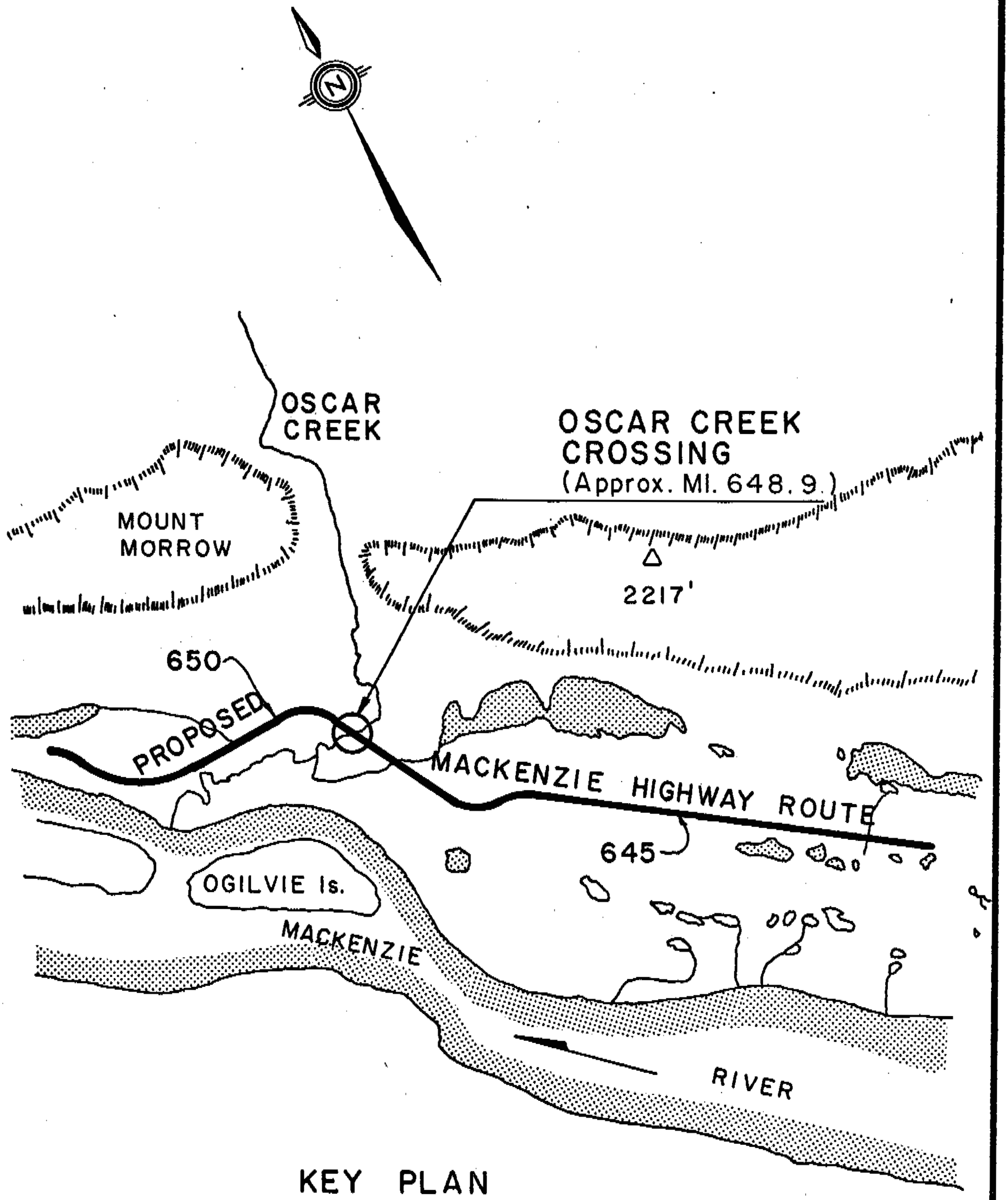
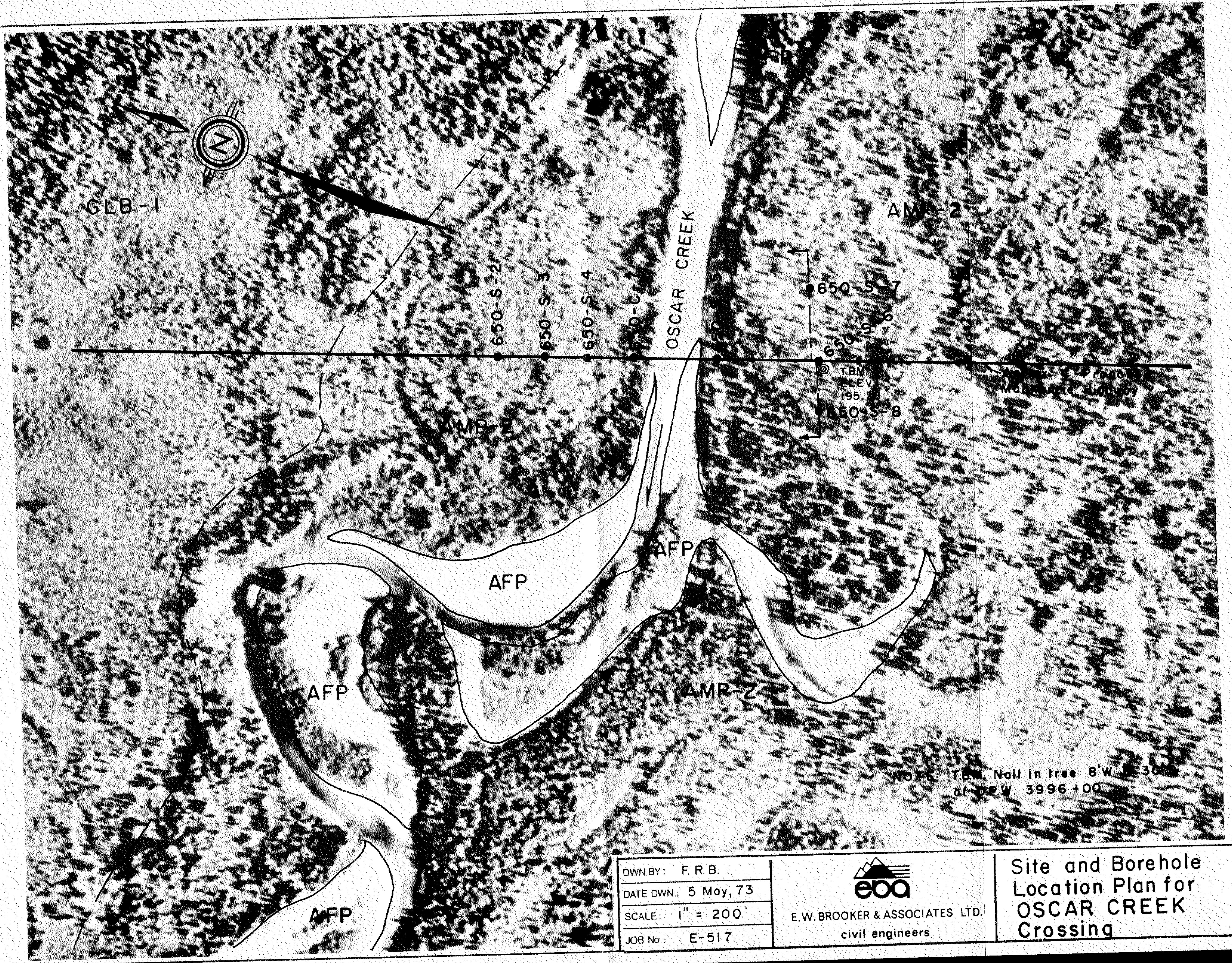


KEY PLAN

EBA Engineering Consultants Ltd.	DATE	SCALE	JOB No.	DWG No.
	2 JAN. 74	1" = 4 mi	E-517	A-1



DWN.BY: F. R. B.
DATE DWN.: 5 May, 73
SCALE: 1" = 200'
JOB No.: E-517

E. W. BROOKER & ASSOCIATES LTD.
civil engineers

Site and Borehole Location Plan for OSCAR CREEK Crossing

DWG.: A – 2
SHT.No.:

TERRAIN LEGEND

Symbol	Terrain Type	Physiographic Features	Materials Description
AFP	Active Flood Plain	Exposed bars in stream or river channel	Sand and gravel in high energy streams to silt and sand in low energy streams
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
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

Drawing No. A-2a

PLATE No. 1

Oscar Creek Crossing. The central cut line is the proposed highway center line. North is to the left. (June, 1973)

PLATE No. 2

Oscar Creek Crossing North is to the right. The cut line is the center line of the proposed highway. (June, 1973)

Drawing No. A-3

E.W. Brooker & Associates Ltd.

