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OBSERVATION OF THE DESIGN AND PERFORMANCE OF THE DEMPSTER HIGHWAY

APRIL 1985

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ABSTRACT

The Dempster Highway is unique in that it is the only "all weather" unpaved highway in Canada which crosses the Arctic Circle. Conventional granular materials for embankment construction are scarce in the unglaciated permafrost terrain traversed by the highway. The unique features of permafrost are the cause of problems which make highway construction difficult: thaw degradation, consolidation and instability of ice-rich soils. The Dempster Highway is 743 kilometers long, connecting the communities of Dawson City, Yukon and Inuvik, N.W.T.

The objective of this report was to observe the highway five years after construction of the final phase (1979) and to observe its performance. Thaw degradation of the permafrost beneath embankments has occurred in some places to depths of a metre or more below original frost line resulting in undulations in the road surface. Isolated problems have resulted from cuts in ice-rich soils, but remedial works have stabilized slopes after 5 years. The majority of culverts installed are still functioning properly (some are over 13 years old). The majority of the highway has retained its design speed (90 km/hr) and the embankments are performing remarkably well. This is encouraging since some sideslopes are inclined at 35 degrees to the horizontal.

The intent of the design philosophy was to preserve the permafrost during the life of the embankment. This has in large part

been achieved, but thaw and compression of the organic mat covering the mineral soils has resulted in subsidence of half a metre or more in many places, during the life of the structure. Subsidence was worst in the first year, but was substantially diminished by the fifth year.

TABLE OF CONTENTS

LIST OF PLATES

LIST OF FIGURES

Page

OBSERVATION OF PERFORMANCE AND DESIGN OF THE DEMPSTER HIGHWAY

I INTRODUCTION

It is the objective of the following study to provide qualitative and some quantitative observations of the design and performance of the Dempster Highway. In addition, engineering recommendations and design improvements are suggested in order to alleviate problems arising from highway construction. The Dempster Highway is unique in that it is the only "all weather" highway in Canada which crosses the Arctic Circle. The terrain traversed by the highway is dominantly continuous permafrost a large portion of which is also unglaciated.

The highway was, and is, intended to be a resource development road with an expected traffic volume of approximately 20 vehicles per day. A large portion of the traffic is heavy industrial or commercial vehicles. The highway has gradually been constructed over a period of 26 years in response to trends in favorable economic and regulatory (governmental) climates. It begins at a junction on the Klondike Highway (No. 2) 40 kilometers east of Dawson City, Yukon and ends at Inuvik, N.W.T. (A length of 743 km.) The last 63 kilometers is called the MacKenzie highway, named in anticipation of a junction with the proposed MacKenzie highway/pipeline alignment.

The underlying philosophy employed in the design and construction of the highway assumes preservation of the permafrost. The alternate approach is to prethaw the alignment via "stripping" the organic cover and allowing 1 or 2 years for thaw settlement to occur prior to commencement of embankment construction. This is not a viable approach because: (1) there is usually insufficient notice of intent to construct; (2) permafrost degradation and concomitant erosion and thaw instability are potential hazards with expensive remedies; (3) aesthetic and environmental laws will be violated by "stripping"; and (4) proper drainage of water from thaw consolidated ice-rich soils may be prohibitively expensive.

II HISTORY

The Dempster Highway follows the route taken by its namesake, Corporal WJD Dempster of the Royal North West Mounted Police, who set out from Dawson City in 1910 to search for the ill-fated patrol of Inspector Fitzgerald.

The highway had its origin in the late 1950's at a time when the Federal Government was encouraging economic development of "the north". In this context, the highway was a "development road" intended to connect specific reserve development areas with existing communities. It began as a winter tractor trail built in the winter of 1954 on behalf of Conwest Exploration Co. Ltd. (Interim Management Plan, 1978).

In spite of interruptions from shifting federal government policy since that time, the road construction has advanced steadily as follows: (See Figure 1 for Locations):

LOCATION (km)	DESCRIPTION	YEAR COMPLETED
0 - 72	Approaching North Fork* Pass	1960
72 - 126	To Chapman Lake	1962
126 - 198	Chapman Lake to Ogilvie River	1970
196	Ogilvie River Bridge	1972
198 - 267	Ogilvie River to Eagle Plain	1972
267 - 287	Eagle Plain	1973
554 - 610	Ft. McPherson to Arctic Red River	1973
675 - 736	MacKenzie Hwy. Jct. to Inuvik	1975
287 - 382	Eagle Plain to Eagle River	1976

