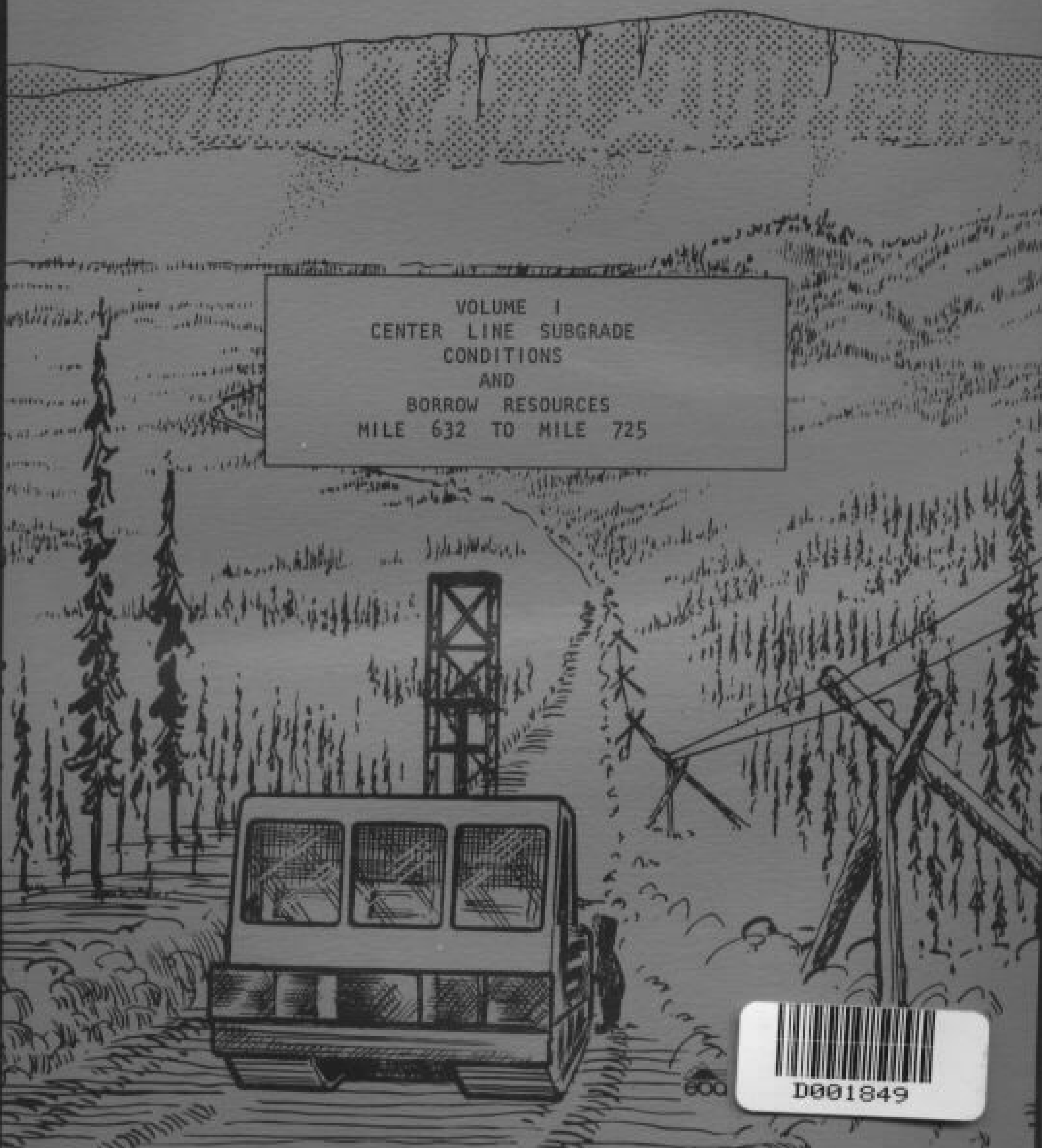
MACKENZIE HIGHWAY GEOTECHNICAL EVALUATION



VOLUME I
CENTER LINE SUBGRADE
CONDITIONS
AND
BORROW RESOURCES
MILE 632 TO MILE 725

D001849

THE ASSOCIATION OF
PROFESSIONAL ENGINEERS
OF ALBERTA
PERMIT NUMBER
P245
ELMER W. BROOKER
& ASSOCIATES LTD.

ISSUE #17

VOLUME I

CENTER LINE SUBGRADE

CONDITIONS

AND

BORROW RESOURCES

MILE 632 TO MILE 725

CONTRACT NUMBER A10/73

FILE NUMBER 9305-52-307

125

E.W.Brooker & Associates Ltd.

TABLE OF CONTENTS

REPORT ORGANIZATION BY VOLUMES

VOLUME XVII Snafu Lake Creek No. 1 and Creek No. 2 Crossings

VOLUME XVIII Donnelly River Crossing

VOLUME XIX Hanna River Crossing

VOLUME XX Elliot Creek Crossing

VOLUME XXI Oscar Creek Crossing

VOLUME XXII Bosworth Creek Crossing

VOLUME XXIII Rabbit Skin (Hare Indian) River Crossing
& Miscellaneous Small Creek Crossings

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1.1 General

The Department of Public Works of Canada (DPW), under Contract No. A10/73, File Number 9305-52-307, authorized Elmer W. Brooker and Associates Limited (EBA), to conduct a geotechnical investigation of the proposed Mackenzie Highway route between Oscar Creek, at Mile 650, and the south bank of the Rabbit Skin River (Hare Indian River), at Mile 724.7. This work was later extended (March 2, 1973) to include the section between Oscar Creek and Bosworth Creek, at Mile 632. The purpose of the investigation was to obtain geotechnical data with respect to center line subgrade conditions, potential borrow resources, and river crossings. Figure 1-1 is a Key Plan of the geotechnical study area.

Planning and mobilization commenced after authorization of the investigation on August 29, 1972, and the field work commenced on December 4, 1972. In all, 84 days were spent in the field and on 75 days some drilling was carried out. Field activities were terminated on March 13, 1973 with the demobilization of the camp, equipment and personnel.

1.2 Engineering Objectives

The geotechnical investigation was oriented towards the fulfillment of the following engineering objectives:

a) to provide a detailed exploration and evaluation of center line soil conditions, including permafrost distribution and properties.

b) to locate and evaluate potential borrow materials.

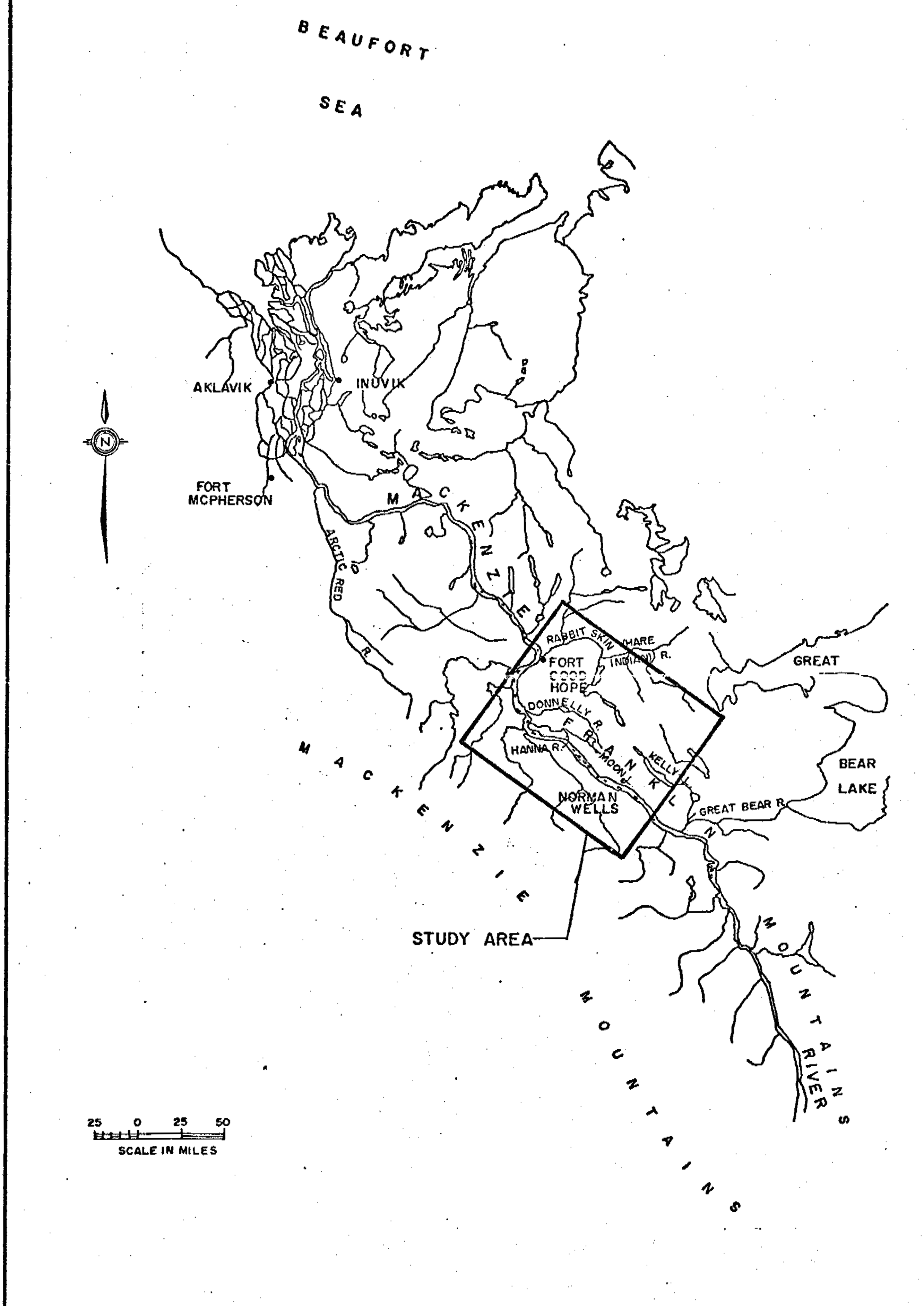


FIGURE I-I KEY PLAN GEOTECHNICAL STUDY AREA
FORT GOOD HOPE to NORMAN WELLS, N.W.T.

c) to provide detailed geotechnical investigations with design recommendations at the major creek and river crossings.

d) to locate thaw sensitive areas and to evaluate the potential magnitude of settlement.

e) to locate and evaluate areas of possible slope instability.

f) to provide a detailed terrain analysis of a two mile corridor centered on the proposed highway route.

g) to recommend additional field and engineering work beyond the scope of this study.

h) to provide geotechnical input to the design and construction of the highway.

1.3 Scope of Work

Preparations for field work included securing portable camp facilities, sub-contracting for drilling and sampling equipment, and planning the expediting and logistics of the winter program. A preliminary air photo interpretation of the highway corridor and an evaluation of the route location and river crossing sites was carried out initially to ensure proper planning of the investigation. In October 1972, preliminary inspection of the river crossings and general route location was carried out by both DPW and EBA staff, utilizing a helicopter.

The field operations consisted of the drilling, logging and sampling of 1287 boreholes, of which 803 holes were on center line and at selected 'special' sites generally on center line. The remaining 484 holes were drilled in the search for borrow materials. Moisture contents and visual classification were obtained for every sample in a field operated laboratory. Selected representative samples were returned to the EBA Edmonton laboratory for extensive classification tests and some special testing, where required.

Following the field program, a terrain evaluation of a two mile corridor, approximately centered on the highway route, was conducted by air photo interpretation, using the borehole logs for ground control.

1.4 Report Organization

The report encompasses 23 volumes of which this the Geotechnical Engineering Study is the first. Volumes II and III present the Air Photo Mosaics and Volume IV presents the Laboratory Data. Volumes V to XIV present the Borehole Logs, which have been subdivided into five sections to coincide with five major geographic sections of the route. These sections are outlined in Table I-1 and in Figures I-2 to I-5, respectively. Volumes XV to XXIII present the detailed river crossing reports in order of occurrence southward. The subject of each volume is given in the Table of Contents of this volume.

TABLE I-1

GEOGRAPHIC SECTIONS OF ROUTE

SECTION I	Rabbit Skin (Hare Indian) River to Snafu (Sucker) Lake	Mile 724.7 Mile 706*
SECTION II	Snafu (Sucker) Lake to Donnelly River	Mile 705 Mile 689
SECTION III	Donnelly River to Hanna River	Mile 688 Mile 669
SECTION IV	Hanna River to Oscar Creek	Mile 668 Mile 649
SECTION V	Oscar Creek to Bosworth Creek	Mile 648 Mile 631.5

* Mileages noted for each section are inclusive.

II. GENERAL FIELD OPERATION

2.1 Field Party Personnel

The field party consisted of thirteen members. The crew and their duties are described briefly in Table II-1. In addition to the permanent staff, a helicopter and pilot were stationed in the camp for much of the program and a Department of Public Works inspector was stationed in the camp after the Christmas break.