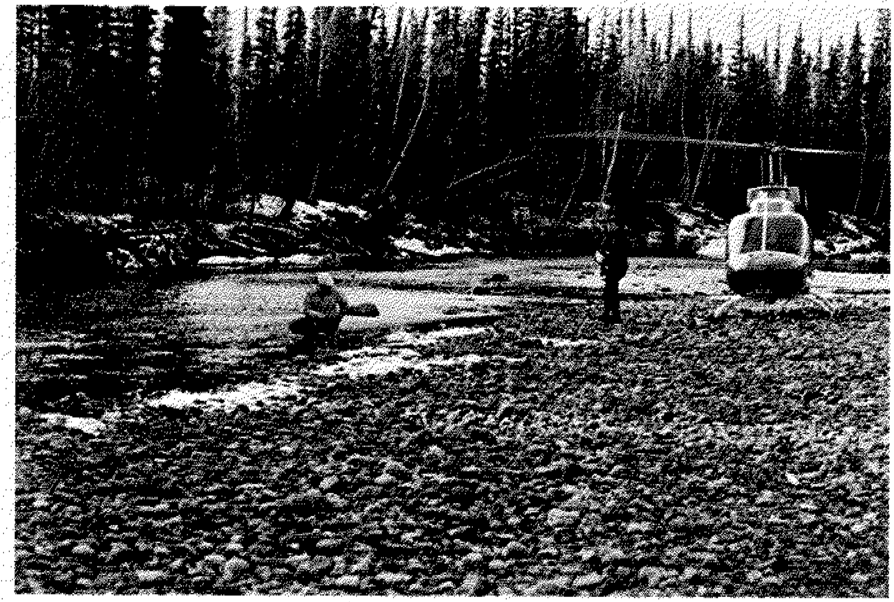


PLATE No. 3

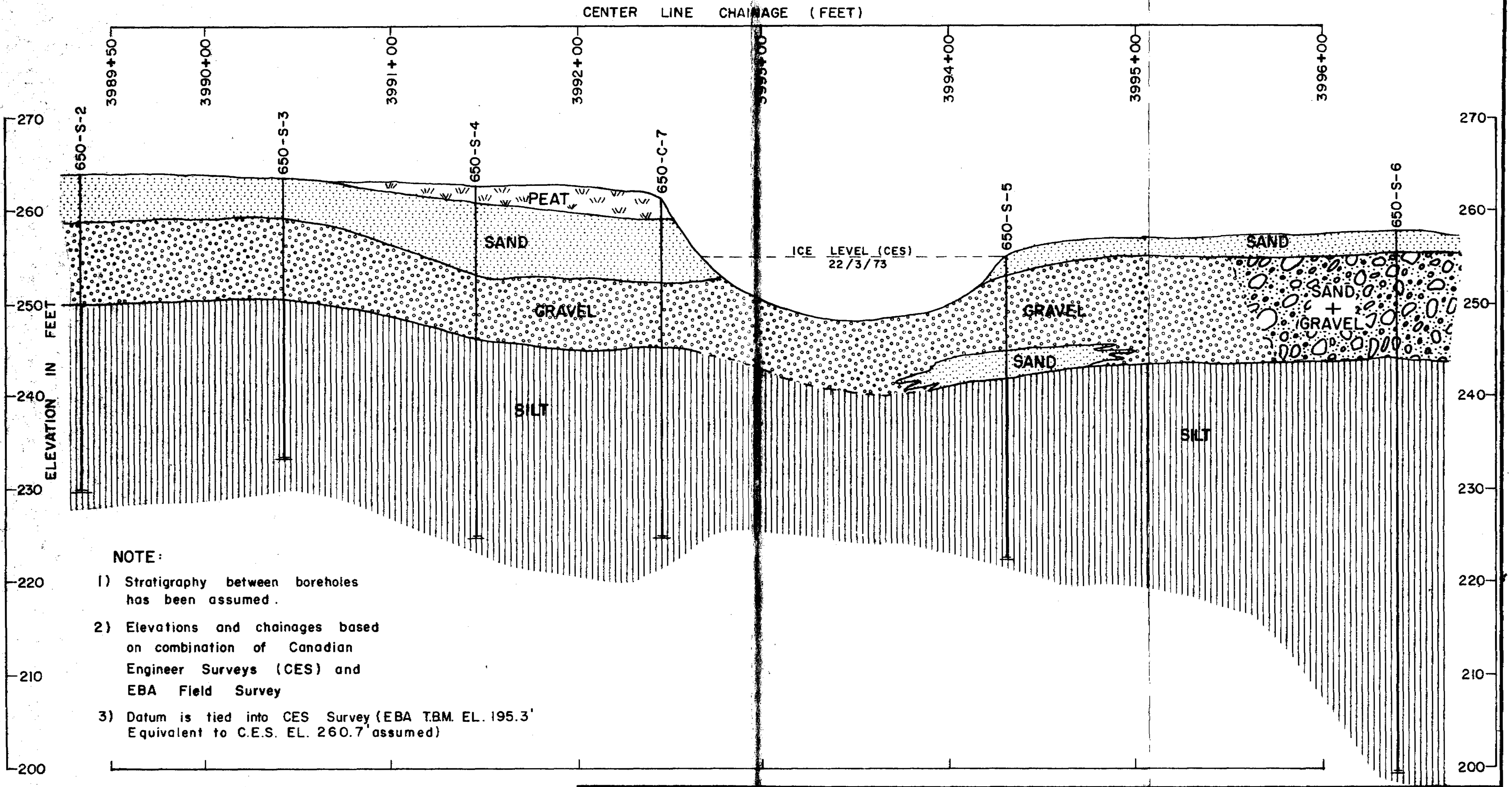
Oscar Creek Crossing
Note size of gravel and differences between creek banks. North is to the right. (October, 1972)

PLATE No. 4

Oscar Creek Crossing
North is to the left. Photo taken very near where the highway crosses the creek. (October, 1972)

Drawing No. A-4

E.W. Brooker & Associates Ltd.



NOTE:

1) Stratigraphy between boreholes has been assumed.

2) Elevations and chainages based on combination of Canadian Engineer Surveys (CES) and EBA Field Survey

3) Datum is tied into CES Survey (EBA T.B.M. EL. 195.3' Equivalent to C.E.S. EL. 260.7' assumed)

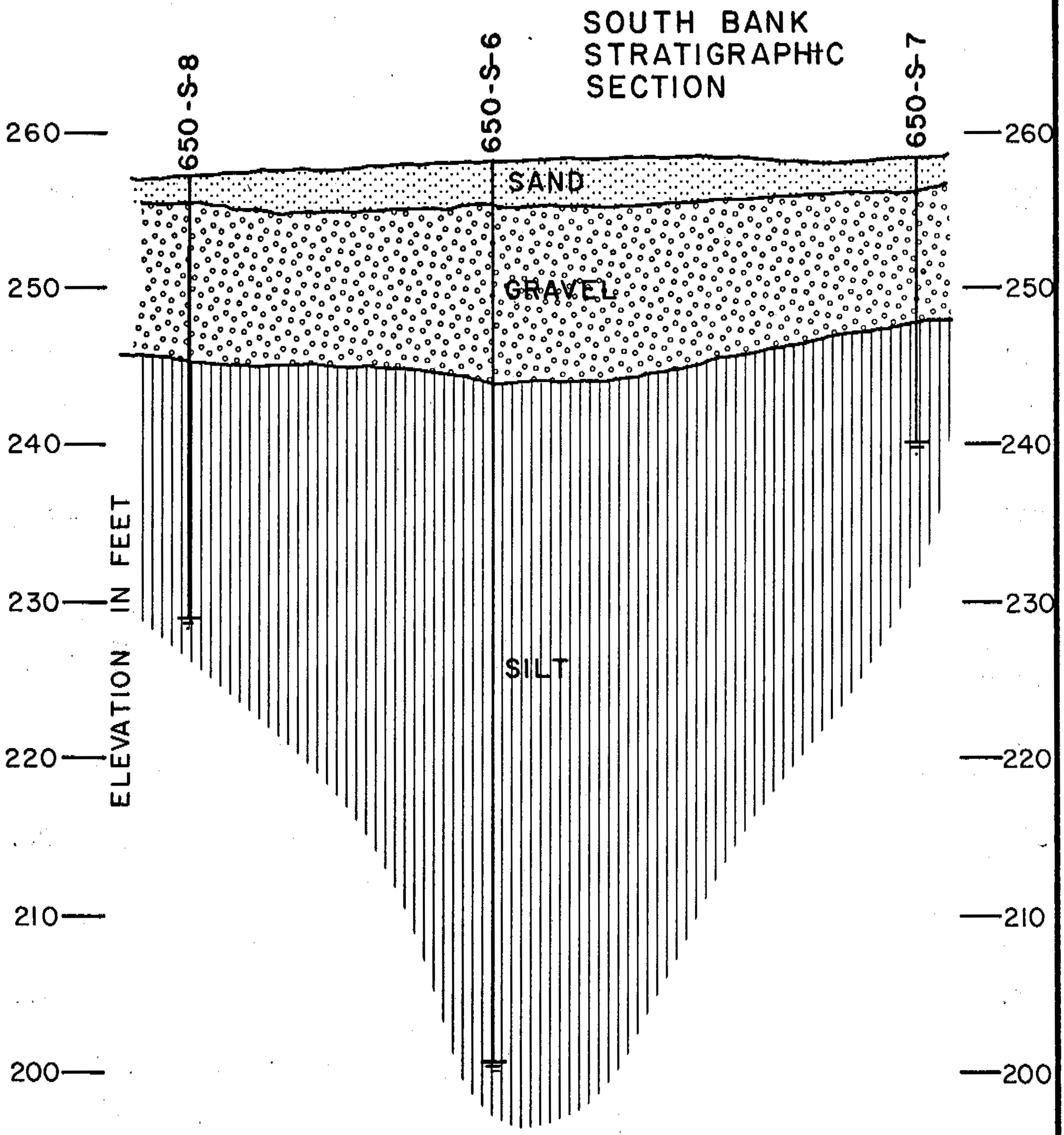
DWN. BY	FRB
DATE DWN.	2 JAN. 74
SCALE	Hor. 1" = 50' Vert. 1" = 10'
JOB No.	E-517

EBA Engineering Consultants Ltd.

OSCAR CREEK CROSSING
℄ PROFILE & STRATIGRAPHY

DWG A-5

SHT. No.



OSCAR CREEK CROSSING
CROSS-SECTION & STRATIGRAPHY

NOTE:

1. Stratigraphy between boreholes has been assumed.
2. All elevations are relative to T.B.M. Nail set in tree 8' W. and 30' S. of D.P.W. ℄ hub. Sta. 3996 + 00 (EL. 195.3' = C.E.S. Survey EL. 260.7' assumed)
3. Scales: Horiz. 1" = 60'
   Vert. 1" = 10'

EBA Engineering Consultants Ltd.	DATE 2 JAN 74	SCALE 1" = 4 mi	JOB No. E - 517	DWG No. A-6

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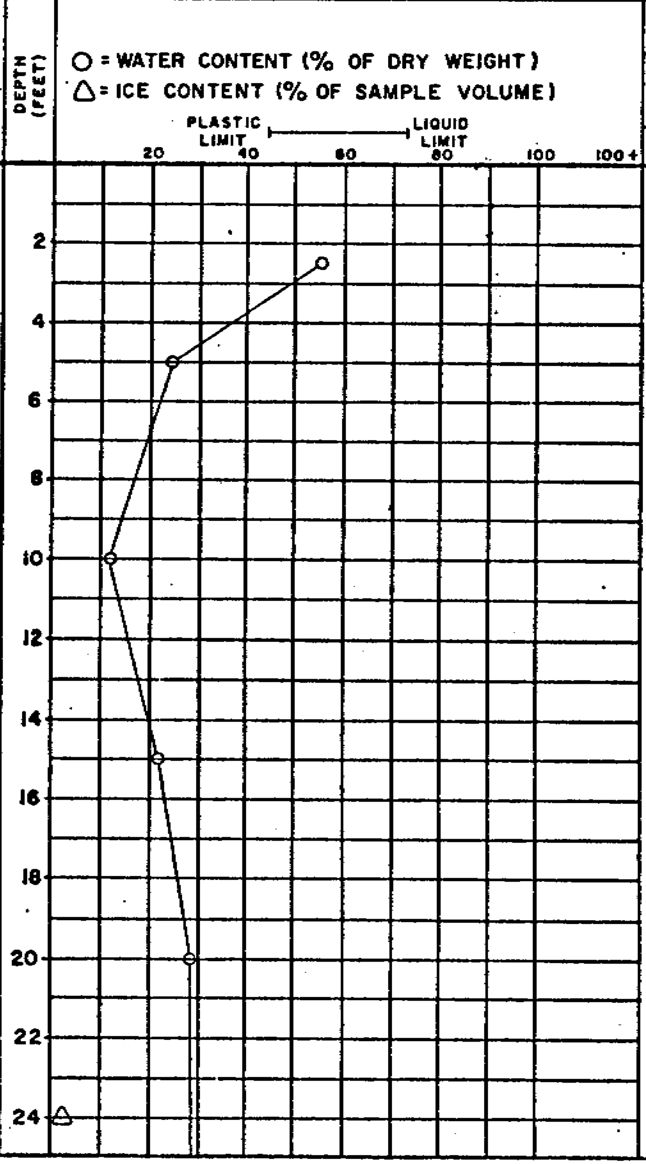
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY**

DWN: ALB	FIELD ENG: NRM	DATE DRILLED: 3/2/73	AIRPHOTO NO: A23003 - 32	CHAINAGE: 3989 + 40 (EST.)	OFFSET
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Fair to S.	VEGETATION: Black Spruce & Birch	ELEV:

TEST HOLE

MILE	B,C,S	NUMBE
650	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
2 4	1 2				SP	SAND - Med. Brown - Medium - Dense - Uniform - With Fine Gravel	F	NB							Oscar Creek
6 8 10 12 14	3				GW	GRAVEL - Med. Brown - Sandy, trace of silt - Dense - Well Graded			-	4	26	70			
16 18 20	4 5				ML to SM	SILT - Grey - Sandy (fine) - Dense -									
22 24					ML	- Grey, Some Clay, Low Plasticity, Sandy		V 0 - 5%							



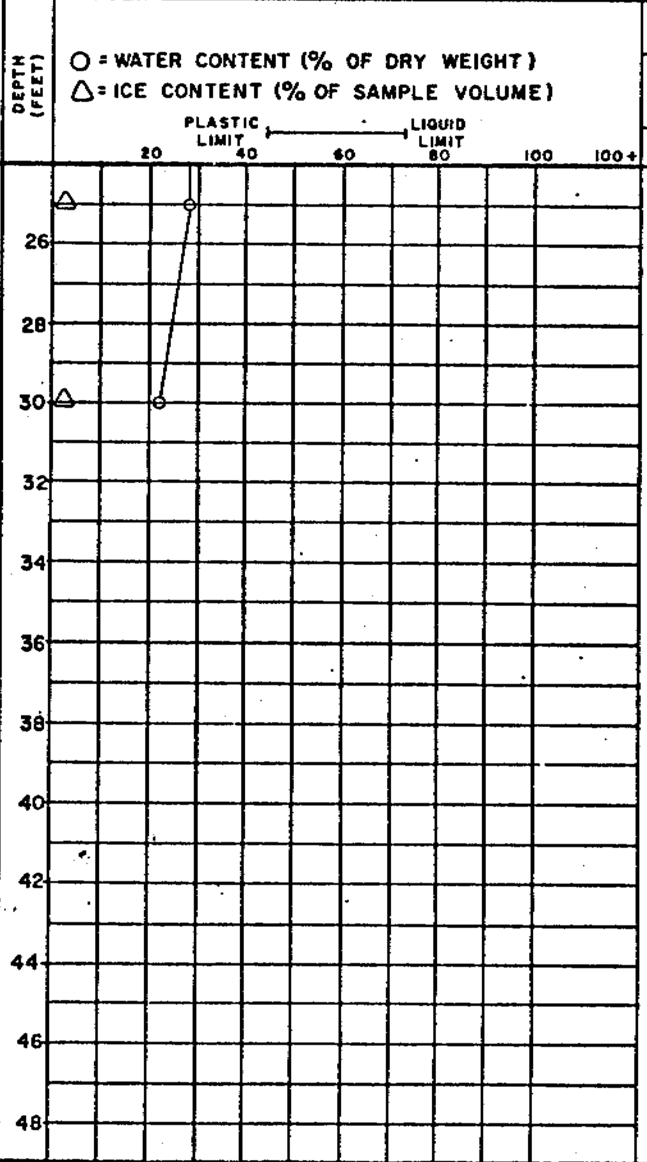
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 3/2/73	AIRPHOTO NO: A23003 - 32	CHAINAGE:	OFFSET
CKD LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Fair to N.	VEGETATION: Black Spruce & Birch	ELEV:

TEST HOLE

MILE	B,C,S	NUMB
650	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 (PCF)	DRY DENSITY (PCF)	REMARKS
	6	⧅				SILT									
26															
28					CL to CI	- Clayey - Low to Med. Plasticity	F	V 0 - 5%							
30	7	⧅													
32															
34						END OF HOLE 34'									
36															
38															
40															
42															
44															
46															
48															



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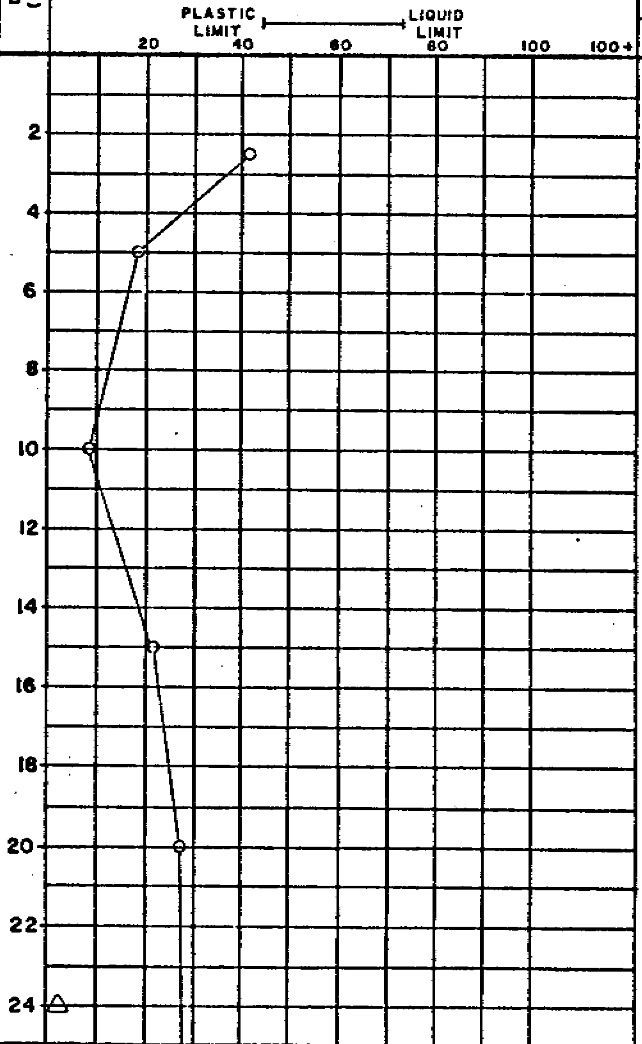
E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 3/2/73	AIRPHOTO NO: A23003 - 32	CHAINAGE: 3990 + 40 (EST.)	OFFSET	
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Fair to S.	VEGETATION: Black Spruce, & Birch	ELEV:	

TEST HOLE		
MILE	B,C,S	NUMBE
650	S	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
2	1	⧅			SP	SAND - Med. Brown - Silty - Dense, Medium - Uniform									Oscar Creek
5	2	⧅			GP	GRAVEL - Med. Brown - Sandy - Dense - Poorly Graded		NB							
10	3	⧅													
15	4	⧅			ML	SILT - Grey - Sandy (Fine) - Dense - Some gravel, some clay @ 15'	F		12	49	30	9			
20	5	⧅													
24					CL	- Grey - Clayey - Low Plasticity		V-0-5%							

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



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E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

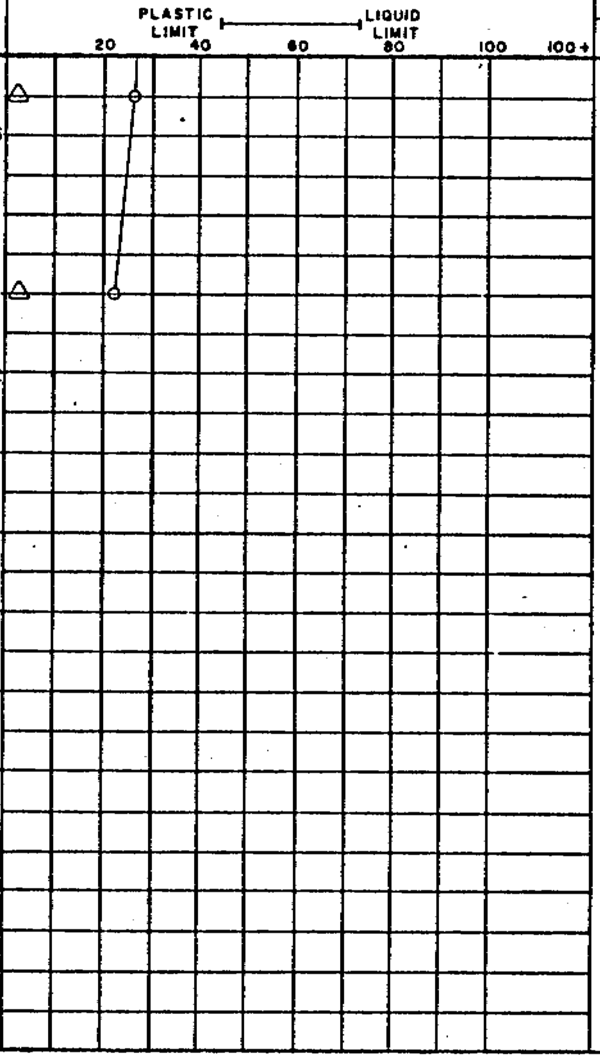
DWN: ALB
CKD: LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED: 3/2/73
RIG: Mayhew
AIRPHOTO NO: A23003 - 32
SURFACE DRAINAGE: Fair to S
CHAINAGE: 3990 + 40 (EST.)
VEGETATION: Black Spruce & Birch
OFFSET:
ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
650	S	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
26	6	▨			CL	SILT - same	F	V 0 - 5%							
30	7	▨				END OF HOLE 30'									

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



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

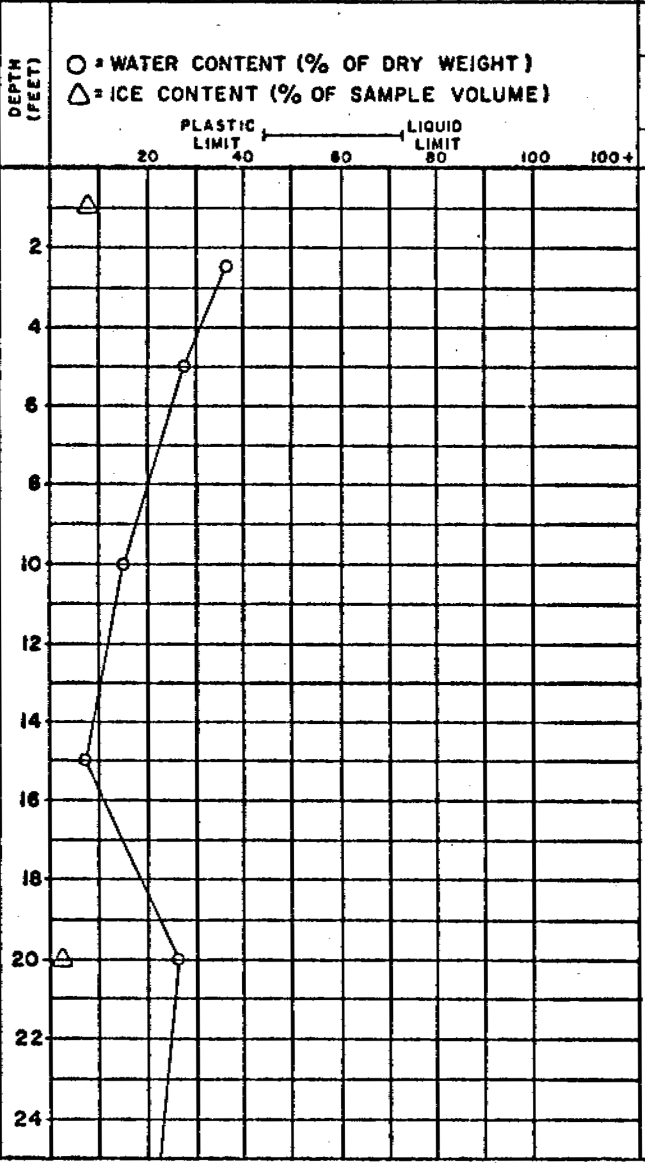
DWN: ALB | FIELD ENG: NRM | DATE DRILLED 3/2/73 | AIRPHOTO NO: A23003 - 32 | CHAINAGE: 3991 + 30 | OFFSET

