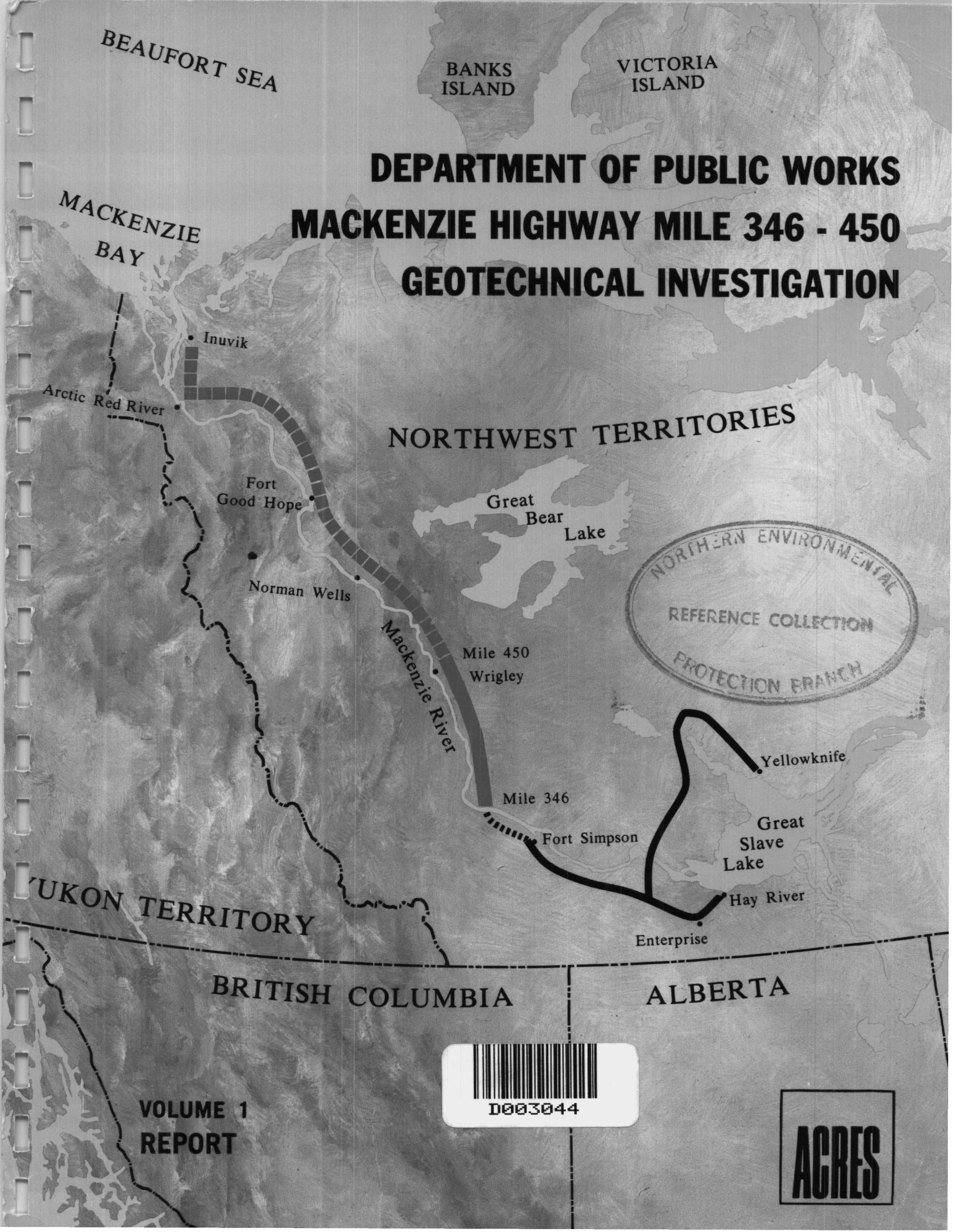
DEPARTMENT OF PUBLIC WORKS
MACKENZIE HIGHWAY MILE 346 - 450
GEOTECHNICAL INVESTIGATION



VOLUME 1
REPORT

ACRES

SYNOPSIS

This report contains the results of an engineering investigation conducted between Mile 346 and Mile 450 on the proposed Mackenzie Highway.

The engineering investigation required a review of all existing pertinent data, photogeologic analysis and mapping, field investigations including drilling and sampling along the centreline of the proposed highway and in borrow areas and the subsequent analysis, amalgamation and checking of all the data obtained.

Possible construction problems have been studied in the light of the results of the field investigations and the following conclusions have been reached:

(a) A satisfactory location has been selected for this section of the proposed highway in that the foundation soils are suitable for the subgrade and acceptable material is available for embankment construction.

(b) The highway can be constructed using normal design and construction techniques, though allowance must be made for the long cold winter season, the frost susceptible nature of some of the soils and the erodibility of the finer grained soils.

(c) Permafrost is neither extensive nor continuous and will not be a major problem in either road design or construction. Where permafrost does occur it is several feet below the ground surface and will not form any impediment to natural drainage.

(d) River crossings can be constructed using normal design and construction procedures.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PHOTOGRAPHS

DEPARTMENT OF PUBLIC WORKS

MACKENZIE HIGHWAY MILE 346 - 450

GEOTECHNICAL INVESTIGATION

PART I

REPORT ON FINDINGS

1 - INTRODUCTION

1.1 - Terms of Reference

The Regional Director of the Department of Public Works, (DPW) Mr. J. A. Brown authorized Acres Consulting Services Limited (Acres) by letter dated August 29, 1972 to undertake the consulting engineering investigation for the geotechnical work on Mile 346 to Mile 450 of the Mackenzie Highway. The work, under the direction of Mr. F. E. Kimball, Project Manager N.W.T. Highways, to be completed by April 30, 1973.

The Terms of Reference state that Acres will undertake a complete soil survey along the length of the designated section of the proposed Mackenzie Highway, Mile 346 to Mile 450, during the winter (November to April 1972/73).

Acres are responsible for three basic areas of investigation:

(a) to locate and identify potential problem areas along the proposed route and to recommend solutions in general terms, i.e. embankment stability, embankment settlement, backslope stability in cuts, etc.

(b) to locate, evaluate and recommend suitable embankment borrow areas along the route within a two mile corridor centered on the route.

(c) to conduct foundation investigations at pre-selected bridge sites and to make bridge foundation recommendations; and to investigate pre-selected approaches to major stream crossings, identify problem areas and recommend general solutions.

In order to comply with the Territorial Land Use Regulations all field operations had to be completed during the winter months commencing no earlier than November 1, 1972.

Work planning for this project started on August 31, 1972 and a preliminary field reconnaissance was conducted at the end of September 1972. Transportation of equipment to the field started on January 10, 1973 and the first drilling started on January 28th. All field operations were completed by March 12, 1973.

1.2 - Project Organization and Operation

Acres provided project management and technical services using experienced personnel from its own organization on a full or part time basis as required.

Other specialized work and support activities were carried out by experienced and capable organizations on a contract basis as shown below:

(a) Drilling equipment and expertise:
Kenting Big Indian Drilling, Calgary

(b) Bulldozing and clearing:
R. Reasons, Contracting, Fort Simpson

(c) Camp trailers:
Nodwell Brothers, General Contractors, Calgary

(d) Camp catering:
Fortier and Associates, Edmonton

(e) All-terrain tracked vehicles:
Flextrac - Nodwell, Calgary

(f) 4-wheel drive trucks:
Avis Rent-A-Car, Hay River

(g) Radio equipment:
Canadian Marconi Co., Calgary

(h) Communications and supply delivery:
Turner Expediting, Fort Simpson

(i) Laboratory testing:
Ripley, Klohn and Leonoff International, Ltd., Calgary

(j) Fuel and lubricants:
Imperial Oil Limited, Fort Simpson

Details of project operations are described in this report in Section 8, "Project Operations"

1.3 - Project Report

This report consists of nine volumes. Volume 1 is divided into two parts:

(a) Part I - Report on Findings

This includes background information to acquaint the reader with the physical characteristics of the area under study, the specific investigations carried out, the study results and recommendations for engineering design and construction.

(b) Part II - Procedures

This includes a description of the project operation, the drilling and sampling techniques and the photogeologic analysis and mapping techniques employed to plan the field investigation.

Volumes 2, 3, 4, and 5 contain Centreline Borehole Logs.

Volume 6 contains Borrow Area Borehole Logs and Borrow Area Testpit logs.

Volume 7 contains Laboratory Test Results on samples taken from the centreline boreholes.

Volume 8 contains Borrow Area Evaluation Sheets and Laboratory Test Results.

Volume 9 contains Photo Mosaics showing the photogeologic interpretations of the area.

2 - BACKGROUND INFORMATION

2.1 - General Description of Proposed Route

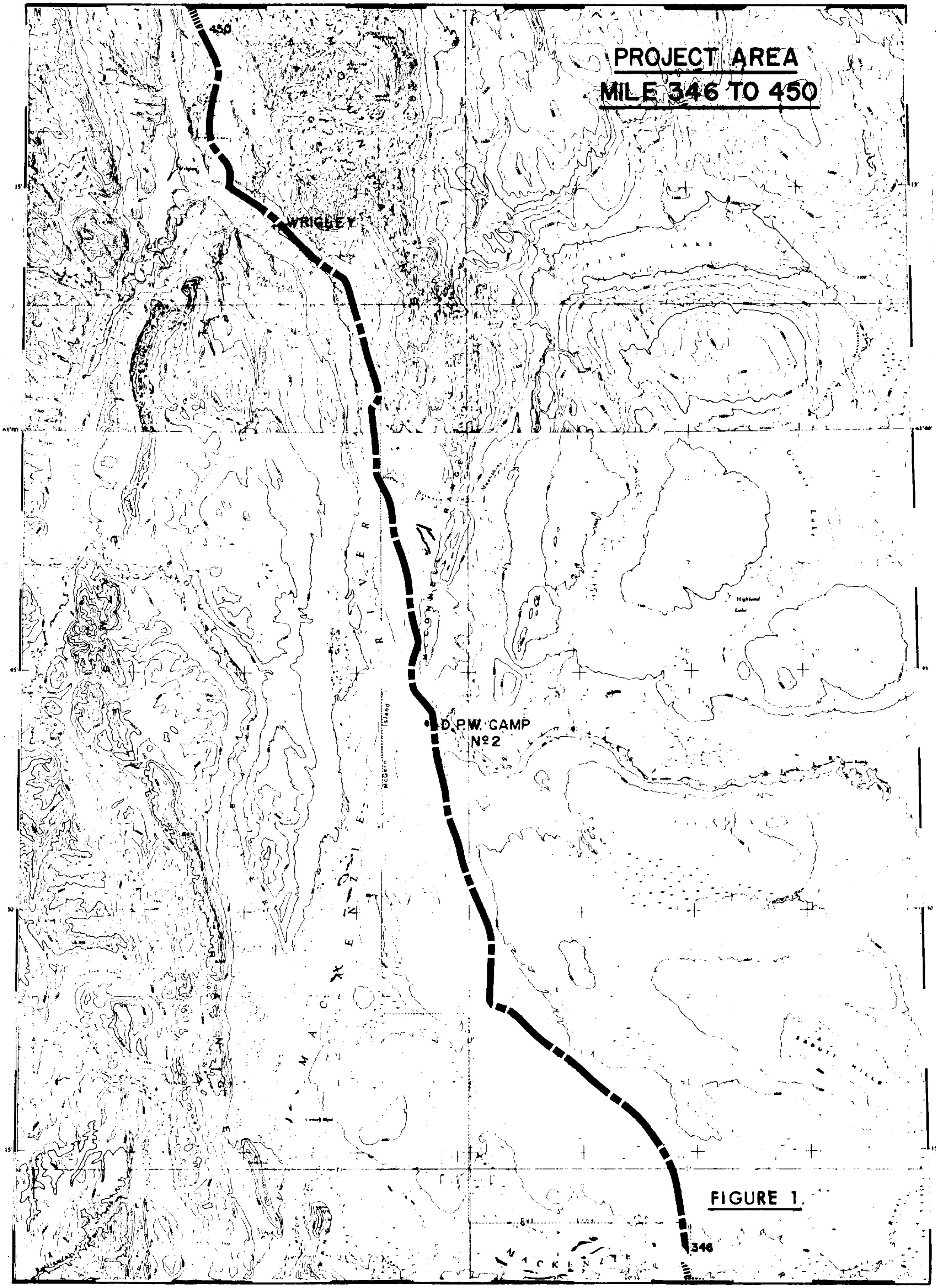
The section of the Mackenzie Highway covered by this report lies between Mile 346 and Mile 450. (Figure 1).

The southern end of this section is at Mile 347 on the north side of the Mackenzie River Crossing, some fifty miles downstream from Fort Simpson. The route trends northerly to Mile 355, northwesterly to Mile 372 and then northerly to the end of the section.

From Mile 347 the route cuts diagonally across Camsell Bend, rejoining the Mackenzie River near Mile 387. From this point northward, the route generally parallels the east bank of the Mackenzie River from 1 to 3 miles inland.

2.2 - Terrain Relief

The section covered by this report lies generally within two sub-divisions of the Interior Plains Physiographic Province. The boundary between these sub-divisions is located approximately at Willow Lake River. The southern section, containing Miles 346 to Mile 395, lies in the Great Slave Plain subdivision. In this subdivision, the major controlling features are the Mackenzie River which has incised its valley through the glacial and post-glaical deposits in the area; the Ebbutt Hills to the east, and the Mackenzie Mountains to the west. Local relief in this section varies from an elevation of approximately 280 feet at the Mackenzie River to a maximum of 695 feet at Mile 359. It then descends to an elevation of 270 feet at Willow Lake River, the


PROJECT AREA
MILE 346 TO 450
450
WRIGLEY
D.P.W. CAMP
Nº 2
MACKENZIE RIVER
346
FIGURE 1

descent being gradual to Mile 391 where it drops quite sharply to the lower terrace along the river bank. Local topography within this southern section is low, generally in the order of 50 feet, as can be seen on Photo (PH) 1. The most prominent physiographic features are low beach ridges resulting from water action during various stages of the post-glacial development of the Mackenzie River.

The northern section, containing Miles 395 to 450, lies in the Mackenzie Plain sub-division. Here the controlling features are the Mackenzie River, the Franklin Mountains to the east and the Mackenzie Mountains to the west. These mountains are part of the Cordilleran System.