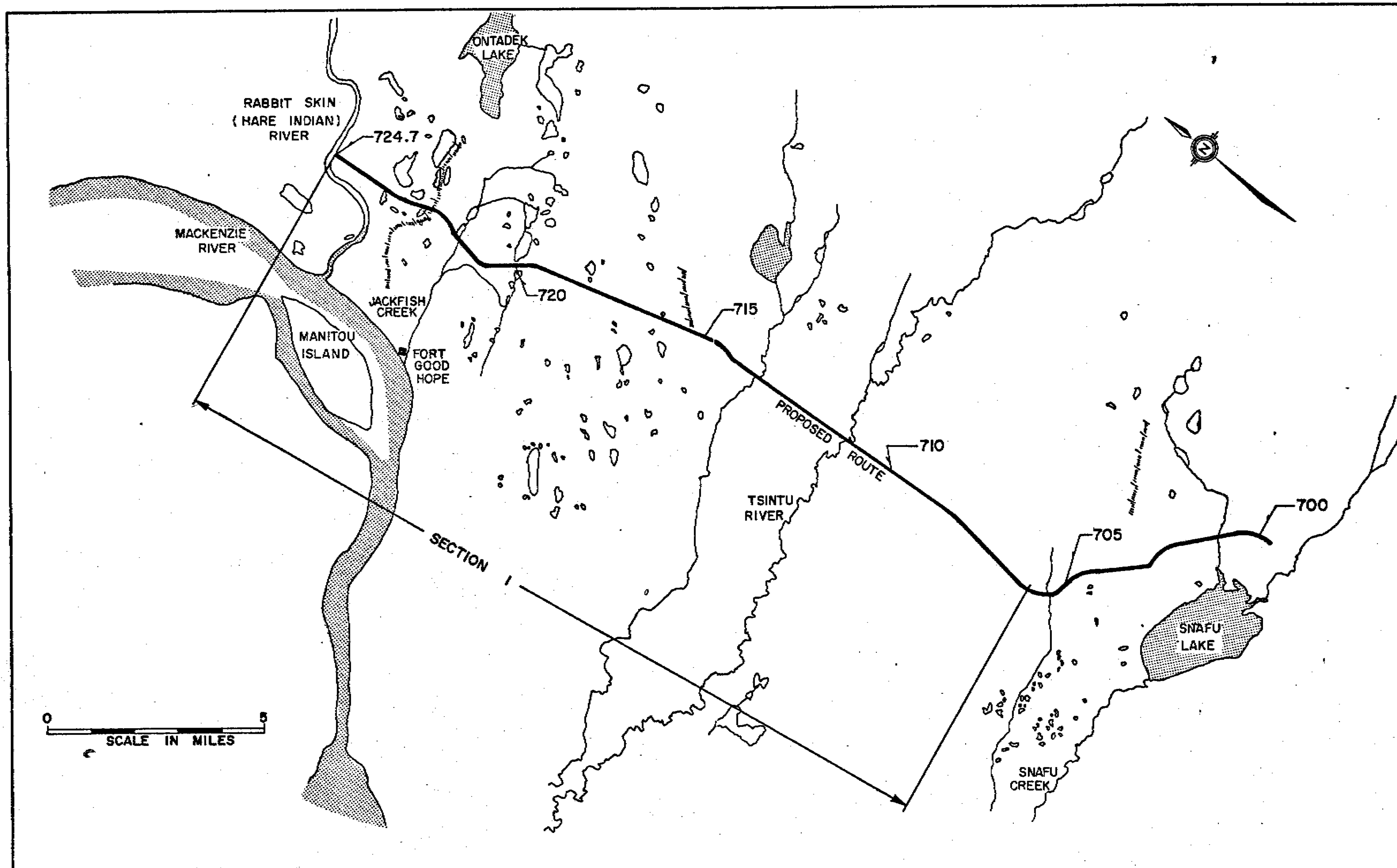


FIGURE I-2 PROPOSED ℄ MACKENZIE HIGHWAY BETWEEN MILE 700 AND MILE 724.7

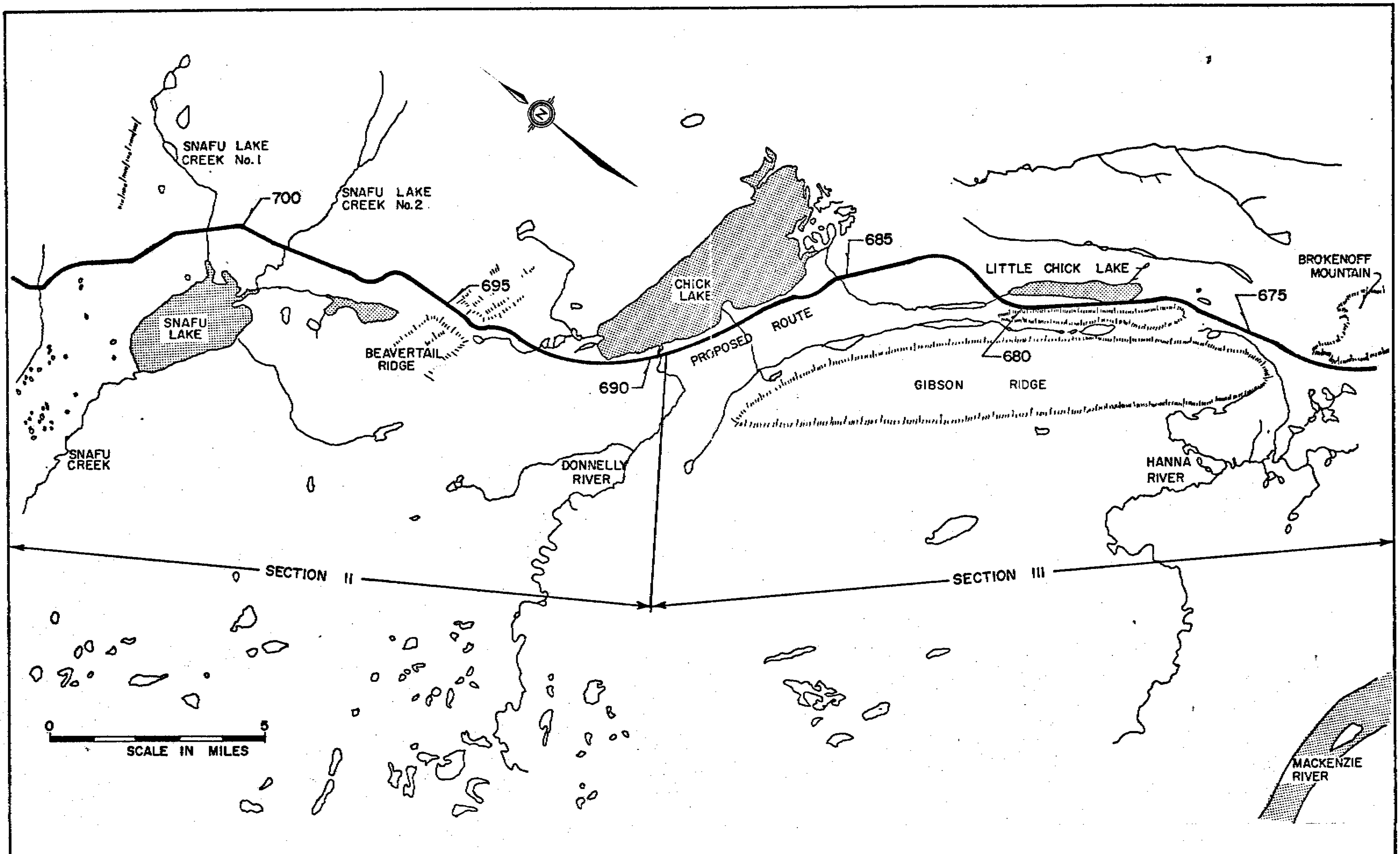


FIGURE I-3 PROPOSED ℄ MACKENZIE HIGHWAY BETWEEN MILE 680 AND MILE 700

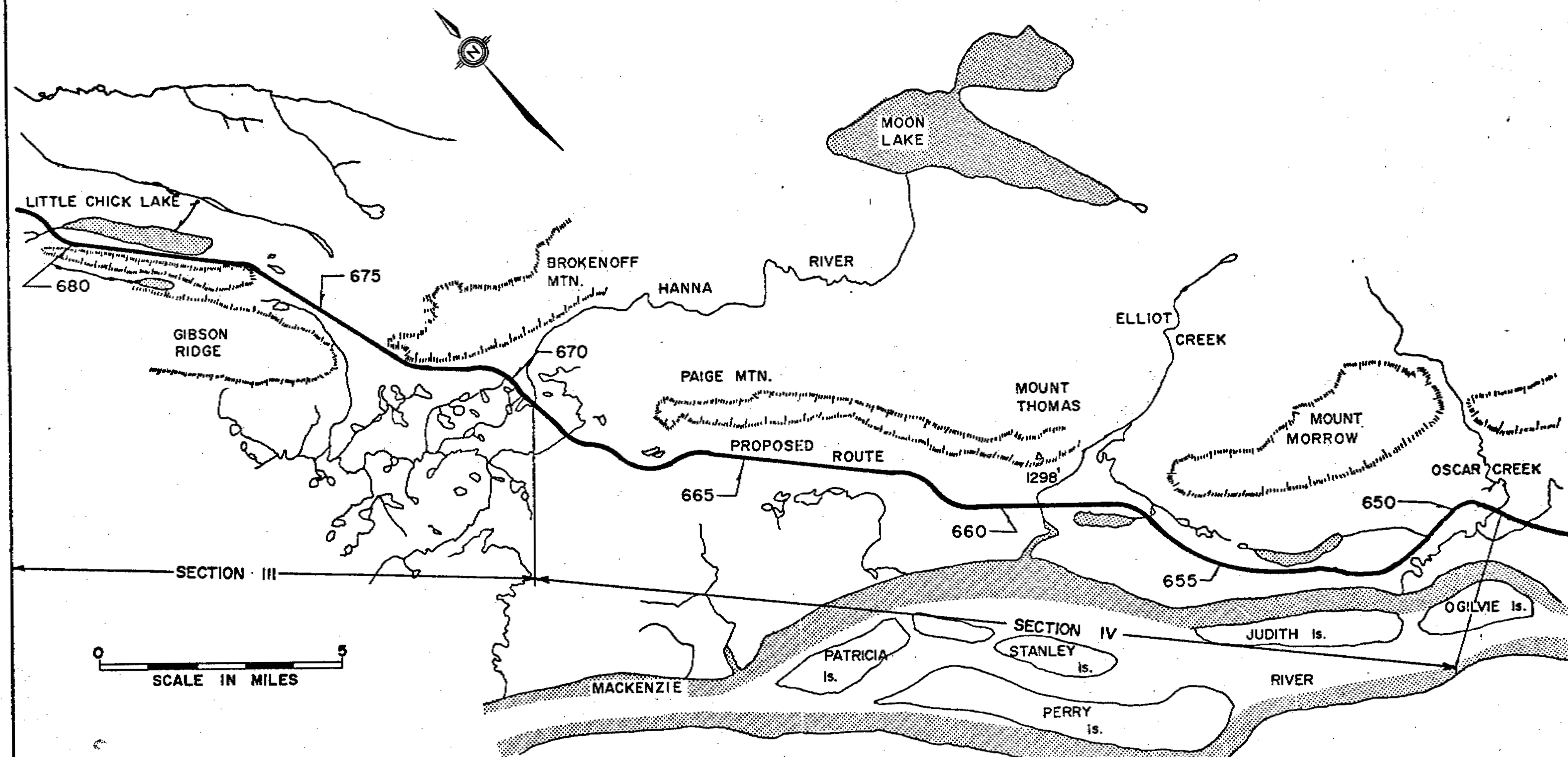


FIGURE I-4 PROPOSED ℄ MACKENZIE HIGHWAY BETWEEN MILE 650 AND MILE 680

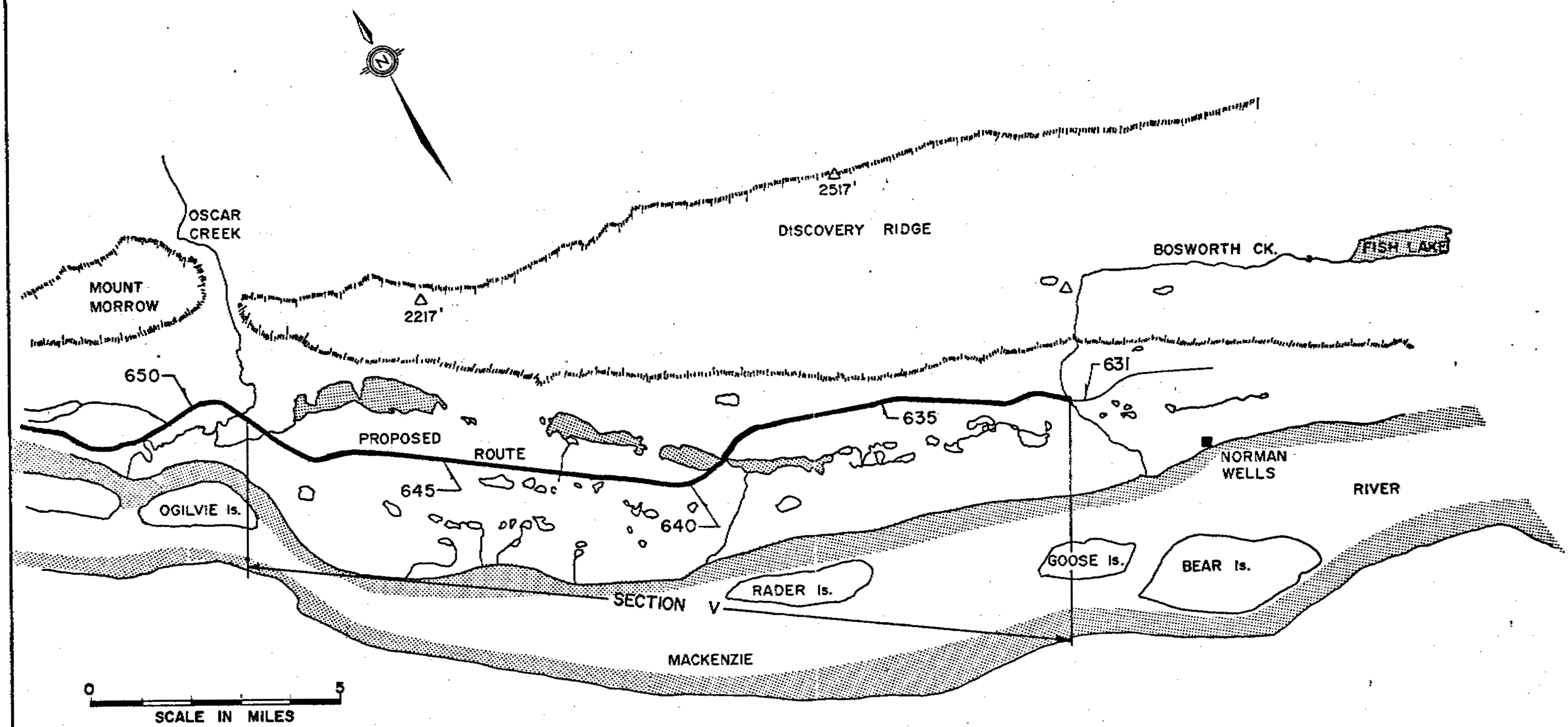


FIGURE I-5 PROPOSED ℄ MACKENZIE HIGHWAY BETWEEN MILE 631 AND MILE 650

TABLE II-1

FIELD PARTY PERSONNEL

PERSONNEL	FIRM	DUTIES
Party Chief and Field Project Engineer (1)*	EBA	Supervise field drilling program - select borrow sites - lay out special and potential borrow borehole locations
Senior Technician (2)	EBA	- locate center line boreholes - prepare field logs - assist in field laboratory operation
Senior Laboratory Technician (2)	EBA	- conduct field laboratory program - direct clearing operations into borrow areas - surveying where required
Drillers (2)	1) Garrity & Baker 1) Mobile Augers	- operate and maintain the respective drill rigs
Driller's Helper (2)	1) Garrity & Baker 1) Mobile Augers	- assist driller
Dozer Operator (1)	Arctic Circle Enterprises	- operater D6-C Dozer
Camp Manager (1)	Northern Geophysical	- maintain camp equipment and supplies - maintain vehicles for transportation
Cook (1)	Northern Geophysical	- prepare meals
Cook's Helper (1)	Northern Geophysical	- assist cook - camp cleanup

* Number of staff in each category.

2.2 Camp

The camp, consisted of an Office-Sleeper Trailer, a skid-mounted Power Supply Shop, a Nodwell FN-110 mounted Kitchen-Dining Room-Sleeper, a Nodwell FN-110 mounted Utility-Sleeper, a fuel sled mounted with four-500 gallon bulk tanks, and a fuel flat deck and supply sleigh. The camp was very mobile and during the winter five camp moves were made without retarding the drilling program.

Field transportation was predominantly by a four wheel drive Crew Cab pick-up truck and a four wheel drive pick-up. Initially, a Bombardier crew-cab was utilized. However, it was discharged part way through the program because of excessive mechanical difficulties. A Bell 206A helicopter and pilot were also available throughout most of the program. The helicopter was stationed in the camp on an occassional charter basis. Use of the helicopter was limited to reconnaissance and to support of the field equipment.

2.3 Drilling Equipment

Two drill-rigs were employed in the geotechnical investigation. One was a Mayhew 500 mounted on a Nodwell RN110. The other was a Texoma mounted also on a Nodwell RN110 tracked carrier.

The Mayhew 500 is an air circulation rotary drill rig. It was equipped to drill 4-1/4 inch to 4-3/4 inch in diameter holes to approximately 90 feet. The Mayhew was also equipped to utilize a modified CRREL core-barrel, standard Shelby Tubes, and a conventional NX core barrel. The Texoma power auger was equipped to dry auger 12 inch diameter holes to a maximum depth of 20 feet.

III. FIELD AND LABORATORY PROGRAMS

3.1 Site Selection

The location of the center line boreholes was generally left to the senior technicians logging the holes. The technicians were instructed to drill an average of 7 to 8 holes per mile, with extra holes to be drilled if conditions were erratic, and fewer if conditions were quite uniform. Holes were generally located on the crests of hills, the bottoms of any lowlying areas, and within areas of possible excessive cuts or fills. In areas where uniform conditions existed, the holes were usually spaced evenly along the center line.

'Special' boreholes were located on the banks of streams, in areas of marginal conditions, or in areas where possible borrow material could be traced on center line. The special holes were usually drilled deeper than the center line boreholes.

For most of the field drilling program, the center line boreholes were drilled in advance of the exploration for borrow materials. The areas to be investigated for potential borrow materials were usually located on the air photos, using the center line borehole data for ground control. Access to a potential borrow area was then prepared by the D6-C dozer, under the supervision of an EBA technician, using the airphotos for a base map.

Potential borrow sources were not investigated within either the Fort Good Hope Development Control Zone, or the Norman Wells Development Control Zone. Approximately five miles of the route lies within the Fort Good Hope Development Control Zone. However, the field party was denied permission for clearing and drilling proposed borrow areas within the Fort Good Hope Development Control

Zone by the Fort Good Hope Settlement Council. The Norman Wells Development Control Zone's northern boundary intersects the proposed highway route at Bosworth Creek. The only drilling in the Norman Wells Control Zone was on the banks of Bosworth Creek, where the extended contract terminated.

3.2 Drilling, Sampling and Logging of Boreholes

3.2.1 General

Center line boreholes were generally drilled to a depth of 18 feet in Sections I and II, and alternately to depths of 9 feet and 18 feet, in Sections III, IV and V. Samples were taken at depths of 2½, 5, 10, 14 and 18 feet, in regular boreholes, in Section I. In subsequent sections, the 14 foot sample was omitted and a 9 foot sample was taken in the 9 foot holes. Special boreholes and some center line and borrow exploration boreholes were extended to depths beyond those indicated. Where this occurred, representative disturbed samples were generally taken at five foot intervals to the bottom of the hole.

Rock coring was attempted, with limited success, in some of the potential borrow areas. Only badly broken core was obtained and recovery was very poor. Shelby tube sampling was attempted in cohesive unfrozen material where possible. Although large areas of unfrozen clay till were encountered, it proved too gravelly for successful Shelby tube sampling. A modified CRREL core barrel was available for coring frozen fine grained soils. However, only a few cores were obtained because of the irregular distribution of soils that could be suitably sampled with the CRREL core barrel and the difficulty encountered in extruding the cores.

In spite of the inability to obtain a representative number of 'undisturbed' samples, with the equipment available, sufficiently representative 'disturbed' samples were obtained to permit accurate classification and logging of soil materials encountered.

Ground ice contents, distribution and classification could not be accurately determined from the disturbed samples. Therefore, the classification and quantities of the ground ice that are recorded on the borehole logs were inferred or estimated from visual observations by the senior technician logging the borehole.

The ice classification used in this report and on the borehole logs is modified from the standard NRC classifications[1]*. Without good core samples of frozen soil, it is impossible to determine if the ground ice is stratified, random, in individual crystals or coatings on larger soil particles. Therefore, a general symbol of 'V' for visible excess ice ** and 'N' for non-visible ice has been used. Since it is generally possible to determine, from drilling performance, whether a frozen soil is well bonded or poorly bonded the modifying symbols B and F, respectively, have been retained to describe the state of bonding for non-visible ice content soils.

---

* Subscripted numbers refer to the List of References presented at the end of this report.

** Excess Ice - Ice in excess of the fraction that would be retained as water in the soil voids upon thawing.

3.2.2 Sample Recovery & Characteristics

The Mayhew 500 is an air circulation rig which returns 'disturbed' samples. The size of the cuttings returned ranged from chips, not greater than one cubic inch, to powder. In most cases excess ice could be visually detected in the chips or cuttings returned to the surface. From experience and through correlation with better methods of drilling and coring, it is believed that the quantity estimates of the excess ice content are reasonably accurate.

The samples obtained from the Texoma, which is a power auger, were much larger and in a much more representative condition than those obtained from the Mayhew 500. Generally, a better evaluation of the form of the ground ice and the quantity of ice in the soil could be made. Representative samples of excellent quality were often obtained on the 12" auger for unfrozen or lightly frozen materials. High ice content soils and rock samples were finely broken, making the form and quantity of ground ice more difficult to estimate.

Because of the better quality of sample obtained with the Texoma, an overlap of the areas drilled with the two rigs was carried out in order to provide a check on the field sampling and classification. In most instances correlation of materials logged with both rigs was good.

3.3 Drilling Coverage and Timing

The field drilling program was restricted to the winter months between freeze-up and break-up. Mild autumn weather caused a late start in the work, and in several places open water, encountered in mid-winter, necessitated delays and lengthy by-passes. Excessively cold weather (in excess of $-50^{\circ}F$) only stopped the drilling for three days during the winter, and stormy weather did not cause any work stoppage. On 75 of the 84 days in the field, some drilling was carried out. Allowing three days for the cold weather, the remaining six days were for mobilization and demobilization of the field camp, at the beginning and completion of the contract, respectively.

Drilling statistics, summarized by the mile for each section, are presented in Table III-1.

3.4 Sample Handling

The borehole numbering system and sample numbering system employed was recommended by the Department of Public Works. This required using the center line mileage of the borehole, followed by a 'C' 'S' or 'B' for center line, special or borrow area borehole, respectively, all followed by a consecutive number assigned in the field.

For example, borehole number 634-C-5 would be the fifth center line hole in Mile 634. As the drilling program proceeded in the direction of decreasing mileage, the first of any of the boreholes would be generally nearest the next highest mile (i.e. borehole 634-C-1 would be closer to mile 635 than it would be to mile 634). The samples were numbered consecutively in the order they were taken for each hole. Thus, a sample at the five foot depth would be the second sample (see Section 3.2.1) and would be numbered 634-C-5-2 for the previously described borehole.

After examination, each sample was placed in a numbered plastic bag, and all bagged samples from the hole were placed in a larger plastic bag. These were returned to the field laboratory where some of each

TABLE III-1

DRILLING STATISTICS

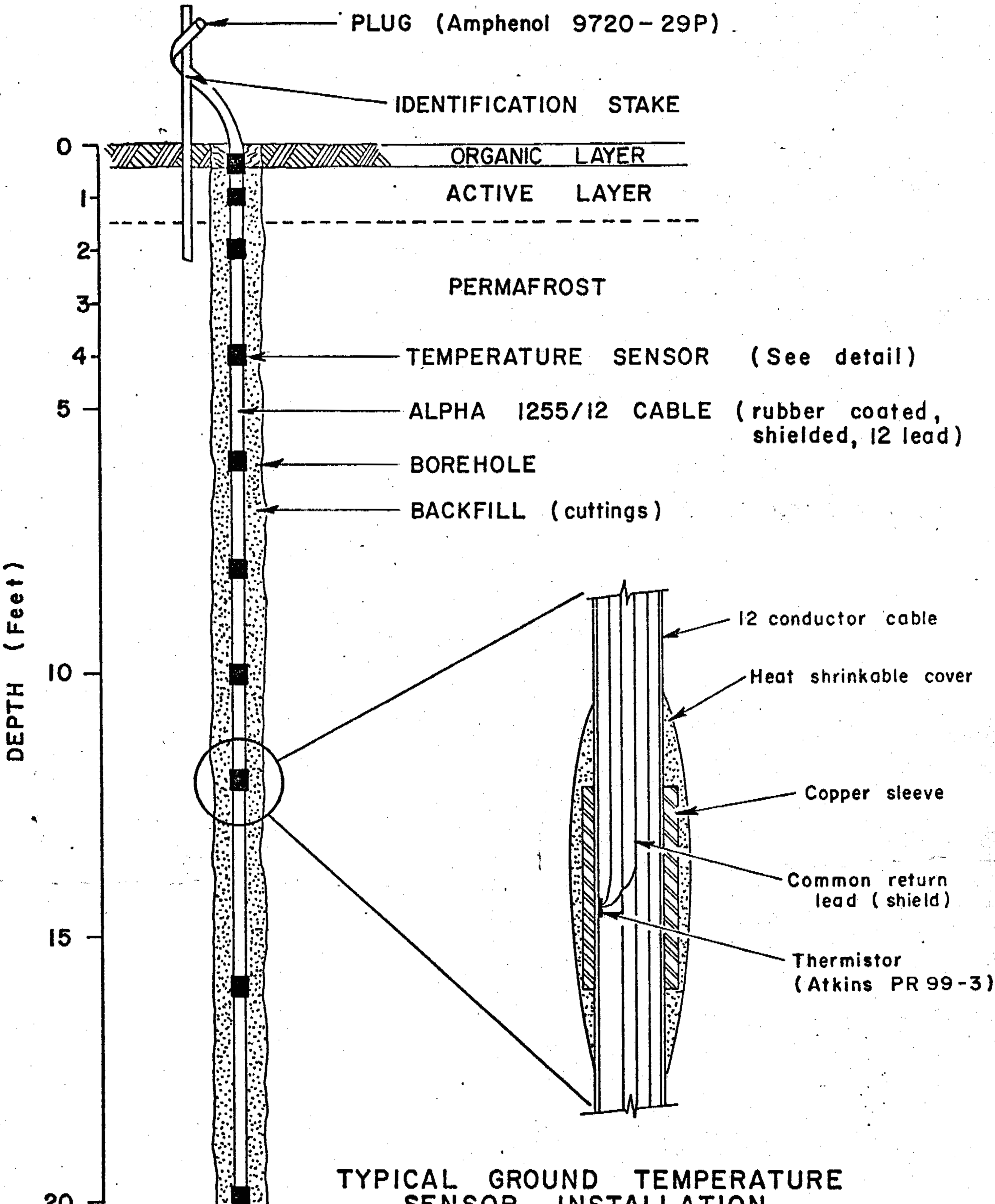
	SECTION I	SECTION II	SECTION III	SECTION IV	SECTION V	TOTAL
Length (miles)	18.7	17	20	20	17.5	93.2
Total No. of Holes	335	255	243	249	209	1287
No. of Centre-Line & Special Boreholes	163	164	174	167	135	803
No. of Borrow Area Boreholes	172	91	69	82	70	484
Total No. of Holes/Mile	17.9	15.0	12.2	12.5	11.7	13.8
Center Line and Special Holes/Mile	8.7	9.6	8.7	8.4	7.7	8.6
Borrow Area Boreholes/Mile	9.2	5.4	3.5	4.1	4.0	5.2

sample was removed for re-classification and moisture content determination. The bags were then re-sealed and the samples from three or four consecutive boreholes were collectively placed in numbered cardboard boxes and retained at the field camp. When a section of the route had been completed, representative samples were selected from the boxes and sent to the EBA Edmonton Laboratory for further testing. The boxes were resealed and taken to the Department of Public Works camp at Fort Good Hope or at Norman Wells, where they are being stored for future reference. CRREL core samples and Shelby Tube samples were also returned to Edmonton.

3.5 Thermistor Installation

At the request of DPW personnel, three ground temperature sensors were fabricated for permanent installation in boreholes along the route. Each module had ten sensors spaced as shown in Figure No. III-1. Thermistors (semi-conductor, resistance temperature sensors) were chosen in preference to thermocouples because of their superior accuracy, increased versatility (rotary switching capacity) and handling convenience. The instruments selected were production items from Atkins Technical Incorporated (PR 99-3 thermistors and 3F01 electronic thermometer). The thermistors were fastened to a copper sleeve, sealed with heat shrinkable tubing, and mounted on a vinyl covered, 12 conductor, telephone cable as illustrated in Figure No. III-1.

One of the three thermistors was damaged on installation and had to be abandoned. The other two are located in Boreholes No. 682-C-4 and 677-S-1, respectively. The initial set of readings, from February 1973, and a set of readings, taken in June 1973, are summarized in Table III-2. Figures III-2 & III-3 show the ground temperature profile in February and June, 1973. These two thermistors



TYPICAL GROUND TEMPERATURE SENSOR INSTALLATION

FIGURE III-1

TABLE III-2

THERMISTOR READINGS

Thermistor No. 1
String No. 98

Borehole 682-C-4
Initial Reading 2:06 pm,
February 16/73

LEAD	COLOUR	DEPTH (feet)	CALIBRATION FACTOR	SUBSURFACE TEMPERATURE (Fahrenheit) 16/2/73	7/6/73
A	Black	0.5	-0.2	9.4	47.0
B	Black	1.0	+0.1	15.3	33.4
C	Black	2	+0.1	20.4	30.1
D	Black	4	+0.1	27.7	30.1
E	Black	6	+0.2	31.8	32.9
F	Blue	8	+0.2	31.5	29.5
G	Red	10	+0.1	32.0	30.8
H	Yellow	12	+0.2	31.2	28.8
J	Brown	16	+0.2	31.6	31.3
K	Green	20	+0.2	30.8	30.2
L	--	--	--	--	--
M	White	Common			

Thermistor No. 2
String No. 99

Borehole 677-S-1
Initial Reading 10:50 am,
February 18/73

LEAD	COLOUR	DEPTH (feet)	CALIBRATION FACTOR	SUBSURFACE TEMPERATURE (Fahrenheit) 18/2/73	7/6/73
A	Black	0.5	+0.2	8.2	43.4
B	Black	1.0	+0.1	12.4	36.9
C	Black	2	+0.2	17.1	31.3
D	Black	4	0	23.9	29.6
E	Black	6	+0.1	26.9	28.9
F	Blue	8	+0.2	29.2	28.5
G	Red	10	+0.2	30.4	28.4
H	Yellow	12	+0.3	30.6	28.5
J	Brown	16	+0.2	30.4	29.2
K	Green	20	0	30.3	29.6
L	--	--	--	--	--
M	White	Common			