CKD LAB | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Fair to S. | VEGETATION: Black Spruce & Birch | ELEV:

TEST HOLE

MILE	B,C,S	NUMBE
650	S	4

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	REMARKS
					PT	PEAT - Black - Fibrous		V: 5 - 10%	Oscar Creek
2 4 6 8	1 2	⧅ ⧅			SM	SAND - Med. Brown - Medium - Dense - - Silty		NB	
10 12 14 16	3 4	⧅ ⧅			GP	GRAVEL - Med. Brown - Sandy - Dense - Poorly Graded - Grey	F		
18 20 22 24	5	⧅			ML to CL	SILT - Grey - Clayey - Low Plasticity - Trace of Fine Sand		V 0 - 5%	



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

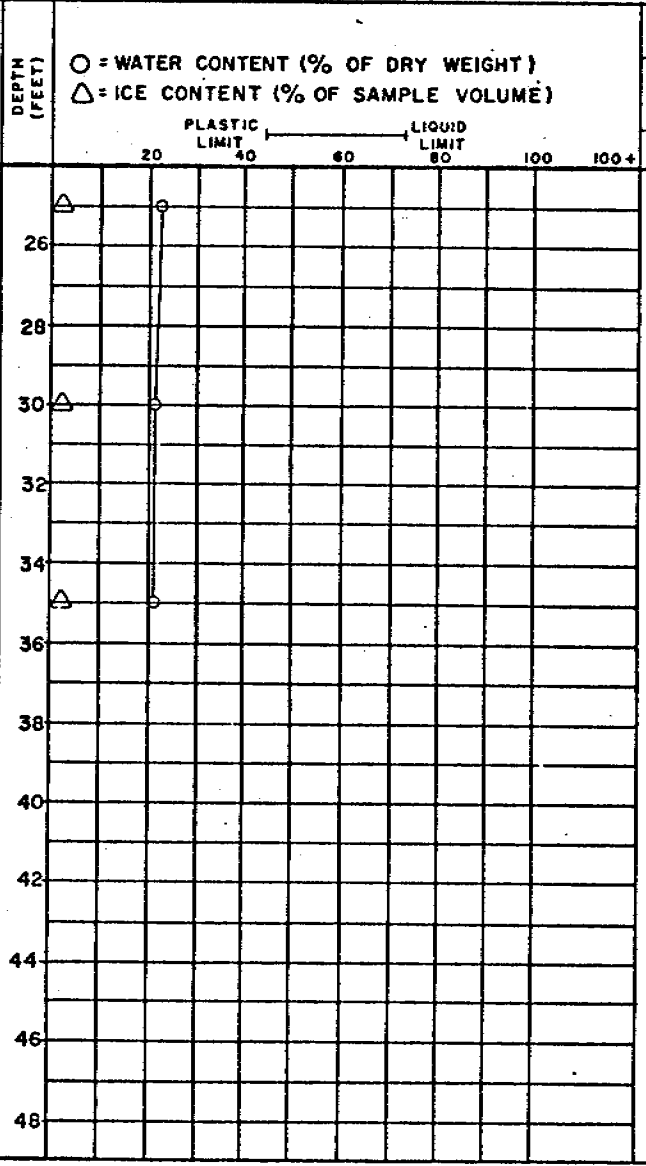
DWN: ALB
FIELD ENG: NRM
DATE DRILLED: 3/2/73
AIRPHOTO NO: A23003 – 32
CHAINAGE: 3991 +45 (EST.)
OFFSET
CKD: LAB
TECH: JK
RIG: Mayhew
SURFACE DRAINAGE: Good to N.
VEGETATION: Black Spruce & Birch
ELEV:

TEST HOLE

MILE	B,C,S	NUMBE
650	S	4

REMARKS

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
	6	▨				SILT		
26					ML to CL	– same		
28								
30	7	▨					F	V 0 - 5%
32								
34								
	8	▨						
36								
38						END OF HOLE 38'		
40								
42								
44								
46								
48								



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

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E.W. BROOKER & ASSOCIATES LTD.

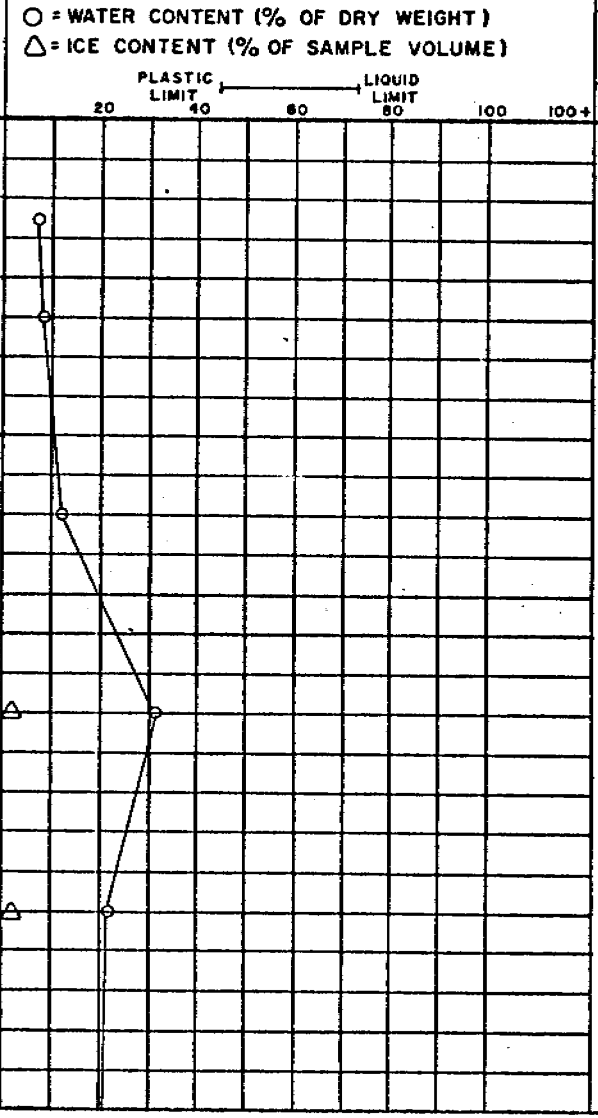
DRILL HOLE REPORT

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: A23003 - 32
SURFACE DRAINAGE: Good to N
CHAINAGE: 3994 + 30 (EST.)
VEGETATION: Black Spruce & Birch
OFFSET
ELEV:

TEST HOLE		
MILE	B,C,S	NUMBE
650	S	5

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
					SP	SAND - Med. Brown - Dense, Fine - Uniform									Oscar Creek
2	1				GP	GRAVEL - Med. Brown - Dense - Sandy - Poorly Graded		NB							
4	2														
6															
8															
10	3				SP	SAND - Med. Brown - Coarse - Dense - Some Gravel									
12							F								
14	4				CL to ML	SILT - Grey - Some Clay, trace of sand - Low Plasticity		V 0 - 5%	16	79	5	-			
16															
18															
20	5														
22															
24															



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E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

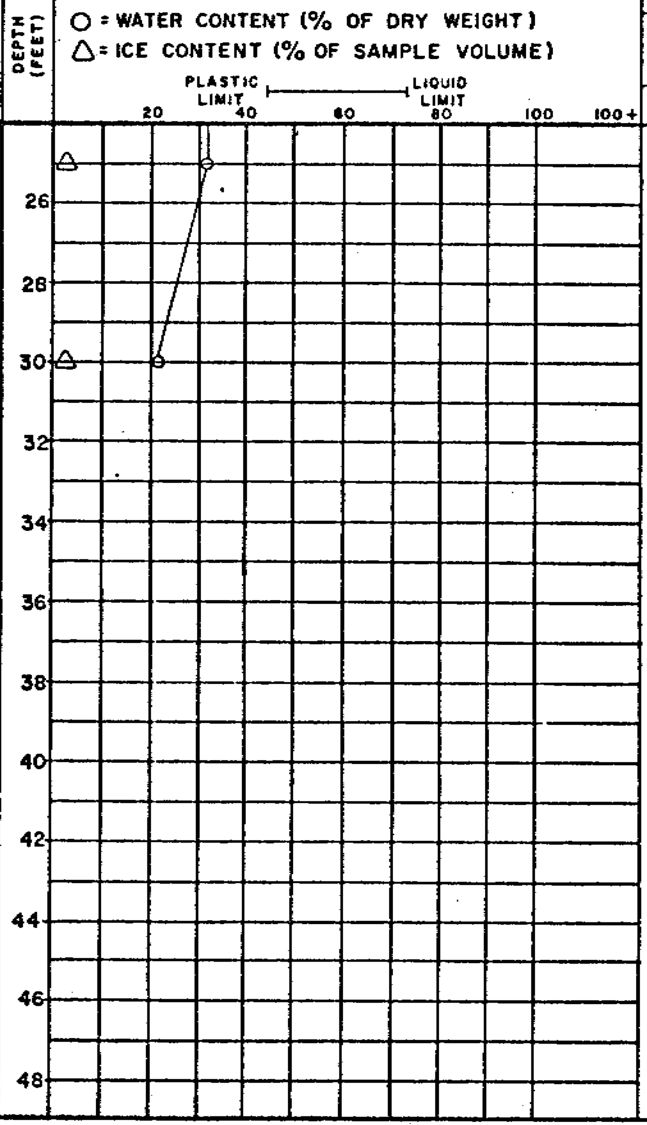
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN ALB
CKD LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED 3/2/73
RIG: Mayhew
AIRPHOTO NO: A23003 - 32
SURFACE DRAINAGE: Good to N.
CHAINAGE: 6994 + 30 (EST.)
VEGETATION: Black Spruce & Birch
OFFSET
ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
650	S	5

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
	6	☒				SILT - same		
26							F	V. 0 - 5%
28					ML to CL			
30	7	☒						
32						END OF HOLE 32'		
34								
36								
38								
40								
42								
44								
46								
48								



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
CKD: LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED: 3/2/73
RIG: Mayhew
AIRPHOTO NO: A23003 - 32
SURFACE DRAINAGE: Fair to N.
CHAINAGE: 3996 + 60 (EST.)
VEGETATION: Black Spruce
OFFSET
ELEV:

TEST HOLE

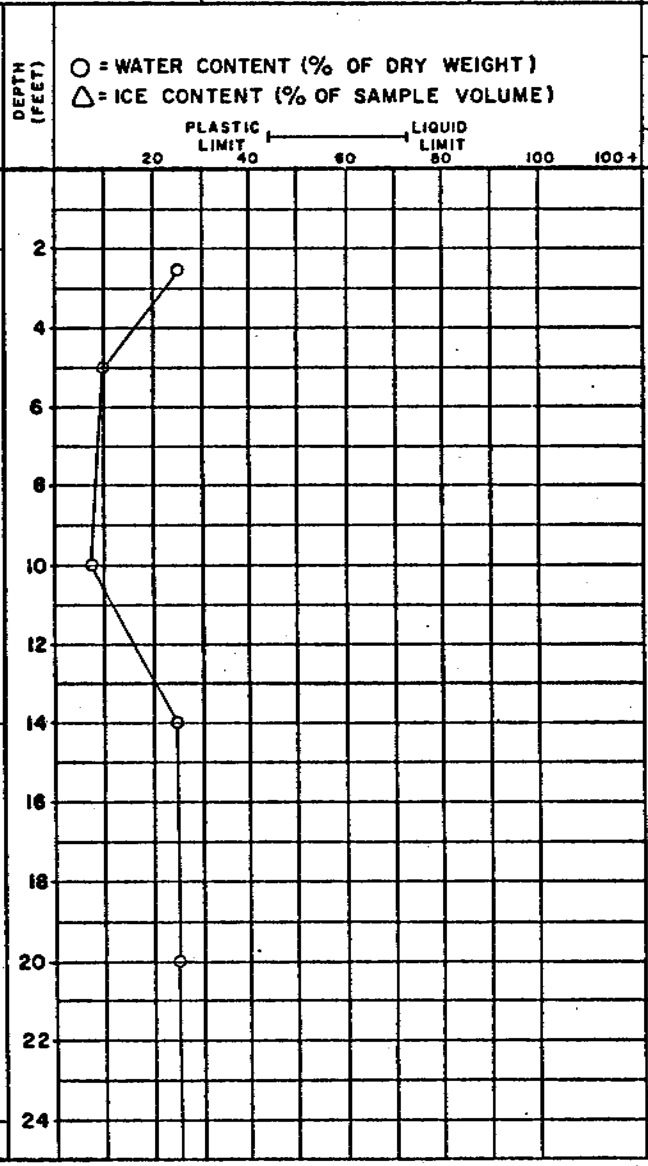
MILE	B,C,S	NUMBER
650	S	6

REMARKS

Oscar Creek

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.)
					SM	SAND - Med. Brown - Fine, Dense - Silty		NB						
2 4 6 8 10 12	1 2 3	⧅ ⧅ ⧅			SP to GP	SAND - Med. Brown, GRAVEL - Coarse sand, - Dense - Poorly Graded		NF to NB						
14 16 18 20 22 24	4 5	⧅ ⧅			ML to CL	SILT - Grey - Some Clay - Low Plasticity	F	NB						

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



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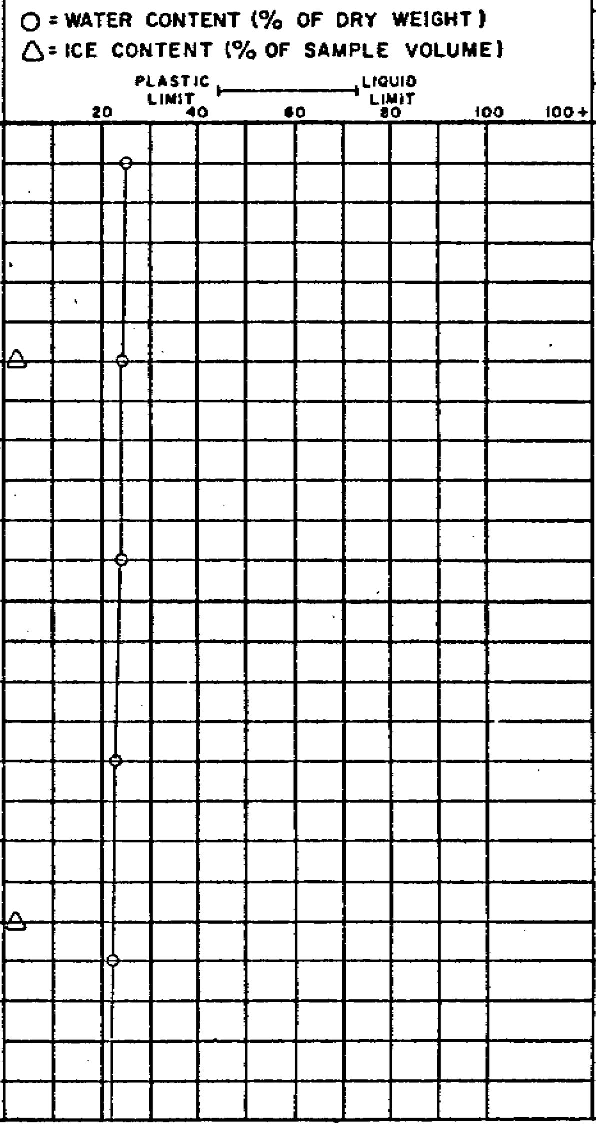
E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 3/2/73	AIRPHOTO NO: A23003 - 32	CHAINAGE: 3996 + 60 (EST.)	OFFSET	
CKD LAB	TECH JK	RIG Mayhew	SURFACE DRAINAGE: Fair to N	VEGETATION: Black Spruce & Birch		ELEV.

TEST HOLE

MILE	B,C,S	NUMBER
650	S	6

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	DEPTH (FEET)
	6	⧅				SILT - same		NB'	
26									26
28					ML to CL				28
30	7	⧅					F	V 0 - 5%	30
32									32
34									34
	8	⧅							
36									36
								NB'	
38									38
40	9	⧅							40
42									42
44									44
	10	⧅							
46								V-0-5%	46
48									48



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

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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: A23003 – 32 | CHAINAGE: 3996 + 60 (EST.) | OFFSET

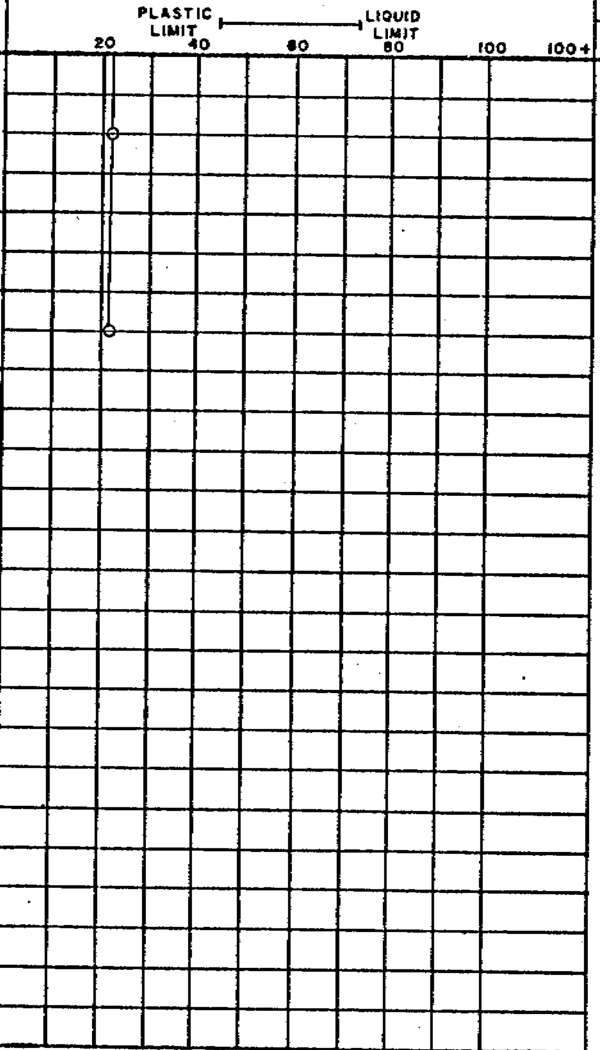
CKD: LAB | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Fair to N | VEGETATION: Black Spruce & Birch | ELEV:

TEST HOLE		
MILE	B,C,S	NUMBER
650	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
						SILT									
50	11	⧅				– same	F	NB							
52															
54					ML – CL										
	12	⧅													
56															
58						END OF HOLE 58'									
60															
62															
64															
66															
68															
70															
72															

○ = WATER CONTENT (% OF DRY WEIGHT)

△ = ICE CONTENT (% OF SAMPLE VOLUME)

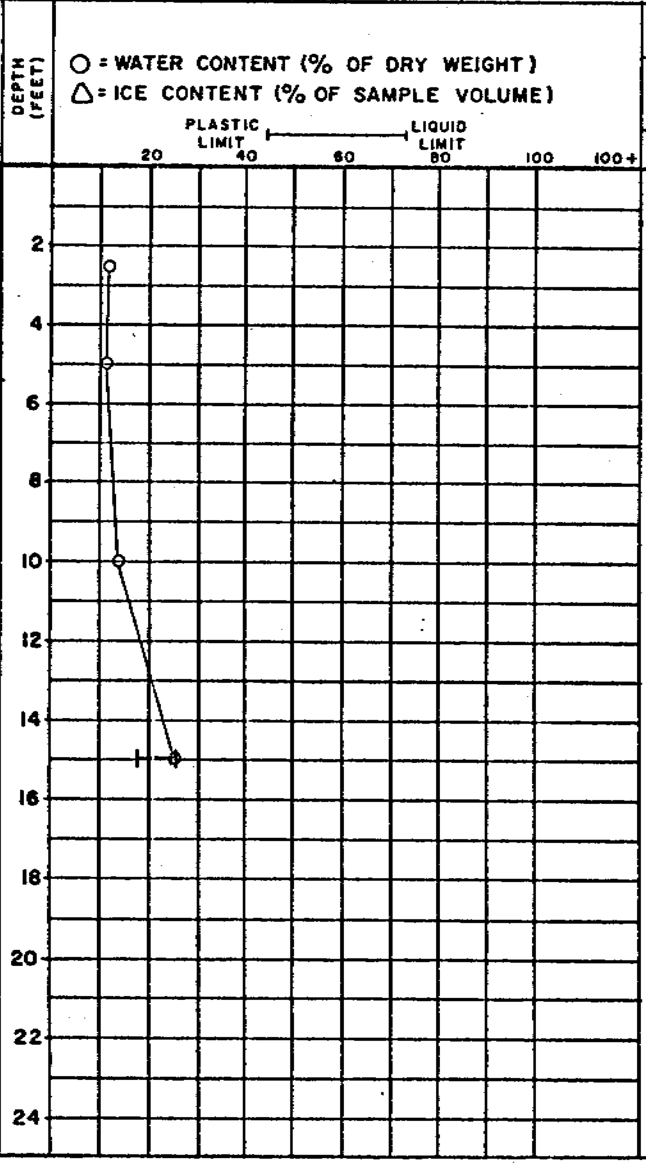


E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED: 3/2/73	AIRPHOTO NO: A23003-32	CHAINAGE: 3996 + 60 (EST.)	OFFSET 160 EAST	
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Poor	VEGETATION: Black Spruce & Birch	ELEV:	

TEST HOLE		
MILE	B,C,S	NUMBER
650	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
0–2					SM	SAND - Med. Brown - Fine, Silty - Dense		NB							Oscar Creek
2	1				GP	GRAVEL - Med. Brown - Sandy - Dense - Poorly Graded		NF to NB.							
4	2														
10	3				ML	SILT - Grey - Some Clay - Low Plasticity	F	NB							
14	4								11	82	7	-			
18						END OF HOLE 18'									