Between Miles 395 and 400 there is a rapid climb to an elevation of approximately 950 feet, the maximum elevation in either section as shown on PH. 2. From this point northward, the terrain is generally rolling, and gently falling northward except for stream gorges where it drops rapidly to the base level of the Mackenzie River and as rapidly climbs again to the general level of the plain. The elevation at Wrigley is approximately 510 feet, rising again to 730 feet by Mile 450. Local topography varies considerably, generally being in the order of 150 feet.

Throughout the northern section, local relief is dominated by glacial and post-glacial type features resulting from the last glaciation (Wisconsin) which covered this area entirely.

2.3 - Regional Geology

The southern part of this section of the proposed highway is considered to be underlain by flat-lying sedimentary rocks of the Fort Simpson Formation, the lowest member of the Upper Devonian

Photo 1:
Typical terrain north from Mile 355.

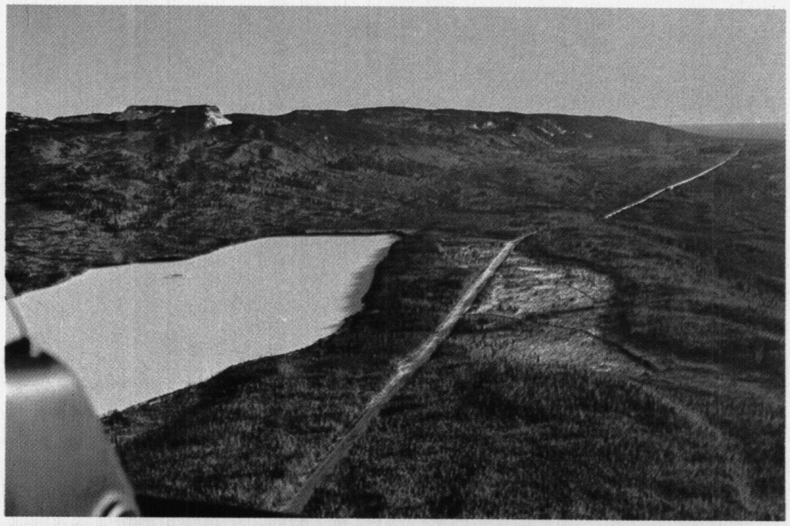


Photo 2:
South along route from Mile 404 - highest elevation in section. Highway joins CNT beyond the Franklin Mountains in background.

Group. This formation includes strata of shale, mudstone, limestone, siltstone and sandstone, but few outcroppings of this formation have been reported in this area.

Commencing at Mile 398, east of the proposed highway centreline, outcrops of limestone of the Nahanni Formation are found. This rock is described as light to dark grey limestone, occuring in thick and medium-bedded massive units. Where exposed in the McConnell Range it has a total thickness of approximately 300 feet. The basal part of this formation, which is mainly covered, may be gradational into the underlying Headless Formation which consists of dark grey thin-bedded and crypto-to fine-grained argillaceous limestone interbedded with calcareous shale. The Nahanni Formation, which is the youngest member of the Middle Devonian Group in this area, forms the main constituent of the McConnell Range. The faulting, which has produced the east-facing scarp of the McConnell Range and has been mapped to within two miles of Willow Lake River may be the source of the numerous hot springs which percolate from the western slopes of the range in this area, shown on PH. 3 and PH. 4. Except for one occurence found upslope at Mile 397, these hot springs are down-slope from the proposed highway location along the sidehill cuts, north of Willow Lake River. There is no indication that the upslope spring has sufficient flow to cause problems at the road location as it dies out before reaching the right-of-way.

The topography of the project area is controlled by glacial and post-glacial formations. The Wisconsin glaciation, which terminated in this area some 10,000 year ago, marked the terrain in the southern section in a northwesterly orientation, and in the northern section in a north northwesterly orientation. This ice sheet was responsible for carving and scouring the underlying bedrock and subsequently for the deposition of moraines and tills. Formations such as eskers, crevasse fillings, kames, and drumlin

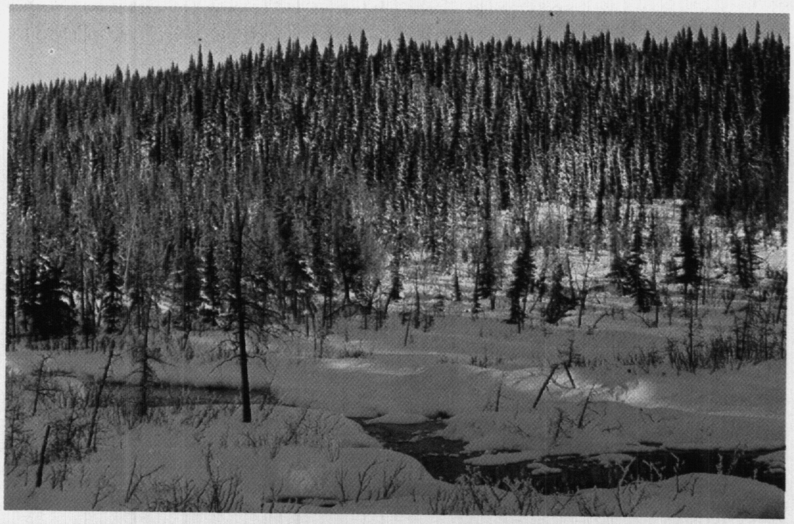


Photo 3:
Hot springs at Mile 396. Highway on hill in background.

Photo 4:
Hot springs lake below highway - Mile 396.

like formations are evident along the route, particularly in the section between Miles 399 and 438. In addition, post-glacial activity, which has resulted in the present Mackenzie River, was responsible for transporting, sorting and redepositing the materials in terraces and beaches.

Due to the geologic history of the area, the frequency, distribution and quality of deposits vary widely. There is also a wide variation in the grain sizes of the materials due in part to the variety of bedrock sources.

2.4 - Climate

With short summers, severe winter temperatures and moderate precipitation, the weather in the project area is typical of the "cool snow-forest climate" classification (Kappen) or of the "Boreal (subarctic) climate" classification (Trewartha). As a result of the low average temperature, evaporation is low and the ground is frozen for approximately 6 months of the year. Precipitation, though low, is sufficient for forest growth.

The weather through the project area is very uniform as can be seen from the records for Fort Simpson and Wrigley (Tables I and II).

The relation between air and ground temperatures and permafrost is discussed under Subsection 2.7 "Permafrost".

Normal frost penetration into the ground beneath undisturbed surfaces is expected to be 6 to 8 feet. However under heavily trafficked and/or exposed areas this could reach 10 feet depending on the subgrade materials.

The mean annual precipitation, approximately 13 inches, is quite uniformly distributed throughout the year. Only in spring, when melting snow creates a rapid increase in available water is excess water experienced. At this time of the year, the ground is still frozen so that much of this water is shed in the spring run-off.

These conditions generally will benefit road construction and maintenance for the following reasons:

(a) Erosion control will be similar to that required for highways in more temperate climates. However, allowance must be made for the greater susceptibility of the tills to erosion due to their low clay content.

(b) Moisture control in the subgrade and base course will not require more specialized techniques than are used in temperate climate areas.

(c) Construction will not be hampered by a long period of wet weather in either the spring or the fall as is generally experienced in temperate climate areas.

Hours of daylight vary between 5-1/2 hours on December 21st and 20 hours on June 21st.

Winds generally are light, averaging approximately seven miles per hour in winter months, though blizzard conditions may occasionally be experienced. However, as this area is well forested, wind effects generally are much reduced.

Winter construction would be hampered by the normal problems of working under extreme cold conditions and short daylight hours. However this is offset by the long season of 12 hours and more of daylight between March and September and by the cool to very warm midsummer temperatures which will be experienced.

TABLE I

AVERAGE CLIMATIC DATA

	FORT SIMPSON Department of Agriculture	WRIGLEY
Location	Lat. 61° - 52°N Long. 121° - 21'N	63° - 12'N 123° - 25'W
Elevation (Ft. ASL)	576	511
Record Period	1941-70	1941-70
Temperature (°F)		
Mean Daily	25.2	22.8
Mean Daily Maximum	34.5	32.1
Mean Daily Minimum	16.0	13.4
Extreme Maximum	92	95
Extreme Minimum	-62	-64
Number of Days with Frost	232	235
Frost Free Period (Days)	89	73
Precipitation (Inches)		
Mean Rainfall	7.86	7.66
Mean Snowfall	47.9	50.6
Mean Total Precipitation	11.64	12.73
Greatest Rainfall (24 hours)	1.29	1.58
Greatest Snowfall (24 hours)	11.6	10.2
Greatest Precipitation (24 hours)	1.29	1.58
Number of Days - Measurable Rain	50	47
Number of Days - Measurable Snow	70	64
Number of Days - Measurable Precipitation	126	108

TABLE II

CLIMATIC DATA

FORT SIMPSON

Year - 1958

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL MAY TO SEP
Growing Degree Days Normal (Approx. 42°)	0	0	0	5	175	471	631	520	147	9	0	0	1944

AVERAGE SOIL TEMPERATURE (°F)

Depth Below Ground Surface	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
20 cm	18.7	18.4	20.7	27.0	37.6	51.5	57.9	56.5	45.2	35.1	28.1	22.1
50 cm	25.7	24.0	24.5	27.6	32.2	41.7	49.5	51.2	45.0	37.4	32.5	29.4
100 cm	31.2	29.9	29.0	29.3	30.6	31.4	37.1	42.3	41.7	37.8	33.7	32.1
150 cm	32.0	32.0	31.8	31.6	31.7	31.8	32.2	36.0	38.5	37.2	34.3	32.6

2.5 - Drainage

Drainage throughout the project area is controlled by the deeply entrenched Mackenzie River and its major tributaries. These tributaries, Willow Lake River, River Between Two Mountains, Smith Creek and Hodgson Creek are also deeply entrenched where they cross the project corridor. However, the secondary drainage system throughout the area, including that of the smaller streams tributary to either the Mackenzie or to the other major streams, is less well developed.

Between Miles 346 and 387, in most cases the route parallels the headwaters of streams tributary to the Mackenzie River. Here the banks are usually shallow, the streams small and many are completely dry or frozen to the bottom during late winter months as shown on PH. 5. With the low relief, the slope of these streams is generally quite flat except where they are intersected close to the Mackenzie River or the other major tributaries, and here more rapid downcutting is in progress and deeper gullies have developed.

North of Willow Lake River the streams have formed a parallel drainage pattern and are deeply incised due to the narrowness of the plain between the Franklin Mountains and the Mackenzie River and the rapid changes in elevation. Substantial erosion gullies have developed even on small drainage courses as they approach the Mackenzie River. The route has been located a sufficient distance from the Mackenzie River for these gullies not to pose any problem to either construction or future maintenance of the highway as can be seen on PH. 6.

Photo 5:
Camp move negotiating creek gully - Mile 362.

Photo 6:
Centreline south from Mile 448.

The slopes and shapes of these gullies, as observed during the terrain analysis study, have provided some of the data for the soils mapping. These features are a guide to the nature of the soils encountered by streams as they cut through various soil horizons.

Some of the more active streams north of Willow Lake River develop heavy winter icing conditions as illustrated on PH. 7 and PH. 8. Streams and springs at Miles 394, 412, 422 and 446 were particularly active. This factor will influence the design of the drainage systems in these areas.

2.6 - Vegetation

The project area is generally forest covered, the type, growth and density of the tree cover being typical of Northern Boreal Forest regions. This ranges from stunted Black Spruce and Tamarack in the muskeg areas to White Spruce on higher, better drained ground and in the valley bottoms where more shelter and uniform moisture supply is available. Stands of Lodgepole Pines and Jackpine grow on well-drained, granular soils, and Poplar and Birch on well-drained tills. Tree sizes vary widely; stands of White Spruce of 10 to 14 inch butt diameter and heights in excess of 50 feet were noted. Typical forest cover is shown on PH. 9 and PH. 10.

Some extensive areas of muskeg and peat were found, some of which appear to have resulted from perched water table conditions. The peat depth was generally 2 to 3 feet, although occasional depths up to 10 feet were noted.

Photo 7:
Icing in creek - Mile 422.

Photo 8:
Icing at Mile 412.

Photo 9:
Start of centreline clearing - Mile 347.

Photo 10:
Highway on CNT centreline - Mile 405.

2.7 - Permafrost

2.7.1 - General

Permafrost is a term, first used some 30 years ago, which describes the thermal condition of earth materials such as soil and rock when their temperature remains continuously below 32°F for a year or longer. This term does not define the earth materials which may be solid rock, gravel, sand, silt, clay, ice or any mixture of these materials. Permafrost develops where a negative heat imbalance occurs at the ground surface, resulting in a depth of penetration of winter freezing which exceeds the depth of summer thawing. Ice is an important component of permafrost, however all water in the ground does not freeze at 32°F if it contains impurities or is under pressure. Permafrost underlies about 50 percent of the land mass of Canada and those areas where the necessary thermal conditions at the ground surface exist are generally divided into two zones as follows:

(a) Continuous Permafrost Zone

In this zone permafrost exists everywhere beneath the ground surface, except in newly deposited unconsolidated sediments, and ranges in thickness from 200 feet at the southern boundary to over 1,000 feet in the Arctic Islands. The depth of annual thaw (Active Layer) varies from a few inches to about 4 feet. The southern boundary of this zone is presently based on an arbitrary line located along the 23°F Isotherm of the mean annual ground temperature (MAGT) as measured just below the zone of zero annual amplitude (annual temperature variation equals zero).

(b) Discontinuous Permafrost Zone

This zone extends from the southern boundary of the continuous zone southward until the mean annual ground temperature isotherm is just above 32°F. In this zone frozen and unfrozen layers exist together, the layers being thin and very scattered at the southern limit and increasing in size, frequency and thickness northward. Along the southern fringe, permafrost occurrences have generally been limited to certain terrain types, for the most part dry peatlands, although other occurrences have been associated with the north-facing slopes of east-west valleys or along heavy-forested stream banks. Northward, it gradually becomes more widespread, being associated with a greater variety of terrain types. In this zone the depth of annual thaw (Active Layer) varies up to 10 feet and does not always extend to the permafrost table.

The Active Layer is the layer of earth materials between the permafrost table and the surface which thaws in summer and freezes in winter. Its depth depends on the same climatic conditions that affect the permafrost.

Climate is a basic factor in the formation and existence of permafrost; and the thickness of the permafrost and of the active layer is mainly influenced by air temperature. However time, relief (slope and aspect), vegetation, drainage, snow cover and soil types also have a bearing on these thicknesses. Changes in any of these conditions can result in changes to the active layer and permafrost regime. The effects resulting from changes in this regime are also related to the types of materials, depth of active layer, extent of permafrost and ice content of the soils.