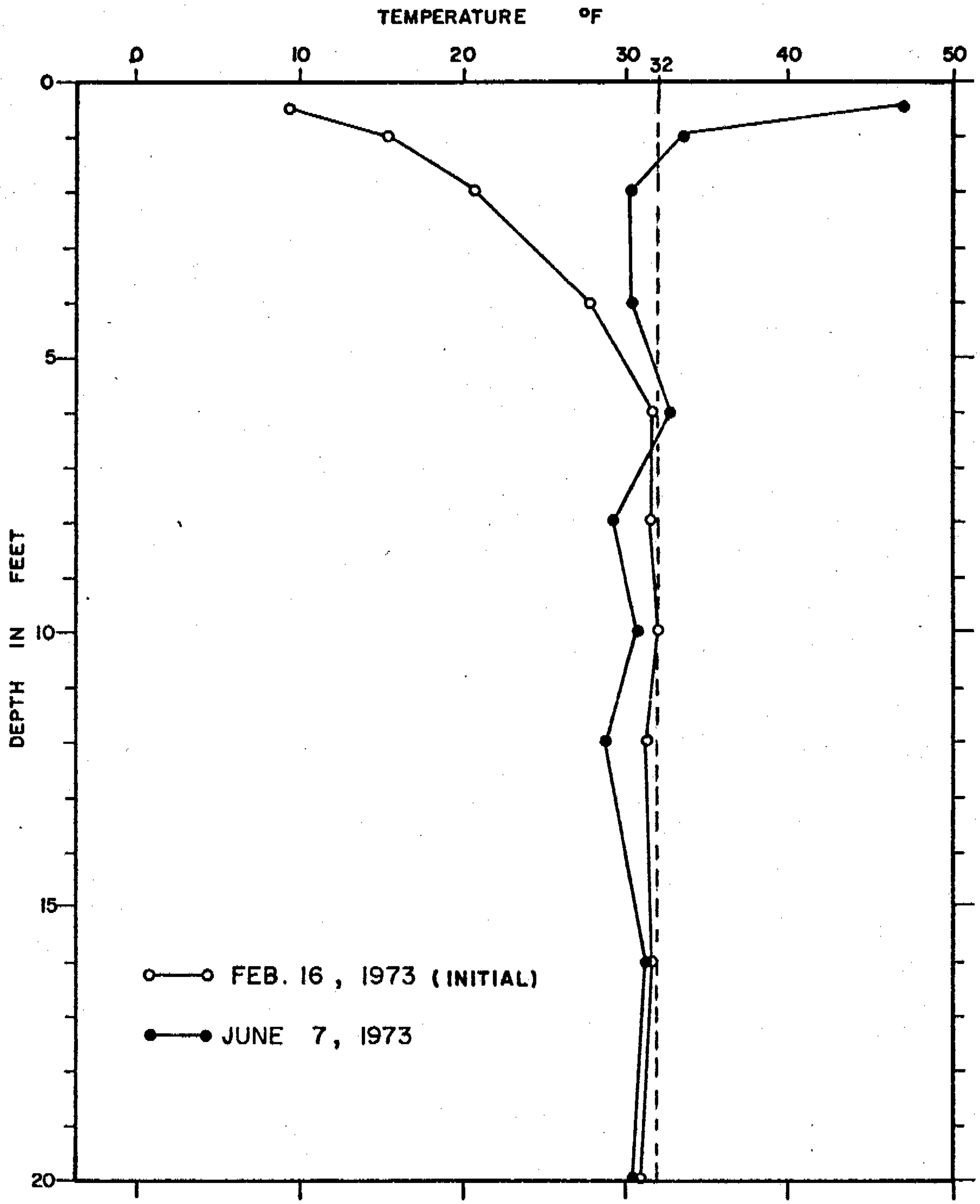


FIGURE III-2 GROUND TEMPERATURE PROFILE FOR BOREHOLE 682-C-4

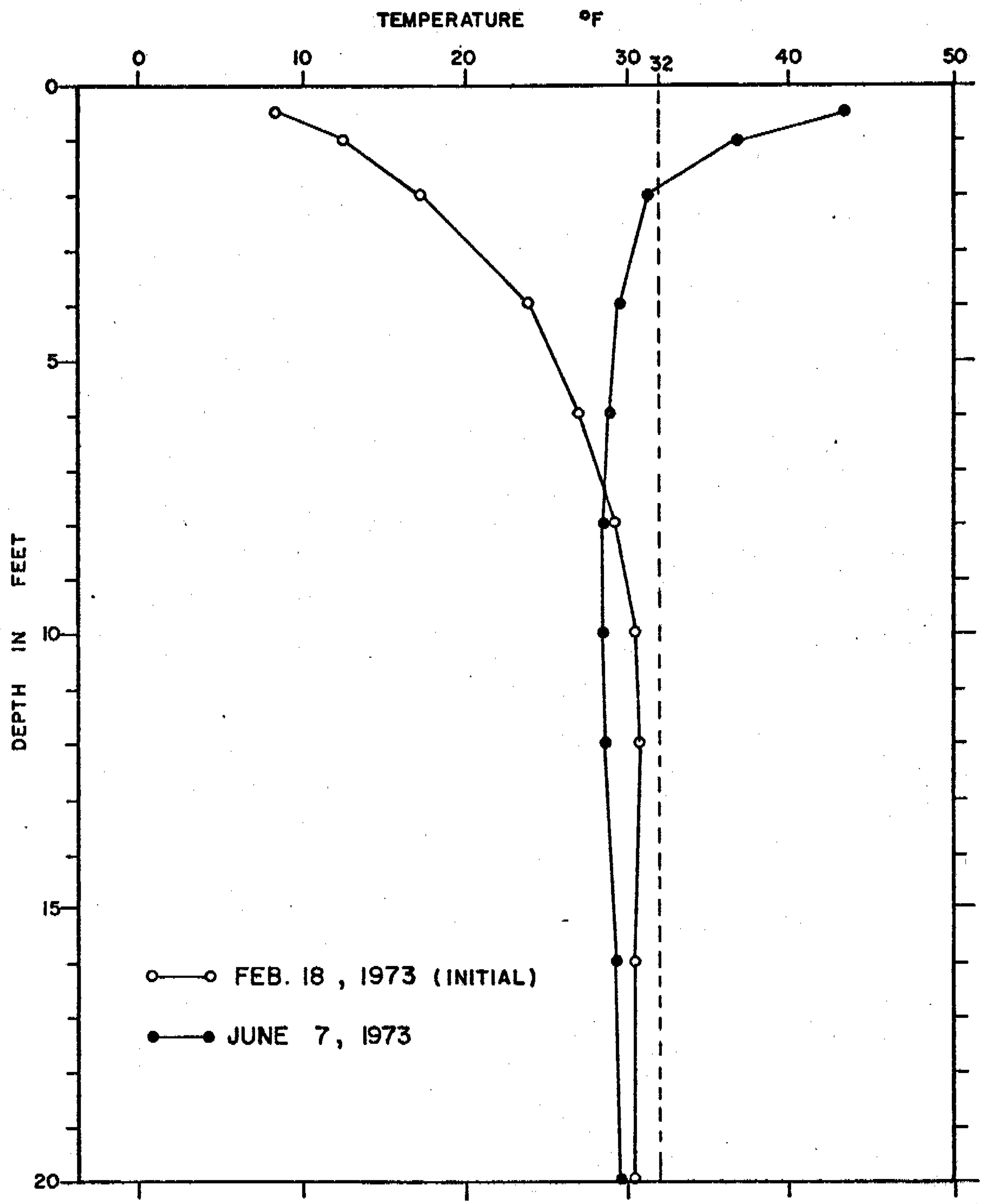


FIGURE III-3 GROUND TEMPERATURE PROFILE FOR BOREHOLE 677-S-1 are still in place. However, String No. 2 was noted to be damaged, when the second set of readings were to be taken, and it was necessary to remove the coupling plug and connect directly to the wire leads. In order to facilitate future readings a wiring diagram, will be presented, on request.

3.6 Laboratory Program

The laboratory program was designed to provide basic classification data for the soils and to verify the field logging of the material. In the field laboratory, moisture contents were determined for every sample and each sample was visually reclassified. The final field borehole logs were prepared based on both the field and laboratory soil classifications.

In the EBA Edmonton laboratory, representative samples were subjected to additional classification tests such as Atterberg Limits, Hydrometer and Sieve Analyses, Organic Contents and Acid Reaction Tests. Some soluble sulphate concentration tests and tests to define the acidity range of the subsoil were also carried out on representative soil samples from the river crossing sites. These results were added to the borehole logs, where applicable, and are summarized, in tabular form, in Volume IV, of this report. Also included in Volume IV, are all the grain size distribution curves for the samples tested.

IV. REPORT PREPARATION

4.1 Borehole Logs

The borehole logs, presented in Volumes V to XIV of this report, have evolved from the original field notes of the logging Technician. The log sheet, as prepared by the Department of Public Works, contains space for soil and ice description and the basic laboratory classification tests. The original field log has been altered to include the results of the field laboratory classification and the Edmonton laboratory tests. The ice classifications used are modified from those outlined in the National Research Council Technical Memorandum No. 73 (see Section 3.2.1). The soils were classified according to the extended Unified Classification System [2]. The extended Unified Classification System is different from the standard system, in that, it defines soils in the intermediate plasticity range. The intermediate range, according to the extended system, is set as the range on the plasticity chart between liquid limits of 35 and 50 percent. Where necessary, the classification of soils was augmented or modified on the basis of textural classification according to the U.S. Engineers Department triangular classification chart [3].

4.2 Mosaics

The airphoto mosaics, of the highway corridor, show an area approximately two miles wide centered about the center line of the highway. The field location of all boreholes, potential borrow areas, and terrain units has been added to the mosaics. The mosaics are presented, in total, in Volumes II and III, and additional copies of each are included, with their respective borehole logs, in Volumes V to XIV.

The center line boreholes and borrow area access locations were located on the mosaics, by scaling the distances on the mosaics between points of known chainages, using the field chainage of the boreholes prepared, by the Department of Public Works Surveyors, during the winter. Alterations were made to the location of some center line boreholes and all borrow area locations when the results of the Canadian Engineering Surveys Ltd. spring survey became available.

The mileage units used on the mosaics were prepared by fixing the river crossings to the Canadian Engineering Surveys mileage and proportioning the mileages in between. It was necessary to proportion the mileages because the mosaics are generally not exactly to a scale of 1":1000', and the location of the highway on the mosaics is not precise.

4.3 Terrain Evaluation

The terrain evaluation is based on an air photo interpretation of features within the two mile corridor centered on the proposed highway. The center line borehole and borrow area boreholes were used in the preparation of the terrain map. Center line terrain summary sheets included in Volumes II and III, have been prepared to be used with the mosaics which show the terrain units. A center line terrain summary and engineering evaluation is also presented as Section VI of this volume and provides most of the geotechnical data necessary for designing the highway.

The terrain analysis followed the basic outline of the air photo study, by J.D. Mollard and Associates, for the Canadian Arctic Gas Study Limited (CAGSL). It was intended that the CAGSL Terrain Study and this terrain study be complimentary. However, the terrain symbols used on the highway study have been modified slightly from those used in the CAGSL terrain study. A legend, describing each unit, in general terms, has been included on the mosaics.

4.4 Borrow Quantities

For the calculation of borrow quantities, the potential borrow area borehole logs were scanned to locate the most suitable material. The approximate average thickness of the suitable borrow material and required stripping depths are summarized in the Borrow Area Summary Table, Appendix A of this Volume. The surface area used in the calculation of the quantities was determined from the mosaics by planimeter, after a careful air photo study of the potential borrow area extent. The extent of the borrow area was based on the borehole logs, and evident physiographic features.

4.5 Theoretical Considerations of Depth of Freeze, Depth of Thaw, Thaw Subsidence and Permafrost Distribution

No attempt has been made within the framework of this report to provide specific predictive values for depth of freeze, depth of thaw, or amount of thaw subsidence anticipated for any section of the route. In order to provide such values, a comprehensive study and analysis, which was considered to be beyond the scope of this investigation, would have to be undertaken of all factors which have an effect on the freezing and thawing phenomena in soils. However, in order to provide an indication of the theoretical methods that may be employed for predicting depth of freeze, depth of thaw and magnitude of thaw subsidence, a discussion has been included in the report concerning these methods. This discussion is presented in detail in Appendix B of this report. In addition, data obtained from this field investigation, has been analyzed to provide some of the design parameters which are required for input to the theoretical methods discussed.

An attempt was also made to ascertain the factors affecting the presence or absence of permafrost along the proposed route. In this regard, a statistical analysis of permafrost distribution, over a portion of the route, was carried out. These results are discussed in detail in Appendix B, Subsection B.1.3(b). Based on the analysis, it was determined that permafrost was encountered in 38 percent of the boreholes between Mile 689 and 725. The incidence of permafrost bore no significant relationship to the peat thickness. However, the incidence of permafrost was significantly lower on the old C.N.T. cut line and in burned over areas (29 and 30 percent, respectively) than in areas where the right of way had been freshly cleared (44 percent occurrence).

The incidence of permafrost in the section studied was found to be significantly greater in sands and silts than in other soil types. The true significance of this trend is difficult to ascertain since the results are affected, to some extent, by limited data for some soil types.

Theoretical studies indicate that, under constant climatic conditions, the presence of permafrost may be dominantly a function of near surface soil water content. Where water contents are high, permafrost will exist and it will be non existent where water contents are low. Data from boreholes between Miles 700 and 725 confirm this trend, however, there is considerable scatter to the data.

V. ROUTE CONDITIONS

5.1 Bedrock Geology

The northern section of the route, from Mile 725 to Mile 695, is on the Anderson Plain division of the Interior Plains. The section of the route from Mile 695 Southward is in the Franklin Mountains division of the Cordilleran Region. The geology of the former section of the route is relatively simple. However, in the latter section, folding and faulting have exposed much more varied and complex geologic conditions. Table V-1 provides a summary of the significant bedrock geology over which the highway must be built. The table presented is a literature[4,5] summary and most of the information is very general. A field geological study was not conducted as part of this investigation.

The pertinent geologic factors, summarized in Table V-1, are:

1) Gypsum beds and gypsum-rich limestones are found through-out the study area. These are the probable source of the sulphur-rich springs, seeps, and streams which are common near the mountains. Sulphate resistant concrete will probably be required wherever concrete is placed in contact with natural materials, or whenever local aggregate is used.

2) Exposed shales in this area are often unstable and subject to mud-flows on moderate to steep slopes, and

3) Bedrock of the Bear Rock Formation is cavernous and therefore is a potential hazard wherever it it is crossed.

5.2 Surficial Geology

All of the surficial deposits, within the study area, are glacial to post-glacial in age. The area north of Brokenoff Gap is covered by glacial age deposits consisting of glacial lake sediments, basal tills, and eskers. Some fluvial or lacustrine sediments exist between Fort Good Hope and the Rabbit Skin (Hare Indian) River. South of Brokenoff Gap, in the Hanna River Basin, a large glacial lake is believed to have developed due to a restricted outflow from the Mackenzie River at some unknown time.

TABLE V-1

BEDROCK GEOLOGY

FORMATION OR GROUP NAME	GEOLOGIC AGE	GEOLOGIC DESCRIPTION	ENGINEERING SIGNIFICANCE
Cretaceous (undivided)	Cretaceous	Shale, siltstone, sandstone and conglomerate in part non-marine. Shales are grey to black, clayey to silty micaceous, concretionary with some local ironstaining and gypsum crystals. The siltstones are dark grey to grey green, micaceous, laminated. Thin bedded bentonitic shales and bituminous beds are not uncommon.	This unit as a whole weathers into large slump blocks on moderate to steep slopes. Bentonite beds can be harzardous on any slope. The gypsum and iron staining indicate potential sulphate attacks on any concrete in contact with the material
Erosional Unconformity			
Imperial Formation	Upper Devonian	Brown and greenish brown fissile shales with subordinate impure, brown, fine-grained sandstone and siltstone beds in part non-marine	The dark coloured shales are known to be subject to mudflows
Fort Creek or Canol Formation	Upper Devonian	The upper units are grey shales, thin sandstones, bituminous shales with coral reefs and limestone. The lower units are dark platey shales	The bituminous shales are subject to spontaneous combustion
Ramparts Formation and Kee Scarp Formation	Middle to Upper Devonian	Kee Scarp part is a thick bedded and massive pale brown limestone. the Ramparts part is mainly medium bedded brown limestone	Stable, durable material. Very durable. Excellent Material
Hare Indian Formation	Middle Devonian	Greenish grey, grey and pale brown shales with calcareous siltstones in thin beds. Locally thin beds of fossiliferous limestone	Considerably less durable rock
Hume Formation	Middle Devonian	Well bedded and rubbly limestone highly fossiliferous shales in middle and lower parts	Stable and durable upper parts
Bear Rock Formation	Silurian to Middle Devonian	Solution brecciated dolomites and limestone with thin to massive beds of gypsum and some anhydrite	Sulphate attack on concrete is possible. The cavernous nature of this unit and the sinks related to it indicate a potential hazard of crossing thin roofed caverns
Erosional Disconformity			
Ronning group Mount Kindle Formation	Lower Silurian to Upper Ordovician	Brownish grey to medium grey finely crystalline dolomite and and some limestones with chert	Chert content make this material unsuitable for concrete aggregate

South of the Paige Mountain spur, almost all glacial sediments appear to have been scoured out and replaced by fluvial sediments. These sediments were deposited as the Mackenzie River, or a meltwater channel in the same valley, meandered near its mountainous eastern channel wall. Only on the high ground south of Mile 640, does a large expanse of ground moraine deposit remain. Locally, lacustrine sediments can be found in abandoned meander channels.

A detailed description of the various surficial deposits is included with the Terrain Legend, accompanying the airphoto mosaics, in Volumes II and III of this report. The probable origin of most deposits is discussed in Part VI of this report.

VI. GEOTECHNICAL ANALYSIS OF THE PROPOSED HIGHWAY ROUTE

6.1 Section I (Mile 725 to Mile 706)

6.1.1 Surficial Materials and Conditions

6.1.1(a) Rabbit Skin (Hare Indian) River Basin

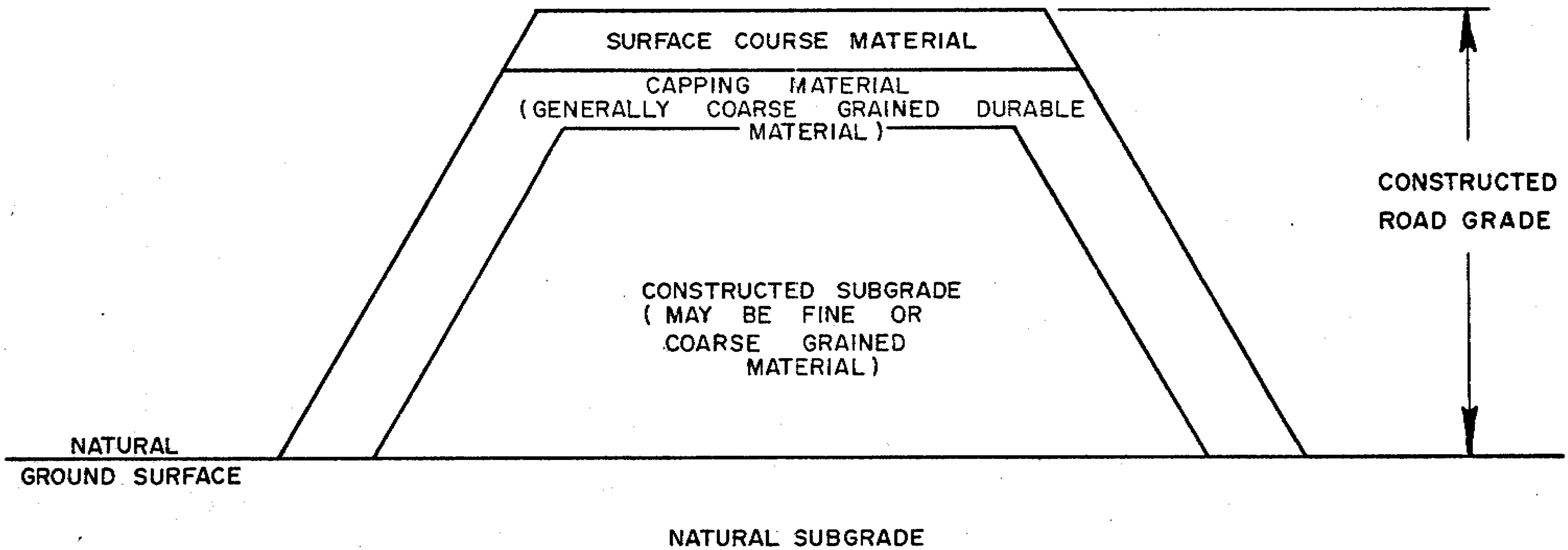
A region of glacio-fluvial or glacio-lacustrine sediments exists between the Rabbit Skin (Hare Indian) River, at Mile 724.7 (south bank), and the Fort Good Hope esker, at Mile 722.5. These sediments were deposited by a flooding ancestor of the present river or in a lake which formed in the late stages of the last glaciation. The sediments are generally silts and clays, which are siltier towards the present river, and are replaced by sandy to gravelly river terraces along the banks of the present river channel. Several small lakes and ponds have developed in lowlying pockets in the basin plain. In these lowlying areas, up to 6 feet of peat and organic soil has formed and localized ice wedge polygons, or their remnants, can be observed near the route proposed for the highway.

6.1.1(b) Fort Good Hope Esker

The esker is the principle land mark of the Fort Good Hope area. It is approximately 100 feet high, and almost 2 miles long. The esker has very steep sides and is only about 300 feet wide, at the top, where the highway crosses it at Mile 722.2. Characteristically, it is made up of well graded sand and gravel with silty inclusions and fillings in kettles. These materials were frozen where they were drilled, but because they are well drained and dry they are poorly bonded. It is expected that the esker will be an excellent source of borrow materials for road grade construction and for gravel surfacing courses. Figure 6-1, following, shows the terminology, concerning the various portions of a highway grade, as referred to in this report.

6.1.1(c) Jackfish Creek Glacial Lake Basin

Sand dunes occur, on either side of Jackfish Creek, from the esker to approximately Mile 720.7. They are presently stabilized by vegetation. The source of the sand is probably washed esker material and beach deposits which are found between the creek and the esker. These dunes are frozen and generally too small to consider for potential borrow material.



NOTE: CAPPING REQUIRED ONLY WITH FINE GRAINED CONSTRUCTED SUBGRADE SUCH AS SILTS, SANDS, TILLS AND SOFT SHALES.

FIGURE VI-I TERMINOLOGY FOR ROAD GRADE SECTIONS

From Mile 718.8 to 718.0, up to 11 feet of glacio-lacustrine clays overlie clay till. The sediments here, and to the north, are generally frozen. However, the tills encountered below these sediments, and south of here, are often unfrozen. The significant feature is that the unfrozen tills have moisture contents near the optimum for compaction. Jackfish Creek, which has the lowest elevation of this section of highway, is crossed at Mile 721.2. The creek valley is characterized by high ice content silts and organic materials. Similar glacio-lacustrine silts and clays extend, from the creek, to 180 feet above the elevation of the esker in Mile 718. A detailed description of the creek area is provided in the Jackfish Creek Crossing Report, Volume XV.

6.1.1(d) Drumlinized Till Plain

From Mile 718 to the Tsintu River, at Mile 711.3, and from Mile 708 to Mile 702, the dominant physiographic features are a series of parallel north-east to south-west trending till ridges and lowlands. Geomorphically, these ridges are best described as molded basal till. The tills are generally low to intermediate plastic clay till to clay-silt till and become siltier near Snafu Lake. They are well graded and contain gravel to cobbles throughout the region. In the lowlying areas, between the ridges, generally 2 to 3 feet of organic soil and peat have formed over the till.

For 34 representative till samples tested, the liquid limit averaged 30.0 percent and the plastic limit and plasticity index averaged 15.6 percent and 14.4 percent, respectively. For the same samples the natural water content was found to average 16.8 percent. Based on the Atterberg Limits, it is anticipated that the optimum moisture content for these tills is about 15 percent.

6.1.1(e) Undifferentiated Ground Moraine

South of the Tsintu River, at Mile 711.3 to Mile 708.2, shale bedrock occurs within 4 to 6 feet of the surface. Organic clay and peat deposits directly overlie the shale, in some areas, or overlie thin deposits of clay till. The overburden is too thin to have developed the drumlinoidal form and does not contain coarse gravel, cobbles or boulders, which are common north of the Tsintu River or south of Mile 708.2.