Figure 1a Completion dates of the Dempster Hwy. contracts

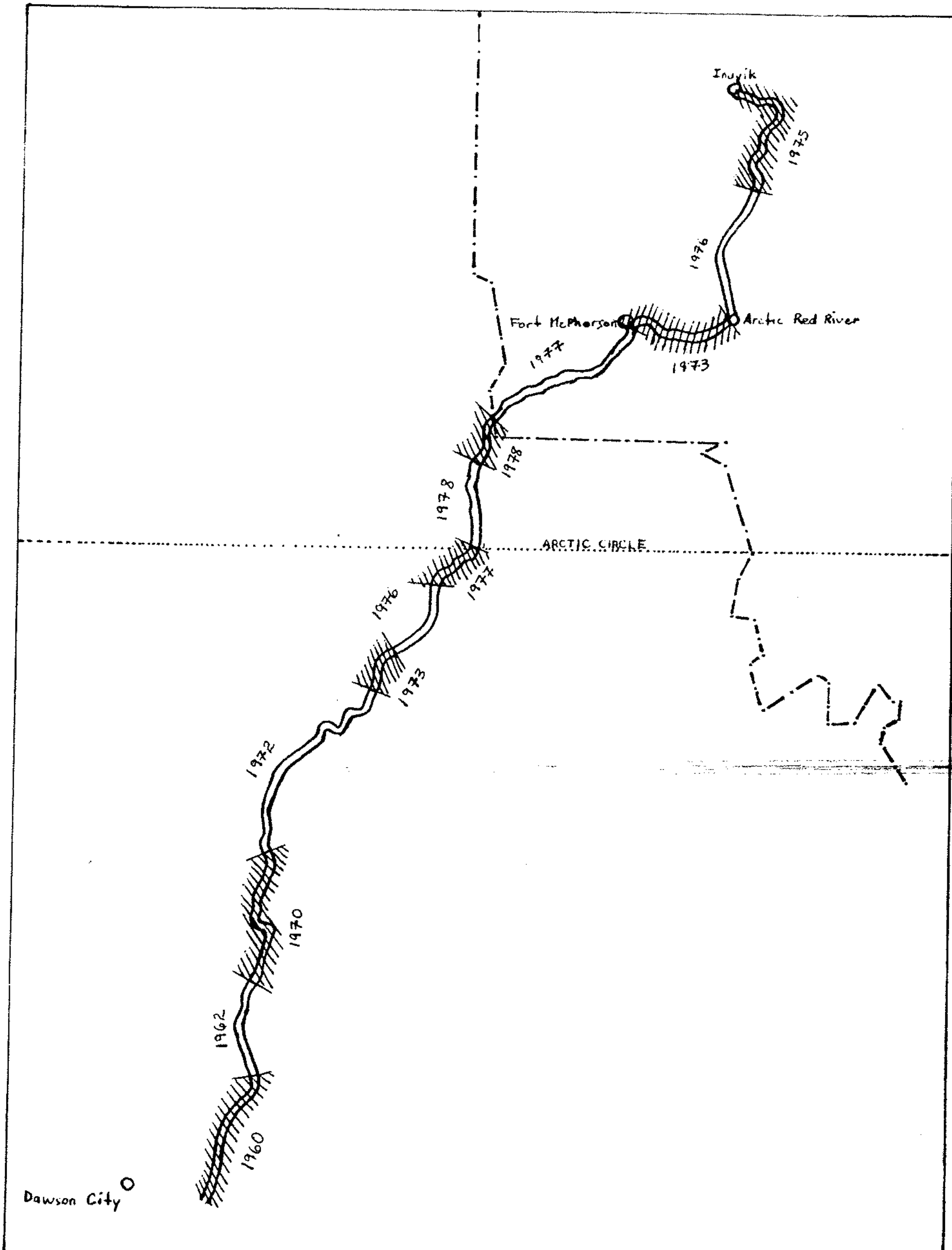
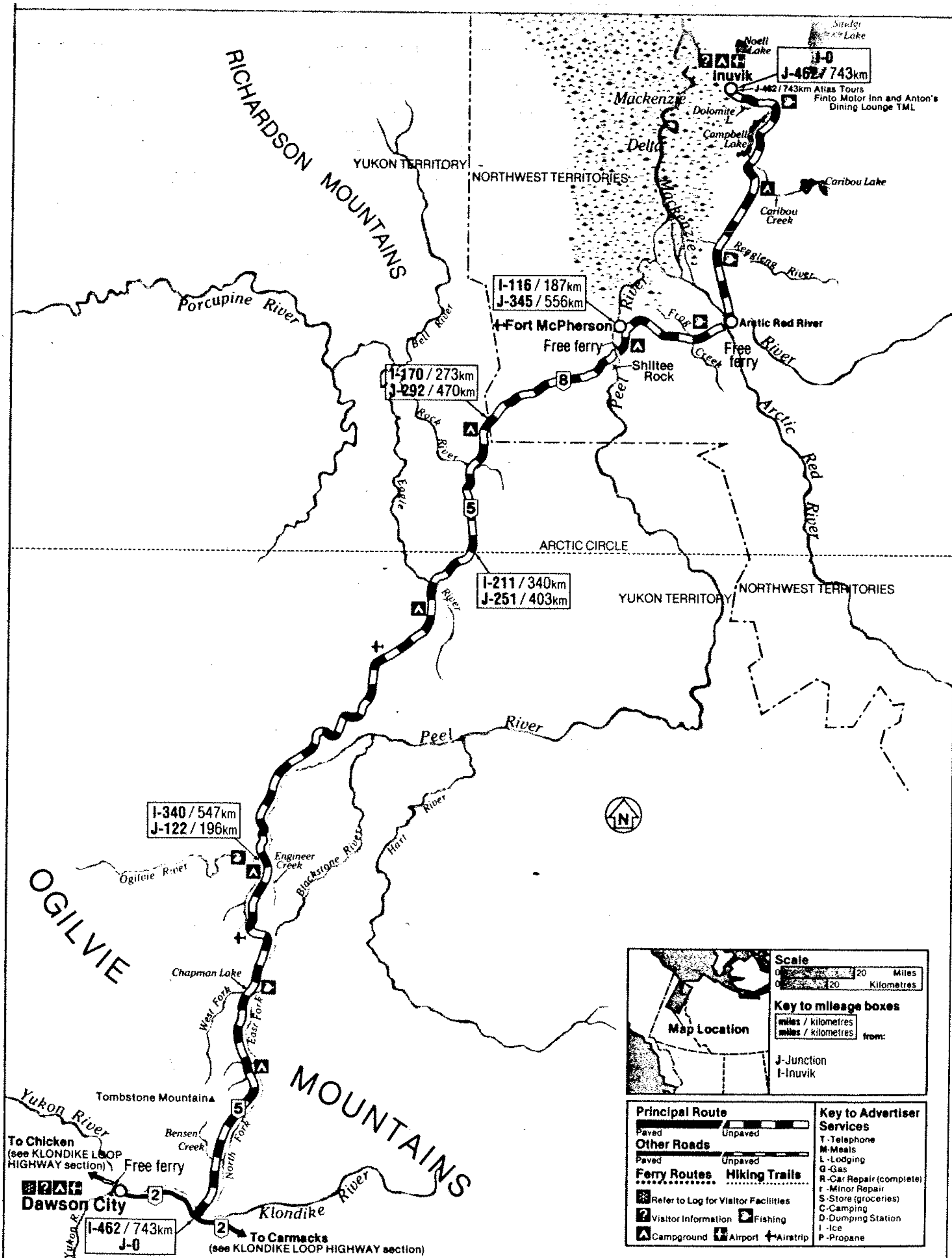


Figure 1b Road map of the Dempster Highway.



LOCATION (km)	DESCRIPTION	YEAR COMPLETED
610 - 675	Arctic Red River to MacKenzie Hwy. Junction	1976
373	Eagle River Bridge	1977
382 - 409	Eagle River North	1977
467 - 554	YT/NWT border to Ft. McPherson	1977
409 - 431	To Rock River	1978
431 - 467	Rock River to YT/NWT border	1978

* North Fork of Klondike River

"Construction contracting has been by public tender with the exception of the two bridges which were built by the Department of National Defence. On completion of a contract, the section of road is handed over to the Territorial Government for maintenance under the Engineering Services Agreement." (Interim Management Plan, 1978)

A program of upgrading along the lower portion of the highway is currently underway. (ie. relocation, widening, more robust design). Highway engineers from Public Works Canada (PWC), Whitehorse have indicated that kilometers 80 to 126 will be serviced this year while kilometers 40 to 80 are to be serviced next year. In addition, the original Bailey Bridges are being replaced this summer.

The original highway (Mile 0 to 78) had a top width of 18 feet (5.49 metres) but reconstruction has widened it to as much as 26 feet (7.95 metres). The present highway comprises a ripped or blasted bedrock pad directly overlying the permafrost and is in turn surfaced with local gravels, where available, or crushed bedrock.

The highway has its origin at the junction with the Klondike Highway, 40 kilometers east of Dawson City, Yukon, and its terminus is Inuvik, N.W.T. - a distance of 743 kilometers.

III BACKGROUND

Most roads in Northern Canada (ie. Yukon, N.W.T., Northern B.C., and Manitoba) have been constructed within the discontinuous permafrost zone wherein different design and construction approaches are more appropriate than in areas of continuous permafrost. The Dempster and MacKenzie highways are examples of the latter case. Difficulties with construction in discontinous permafrost can be mitigated or avoided entirely simply by pre-thawing the ground, or by relocating the highway alignment to unfrozen terrain; however, with the more substantial frozen ground thicknesses of continuous permafrost, these approaches are not generally feasible.

The factors influencing the location, design and construction of the highway in continuous permafrost include: topography, soil and rock conditions, permafrost conditions (extent, thickness, temperature), drainage, economics, construction scheduling, environmental impact and availability of construction materials.

There are two design concepts currently in use: (1) the Active design approach, whereby sufficient time is allowed for the permafrost to degrade before construction commences; and (2) the Passive design approach in which the permafrost is preserved. The Dempster Highway embodies the latter approach with the provision that thawing and compression of the active layer of permafrost would occur within the first few summers following embankment construction.

The following aspects of the Dempster study area were examined in the process of design and construction execution: topography;

drainage; physiography; geology; vegetation; permafrost; and glacial history.

A <u>Topography</u>

Topographically, the area of the Dempster Highway is characterized primarily by the Tintina Trench at the south, by the Ogilvie and Richardson Mountain ranges (with the Eagle Plain between the two), and by the MacKenzie Lowlands at the north end of the highway. (Figure 2)

B <u>Drainage</u>

As reported in a Corridor Planning project (1982), two main drainage systems comprise the region of the Dempster Highway corridor: the Yukon system flowing to the west, as part of the Pacific watershed, and the Peel and MacKenzie Rivers flowing north, as part of the Arctic System. (Figure 3)

Initially, the highway follows the North Klondike River (Yukon System), then follows the Blackstone River, Big Creek and Ogilvie River (Peel System), then climbs from the Peel System to the Eagle Plain, where drainage is toward the Porcupine River. As the highway passes through the Richardson Mountains, the drainage is again towards the Peel River. The final section of the highway from Ft. McPherson to Inuvik is entirely within the Arctic System, which follows the MacKenzie River.

The drainage systems were a major consideration in the location of the highway. With the exception of the Eagle Plain and the pass through the Richardson Mountains, where the alignment follows ridges to avoid watercourses, the highway generally follows the major watercourses.

