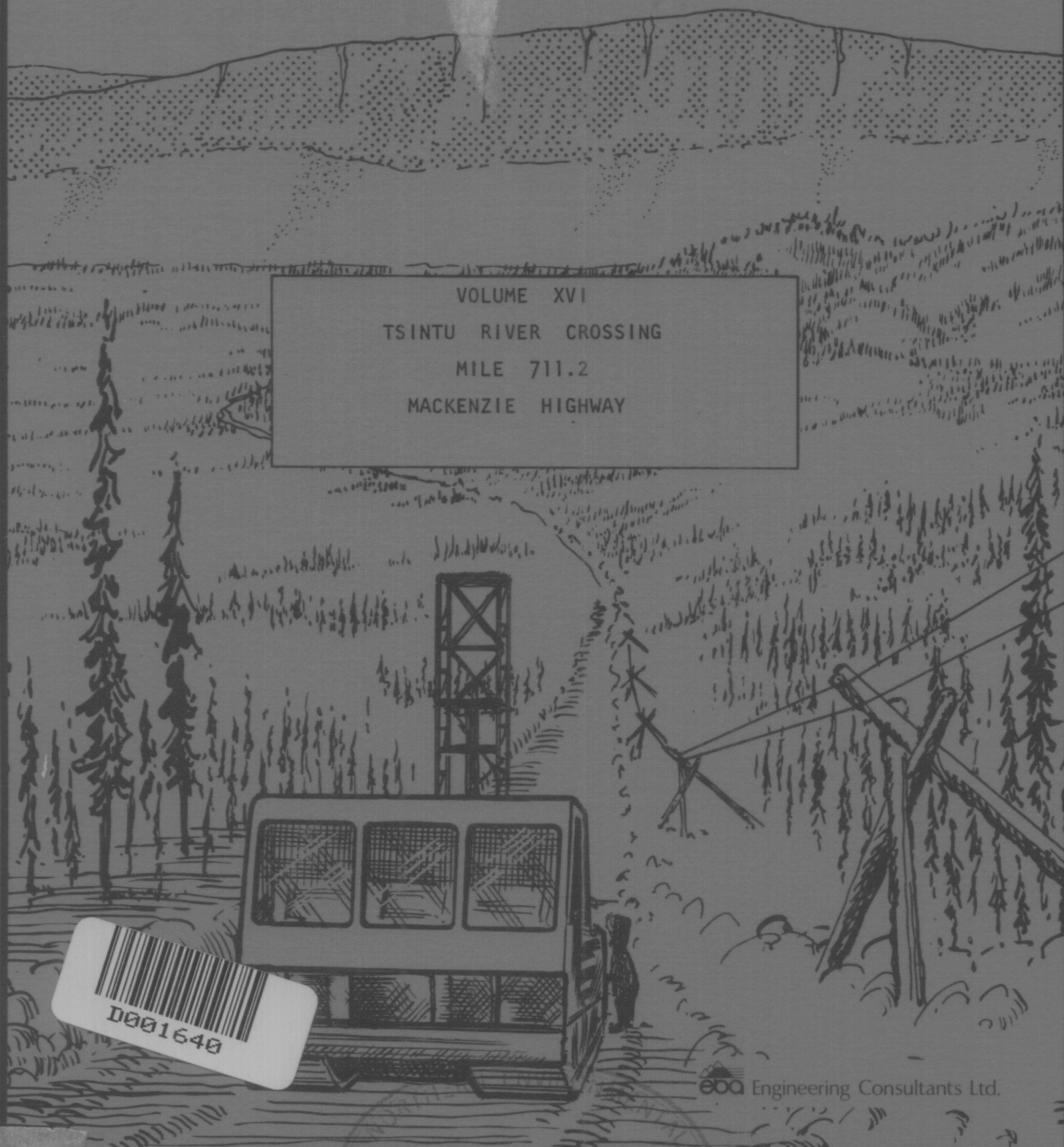
MACKENZIE HIGHWAY

GEOTECHNICAL EVALUATION



VOLUME XVI

TSINTU RIVER CROSSING

MILE 711.2

MACKENZIE HIGHWAY

eba Engineering Consultants Ltd.

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MACKENZIE HIGHWAY
GEOTECHNICAL EVALUATION
VOLUME XVI
TSINTU RIVER CROSSING - MILE 711.2
MACKENZIE HIGHWAY

Submitted To:

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

MARCH, 1974

eba Engineering Consultants Ltd.

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I INTRODUCTION

In conjunction with a geotechnical engineering study carried out from Mile 725 to Mile 632 of the proposed Mackenzie Highway, several major river and stream crossings were investigated. The Tsintu River 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 river crossing, are reported herin.

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

II. GEOTECHNICAL DATA AQUISTION

2.1 Field Testing

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

The special borehole and three center line boreholes were designated Boreholes 711-S-2 and 711-C-7 to 711-C-9, inclusive, respectively. Detailed borehole logs are presented in consecutive order in Appendix B.

The special borehole was drilled with a Texoma Super Economatic power auger, fitted with a 12 inch diameter stub auger. All center line 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 19 feet, and averaged 15.5 feet in depth. Sampling consisted of representative bag samples, obtained at depths of 2 1/2, 5 and 10 feet, and at depth intervals of about 4 feet, thereafter, to the bottom of each borehole. 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 and grain size distribution 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 the 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 the 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 the grain size distribution curve, Drawing No. C-1, Appendix C. Drawing No. C-2, Appendix C, presents a partial 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 the Tsintu River at Mile 711.2, approximately 10 miles south of Fort Good Hope, N.W.T.. Drawing A-1, Appendix A, is a Key Plan of the Tsintu River area and Drawing No. A-2, Appendix A, presents a detailed Site Plan. Plates No. 1, and No. 2, Drawing No. A-3, Appendix A, show the crossing from the air in June, 1973,

The Tsintu River drains a relatively large area extending east of the Mackenzie River and crossing site. The Tsintu River basin is flanked by the Jackfish Creek and Snafu River watersheds. The topography in the Tsintu River basin is relatively flat. The summer water flow is expected to be moderate and a low base flow is beleived to exist throughout the winter months.