6.1.2 Borrow Resources

In the Fort Good Hope Development Control Zone, which extends on center line, from the Rabbit Skin River south to about Mile 719.3 the proposed highway crosses several areas judged to contain good to excellent granular borrow material. However, the field party was not permitted to explore for borrow within the Fort Good Hope Development Control Zone. Consequently, detailed information cannot be presented. It is expected, however, that good gravel borrow material exists in the river terrace nearest the Rabbit Skin (Hare Indian) River and that good to excellent sand and gravel could be obtained from the esker and the outwash delta to the west of the esker.

The first area explored for borrow, outside the Control Zone, was the extreme east end of the esker. The drilling in potential borrow area number one (BA 1) proved that the esker is a source of large quantities of high quality sand and gravel for road grade construction and for gravel surfacing courses. A grain size distribution curve, from borehole 721-B-4 on the esker, is presented in Volume IV of this report.

For the next several miles, there are no materials of apparent suitable quality or sufficient quantity to be used for borrow material. Sand dunes exist within the Control Zone, but generally they are small and, based on limited drilling, they were found to be frozen below a 10 foot active zone. Moisture contents in excess of 20 percent were prevalent. It is expected that the sand would not be economically rippable because it is frozen. In addition, if it were allowed to thaw the high water content would make excavation and placing extremely difficult. If the sand was placed as road grade material while still frozen, it would probably be unstable for a period of time when it thawed.

A ridge of exposed limestone (Upper Ramparts Formation) constitutes a possible borrow source at Mile 718.7 (BA2A & BA2B). It is believed that this limestone would require drilling and blasting, but that substantial quantities of good material could be obtained here. Drainage and stripping requirements are not expected to adversely affect the development of a quarry at this location. Anticipated pit conditions are summarized in Table VI-1 and in Appendix A of this Volume.

Additional sources of good borrow material are scarce in the remainder of this section. There are, however, two alternative sources of secondary quality material which could be developed. One source is the unfrozen tills, which were found between Mile 718 and Snafu Lake, at Mile 703; and the other is shallow shale bedrock which generally occurs at depths greater than 10 feet, except between Mile 713.3 and Mile 706.

These materials are considered to be less desireable than granular materials or quarried limestone because the shales will degrade into silt and clay within a few years. The tills also have a high percentage of silt and clay. The silt and clay content in these materials is undesireable because they are highly frost susceptible and easily eroded. The till will require a capping of granular material or broken limestone when it is placed, and the shale will require a similar capping within a few years.

Table VI-1 summarizes the borrow areas, for this section, which are believed to have the most favorable materials and conditions for development. Appendix A presents a summary of all potential borrow areas explored.

6.1.3 Drainage Considerations

Jackfish Creek is the only major drainage channel encountered between the Rabbit Skin River and Mile 718. A discussion of the creek crossing is presented in Volume XV of this report. A major spring runoff channel at Mile 724.3 is the only other drainage feature which could be a problem north of Jackfish Creek.

Between Mile 718 and 714, the route crosses two wide peat lands or fen areas. The potential for spring flooding is high and the soil conditions are poor. A re-routing has been proposed for this region to by-pass some of the problems expected in this area. This re-route is discussed in more detail in subsection 8.3 of this report.

In the drumlinized till plain, north of the Tsintu River, considerable spring flooding is anticipated. There will be a need for culverts to join all lowlying regions. A creek crosses the highway at Mile 713.6. It is expected that a considerable volume of water will pass through the lowlands on either side of this stream at the time of break-up. Consequently, a major culvert is anticipated to be required here.

TABLE VI-1

SUGGESTED BORROW AREAS - MILE 725 TO MILE 706

BORROW AREA NO.	ACCESS MILE	MATERIAL DESCRIPTION	POTENTIAL USAGE	ANTICIPATED PIT CONDITIONS	ESTIMATED QUANTITY	COMMENTS
1	721.5	Sand and Gravel	Surface Course Subgrade	Well Drained	Very Great	Ft. Good Hope Esker
2A & 2B	718.7	Limestone	Surface Course	Well Drained	260,000 yds. Minimum	Outcrop
9	715.3	Clay Till	Subgrade	Unfrozen, Low M/C	Uncertain	
10B & 11	714.5	Limestone	Surface Course	Consider-able Stripping	78,000 yds Minimum	
13 & 14	713.8	Clay Till	Subgrade	Unfrozen, Low M/C	Uncertain	Large Area
16A & 16B	713.2	Clay Till	Subgrade	Unfrozen, Low M/C	Uncertain	Large Area
17	712.7	Shale	Surface Course	May be Wet	160,000 yds Minimum	
19 & 20	709 to 710	Shale	Surface Course to Subgrade	Wet in Spring	Very Great	Potential Large Pit Location
22	706.7	Shale	Subgrade	May be Wet in Spring	200,000 yds Minimum	

The Tsintu River crossing is considered in greater detail in Volume XIII of this report. South of the Tsintu River, at Mile 711.3, a wide lowlying peaty area extends for half a mile. This lowlying area should receive careful attention when placing culverts because it will probably be flooded every spring.

Between Mile 708.6 and Mile 708.3, a lowlying region exists which could be completely flooded in the spring. It is believed that there will be a need for a major culvert system through this region. From Mile 708.3 to Snafu Lake, lowlying areas in the drumlinized till plain will be major channels for spring runoff. These channels will each require culverts.

6.1.4 Construction Problems and Recommendations

The principle design consideration and construction problem anticipated for this section of the highway will be to develop adequate supplies of good quality borrow materials. The only sources of good material are the Fort Good Hope esker, the bedrock ridge at Mile 718 and the limestone bedrock at Mile 714.5. However, much of the esker lies within the Fort Good Hope Development Control Zone and may, therefore, not be available as a source of borrow material.

South of approximately Mile 714, it is considered necessary to develop poorer quality borrow from either the unfrozen tills or from the shale bedrock. The till is more readily available and very little stripping would be required. The shale is generally deeper than 10 feet, except between Mile 713.3 and 706, where the depth to shale within the potential borrow areas was noted to range between 5 and 10 feet.

The tills are frost susceptible and, therefore, are slightly undesireable for road bed materials. However, they generally exist at natural moisture contents very near to the optimum moisture content for placement and compaction, they are unfrozen and therefore should be easy to excavate, and they are readily available. It is recommended that use of the unfrozen tills be confined to a summer construction program. During a late winter construction program, their use is regarded as undesireable because they will be seasonally frozen for a considerable depth making excavation difficult. In addition, it is desireable that the till be excavated, placed and compacted under above freezing conditions. Any delays during a winter construction schedule would allow freezing. However, in some areas where borrow area access difficulties will be encountered, an early winter construction schedule may be desireable.

The shales are believed to be compaction shales* and it appears, from uncontrolled slaking tests, that the material will break down relatively quickly. The resulting material is highly frost susceptible silt and clay which is also readily subject to scour. It is considered that this material would serve as a suitable base course for an initial two or three years, but would slowly break down into easily eroded, highly frost susceptible material. Therefore, it is believed that the shale will require a protective covering of gravel or crushed limestone or dolomite if it is to be incorporated into a road grade.

---

* Compaction Shales: are shales that have formed predominantly by compaction of silt and clay sediments with limited recrystalization and little or no cementing. The compaction bonds are usually easily broken by the swelling and shrinking of the clays as they alternately absorb and release moisture under normal weathering processes.

6.2 Section II (Mile 705 to Mile 689)

6.2.1 Surficial Materials and Conditions

6.2.1 (a) Snafu Lake Glacial Basin

From Mile 705 to Mile 700, glacio-lacustrine silts and clays, and organic soils and peat, overlie the drumlinized till plain described in subsection 6.1.1(c). The glacio-lacustrine sediments average about 6 feet thick in the lowlying areas, but are absent from the drumlinoid features. The peat and organic soil in this area is generally thicker than elsewhere. Consequently, it is believed that these areas will require more road bed fill to maintain the grade. The high water and ice contents in these soils will probably lead to significant settlements which will continue for a considerable period of time.

6.2.1(b) Beavertail Gap (Mile 695.1 to 694.1)

From Mile 699.7 to 695.1, the route passes over undifferentiated ground moraine. In this section, local peat cover is commonly up to four feet thick. Silt tills and clay tills, which are probably unfrozen in the summer season, overlie shale bedrock at about 13 feet. For the till samples tested in this region, the average liquid limit was 30.0 percent and the average plastic limit and plasticity index was 16.1 percent and 13.9 percent, respectively.

Within the gap, Mile 695.1 to 694.1, the surficial geology is very complex. Sand and gravel kames appear in one section of the route, and at least 9 feet of gravel overlies an apparent bedrock high at Mile 694.1. Two glacial lake arms; one from Snafu Lake and one from Chick Lake, approach the pass from their respective sides. It is believed that at one time the two lakes were joined through the pass. On the higher slopes, all glacio-lacustrine sediments probably have been removed by slopewash and the underlying silt tills and clay tills are exposed. These tills have a moderate to high ice content, in the upper 10 feet, where they are overlain by organic rich soil or peat. Active solifluction and slopewash continue and in several areas relatively thick deposits of peat have built up.

Alluvial fans have developed where small streams drain off Beavertail Ridge. Although center line and borrow area borings did not reveal much of the nature of these deposits it is believed that they represent an unproven but potential source of borrow sands and gravels.

6.2.1(c) Chick Lake Glacial Basin

From Mile 693.0 to Mile 691.6, the route crosses a glacial lake basin which now contains Chick Lake. High ice content lacustrine clays, overlain by several feet of organic soil and peat are common. Ice contents are estimated to average up to 25 percent in the top 10 feet of the subgrade.

Silt tills and clay tills are found, at or near the surface, between Mile 694.1, in the north, and the Donnelly River (Mile 689.7) in the south. A thin mantle of organic soils, lacustrine clay and silt, or solifluction silt occurs in local depressions. It is believed that the tills were completely overlain by glacio-lacustrine sediments which have been largely stripped away by slopewash and solifluction. Generally, the lacustrine silts and solifluction silts are high ice content material which on these moderate to gentle slopes, could be unstable.

6.2.2 Borrow Resources

An irregular deposit of well graded gravel was located at Mile 703.9 (BA25). The quantity available is uncertain, but based on a grain size distribution curve of this material, presented in Volume IV, the quality is considered to be good. It is recommended that additional drilling or test pitting be carried out to better map this deposit.

At Mile 703.4 (BA26), an apparent limestone bedrock high was located. It is not known for certain that the limestone is truly bedrock and not part of a very large boulder or collection of boulders. The limited geologic data from this area do not indicate that limestone bedrock should be found here. The limestone would, however, provide a good source of borrow material. It is believed that it will require drilling and blasting.

In the Beavertail Gap area, there are several possible borrow sources. However, from Mile 703.4 to Mile 696.1, no suitable sources of borrow were discovered. At Mile 696, BA33 and BA34 should provide some good quality sources of sand and gravel borrow material. At Mile 695.5 (BA35), limestone was located, and at Mile 694.1 (BA37) good to excellent gravel borrow was located. If additional material is needed, the exposed limestone bedrock on either side of the route should provide good to excellent material, but access will be more difficult and drilling and blasting will be required. It is possible that the bedrock may contain some of the gypsum-rich Bear Rock Formation which is soft and highly susceptible to solution.

From the Beavertail Gap to the Donnelly River, there does not seem to be any suitable borrow. On the north-west bank of the Donnelly River (BA41), shale bedrock occurs at relatively shallow depths. The material is similar to the shales described in subsection 6.1.4 and will require long term protection similar to that recommended in subsection 6.1.4. In addition, the shale bedrock is overlain by an average of 7 feet of high ice content material, which will result in difficulties in excavating and maintaining the pit during the summer period.

Table VI-2 summarizes those potential borrow areas in which good material was located and which are expected to be the best for development. A complete summary of all potential borrow areas explored is presented in Appendix A.

6.2.3 Drainage Considerations

The drumlinoid features north of Snafu Lake will channel considerable runoff towards the lake and some lowlying areas near the lake are expected to be flooded in the spring. Up to 55 percent of the portion of the route from Mile 705.2 to Mile 702.4 is on lowlying land that may be flooded in the spring. However, an acceptable alternate route, short of an extensive relocation, could not be found for this area.

The two major creeks feeding into the west end of Snafu Lake are major drainage channels which are covered in Volume XVII of this report.

TABLE VI-2

SUGGESTED BORROW AREAS - MILE 705 TO MILE 689

BORROW AREA NO.	ACCESS MILE	MATERIAL DESCRIPTION	POTENTIAL USAGE	ANTICIPATED PIT CONDITIONS	ESTIMATED QUANTITY	COMMENTS
25	703.9	Gravel	Surface Course	May be Irregular Distribution	300,000 yds. Uncertain	
26	703.4	Limestone	Surface Course	May be Flooded in Spring	80,000 yds. Minimum	
33	696.1	Gravelly Silt Till	Subgrade	Unfrozen Low M/C	650,000 yds.	
34	696.0	Gravel and Sand	Subgrade to Surface Course	Unfrozen May be Wet	460,000 yds.	May be too Wet to Handle
35	695.5	Limestone	Surface Course	Well Drained	Very Great	
37	694.1	Gravel	Surface Course	Unfrozen, Well Drained	180,000 yds. Minimum	
41	690.0	Shale	Subgrade to Surface Course	Wet, Underlies High Ice Content Material	130,000 yds. Minimum	

From Mile 699.7 to Mile 694.0, the route is cross-slope to the natural drainage direction for the area. There are no pronounced streams in this area, but there are very many slopewash gullies which will channel runoff. A culvert for each of these channels will probably be required. Channelling of the runoff could lead to major erosion and/or ponding, resulting in washing out of the road. A careful study of the problem of channelling runoff is required here.

The route runs downslope, parallel to the natural slope, from Beavertail Gap to Mile 692.5. There is a danger that the highway bed will channel drainage resulting in deep scouring in this area. This problem is compounded by the existence of high ice content material in this section. The lack of similar features associated with the CNT line, however, is somewhat re-assuring. However, a flow breaker system should be considered where the gradient is steepest. A theoretical discussion with respect to flow velocities in channels, is presented in Appendix C of this report.

The route is aligned across the natural slope between Mile 692.5 and the Donnelly River. Some deep gullies are crossed wherein drainage from the slopes to the south-west is concentrated. These gullies will require major or multiple culverts and a system to protect against their blockage with slopewash sediments. Riprapping around culvert entrances and exits is considered necessary in order to avoid severe scour and washout of culverts.

The Donnelly River crossing is discussed in detail in Volume XVIII of this report.

6.2.4 Construction Problems and Recommendations

In Section II, it is considered that the major design consideration will be drainage. From the east end of Snafu Lake to Beavertail Gap (Mile 700 to 695), the route runs cross-slope and therefore cuts across the natural drainage system for the area. From Beavertail Gap, at Mile 694 to Mile 691.5, the drainage parallels the highway over thaw sensitive lacustrine sediments and high ice content tills. It will be necessary to ensure that the highway does not channel runoff which could lead to deep scour and road bed failure. From 691.5 to the Donnelly River, at Mile 689.7, there are several deep gullies crossing the highway where large or multiple culverts and substantial fills will be necessary. The use of all rock fills is recommended for areas where ponding of runoff may occur.

On the north side of Snafu Lake, in the lowlying areas, several feet of peat and organic soil has accumulated. To protect the peat from thawing, extra thickness of road bed material will be required. The channelling effect of runoff on this material by culverts must be considered. The high ice content of the frozen peat makes it very susceptible to large thaw-settlements.

In Beavertail Gap, the route, as it is presently located, will require a deep cut possibly through bedrock (Mile 694.1). It was not possible to penetrate more than 9 feet of gravel, with either drill rig. However, it is believed that bedrock exists below the gravel. In subsection 7.3 of this report, an alternate route is proposed which will avoid the cut and follow better subgrade conditions south of the Gap. If, however, the route is maintained in its present location it is considered that a stable cut can be made in the granular material. A backslope of about 1.5:1, in the granular material, should be considered for design purposes. If bedrock is encountered, backslopes may be steepened.

6.3 Section III (Mile 688 to Mile 669)

6.3.1 Surficial Materials and Conditions

6.3.1(a) Donnelly River to Little Chick Lake

South of the Donnelly River, to Mile 686.8, the cross slopes are not as steep as north of the river. Therefore, glacio-lacustrine clays generally have not been stripped away by slopewash. The clays are often organic in the upper portion and ice contents, in the upper ten feet, average 30 percent or greater between Mile 691 and Mile 687. The glacio-lacustrine sediments are replaced by an almost featureless till plain between Mile 686.9 and Mile 680.8. In localized areas, several feet of peat or organic soils have developed, but generally clay tills and clay-silt tills extended to the depth of drilling (generally 18 feet maximum). The tills generally have a low ice content and are not nearly as thaw-sensitive as the glacio-lacustrine clays. Greater than normal road bed fill thicknesses are believed to be warranted between Mile 691 and Mile 687 to limit thaw subsidence effect in the high ice content foundation materials.

6.3.1(b) Little Chick Lake to Brokenoff Gap

A bedrock (limestone and shale) ridge, trending northwesterly to southeasterly; is exposed on the west side of Little Chick Lake between Mile 680.8 and Mile 676. The route proposed for the highway is along the east side or backslope of this ridge, which is overlain by an average of eight feet of clay till. Between the bedrock ridge and the bottom of the valley containing Little Chick Lake, silts which may be lacustrine, occur with and over the till. Visual estimates of excess ice content range up to 90 percent for the silts, but generally are below 30 percent for the till. Similarly, in the till, over the bedrock ridge, ice contents are estimated to be below 30 percent; except wherever peat or organic soils occur ice contents appear much higher. These soils are, therefore, very thaw-sensitive and protection from running or ponded water will be required. Substantial thicknesses of road grade fill will be required to minimize subsidence.

The route and bedrock ridge diverge near Mile 678 and the route continues over flat to moderately sloping ground moraine. Peat deposits, locally up to six feet thick, overlie clay till and silt till down to bedrock at 14 to 18 feet. Excess ice contents in these materials are generally less than 40 percent.

6.3.1(c) Brokenoff Gap

Very complex conditions and stratigraphy occur in Brokenoff Gap, between Brokenoff Mountain and Gibson Ridge. Irregularly frozen ground conditions exist in this section and open streams were observed in mid-winter. It is believed that both these features are related to concentrated ground water seepage through the Gap from the surficial materials east of the Gap and high bedrock shoulders to the north and south of the Gap. This seepage is believed to account for the very soft ground conditions encountered in silty soils at the bottom of the gap. These soft conditions make a summer construction program very difficult, in that, access to borrow areas and along center line will be possible only after a working grade has been prepared on these materials. The advantage of summer construction is that consolidation of the thawed foundation soil will commence immediately on placement of the fill and will be relatively uniform; whereas, fill placed in the winter may undergo appreciable differential settlement, associated with thaw subsidence of the seasonally frozen foundation, followed by consolidation settlement of the foundation soils. Winter construction in this area, however, is desireable with respect to access to borrow areas and along center line.

The route passes over a moderately sloping till plain with some local peat deposits up to six feet thick between Mile 676.3 to 675.8. This material appears to have an estimated excess ice content between 10 percent and 40 percent to 10 foot depths. At about Mile 675.8, the route crosses the high water-line of a large glacial lake which formed in the Mackenzie Valley. Lacustrine silt and some clay, noted to the depth of drilling, have been deposited on these moderate to steep slopes. The ice content of the silts are generally lower than would be expected which may indicate that the material may be only seasonally frozen. If this is true, the potential for extensive solifluction movements is very high.

Patches of better drained soil exist in several places in the lower gap. They have been interpreted as hummocky moraine, but due to only limited borehole data for this material, which indicates sand, it is possible that these features are remnant deltaic or alluvial fans, or perhaps kames. They generally exist as raised mounds. The surrounding flat lying silts are generally unfrozen and very soft.

6.3.1(d) The Lower Hanna River Basin

Over five miles of the route, from Mile 673.8 to Mile 668.5, traverses a large glacio-lacustrine basin, which occupies the area bounded by the Rouge Mountain - Carcajou Ridge - East Mountain upland to the south, and the Brokenoff Mountain - Gibson Ridge - Bat Hills upland to the east and north. The route runs along the front (west side) of Brokenoff Mountain where well developed talus slopes should supply excellent borrow materials. However, subgrade supporting soils are, high ice content, glacio-lacustrine clay and silt sediments. The terrain units, based on the morphology, subdivide this region into peat covered areas and thermokarst-lake areas;each having very high ice content materials. The top ten feet of the area has an average ice content of 40 to 70 percent. Extreme care will be necessitated in design, construction and maintenance of a highway across this area. Extensive fills will be required to prevent massive thaw-subsidence. Maintenance and upgrading of areas, where subsidence does occur, is anticipated to be a long term requirement. Re-routing in this area does not appear practical.

6.3.2 Borrow Resources

Table VI-3 summarizes those potential borrow areas in which good material and acceptable pit conditions are anticipated.

South of the Donnelly River, poor quality shale borrow was located at Mile 687.9 (BA43). This material is considered usuable but of poor quality, and is expected to quickly degrade on exposure. The material should be used with a gravel or broken limestone capping.

In Mile 686, the route parallels the crest of a limestone ridge on the back of Gibson Ridge. Good to excellent borrow material can be obtained anywhere along this ridge, but drilling and blasting the limestone will be required. This ridge continues past Little Chick Lake, almost to the entrance to Brokenoff Gap.

At the entrance to Brokenoff Gap, a poor quality source of borrow material was found in a low shale bedrock ridge. This appears in BA 52 and 53, in Mile 676. In the gap, there are several possible sources of borrow material. The bedrock walls could be developed. However, they are generally quite steep and access is difficult. Features similar to the sand mound in BA 57 are found at lower levels in the Gap, but they have not been drilled and the quality and quantity of material has not been proven. BA 57 was too wet for development.

TABLE VI-3

SUGGESTED BORROW AREAS - MILE 688 TO MILE 669

BORROW AREA NO.	ACCESS MILE	MATERIAL DESCRIPTION	POTENTIAL USAGE	ANCICIPATED PIT CONDITIONS	ESTIMATED QUANTITY	COMMENTS
43	687.9	Shale	Subgrade to Surface Course	Underlies High Ice Content Material	250,000 yds. Minimum	
45 & 46	688.5 to 685	Limestone	Surface Course	Well Drained	Very Great	Bedrock Ridge
47,48 & 49	680.5 to 678	Limestone	Surface Course	Well Drained	Very Great	Bedrock Ridge
52 & 53	676.6	Shale	Subgrade to Surface Course	May be Wet	Uncertain (Large)	
58B	675.0	Gravel	Surface Course	Friable Frozen Low M/C	660,000 yds.	
60A & 60B	672.8 to 671.5	Limestone & Dolomite	Surface Course	Well Drained	Very Great	Talus Slopes
61	671.4	Gravel	Surface Course	Friable Frozen Low M/C	700,000 yds. Minimum	

In the lower Hanna River Basin, the main source of borrow material will be the talus slopes and rock glaciers on Brokenoff Mountain. The quantity of talus material from both sources should be sufficient. Although the rock glaciers have a much flatter surface, and therefore, provide more convenient access, it is possible that they will be finer and have a higher ice content. The pits attempted in the winter with the D6-C dozer, without a ripper tooth and with ground protection pads on the dozer blade, did not penetrate more than a few feet. Consequently, an insufficient amount of data was obtained with respect to the quantity and quality of the talus material. Test pits excavated into both the rock glaciers and talus slopes during the summer would quickly prove the quantity and quality of the materials.