FIGURE 2

PHYSIOGRAPHIC DIVISIONS OF NORTH CENTRAL YUKON TERRITORY

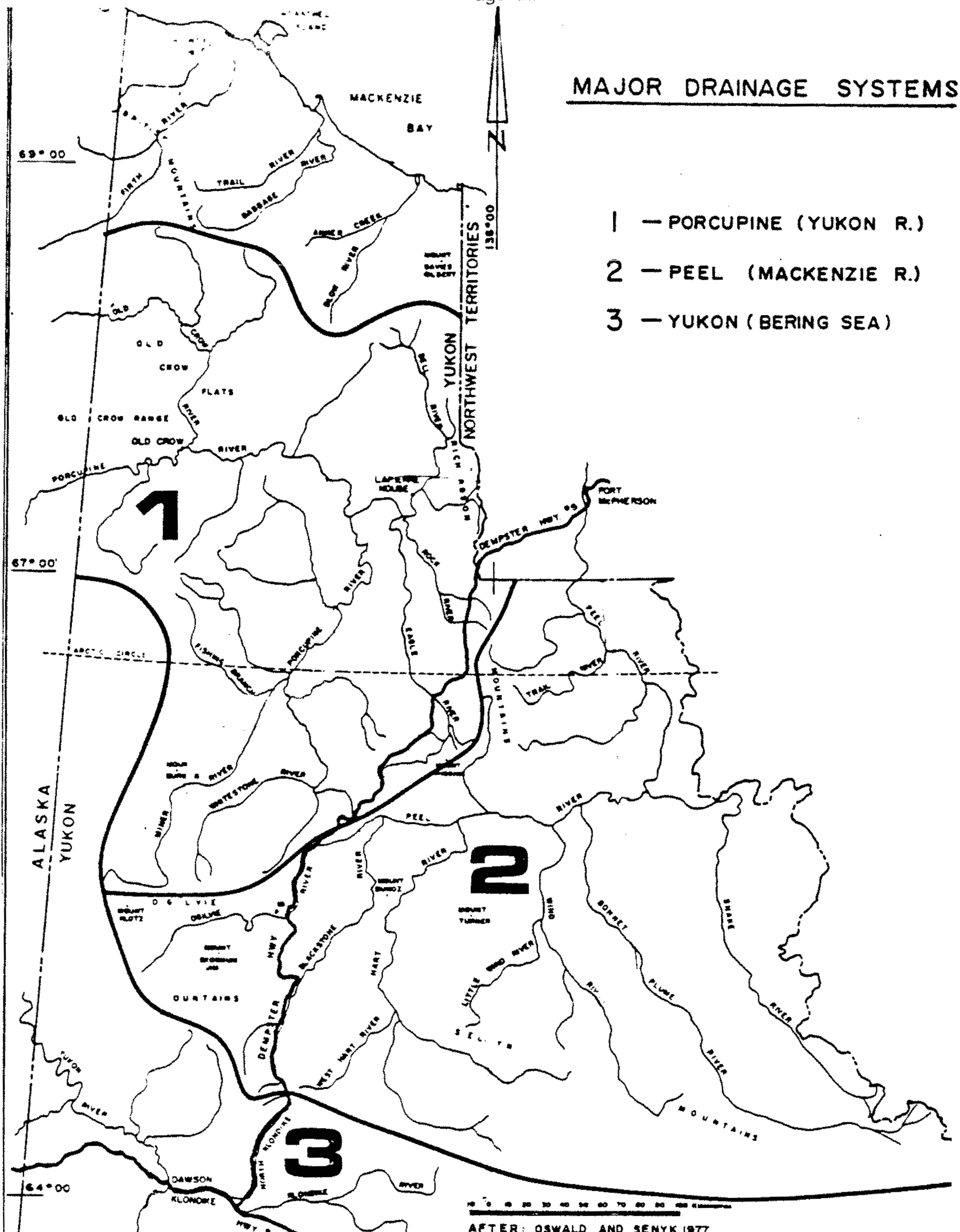


Figure 3 Major drainage systems of the Yukon Territories.

C Physiography

The terrain of the Dempster Highway may be partitioned as follows, according to the physiographic divisions depicted in Bostock's interpretation (Figure 2):

Section 1	Flood plain deposits of Tintina Trench and the river valley of the Ogilvie Mountains
Section 2	Barren alpine tundra and glacial features in the open Taiga Valley
Section 3	River valley in central Ogilvie Mountains
Section 4	River terraces in the open Peel Valley
Section 5	Unglaciated elevated plain (Eagle Plain) of the Porcupine plateau characterized by broad valleys with colluvial slopes rising gently to meet steeper valley walls
Section 6	Barren alpine tundra on western slopes of Richardson Mountains
Section 7	Richardson Mountains (barren)
Section 8	Barren alpine tundra in eastern slopes of Richardson Mountains
Section 9	Lake and swampland covering the Peel plain moraines

D Geology

The geology of the Dempster Highway corridor was examined in a report by Schultz International Ltd., (1972) and is summarized below.

The highway begins in the Tintina Trench, which separates the southern Ogilvie Mountains from the Klondike Plateau. "The floor of the Trench is occupied by faulted and tilted early Tertiary coal bearing sediments and overlain by terrace and flood plain deposits of the

Klondike River." The southern Ogilvie Mountains consist of Precambrian to Middle Jurassic quartzite, chert and slate with occasional carbonate rock.

Section 2 is completely contained in the Ogilvie Mountains and commences near the Northfork pass, a morainal feature of the last glaciation. The highway crosses the Taiga Valley which is underlain by weak late Devonian slates and re-enters the mountains at Kilometer 128.

Section 3 of the highway begins at Kilometer 128 and follows the Blackstone River to Kilometer 152 where it climbs to the headwaters of Big Creek, crosses the Ogilvie River, then leaves the Mountains at Kilometer 216. Precambrian to Permian rocks generally comprise this section.

The highway enters the wide Peel River Valley in Section 4 and follows the Ogilvie River on flood plain deposits.

Section 5 of the highway begins at Kilometer 248 and follows the plains ridge between the Peel and Porcupine drainage systems. "The Eagle Plain consists largely of Cretaceous sandstones and shales, giving place to Permian, Carboniferous and Devonian rocks in the approaches to the Richardson Mountains." Figures 4 (a) and 4 (b) illustrate one geologic interpretation by Huculak et al, (1978). The plateau is unglaciated, while the River occupies a valley formed by a major melt water channel.

In Section 6 the highway skirts the western side of the Richardson Mountains near a series of foothills known as "whalebacks" which comprise hard, fissile, severly fractured shales.