Aerial photographic interpretation of the surficial geology of the immediate area of the Tsintu River Crossing, is shown on Drawing No. A-2 Appendix A. The surficial materials are believed to be alluvial meander plain and ground moraine or ridge-and-knoll moraine deposits. Drift covered bedrock outcrops were also noted at isolated locations. Depressions were noted to be infilled with peaty soil. 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 TSINTU RIVER CROSSING

MATERIAL	DESCRIPTION	APPROXIMATE DEPTH BELOW EXISTING GRADE (FT)	RANGE OF THICKNESS (FT.)
PEAT	dark brown, silty & clayey, unfrozen to frozen, NB-V20%, moisture content (M/C) 7% to 64%	0 - 2.5	1.5 - 3.5
SILT (Boreholes 711-C-7 &711-C-8, only)	grey, organic, low plasticity, unfrozen, M/C 23% to 35%	3.5 - 6.5	2.5 - 3.5
CLAY (TILL) (Boreholes 711-C-8 &711-C-9, only)	dark brown, sandy, silty gravelly, low plasticity, some shale, unfrozen, M/C 10% to 22%	4.2 - 10	4 - 7.5
GRAVEL (TILL) (Borehole 711-S-2, only)	brown, sandy, some silt & clay, boulders & cobbles, unfrozen, low plasticity, M/C 4% to 14%	1.5 - 5(max.depth of penetration)	Not Established
SHALE	dark grey, very soft, low plasticity, silty clay shale, unfrozen,M/C 7% to 14% (avg. 10%)	8.7 - Max. Depth of Penetration	Not Established

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

1. The maximum depth of borehole penetration was 19 feet.

2. Permafrost was not observed at any of the borehole locations within the depth of drilling penetration.

3. The gravel till,noted near the surface in Borehole No. 711-S-2, is probably of the same origin as the clay till noted in Boreholes 711-C-8 and 711-C-9. Atterberg limits from a depth of 5 feet in Borehole 711-S-2 and 10 feet in Borehole 711-C-7, are comparable, hence quantitative indication of material similarity is available.

4. No borehole information is available in the creek bed to indicate the type and nature of underlying sub-surface materials.

IV. CONCLUSIONS AND RECOMMENDATIONS

4.1 Foundation Types

Only pile foundation systems founded on or in bedrock are recommended for the support of a bridge structure at the Tsintu River crossing. 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. Driven closed end pipe piles
2. Driven steel H-piles
3. Cast-in-place concrete end bearing piles.

4.2 Foundation Design

A major factor affecting the design of pile foundations at the Tsintu River is the absence of permafrost and presence of bedrock materials at shallow depth (about 9 feet below grade). The absence of frozen zones is considered justification to allow the use of dynamic pile formulae as a supplemental approach to the determination of pile capacities in conjunction with pile load tests. The placement of piles through pile driving techniques will likely be the most expedient method of installation.

Because of a lack of data, with respect to soil strength, pile designs presented herein are largely based on empirical data , and must be considered, only preliminary in nature. Confirmation of the design parameters presented herein through addtional 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. It is considered that the only positive method of foundation support that will permit relatively high loads, without excessive settlements at the Tsintu River Crossing, is a pile foundation system achieveing support on bedrock existing beneath the site. For preliminary design purposes, it is believed that consideration should be give to the use of closed end pipe piles, steel H-piles or cast-in-place concrete end bearing piles, in bedrock.

4.2.1 Closed End Pipe Piles

It is recommended that closed end pipe 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 testing, however, pile lengths of about 60 feet are expected to be necessary(assuming 10 feet of abutment fill). Installation of pipe piles is not expected to require the use of drilling equipment. It is recommended that the piles be installed by driving to the specified set or refusal criteria. A minimum driving energy of 24,000 foot pounds is recommended. 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.

4.2.2 Steel H-Piles

Steel H-piles are also believed to be feasible, as preboring is not expected to be required. Installation without preboring is anticipated to provide high end bearing and skin friction resistance. Confirmation of this can be achieved through the driving of test H-piles at the site. A preliminary design load capacity of about 185 kips may be used for a nominal 12 inch by 12 inch Steel H-pile, weighing at least 53 pounds per foot, driven to 'refusal'. It is considered that 'refusal' will constitute a penetration of less than 0.1 inch per blow, measured over the last foot of driving with an energy of 24,000 foot-pounds.

4.2.3 Cast-in-place Concrete End Bearing Piles

Cast-in-place concrete end bearing piles are also considered feasible at the site. Installation would be by standard drilling techniques, utilizing auger drilling machines. A minimum shaft diameter of about 30 inches is recommended to facilitate inspection and hand cleaning. An allowable end bearing capacity of 20,000 pounds per square foot is recommended for preliminary design calculations, assuming the bell base is placed below a depth of 20 feet from existing grade.

4.2.4 General

It is recommended that pile driving records be kept for all driven piles, for immediate review by the geotechnical consultant. A pile load test is also recommended prior to, or at the outset of pile installation to confirm the load carrying capactiy of the piles and permit a correlation to the driving records. Preboring may be required through seasonally frozen soil or dense granular fill, to ensure pile alignment and prevent damage to the pile element. 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.

4.3 Negative Skin Friction

The effect of negative skin friction, on individual piles and pile groups, will be dependent upon the occurrence and magnitude of both consolidation settlement and thaw settlement within the fill surrounding the piles and the natural subgrade soils. At the crossing site, it is considered that the near surface peat, silt and clay till layers are relatively thaw unstable but all materials noted below an average depth of about 9 feet from existing ground surface are thaw stable. Consequently, negative skin friction effects are anticipated on foundation elements within near surface materials if these materials are frozen when fill is placed and thawing occurs at a later date. At the time of this investigation (December, 1972 and January, 1973), permafrost was not noted and the seasonal frost depth was limited to about 1 foot. Substantial negative skin friction effects may be mobilized in road grade fill surrounding piles if loss of subgrade support occurs.

To limit the amount of thaw-settlement of the subgrade, the loss of subgrade support, and the magnitude of negative skin friction, fills should be placed during the summer season. In order to further limit potential negative skin friction, due to settlement of the fill itself, it is recommended that fills be pladced to final grade and the installation of piles be carried out through the fill. The maximum time interval, which is consistent with the construction schedule, should be allowed between these two phases of construction. Consolidation settlement of the peat, silt and clay (till) is expected to be of significant magnitude and thus a contributing factor to negative skin friction.