The outwash delta that has developed in front of the Hanna Gap (BA 61, Mile 671.4) appears to be a good supply of sand and gravel borrow. This material is generally unfrozen and readily excavatable. No major problems with pit development are foreseen in this material.

6.3.3 Drainage Considerations

The portion of the route between the Donnelly River and Little Chick Lake is generally drained through small gullies into Chick Lake. The route runs parallel to the lake shore and across the drainage path of these gullies. There will be a need for some minor culverts in this region, but the amount of runoff does not appear to be great.

At Mile 687.9, there is a deep wide drainage channel which, from the size of the delta and valley, must flow with a considerable volume in the spring. This channel is discussed in more detail in Volume XXIII of this report.

A second large stream is crossed at Mile 684.6. However, it is not known how much runoff will be channelled through this valley. It is likely that major or multiple culverts will be required.

At the outlet to Little Chick Lake, at Mile 680.9, is a creek crossing which will require a major culvert. This creek crossing is discussed in more detail in Volume XXIII. From the crossing to Brokenoff Gap the cross slopes, which the route traverses, are not expected to have large runoffs.

Several streams funnel together in Brokenoff Gap and a complex system of culverts will be necessary. Open water was noted in a couple of these streams in mid-winter, which indicates that icing in culverts may be a problem; particularly in the lower levels of Brokenoff Gap.

From the bottom of Brokenoff Gap, at Mile 673.5, to beyond the end of the Section at Mile 668.6, drainage conditions are exceptionally poor. The nearly flat lying, high ice content, glacio-lacustrine sediments and peat are exceptionally thaw sensitive. Consequently flooding, scour and ponding will have a major detrimental effect on the stability of these sediments. There are several streams and melt water channels through this area which may have some flow of sulphur rich water in the winter. Potential icing in these streams and channels should be a design consideration in this section of line.

6.3.4 Construction Problems and Recommendations

The lower Hanna River Basin is believed to be the most sensitive permafrost region encountered in this study area. The high ice content in the subsurface soils could contribute to major road bed failures and subsidence if controlled construction practices are not followed. The route has been examined for possible relocation but a significantly better alternate could not be found short of a major re-location.

This area, because of its sensitive nature, should be instrumented to provide long term information of the highway performance and to obtain valuable information for northern road construction. The advantage of using this area as a test section is that it has uniform,very adverse conditions over several miles of road. Trial designs and construction methods could be compared with respect to thermal influence of the constructed grade on the natural soil and their long range performance. It is recommended, therefore, that a carefully planned thermal analysis and thaw-settlement analysis be set up for this area. It would be desireable to have such a study initiated as soon as possible, to provide pre-construction through to post-construction data.

Borrow can be obtained from the limestone ridge which parallels the highway and which the highway follows between Chick Lake and the top of Brokenoff Gap. These materials should be of excellent quality, are readily available and should be present in sufficient quantity for construction of the highway grade.

In the Gap, there are several unproven but potential sources of borrow in low kames or deltaic mounds. If these proved unsatisfactory the bedrock walls could be developed. However, they are quite steep and access is difficult.

The development of the talus slopes of the Norman Range for borrow presents several slope stability considerations. The most apparent of these is the stability of the talus slopes themselves. They are presently at about their angle of repose and, therefore, any increase through excavation at the toe of these slopes will cause oversteepening and the possibility of some movement. The hazards involved in sliding will depend on the rate of excavation, the height of the excavation, the size of the talus rock and the method of excavation.

The possible existence of easily eroded or deeply weathered rock, on or just behind the cliff face, is significant because its nature leads to instability of the face. A pair of large rock slides on the face, beside Mile 656, extend over 4300 feet of the face. It is believed that these slides are directly related to the existence of the easily eroded bedrock units behind the original rock face.

It is beyond the scope of this investigation to determine the stability of the cliff face, above potential borrow sources in the talus slopes of the Norman Range. However, the potential for massive instability does exist and should therefore be considered. It is recommended that in this area, where the development of talus slopes is necessary, a careful inspection of the slopes and upper cliffs be performed.

6.4 Section IV (Mile 668 to Mile 649)

6.4.1 Surficial Materials and Conditions

6.4.1(a) Paige Mountain Spur

The section of the route from Mile 668.5 to Mile 667 passes over a breached anticlinal arm which extends from Paige Mountain to Carcajou Ridge. The ground rises on a long steep grade, on the north side dip slope, and crosses the first outcrop of breached bedrock at Mile 668.0. This geologic unit is recrossed on the other side of the anticline approximately 0.7 miles to the south. The significance of the anticline is that the gypsum-rich Bear Rock Formation also outcrops twice within the same 0.7 miles. Many large sinks in the Bear Rock Formation can be found within two miles of the crossing and two of these, only 3000 feet from the center line, are shown on the air photo mosaics. Several filled sinks and similar collapse features appear much closer to the proposed center line. Borehole 667-C-2 penetrated 8 feet of peat, which may be the infilling of such a sink and may or may not be stable.

6.4.1(b) Paige Mountain to Elliot Creek

The surficial deposits south of Mile 667.3 are more fluvial, or glacio-fluvial, in origin than lacustrine. The sediments found in this region resemble flood plain sediments more than lacustrine sediments. Strand lines or river terraces are composed of well sorted sand and the sediments below the strand lines are medium to high ice content lacustrine-like clays; whereas, the sediments above the lowest strand line are dominantly flood plain like, low ice content, silts with some clay bedding.

The sediments here are much siltier and have a much lower average ice content than those in the lower Hanna River Basin and almost certainly have a different origin. It is believed that they represent flood plain material which was deposited in early post-glacial times before the Mackenzie had incised itself as deeply as at present. These materials are only moderately thaw-sensitive but solifluction may be very active throughout the area.

6.4.1(c) Elliot Creek to Oscar Creek

The lower parts of Elliot Creek (Mile 659.3), Oscar Creek (Mile 649.0) and the string of ox-bow like lakes and channels, extending from Elliot Creek to past Norman Wells, represent abandoned channels of the Mackenzie River. On either side of these channels, flood plain and meander plain sediments and river terraces can be found. The flood plain sediments are dominantly back water silts and clays which are generally poorly drained. The meander plain sediments are generally silts with some sands and are better drained. The terrace deposits are usually silts to sands with only infrequent gravel. The terraces are well drained and the route proposed for the highway traverses these deposits as it crosses this region.

6.4.2 Borrow Resources

Table VI-4 lists the potential borrow areas, between Mile 668 and Mile 649, which were explored and found to have suitable borrow material and conditions for pit development.

The bedrock ridge, across the route at Paige Mountain, is an excellent source of good quality rock borrow (BA63, BA65A, and BA65B). The limestone outcropping on the ridge is hard and durable, and will require drilling and blasting. However, it is recommended that rock of the gypsum-rich Bear Rock Formation be avoided as it is extremely solution susceptible. The material from this ridge may be required for road grade construction and as a wearing surface for many miles of the proposed highway.

There are several beach ridge features, containing small quantities of sand , which would serve as secondary borrow between the ridge and Mile 662. This material, however, is irregularly frozen, and where it is frozen it generally will be too wet to excavate and

TABLE VI-4

SUGGESTED BORROW AREAS - MILE 668 TO MILE 649

BORROW AREA NO.	ACCESS MILE	MATERIAL DESCRIPTION	POTENTIAL USAGE	ANTICIPATED PIT CONDITIONS	ESTIMATED QUANTITY	COMMENTS
63	668.5	Limestone	Surface Course	Good	Very Great	Bedrock
65A & 65B	668 to 667.6	Limestone	Surface Course	Good	Very Great	Bedrock
74	662.7	Sand and Gravel	Subgrade	May be Wet	180,000 yds.	
77	659.7	Gravel and Sand	Subgrade	May be Frozen and Wet	300,000 yds. Minimum	
79 & 80	658.5 to 657.2	Sand and Gravel	Subgrade to Surface Course	Friable Frozen Low M/C	Very Great	
86A & 86B	653.4 to 652.8	Sand	Subgrade	Unfrozen Locally Wet	Very Great	River Terrace
91A & 91B	649.1	Gravel and Sand	Subgrade to Surface Course	Irregular-ly Frozen and Wet	260,000 yds.	

place. Potential borrow area 74 (BA74), at Mile 662.7, is likely to be the best area for development. This material should be used only as part of a composite road grade and suitable erosion protection should be provided.

About 0.5 miles southwest of the proposed highway center line, at Mile 664.5, are two very large sand dunes. Although these were not drilled because of access difficulties, it is expected that they will be similar to the dunes found south of this area. These two large dunes are believed to contain unfrozen, to frozen, friable, poorly graded fine sand, which should provide an additional source of secondary borrow material.

East of the route, from Elliot Creek to Mile 657.1, is a kettle marked moraine or outwash complex. Gravel and sand borrow material of good quality was located in BA 79, and 80, at Miles 657.8 and 657.6, respectively; and similar material probably exists nearer to Elliot Creek. This material should provide a large quantity of good to excellent granular material which may be suitable for road grade construction and surfacing.

Potential borrow areas 86A and 86B, at Mile 653.4, revealed unfrozen, generally dry sand, which should be acceptable as borrow. On the north bank of Oscar Creek, an irregular source of gravel, which is probably part of an outwash terrace, was located. The material is partially frozen and locally is wet. In spite of this, it is believed that good material may be located here with further exploration.

6.4.3 Drainage Considerations

The bedrock ridge, between Paige Mountain and Carcajou Ridge, forms the southern boundary of the high ice content, poorly drained, lacustrine sediments of the lower Hanna River Basin. South of Paige Mountain the materials are generally fluvial silts and clays, in the north, to silts and sands in the south. The highway is routed along the front of the Norman Range and the surface runoff from the mountains must all cross the highway. Concentrated runoff can be expected in all gullies and spring flooding can be expected in all lowlying basins.

Drainage considerations at Elliot Creek and at the unnamed creek, at Mile 657.2, have been discussed in more detail in Volumes XX and XXIII, respectively. Drainage conditions in the wide flood plain of Oscar Creek are discussed in Volume XXI of this report.

Between Mile 666.8 and Mile 662, exists several potentially hazardous drainage channels which cross the highway. It is believed that these drainage channels may be potential mud slide channels. In addition, from Mile 665.3 to Mile 661.6, the route parallels a beach ridge which concentrates runoff and potential mud slides, into narrow channels which cross the highway. Several of these channels, when drilled, were unfrozen and very soft, indicating the possible existence of mud flows or concentrated solifluction. These areas will require extra attention in the designing and placing of culverts. In particular, the channels at B.H. 664-S-1 (Mile 664.8), and between 664-S-2 and 664-C-4 (Mile 664.3), are potentially the most hazardous.

The poorest drainage conditions in this section of highway occur between Mile 661.6 and Elliot Creek crossing. Soil conditions reflect the poor drainage in the form of an above normal accumulation of peats and organic soils, and medium to high ice contents. A major culvert will be required at the location of B.H. 660-S-1 (Mile 660.1).

South of Elliot Creek (Mile 659.3), the proposed route is again aligned across the local drainage. From Elliot Creek to the large unnamed creek, at Mile 659.2, the route crosses several small channels, each of which will require a culvert. From Mile 657 to 651.2, the route follows a series of well drained terraces with very few problem areas, with respect to drainage, anticipated.

Two large creeks are crossed in the approach to the Oscar Creek crossing. The first creek is at Mile 651.1 and the second at Mile 650.1. Neither of these should present major drainage problems, but major culverts or culvert systems will be needed to handle concentrated spring runoff.

6.4.4 Construction Problems and Recommendations

The major problem in this section of the highway is the need to maintain good cross-slope drainage and to be aware of the locations of potential mud flows. The regions from Mile 667 to 662, and Mile 659 to 657, are the most hazardous in this respect. To re-route the highway through this area does not presently seem practicable unless major mud flows are actually observed. In addition, there does not appear to be a much better alignment than the one presently proposed.

A re-location of the route through the region from Mile 662 to Elliot Creek (Mile 659.3) has also been considered. Subgrade conditions are expected to be adverse through this region and extra care and an increased thickness of road grade material will be warranted. Several alternate re-alignments have been studied, but none seem to offer conditions significantly better than those expected on the present alignment.

A third problem area, for which a re-route has been considered, is the Paige Mountain Spur crossing. Sink holes in the gypsum-rich Bear Rock Formation may possibly underlie the highway on both sides of the anticlinal ridge. The 8 feet of peat, penetrated by B.H. 667-C-2, tends to indicate that the center line is presently located over a filled sink. A major relocation away from the unit does not seem presently feasible because of confinement imposed by the mountains to the east and a deeply incised major stream to the west. Minor changes in the route over the ridge should be considered only after more detailed field inspections. Any potential alternate routes should be drilled and cored in detail before any road bed material is placed. A field inspection of this nature may or may not reveal potential alternates.

Although much of the sands in the borrow pits recommended in subsection 6.4.2, have generally low moisture contents, some wet sands were encountered in the potential borrow areas. The dry sands should allow relative ease of excavation and placement within road grades. However, frozen high moisture content sands will be difficult to excavate and when placed under winter conditions will result in road grade fill which will possess high concentrations of voids. Thawing in the spring will result in considerable settlement of the fills and soft, probably untrafficable , grades until drainage and densification of the fills occur.

Use of sand borrow, whether dry or wet, should only be considered for the construction of composite or fully capped road grades to minimize erosion of road grades by wind and running water.

6.5 Section V (Mile 648 to Mile 632)

6.5.1 Surficial Materials and Conditions

6.5.1(a) Oscar Creek Meander Plain

The highway crosses Oscar Creek where it follows an abandoned channel of the Mackenzie River. The lowlying area of the channel swings to the south, away from Oscar Creek, and into a system of long narrow lakes which extend, from Elliot Creek, south past Norman Wells. The intersection of the highway and this lowlying region, including Oscar Creek, extends from Mile 649 to Mile 648. The materials and conditions at the Oscar Creek crossing are discussed in detail in Volume XXI, of this report.

The lowlying area between the river crossing and the top of the bank, at Mile 648.1, presents several problems. The materials encountered here, are generally fluvial sands and gravels which were found to be unfrozen to frozen with a very low ice content. However, they are wet where unfrozen. Old meander channels, which are evident on the air photos, may contain several feet of poorly consolidated organic material.

High ice content silt and clay were found on the slope, from the level of Oscar Creek, up to the old flood plain level. The highway crosses this slope at a gentle grade and stability should not be a problem if adequate drainage is maintained and severe degradation of the permafrost does not occur.

6.5.1(b) Mile 648 to Mile 639

Mile 648 marks the approximate mileage of the top of the west bank of an abandoned Mackenzie River channel. Between Mile 648 and Mile 639, the highway crosses over a relatively flat sand plain formed from either flood plain or meander plain deposits. At Mile 639 the highway crosses from the west to the east side of the abandoned channel.

The sand in this area was generally frozen. However, some unfrozen sand was encountered. The excess ice content seldom exceeded 10 percent and was usually under 5 percent. Where peat deposits have formed in shallow depressions and stream channels, ice contents are generally higher. Based on noted ice contents, it is believed that severe thaw-settlement of road grades,founded on the sand, will not likely occur if the permafrost degrades.

The sand is generally fine and silty, with the exception of sand dune material which is dominantly poorly graded and free of silts. The sand dune material was relatively dry and unfrozen. However, the fluvial sand has moisture contents averaging 20 percent to 25 percent,and often moisture contents are higher in the upper 3 feet of organic soil or peat cover. The high average moisture content could result in soft subgrade conditions if the sand is disturbed.

Silt and clay, which generally have an ice content of 30-40 percent, in the upper five to eight feet, occur south of Mile 640.4 to Mile 638.8. There is a potential for significant thaw-settlement damage to the highway if the permafrost degrades in this area. Significant thicknesses of road bed fills will be required to protect the permafrost in this area.

6.5.1(c) Mile 639 to Bosworth Creek

The level of the land rises from the Creek crossing at Mile 639, and the soils are significantly different than those to the west of the creek. Silt tills and clay tills, underlying unconsolidated slopewash deposits, are the major soil types in this section of highway route. South of approximately Mile 644.5, shallow limestone bedrock underlies the tills. The tills were generally found to be seasonally frozen to 8 to 12 feet at the time of the field investigation. However, unfrozen tills were encountered in 60 percent of the boreholes. Near the surface, in peat or organic soils, the estimated excess ice content is approximately 25 percent, but below the seasonally frozen soils the ice content averages 0-10 percent. Thaw settlements, in excess of 2 feet are not anticipated even in the higher ice content materials.

6.5.2 Borrow Resources

Table VI-5 presents a summary of the suggested borrow areas. These areas were found to have materials and conditions which are believed favorable to development. All potential borrow areas explored are summarized in Appendix A.

Only four natural deposits, considered worthy of being exploited for borrow materials, were found in this section of highway. These are the sand dunes at Borrow Areas 94 and 95, in Mile 646, the kame complex at Borrow Area 107, at Mile 637, the bedrock ridges paralleling the highway from Mile 637 to 634, and the esker complex, just north of Bosworth Creek, at Mile 632. All other deposits, with the exception of those in BA113, proved to be of unsuitable material or had unsuitable moisture or ice contents. BA113, which appears to be in a river terrace, appears to be small and of uncertain quality.

TABLE VI-5

SUGGESTED BORROW AREAS - MILE 648 TO MILE 632

BORROW AREA NO.	ACCESS MILE	MATERIAL DESCRIPTION	POTENTIAL USAGE	ANTICIPATED PIT CONDITIONS	ESTIMATED QUANTITY	COMMENTS
94 & 95	646.7 to 646.2	Sand	Subgrade	Dry	1,300,000 yds.	Sand Dunes
107	637.0	Sand and Gravel	Subgrade to Surface Course	Low Ice Content Irregular M/C	850,000	Kame
109A to 111	637.0 to 634	Shale and Limestone	Surface Course	Well Drained	Very Great	Bedrock
112	633.3 to 632.7	Limestone	Surface Course	Well Drained	100,000 yds. Minimum	Bedrock
113	633.0	Sand and Gravel	Subgrade	Partly Frozen and Wet	200,000 yds.	
114	632.0	Sand and Gravel	Subgrade to Surface Course	Possibly Wet	800,000 yds.	Eskers

The sand dune material will probably only be suitable for subgrade construction as it will be very susceptible to wind and water erosion. Capping of this material with more competent material will be required. The sand and gravel deposits of BA107 (Mile 637.0) and BA 114 (Mile 632.0) both have irregular frozen patches with higher moisture contents. However, these deposits are believed to be the best quality material. If the frozen material is placed in the summer, it should thaw and drain quickly enough to provide a suitable subgrade. However, the high moisture contents may lead to undesireably soft borrow pit conditions. If the frozen material is placed in the winter, road grades will be subject to considerable settlements when they thaw.

The shallow bedrock, which was explored in Borrow Areas 109A, 109B, 110A, 111 and 112, should provide good durable limestone borrow and good, dry, probably ripable shale for road grade construction. The bedrock deposits will be the driest and therefore the most easy to place. The shales of this area appear more durable than those in the Fort Good Hope area and should provide an adequate road grade.

6.5.3 Drainage Considerations

Extensive spring flooding in the lowlying Oscar Creek Meander Plain is expected. A more detailed discussion of the Oscar Creek Crossing is presented in Volume XXI of this report. A major creek, which is expected to require a large culvert, crosses the highway alignment at borehole 648-S-1, Mile 648.2. When this hole was drilled in early March, 1973, open water was encountered. This indicates that a problem could develop with icing of culverts and flooding of the highway during the winter.

The drainage pattern from Mile 647 to 640 is poorly developed. Several main streams exist which will require culverts. In particular, the streams at Mile 646.9, Mile 645.9, Mile 644.3, Mile 642.6, Mile 641.8, and Mile 641.2 will require special attention. The last of these is a naturally in-filled pond which is crossed by the highway. The in-filling is organic rich and highly compressible. Extensive fill will be required in constructing a road grade across this pond.

A discussion of the drainage at the creek, at Mile 639, is presented in more detail in Volume XXIII of this report. On the east side of this creek the highway alignment is across the natural slope and drainage paths. There are many creek crossings in this section, however, most of them are intermittent and none are very large. The crossings at Mile 676.9 and at Mile 634.0 are the largest of these.

6.5.4 Construction Problems and Recommendations

This section of highway overlies generally low ice content material. Thaw settlements are not expected to be a major problem in this area. However, wherever organic soils and peat occur greater settlements can be expected. Soft natural subgrade conditions could develop in the high moisture content sands north of Mile 640, if they are disturbed or if they are exposed in a cut. Use of these materials for road grade construction will result in soft, possibly untrafficable grades, until drainage occurs.

In the lowlying area from Oscar Creek to Mile 648, there is a possible hazard of extensive flooding in the spring. It is believed that it will be necessary to use high quality borrow to provide adequate protection from scour of fine grained soils used in the road bed. Icing up of culverts and flooding are potential problems at the creek at Mile 648.2.

South of Mile 639, supplying adequate drainage will be a major problem because the highway cuts across most natural drainage channels. There is a potential for spring flooding and mud flows in this area and washing out of parts of the road may occur.

VII. RE-ROUTINGS

7.1 General

Route location has generally been carefully selected and a great deal of attention has been paid to expected subgrade soils and permafrost conditions, slopes, cross-slopes, borrow sources and drainage requirements. Therefore, only a few suggestions for re-routings are considered necessary.

In several places undesireable conditions cannot be avoided. These areas are expected to be long-term maintenance areas and warrant extra design and construction care. These areas in order of occurrence southward are:

1) Jackfish Creek, at Mile 721
2) The north-west end of Snafu Lake, at Mile 703
3) The west end of Chick Lake, at Mile 693 to 691
4) Brokenoff Gap, at Mile 676 to 673
5) The lower Hanna River Basin, at Mile 673 to 668 and
6) From Mile 662 to Elliot Creek, at Mile 659.

A minor re-route may be warranted at Mile 708 where the proposed alignment crosses a shallow peat filled depression. A by pass, approximately 500 feet to the west, would avoid much of this poor material. The major disadvantage of this by pass is that three additional curves would be required. This re-route is not considered necessary unless very adverse conditions are found at the time of construction.

A similar re-route may be warranted, but only after more detailed field inspections, for the Paige Mountain Spur at Mile 667. The possibility exists that the present route may be located over an in-filled, possibly unstable sink hole.

7.2 Mile 714.6 to Mile 717.9

A re-routing of the highway between Mile 714.6 and 717.9, to obtain better natural subgrade conditions, appears to be justified. Approximately 15 percent of the originally planned route, within these mileages, is over very poor subgrade material consisting of several feet of peat and organic soil with high ice contents.

The proposed alternate, shown on Figure VII-1, is longer by approximately 4300 feet or 25 percent (measured PI to PI). However, on this re-route the road would encounter only about 2.6 percent of the very poor subgrade materials. Also, the route would be shifted approximately 3300 feet to the east and up the ridge on which Potential Borrow Area 9 was drilled. On the crest of the ridge, along which the highway is routed, it is suspected that shallow bedrock may be found which would be an additional source of borrow in an area lacking good borrow resources.