The occurrence of ice in permafrost and also in the active layer is a direct function of the moisture content of the soils. When insufficient moisture is available to fill the soil voids, ice will be present in the form of individual crystals scattered through a soil sample or in the form of ice coating on the soil particles which will cement the particles together into a solid mass. However, when the volume of moisture exceeds the volume of voids, excess ice occurs. When this condition is present ice will be segregated from the soil in the form of lenses, layers, wedges, up to thick massive, almost pure, ice formations. The amount of ice and the size and thickness of the formations is dependent on the amount of soil moisture available and the proximity of reservoirs of water. A system for identification of these conditions has been developed by the National Research Council of Canada and ACFEL. The classification table is shown in Figure 2.

The significance of underground ice depends on whether there is excessive ice present in any form. When excess ice is present, thawing may result in volume changes, settlement and possible loss of soil strength. When there is no excess ice, there will be little or no settlement and loss of strength conditions will be limited.

Other major conditions which are affected by the presence of permafrost are drainage, vegetation and ground surface features. Though these factors are of particular significance in the continuous zone, the effects become less significant with distance southward through the discontinuous zone. Where the active layer is 6 to 10 feet in thickness, permafrost will not form any major impediment to natural drainage nor will it have any major effect on the development of surface features or drainage systems.

<table>
<tr><td rowspan="2"></td><td colspan="2">MAJOR GROUP</td><td colspan="3">SUB-GROUP</td></tr>
<tr><td>DESCRIPTION</td><td>DESIGNATION</td><td colspan="2">DESCRIPTION</td><td>DESIGNATION</td></tr>
<tr><td rowspan="7">DESCRIPTION OF FROZEN SOIL</td><td rowspan="3">SEGREGATED ICE IS NOT VISIBLE BY EYE</td><td rowspan="3">N</td><td colspan="2">POORLY BONDED OR FRIABLE</td><td>Nf</td></tr>
<tr><td rowspan="2">WELL BONDED</td><td>NO EXCESS ICE</td><td>Nb n</td></tr>
<tr><td>EXCESS ICE</td><td>Nb e</td></tr>
<tr><td rowspan="4">SEGREGATED ICE IS VISIBLE BY EYE
(ICE 1 INCH OR LESS IN THICKNESS)</td><td rowspan="4">V</td><td colspan="2">INDIVIDUAL ICE CRYSTALS OR INCLUSIONS</td><td>Vx</td></tr>
<tr><td colspan="2">ICE COATINGS ON PARTICLES</td><td>Vc</td></tr>
<tr><td colspan="2">RANDOM OR IRREGULARLY ORIENTED ICE FORMATIONS</td><td>Vr</td></tr>
<tr><td colspan="2">STRATIFIED OR DISTINCTLY ORIENTED ICE FORMATIONS</td><td>Vs</td></tr>
<tr><td rowspan="2">DESCRIPTION OF SUBSTANTIAL ICE STRATA</td><td rowspan="2">ICE
(GREATER THAN 1 INCH IN THICKNESS)</td><td rowspan="2">ICE</td><td colspan="2">ICE WITH SOIL INCLUSIONS</td><td>ICE + SOIL TYPE</td></tr>
<tr><td colspan="2">ICE WITHOUT SOIL INCLUSIONS</td><td>ICE</td></tr>
</table>

ICE CLASSIFICATION CHART

NRC/ACFEL ICE CLASSIFICATION SYSTEM

FIGURE 2

2.7.2 - Permafrost And Active Layer Conditions Mile 346 to 450

The project area lies in the middle of the Discontinuous Permafrost Zone - just north of the 25°F Isotherm which passes through Fort Simpson. Permafrost in this zone is generally considered to be widely scattered and in most areas to be limited to those locations where the ground is well insulated, such as under thick dry peat, on north-facing slopes or under heavy tree cover in stream valleys. As latitude and elevation increase, the mean annual air temperature decreases and the occurrences of permafrost could be expected to be more frequent.

Using the generally accepted 6°F average temperature differential between Mean Annual Air Temperature and Mean Annual Ground Temperature at the level of zero annual amplitude, permafrost in this area, if it is present, will have temperatures in the order of 29° - 31°F. This means that permafrost, where it exists, will be fairly sensitive to environmental changes, and reductions in ground cover or insulation could cause it to recede or even disappear entirely.

Ground which could be considered as being in a permafrost condition was encountered in only a few holes. This was identified by the presence of an unfrozen layer of ground between two frozen layers, however as temperature readings were not required for this study, no positive identification could be made. Ice lenses were encountered in only one hole at Mile 394. Only two or three occurrences of minor ice lensing (1/8 to 3/16 inches thick) were noted in all of the test pits excavated and no occurrences of excessive or massive ice formations were observed.

Where frozen ground was encountered throughout the full depth of a hole without finding any line of demarcation or layer of thawed ground, the inference was made that the upper 6 to 10 feet was the active layer and the remainder permafrost. Generally, any ice found, in either the active layer or in the permafrost, was in the form of fine crystals scattered through the soil mass.

From the soil moisture data gathered and soils investigated, there is no indication that excess ice will be encountered either in the permafrost or in the active layer.

Permafrost was not encountered at any of the river crossings investigated.

2.8 - Existing Roads, Trails and Seismic Lines

The significance of trails and roads relative to the highway is that they provide an excellent record of the effects of frost and moisture on the natural subgrade, although they provide little in the way of information relative to engineering design and construction.

During the airphoto reconnaissance and terrain analysis study, evidence of old trails and narrow roads, probably wagon and sleigh roads were noted, particularly on the northern end of the project area. The area has been criss-crossed by seismic lines of various ages, particularly the area south of Willow Lake River. The Canadian National Telecommunications line (CNT) and the accompanying road, used annually as a winter road, parallels or is part of the proposed highway from Mile 384 north.

The winter road and a number of seismic lines were used for access to drill hole sites and borrow areas, for moving camp and delivery of supplies.

It was noted during both the terrain analysis study and the field operations that no major settlement or slump had occured, nor had any drainage courses of significance developed along these lines, trails or roads. There is no evidence of any major rutting on the winter road even though this has been used for a number of years and has been subjected to heavy traffic. Several instances of localized sloughing of seismic shot holes were noted but these have only progressed to the point where the natural angle of repose of the underlying soil has been reached. Along these seismic lines, good regeneration of vegetation has taken place.

The fact that no settlement, slumping or water erosion was observed in any of the terrain traversed by these roads, trails or lines, indicates that permafrost if present is too far below the surface to exert significant control on the development of surface features and hence should have only a minor effect on the design and construction of this section of the highway. Generally, the only areas which would be affected by the presence of permafrost would be in deep cut areas where isolated patches of permafrost might be present. If these have high moisture content, they could create localized problems during construction.

3 - HIGHWAY LOCATION

3.1 - Soil Conditions Along the Proposed Highway Centreline

Centreline Borehole Logs are contained in Volumes 2, 3, 4 and 5 of this report. The soils encountered along the route have been broken down into a number of subdivisions based on the data obtained during field investigation and laboratory analysis. Volume 7 contains Laboratory Test Results of the samples obtained during the field investigation.

In the southern section from Mile 346 to 395, the subgrade soils are uniform, being generally clayey to silty sandy till. Throughout this section the subsoils will provide good foundation for the road embankment and any other structures which may be constructed. Few major cuts or fills are anticipated in this section.

Most of the major engineering features such as bridges, heavy cuts and fill embankments are located in the northern section from Mile 395 to 450. In this section the subsoils vary widely from gravelly till to glacio-lacustrine clay-silts.

The soils encountered in the various terrain sub-divisions along the route are described in the following paragraphs:

(a) Miles 346 to 349

In this section the subsoil is composed of a plastic clayey silt having an average moisture content of 20 percent, a Plasticity Index (Ip) of 15 and a Plastic Limit (Wp) of 15. The laboratory test data indicates this material is normally consolidated. No significant ice content was noted.

(b) Miles 350 to 369

The subsoils underlying this area are mainly composed of silty clayey till with considerable sand and gravel content. The clay content varies from 25 to 5 percent decreasing from south to north. The average moisture content is 13 percent, the average Ip is 12 and the average Wp is 15. The maximum depth of annual frost penetration observed was 9 feet. Random ice crystals were noted in samples taken but no ice lensing or massive ice formations were encountered.

(c) Miles 370 to 389

From Mile 367 through this section, there is a progressive transition from the clayey silty tills at the south to silty sandy tills with decreasing clay content to Mile 374. This transition then reverses and the material changes to silty clayey till with an increasing clay content to Mile 389.

In the silty sandy till, Miles 367 to 374, the clay content varies from 2 to 15 percent, and the average moisture content is 10 percent. Between Miles 375 and 389 the clay content increases from 5 to 30 percent, with an average moisture content of 15 percent, a Ip of 12 and a Wp of 18. No visible ice was noted.

(d) Miles 390 to 396

The subsoils in this area are transitional from silty clayey tills in the southern end to fine sandy tills towards the northern end. The transition is gradual towards a terrace composed mainly of fluvial fine sand at mile 392 to 393. This hill probably overlies the till in the area, as beyond it the silty till pattern continues to Mile 395.

These tills are all of low clay content, averaging 10 percent, and have relatively high moisture contents, average 25 percent. However no significant ice lensing or excess ice was noted. Permafrost was considered to be encountered in some holes between Miles 387 and 396. Active springs were located close to the hill at Mile 393 as described in Sub-section 3.2 "Potential Slide Areas" and Section 6 "Deep Cuts and Embankments" and as shown on PH. 11.

(e) Miles 396 to 400

Subgrade soils change from Mile 396 northward, to sandy gravelly till up to 10 feet in depth, underlain by shale bedrock on the sidehill location. The sandy till is transitional to silty till as the lower-lying areas beyond Mile 400 are reached.

(f) Miles 401 to 410

Subgrade material is a silty till, relatively uniform in nature having a low moisture content, average 15 percent. No significant ice lensing was encountered although random ice crystals were noted in samples taken. Some localized permafrost was considered to be present.

(g) Miles 411 to 425

The soils vary from silty clayey till to sandy silty till and beach deposits containing widespread larger granular material, the transition occuring in the northern approaches to River Between Two Mountains at Mile 411. The moisture content is low, average 10 percent, the Ip is 10 and the Wp is 13. Some random ice crystals were noted in samples taken. The sandy silty till continues throughout the section to Mile 425 with an average clay content of less than 5 percent.

Photo 11:
Icing from spring - Mile 393.

Photo 12:
Gravel in trench on centreline, north bank of Smith Creek - Mile 430.

(h) Miles 426 to 428

A gradual transition from sandy till to clayey till occurs at Mile 426. The clay content becomes relatively high, 30 percent, and the moisture content averages 20 percent, the Ip is 10 and the Wp is 20. These soils change again to a granular till at Mile 427 continuing on to Mile 428. Little or no ice content was noted.

(i) Mile 429

This area includes the southern approach to Smith Creek. Here the subsoils are soft, saturated, clayey silts. The moisture content averages 25 percent, the Ip is 15 and the Wp is 20. These subsoils may undergo significant settlement and stability problems could be encountered. This is discussed under Subsection 3.2 "Potential Slide Areas" and Section 5 "River Crossings".

(j) Miles 430 to 438

This lower-lying flat plain surrounding the village of Wrigley is underlain by predominately dense, sandy gravel up to Mile 435. The gravel content, see PH. 12, increases from Mile 435 to 438 at which point the plain ends. The granular material is classified as a glacial outwash, gradational from north to south, being coarser at the north and finer at the south. This is a very large deposit of excellent granular material with a low moisture content of 6 percent.

(k) Miles 438 to 450

The subsoils are fairly uniform throughout this section being composed of clayey silts and silty clays throughout the 15 to 30 feet depth investigated. The clay content averages from 30 to 50 percent, the moisture content is high, averaging 30 percent, the Ip averages 20 and the Wp averages 25. These clays are normally consolidated lacustrine deposits and as such are soft and highly compressible. Embankments will be subjected to significant initial settlement and stability problems could be encountered. This is discussed under Section 4 "Borrow Material Sources" and Section 6 "Deep Cuts and Embankments".

3.2 - Potential Slide Areas

During the course of the terrain analysis studies, substantiated by the field investigations, four old slide areas were noted which are on or near the right-of-way, however there are no indications that they may again become active. Nevertheless, they pose a potential hazard to road construction and are discussed in the following subsections.

(a) Mackenzie River - Mile 346

The first of these slide areas is along the banks of the Mackenzie River near the south end of the study area at Mile 346. Although this area is not located on the alignment investigated, one of the initial road locations skirted the top of the slide. The alignment as investigated relative to this slide area is shown on Figure 3.

(b) Willow Lake River - South Side - Mile 394

A potential slide area was noted at Mile 394. At this point, the road location drops over the edge of a terrace as it descends to the level of the southern approach to Willow Lake River crossing. This location is close to the point where the escarpment is cut by this slide. The slide is not new and there are no indications of recent activity. A deep drill hole on the present alignment at the bottom of the escarpment indicated the presence of a layer 5 feet thick of stiff silty clay with visible ice, at a depth of 25 feet below the ground surface.

In addition to the slide area, an active spring is located at the bottom of this escarpment which drains to the north-west and produces extremely heavy icing conditions across the proposed right-of-way during the winter. This ice depth increased approximately 15 inches during February. It is considered that by relocating the road closer to the CNT right-of-way, this spring and slide area can be avoided. Several holes were drilled to determine sub-surface conditions on both the CNT right-of-way and the area between the CNT right-of-way and the present road location. It was found that the clay horizon did not continue into this area. Therefore, it is recommended that the road be relocated as shown on Figure 4.

(c) Mile 428

Approximately 1,000 feet south of Mile 428, an old slide area was mapped, some 500 feet of which are within 100 to 150 feet of the proposed centreline. This area appears to be inactive and realignment is not essential. However design of the drainage system should ensure that runoff is conducted away from this slide area.

(d) Mile 429

North of Mile 429, the alignment proceeds downgrade to the elevation of the south approach to the Smith Creek crossing. The alignment selected runs down an old slide area on the hillside above the river gulley.

Terrain analysis and mapping of this area confirm that no approach can be made without crossing a slide area since the slide activity extends for several miles along the south bank of this creek. Particular attention will be required in the design and construction of the cuts and embankments at this location. Recommendations respecting methods of achieving a stable roadbed at this site are covered in Section 6 "Deep Cuts and Embankments".