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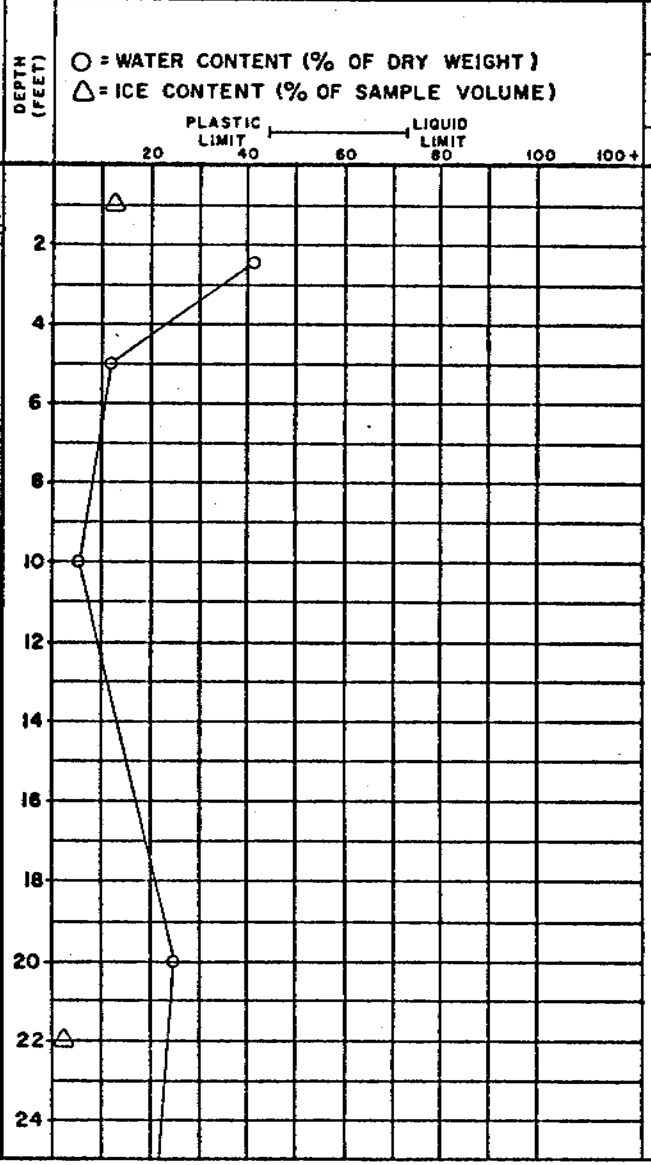
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY MVPL1

DWN: AJB | FIELD ENG: NRM | DATE DRILLED: 21/2/73 | AIRPHOTO NO: A23003 - 32 | CHAINAGE: 3992 +40 | OFFSET
CKF: GRG | TECH: JK | RIG: Mayhew | SURFACE DRAINAGE: Good to S. | VEGETATION: Black Spruce & Birch | ELEV:

TEST HOLE MVPL

MILE	B,C,S	NUMB
650	C	7

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					PT	PEAT - Black - Sandy - Fibrous		V- 10-15%							
2	1	◸			SM	SAND - Med. Greyish Brown - Silty, Fine		NB							
5	2	◸													
9					GW	GRAVEL - Med. Brown - Sandy - Well Graded - Dense	F								
10	3	◸													
15	4	◸													
16					ML CL	SILT - Grey, some clay - Trace of sand - Low Plasticity									
19								V- 0-5%							
20	5	◸													
24															



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

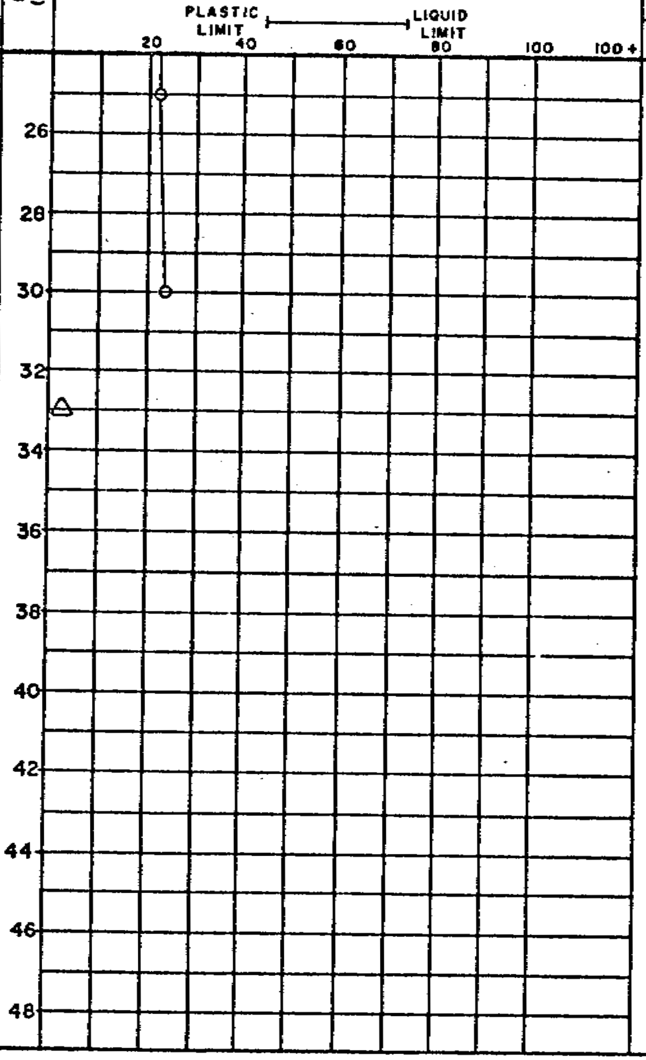
DWN: ALB	FIELD ENG: NRM	DATE DRILLED 21/2/73	AIRPHOTO NO: A23003 - 32	CHAINAGE: 3992 + 40	OFFSET
CKD GRG	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Good to S.	VEGETATION: Black Spruce & Birch	ELEV:

TEST HOLE MVF

MILE	B,C,S	NUM
650	C	7

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
	6					SILT - Same									
26					ML CL			NB to Unfrozen							North Bank of Oscar Creek
28							F to U								
30	7					-									Hole terminated @ 36' drill bit mudded up.
32															
34															
36						END OF HOLE 36'									
38															
40															
42															
44															
46															
48															

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



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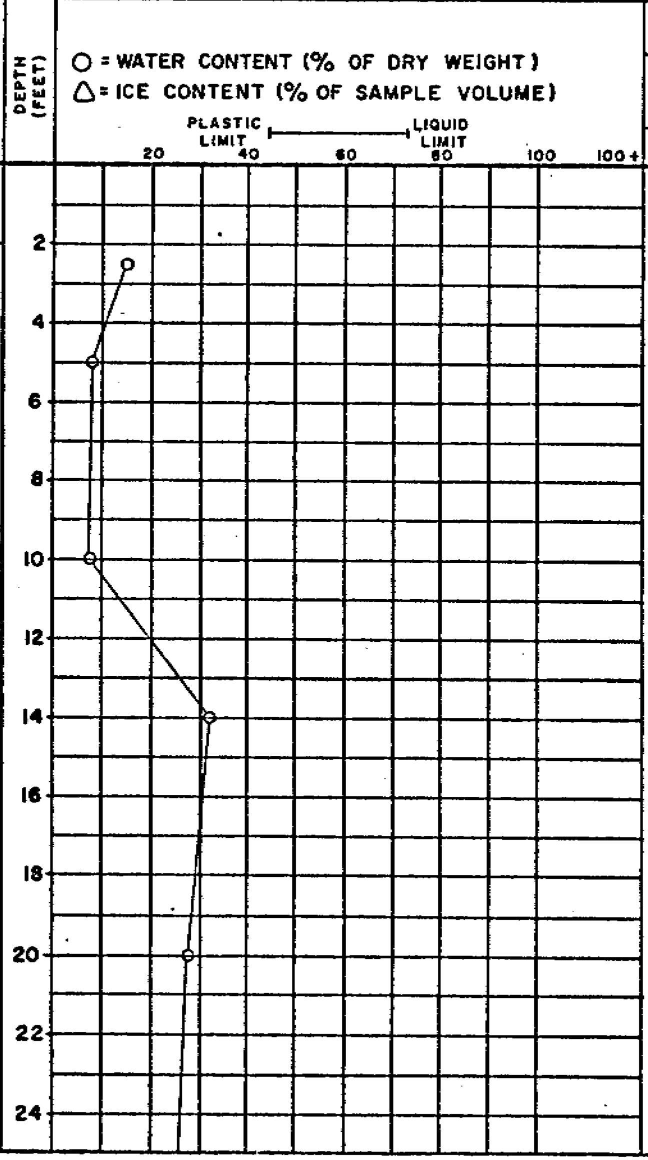
E.W. BROOKER & ASSOCIATES LTD. | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN: ALB
CKD: LAB
FIELD ENG: NRM
TECH: JK
DATE DRILLED 3/2/73
RIG: Mayhew
AIRPHOTO NO: A23003 - 32
SURFACE DRAINAGE: Fair to W.
CHAINAGE: 3996 + 60 (EST)
VEGETATION: Black Spruce
OFFSET 110' WEST
ELEV:

TEST HOLE		
MILE	B,C,S	NUMBE
650	S	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
					SM	SAND - Med. Brown - Fine Silty - Dense		NB							Oscar Creek
2	1				GW	GRAVEL - Med. Brown - Sandy, trace of silt - Dense - Well Graded .		NF to NB							
4	2								-	2	24	74			
6							F								
8															
10	3														
12					ML to CL	SILT - Grey - Some Clay - Low Plasticity		NB							
14	4														
16															
18															
20	5														
22															
24							F								



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E.W. BROOKER & ASSOCIATES LTD.