7.3 Beavertail Gap to Chick Lake (Mile 691.9 to Mile 694.7)

A proposed alternate route for the highway, between Mile 691.9 at the west end of Chick Lake and Mile 694.7 in Beavertail Gap, is shown on Figure VII-2. In Beavertail Gap, at Mile 694.2, the route as it has been located orginally, crosses a ridge in a deep cut with a curve in the cut. By shifting the route approximately 400 feet to

PROPOSED RE-ROUTE BETWEEN MILE 717.9 AND MILE 714.6

FIGURE 7-1

PROPOSED RE-ROUTE BETWEEN MILE 694.7 AND MILE 691.9

FIGURE 7-2

the west, this cut is avoided. It is considered desireable to avoid the cut because the maximum depth drilled on the ridge is nine feet, and although only gravel was encountered, it is believed that bedrock may be exposed in the cut. In addition, if the route is shifted away from the ridge it will permit full development of the ridge for borrow materials.

It is believed that the relocated route would encounter lower ice content tills than the original route from Mile 694.2 to Mile 693, and the higher elevation of the relocated section should provide better drainage conditions. From Mile 693 to Mile 692, the conditions on both the original and proposed route are expected to be similar.

The original route is shorter than the proposed re-route, but only by approximately 100 feet, measured from PI to PI.

VIII. SUMMARY

8.1 General

A geotechnical evaluation of the proposed Mackenzie Highway, between Norman Wells and Fort Good Hope, has been reported herein. In total, 1287 boreholes were drilled, of which 484 were in search of borrow materials. The remainder of the holes were drilled on center line to evaluate natural subgrade conditions.

Disturbed samples were obtained from all boreholes and a limited number of undisturbed samples were collected. All soil samples were field classified and a visual estimate was made of the quantity and form of ground ice conditions. In the field laboratory, moisture contents were determined for every sample and a visual classification was undertaken. Selected samples were returned to Edmonton for basic classification tests.

Detailed river crossing reports for this section of highway have been prepared and are presented as Volumes XV to XXIII of this report. All data and borehole logs have been presented as Volumes II to IV, and V to XIV, respectively, of this report.

It is believed that most of the objectives of this study have been accomplished. Sufficient data on the natural subgrade materials and conditions has been obtained. Potential sources of acceptable borrow materials have been identified; and a terrain analysis of a two mile corridor, centered on the highway, has been provided.

An attempt to analyse those factors affecting the origin and distribution of permafrost has been made with only limited success. The amount of data available for this kind of analysis has permitted a statistically significant examination of permafrost occurrence on a scale not generally possible. However, the lack of conclusive results emphasizes the complex nature of the ground thermal regime. It is believed that equations presented can be employed in the evaluation of permafrost behavior during and after highway construction.

8.2 Problem Areas

The route location has been carefully selected and generally the subgrade conditions encountered are believed to be the best available. Only two route re-locations are considered to be warranted. However, several other problem areas are listed as follows:

1) Jackfish Creek, at Mile 721
2) The northwest end of Snafu Lake, at Mile 703
3) The west end of Chick Lake, at Mile 693 to 691
4) Brokenoff Gap, at Mile 676 to 673
5) The lower Hanna River Basin, at Mile 673 to 668 and
6) From Mile 662 to Elliot Creek, at Mile 659.

The lower Hanna River Basin is the most thaw sensitive section of the route. The materials have very high ice contents over several miles of road and local drainage is very poor. Very careful design and construction practices must be employed to protect the permafrost or extensive road settlements will occur.

This area , because of its sensitive nature, should be instrumented to provide long term information on the road performance and to obtain valuable information on northern road construction. The advantage of using this area as a test section is that it has uniform conditions extending over several miles of road. Trial designs and construction methods could be compared with respect to their thermal effects on the natural subgrade and their long range performance. It is recommended, therefore, that a network of temperature measuring installations be placed in this area, on center line, off center line and far enough away from the proposed highway to be free from the effects of construction. These should be installed as soon as possible, to provide pre-construction through post-construction data.

The other major subgrade problem that is foreseen, concerns the possibility of constructing road grade over thin-roofed caverns and sink holes, existing in gypsum-rich limestone bedrock of the Bear Rock Formation. It is believed that this formation may out-crop in Beavertail Gap, at Mile 695, Brokenoff Gap, at Mile 675, and the ridge between Paige Mountain and Carcajou Ridge, at Mile 667.5. A detailed geological and/or geophysical study of these areas may be warranted. If such a study is undertaken, the foundation bedrock at the major river crossings should also be investigated.

8.3 Borrow Pit Development

Tables VI-1 to VI-5 list and evaluate those potential borrow areas, for each section of the highway, which have been proved by drilling and are believed to be most suitable for development. These potential borrow areas are expected to provide most of the material required for the highway. Several other unproven areas are discussed in the main body of this report and all areas explored for borrow are summarized and evaluated in Appendix A. Test pits will probably be required at several of the suggested pit locations to more accurately evaluate permafrost conditions, ground moisture conditions, and material quality and quantity.

In Section III of the highway, south of Brokenoff Gap, it is quite likely that the talus slopes along the west side of Brokenoff Mountain, between Mile 673 and 671, will be developed for borrow resources. The cliff faces above the talus slopes should be inspected, before any development, for signs of instability. Tension cracks or deeply weathered zones may have weakened the rock face to the extent where slides could possibly be initiated by the effects of blasting or heavy equipment working below.

Respectfully Submitted,

EBA ENGINEERING CONSULTANTS LTD.

N.R. MacLeod

PROFESSIONAL ENGINEER
L. A. BALANKO
ALBERTA

L. A. Balanko, P. Eng.

L. B. Smith, P. Eng.

Reviewed By:

G. R. Gilchrist, P. Eng.

D. W. Hayley, P. Eng.

E. W. Brooker, P. Eng.

LIST OF REFERENCES

1. Guide to a Field Description of Permafrost: Tech. Memo. No. 79. Associate Committe on Soil and Snow Mechanics, N.R.C. of Canada.

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

3. Means, R.E. and Parcher J.V., 1963, Physical Properties of Soils, Charles E. Merril Books Inc., Columbus, Ohio.

4. Hume G.S. 1953: The Lower Mackenzie River Area, Northwest Territories and Yukon, Geol. Surv. Can. Mem. 273.

5. Canada, 1969: Map 4 - 1969 Geology, Fort Good Hope, District of Mackenzie. 1:250,000, Geol. Surv. Can.

TABLE A-1

SUMMARY OF POTENTIAL BORROW AREA CONDITIONS

SECTION I (MILE 724.7 TO MILE 706)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
1	721.5	721-B-1 to 7	4.5 miles	Sand and Gravel	Good to Excellent	Dry, U/F[2] NF[3]	0	18+[4]	Very great	Excellent but long haul distance
2A	718.7	718-B-10 to 15 718-B-28,29	660	Limestone Bedrock	Good to Excellent to moderate Ice content silt and clay	Overlain by Low to	5	8+	160,000+	Good Material Fair Conditions
2B	718.7	718-B-31 to 38	2000	Limestone Bedrock	Good to Excellent	Will require blasting	8	10+	100,000+	Good Material Fair Conditions
3	718.3	718-B-16 to 19	400	Limestone Bedrock	Good	Limited Quantity Moderate to High Ice Content	7	11+	Uncertain	Fair to Poor Conditions Unsuitable Development
4	717.9	718-B-20 to 27	375	Clay-Silt Till	Poor	Irregularly Frozen				Unsuitable
5	717.8	717-B-1 to 5	320	Silt Till and Clay Till	Unsuitable					Unsuitable
6A	717.4	718-B-1,2,3,7 717-B-6 to 10	400	Clay-Silt Till	Unsuitable	Low Ice Content				Unsuitable
6B	717.6	718-B-9	550	Clay Till to Silt Till	Unsuitable	Randomly Frozen				Unsuitable
6C	717.0	718-B-4,5, 6,8	400	Clay Till	Unsuitable	U/F to Partially Frozen				Unsuitable
7	717.0	717-B11 to 18	560	Clay Till Silt-Clay Till	Unsuitable	Frozen				Unsuitable
8	716.7	716-B-1 to 7	550	Silt Till & Clay Till	Unsuitable	Frozen to U/F				Unsuitable
9	715.3	715-B-1 to 5	800	Clay Till	Poor to Unsuitable	U/F, low M/C[5]				Poor
10A	714.6	715-B-6 to 11	550	Clay Till to Silt Till	U/F to Frozen Low M/C					Unsuitable
10B	714.6	715-B-12 to	2000	Shale and Limestone	Good	Deep	10	9	Uncertain	Poor to Fair
11	714.4	714-B-1 to 5	550	Limestone or Dolomite	Good	Deep	13	6+	77,800+	Very deep Otherwise Good
12	714.0	714-B-6 to 11	550	Clay Till & Silt Till	Poor to Unsuitable	U/F, Low M/C				Poor
13	713.8	714-B-12 to 17	550	Clay Till & Silt Till	Poor	U/F				Poor
14	713.7	713-B-7 to 12	550	Clay Till	Poor	U/F				Poor
15	713.5	713-B-13 to 17	550	Shale	Fair to Good	U/F Material Dry, Dense, Hard	12	6+	Limited	Good Material but Quantity Small

1. Numbers are inclusive.
2. U/F - unfrozen
3. NF - Non visible ice- friable
4. 18+ - in excess of 18 feet of material. Actual amount unknown.
5. M/C - Moisture Content

TABLE A-1 (Cont'd)
SUMMARY OF POTENTIAL BORROW AREA CONDITIONS
SECTION I (Mile 724.7 To Mile 706)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
16A	713.2	713-B-1,2,3 713-B-18 to 22	300	Clay and Clay Till	Poor	U/F, Low M/C				Poor
16B	713.2	713-B-4,5,6	300	Clay Till and Silt Till	Poor	U/F, Low M/C				Poor
17	712.7	712-B-1 to 5	550	Shale	Fair	Dry, Dense, Hard	9	9+	160,000+	Fair to Good but Deep
18	711.0	711-B-1 to 6	400	Shale	Fair	High Ice Contents & Possible Flooding				Unsuitable Conditions
19	710.0	710-B-1 to 9	400	Shale	Fair	Underlies Randomly Frozen Material	9	9+	256,700	Fair to Poor
20	709.0	709-B-1 to 6	550	Shale	Fair	Underlies High to Medium Ice Content Materials	9.5	12+	1,302,200+	Poor to Unsuitable Conditions For Fair Material
21	707.9	708-B-1-2b 708-B-2 to 5b	800	Silt Till & Clay Till	Unsuitable	Frozen				Unsuitable
22	706.7	706-B-1 to 10	450	Shale	Fair	U/F, Very Soft	7	11+	220,000	Fair to Poor

TABLE A-2

SUMMARY OF POTENTIAL BORROW AREA CONDITIONS

SECTION II (MILE 705 TO MILE 689)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
23	705.8	705-B-1 to 10	550	Silt Till to Clay Till	Unsuitable	Randomly Frozen				Unsuitable
24	704.1	704-B-1 to 7	550	Silt Till & Clay Till	Unsuitable	Randomly Frozen				Unsuitable
25	703.9	703-B-1 to 5	2000	Well Graded Gravel	Good to Excellent	May be Irregular	1	10.5	307,200	Good Material but Quantity Uncertain
26	703.4	703-B-6 to 11	450	Limestone Bedrock	Good	Irregular Distribution and Depth	9	9+[1]	80,000+	Good but Deep
27	702.3	702-B-1 to 5	450	Silt Till	Poor to Unsuitable	U/F[2], Low M/C[3]				Unsuitable
28	701.0	701-B-1 to 5	1500	Silt Till to Sand Till	Poor to Unsuitable	Partially Frozen	0	18+	173,300+	Poor to Unsuitable
29	699.6	699-B-1,2	600	Silt Till to Clay Till	Poor to Unsuitsble	High Ice Content				Unsuitable
30	698.3	698-B-1 to 4	400	Silt & Clay Till	Poor to Unsuitable	High Ice Content				Unsuitable
31	697.1	697-B-1 to 6	400	Sand and Gravel	Good to Excellent	Frozen - High M/C	0	11	101,850	Unsuitable Too Wet
32	697.0	696-B-1 to 7	400	Gravel and Sand	Good to Excellent	Irregular Deposit Partially Frozen	2	10	240,700	Unsuitable
33	696.1	696-B-8 to 11	400	Gravelly Silt Till	Poor to Fair	Low M/C, U/F	1	12	657,800	Poor to Fair
34	696.0	695-B-1 to 5	400	Gravel and Sand	Good	U/F	0	18+	466,700	Good
35	695.5	695-B-6 to 12	900	Gravel Till over Limestone	Fair to Good Good	Lightly Frozen Fair To Poor Variable Surface	1 11	11 7+	513,300 326,700+	Fair to Poor Good but Deep
36	694.9	694-B-1,2	200	Sand and Gravel	Excellent	Partly Frozen	0	7+	67,410+	Poor to Unsuitable
37	694.1	694-B-3,4	500	Gravel	Excellent	U/F	0	8+	183,700+	Excellent
38	693.2	693-B-1 to 5	500	Silt and Clay	Unsuitable	High Ice Content				Unsuitable
39	691.3	691-B-1,2	450	Clay	Unsuitable	High Ice Content				Unsuitable
40	690.9	690-B-1,2,3	450	Clay	Unsuitable	High Ice Content				Unsuitable
41	590.0	689-B-1 to 4	900	Shale	Fair to Good	High Ice Content Material Above	7	9+	133,300+	Fair to Good

1. 9+ - In excess of 9 feet of material. Actual amount unknown.
2. U/F Unfrozen
3. M/C Moisture Content

TABLE A-3

SUMMARY OF POTENTIAL BORROW AREA CONDITIONS

SECTION III (MILE 688 TO MILE 669)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
42	688.8	688-B-1,2,3	1300	Clay	Unsuitable					Unsuitable
43	687.9	688-B-4 to 7	3000	Shale	Fair	Underlies High Ice Content Clay	10	6+ [1]	252,000+	Poor to Good
44	686.5	687-B-1	7900	Limestone	Good to Excellent	Will Require Blasting	3	6+	6,670/100+ [2]	Good
45	686.5	686-B-1,2	2200	Limestone	Good to Excellent	Will Require Blasting	4	5.5+	7,450/100+	Good
46	685.0	685-B-1 to 4	2400	Limestone	Good to Excellent	Will Require Blasting	7	5+	5,000/100+	Good
47	680.5	680-B-1,2	1300	Limestone	Good to Excellent	Will Require Blasting	2	10+	25,930/100'	Good
48	679.9	679-B-1,2,3	700	Limestone	Good to Excellent	Will Require Blasting	2	7+	8,040/100'	Good
49	679.4	679-B-4 to 7	1400	Limestone	Good to Excellent	Will Require Blasting	3	16+	1,801,500	Good
50	677.6	677-B-1 to 6	900	Shale	Fair to Good	Too Deep	14			Unsuitable
51	676.7	676-B-1,2	1250	Shale	Fair	Too Deep	16			Unsuitable
52	676.6	676-B-3	500	Shale	Poor to Fair	Underlies Medium Ice Content Material	9	9+	Extent Uncertain	Poor to Fair Area For Development
53	676.5	676-B-4,5	750	Shale	Poor to Fair		7	11+	134,400+	Poor to Fair
54	675.8	675-B-1,2,3	500	None	Unsuitable					Unsuitable
56A	676.1	675-B-4,5	(3100) [3]	Silt and Clay	Unsuitable					Unsuitable
56B	676.0	675-B-9,10 11		Silt Till	Unsuitable	Frozen				Unsuitable
57	675.4	674-B-8,9	500	Sand	Fair to Good	High M/C Low Ice Content				Unsuitable
58A	675.0	674-B-1	2000	Silt	Poor to Unsuitable					Unsuitable
58B	675.0	674-B-2,3	5000	Gravel	Good to Excellent	NF [4], Low M/C	0	16	663,700	Good to Excellent
58C	675.0	674-B-4 to 7	5300	Silt	Poor to Unsuitable	Frozen				Unsuitable

1. 8+ In excess of eight feet of material. Actual extent unknown.
2. 6670/100'. On linear features such as bedrock ridge quantities are given per 100 feet of ridge length.
3. (3100) Distance or access to borrow area by a route more direct than that used by Drilling Party.
4. Non visible ice-- friable.

TABLE A-3 (cont'd)

SUMMARY OF POTENTIAL BORROW AREA CONDITIONS

SECTION III (MILE 688 TO MILE 669)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
59	(674.0)	673-B-1,2,3	4900	Sand	Fair to Good	Frozen Irregular High Ice Content	6.5	11.5+	Limited	Unsuitable
60A	(672.7)	672-B-1 (test pit)	2500	Talus Slopes	Good to Excellent	Good	0	Unknown	Very Great	Excellent
60B	(672.0)	671-B-1 (test pit)	5400	Talus Slopes	Good to Excellent	Good	0	Unknown	Very Great	Excellent
61	(671.4)	670-B-1,2,3	1300	Gravel	Good	NF	2	18+	733,000+	Good to Excellent
62A		670-B-4,3,6	4000	Sand & Gravel	Good to Excellent	NF to Low Ice Content	1.0	6	333,000	Good in Top 6' Only
62B	670.2	670-B-7 to 10	5500	None	Unsuitable					Unsuitable

TABLE A-4
SUMMARY OF POTENTIAL BORROW AREA CONDITIONS
SECTION IV (MILE 668 TO MILE 649)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
63	(668.5)	668-B-1 & 5	6500 (3800)[1]	Limestone	Excellent	Will Require Blasting	40	9+[2]	11,670/100+[3]	Good to Excellent
64	667.7	668-B-2,3,4	4700 (200)	Sand, Silt Till	Poor	Possibly High Ice Content & Wet	0	7	28,500	Poor to Unsuitable
65A	668.0	668-B-6 667-B-1	1400	Limestone	Excellent	Will Require Blasting	1	21+	62,220/100+	Excellent
65B	667.6	667-B-2	1500	Limestone	Excellent	Will Require Blasting	3	9+	236,700	Excellent
66	668.0	667-B-7,8	1700 (500)	Silt Till	Poor to Unsuitable	Frozen				Unsuitable
67	667.3	667-B-5,6	5400 (800)	Silt	Poor to Unsuitable					Unsuitable
68	667.3	667-B-11	1700	Sand	Fair	U/F[4]	0	15+	116,700	Poor to Unsuitable
69	667.3	667-B-9,10	1800	Silt, Clay Till	Poor to Unsuitable	Unfrozen				Unsuitable
70A	667.3	667-B-3	1000	Sand	Fair to Poor	Frozen May be Wet	0	6	93,300	Poor to Unsuitable
70B	667.3	667-B-4	2650	Sand	Fair	U/F May be Wet	5.5	8.5	157,400	Poor to Unsuitable
71	666.5 (666.6)	666-B-1 to 4	7200 (4550)	Shale	Poor		2.5		Limited Area Uncertain	Poor
72	664.7	664-B-1,2	1100	Silt and Silt Till	Unsuitable	Frozen				Unsuitable
73A	663.7	664-B-3,4,5	4200	Sand and Sand Silt Till	Poor to Unsuitable	Frozen	3	15+	166,700	Unsuitable To Poor
73B	663.7	664-B-6,7,8	2100	Silt Till	Unsuitable	Frozen High Ice Content				Unsuitable
74	662.7	662-B-1 to 4	1000	Sand and Gravel	Good	U/F to NF[5] May be Wet	1	9.5	183,000	Fair to Good
75	661.7	662-B-5 to 9	1500	Sand and Gravel	Good	Irregularly Frozen to NF Some Clay and Silt Beds, Wet				Poor to Unsuitable
76	661.7	661-B-1,2	2800	Clay and Silt	Unsuitable					Unsuitable
77	659.7	661-B-3 to 6	400	Gravel and Sand	Fair to Good	U/F to NF Wet	0	15+	300,000+	Fair to Poor
78	659.0	659-B-1,2	1300 (550)	Gravel	Good	Thin		Thin Deposit		Unsuitable
79	657.7	658-B-1,2,3	600	Sand and Gravel	Good to Excellent	Frozen Low Ice Content to NF	10	34+	Area Uncertain Large	Good

1. (3800) Distance to borrow area by a route more direct than that used by the Drilling Part which is given above.
2. 9+ Thickness of material in excess of 9 feet. Actual quantity unknown.
3. 11,670/100' On linear features, such as bedrock ridge, quantities are given per 100 feet of ridge length.
4. U/F - unfrozen
5. NF - Non visible ice - friable

TABLE A-4 (cont'd)
SUMMARY OF POTENTIAL BORROW AREA CONDITIONS
SECTION IV (MILE 668 TO MILE 649)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
80	657.5	658-B-4,5	500	Sand and Gravel	Good to Excellent	NF	0	10	Uncertain Large	Good to Excellent
81	657.1	657-B-1,2	400	Clay	Unsuitable	Too Wet & Frozen				Unsuitable
82	655.8	656-B-1,2	500	Sand	Fair	Wet Below 10' Frozen Low Ice Content to NF	0	18+	626,700	Unsuitable
83	654.8	655-B-2,3,4	400	Silt	Unsuitable					Unsuitable
84	654.0	655-B-1 654-B-1	3000 or 3300	Sand	Fair	NF to U/F May be Wet Where Frozen	0	15+	366,700+	Poor to Unsuitable
85	654.0	654-B-2,3	400	Clay	Unsuitable	Too Wet & Frozen				Unsuitable
86A	653.4	654-B-4 653-B-1,2,3	400	Sand	Fair to Good	U/F Locally Wet	0	15+	511,100	Fair to Poor
86B	653.4	653-B-4	4200	Sand	Fair to Good	U/F	0	15+	822,200	Fair to Good
87	652.1	652-B-2,3,4	1100	Sand	Fair	Irregularly Frozen, Wet	0	7	Uncertain	Poor to Unsuitable
88	651.5	652-B-1	1650	Sand	Poor to Fair	U/F, Too Wet	0	6	Uncertain	Poor to Unsuitable
89	650.8	651-B-1,2,3	400	Silt & Clay	Unsuitable	Frozen, Wet				Unsuitable
90	650	651-B-4,5	900	Silt & Clay	Unsuitable	Frozen & Wet				Unsuitable
91A	649.1	650-B-1	1100	Gravel	Good	U/F to NF May be Wet	4	11+	260,700	Good
91B	649.1	649-B-2 to 5	1100	Gravel and Sand	Good	Irregular Distribution	2	16+	Area Uncertain	Fair

TABLE A-5

SUMMARY OF POTENTIAL BORROW AREA CONDITIONS

SECTION V (MILE 648 TO MILE 632)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
92	648.3	648-B-1,2	1400	Sand	Poor to Fair	Poor, Wet	0	18+[1]	106,700	Poor to Unsuitable
93	647.8	647-B-1,2	1200	Sand	Poor to Unsuitable	Frozen Low Ice Content, Wet	0.5	8.5	Uncertain & Limited	Unsuitable
94	646.7	646-B-1,2,3	400	Sand	Good to Fair	U/F[2], Dry	0.5	32	1,090,400	Fair to Good
95	646.2	646-B-1A,2A 3A, 4A	500	Sand	Fair to Good	U/F, Dry	0	32	237,000	Fair to Good
96	645.4	645-B-1,2	500	Sand	Poor to Unsuitable	Frozen				Unsuitable
97	645.3	645-B-3	550	Sand	Poor to Unsuitable	Frozen			Small	Unsuitable
98B	645.2	645-B-4,5,6	4300	Clay, Silt Till	Unsuitable	Frozen				Unsuitable
98A	645.2	645-B-7	2300	Sand	Poor to Fair	Frozen, High M/C[3]	0.5	18+	426,700	Poor to Unsuitable
99	643.1	643-B-1,2,3	2500	Silt and Sand	Poor	Frozen High Ice Content				Unsuitable
100	641.6	641-B-1,2	3300	Sand	Fair to Poor	Frozen, High M/C				Poor to Unsuitable
101	641.3	641-B-3,4,5	400	Sand	Poor to Unsuitable	Frozen, High M/C				Unsuitable
102A	640.8	641-B-7,8,9	1000	Sand	Poor to Unsuitable	Frozen, High M/C				Unsuitable
102B	640.8	641-B-6	400	Sand	Poor to Fair	Wet	3.0	15+	327,800	Poor
103	640.0	640-B-1	2400	Sand	Poor	Frozen				Unsuitable
104	639.5	640-B-2,3	800	Clay	Unsuitable	High Ice Content				Unsuitable
105	638.1	638-B-1	1250	Sand, Silt Till	Poor to	U/F Low M/C				Unsuitable
106	637.8	638-B-6 to 9	3600	Silt, Clay & Silt Till	Unsuitable	High Ice Content				Unsuitable
107	637.0	637-B-1,2,3	1100	Sand and Gravel	Good to Excellent	Irregular M/C Low Ice Content to NF[4]	0	32	853,300	Excellent but May be Wet
108	636.9 (637.8)	638-B-2 to 5	2800	Silt and Clay Silt Till	Poor to Unsuitable	Irregularly Frozen				Unsuitable
109A	635.9	636-B-1,2,3	2650	Shale	Poor to Fair	Soft probably Ripable	8	10+	38,520/100'[5]	Poor to Good
109B	635.9	636-B-4 to 7	1000	Shale	Poor to Fair	Soft probably Ripable	8	10+	26,700/100'	Poor to Good

1. 18+ In excess of 18 feet of material. Actual depth unknown.
2. U/F Unfrozen
3. M/C - Moisture Content
4. NF - Non visible Ice - friable
5. 38, 520/100' On linear feature, such as bedrock ridge, quantities are given per 100' of ridge length.