3.3 - Recommended Relocations

As well as the one relocation recommended in subsection 3.2 "Potential Slide Areas" one other area has been noted and is described below. These two relocations total approximately 7.4 miles of the 104 miles investigated.

Miles 370 to 372

A series of low beach ridges occur a few hundred feet to the east of the present right-of-way. Drill holes in these ridges indicate the presence of dry granular materials which would provide good subgrade and base material. In addition, a suitable grade could be designed for this area, permitting balanced cut and fill construction.

This relocation would raise the grade line providing improved drainage for the subgrade and base. The proposed relocation is shown on Figure 5.

4 - BORROW MATERIAL SOURCES

4.1 - General

Preliminary selection of possible borrow material sources was carried out during the planning phase of the work by photogeologic analysis. Mapping of potential sources was done using 1 inch equals 3,000 feet and later on 1 inch equals 1,000 feet photography, the latter was also used for plotting drill holes and access road location. This procedure is discussed in Section 9, "Photogeologic Analysis and Mapping".

The field investigations which followed, permitted the evaluation of the materials contained in the various landforms and deposits with a minimum of drilling and sampling. The correlation of field and airphoto investigations gave the knowledge that, if the material was acceptable, the deposit in general would provide material of the same or similar quality. Borrow Area Borehole Logs are contained in Volume 6 of this report.

The selection of the access routes to be used for the investigation of each potential borrow source was planned with particular attention directed to the protection of the local terrain and forest. Although a large number of potential borrow sources had been mapped, the quality of the materials in each was unknown. It was therefore planned that access would be gained using routes which would minimize permanent marring of the terrain. As the ground would be frozen, optimal use was to be made of seismic lines, open bushland and marsh areas, avoiding heavy timber wherever possible. After borrow requirements have been established, the roads to the selected pits can be chosen on the basis of the best location for a haul road relative to the construction requirements.

Agreement on this procedure was reached during meetings in the field with representatives of the Department of Public Works and the Department of Indian Affairs and North Development. This procedure was followed throughout the course of the field work.

In much of the area, the trend of the landforms is such that many of the borrow sources extend across the right-of-way. Certain of the centreline holes were therefore sited to provide further substantiation of the materials encountered in the borrow sources. In this manner the location and number of drill holes required for borrow source evaluation was kept to a minimum. The additional data obtained through test-pitting, complements that provided by the drilling, as detailed in Subsection 4.2 "Test-pitting at Borrow Locations".

Embankment quantities, estimated from the preliminary profiles provided by the Department of Public Works, were also used as a guide to define the extent of exploration required.

In locating borrow areas, consideration was given to quality, quantity, accessibility, workability and haul distance of the potential sources. Those sources located in landforms elevated above the level of the surrounding topography were generally well drained and characterized by very low moisture contents. This may permit operation of these pits on a year-round basis. In general, all the material in those sources situated higher than the surrounding terrain would be available except for that left on the pit floor to maintain good drainage during the operation of the pit. Quantities available have been estimated on this basis.

In those sources where the material was located near or below surrounding ground level, or where the exploited pit would be below normal ground level, the quanities available have been estimated on a more conservative basis. It was considered that operations in these pits would be to shallow depths to ensure no interference from excessive water collecting in the pit. In practice, certain of these pits may yield much greater quantities than estimated, as water collection may not be a problem.

Suitability of borrow areas for possible year round workability is related to the moisture content and the texture of the materials they contain. In coarse-grained, well-drained deposits, though annual frost penetration may be deep or permafrost may be present, there will be insufficient moisture present to freeze the particles into a solid mass and these deposits will remain workable throughout the year. In fine grained granular deposits even with relatively low moisture content, there may be sufficient moisture to freeze the soil particles into a solid mass which will be difficult to excavate, and which will break into solid lumps. This will make the working of such deposits and the placing of these materials both difficult and expensive. If frozen fine-grained granular soils having excess moisture are used in embankment construction, voids may develop in the embankment when the ice melts and the excess moisture may cause temporary loss of strength until the soils drain. For this reason, soils with high moisture content normally are not placed when in a frozen state.

In general, the borrow sources investigated contained materials suitable for road construction ranging from tills to outwash gravels. The only major area where suitable borrow material is absent was between Miles 440 and 450 where the entire area consists of silty clays and clayey silts.

In some of the shallower borrow sources, strata of poorer materials are inter-bedded with other strata of good quality materials. By the blending of appropriate quantities of each, a much larger source of suitable embankment materials may be obtained. This can be done by opening a pit of sufficient depth so that layers of both materials will be cut and natural blending will be achieved during excavation.

The following summarizes the various borrow sources by section. The location of each source can be seen on the Photo Mosaics in Volume 9 of this report. The Borrow Area Borehole Logs and Test-Pit Logs are contained in Volume 6. Laboratory test results on samples, as well as estimates of available borrow material are contained in Volume 8 - Borrow Area Laboratory Test Results and Evaluation Sheets.

4.1.1 - Miles 346 to 380

These materials generally consist of well-graded sandy to silty clayey till, however glacio-fluvial sands and gravels (Mile 370 to 375) and local beach ridges are also encountered. Clay content in the tills averages 15 percent, gravel averages 8 percent and moisture content ranges from 15 to 18 percent. Borrow areas are generally located in low ground moraine or beach ridges. In general, materials throughout this section are well suited for embankment construction. (See PH. 13, PH. 14 and PH. 15)

Although ice crystals were noted, they were randomly located through the material and no ice lenses were observed.

4.1.2 - Miles 380 to 390

In this section, a noticeable transition in the grain size of borrow material occurs. The material becomes coarser as the terraces to the north are approached. Silty clayey tills were still found at depths of 20 feet below the surface. The

Photo 13:
D-7 trenching test pit - 355-BT-1.

Photo 14:
Test pit - pick at 5' level - 364-BT-1.

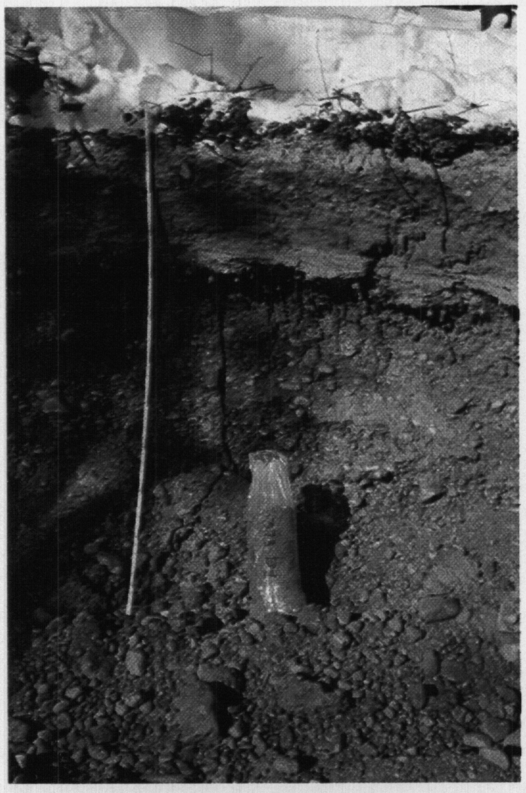


Photo 15:
Test pit - sample at 3' level - 373-BT-1.

Photo 16:
Test pit - sample at 4' level - 400-BT-1.

materials are all well-drained and ice free, with a low moisture content averaging 10 percent. These materials will be suitable for embankment construction.

4.1.3 - Miles 390 to 396

This section straddles Willow Lake River, Mile 394. The borrow material available is basically a silty to clayey till except for the area in Miles 392 to 393 where a sandy terrace overlies the till. The clay content averages 15 percent and the moisture content averages 20 percent. In the sandy terrace, a fine friable sand is the predominant component, the clay content being less than 5 percent.

Sufficient borrow material is available within this section to complete the embankments required and the bridge crossing approaches.

4.1.4 - Miles 396 to 397

Sufficient quantities of fine-grained gravel, which can be obtained in the centreline cuts and in the adjacent borrow areas, were proven in this section. These materials generally occur in shallow deposits of 5 to 15 feet. They are underlain by shale bedrock.

4.1.5 - Miles 398 to 409

Borrow materials within this section vary from well-graded silty tills with some clay and gravel to tills with a high gravel content. The moisture content averages 13 percent and the gravel content ranges from 5 to 10 percent with sand contents up to 40 percent.

Photo 17:
Test pit - sample at 4' level - 407-BT-1.

Photo 18:
Test pit - sample at 6' level - 420-BT-1.

Little variation in texture or colour was noted through this section. (See PH. 16 and 17.)

In view of the gradation and gravel content, the borrow materials within this section can be compacted to high density and are therefore suitable for any high embankments which may be required. Large quantities of material are available through this section. The bottom of a number of the borrow pits in this area would generally be at the same elevation as or slightly higher than surrounding terrain and would therefore have excellent drainage.

4.1.6 - Miles 410 to 425

Within this section, the borrow areas reflect the transition apparent in the centreline holes. With the exception of several borrow areas at approximately Mile 414, the other potential borrow sources are beach deposits and sandy till with considerable silt and gravel content but no clay. These materials are well-graded. The water content is low, averaging 10 percent and the materials are ice free. (See PH. 18).

At Mile 414, the material is a well-graded silty, clayey till with considerable gravel content, approximately 15 percent. The clay content averages 15 percent and the water content is low, averaging 10 percent. No ice lenses were noted though random ice crystals were seen in some samples. This material would make excellent embankment material.

4.1.7 - Miles 426 to 429 (at Smith Creek)

In this section, the borrow material is clayey silty till with a low gravel content of 5 percent. The clay content is approximately 30 percent and the Ip is 20. These materials are similar in characteristics to those encountered in the centreline exploration.

The materials within this section are not as suitable for embankment construction as the alternate sources of good quality material available from the sections to the north and to the south.

4.1.8 - Miles 430 to 437

Most of this section falls between Smith Creek and Hodgson Creek. The village and airport of Wrigley are located within this section. Substantial quantities of sandy gravel between Miles 430 and 437 are located in this area. These are glacial deposits. (See PH. 19, 20, 21 and 22).

Previous work done by Pemcan '72 Services, plus the photogeology and centreline drill holes confirmed the availability of gravel in this area. The previous work done in this area has shown that there is more than sufficient gravel available thoughout this section for embankment requirements.

Because of the excellent quality and large quantity of material available in this area, consideration should be given to the movement of this material to other sections, particularly areas to the north, which do not contain sufficient desirable materials.

4.1.9 - Miles 437 to 440

Exploration within this section indicates a change in soil from sandy gravel at Mile 437 to silty and clayey till at Mile 439. Here too, the sources would make good embankment materials and

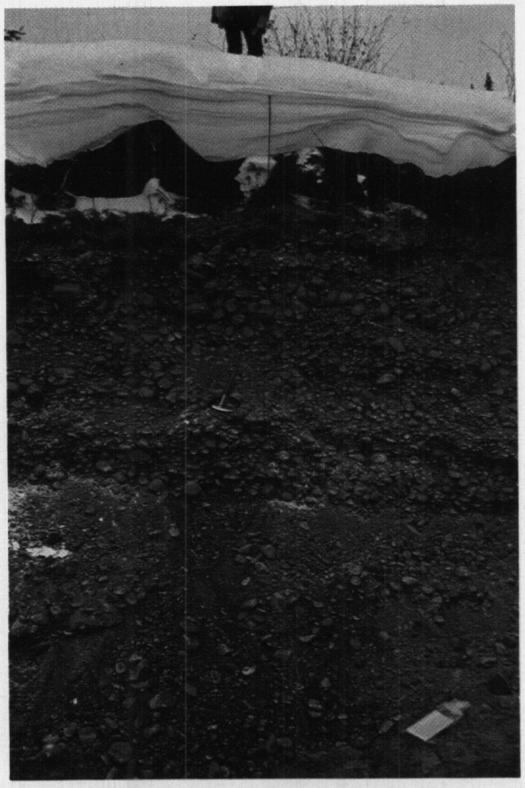


Photo 19:
Test pit - upper 10' level - 430-BT-1.

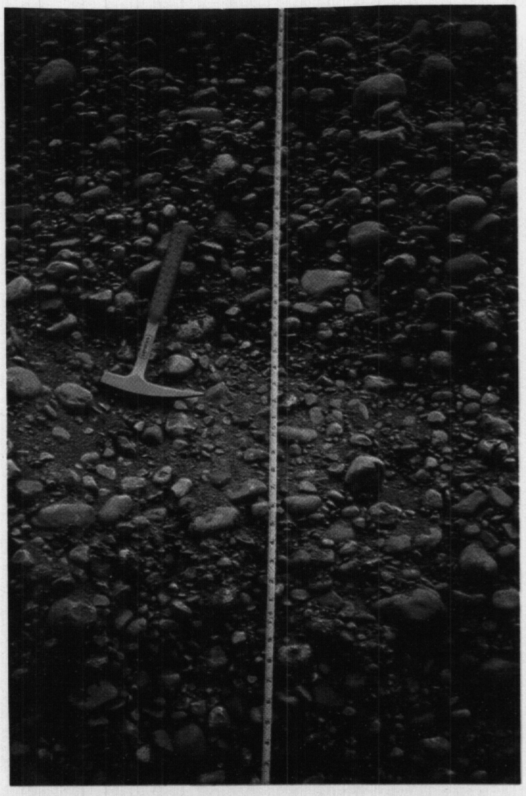


Photo 20:
Test pit - pick at 5' level - 430-BT-1.

Photo 21:
Test pit on centreline, north bank of Smith Creek.
Pick at 8' level - 431-BT-1.

Photo 22:
Test pit - pick at 8' level - 433-BT-1.

would provide sources for sections to the north. Moisture content averages 15 percent and no ice was observed. (See Ph. 23).

4.1.10 - Miles 440 to 450

All borrow material sources explored throughout this area are similar in quality to that found in the centreline holes. The material is basically a lacustrine silty clay and clayey silt described in Subsection 2.2 "Soil Conditions Along the Proposed Highway Centreline". This material is not considered as suitable for fill and embankment construction as the alternate sources available. One of these alternate sources is in the borrow areas to the south. Another borrow source and one which would provide good sub-base materials is in the talus slopes at the foot of Mt. Gaudet. (See PH. 24). A third source would be to develop a quarry in the outlier directly to the north of Mt. Gaudet.