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 the soil thaws and consolidates. Table 4.3.1 [4] presents suggested values for negative skin friction in typical soils. At the Tsintu River Crossing the thickness of fill placed, timing and method of placement will significantly effect the depth and rate of thaw wherever the soil is frozen.

4.4 Frost Heave of Piles

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

In order to prevent pile heave, it is necessary to check the pile design to ensure that the available resisting forces provide an adequate factor of safety against seasonal frost heaving. In general it has been found that 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 [4] and may be used for preliminary design purposes.

TABLE 4.3.1

NEGATIVE SKIN FRICTION OF UNFROZEN SOIL FOR PILE DESIGN

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

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

TABLE 4.4.1

FROZEN SOIL ADFREEZE BOND STRENGTH FOR PILE DESIGN
(After Woodward Lundgren And Assoiates, 1971) (4)

Design Category		Applicable Criteria: Segregated Ice Condition	Applicable Criteria: Water Content of Soil %	Design Adfreeze Bond Stress, for Frost Heaving Soils[1] (PSF)
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.

4.5 Subgrade Considerations on Center Line

As indicated in Table 3.2.1, the stratigraphy on center line, on both sides of the Tsintu River is similar. An organic cover, averaging about 2.5 feet in thickness (ranging from 1.5 to 3.5 feet), was noted at all borehole locations. The organic layer ranged from unfrozen to frozen, with visible ice being observed (up to 20 percent by volume noted). The natural moisture content ranged from 7 to 64 percent. A layer of silt was logged in the vicinity of the river channel between average depths of 3.5 and 6.5 feet (thickness ranges from 2.5 to 3.5 feet). Although the silt layer was unfrozen, the natural moisture content was relatively high (23 to 35 percent). Clay till and gravel till were noted between either the surface organic layer or silt and the underlying shale bedrock. The till layers were unfrozen and existed at a relatively low natural moisture content (ranged from 4 to 22 percent). The underlying shale is unfrozen, and was noted to possess a relatively low moisture content (average moisture content 10 percent, ranging from 7 to 14 percent). The near surface soil layers are expected to be relatively soft during the summer season. However, a summer construction schedule is recommended to avoid subsequent thaw-settlement problems that would be associated with winter fill placement. 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 moderate to good shear strength conditions will exist in the gravel and clay till, and shale, but low shear strength will exist in the peat and silt layers.

Assuming that a summer construction program is undertaken, it is anticiapated that the foundation soil will be entirely frost free, thus thaw-settlement will be non-existant. If winter construction is selected, it is anticipated that thaw-settlement of the approach fill will range from 0.5 to 1.0 feet. These estimates of settlement do 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 need not be followed at this site. It is beleived that if the organic layer can be stripped, without serious construction problems, significant settlement can be avoided. Fills for bridge approaches should be constructed with allowance being 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 at the Tsintu River Crossing. Local fine grained materials, such as silt, or clay and gravel till, are not considered suitable for abutment or approach fills. The thickness of road grade material required to prevent seasonal frost penetration of the subgrade 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 summer period to avoid construction on a frozen subgrade and subsequent thaw-settlement. Possible damage to the existing ground cover and slopes by construction equipment is most critical at this time of year, hence special precautions are advised. Placement of the fill should be undertaken by end dumping with subsequent spreading by dozing equipment. A minimum initial lift thickness of 2 feet is suggested. Depending on construction completion schedules, placement of fills may be staged for several seasons or carried to completion as construction progresses.

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

4.6 Slope Stability Considerations

No evidence of recent slope instability was detected on either river bank in the immediate vicinity of the proposed crossing. The slope gradient along center line ranges from negligible to about 16 degrees (about 0 to 28 percent grade) at the river banks. Cursory slope stability calculations, using implied shear strength values for thawed materials and the surveyed slope configuration, indicate an adequate factor of safety with respect to slope stability. Consequently, it is believed that approach fills can be constructed on the proposed alignment in comparative safety with respect to natural slope stability. However, it is recommended that excessive fill thickness be avoided near the crest of the river banks. 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 river 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 river 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 minimize erosion on the slope parallel to the fill. Every effort should be made to preserve the vegetal lining of all designed water courses and wherever this is impossible, coarse gravel should be used as channel lining. Transverse flow breakers should be provided at frequent intervals to reduce the rate of runoff along the fill and thereby reduce the potential for eroision 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 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 tested to determine the soluble sulphate concentration and soil acidity. However, it is recommended that the use of Type V Sulphate Resistant Cement be considered, for preliminary design puposes, for all concrete in contact with the natural soil. Soil sulphate analyses are recommended prior to construction. A minimum '28 day' compressive strength of 3000 pounds per square inch is recommended for all concrete forming foundation elements.

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

4.9 Additional Studies

In order to more accurately assess such factors as insitu shear strength, 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 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 superivsed and documented pile driving and pile load tests. Although preferable, these tests need not be carried out at actual bridge crossing 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 the Tsintu River and proposed highway crossing. This knowledge has been derived from visual, physical and analytical considertions 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