TABLE A-5 (cont'd)
SUMMARY OF POTENTIAL BORROW AREA CONDITIONS
SECTION V (MILE 648 TO MILE 632)

BORROW AREA NUMBER	ACCESS MILE	BOREHOLE NUMBERS	ACCESS DISTANCE (FEET)	BORROW MATERIAL	QUALITY OF MATERIAL	EXPECTED PIT CONDITIONS	AVERAGE STRIPPING DEPTH (FEET)	AVERAGE THICKNESS (FEET)	ESTIMATED QUANTITY (CU.YD.)	OVERALL BORROW EVALUATION
110A	635.0	635-B-1,2,3,4	1950	Shale	Poor to Fair	Soft Probably Ripable	7	11+	1,617,410	Poor
110B	635.0	635-B-5	800	Gravel	Good	Good	0	14	Uncertain 1,078,500	Good
111	634.5	635-B-6,7,8	1200	Limestone	Good	Good	1	8+	563,000+	Good
112	633.0	633-B-1,2	1100	Limestone	Good	Good	5	4	106,700+	Good
113	633.0	633-B-4,5	2300	Sand and Gravel	Excellent	Wet Where Frozen, Frozen to U/F to NF	7	18	200,000	Good Possibly Too Wet
114	632.0	632-B-1,2	400	Sand and Gravel	Fair to Good	Silty Layers Possibly Wet	0	18+	810,700+	Fair to Poor if Wet
115	632.0	632-B-3,4	2500	Silt-Clay Till	Unsuitable	High Ice Content				Unsuitable

APPENDIX B

THERMAL ANALYSIS

B.1.1 Introduction

The determination of the effect which construction activity has on permafrost, is the analysis of the ground thermal regime and, therefore, is fundamental to geotechnical design in the Arctic. In the past, the design of structures in the north has been hampered by the limited amount of precise field data with regard to the various factors affecting ground temperatures. It is, therefore, appropriate to analyse all available borehole data with a view to establishing the factors which affect the ground thermal regime.

Specifically, some data from this study was used to provide an indication as to the predictive accuracy of the Modified Berggren equation for predicting depth of freeze and depth of thaw. An attempt was also made to establish the factors affecting the incidence of permafrost along a portion of the proposed highway route.

B.1.2 Theory

B.1.2(a) Depth of Freeze

The modified Berggren equation * is useful for determining the depth of freeze where a thawed, homogeneous deposit is subjected to freezing temperatures at the surface. The expression for the depth of freeze is:

$$X_f = \lambda \sqrt{\frac{48K_f \; nI_f}{L_s}} \qquad \text{(B-1)}$$

* Reference: Adrich H.P. and Payter, H.M., 1953, ' Analytical Studies of Freezing and Thawing of Soils', U.S. Corps of Engineers, Technical Report No. 42.

where X_f denotes the depth of freeze (ft)

K_f denotes the thermal conductivity of the frozen material (BTU/FT HR $^\circ$ F)

L_s denotes the volumetric latent heat of freezing of the soil (BTU/CU FT)

I_f denotes the air freezing index ($^\circ$ F days)

n denotes the ratio between the ground surface freezing index and the air freezing index

and λ denotes a dimensionless correction coefficient

Equation (B-1) can be rewritten as:

$$X_f = J_f \sqrt{n I_f} \tag{B-2}$$

where

$$J_f = \lambda \sqrt{\frac{48 \quad K_f}{L_s}} \tag{B-3}$$

It can be shown that the freezing thermal constant, J_f, is primarily a function of soil type and soil water content. The freezing thermal constant, J_f, has been plotted as a function of soil water content in Figure B-1. One curve is obtained for coarse grained soils (sands and gravels) and a second curve is obtained for fine grained soils (silts and clays). The thermal constant for peat is indicated on the figure and is essentially constant for water contents greater than 50%.

B.1.2(b) Depth of Thaw

Equation (B-1) can also be used for the prediction of depth of thaw in a homogeneous frozen material subjected to thawing temperatures at the surface.

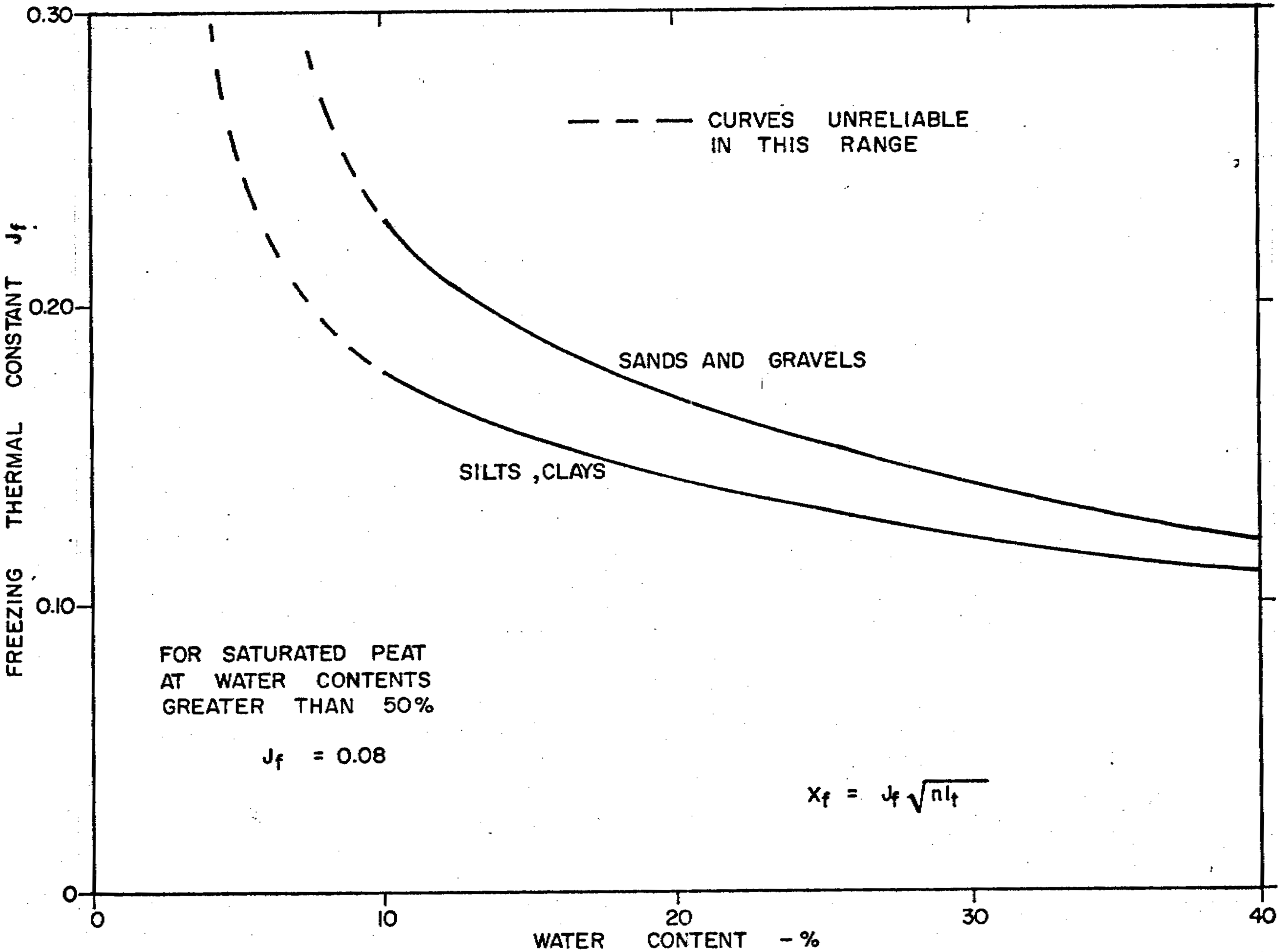


FIGURE B-1 WATER CONTENT VERSUS FREEZING THERMAL CONSTANT

In this instance the unfrozen thermal conductivity, K_t, is substituted for the frozen conductivity in equation (B-1). As before, the depth of thaw will be given by:

$$X_t = J_t \sqrt{n I_t} \qquad \text{(B-4)}$$

where

$$J_t = \lambda \sqrt{\frac{48\ K_t}{L_s}} \qquad \text{(B-5)}$$

and I_t denotes the air thawing index

Again, the thermal constant for thawing, J_t, is primarily a function of water content and soil type and has been plotted as such on Figure B-2.

B.1.3 Data Analysis

B.1.3(a) Depth of Freeze

The borehole data was initially analysed with a view to establishing the predictive accuracy of equations (B-2) and (B-4).

The depth of freeze, X_f, was determined from Figure B-1, provided the average, near surface, water content had been determined for the particular borehole. The air freezing index, I_f, was determined from the average monthly air temperature records for the period from September, 1972 to March, 1973. The air freezing index was found by integrating the air temperature - time curve, below 32^{o} F, up to the date at which the borehole was drilled. The typical configuration of the frozen ground which was found in many boreholes, and which permits

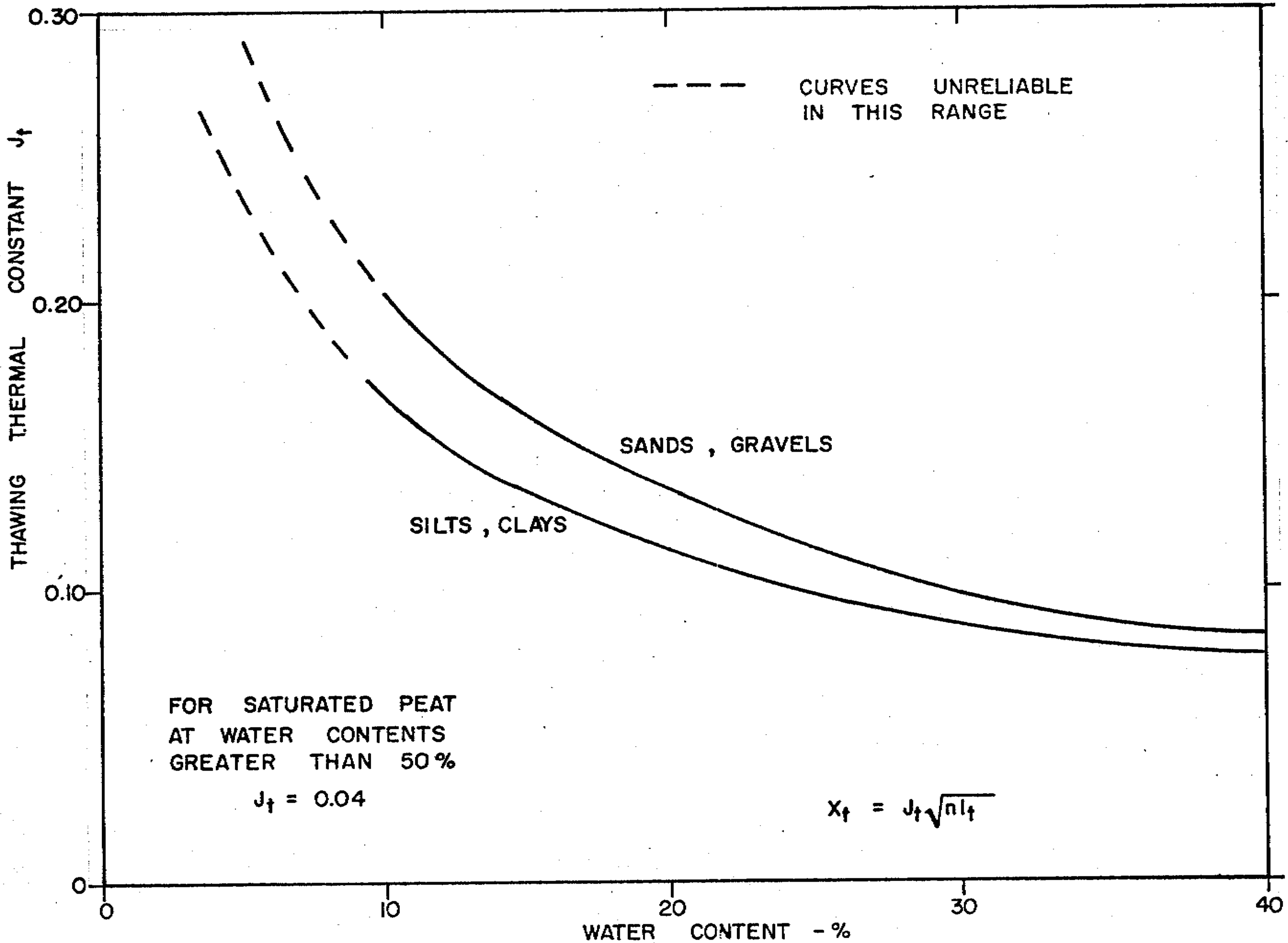


FIGURE B-2 WATER CONTENT VERSUS THAWING THERMAL CONSTANT thermal analysis for both the freezing and thawing situation is shown in Figure B-3. It is therefore possible to solve equation (B-2) for the ratio, n, between the ground surface freezing index and the air freezing index. The results are presented in Table B-1. Since the analysis is fairly time consuming, it was confined to that portion of the route between miles 632 and 648. The values for the ratio, n, for 38 boreholes, are seen to vary from 0.05 to 4.2.

Information published in the literature indicates that the value for the 'n' Factor should be between 0.3 and 1.0, depending on surface cover and snow depths. The mean value for the freezing 'n' Factor, for the 39 cases analysed, was calculated to be 0.5 with a standard deviation of 0.9. It is probable that the large range in values for the 'n' Factor is the result of the difficulty in obtaining representative water contents for the soils. The freezing 'n' Factor is also extremely sensitive to density and depth of snow cover and no account of these effects was taken in the analysis.

The analysis indicates that equation B-2 will provide an approximate estimate for the depth of freeze in homogeneous deposits. The equation is severely limited, however, in that such effects as surface cover, snow depth and variations in soil type and water content with depth, are not explicitly accounted for in the equation.

B.1.3(b) Depth of Thaw

As mentioned previously, the configuration of the frozen ground in many boreholes is as shown in Figure B-3. The soil which had thawed the previous summer had not yet completely refrozen in many instances. This is the result of an unusually mild Autumn and winter up until Christmas.

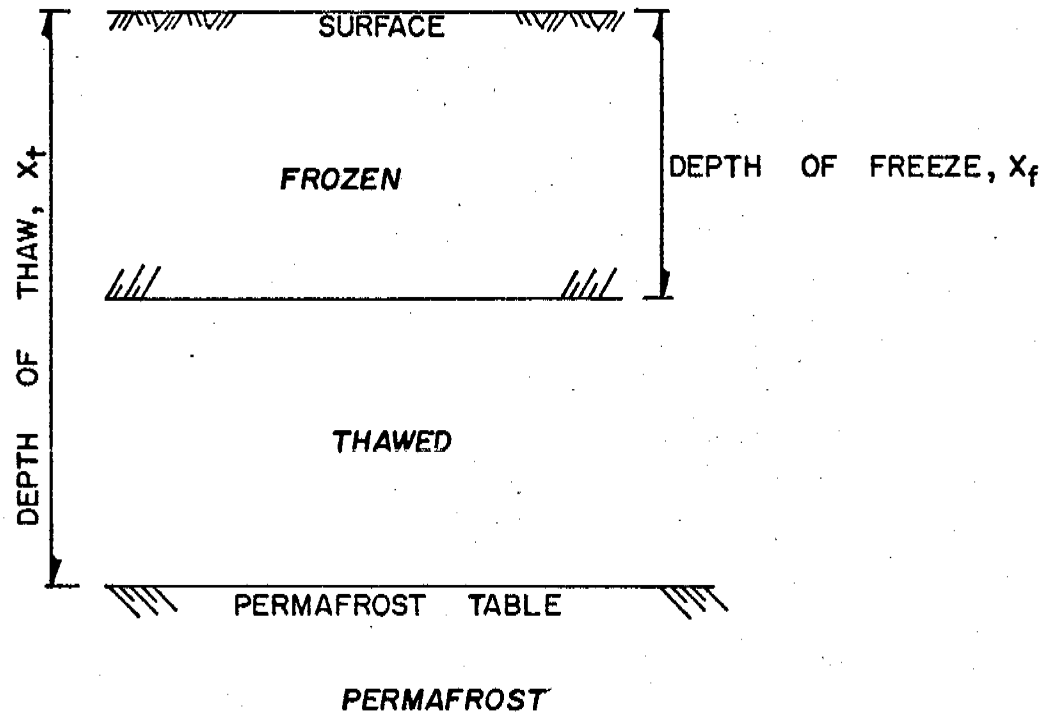


FIGURE B-3 CONFIGURATION OF FROZEN GROUND WHICH PERMITS THERMAL ANALYSIS

TABLE B-1

DATA ANALYSIS

DEPTH OF FREEZE

MILE	HOLE	LOGGED DEPTH OF FREEZE X=	DOMINANT SOIL	WATER CONTENT (%)	Jf (from Fig. 1)	$n = \frac{(X/J_f)^2}{I_f}$
640	C-5	3.0	SM	35	0.116	.12
	C-3	7.0	PT	166	0.080	1.36
639	C-4	5.0	CL	65	0.100	.45
638	C-6	5.0	CL	15	0.156	.18
638	C-3	2.0	PT	50	0.077	.12
637	C-7	3.5	CL	12	0.168	.08
	C-2	4.5	CL	35	0.116	.27
	C-1	14.0	ML	50	0.104	3.23
635	C-4	6.0	CL	20	0.142	.32
	C-1	6.0	GW	3	0.208	.15
634	C-1	5.0	CW	5	0.160	.17
633	C-4	12.0	PT	100	0.078	4.2
632	C-4	10.0	CL	28	0.124	1.16
641	C-3	5.0	PT	93	0.078	.73
644	C-1	4.0	SM	32	0.136	.15
644	C-2	5.0	SM	27	0.148	.20
644	C-4	5.0	SM	25	0.153	.19
644	C-5	6.0	SM	13	0.204	.15
644	C-6	7.0	SM	5	0.144	.42
645	C-3	3.0	SM	32	0.136	.09
645	C-4	3.0	SM	25	0.153	.07
645	C-5	5.0	SP	22	0.163	.17
646	C-7	11.0	SM	29	0.156	.89
647	C-1	3.0	CL	34	0.116	.12
647	C-4	5.0	ML	30	0.122	.30

TABLE B-1 (cont'd)

DATA ANALYSIS

DEPTH OF FREEZE

MILE	HOLE	LOGGED DEPTH OF FREEZE	DOMINANT SOIL	WATER CONTENT (%)	Jf (from Fig. 1)	$n = \frac{(X/J_f)^2}{I_f}$
648	C-3	4.0	SP	15	0.192	.08
648	C-5	4.0	SM	8	0.125	.18
648	C-6	4.0	SM	22	0.163	.11
632	S-1	10.0	CL	15	0.156	.73
632	S-6	5.0	GW	12	0.212	.10
632	S-11	13.0	ML	17	0.150	1.34
632	S-12	14.0	CL	40	0.110	2.89
633	S-1	3.0	ML	9	0.186	.05
634	S-2	3.0	CL	50	0.104	.15
635	S-1	3.0	ML	15	0.157	.07
643	S-1	4.0	ML	15	0.157	.12
645	S-1	3.0	SM	32	0.136	.09
648	S-1	3.0	PT	115	0.078	.26

AIR FREEZING INDEX I_t = 5616 °F Days

FREEZING 'n' FACTOR

AVERAGE VALUE - 0.5

STANDARD DEVIATION-0.9

Computer analyses have indicated that refreezing of the active layer takes place primarily from the surface downwards. That is, the distance to the permafrost table, on Figure B-3, will correspond closely to the maximum depth of thaw which occurred in the previous fall. Therefore it is possible to use equation (B-4) to solve for the thawing 'n' Factor which operated the previous summer.

The air thawing index was determined from the average monthly temperature records between March and November, 1972. The air thawing index was found by integrating the air temperature time curve, above 32 ^{o} F, over the entire summer period. The thawing thermal constant, J_t, was determined from Figure B-2, provided the near surface water content data was known. The ratio between the ground surface thawing index and the air thawing index was then calculated from equation (B-4). The results are presented in Table B-2.

The maximum depth of thaw which occurred during the previous summer could be deduced in 29 of the boreholes drilled during the winter. A review of the literature indicates values for the thawing 'n' Factor generally ranging from 0.8 to 1.5. An average value of 2.3 was calculated, from the data presented in Table B-2, with a standard deviation of 1.9. The average value is somewhat high and could be due to the fact that only boreholes, where the active layer had not completely thawed, may be included in the analysis. The depth of thaw for these boreholes would therefore be biased on the high side. There is considerable scatter to the data as indicated by the value for the standard deviation. This is partially due to the difficulty in obtaining representative near surface water contents. Furthermore, the type of surface cover and the effect of stratified deposits was not accounted for in the analysis.