4.2 - Test-Pitting at Borrow Locations

In order to obtain larger representative samples and to develop a better appreciation of the type and quality of materials in the borrow areas, a program of test-pitting of the potential borrow sources was carried out. The test-pits were opened using a D7 bull-dozer, equipped with a blade ripper to penetrate the surface frost. The bulldozer was able to excavate trenches from 3 to 12 feet in depth depending on the nature of ice bonding in the soil. These trenches enabled a detailed examination to be made of the materials in the borrow sources. In addition, large grab samples were taken from the test-pit face. Colour photographs were taken of the materials in every test-pit, as represented in PH. 13 to PH.23 inclusive.

Photo 23:
Test pit - pick at 7' level - 437-BT-1.

Photo 24:
Talus slopes, Mt. Gaudet - Mile 445.

4.3 - Rock Fill

4.3.1 - Mile 396 - North of Willow Lake River

The mountains forming the south end of the McConnell Range just north and east of the Willow Lake River crossing are composed of fairly massive limestone 300 to 400 feet thick. This material would form an excellent source of crushed rock for concrete aggregate for any structures required in this area. It would also be excellent as rip-rap for embankment protection or for protection of bridge piers against scouring.

4.3.2 - Mile 445 - Mt. Gaudet

The material in this mountain is limestone. Due to the steep face to the south east, large amounts of talus have collected at the foot of this slope. This could provide an excellent source of material for a subgrade without establishing a rock crushing operation. After such a subgrade had been built, finer material could be transported from borrow sources in the sections immediately to the south. There are several small outliers to this mountain closer to the right-of-way which would provide good quarry sites should they be needed.

5 - RIVER CROSSINGS

5.1 - General

One of the terms of reference of the geotechnical investigation was to identify problem areas on the approaches and to investigate the bridge foundation conditions at certain pre-selected major stream crossings. Based on the findings, Acres is required to make foundation recommendations to the consultants responsible for the bridge design.

The original list of crossings was as follows:

(a) Willow Lake River

(b) River Between Two Mountains

(c) Hodgson Creek

To this list, the bridge consultants added a fourth site where a bridge was contemplated, namely

(d) Smith Creek.

During the investigation the merits of various types of foundations and their construction were reviewed. In view of the prevailing climatic conditions, the remote locations and associated limited accessibility to construction materials, piled foundations appear to be the best suited for the soil conditions in most cases. Other factors which influenced this conclusion are the freezing, thawing, river ice and flood conditions which occur at these sites.

As stated in Subsection 2.7 "Permafrost", generally no indications of permafrost were found at any of these crossings, either in the banks or under the stream beds.

Individual foundation reports on each of the major stream crossings have been prepared for the bridge consultants. Each of these reports contain the detailed results of the investigation on the stream bed, the banks and one mile of centreline on either approach to the crossing.

The specific crossing investigations are discussed briefly in the subsections which follow:

5.2 - Willow Lake River

5.2.1 - General Data

Location (See PH. 25)	- Mile 394
Total length	- 900 feet
Number of spans proposed by bridge consultant	- 5
Number of spans recommended by environmental consultant	- 4

Bridge Consultant - T. Lamb, McManus Associates
Environmental Consultant - F.F. Slaney & Company Ltd.

5.2.2 - Subsoil Data

Five boreholes were drilled to depths varying from 50 to 100 feet to investigate the subsoil conditions for the pier foundation.

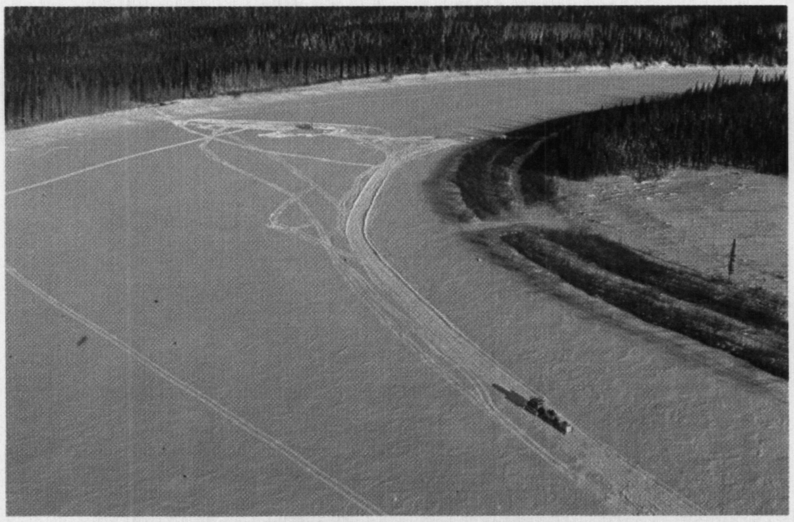


Photo 25:
Drilling on ice, Willow Lake river crossing - Mile 394.

Photo 26:
Drilling - north bank of River Between Two Mountains - Mile 411.

The subsoil data from these five holes were reviewed and it was found that profiles are similar and can be summarized as follows. The subsoils are composed of sand and gravel of medium density, having an average blow count (N) of 25 per foot using a standard penetration test, to a depth of 20 feet below the river bottom. This is underlain by dense silty sand with some clay and gravel with an average blow count of 32 per foot to a depth of 30 feet below the river bottom. From 30 to 60 feet the soils are composed of medium plastic, stiff silty clays having an average blow count of 31 per foot.

5.2.3 - Foundation Design

Pile supported foundations are recommended for all piers and abutments at this location. Steel friction piles are recommended and will have sufficient capacity if driven to a depth of 50 feet or refusal. Accurate foundation analyses can be undertaken using the available borehole data when bridge loads and pile type are known.

5.3 - River Between Two Mountains

5.3.1 - General Data

Location (See PH. 26)	- Mile 411
Total length	- 260 feet
Number of spans proposed by bridge consultant	- 3

Bridge Consultant - Reid, Crowther & Partners Limited.

5.3.2 - Subsoil Data

Three boreholes were drilled to depths of 40 to 60 feet. The borehole data for the above locations indicates a uniform profile with little variation in soil characteristics and composition. The subsoils are composed of very dense sand and gravel with some silt and boulders. The standard penetration test shows that the 'N' values were greater than 40 blows per foot, indicating the dense nature of the sub-strata.

5.3.3 - Foundation Design

Pile supported foundations are recommended for all piers and abutments at this location due to scour considerations. Steel H piles are recommended and will have sufficient capacity if driven to a depth of 30 feet or refusal. No significant settlements would be expected in the above subsoils. Accurate foundation analysis can be undertaken using the available borehole data when bridge loads and pile types are known.

5.4 - Smith Creek

5.4.1 - General Data

Location (See PH. 27)	- Mile 429
Total length	- 50 feet
Number of spans proposed	- 1

Bridge Consultant - Reid, Crowther & Partners Limited

5.4.2 - Subsoil Data

Two holes were drilled at this site to depths up to 31 feet. The subsoil varies from clayey silt on the south bank to dense sand and gravel on the north bank. The clayey silts are of recent origin, probably resulting from the old slides which have occured on the hills to the south. They are normally consolidated and highly compressible. The subsoils on the north bank are dense sands and gravels.

5.4.3 - Bearing Capacity

At this site it is recommended that consideration be given to the use of a culvert system as discussed in Section 5.4.5 instead of pile foundations. If however, pile foundations are considered, it is anticipated that an abutment on the north bank would remain relatively stable while an abutment on the south bank could undergo significant settlement. If pile foundations are to be used, load bearing tests may be required to determine the bearing capacity for any pile system.

5.4.4 - Additional Considerations

The recent slides which have occured on the slope above the stream present a potentially serious problem. Though there is no indication that these slides are presently active, the soils are such that any change in moisture content or loading might induce further movement. Such movement could endanger any bridge abutment which might be constructed on the south side of this stream.

Photo 27:
North bank of Smith Creek - Mile 429.

Photo 28:
Hodgson Creek at crossing - Mile 436.

5.4.5 - Recommendations

In view of the relatively unstable condition of the south bank and of the small size of this stream, it is recommended that consideration be given to the use of a fill and culvert at this crossing. Culverts are relatively light structures and do not impose any serious foundation problems. They will also be able to withstand any unexpected loading which might be imposed by slides from the adjoining slopes. It is also recommended that the grades be modified so as to give a height of fill no greater than 20 ft. at this point.

5.5 - Hodgson Creek

5.5.1 - General Data

Location (See PH. 28)	- Mile 436
Total length	- 150 feet
Number of spans proposed by bridge consultant	- 1

Bridge Consultant - Reid, Crowther & Partners Limited

5.5.2 - Subsoil Data

The two holes drilled for the abutment foundation at the south and north banks of the Hodgson Creek are 50 and 43 feet deep respectively.

A review of the borehole data from these two locations indicates that the subsoils are composed of very dense stratified sand and fine gravel to a depth of 45 feet and firm silty clay below 45 feet. The 'N' values from the standard penetration tests taken at 5 feet intervals varied from 45 to 100 blows per foot.

5.5.3 - Foundation Design

Pile supported foundations are recommended for all piers and abutments at this location. Steel friction piles are recommended and will have sufficient capacity if driven to a depth of 50 feet or refusal. Accurate foundation analyses can be undertaken using the available borehole data when bridge loads and pile type are known.

6 - DEEP CUTS AND EMBANKMENTS

6.1 - General

During the investigations particular attention was paid to those areas where deep cuts or fills were anticipated. The preliminary road profile on the mile sheets, provided by the Department of Public Works, was used as a guide in selecting those areas where the cuts or fills were planned.

The depth of drilling at the location of the deep cuts was increased to ensure that wherever possible, the drill hole depths would be taken to 15 feet below the road grade surface.

Following the completion of the field work, the profile of the entire route was reviewed in conjunction with the borehole and laboratory data to determine the magnitude of all cuts and embankment fills, their location and the associated stability problems under the existing subgrade soil conditions. The project area can be divided into two major sections:

(a) The section south of Willow Lake River, Miles 346 to 395, and

(b) The section north of Willow Lake River, Miles 395 to 450.

These are discussed separately in the following subsections.

The mileages and locations of the various areas discussed are shown on the 1 inch equals 1,000 foot scale Photo Mosaics contained in Volume 9 of this Report.

6.2 - Miles 346 to 395

Throughout this section, the only significant cuts or embankments are those cuts at the approach to the Mackenzie River crossing and the cuts and embankments at and to the south of Willow Lake River. The remainder of this section will be mainly on shallow embankments with heights of 3 to 6 feet and with shallow cuts up to 10 feet in depth. Similarly, any cuts indicated or any which would be developed, should the suggested relocation be adopted, are unlikely to be in excess of 10 feet in depth.

The subsoils found in this area are generally composed of silty sandy tills in the north, varying to silty and sandy clayey tills in the south. No significant permafrost was found in this area and the annual frost penetration was found to be 8 to 9 feet. In the materials encountered at depth the moisture content was similar to or less than that in the overlying soils.

In these subgrade soils, and considering the depth of annual frost penetration, soil moisture conditions would be relatively stable and thawing of any underlying permafrost encountered would not add any significant amounts of moisture to the subgrade soils.

6.2.1 - Cut at Mile 347

This involves a cut approximately 27 feet in depth where the road climbs up steep banks from the base level at the Mackenzie River crossing onto the first terrace above the river.

Boreholes to depths of 40 feet revealed a dense stratified subsoil consisting of a fluvial clay-silt. The average moisture contents vary, being 21 percent in the upper sandy silt, 18 percent in the silty sand and 26 percent in the clayey silt.

In the clayey silt the Ip is 35 and the Wp is 21. The 'N' values obtained for standard penetration tests averaged 75 blows per foot in the silty and sandy alluvium and 45 blows per foot in the silty clayey alluvium.

It is recommended that excavated slopes should be cut to no steeper than 3 horizontal to 1 vertical for reasons of stability.

In view of the fine-grained nature of the soils which will be encountered throughout this cut, it is recommended that the drainage system be designed so that drainage flow in the cut will be restricted to local runoff from the cut, all other water being conducted away from the cut. Slope protection measures such as the planting of vegetation or the placing of coarser granular material to control erosion are also recommended.

6.2.2 - Cut - Mile 393 to 394 (On Relocated Alignment)

This involves a deep cut of approximately 35 feet where the road drops over the edge of the sandy terrace at this location. Boreholes of 80 to 100 feet in depth revealed that the material is composed of generally medium dense fine uniform friable sand with negligible silt content. The upper 10 feet of sand are loose and dry. Below this layer, to depth, the sand is of medium density with an average 'N' value of 31 blows per foot. Water contents are low throughout.

It is recommended that the cut backslopes should be no steeper than 2.5 horizontal to 1 vertical.

In view of the fine grained nature of the soil which will be exposed in this cut, it is recommended that the drainage system be so designed so that drainage flow in the cut will be restricted to local runoff. Suitable slope protection measures such as the planting of vegetation or the placing of coarser granular slope protection are recommended.

6.2.3 - Embankment - Miles 393 to 394

Immediately following the cut described above, a deep embankment will be required to carry the road down to meet the approaches to Willow Lake River crossing.

At the location originally proposed, a very active spring was found which drained across the right-of-way from east to west. As this spring could affect the stability of any embankment foundation and will also cause icing conditions, relocating the highway to the east closer to the CNT right-of-way is recommended as discussed in Subsection 3.2 "Potential Slide Areas". Test holes which were drilled on the recommended re-alignment confirm that the sandy subsoils prevail through this area.

The analysis of the embankments in this area was based on the recommended realignment. The embankment can be constructed using the sand till from the cut immediately to the south and also from the borrow sources nearby. Sideslopes of this embankment should be no steeper than 2.5 horizontal to 1 vertical for stability.

As the material in both the cut and from the nearby borrow sources is generally a uniform fine-grained sand, it is recommended that measures be taken to protect the slopes from erosion. Suitable erosion protection could be achieved through the use of planted vegetation, coarser granular material or rip-rap hauled from other sources.

6.2.4 - South Approach Embankment - Willow Lake River Crossing - Mile 394

Boreholes along the approaches to the south bank of the river indicate that the subsoils are generally a fluvial clay-silt having a relatively low moisture content. As such, the subgrade material will provide adequate bearing strength for this embankment which varies from 20 to 30 feet in height.

It is recommended that the embankment sideslopes be no steeper than 3 horizontal to 1 vertical for reasons of stability.