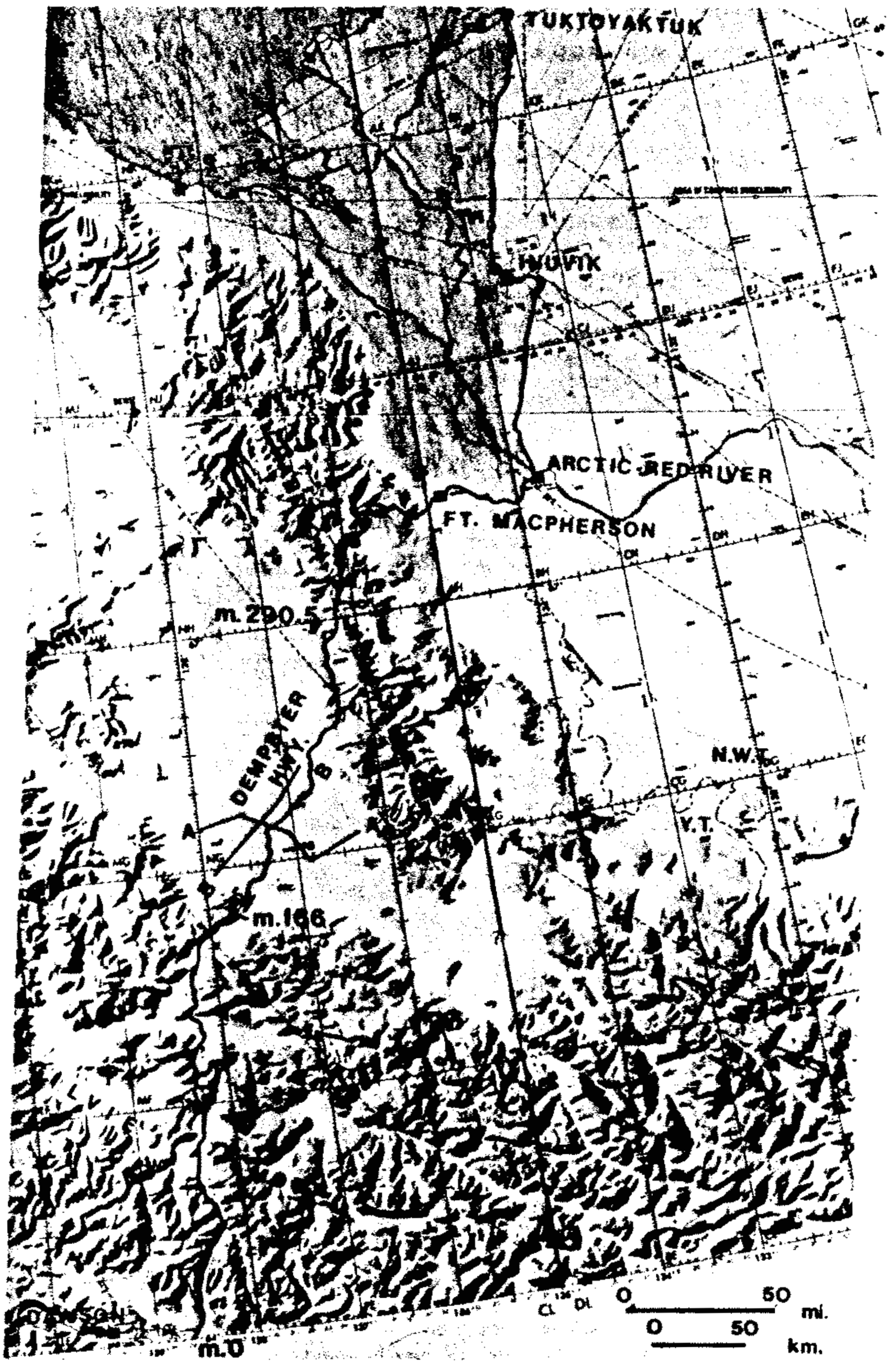


Figure 4a Detailed location of the Dempster Highway and stratigraphic cross-sections.

Section 7 begins at Kilometer 488 near George's Gap.

Section 8 starts in the eastern upland slopes of the Richardson Mountains and ends at the Peel River. The eastern slopes generally comprise Cretaceous Sandstones and shales with well preserved glacial landforms and ridges of resistant sandstone.

The Peel Plateau of Section 9 is underlain by Upper Devonian rocks. Evidence of a broad belt of hummocky moraine is apparent between the Peel and MacKenzie Rivers; this is perhaps where the Laurentide ice sheet halted its northwesterly movement.

Huculak et al, (1978) have pointed out that "Near Inuvik there are outcrops of shales and dolomites, and at Arctic Red River, there are outcroppings of shales and sandstone."

E Vegetation

Vegetative cover along the Dempster Highway varies from spruce forest to tundra plant communities. The southern extent of the highway has spruce forest associations which include willow, aspen, cottonwood and birch. Deciduous trees thrive along the river valleys where flood plain deposits, water availability and shelter from high winds and extreme cold provide a suitable environment. The deciduous trees also occupy higher ground as a reforestation stage following forest fires. Black Spruce communities dominate the wet poor draining soils and are often associated with sphagnum moss and muskeg. White Spruce are common in free draining soils. Black Spruce and White Spruce generally comprise the more northerly forest and survive only in the hollows of the Eagle Plains plateau. Tundra-like plant communities including labrador tea, cotton-tussocks, sedge grass and reindeer moss occupy the alpine tundra of the Richardson Mountains.

F Permafrost

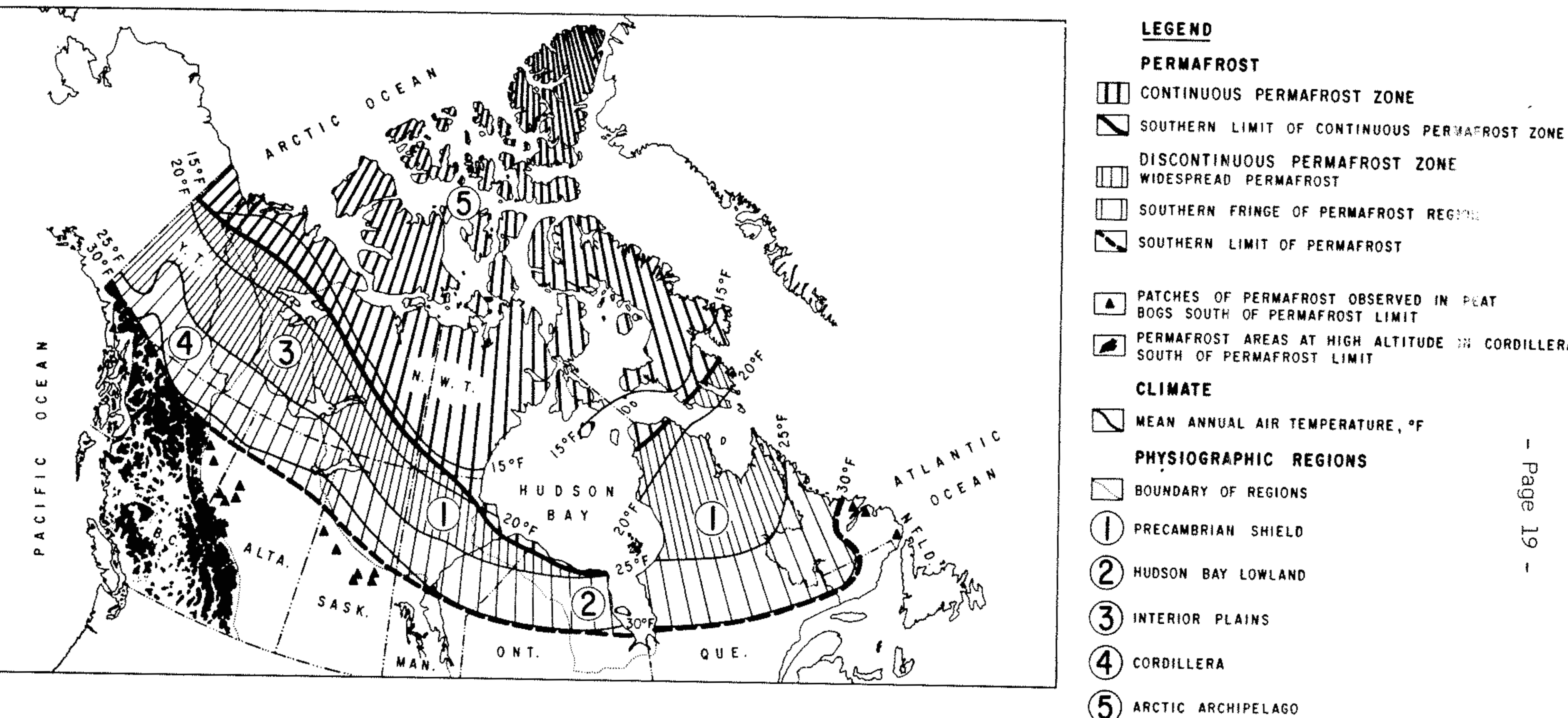
Contrary to interpretations by other researchers (Figure 5), continous permafrost (rather than discontinous) was encountered by local highway engineers, who assert that except for stream bed areas, lakes and ponds, the area occupied by the Dempster Highway may be regarded as continous permafrost. It will be seen that this has a great impact on the design philosophy and construction approach. The thickness of the permafrost along the highway is generally unknown, but is probably up to 100 metres or more thick, particularly in the northern section.

A variety of ice-rich and ice-poor soils have formed as a result of climatic and geologic processes. Peat deposits with water contents varying from 6880 (Kilometer 128) to 220 percent (Kilometer 190) have been recorded in a previous study by Schultz International Ltd., (1972). Ground ice in vein or reticulate form or lenses a metre or more thick are not uncommon in areas such as the Ogilvie Mountains. These and other manifestations of ground ice tend to produce thaw-unstable soils. (See PLATES 27 and 32).