TABLE B-2

DATA ANALYSIS

DEPTH OF THAW

MILE	HOLE	APPARENT DEPTH OF THAW (X)	DOMINANT SOIL IN TOP 3 FT	WATER CONTENT AT 2.5 (%)	J_t	$n = \frac{(X/J_t)^2}{I_t}$
643	C-1	14.0	SP	6	0.264	1.02
644	C-1	8.1	SP	32	0.094	2.69
644	C-2	10.0	SM	27	0.106	3.22
643	S-1	19.0	ML	15	0.132	7.49
645	S-1	12.0	SM	32	0.094	5.89
723	C-7	9.0	ML	50	0.076	5.07
722	C-5	9.0	ML	10	0.162	1.12
722	C-9	10.0	CL	12	0.148	1.65
721	C-2	11.0	SP	3	0.300	0.49
721	C-3	9.0	SM	3	0.300	0.33
721	C-6	2.0	SM	5	0.290	0.02
721	C-7	2.5	SM	27	0.106	0.20
720	C-5	8.5	ML	35	0.080	4.08
720	C-7	9.0	ML	13	0.142	1.45
720	C-8	9.5	CL	24	0.099	3.33
719	C-5	9.0	CI	75	0.076	5.07
692	C-5	7.0	ML	51	0.076	3.07
689	S-1	9.0	CL	30	0.086	3.96
677	C-4	6.0	ML	15	0.132	0.75
674	C-5	4.0	ML	13	0.142	0.29
665	C-6	7.0	SC	32	0.094	2.01
664	C-2	8.0	MC	27	0.092	2.74
664	C-3	7.0	MC	25	0.096	1.92
664	C-4	9.0	ML	20	0.110	2.42
664	C-5	4.0	SM	10	0.200	0.15
664	C-6	7.0	ML	20	0.110	1.47

TABLE B-2 (cont'd)

DATA ANALYSIS

DEPTH OF THAW

MILE	HOLE	APPARENT DEPTH OF THAW (X)	DOMINANT SOIL IN TOP 3 FT	WATER CONTENT AT 2.5 (%)	J_t	$n = \frac{(X/J_t)^2}{I_t}$
663	C-2	4.0	ML	15	0.132	0.33
663	C-7	12.0	ML	5	0.234	0.95
662	C-1	9.0	ML	23	0.102	2.82

AIR THAWING INDEX, I_t = 2765 ^{o} F DAYS

NUMBER OF SAMPLES: 29

THAWING 'n' FACTOR

AVERAGE VALUE -2.3

STANDARD DEVIATION -1.9

The results indicate that equation (B-4) will provide an approximate indication of depth of thaw. As with the equation for prediction of depth of freeze, equation (B-4) is severely limited in that it does not explicitly account for surface cover, and variations in soil type and soil water content with depth.

B.1.4 Factors Influencing the Occurrence of Permafrost

A review of the data from 332 boreholes, located between miles 689 to 724, revealed that permafrost was encountered in 38% of the boreholes. In regions which had been burned over by fire, in this section of the route, permafrost was encountered in 30% of the boreholes. This is significantly different from the overall incidence of permafrost, and is very close to 29% for the presence of permafrost on the old cut line. Therefore, it would seem that the incidence of permafrost under open or exposed areas will be approximately 30%; whereas, in tree covered areas it will be approximately 44%, which was found for boreholes drilled where the right of way had been freshly cleared. It is apparent that considerable degradation of the permafrost has occured below the C.N.T. line.

The effect which depth of peat has on the incidence of permafrost, between Miles 684 and 724, was also investigated. The average depth of peat in permafrost areas was found to be 1.0 feet with a standard deviation of 1.5 feet. In non permafrost areas the average peat depth was 0.9 feet with a standard deviation of 0.9 feet. It is apparent that the difference between the two average values is insignificant.

The incidence of permafrost as a function of soil type was also investigated and the results are presented in Table B-3 below:

TABLE B-3

INVESTIGATED INCIDENCE OF PERMAFROST
IN VARIOUS SOIL TYPES
MILES 684 TO 724

SOIL TYPE (EXTENDED UNIFIED CLASSIFICATION)	G	S	ML	CL	CI	CH	OH
PERCENT OF TOTAL NO. OF BOREHOLES, OF INDIVIDUAL SOIL TYPES, WHERE PERMAFROST WAS LOCATED	33%	65%	81%	25%	34%	25%	36%

From the above table, it appears that permafrost is more likely to be found in sands and silts as compared to other soil types. The analysis indicates that 65 and 81 percent of the boreholes that revealed sand and silt, respectively, also revealed permafrost conditions. It is difficult to establish the true significance of the results, however, since they are affected, to some extent by the number of samples of each soil type encountered. For example, gravel was encountered in only 3 of the boreholes in the area analysed, and hence, the value of 33 percent found for the incidence of permafrost in gravel could be significantly in error.

Theoretical studies have indicated that the presence of permafrost may be dominantly controlled by the ratio of the frozen to thawed thermal conductivities of the soil. Since the thermal conductivities are primarily a function of soil water content, it would seem probable that the incidence of permafrost could be directly related to near surface soil water content. Specifically, for given climatic conditions, surface cover and soil type; permafrost can exist where water contents are higher than some critical value, and will be non-existent where the near surface water contents are lower than this critical value.

Data from miles 700 to 724 was analysed in this respect. The average near surface water content of the boreholes, where permafrost was found, was calculated to be 28.1%, which is significantly greater than the average water content of 15.3%, measured where permafrost was not found. This trend supports the theoretical prediction. However, there was found to be considerable scatter to the data. For example, in many instances where near surface water contents greatly exceeded 15.3% permafrost was not found to exist. Conversely, permafrost was found in many boreholes where water contents were very much less than 28.1%. The scatter to the data reflects the effects which other factors such as surface cover, soil type, and micro-climate have on the existence of permafrost.

B.1.4 Conslusions

It is concluded that equations (B-2) and (B-4) will provide approximate estimates for depth of freeze and depth of thaw, respectively. The accuracy of prediction is severely limited in that such factors as surface cover, snow depth, varying water content with depth, and varying soil type with depth are not explicitly accounted for in the equations. Therefore, considerable judgement is required in their use.

Permafrost was encountered in 38% of the boreholes between miles 689 and 724. The incidence of permafrost bore no significant relationship to the peat thickness. However in burn areas and on the old C.N.T. cut line the incidence of permafrost was significantly lower, (30% and 29%, respectively) as compared to that found where the right of way had been freshly cleared (44%).

The incidence of permafrost in this section was found to be significantly greater in sands and silts as compared to other soil types. The true significance of this trend is difficult to ascertain since the results are affected to some extent by limited data for some soil types.

Theoretical studies indicate that under constant climatic conditions the presence of permafrost may be dominantly a function of near surface soil water content. Where water contents are high, permafrost will exist and it will be non existent where water contents are low. Analysis of data from boreholes between Miles 700 and 724 confirm this trend, however, there is considerable scatter to the data.

B.2 Settlement

B.2.1 General

Numerous methods for predicting the magnitude and rate of settlement, associated with thawing of soils under embankments, have been presented in the literature. Generally, the amount of settlement will be a function of the compressibility of the thawed soil and the depth to which it thaws. That is:

$$S = M X_t \tag{B-6}$$

where S denotes the settlement which occurs.

M denotes the volumetric compressibility of soil under specific stress conditions.

and X_t denotes the depth to which the soil thaws.

It is apparent that the accuracy to which settlements can be predicted is a function of both the accuracy to which the depth of thaw and the compressibility of the soil can be determined.

B.2.2 Depth of Thaw

A wide variety of analytical and numerical methods of determining the depth of thaw in frozen ground are available. The Modified Berggren Equation, Equation B-1, represents one of the simpler methods for predicting depth of thaw. The accuracy of the Modified Berggren Equation is limited, in that, it can not explicitly account for variations in surface cover, snow depth and inhomogeneous soil thermal properties. The equation can be used where approximate estimates for depth of thaw and settlement are required.

Where more accurate predictions are required, it is necessary to predict the depth of thaw accurately through the use of more sophisticated numerical solutions programmed for the computer. Both finite difference and finite element computer programs are privately available which incorporate explicitly into their solutions, the ground surface heat balance, as well as, variations in soil thermal properties with depth. The accuracy of thermal predictions is restricted only by the accuracy to which the various input parameters can be determined.

The depth of thaw and, hence, the amount of settlement which occurs below a gravel highway fill, will depend, in general, upon the following factors:

1) ground surface temperature variation with time

2) thermal properties and depth of gravel

3) thermal properties and depth of subgrade soils

4) the construction schedule

These factors can be accounted for in existing computer programs. The optimum depth of gravel, which will ensure the thaw plane does not penetrate into highly compressible sub-soils, can be determined as a function of these factors.

B.2.3 Soil Compressibility

The volumetric compressibility of the thawed soil can be determined through a variety of methods which employ standard soil mechanics procedures. One of the simpler techniques will be reviewed here.

The volumetric compressibility, M, can be defined as:

$$M = \frac{e_i - e_t}{1 + e_i}$$

where e_i denotes the initial (frozen) void ratio of the soil.

and e_t denotes the final (thawed) void ratio of the soil.

Provided the soil is completely saturated with water, the compressibility M, will be a function of the initial and final soil water contents.

$$M = \frac{1.1\,w_i - w_t}{1/G_s + 1.1\,w_i}$$

where w_i denotes the initial soil water content (by weight)

w_t denotes the final soil water content (by weight)

G_s denotes the specific gravity of the soil particles and generally has a value of about 2.65 for mineral soils.

and the factor 1.1 accounts for the volume change associated with the ice/water transformation

In practice the initial water content is determined from disturbed samples taken in the field. The final water content can be estimated for the soil, or if greater accuracy is required, it can be determined from the results of thaw-consolidation tests on undisturbed samples.

B.2.4 Conclusions

Accurate prediction of the settlement below a gravel highway is seen to depend on the accuracy to which the thaw depth and compressibility of the soil can be determined. Where approximate estimates of depth of thaw and settlement are required, it is possible to employ the simpler analytical thermal solution.

The accurate prediction of settlements requires the use of more sophisticated computer solutions which can account for the majority of factors which significantly affect depth of thaw.

The compressibility of the thawed soil can be determined using standard soil mechanics procedures. The magnitude of the settlements can therefore be determined under a wide variety of temperature and soil conditions. The design depth for the gravel forming the roadbed, which will ensure settlements are not excessive, can therefore be determined in a rational way.

APPENDIX C

RECOMMENDED FLOW VELOCITIES FOR DRAINAGE CHANNELS

C.1 INTRODUCTION

There are many complex factors affecting the design of the drainage systems to be employed in northern highway construction. Construction of the highway will invariably result in some re-arrangement of the natural drainage pattern, which will require channelling of runoff over new courses, or the concentration of more water in some old channels. In these areas the potential for scour erosion is, therefore, increased. Where new drainage courses are developed, the problem of thermal erosion also exists.

In the north, the greatest concentration of water comes during the spring runoff, when almost every lowlying area becomes a channel for runoff. These areas are protected from erosion by the natural vegetation and by the seasonal frost in the ground. As the seasonal frost degrades the potential for erosion increases. In a natural stream channel the permafrost is in thermal equilibrium with the stream, and the bed of the stream is in balance with the rate of flow. In a new channel this equilibrium will not have established. Consequently, it is desireable that a new channel and flow velocities of runoff be adjusted to prevent scouring of the exposed channel material, by the runoff.

C.2 Design Equations

Design equations exist for open channel flow, which relate flow velocity to the gradient of the channel and the cross-sectional configuration. The Manning formula[1*], is such an equation, which is commonly employed for open channel flow calculations. The formula is as follows:

---

* Reference
1. U.S. Department of the Interior, Bureau of Reclamation Canals and Related Structures. Design Standards No. 3.

$$V = (1.486/n)\ r^{2/3}\ s^{1/2} \tag{C-1}$$

where V = velocity of water, in feet per second,
s = slope of energy gradient, in feet per foot,
r = hydraulic radius (water area divided by wetted perimeter), and
n = coefficient of roughness (Mannings 'n').

For uniform channel sections covered with sand and gravel, the Mannings's 'n' may be determined by the Strickler equation, which is,

$$n = 0.0342\ d_{50}^{\ 1/8} \tag{C-2}$$

In the equation, d_{50} equals the size, in feet, for which 50 percent of bed material by weight is finer. Values of Manning's 'n' have also been determined for other soil types. Table C-1 presents recommended values of Mannings 'n' for the soils found along this section of the highway.

In unlined channels, the velocity should be such as to prevent cutting of the channel prism or deposition of silt. The maximum velocity allowable to prevent cutting, or the minimum allowable to prevent silt deposition, will depend upon soil characteristics, sediment load in the water, and other natural factors. However, general limits can be set down. The Kennedy formula for sediment laden water flowing in a bed of similar material is,

$$V_s = CD^{0.64} \tag{C-3}$$

TABLE C*-1

Manning's'n' for Natural Stream channels and Design Velocities for Non-Silt, Non-Scour Conditions for use on the Mackenzie Highway.

MATERIAL	MANNING n	RECOMMENDED DESIGN VELOCITIES (fps)
Stiff Clay	0.025	2.00
Colloidal Silt	0.025	1.00
Non-Colloidal Silt	0.020	2.00
Fine Sand	0.020	1.25
Coarse Sand	0.020	1.25
Silty Sand	0.020	1.50
Silt Till	0.025	2.00
Clay Till	0.025	2.00
Fine Gravel	0.020	2.00
Coarse Gravel	0.025	3.00
Well Graded Gravel	0.025	3.00
Cobbles	0.035	5.00
Broken Stone	0.035	3.00
Shale	0.025	4.00
Vegetal-lines	0.033	3.5 maximum

1. Table C-1 is based on channel depths being between 0.5 and 3.0 feet.
2. Aging of channels permits velocities to be increased by 30%.
3. For vegetal-lined channels increase the given values by up to 1.5 (fps) but not exceeding maximum values given.
4. Gradients should not exceed 4 percent for any drainage course.

---

* References:

1. Seelye, E.E., 1956: Foundations, Design and Practice

2. Handbook of Steel Drainage and Highway Construction Products. Highway Task Force, American Iron and Steel Institute.

Where V_s = velocity for nonsilt and nonscour,
D = depth of water in feet, and
C = coefficient for various soil conditions.

Typical values for the coefficient C are as follows:

MATERIAL	C
Fine, light, sandy soil	0.84
Coarser, light, sandy soil	0.92
Sandy, clayey silt	1.01
Coarse silt or hard soil debris	1.09

A suggested modification of the Kennedy formula for clear water is,

$$V_s = CD^{0.5} \tag{C-4}$$

Figure C-1 shows the relationship of V_s to D for various water depths, for clear water, and Figure C-2 shows this relationship for sediment laden water.

After a channel is in operation for an extended period, heavy concentrations of fine sediments in the flow may cause cementing (cohesion) of some fine sands in the bed. This often results in an increase in the nonscour velocities of up to 50 percent. The amount of increase suggested for the design of drainage systems for the Mackenzie Highway, should not exceed 30 percent.

Design velocities based on non-silt, non-scour conditions and Manning 'n' values are presented in Table C-1. These values can be employed in Equation C-1 to solve for design gradients whenever the materials and drainage channel size are known. However, it is recommended that design gradients should not exceed 4 percent.

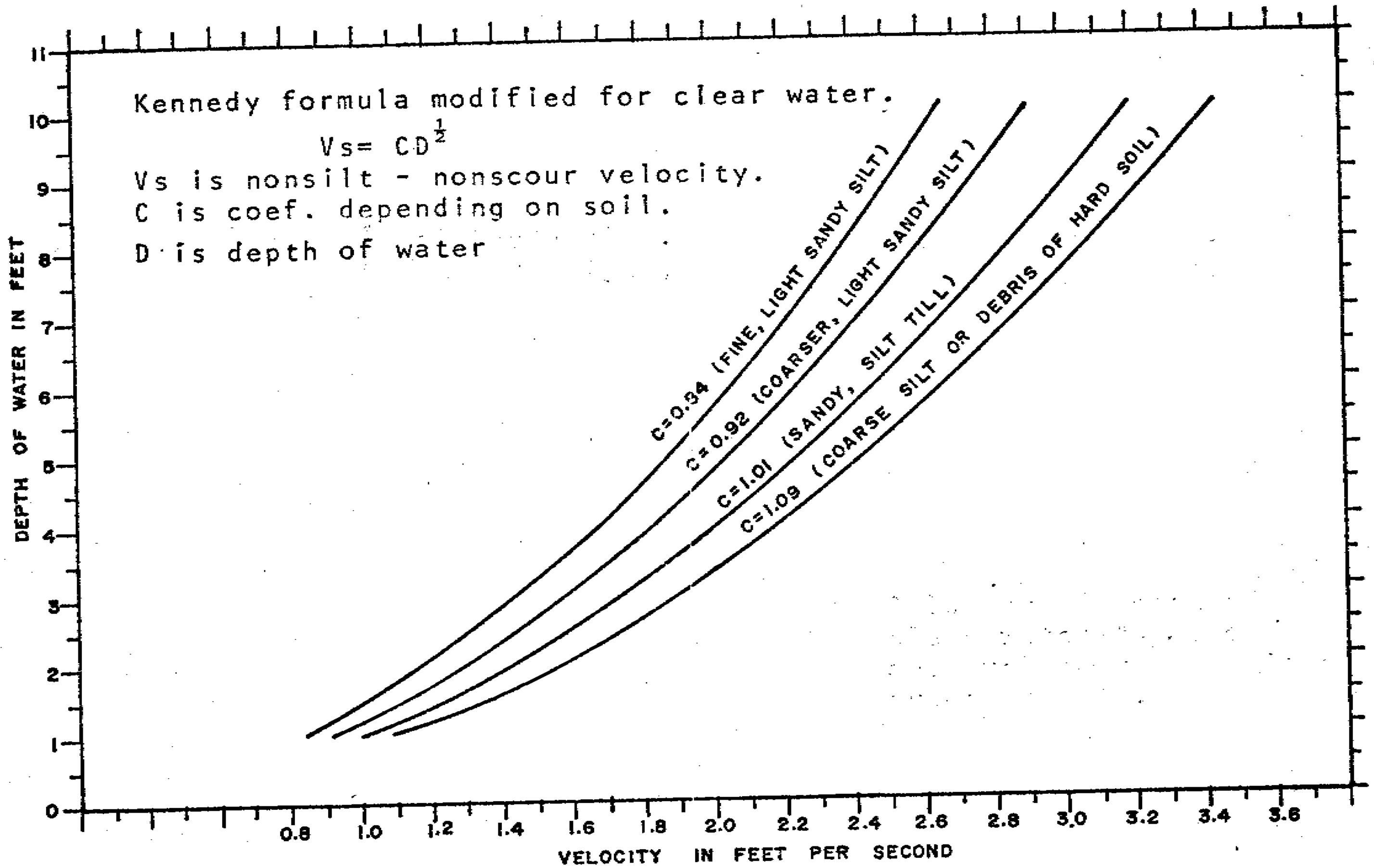


FIGURE C-1 RELATION OF DEPTH TO ALLOWABLE VELOCITY FOR CLEAR WATER

AFTER U.S. DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION

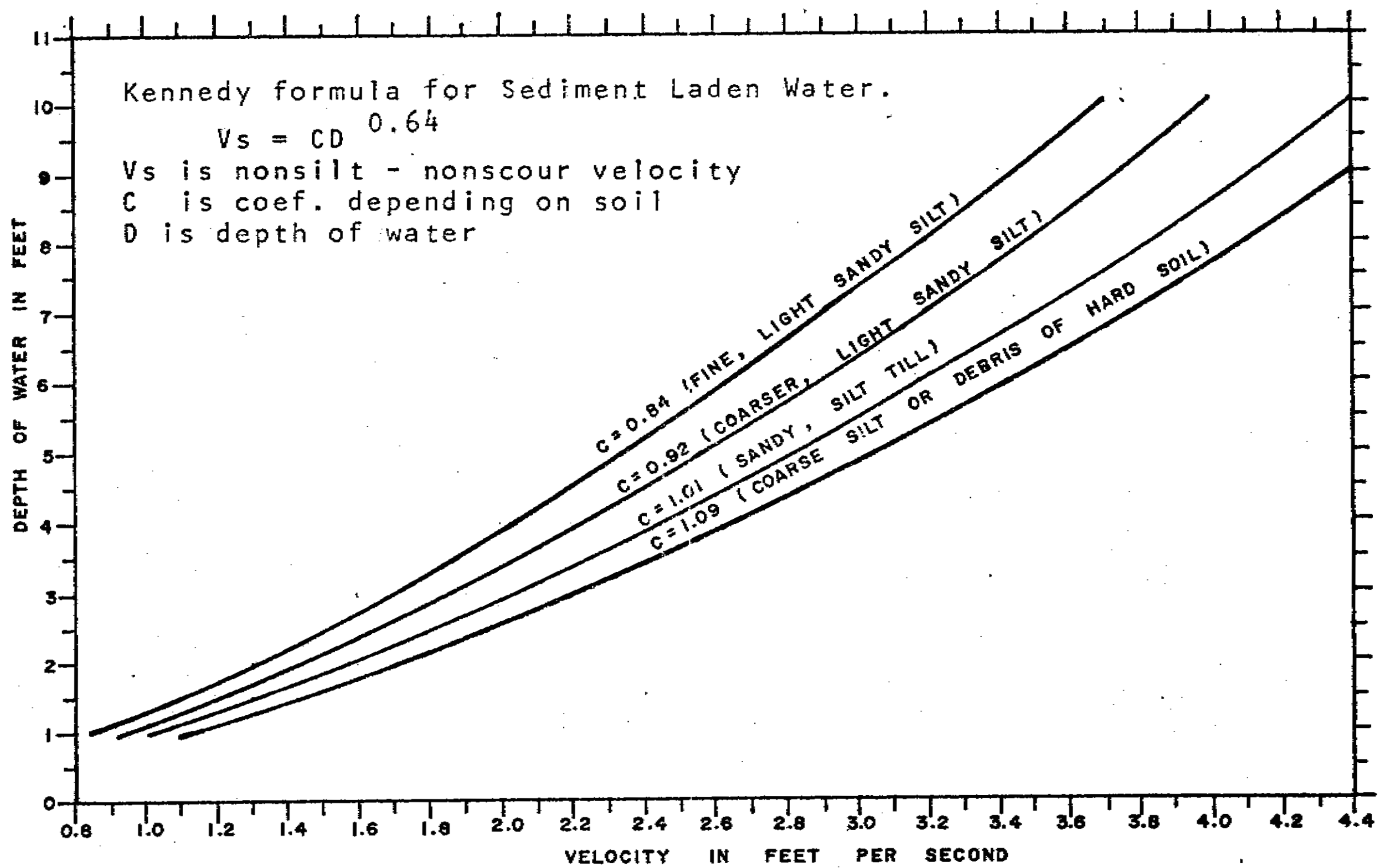


FIGURE C-2 RELATION OF DEPTH TO ALLOWABLE VELOCITY FOR SEDIMENT LADEN WATER