6.3 - Miles 395 to 450

Most of the major cuts and fills in the project area occur within this section. During the investigation it was found that there were six locations involving deep cuts and embankments which would require individual study. An analysis of the stability of the slopes and foundation subsoils has been carried out for these areas. The problems associated with each location have been analyzed in conjunction with the foundation conditions and borrow materials found in each area. The conclusions reached relative to each location are discussed in the following subsections.

6.3.1 - Embankment - Miles 395 to 399

Embankments to be constructed through this area include the northern approaches to Willow Lake River Crossing. The fill materials which will be used to construct the embankments through this section will probably be composed of materials similar to that found in the foundation subsoil, based on borrow pit investigation in this area. The embankment will possess, therefore, characteristics somewhat similar to those of the subsoil. Analysis indicates that the saturated or short term base failure conditions will be the most critical since the fill and foundation material will be relatively homogeneous. Analyses carried out on these embankments with heights varying from 20 to 30 feet, indicate that embankment slopes should be no steeper than 2 horizontal to 1 vertical for fills less than 10 feet in height and no steeper than 2.5 horizontal to 1 vertical for fills over 10 feet in height.

6.3.2 - Cuts - Miles 395 to 399

The cuts required in this area are generally sidehill cuts, with centreline depths up to 30 feet maximum as indicated on the preliminary profiles. Through this area, boreholes, generally to depth of 5 to 15 feet below the grade line, indicate that the soils generally are dense, dry, silty sandy or gravelly tills with some clay. In several holes, shaley bedrock was encountered.

It is recommended that the backslopes of these cuts be no steeper than 2 horizontal to 1 vertical for cuts below 15 feet in depth and no steeper than 2.5 horizontal to 1 vertical for cuts deeper than 15 feet. Where competent rock is encountered in the bottom of cuts, normal rockcut backslopes of 1 horizontal to 5 vertical would be satisfactory, where weathered rock is encountered, 1 horizontal the 1 vertical backslopes are recommended. However on the top of any such rock cut, a 10 foot wide berm should be used before commencement of the earth backslope.

6.3.3 - Embankment - Miles 426 to 429

Boreholes along the centreline and in the borrow areas in this section indicate that the soils are generally clayey silts having clay contents up to 30 percent and with relatively high moisture contents. The foundation subsoils and the embankment borrow materials are relatively plastic with a liquid limit of about 40 and a Wp of 15.

Using field identification and laboratory test data for these soils, analyses were made to assess the safe bearing capacity, allowable slopes and probable subgrade and embankment settlements for embankments up to 40 feet in height. The embankment fill material was assumed to be composed of materials similar to those found in the subgrade and to be compacted to 95 percent of Standard Proctor density. Stability analyses were done for both short and long term failure conditions.

These analyses show that the saturated or any short term failure conditions are the most critical. Under these conditions, for embankments of up to 40 feet in height built with materials similar to those found in the subsoils, i.e. from nearby borrow sources, a maximum slope 2.5 horizontal to 1 vertical is recommended. For any embankments greater than 40 feet in height, slopes of 3 horizontal to 1 vertical are recommended.

Settlement calculations indicate that the total estimated settlement of the subgrade would be in the order of 9 inches. In the embankments, settlements in the order of 4 inches would be anticipated. However most of this settlement will take place within the construction period or during the first six months.

In view of the plentiful sources of good granular borrow materials immediately to the south or just north of Smith Creek, it is recommended that consideration be given to constructing the embankment from the above mentioned sources of superior borrow material.

6.3.4 - Cuts - Miles 426 to 429

The cuts required in this area are generally shallow in depth. Backslopes should be no steeper than 2 horizontal to 1 vertical.

6.3.5 - Approach Cut and Embankment Smith Creek Crossing - Mile 429

(a) South Approach

The terrain in this area slopes sharply toward the river. The subsoils are composed of soft silts and clays, resulting from the slides which have occured in the bank.

Subsurface investigation in this area was carried out to a depth of 30 feet. The soil was found to be saturated and the clay content increased progressively downslope.

The clays are normally consolidated and have high water content and plasticity index.

It is recommended that the maximum height of fill used at this point be no more than 25 feet due to the weak highly compressible foundation. Because the subgrade soils have low bearing capacity and are highly compressible, care must be taken in the design, construction and maintenance of the embankments in this area. It is suggested that a procedure which will reduce the potential construction and maintenance problems is two stage construction as follows:

Stage I - Construct a complete drainage system including parallel and off-take ditches. Following completion of the drainage system, place and shape a sand filter 15 to 18 inches in depth, followed by 18 feet of the embankment fill then, leave the area to consolidate for 3 to 4 months. Construction traffic could travel on this surface providing the surface is reshaped to remove ruts.

Care will have to be taken in the placing of the culvert under the fill if this recommended approach is adopted to allow for differential settlement of the culvert.

Stage II - In the second stage, the embankment is completed, shaped and the surfacing course placed.

This procedure will reduce post-construction settlement and increase the bearing capacity of the subsoils. Embankment slopes of 3 horizontal to 1 vertical are recommended.

(b) North Approach

The subsoils on the north side of the river are a dense sandy gravel. These subsoils will provide good foundation for any embankments which may be constructed on these soils. Embankment slopes of 2 horizontal to 1 vertical are recommended.

The cut which will be required on this approach will be approximately 30 feet in depth. It is recommended that the backslopes of this cut be no steeper than 2.5 horizontal to 1 vertical.

6.3.6 - Embankment and Cut - Miles 448 to 449

In this area the cuts are expected to be up to 20 feet in depth and the fill embankments to be up to 35 feet in height.

The soils in this area are the same as those which predominate from Miles 438 to 450, i.e. silt with a relatively high clay content. These are glacio-lacustrine in origin and are continuous to depths of 15 to 30 feet reached in the centreline drilling program. Both frozen and unfrozen soils were encountered with high moisture content, these soils had average liquid limits between 45 and 50 percent.

Any high embankments will undergo significant settlement when constructed on these normally consolidated clays. Analyses made indicate that settlements in the order of 1 foot may be experienced.

Recommended embankment sideslopes are no steeper than 2 horizontal to 1 vertical for embankments less than 10 feet in height and 3 horizontal to 1 vertical for higher embankments.

Based on stability considerations, excavated backslopes should be no steeper than 2.5 horizontal to 1 vertical.

7 - CONCLUSIONS AND RECOMMENDATIONS

7.1 - Conclusions

After review and analysis of all data obtained in this investigation, the field conditions which will be experienced and the borrow materials available, it is concluded that a highway capable of meeting modern loading requirements can be constructed through the section investigated without the use of unusual or special design procedures and construction techniques.

The following specific conclusions were reached:

(a) A satisfactory location has been selected for the highway alignment. As discussed in Section 3 "Highway Location" there are potential problem areas which can be avoided by minor relocations. In total, these relocations represent approximately 7.4 miles of the 104 miles investigated.

(b) Permafrost should not be a major problem in either the road design and construction or in the establishment of drainage systems. The permafrost encountered was at sufficient depth that it will not significantly affect surface conditions.

(c) Normal highway and drainage design and construction techniques can be used with due allowance being made for local conditions. Winter embankment construction may be desirable in several sections of the project area as discussed in Subsection 7.2.4.

(d) Subgrade soils are generally suitable for embankment foundation purposes and in many cases can be used for sub-base and even base material. However, in several locations, particularly in the north end of the section, the materials are normally consolidated clays or silty clays which will require particular attention during design and construction.

(e) There is a plentiful supply of suitable material for embankment construction. The distribution of this supply is irregular, particularly in the northern 10 to 12 miles where the borrow sources contain silty-clay material which is not as suitable for embankment construction as material from alternate sources. In some other areas the borrow materials are fine-grained granular materials which may require slope protection on embankments, cuts and river crossing approaches to prevent erosion.

(f) River crossings, including the approaches to these crossings and the bridge foundations, can be constructed using normal design and construction methods. Some winter construction at these sites may also be advantageous in that pile-driving and other work may be done off the river ice.

7.2 - Recommendations

7.2.1 - Recommended Relocations

In the 104 mile section of the proposed highway investigated relocations are recommended totalling approximately 7.4 miles. One relocation is at Willow Lake River - South Side - Mile 394 as detailed in Subsection 3.2 "Potential Slide Areas" of the report. The other is detailed in Subsection 3.3 "Recommended Relocations".

In addition, a slide near the highway alignment at Mile 428 should be considered during detailed engineering design and may result in one additional relocation, approximately 3,000 feet in length.

7.2.2 - Drainage

Moisture control will probably be a major factor in maintaining the road and embankment stability. Most of the subgrade and borrow materials are dry and every effort should be made during design, construction and maintenance of the road to maintain and improve this condition. During construction the use of additional culverts may be desirable in some locations to ensure that proper drainage is maintained while the final drainage system is being developed. The design of the drainage system should be such that ponding of water along the right-of-way will be eliminated. The spring run-off period will probably be the most critical as the ground will be frozen, water storage will be minimal and therefore, run-off will be high. The number and sizing of culverts, and the design and protection of culvert inlets and outlets including the use of embankment protection will assist in controlling erosion in these areas.

The majority of the soils encountered in the project area (tills and fluvial silty sands) are characterized by relatively low clay contents and high silt and sand contents. As a result these soils will have a relatively low resistance to scour by flowing water, and care must be exercised in the design and construction of the highway drainage system to minimize run-off erosion.

It is recommended that the maximum allowable velocity in unprotected drainage channels be 1.5 feet per second to minimize erosion damage. Freshly excavated drainage channels will be highly susceptible to run-off damage and extensive use of scour control methods (such as vegetation protection, coarse granular channel protection and properly designed check structures) is strongly recommended.

In the fill areas, the natural ground should be left intact to the maximum possible extent. Use of frequent culverts under the embankment is recommended to minimize ditch flows and to prevent ponding of water against the embankment. Where high embankments have been constructed of fine-grained material, coarse granular slope protection or vegetation slope protection is recommended to prevent run-off damage to the sideslopes.

Ditches in cut areas must be designed for peak run-off conditions. In major cuts, natural ground cover at the top of the slope should not be disturbed and interceptor ditches should be used to minimize run-off into the cut. Velocities in ditches must be carefully controlled and suitable erosion protection measures taken.

7.2.3 - Embankment Construction

A large part of this section of the highway will be constructed on embankments, hence the ultimate performance of these sections of the highway will depend to a large extent on the construction methods adopted and the construction quality control maintained.

Compaction of the subgrade will not usually be required due to the dense, high strength nature of most of the foundation soils. Foundation stripping should follow normal procedures for earthwork in this area.

The selection, handling, placing and compaction of borrow materials will require close attention.

Placing of fill should be done in lifts not exceeding nine inches and compaction should be done with a vibrating sheepsfoot roller. Fills less than 15 feet in height should be compacted to a 95 percent Standard Proctor density and higher fills to 98 percent Standard

Proctor density. Standard compaction testing of borrow material, when design is finalized, is recommended as is standard quality control testing during construction to ensure rigorous compliance with the specifications.

Much of the sub-base for the road and some of the fill material in the study area which will be utilized will be obtained from either till or fluvial deposits, containing relatively high percentages of silts and sands and low percentages of clay sized particles.

These soils are frost-susceptible and frost heave may be a problem in areas where a high water table exists or water is ponded against the fill.

Measures recommended to eliminate or reduce the severity of frost heave problems are outlined below:

(a) Use of clean, free-draining granular fill to the maximum possible extent.

(b) Use of a 2 feet thick layer of well compacted, free-draining granular fill separating the embankment from the sub-base in areas of high water table where granular borrow is scarce and embankments must be constructed from silty till or fluvial borrow material.

(c) Use of properly designed and constructed roadside and interceptor ditches, whenever possible, to lower the water table by a minimum of 4 feet in wet, boggy areas where a high water table exists.

(d) Properly controlled compaction of embankment soils to a minimum of 95 percent Standard Proctor density. Moisture content must be rigorously controlled and it is recommended that no fill be placed more than 2 percent over the optimum moisture content.

Right-of-way width should follow normal highway practice in order to maintain the present, relatively dry conditions of the subgrade. As the climate is cool and moist evaporation will be slow and full right-of-way width will ensure more rapid drying. This will also permit proper drainage and backslope design.

7.2.4 - Construction Procedures

In general, construction requirements have been outlined in the previous subsections. However in two areas, namely Mile 426 to Mile 429 and Mile 438 to Mile 450 where the subsoils are of poor quality, consideration should be given to the use of construction techniques which will permit additional time for drainage and consolidation of the subgrade prior to completion of the embankment and also minimize deformation and rutting of the subgrade.

This would involve the construction of ditches, placing and shaping an 18 inch sand filter course followed by the placing and shaping of a good granular or rock fill base up to 70 percent of the total height for fills greater than 10 feet in height. This should then be left to drain and consolidate for a period of 3 to 6 months following which the final lifts would be placed.

In the specific areas mentioned, it is recommended that consideration be given to the construction of the initial embankments during the winter months. This would serve to prevent deformation and rutting of the subgrade by construction equipment and would save construction time in that it would be possible to complete the embankment early the following fall without major settlement being experienced later. Casual construction traffic could use the surface during the consolidation period.

Although extensive permafrost is not anticipated, patches may be encountered in deep cuts particularly on north-facing slopes. This should not cause any serious problems providing standard northern construction techniques are used in such cases. This would involve the flattening of backslopes in permafrost areas and the selective placing of filter materials on exposed permafrost, if it contains excess moisture, in order to permit draining and consolidation of the soil without sloughing.

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DEPARTMENT OF PUBLIC WORKS

MACKENZIE HIGHWAY MILE 346 - 450

GEOTECHNICAL INVESTIGATION

PART II

PROCEDURES

8 - PROJECT OPERATIONS

8.1 - Planning

In view of the limited shipping season remaining after August 29, 1972, the date on which the contract was awarded, in which to mobilize and transport equipment, fuel and supplies to Hay River for barging to the starting point at Mile 346, two alternatives were considered:

(a) To carry out immediate mobilization with whatever equipment was available and finalize the field program later.

(b) To plan the program, obtain equipment and mobilize to Fort Simpson and use the winter road for access as soon as the Mackenzie River ice bridge was available.

A preliminary analysis indicated that the field work could be completed within a two month period and that winter mobilization would permit completion of the work within the scheduled period.
It was, therefore decided, to delay mobilization of the equipment, supplies and personnel for the field investigation until January 1973.

Detailed office planning for the field operations was carried out, consisting of selection of equipment, personnel requirements and the drilling and sampling procedures to be used.