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

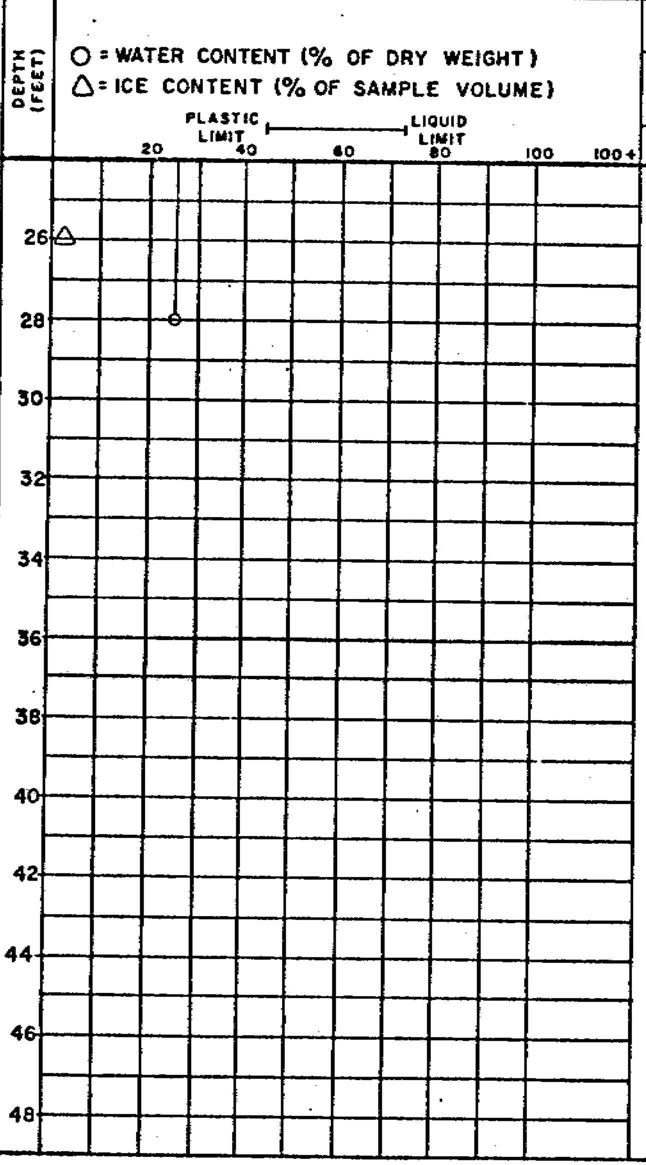
DWN: ALB	FIELD ENG: NRM	DATE DRILLED: 3/2/73	AIRPHOTO NO: A23003 - 32	CHAINAGE: 3996 + 60 (EST.)	OFFSET 110' WEST
CKD: LAB	TECH: JK	RIG: Mayhew	SURFACE DRAINAGE: Fair to W.	VEGETATION: Black Spruce	ELEV:

TEST HOLE

MILE	B,C,S	NUMBER
650	S	8

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					ML to CL	SILT - Same	F	V 0-5%							Oscar Creek
28	6														
30						END OF HOLE 28'									
32															
34															
36															
38															
40															
42															
44															
46															
48															

GRAIN- SIZE ANALYSIS



GRAIN SIZE DISTRIBUTION

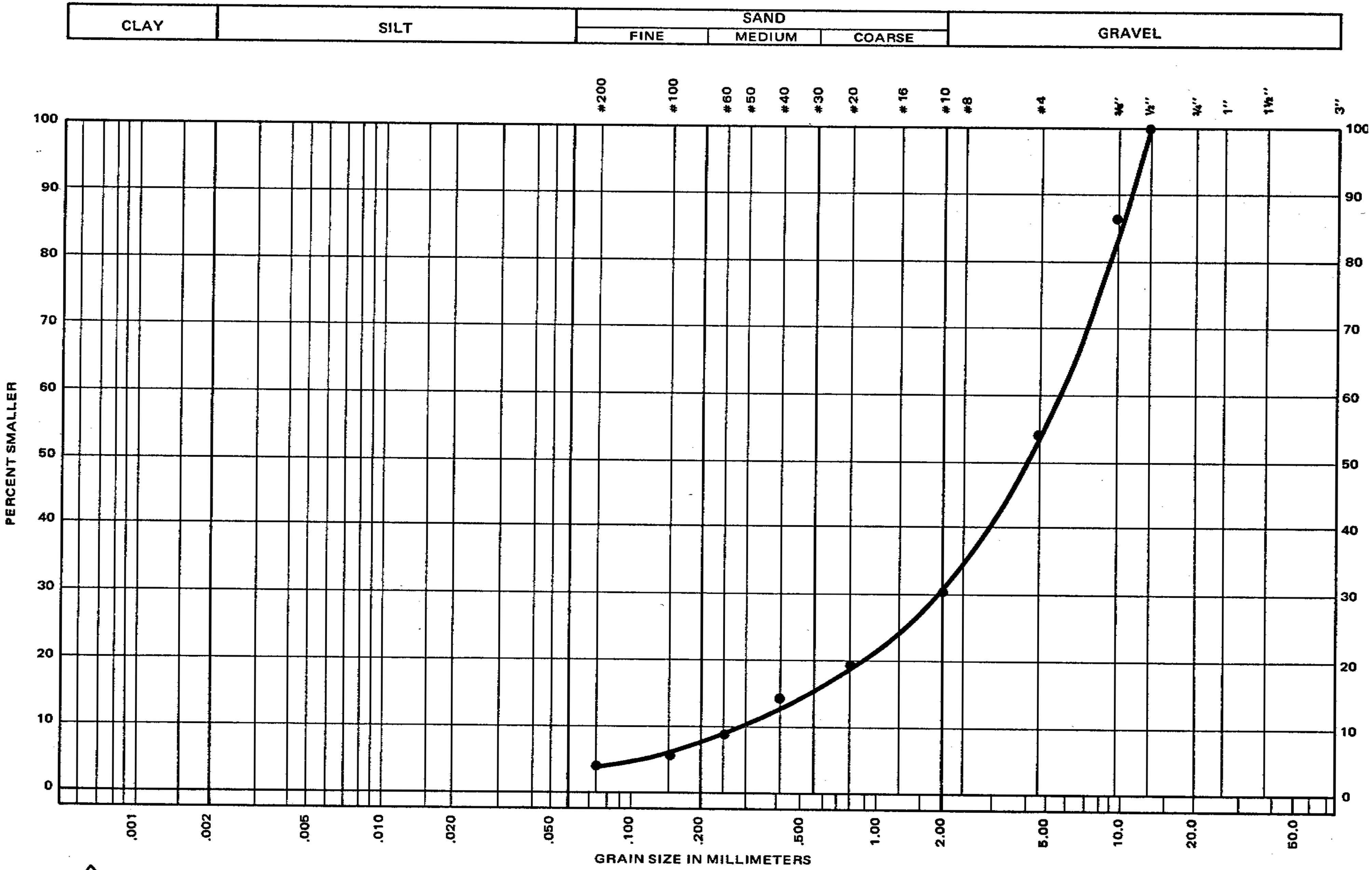


FIGURE C-1

eba

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION Gravel, Sandy, Trace of Silt

PROJECT Mackenzie Highway

JOB No. E-517 DATE April 4/73

SAMPLE No. 650-S-2

DEPTH 10'

GRAIN SIZE DISTRIBUTION

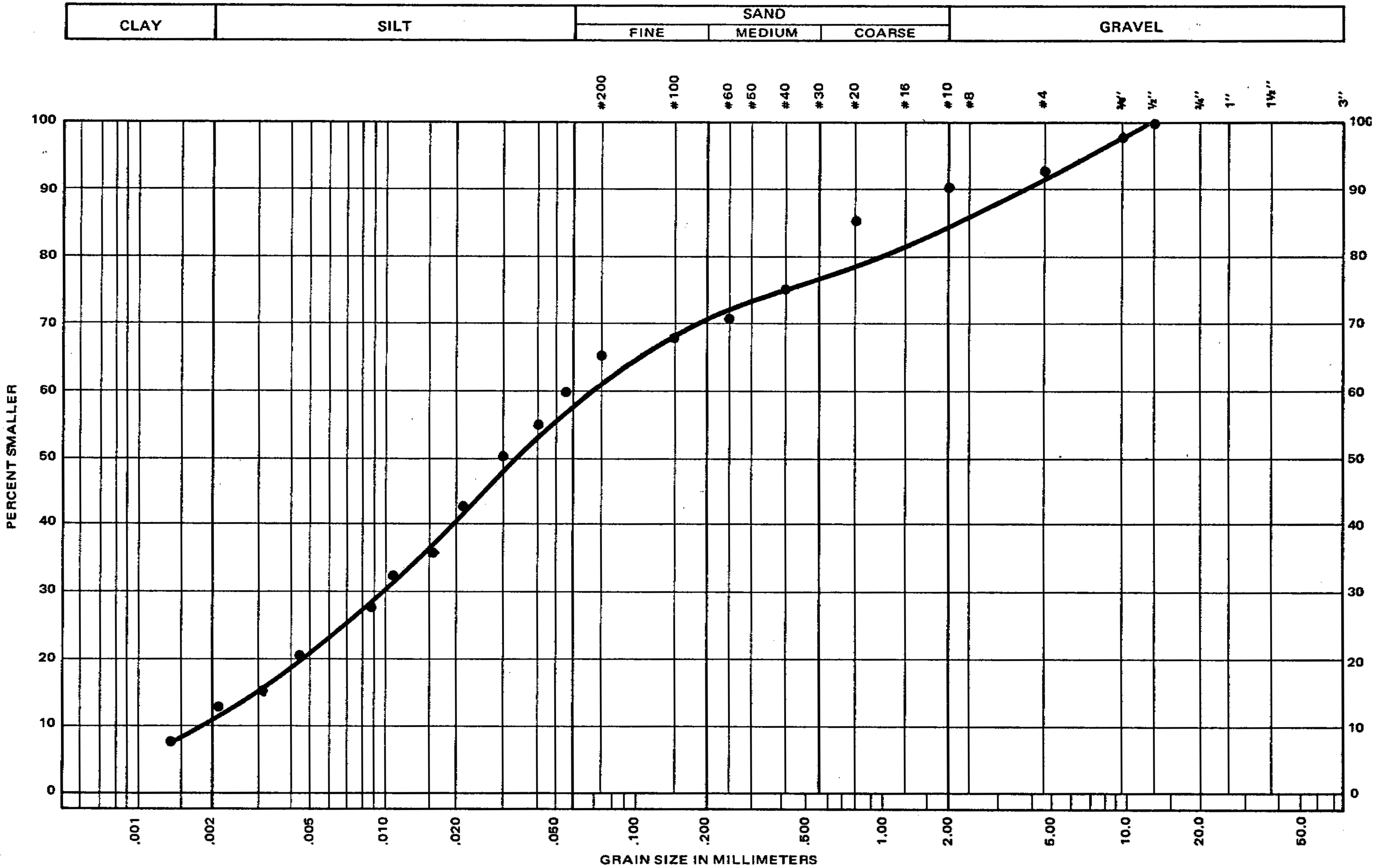


FIGURE C-2

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION Silt, Sandy, Some Clay, Trace of Gravel

PROJECT Mackenzie Highway

JOB No. E-517 DATE April 12/73

SAMPLE No. 650-S-3

DEPTH 15'