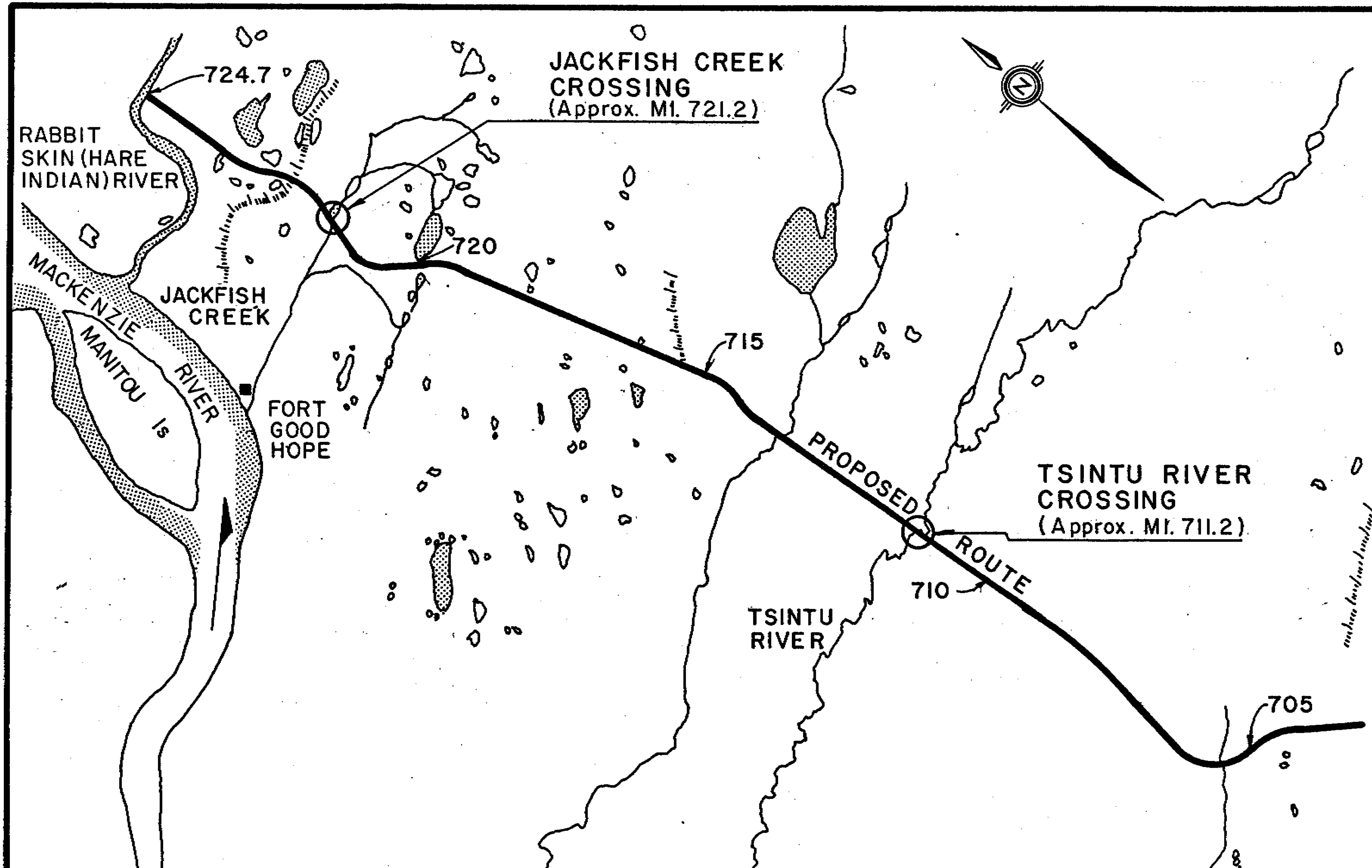
LIST OF REFERENCES

1. Yong, R.N. and Warkentin, B.P., 1966: Introduction to Soil Behavior. The MacMillan Company, New York.

2. Means, R.E. and Parcher, J.V., 1963: Physical Properties of Soils. Charles E. Merrill Books Inc., Columbus, Ohio.

3. Pihlainen, J.A., and Johnston, G.H., 1963: Guide to Field Description of Permafrost. NRC Tech. Mem. 79.

4. Woodward - Lundgren and Associates 1971: Results of Pile and Anchor Installation and Load Tests, and Recommended Design Procedures. Trans Alaska Pipeline System (Unpublished).



KEY PLAN

	DATE	SCALE	JOB No.	DWG No.
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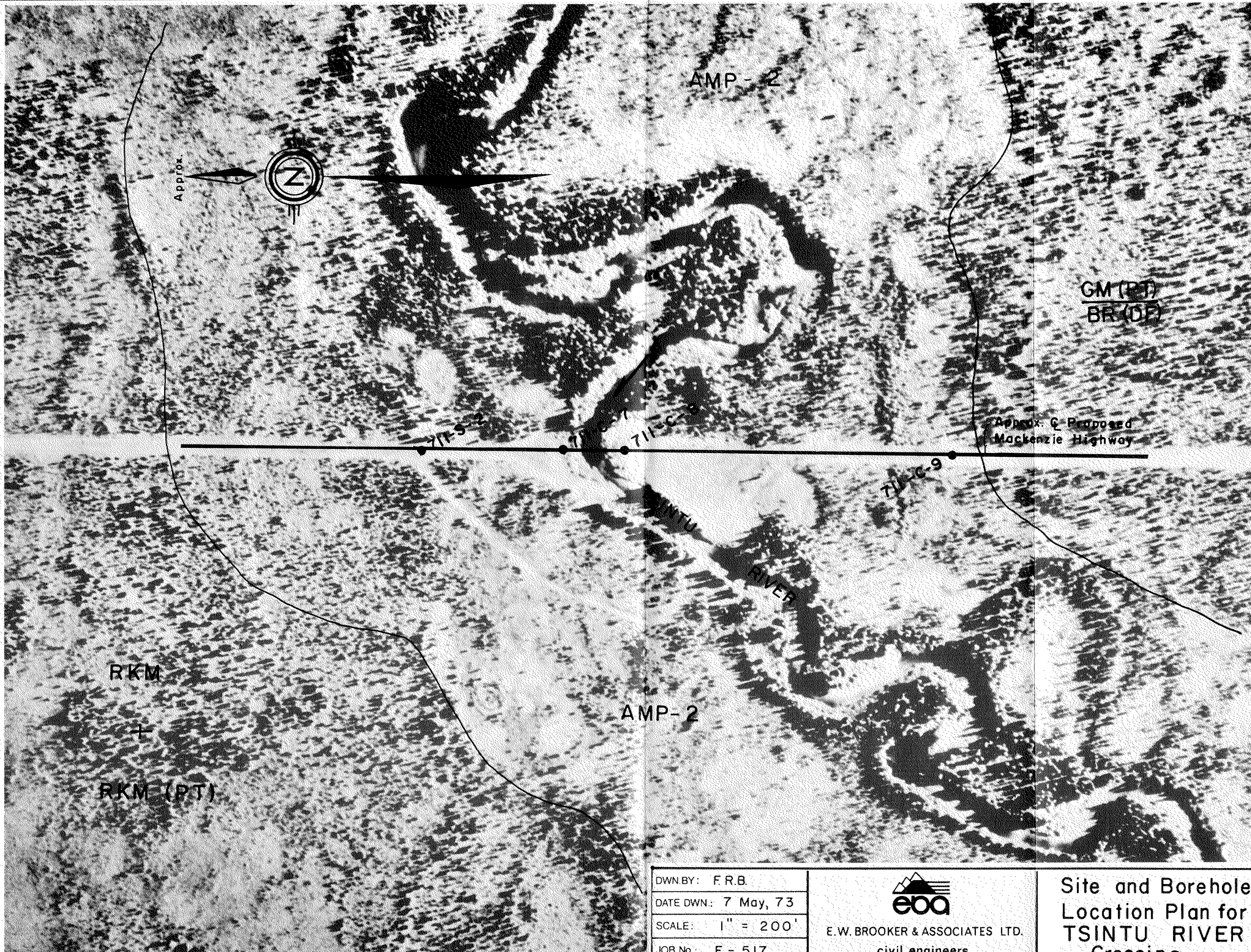