Several factors which influenced the choice of equipment, planning, scheduling and logistics of the operations were:

(a) Effect of the extreme cold temperatures on personnel and equipment.

(b) Working safety and health in extreme temperatures and isolated location.

(c) Logistics of communications and supply deliveries in isolated areas.

(d) The equipment, spare parts and supplies to be carried with the camp to maintain operational efficiency.

(e) Logistics of supplying fuel to the project.

In view of the extreme low January and February temperatures which would necessitate keeping equipment running 24 hours per day, it was planned to optimize equipment utilization by working round-the-clock. With these guidelines established, a work schedule was developed which indicated that two drills working on a 24-hour per day basis could complete the work within the required period.

It was estimated that the field party would total 23 to 25 persons. The requirements for all support equipment and vehicles were then based on these figures and the various aspects are discussed under the following headings.

8.2 - Camp and Services

Although initially it was planned to use sleigh mounted mobile camp units, a brief market study indicated late deliveries and a current shortage of both sleigh-mounted units and sleighs. The second choice was to use trailer camp units on wheels with under-axle skids, constructed to withstand the stress of traversing rough terrain and capable of being towed in a train of units pulled by a bulldozer. Short length, 32 feet, units were selected to ease travel over rough ground.

Accommodation requirements indicated that three sleeper units, one office sleeper, and the sleeping quarters in the utility trailer would provide adequate accommodation. A kitchen-diner having an 18-man seating capacity was considered to be suitable, since with the round-the-clock operation there would seldom be the need to accommodate the full complement. A utility unit and wash trailer with toilet and wash facilities completed the camp requirements.

A 30 KW diesel-electric plant mounted in a trailer along with a 10 KW diesel-electric standby unit and workshop and storage space was selected on the basis that approximately 2 KW of power would be needed per sleeping unit and 5 KW in each of the kitchen and utility units. For emergency use the standby would provide sufficient power to keep the camp operational.

8.3 - Support Equipment

The following objectives guided the selection of the support equipment planned for use in this work both as to type and quantity.

8.3.1 Personnel Transport

This would be required to carry:

(a) drill personnel and inspectors to and from the drill equipment.

(b) fuel and spare parts to the drills.

(c) the drill foreman and mechanic.

(d) the geologist for flagging holes and reconnaissance.

(e) the party chief throughout the work area.

(f) personnel under emergency situations.

Some of the equipment had to have all terrain capability under extreme conditions of temperature and snow cover, and some were to be rubber-tired 4-wheel drive units to provide more rapid movement on cleared lines and winter roads.

Based on the foregoing, the following transport equipment was chosen:

(a) One FN 20 all-terrain unit equipped with a winch and crew cab.

(b) Three 4-wheel drive trucks, one being a crew cab model, all equipped with winches, tire chains and spare fuel tanks.

8.3.2 - Camp Support Unit

An FN 550 all-terrain tracked unit was selected to provide mobility for moving fuel, towing camps, hauling water and general utility usage.

8.3.3 - Heavy Support Equipment

Two D7H bulldozer units or the equivalent, were considered necessary for:

(a) snow ploughing as required on the centreline and other cleared lines.

(b) clearing of portions of the line and of access routes to drill hole locations.

(c) towing the camp units.

8.3.4 - Fuel Sloops and Storage Sheds

To enable sufficient fuel, both gasoline and diesel, for 7 to 10 days operation to be carried with the camp, fuel sloops with 2,000 gallons capacity each were preferred to barrels, as these units could be towed to meet fuel tankers on the winter road. Storage sheds for spare parts, small tools, lubricants and materials were also planned.

8.3.5 - Radio

A review of radio communication systems available indicated that single side band radios working through the CNT telephone system would be suitable for providing external communications. For continuous communication between personnel to minimize time lost in effecting repairs, locating drill sites and advising of emergency situations, a VHF system was selected with a base station in the office and mobile sets in all drill rigs, trucks and the all-terrain vehicles.

8.3.6 - Drills

This equipment is discussed under Section 10 "Drilling and Sampling". One aspect, however which should be emphasized, was the need to carry an extensive supply of spares of those items which experience indicated were the most likely to be required. Some of these were complete replacement units which would enable repairs to be effected rapidly, others were parts which could be replaced in a minimum down-time.

8.4 - Mobilization

Preliminary field reconnaissance was undertaken at the end of September 1972 to gain first hand knowledge of the area through which the alignment was located and to assess services and support available in the Fort Simpson area.

Airphoto analysis and mapping was started immediately and continued through part of September, October, November and December as aerial photography and mosaics became available. A more detailed description of this phase of the work is described in Section 9 "Photogeologic Analysis and Mapping".

It was planned to mobilize to Fort Simpson in January 1973 and move out from that point as soon as the winter road and Mackenzie River crossing permitted.

Transportation of equipment commenced January 10th and continued through January 24th. An advance party of field staff moved to Fort Simpson on January 19th followed by the balance of the party on January 24th.

Despite delays experienced in gaining access along the winter road due to the slow start being made in its preparation, all units and crew were at DPW Camp #2 at Willow Lake River by January 27th. Drilling operations with the first drill commenced January 28th, and the second drill on January 30th. Full scale round-the-clock drilling operations with both units commenced January 31, 1973.

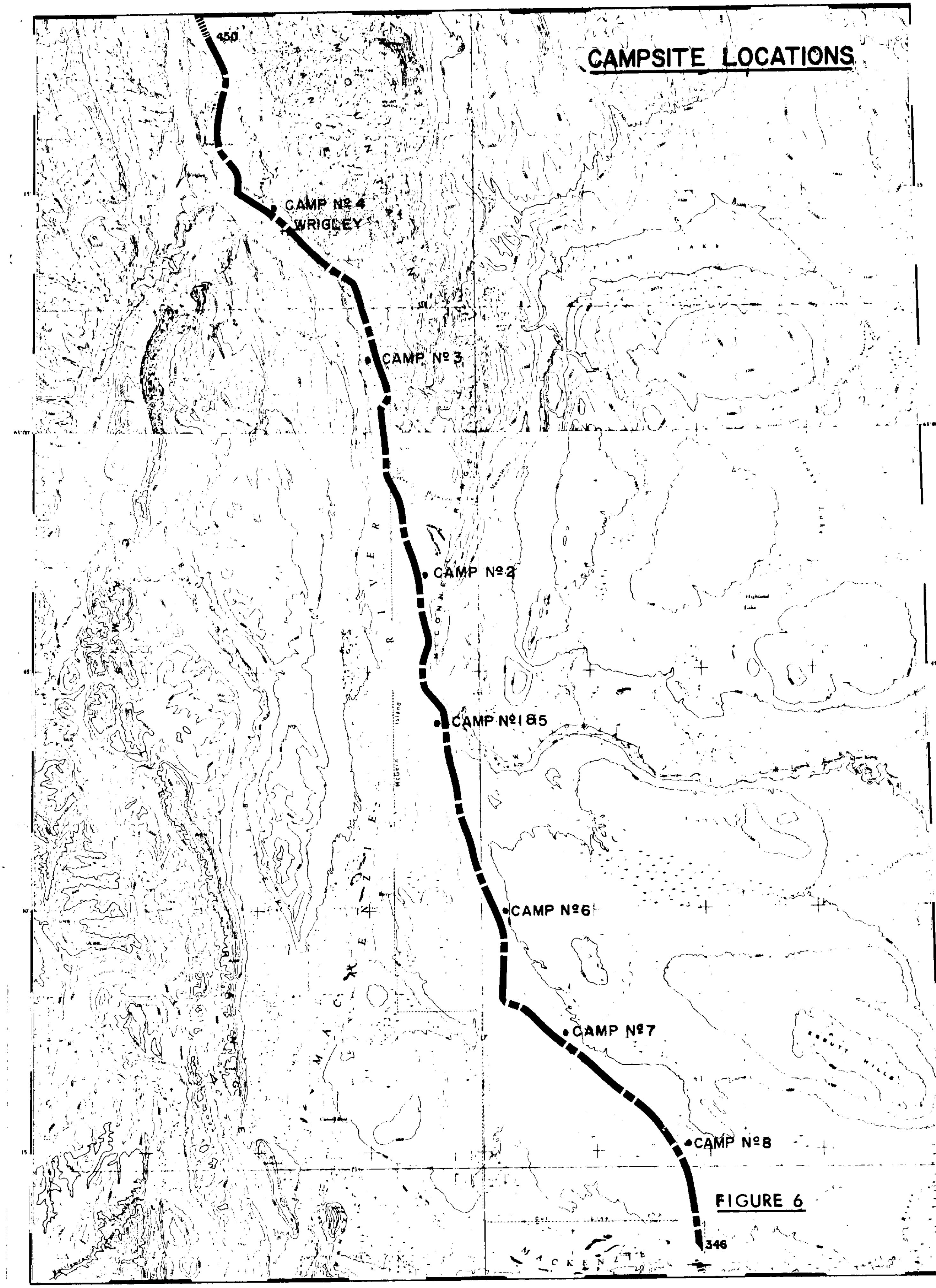
8.5 - Operations and Schedule

The schedule called for an average of 2-1/2 miles per day of centreline and borrow site drilling.

Camp moves were scheduled to take place when the drills had moved 4 to 5 miles ahead of the camp; moves of 10 miles maximum were planned. In practice, due to the ready access by winter road it was possible to operate drilling units 8 to 10 miles from the camp without major loss of time.

Orginally it had been planned to mobilize to a point north of Wrigley and commence operations working from north to south, however, this was modified upon the direction of the Department of Public Works to start at Willow Lake River. The field operations therefore moved north from Willow Lake River to the end of the line, Mile 450, then made a long move back to commence work south of Mile 385 and continued south from this point, completing the drilling at Mile 346 on March 8th. This change created a problem in that all detail mapping done to date had been from the north end of the section. Campsite locations used during the operations are shown on Figure 6.

The planned and achieved operations schedules are shown in Figure 7.


CAMPSITE LOCATIONS
450
CAMP Nº 4
WRIGLEY
CAMP Nº 3
CAMP Nº 2
CAMP Nº 1&5
CAMP Nº 6
CAMP Nº 7
CAMP Nº 8
FIGURE 6
346

8.6 - Equipment Performance

In general the equipment performed exceptionally well and overall down-time was low. However, weakness in suspension and drive trains showed up in the 4-wheel drive units, and these problems were compounded by having to obtain replacement parts from distant automotive distributors.

8.7 - Radio Communications

8.7.1 - Single Side Band

Despite efforts during planning to reduce operating problems to a minimum including selecting the optimum aerial direction and having 60 foot crank-up type towers, atmospheric conditions were such that this system was not a reliable means of communication with outside centers.

8.7.2 - Internal VHF Mobile System

The results achieved with this system were excellent. A range of 10 to 15 miles was generally achieved by all units and base station transmissions were audible up to 25 miles. This system prevented much loss of time and unnecessary movement of personnel and vehicles.

8.8 - Equipment Utilization

The following table shows the time major equipment was operationally available.

TYPE	OPERATIONAL AVAILABILITY
Drill #1	90 percent
Drill #2	85 percent
D6C Bulldozer	99 percent
D7 Bulldozer	100 percent
FN 20	80 percent
FN 550	85 percent

8.9 - Delivery of Fuel and Supplies

As planned, during the period from January 27th to February 27th, deliveries of all fuel, supplies, and personnel was by ground transport, this system working well with no delays experienced.

However, once the operation moved south of Willow Lake River, where the new alignment diverged completely from the winter road greater difficulties were experienced. It was not possible to move bulk fuel tankers over the old seismic lines, but the camp fuel storage tanks had sufficient capacity to permit completion of the work. If additional fuel deliveries had been required, a major problem in arranging deliveries to fuel sloops hauled to points along the winter road would have been that of communications, due to the unreliability of the single side-band system.

Deliveries of supplies, spare parts and personnel to the camp sites south of Willow Lake River were carried out using the 4-wheel drive vehicles and by helicopter delivery from Fort Simpson, arranged through Turner Expediting.

8.10 - Demobilization

Upon completion of the work it was planned to move the camp to the winter road for return to Fort Simpson where units would be assembled for dispersal.

This commenced on March 8th and by March 9th the complete camp had been moved to the winter road via an existing seismic line. From this point the drills, vehicles and tractors which hauled some of the camp units, moved to Fort Simpson. Wheeled tractors obtained in Fort Simpson pulled the remaining camp units into the Village. Here all equipment was checked, cleaned, and packed, and the return hauls to the points of origin commenced on March 12th.

Personnel were demobilized upon arrival at Fort Simpson with the exception of a small crew held for camp clean-up and packing for the return hauls.

During the period March 9 - 12, the remainder of the test-pitting in the area between Miles 420 and 390 was completed by a small geotechnical party supported by one D7 bulldozer and several personnel vehicles.

9 - PHOTOGEOLOGIC ANALYSIS AND MAPPING

9.1 - General

Prior to field operations, extensive use was made of aerial photography in reconnaissance, photogeologic analysis and mapping. This was done in several stages and to varying degrees of detail based on the scale of the aerial photography available. In conjunction with this work, topographic maps, scales 1:250,000 and 1:50,000, geomorphological maps and geological reports and maps were studied. Discussions were held with members of the Geological Survey of Canada and other geotechnical personnel who had worked in this area to elicit basic background data and information for this program.

This aspect of the work was emphasized during the planning stage as it allowed optimizing of the field program with a consequent reduction in the period of field operation.

9.2 - Photogeologic Analysis Techniques

9.2.1 - General

Photogeologic techniques enable accurate analyses to be made of the terrain in terms of the soil types, bedrock and the geological history of the area. These make use of a series of complementary diagnostic keys, each of which provides part of the basic information. The major keys used in this analysis are:

(a) Topography - shape, pattern, relief

(b) Erosion - shape of gulleys, slope, resultant deposits

(c) Drainage - type of drainage, present and ancient drainage patterns

(d) Vegetation - ground cover, tree types

(e) Tone - light, dark

Using these techniques, the soils and bedrock in an area can be identified and typed. Subsequent field exploration will then be limited to confirmation of the data collected in the photogeologic study, identification of boundaries and of variations in soil types in transitional zones and obtaining of samples to provide details of the quality of materials in a deposit.

9.2.2 - Application to This Project

The application of these techniques to this project is basically as follows:

(a) to evaluate the subgrade soils along the alignment

(b) to select up to four sources of borrow materials per mile

(c) to determine potential problem areas

(d) to recommend possible alignment changes where problem areas exist or where investigations indicate subgrade conditions could be improved.