A variety of permafrost features were observed along the highway corridor during the field reconnaissance undertaken in June, 1984:

(1) Beaded Streams; Eagle Plains valleys and MacKenzie River basin

(2) Frost Boils; Eagle Plains



igure 5 Permafrost in Canada (after Brown (1970))

(3) Frost Polygons; Blackstone River and Richardson Mountains

(4) Unsorted stone circles; Eagle Plains

(5) Solifluction strips, lobes, scars; Richardson Mountains

(6) Skin Flows; Ogilvie Mountains

(7) Thermokarst; MacKenzie Basin

(8) Ground Ice in a variety of forms

(9) Peat Plateaus; Tintina Trench

(10) Felsenmeer; Richardson Mountains

(11) Nivation Hollows; Richardson Mountains

In general, the ice rich soils are protected by an insulating mat of surface vegetative cover which restricts seasonal thawing and freezing to depths of 0.6 to 1.0 metres below ground surface. Compression or destruction of the mat allows thaw to penetrate deeper, leading to instability in ice-rich soils.

G Glacial History & Geomorphology

A measure of uncertainty exists about the exact extent of glaciation in the vicinity of the highway. According to Prest's interpretation (1969), nothing west of the Richardson Mountains and north of Dawson City was glaciated, with the exception of a few small isolated areas (See Figure 6). However, Oswald and Senyk present a different interpretation, as shown in Figure 7. The author's experience supports the former case.

The Eagle Plain Plateau is definitely unglaciated, as well as the approaches to the west pediment of the Richardson Mountains,

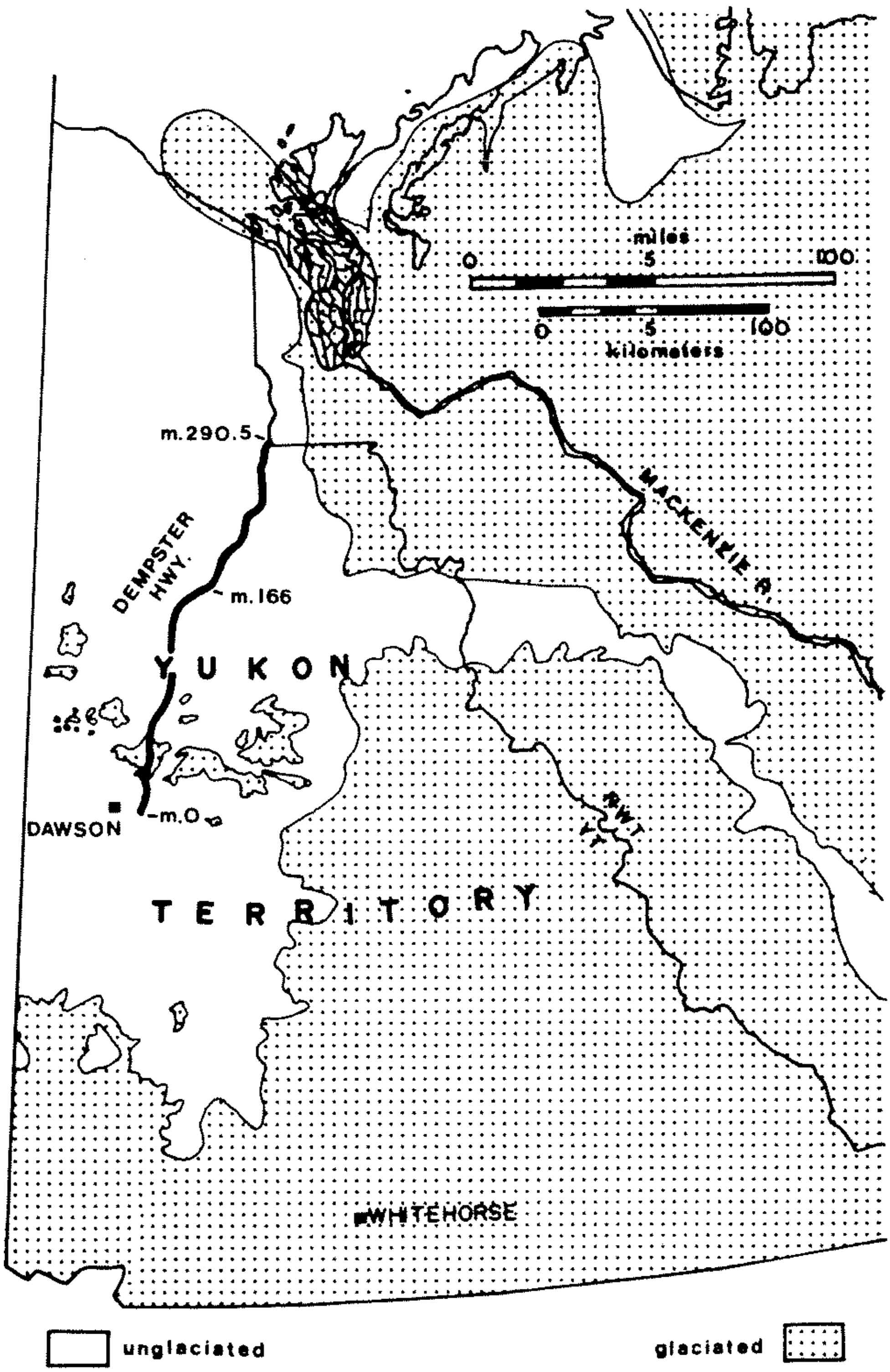


Figure 6 Map showing extent of glaciation (after Prest (1969)).

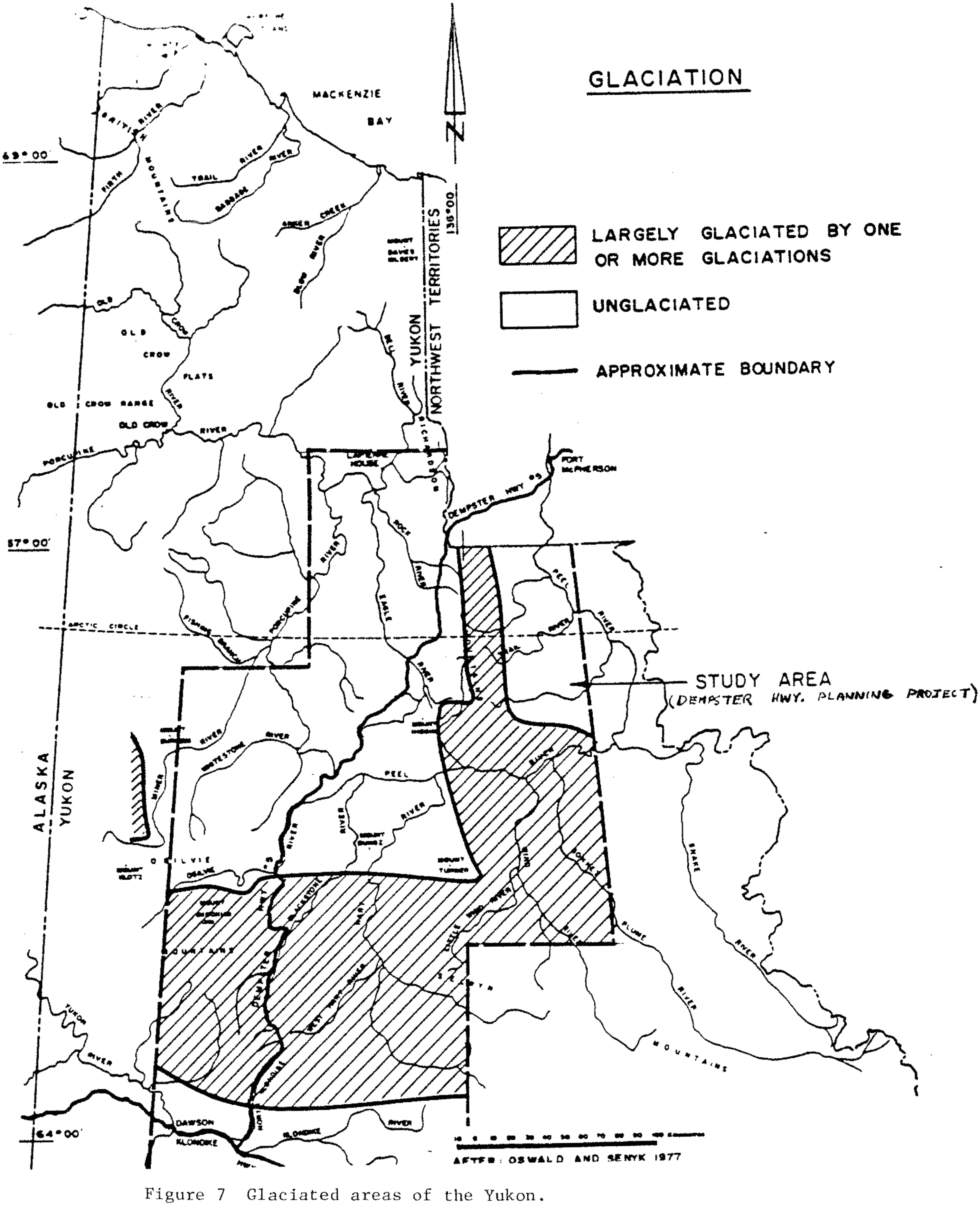


Figure 7 Glaciated areas of the Yukon.