DWN. BY: F.R.B.		E.W. BROOKER & ASSOCIATES LTD. civil engineers	Site and Borehole Location Plan for TSINTU RIVER Crossing	DWG.: A - 2
DATE DWN.: 7 May, 73				
SCALE: 1" = 200'				SHT. No.:
JOB No.: E - 517				

TERRAIN LEGEND

SYMBOL	TERRAIN TYPE	PHYSIOGRAPHIC FEATURES	MATERIALS DESCRIPTION
AMP-2	Alluvial Meander Plain (excluding the Mackenzie River Plain)	Flood plains filling bottom of the stream or river valley	Fine silt, sand or gravel as channel deposits
BR	Bedrock	Outcrop to continuous ridge	Exposed rock to rock with generally less than 5 feet of cover
GM	Ground Moraine (undifferentiated)	Flat to broad gentle slopes	Silt till to clay till usually some sand and gravel
RKM	Ridge-and-knoll Moraine	Drumlinized till plain Rolling large linear features	Molded basal till low plastic silty-clay till

Topstratum Phases (Associated with Terrain Types)

Df	Thin (0 - 10 feet) of drift over bedrock surface
PT	Mixed bog and fen peats in post glacial ponded depression

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.

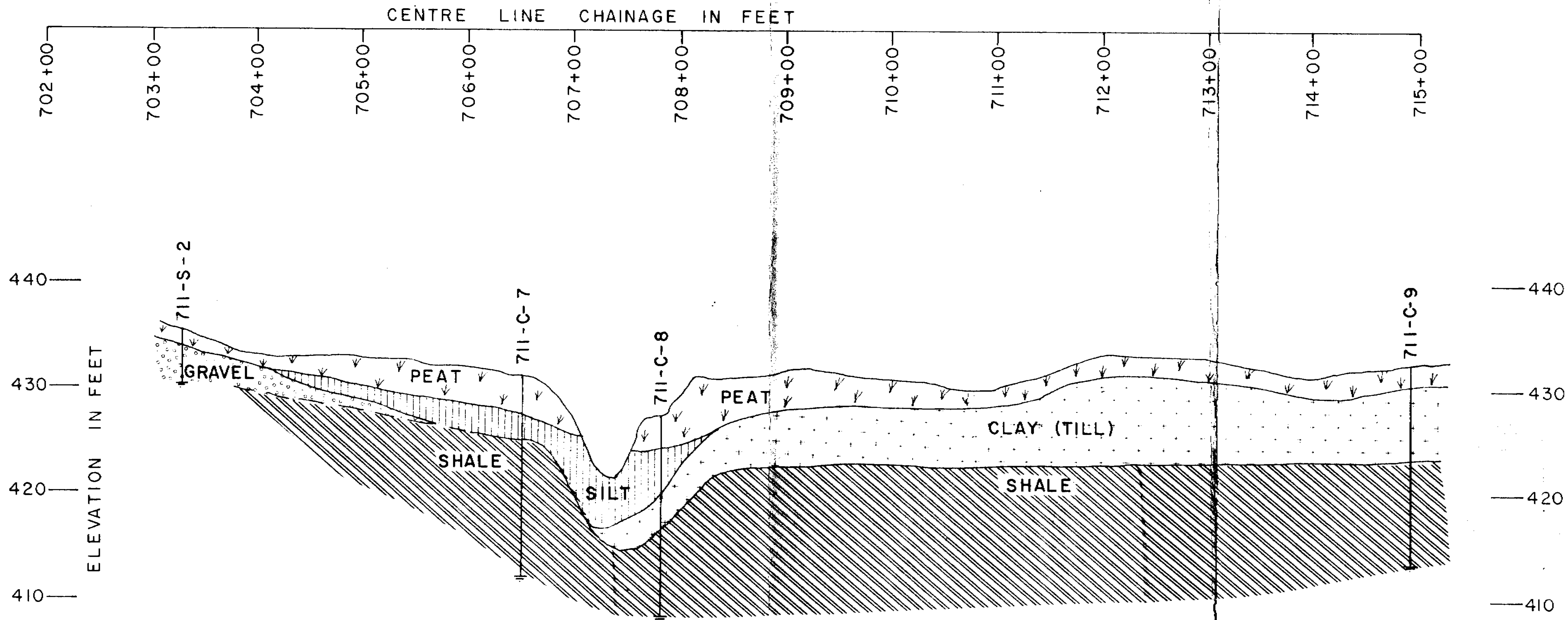
PLATE No. 1

Tsintu River Crossing
North is to the top of the plate. The proposed crossing is on the main north-south cutline.

PLATE No. 2

Tsintu River Crossing
North is to the right of the plate.