Of the above items, only (a) and (b) will be expanded at this point since items (c) and (d) are basically an outcome of the evaluation done under item (a). Furthermore these are discussed in detail under Subsections 3.2 "Potential Slide Areas" and 3.3 "Recommended Relocations".

The subgrade soils along the alignment were evaluated in two stages, firstly by general soils mapping, secondly by the amalgamation and evaluation of all the data obtained. The results of this work are discussed in detail in Subsection 3.1 "Soil Conditions Along the Proposed Highway Centreline".

The general philosophy followed in the initial selection of borrow sources was that all deposits which the photogeologic techniques indicated as being potential sources of high quality construction materials were identified. The four most promising sources per mile were then chosen on the basis of quantity, accessibility, topography, drainage and moisture content.

Certain factors cannot be evaluated from airphotos. This is particularly true of the exact grain size distribution within certain landforms. For example, a drumlin may contain fine-grained or coarse grained till, or may vary from fine to coarse without exhibiting surficial characteristics permitting definition of grain size variation from aerial photography. However in formations which have resulted from flowing water, the sorting action of water will be reflected in the deposit in that grain sizes will be gradational from coarse to fine, moving downstream from the source.

The division of the soil types in the project area was limited to their engineering use identification, in which frictional soils (i.e. sands and gravels), transitional soils, cohesive soils, organic and peat materials along with the surface features such as slides, slumps and springs, were identified. These are considered to be the features which have a major bearing on the road location and construction. The soil classification chart used is shown on Figure 8 - Unified Soil Classification System.

9.3 - Reconnaissance

Initially, 1 inch equals 1 mile aerial photography, obtained from the Geological Survey of Canada, covering the area from Mile 346 to Mile 411 plus uncontrolled mosaics at scales of 1:50,000 and 1:24,000 as well as topographic and geological maps and reports were used in developing a basic knowledge of the area and a concept of the program.

Subsequently a four day field reconniassance was made between September 25th and September 29th, during which a general appreciation was obtained of the terrain, land forms, geology, route alignment, river crossings, accessibility and ease of movement. During this period additional data and background was gained through a brief study of 1 inch equals 3,000 feet and 1 inch equals 1,000 feet aerial photographs available at DPW field offices.

9.4 - Photography and Mapping
Scale 1 inch equals 3,000 feet

Subsequent to the field reconnaissance, a set of 1 inch equals 3,000 feet aerial photographs covering the corridor to be studied during this investigation was supplied by the Department of Public Works and more detailed analysis and mapping commenced. Using these photographs, potential borrow sources were identified.

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (little or no fines)	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	more than 50% of coarse fraction RETAINED on no. 4 sieve	GRAVELS WITH FINES (appreciable amount of fines)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
more than 50% of material is LARGER than no. 200 sieve size	SAND AND SANDY SOILS	CLEAN SAND (little or no fines)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	more than 50% of coarse fraction PASSING no. 4 sieve	SANDS WITH FINES (appreciable amount of fines)	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	liquid limit LESS than 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
more than 50% of material is SMALLER than no. 200 sieve size	SILTS AND CLAYS	liquid limit GREATER than 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

SOIL CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

FIGURE 8

Access routes to borrow sources, trails, and seismic lines for movement of camps and equipment, and potential campsites and water sources for field operations were selected.

This information, mapped on mosaics at a scale of 1 inch equals 3,000 feet was then submitted to the Department of Public Works to accompany the application for an extension to the Land Use Permit covering the Project Area (N72E173). This extension was required to permit investigation of the potential borrow sources outside the proposed highway right-of-way.

9.5 - Detailed Photogeologic Analysis 1 inch equals 1,000 feet

Photography and mosaics at a scale of 1 inch equals 1,000 feet were received from the Department of Public Works early in January and detailed mapping was commenced. Although there was insufficient time in which to complete all the mapping prior to mobilization, some 24 miles, starting from Mile 450 were completed prior to commencement of field operations.

The data was mapped on 1 inch equals 1,000 feet scale photo mosaics covering the centreline and investigation corridor through the project area. Mapping continued during the field operations although in less detail as working time available for detailed mapping was limited.

In addition to the mapping of the soil formations, the locations of the proposed boreholes were marked on the mosaics. In this manner the relationship between centreline and borrow area holes was established and the information obtained from the centreline holes was used in proving borrow areas.

The borehole locations, borrow sources and soils mapping are shown on the alignment sheets contained in Volume 8 "Photo Mosaics".

The final mapping and checking of this information was completed upon return to the office.

10 - DRILLING, SAMPLING AND TESTING

10.1 - General

Initially the boreholes for the drilling program were located on 1 inch equals 1,000 feet mosaics and photographs. This was done in conjunction with the "Photogeologic Analysis and Mapping" as described in Section 9. During the field operation supplementary holes were added as required to define more clearly problem areas. In some instances holes were deleted as the geological formations were sufficiently defined and additional holes would have been unnecessary.

10.2 - Drilling Equipment

In planning the drilling operations, several major factors guided the selection of the equipment to be used. These were generally as follows:

(a) The drills should be mounted on self-propelled all-terrain tracked equipment, each complete with its own winch so as to be self-sufficient from the standpoint of mobility.

(b) As holes deeper than 100 feet were not envisaged, small drill rigs could be used. Speed of drilling, mobility and versatility were of greater importance than depth capability.

(c) As a wide variety of subsoil materials were anticipated, drilling and sampling techniques were to be versatile, and ancillary equipment such as mud pumps and casing would be required.

(d) As rock drilling or coring capability would only be required for a small percentage of the program, the time expended on it would have little bearing on the duration of field operations.

(e) Drill equipment was to be of a type that repair and replacement of worn and failed parts could readily be undertaken in the field.

(f) The drilling units should be identical to enable interchangeability of parts and reduce the number of spare parts carried.

The drilling equipment selected consisted of two reinforced Gardner-Denver 200 'helidrills', mounted on Foremost 60 tracked vehicles. These drills were rigged with a mast and kelly which could handle 10 foot rod lengths. Each drill carried a total of 120 feet of rod and 70 feet of casing. Tungsten carbide insert type coring bits, rock bits, diamond coring bits, core barrels, standard two inch Split Spoon samplers, three inch split barrel samplers, 140 pound and 350 pound hammers, standard three inch Shelby tubes, cone penetrometers, 4-1/2 inch and 5-1/2 inch soil augers were the major items of drilling and testing equipment carried, quantities of each being determined by anticipated usage.

Ancillary equipment including a mud pump and water tank were sled mounted.

These drills use compressed air and reverse flow drilling which blows cuttings to the surface. As the cutting return is rapid, it is possible to obtain accurate grab samples from any depth interval.

10.3 - Drilling Program

10.3.1 - Centreline Boreholes

Centreline holes were spaced so that the materials within the formations and the transition soils at boundaries could be accurately delineated. The depth of hole to be drilled was a minimum of 15 feet below ground surface, deepened as required, particularly in areas where cuts were indicated on the preliminary centreline profile data supplied by Department of Public Works. In special locations such as bridge foundations and very deep cuts, holes were drilled to 100 feet depth.

As well as the regular drill hole depths of 15 feet, certain centreline holes were drilled to depths of 25 to 40 feet to enable Neutron Density testing to be carried out. This was done on advice from the Department of Public Works who had been requested to permit this additional work by Mackenzie Valley Pipeline Research Limited. In total, 690 centreline holes were drilled over the 104 mile section. The data on these holes is contained in the Centreline Borehole Logs, see Volumes 2, 3, 4, and 5 of this report.

10.3.2 - Borrow Source Boreholes

Initially up to 3 or 4 potential borrow sources per mile were identified, the best one or two were then selected for drilling and sampling, the remaining areas being held in reserve should the others prove unsatisfactory.

The number and location of drill holes used to confirm the presence of suitable borrow materials varied depending on the type, extent and topography of the formation. In total 184 boreholes were drilled in borrow areas. The data on these holes is contained in Volume 6 "Borrow Area Borehole Logs" of this report.

The depth of holes in the borrow areas ranged from 15 to 80 feet depending on the particular requirements, materials and formation being sampled. It was estimated that, per mile of proposed road, an average of some 50,000 cubic yards of fill materials would be required. As most of the borrow areas were found to contain more than 100,000 cubic yards of usable material, generally only the best source in each mile was drilled and sampled. An estimate of usable borrow material for each borrow source has been made. The evaluation sheets are contained in Volume 8 "Borrow Area Laboratory Test Results and Evaluation Sheets".

10.4 - Sampling

Sufficient sampling equipment was mobilized with the drills to take disturbed or undisturbed samples of either frozen or thawed materials and make standard penetration tests.

Generally disturbed samples were taken for field identification, moisture testing, grain size analysis and Atterberg limits at 5 foot intervals or at changes in soil type. During the drilling the field technicians collected the disturbed samples from the drill hole as shown on PH. 30. This material was visually examined and identified, any ice in the sample noted, and the data recorded on the drill log report.

Photo 29:
Drilling with mud - 80' hole in dry sand - edge of terrace - Mile 393.

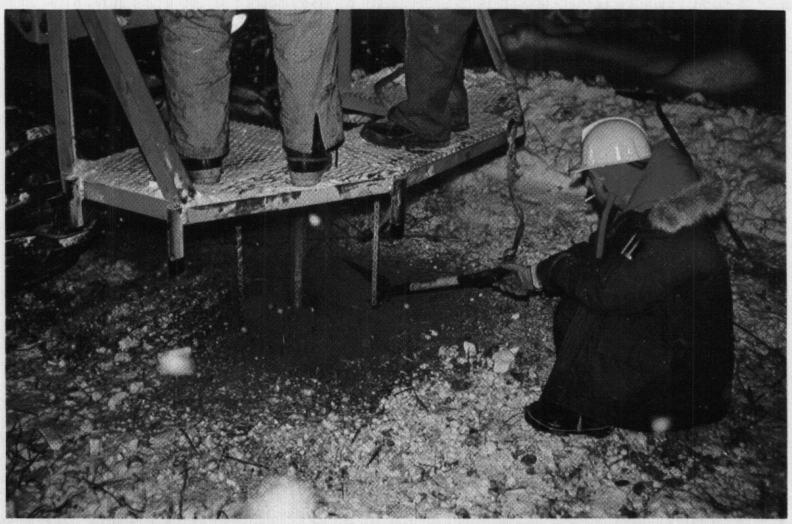


Photo 30:
Sampling during night drilling operations.

Undisturbed samples were taken using a standard Split Spoon sampler if the material was granular and with Shelby tubes if cohesive material was encountered.

At all bridge site locations, in addition to routine sampling, standard penetration tests were conducted and samples taken from the Split Spoon if the material was granular and with Shelby tubes if the material was cohesive. High embankment or suspected problem areas were tested in a similar manner.

Sampling of frozen ground was done using either a standard Split Spoon sampler or 3 inch Shelby tubes where possible.

When mud was used to assist in drilling, the mud pumping circulation was also observed as a further guide to the nature of the soils being penetrated. Drilling with mud is shown on PH. 29.

10.5 - Test-pitting of Borrow Sources

In order to provide a better visual examination of borrow materials and to obtain more representative samples of the materials contained in the sources, test pits were dug by bulldozer, described in Subsection 4.2 "Test-Pitting at Borrow Locations".

The trench faces were examined in detail and colour photographs taken of each face exposed. A large bulk sample was taken down the face of the trenches for laboratory testing and to provide reference samples which were left with the Department of Public Works in Fort Simpson.

The locations of the test pits are shown on the Photo Mosaics in Volume 8 of this report. Colored prints of some of the sites tested are included as PH. 13 to PH. 23 inclusive of this report, and the remainder of the photographs are available as coloured slides.

10.6 - Sample Handling

Disturbed samples were placed in plastic bags, excess air removed, and the bag sealed and labelled at the drill rigs. Unfrozen split spoon samples were generally handled in the same manner. If the split spoon samples from frozen material were in good condition they were treated as undisturbed samples.

Undisturbed samples were wrapped in aluminum foil and taken to the field laboratory. Shelby tube samples were sealed in the laboratory and maintained in a frozen or unfrozen condition depending on the insitu conditions where the sample had been taken. Frozen samples were stored in boxes at the camp until ready for shipment.

Samples which were to be tested in the Calgary laboratory were double wrapped in plastic bags, identified and then a series of these bags were placed in heavy canvas bags for shipment.

Undisturbed frozen samples and unfrozen samples were separated and each type shipped in insulated containers. To monitor the temperatures inside the containers during transit, a max./min. thermometer was placed in each container. The laboratory recorded the temperature on arrival and placed frozen samples in a deep freeze unit until they were required for testing. None of the samples shipped in this manner suffered a change in state during transit.

Representative samples of all materials were placed in plastic bags and deposited with the Department of Public Works for future reference.

10.7 - Sample Testing

10.7.1 - Field Testing

In view of the number of samples to be tested, the importance of soil moisture conditions, and the necessity of being able to obtain additional samples for check testing; moisture testing was done in the field; with part of the office trailer being equipped for this purpose. A Quincy testing oven and a Soil Test Infra-red oven were installed along with a triple beam balance and the necessary containers and laboratory utensils. The Quincy oven was used for testing all samples taken. The infra-red oven being used only for quick tests on parts of samples and for check tests on granular materials, never where organic soils or clays were encountered. Moisture testing was done on a daily basis.

In all approximately 4,900 moisture content tests were performed. The results of this testing have been plotted on the drill logs which are contained in Volumes 2, 3, 4, 5 and 6 of this report.

10.7.2 - Laboratory Testing

All other sampling was conducted in Ripley, Klohn and Leonoff's soils testing laboratory in Calgary. The routine tests on all samples used for classification of the materials summarized as follows:

(a)	Grain size analysis;	Number tested = 771
(b)	Atterberg limits where cohesive soils encountered;	Number tested = 314
(c)	Moisture content check tests on approximately 10 percent of the samples;	Number tested = 89

The special testing carried out included the following:

(a) Standard Proctor Compaction tests.

(b) Drained Triaxial tests on remoulded samples.

(c) Undrained Triaxial tests on remoulded samples.

(d) Thaw consolidation tests.

(e) Consolidation tests.

(f) Undrained triaxial tests on undisturbed samples.

Some of the data has been plotted on the Borehole Logs. Complete test results on samples taken from centreline boreholes are contained in Volume 7 "Laboratory Test Results" and the results on samples from borrow sources are contained in Volume 8 "Borrow Area Laboratory Test Results and Evaluation Sheets".