but evidence of glaciation in the Ogilvie Mountains at North Fork Pass (a morainal feature) corroborates Prest's interpretations. The Eagle Plain comprises an uplifted sedimentary basin, which has been dissected by fluvial erosion to create a region of board rounded ridges. The valleys contain relatively wide flood plains occupied by meandering streams. Weathered bedrock has been carried downslope by mass wasting and fluvial action. However, the Eagle River now occupies a valley formed by a major melt-water channel.

Similar processes have altered the faulted, northward plunging anticline which is the Richardson Mountain range.

As previously stated, the Peel Plateau contains a broad belt of hummocky moraine which indicates the limit of westerly ice advance at that latitude.

The rivers of the Ogilvie and Richardson Mountains are young and create slope instability in the active process of channel formation. Except for the MacKenzie and Lower Peel Rivers, none of the rivers have reached any stage of maturity. (Schultz International)

IV PREVIOUS STUDIES

Several studies addressed different aspects of the Dempster Highway and were undertaken before, during and after construction. These studies provide background information related to the highway design, construction and maintenance. To the author's knowledge, only one study was undertaken subsequent to construction for the purpose of evaluating the highway performance; this study examined only the portion within the Northwest Territories: (Cook and Huculak, 1980).

The following are summaries of the relevant portions of studies previously undertaken to examine the Dempster highway.

A AESL Consulting Engineers

In August 1972 Associated Engineering Services Ltd. (AESL) was commissioned to review design features, availability and use of materials and construction techniques employed on the Dempster Highway from Fort McPherson to Arctic Red River and on the MacKenzie Highway from Inuvik south to milepost 931. As this was a lengthy report, only the terms of reference and conclusions are given.

The terms of reference were as follows:

(1) To determine through an analysis of the design concept and construction practices employed, on the sections of road under construction between Ft. McPherson and Inuvik, whether the native fine-grained materials can be used successfully for the construction of a stable road grade in permafrost.

(2) To determine if the present concept of construction practice and material are required to produce a satisfactory highway.

(3) To recommend design concepts and construction practices that should be adopted on these and other similar projects.

(4) To provide construction cost comparisons for original concept, present concept and any recommended changes.

CONCLUSIONS

(1) Native fine-grained soils may not be used for complete all-weather road embankments. They can be used for the internal portion or core of the embankment provided that competent materials, such as gravel, shale, etc., are used to provide load bearing strength and side slope erosion protection.

(2) The design philosophy and material selection was adequate at the time but modifications are suggested.

(3) No fine-grained materials may be used in high embankments; only coarse-grained materials such as shales or broken rock are to be used.

(4) Materials selection criteria: availability, construction schedule; alternative design feasibility, consequence of road failure.

(5) No blanket statement can be made about the cost advantage of one embankment design over another when the only difference is the material type used in design.

(6) No embankment failures; no need to abandon any section; Big Cut at Mile 343.2 may be abandoned for other reasons.

B Richardson, N.W.; 1974 (Km 266 - 465)

Introduction

The object of a study initiated in 1972 by Neil W. Richardson and E Karl Sauer, was to establish a rational basis for field testing and subsurface exploration programs for highway development in the unglaciated sedimentary areas of the Arctic. Terrain classification of Mile 166 to Mile 290.5 (Km 265.6 to 464.8) of the Dempster Highway was carried out on the basis of geology, vegetation, topography and patterned ground.

Physical characteristics and vegetation

The study area occupies the terrain north of the Ogilvie Mountains, including the Eagle Plain plateau and the approaches to the Richardson Mountains up to the Yukon/N.W.T. border (Mile 290). The Eagle Plain plateau consists of shales, sandstones and siltstones of middle Cambrian to Tertiary age, while the Richardson Mountains comprise shales, sandstones and limestone.

Vegetation is grouped as one of 5 plant associations for ease of mapping. The first and most abundant association is the Black Spruce - Sphagnum association which commonly occupies frozen, fine-grained sediments in lower altitudes and includes tamarack, alders, willows, dwarf birch, Arctic Labrador tea, lichens, and Arctic cottongrass.

The second most proliferous association, cottongrass tussock tundra, thrives on frozen, poorly drained, fine-grained sediments at higher elevations.

The lichen association is found on the sandstone outcrops and other coarse grained deposits of the Eagle plains and includes dwarf birch, shrubs, bearberry, and cranberry bushes.

The birch association consisting of paper birch, alder, dwarf birch, and willow, occupies the wet areas along drainage courses.

The flood plain association is restricted to the Eagle River Valley and includes willows, alders, balsam poplar, and White Spruce (in order of increasing distances from the stream).

Terrain Units and Associated Construction Difficulties

The terrain from Mile 166 to Mile 176 comprises ridge and cliff-forming sandstone strata interbedded with thicker highly weathered shale beds. Beyond Mile 176 the sandstone appears to be more susceptible to weathering and layers of thaw unstable silty clay commonly exist beneath thick vegetative cover. The clay overburden was difficult to remove from borrow pits during wet conditions and tended to become mixed with the sandstone subgrade, hence severely reducing the shear strength of the embankment. Surficial shale deposits were usually ice-rich and thaw-unstable.

Mile 215 to Mile 217, the Lower Cretaceous interval, consists of shale similar to the Eagle Plain Formation with lower erosion resistance which might be inferred from the subdued topographic relief.

Mile 217 to Mile 220.5, the Ettrain Formation consists of erosion-resistant limestone, the upper 1½ metres of which is ice-rich felsenmere. The quality of borrow material may be inferred from the steep sharply defined ridges.

The Hart River Formation, Mile 220.5 to Mile 230 consists of thaw unstable shales of low erosion resistance; at the surface, at least. Smooth rounded hills, landslides, slumps and flow features on aerial photographs are indicative of the material quality.

The Western outcrop interval, Mile 225 - 240, consists of interbedded silty shale and sandstone. The sandstone is a suitable borrow material, but the shale should be left undisturbed.

The Eastern outcrop interval, Mile 240 - 257, is predominatly sandstone and siltstone with minor shale beds.

The Canol Formation, Mile 257 to Mile 271, consists of dark grey to black shale.

Mile 271 to Mile 254 comprises shale and siltstone strata with near vertical dips and occasional alluvial overburden. Borrow should be secured from lithified shale and sandstone deposits. The major problems anticipated are drainage and disturbance of vegetation.

Mile 254 to 290.5 comprises four geologic units of shale sandstone. The principal source of borrow is the highly resistant ridge sandstones and shales of the hogback ridges.

Conclusions

The engineering significance of the terrain units mentioned is readily apparent. The steep, sharp, erosion resistant ridges encountered along the corridor indicate a suitable erosion resistant borrow. Patterned ground alerts the observer to the perils of ground ice, which is manifested in various forms. Vegetative cover type and thickness provides clues to the extent of ice and frozen ground.

Since gravels are generally absent in the study area, it is necessary to excavate bedrock. The sandstone is generally the best borrow, but varies in quality and ice content (often high ice content near surface), while the shales and siltstones are generally poor quality borrow due to thaw instability (high ice content) and inferior durability.

The type of vegetation and drainage seen in air photos are important indicators of potential problems that may occur regarding subsurface drainage disruption, icings, slope stability, creep, and potential subsidence.