Drawing No. A-3



NOTE:

1. Stratigraphy between boreholes has been assumed
2. Elevations and profile taken from C.E.S. profile plan of Proposed ℄ for Mackenzie Highway.
3. Scales: Horiz. 1"=100'
   Vert. 1"=10'

DWN BY: F.R.B.	eba Engineering Consultants Ltd.	TSINTU RIVER CROSSING ℄ PROFILE & STRATIGRAPHY	DWG: A-4
DATE DWN:			SHT.No.
SCALE: As Noted			
JOB No: E-517			

E.W. BROOKER & ASSOCIATES LTD. — DRILL HOLE REPORT

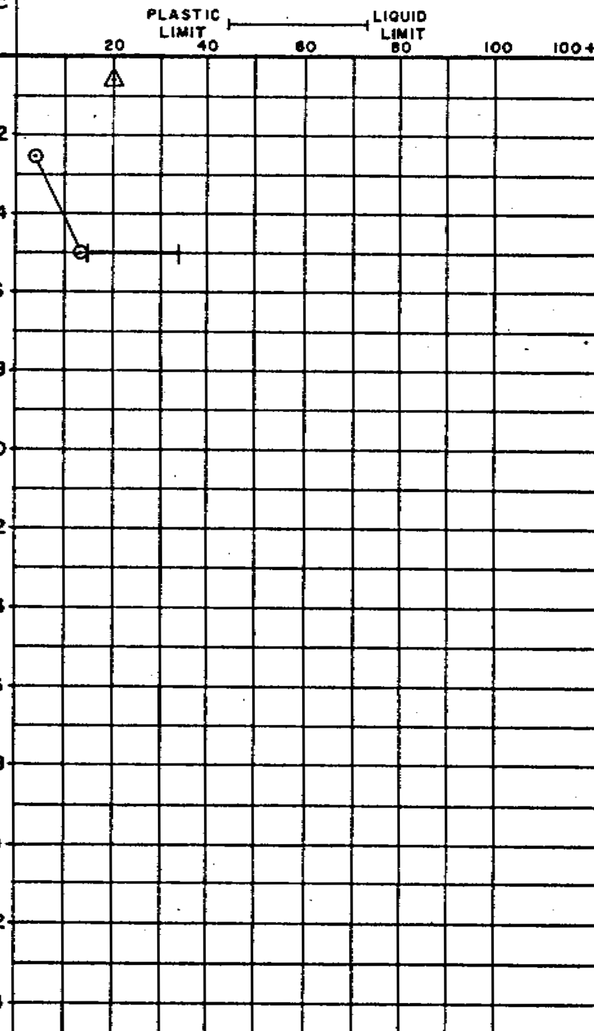
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: NRM	DATE DRILLED 1/11/73	AIRPHOTO NO: A23002 - 108	CHAINAGE: 703 +30	OFFSET
CKD: GRG	TECH: DF	RIG: Texoma	SURFACE DRAINAGE: Well Drained	VEGETATION: Burnt Out Black Spruce	ELEV: 435.5'

TEST HOLE		S12
MILE	B,C,S	NUMBER
711	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
0–1					Pt	PEAT - Dark Brown, Organic	F	NB -V 20%							
2	1					GRAVEL (TILL) - Med. Brown, Sandy, Some Silt & Clay, Low Plasticity, Angular & Subangular, Boulders or Cobbles	U	Unfrozen	(11)		32	57			
4					GW										
5	2														
8						END OF HOLE 5'									

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



E.W. BROOKER & ASSOCIATES LTD. — DRILL HOLE REPORT

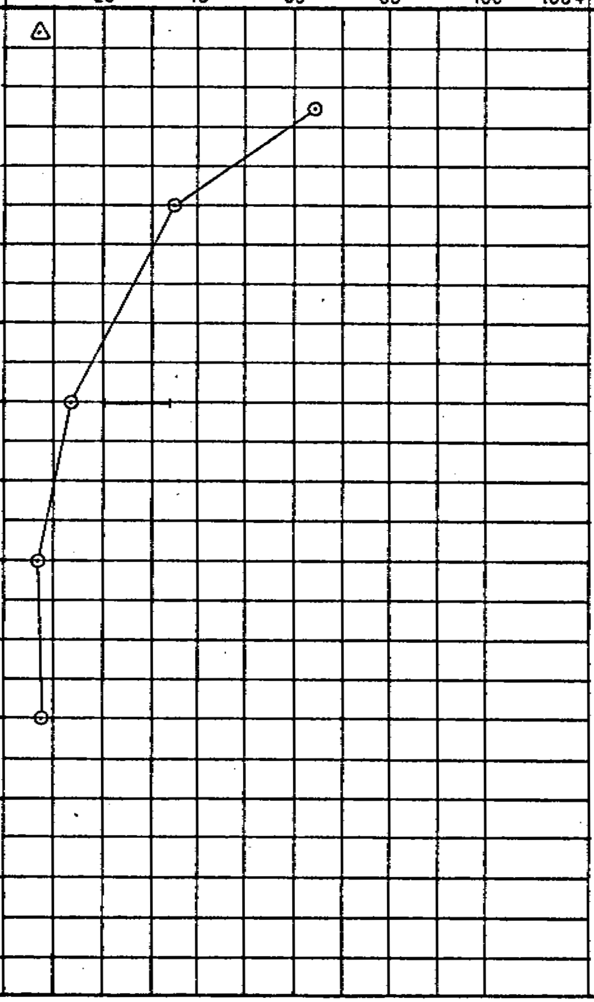
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: LAB	DATE DRILLED 12/13/72	AIRPHOTO NO: A22859-128	CHAINAGE: 706 + 50	OFFSET
CKD: GRG	TECH: TJ	RIG: Mayhew 500	SURFACE DRAINAGE: Good to South	VEGETATION: Willow - Grass	ELEV: 431.6'

TEST HOLE C105		
MILE	B,C,S	NUMBER
711	C	7

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–3.5	1 (at 2)				Pt	PEAT - Dk. Brown, Silty, Clay Peat	F / U	V 5-10% / Unfrozen
3.5–6	2 (at 4.5)				OH	SILT - Grey, Organic	U	Unfrozen
6–19	3 (at 10), 4 (at 14), 5 (at 18)				CL	SHALE - Dk. Grey, Very Soft, Silty Clay Shale, Low Plasticity	U	Unfrozen
19						END OF HOLE 19'		