GRAIN SIZE DISTRIBUTION

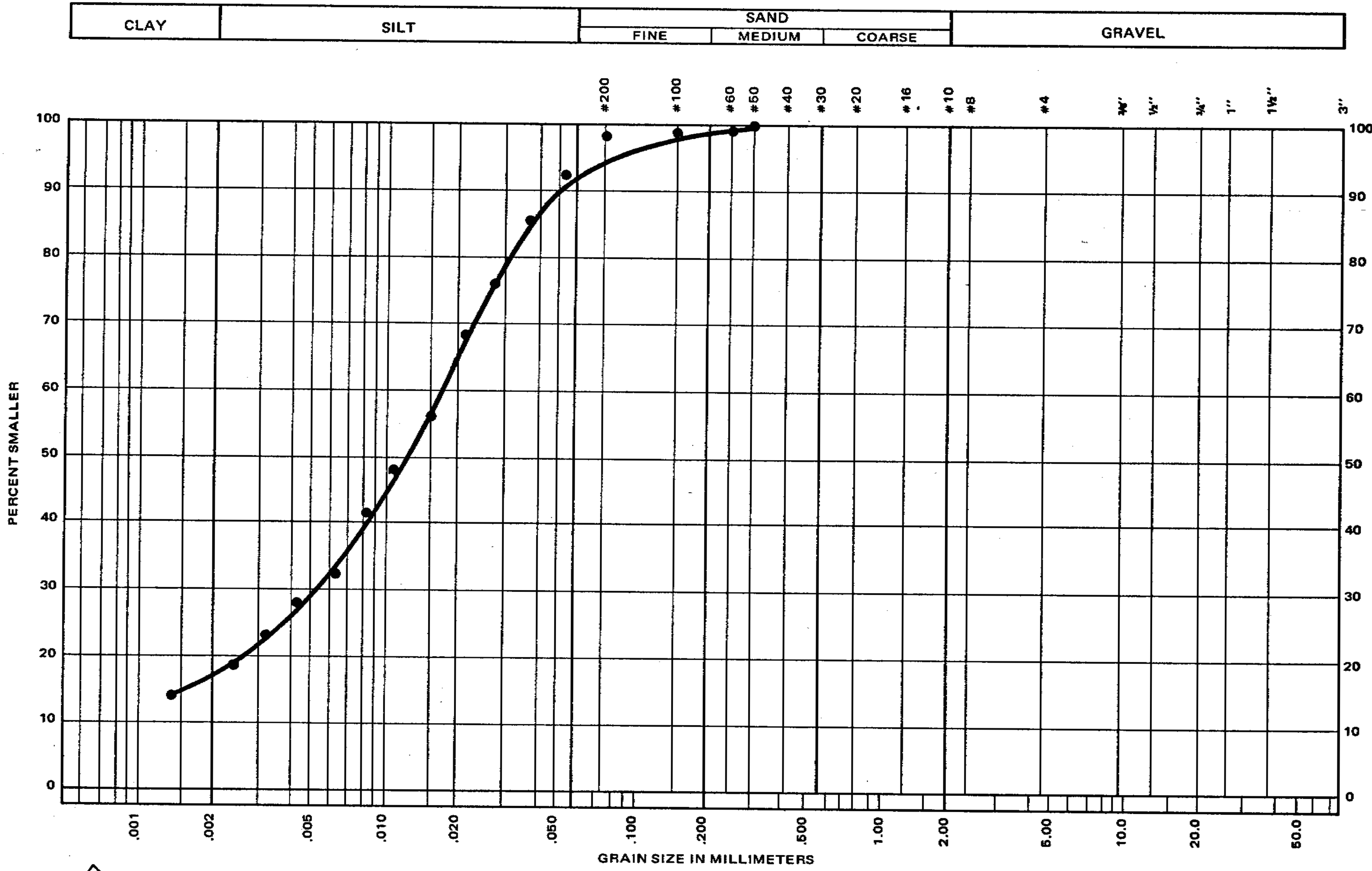


FIGURE C-3

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION Silt, Some Clay, Trace of Sand

PROJECT Mackenzie Highway

JOB No. E-517 **DATE** April 9/73

SAMPLE No. 650-S-5

DEPTH 15'

GRAIN SIZE DISTRIBUTION

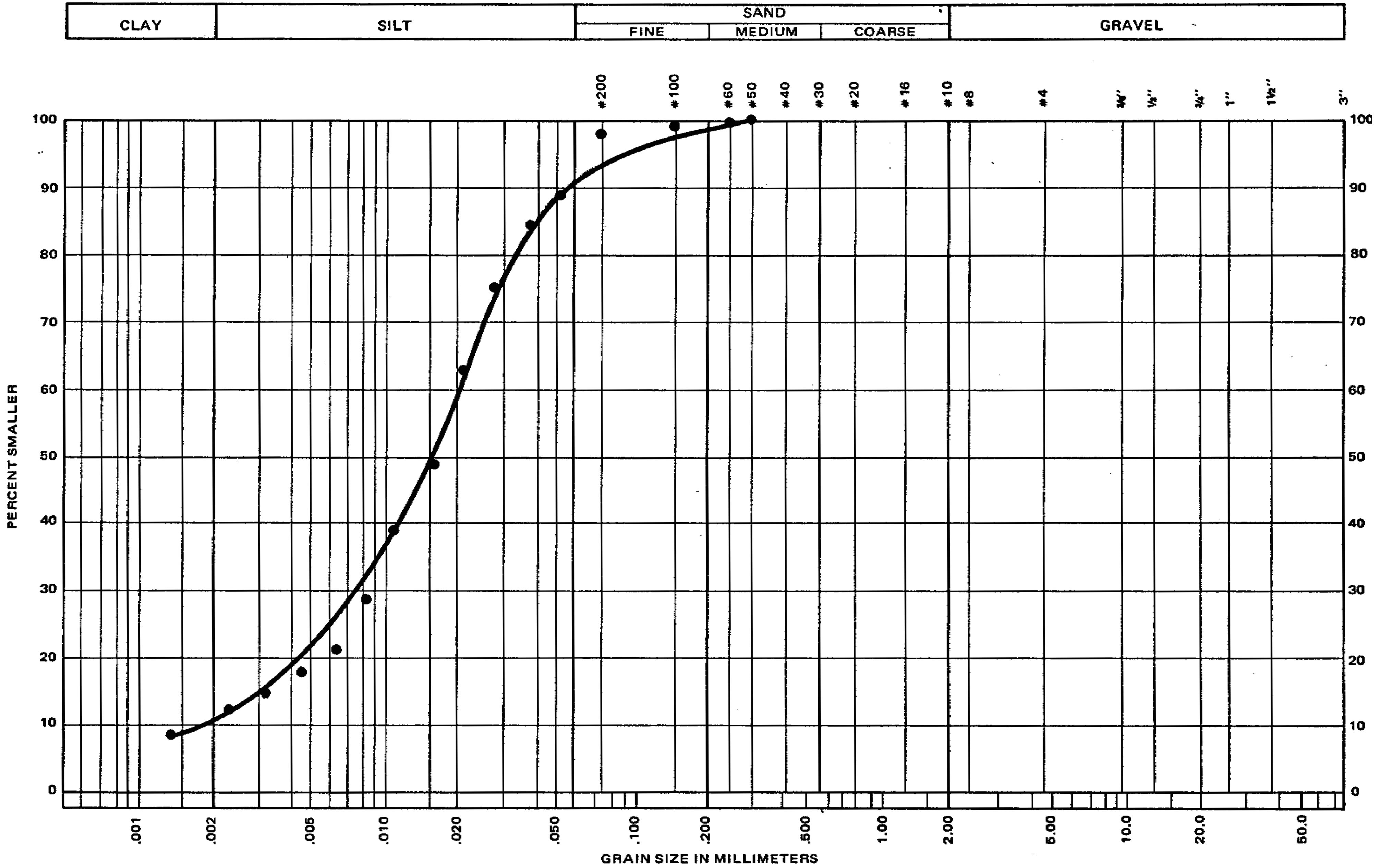


FIGURE C-4

eba

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION Silt, Some Clay, Trace of Sand

PROJECT Mackenzie Highway

JOB No. E-517 DATE April 9/73

SAMPLE No. 650-S-7

DEPTH 15'

GRAIN SIZE DISTRIBUTION

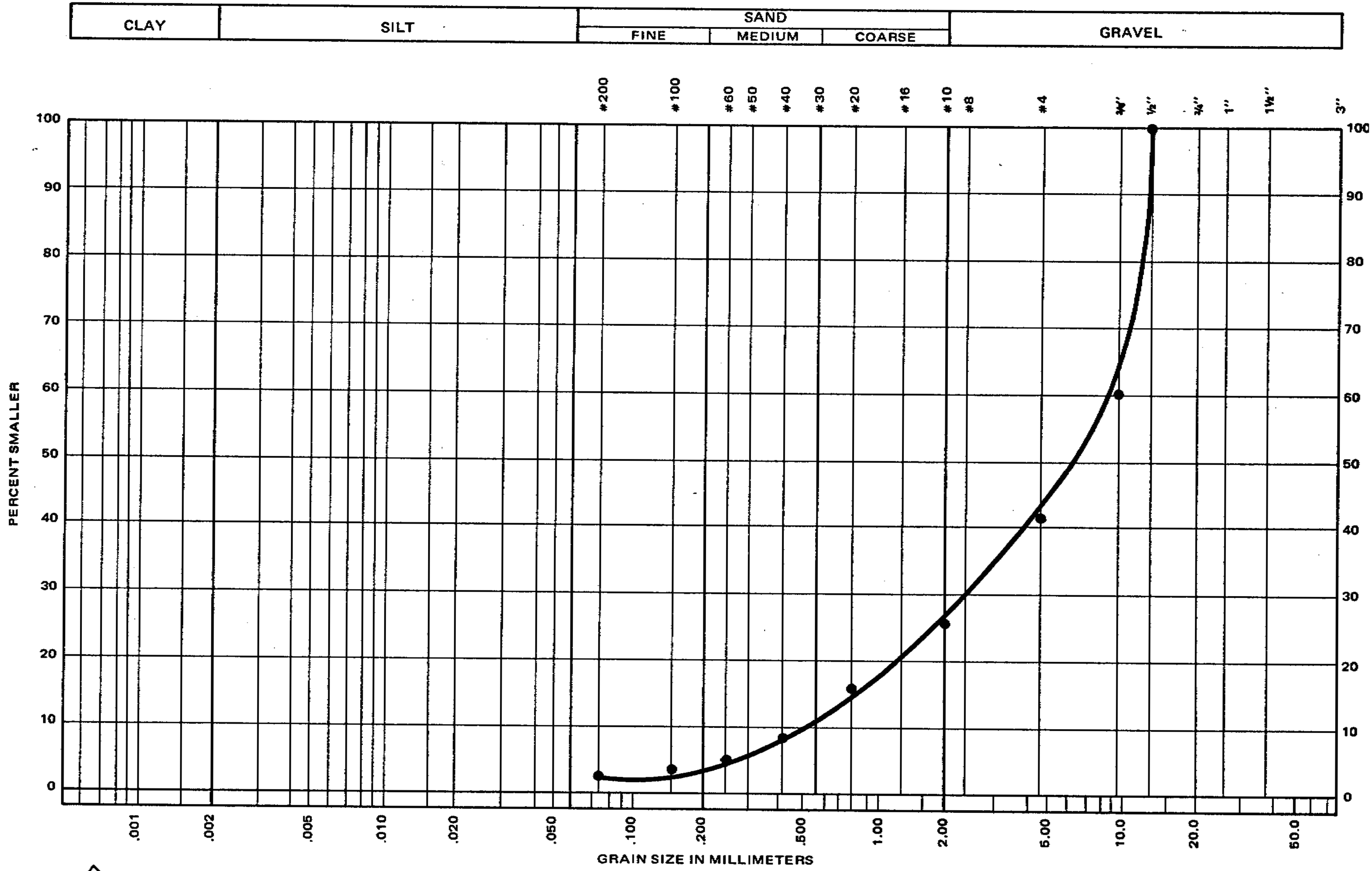


FIGURE C-5

E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION Gravel, Sandy,
Trace of Silt

PROJECT Mackenzie Highway
JOB No. E-517 DATE April 5/73
SAMPLE No. 650-S-8
DEPTH 5'

SUMMARY OF TEST RESULTS

JOB No.

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		
650-S-2	10						4	26	70	GW	
-3	15					12	49	30	9	ML	
-5	15	30.5	28.3	21.6	6.8	16	79	5	0	CL	
-7	15	25.0	25.0	19.2	5.8	11	82	7	0	CL- ML	
-8	5						2	24	74	GW	

DWG. No. C-6

EBA ENGINEERING CONSULTANTS LTD.