C Huculak, N.A., and others; 1978 (Km 616 - 743)

Introduction

The authors of this paper (N.A. Huculak, J.W. Twach, R.S. Thomson, R.D. Cook) were intimately involved in the design and construction of the Dempster Highway. The report covers the design criteria, pre-engineering activities, construction techniques and road performance, up to the highway completion in 1978.

Geotechnical Investigation

Airphoto interpretation was initially used to define the terrain and potential borrow sites. When the obvious borrow sites (glacial-fluvial deposits, bedrock exposures, kames and till ridges) were proved unsuitable by test drillings, field investigations concentrated on locating bedrock with shallow overburden. Borrow was limited to areas of 3 metres of overburden and spaced at regular intervals with a maximum haul distance of 10 miles.

"Drilling programs generally consisted of 5 to 6 holes on centerline per mile plus an average of 10 holes per mile during borrow search. Exploration was carried out in winter to minimize damage caused by using tracked vehicles, mobile camps and helicopter support. Almost all samples were disturbed "grab" samples."

Embankment Design and Specifications

The highway was constructed to an "all-weather" unpaved surface, subject to spring thaw and fall freeze-up periods at the ferry crossings at the Peel, MacKenzie and Arctic Red Rivers. The subgrade design width varied between 24 and 28 feet. Except where gradients exceeded 6%, the highway design speed was 50 to 60 mph (80 to 95 km/hr). Side slopes were designed at 3 to 1 slopes and desired curvature was 5 degrees with a maximum allowable of 15 degrees.

The design philosophy was to preserve the permafrost to a tolerable degree of grade distortion; therefore, it was desireable to provide uninterrupted cross-drainage during the embankment settlement (caused by thermal degradation) and to locate structurally competent, thaw stable materials which could be used for both embankment and surfacing.

The initial design concept/objective was to establish the minimum depth of fill which would prevent advance of thaw into ice-rich inorganic solid, (ie. to limit the depth of thaw to the original ground surface or within the active layer). "Assuming winter construction, the saturated surface organic layer when frozen would act as a heat sink, however, once thawed under a fill, compression of the peat and organic material would occur, reducing the natural insulating properties, and allowing the thaw plane to migrate deeper in succeeding summers." (Huculak, et al, 1978)

Progressive thaw and subsidence of the side slopes were considered to be the major drawbacks of the embankment. However, it was considered that this strip along each side of the highway would self heal in three to four years with minimal slumping. Only if ice-rich mineral soil extended across the entire road width, would major surface distortion occur and repair be necessary. Based on previous experience with permafrost and theoretical predictions, a minimal fill height of 4.5 feet (1.4 m) was necessary.

Highway location and drainage

Because of the abundance of ice-rich, fine-grained thaw unstable surficial soils, it was important to avoid areas requiring excavation. Low lying flat ground was the preferred terrain to accommodate geometrics and the overlay construction method. It was necessary to blanket cuts in thaw unstable materials with quarried bedrock.

Special attention was given to the provision of reliable cross drainage in regions of ice-rich fine-grained soils, particularly to avoid runoff along the thermally disturbed toe of the embankment. It was necessary to overcome several problems:

- absence of historical hydrological data
- poorly defined drainage basins
- thaw unstable subgrade, that would settle random amounts after culvert placement
- rapid, short term spring runoff over the frozen tundra
- tendency for complete ice filling of small culverts prior to spring runoff

The solution was as follows:

- 30 inch (77 cm) minimum culvert diameter
- flexible corrugated metal culverts
- culvert inverts located at or slightly above entrance ground elevation
- attempt to err on the high side in terms of culvert size
- smaller relief pipes or steam pipes were installed at locations requiring pipes larger than 60 inches diameter

Construction Technique

Due to the high ice and silt content of surficial soils, excavated shale and sandstone bedrock was the preferred embankment material. It was quarried by drill and blast or "ripping" methods, and dumped and compacted by tracking of the prime movers on the embankment. Compaction equipment was used on the final grade to break down larger fractions and provide a tighter surface.

All contractors chose to work year round with a two month break between mid-December and mid-February. Production and performance was lower during winter, but equipment mobility was improved over the frozen tundra. Removal of waste (ie. ice-rich materials) and drain installation was also easier during winter. Prudent contractors scheduled their operations and used the navigation season of the MacKenzie River to obtain the numbers and types of equipment necessary in advance.

Surfacing was delayed 2 to 3 years to allow for subsidence and warping of the embankment due to thermal degradation. Lack of granular deposits in the delta area necessitated the use of quarried limestone for surfacing. Addition of fine-grained materials as a binder was necessary for poorly graded granular materials.

Road Performance

Embankment settlement and distortion varied between 10 and 100 cm, being predictably higher in ice-rich terrain. Sections built during the winter distorted more than those constructed during the summer as a result of embankment spreading due to displacement of the soft, saturated in-situ ground when thawing occurred.

D Public Works Canada; 1980 (Km 615 - 743)

Introduction

In 1980 an evaluation of design and construction methods on long term embankment performance was undertaken by Cook and Huculak. Mile 892 to 972 of the MacKenzie Dempster Highway was evaluated in terms of: embankment height; materials, subgrade soils, orientation and trafficability; water and drainage measures; construction season; and culvert performance.

Observations

The minimum fill height (1.4 m) was chosen on the basis of theoretical calculations and observations at Inuvik Airport regarding thickness required to preserve the permafrost. In practice, 1.4 metres of fill was not sufficient to limit thaw to original ground height, but embankment serviceability was still good. Sections constructed in stages with an initial metre thick pad performed better than those with only 0.6 metre thick initial pad. The worst settlement occurred from Kilometer 719 to 925 (Mile 956 - 960); over 1.6 metres of fill was placed, but the surface remained only 0.6 metres above adjacent terrain. It is more economical to recondition (fill depressions, scarify, restore design cross-section) a lower embankment than build a higher one initially to reduce post-construction maintenance. Subsidence of the toe of the embankment and slumping of the shoulders are likely occurrences, but will be stabilized over a period of 3 - 4 years by regular maintenance.

Attempts were made to use fine-grained embankments in 1971 - 72 but this practice was abondoned in favour of bedrock materials (shale, sandstone, limestone). Embankment performance if generally not influenced as much by construction materials as other factors.

The amount of embankment settlement was function of thaw depth, lithology of subgrade soils, moisture content, and thickness of the peat or organic cover. Highway performance was better where old burn areas were crossed, perhaps because of reduction in ice content from loss of insulation or because burn areas are usually worse on higher, better drained terrain. Performance of the highway was generally better over rolling or high terrain not because of better soils but because of better drainage.

Thawing of south facing slopes was usually greater than north facing slopes but slumping was not noticeably worse.

Settlement and embankment distortion were usually greatest where water was allowed to pond adjacent to the embankment.

Summer stages construction produced a stable embankment as much as 2 years earlier than winter stages construction but required more fill initially. There is no long term advantage to either method; drainage and embankment height are more important.

Conclusions

In 1980, 5 - 8 years after construction, 90 percent of the highway examined can be driven comfortably at design speed (90 km/hr). The remainder requires a speed reduction of 15 - 25 km/hr. However, the highway has had very little heavy truck traffic until 1979. Heavy truck traffic is not recommended in the initial 2 - 3 years after construction because more extensive maintenance will be required.

A design height of 1.4 metres is not sufficient for the passive design intended. However, it is not economical to use a higher initial design height; a lower initial design height should be considered except when crossing wet, poorly drained terrain or where there will be heavy truck traffic in the initial 2 - 3 years following construction.

The season of construction may be important in the initial 2 - 3 years after construction, but not in the long term (5 - 7 years).

Embankments less than 1.6 metres high performed poorly in poorly drained, wet terrain. The performance does not appear to be related to the amount of cross-drainage provided (culverts), but rather to embankment settlement into the vegetative mat and subgrade soils creating a depression.

Thaw was greater on south facing slopes, but slumping was not correspondingly greater.

Advanced thaw degradation occurred on the upslope toe of embankments located on general cross-slopes. There was slight increase in slumping, settlement and distortion of the upslope shoulder.