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



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

REMARKS

30' North of North Bank of Tsintu River

Surrounded by 25-35' Spruce

E.W. BROOKER & ASSOCIATES LTD. — DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

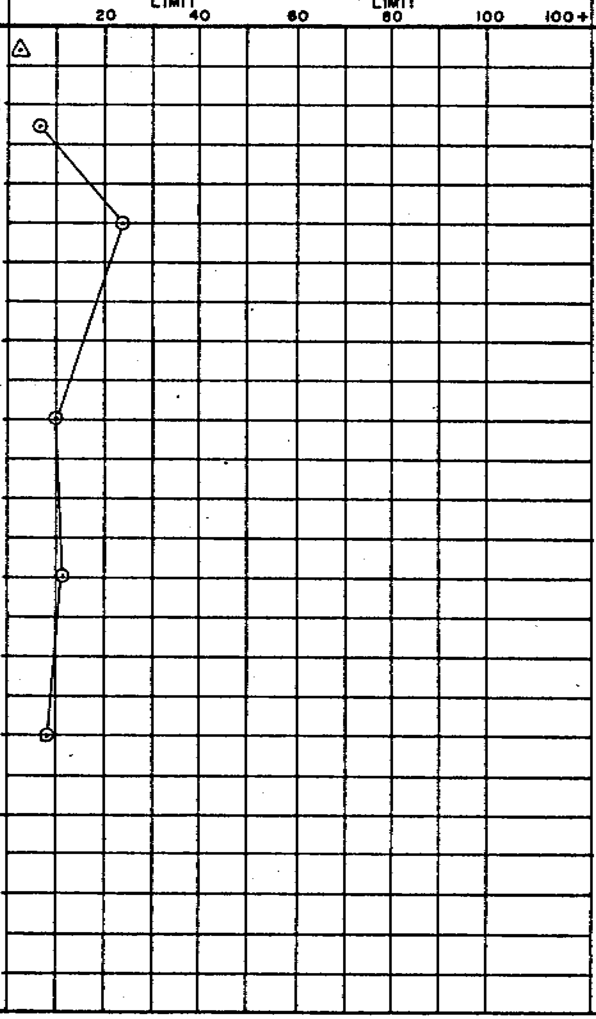
DWN: ALB	FIELD ENG: LAB	DATE DRILLED 12/13/72	AIRPHOTO NO: A22859-128	CHAINAGE: 707 + 82	OFFSET
CKD: GRG	TECH: TJ	RIG: Mayhew 500	SURFACE DRAINAGE: Good to North	VEGETATION: Red Willow	ELEV: 427.8'

TEST HOLE C106		
MILE	B,C,S	NUMBER
711	C	8

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–3.5	1 (at 2)				Pt	PEAT - Dk. Brown, Organic, Silty Peat	F / U	V 2-5%
3.5–7	2 (at 4.5)				OH	SILT - Grey, Organic Silt, Low Plasticity		
7–11	3 (at 10)				CL	CLAY (TILL)- Dk. Brown, Very Gravelly, Silty, Low Plasticity		Unfrozen
11–19	4 (at 14), 5 (at 18)				CL	SHALE - Dk. Grey, Very Soft Silty Clay Shale		
19						END OF HOLE 19'		

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

PLASTIC LIMIT ├───┤ LIQUID LIMIT



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

REMARKS

40' South of South Bank of Tsintu River.

Surrounded by 25-35' Spruce

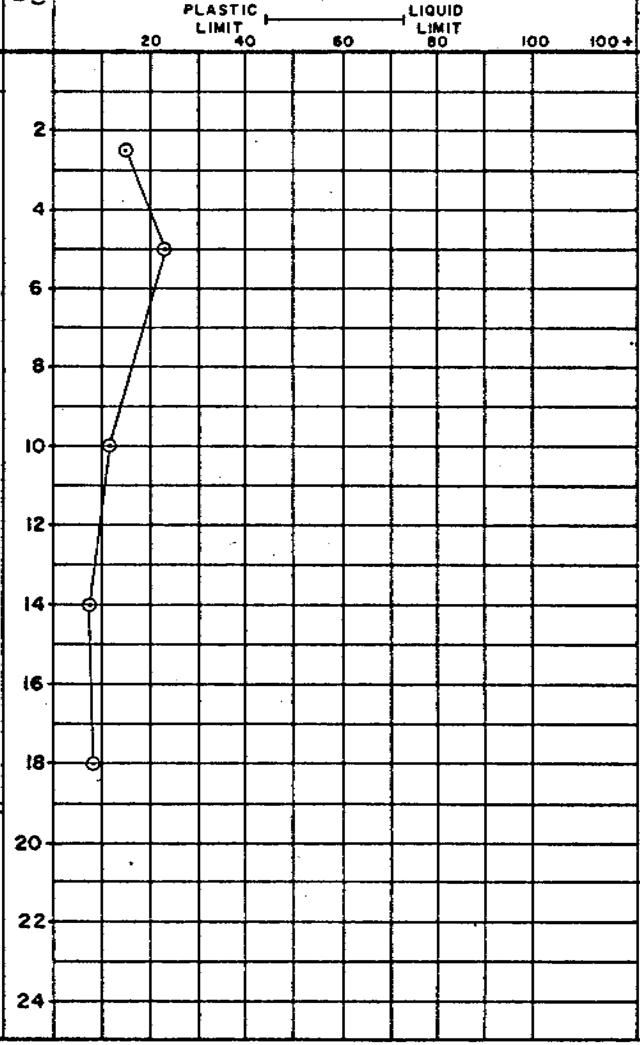
E.W. BROOKER & ASSOCIATES LTD.	DRILL HOLE REPORT	DEPARTMENT OF PUBLIC WORKS, CANADA MACKENZIE HIGHWAY

DWN: ALB	FIELD ENG: LAB	DATE DRILLED 12/13/72	AIRPHOTO NO: A22859 - 128	CHAINAGE: 714 + 94	OFFSET		TEST HOLE C107
CKD: GRG	TECH: TJ	RIG Mayhew 500	SURFACE DRAINAGE: Good to North	VEGETATION: Thin Spruce 15-25'	ELEV: 432.4'		

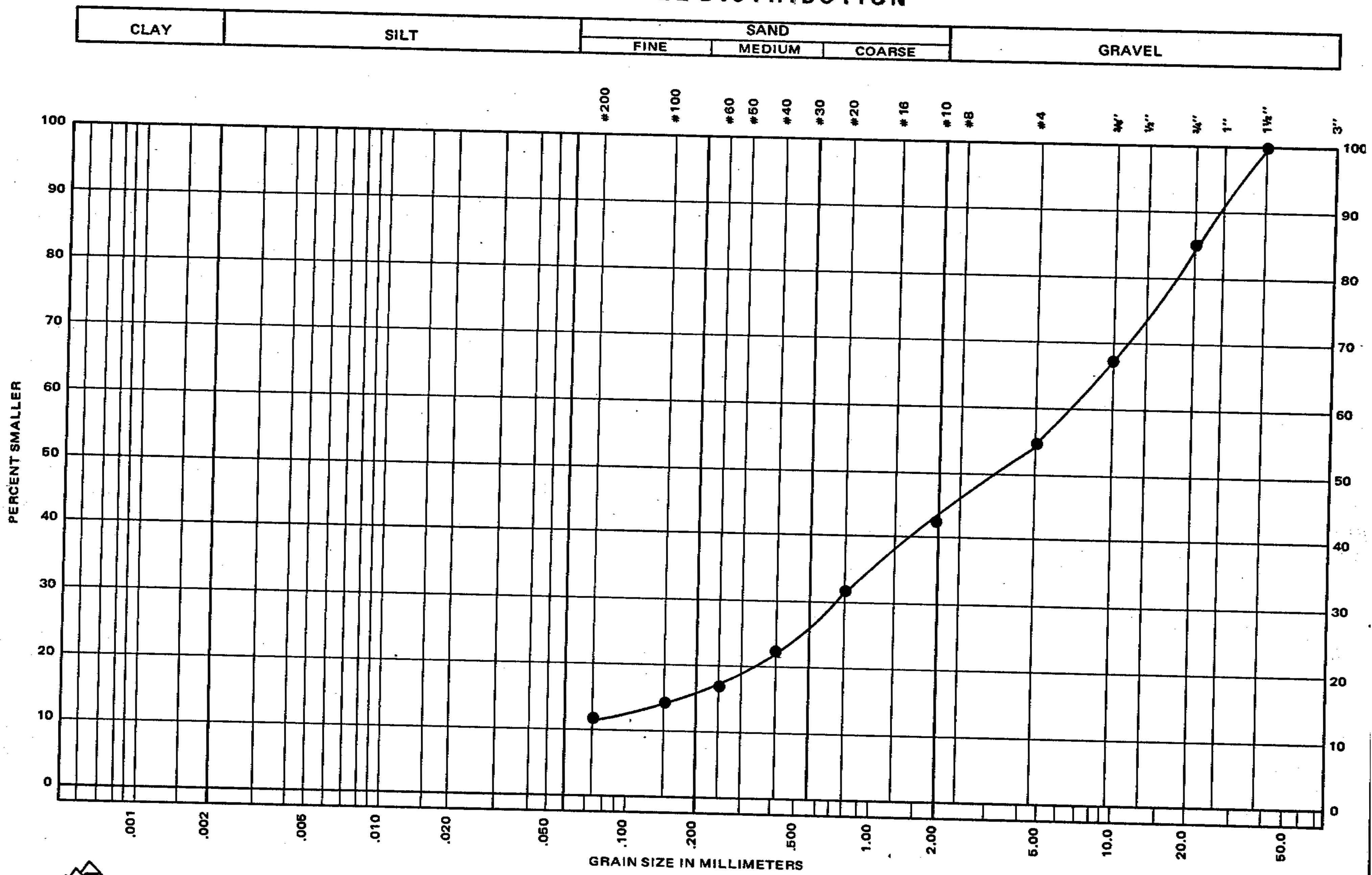
MILE	B,C,S	NUMBER
711	C	9

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	GRAIN-SIZE ANALYSIS: CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
0–1					Pt	Organic Topsoil	F / U	NB							
2	1					CLAY (TILL) - Dk. Brown, Sandy, Silty, Some Gravel & Shale Fragments, Low Plasticity, Moist		Unfrozen							
5	2				CL										
10	3				CL	SHALE - Dk. Grey, Very Soft, Silty Clay Shale									
14	4														
18	5														
19						END OF HOLE 19'									

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



GRAIN SIZE DISTRIBUTION



E. W. Brooker & Associates Ltd.

SAMPLE DESCRIPTION Gravel - Sandy, Some Silt & Clay, Well Graded

PROJECT Mackenzie Highway
JOB No. E-517 DATE Feb. 9/73
SAMPLE No. 711-S-2 Tsintu River
DEPTH 2-1/2'

Drawing No. C-1

<table>
<tr><th rowspan="2">BORE HOLE</th><th>DEPTH</th><th>NATURAL WATER CONTENT</th><th colspan="3">Atterberg Limits</th><th colspan="4">MECHANICAL ANALYSIS (M.I.T. CLASSIFICATION)</th><th rowspan="3">SOIL CLASSIFICATION (UNIFIED)</th><th rowspan="3">REMARKS</th></tr>
<tr><th></th><th></th><th>W_L</th><th>W_P</th><th>PI</th><th></th><th></th><th></th><th></th></tr>
<tr><th></th><th>feet</th><th>%</th><th>%</th><th>%</th><th>%</th><th>% CLAY</th><th>% SILT</th><th>% SAND</th><th>% GRAVEL</th></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>711-S-2</td><td>2-1/2</td><td></td><td></td><td></td><td></td><td colspan="2">11</td><td>32</td><td>57</td><td>GW</td><td></td></tr>
<tr><td></td><td>5</td><td>14.0</td><td>33.7</td><td>14.8</td><td>18.9</td><td></td><td></td><td></td><td></td><td>CL</td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>711-C-7</td><td>10</td><td>13.7</td><td>33.5</td><td>20.0</td><td>13.5</td><td></td><td></td><td></td><td></td><td>CL</td><td></td></tr>
</table>

DWG. No. C-2

EBA ENGINEERING CONSULTANTS LTD.