V CURRENT STUDY

A Design Criteria

General

Design and construction of roadways, railways and airfields in permafrost areas are similar in many ways, but the construction problems encountered will differ depending on the intended use of the structure. Accordingly, design standards vary to accommodate the level of service required and capital/maintenance cost and trade-offs are made.

The seven site attributes discussed in Section III (Background) may be summarized in three criteria governing site selection: (1) terrain conditions; (2) availability of construction materials; and (3) environmental impact. Three potential routes were examined before the present alignment of the Dempster Highway was chosen. A critical evaluation of specific aspects of the route selection process is beyond the scope of this report since extensive geotechnical, geometric and aerial photographic data is required. However, in retrospect, the chosen alignment appears to be quite satisfactory from economic and engineering viewpoints.

Three conditions are commonly considered in the design of structures in permafrost: (1) frost action; (2) drainage; and (3) ground thermal regime (Johnston, 1981). The majority of embankment material used in the Dempster Highway were non-frost susceptible. Major ancilliary structures such as bridge piers, abutments and culverts were designed to minimize the effects of frost jacking and thermal degradation.

Proper drainage of highway structures is essential since ponded water acts as a "heat sink" leading to thermal degradation of permafrost and possible subsidence. Liberal use of culverts in the Dempster Highway was made to avoid interrupting natural drainage. Ditch blocks, rip-rap and river training structures were used to reduce potential erosion (Hudson, 1984). The underlying design principle is to preserve the permafrost and thereby reduce the possiblity of thermally induced subsidence (Thomson, 1984).

Embankments

The road surface width varied between 7 and 8 metres depending upon height of fill, sharpness of horizontal curves, and other factors. The minimum embankment height specified for newer embankments (1970 and newer) was 1.4 metres; sideslopes varied between 3:1 (cut/fill under 1.5 m) to 1½:1 (fill over 3 m high). Cut slopes in rock were specified as ¼ :1.

Initial specifications were intended to provide for the use of fine-grained materials within embankments, but these materials were suitable in only a few cases and locations. Quarried rock was then required where sands and gravels were not available.

Construction seasons and scheduling was generally left to the discretion of the contractors with restrictions applied to travel over the tundra, especially in summer. (AESL, 1972)

Culverts

Culvert diameters range in size from 0.76 m to 4 m. The large minimum size was intended to obviate potential icings problems and section reduction due to embankment deformations. Culverts were used liberally in low, flat topography or where natural drainage was indistinct. Culvert types included corrugated steel pipe (CSP) for smaller size applications and corrugated structural (steel) plate pipe (CSPP) for larger applications. Different shapes (round, elliptical arch) were used to take advantage of flow characteristics to mitigate icings and increase efficiency. A reverse (positive) camber was placed on the long dimension of culverts to counter the effects of potential sagging from embankment subsidence at counterline and to prevent ponding of water in culverts (Hudson, 1984). Culvert inlets were also elevated above ditch in anticipation of subsidence. Gravel was the recommended bedding material and rip-rap was required for erosion protection. Materials with potential for thaw consolidation were not permitted adjacent to culvert installations after trial use of fine-grained materials proved unsatisfactory for backfill.

B Qualitative and Quantitative Observations

A field investigation was undertaken by the author in June, 1984 during which both quantitative and qualitative observations of the performance of the Dempster Highway were made. These observations included transit and level surveys, drawings, photographs and detailed notes.

Annotated photographs are presented in the order of increasing latitude with reference to the following aspects of highway performance: (1) cut/fill sloughing; (2) permafrost degradation near fills, cuts, bridges, culverts, borrow pits and ditches; (3) road heave; (4) road subsidence; (5) drainage disruption; and (6) road trafficability.

One objective of the investigation was to obtain a profile of thaw settlement of highway cross-sections from existing records, supplement it with "as-built" geometry and up-date it with level surveys taken in the course of the field work. This endeavor was not fruitful for several reasons. The selected sections from as-built drawings could not be located in the field because the landmarks or original survey markers had weathered badly or were not visible. The local highway engineers affirmed that in the course of maintenance, the surface elevations may have been either raised or lowered, making subsidence indeterminate. A test section north of the Eagle River Bridge is being monitored currently, but pertinant data is not yet available.

The following photographs are typical examples of the worst conditions encountered along the highway. This does not imply that the entire highway is similar; quite the contrary, the highway is in excellent condition considering the type of terrain it traverses.

PLATE 1: Reconstructed embankment, about 5m. high. Original culvert is slightly below ditch level and contains remnant aufeis (Kilometer 6).

PLATE 2: Spruce forest/sphagnum moss plant association. "Broken back" culvert in a deep fill (centre). Remnant aufeis and icing area visible (right).

(Kilometer 29)

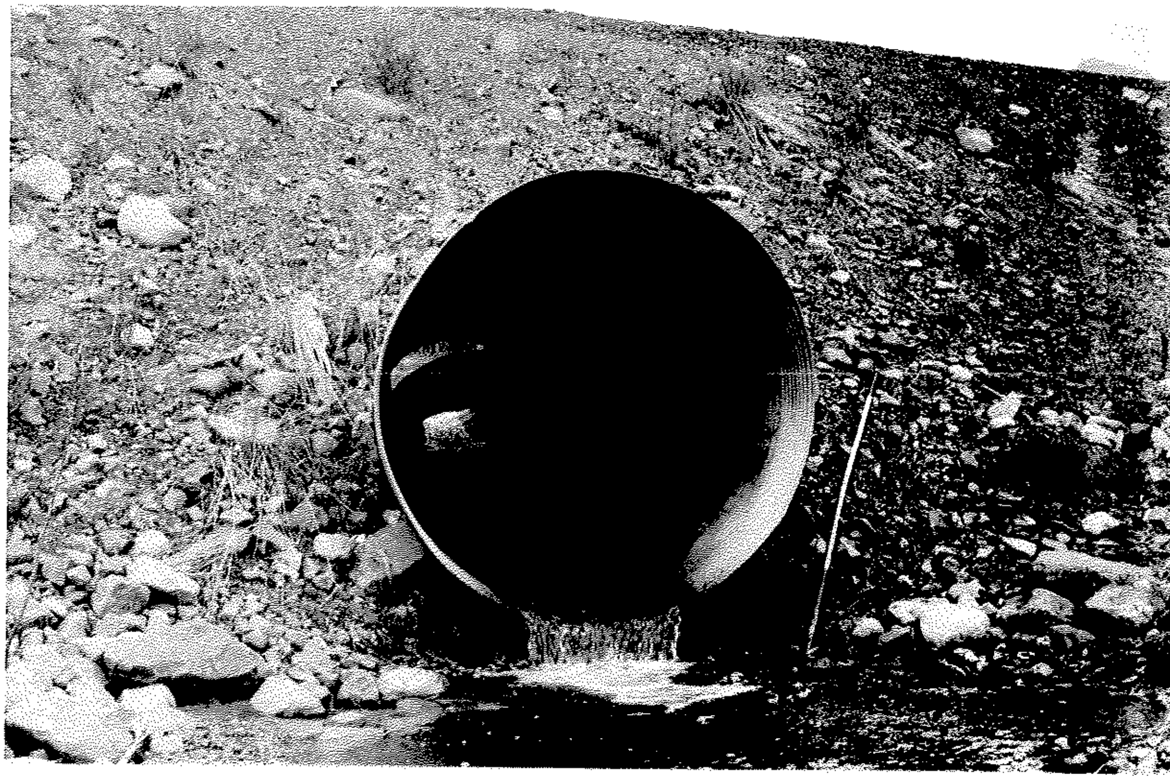


PLATE 3: Culvert uncoupled same culvert as in PLATE22. Culvert diameter is 2 m.; installed about 1960.

PLATE 4: Overdeck Bailey bridge at Blackstone River. Remnant aufeis (right) is almost at truss level.
(Km. 78)

PLATE 5: Highway reconstruction underway near Km. 81 in alpine "tundra" of the Ogilvie Mountains (looking north). Remnant aufeis is visible at the left.

PLATE 6: Backslope cut at 28 degrees to the horizontal in silty sand. See Figure 14 for cross-section. (Km. 87)

PLATE 7: Looking north from Km. 122 (Built 1962). Surficial soils consist of 0.1 - 0.5 m. of saturated organics overlying ice rich silts or sands.

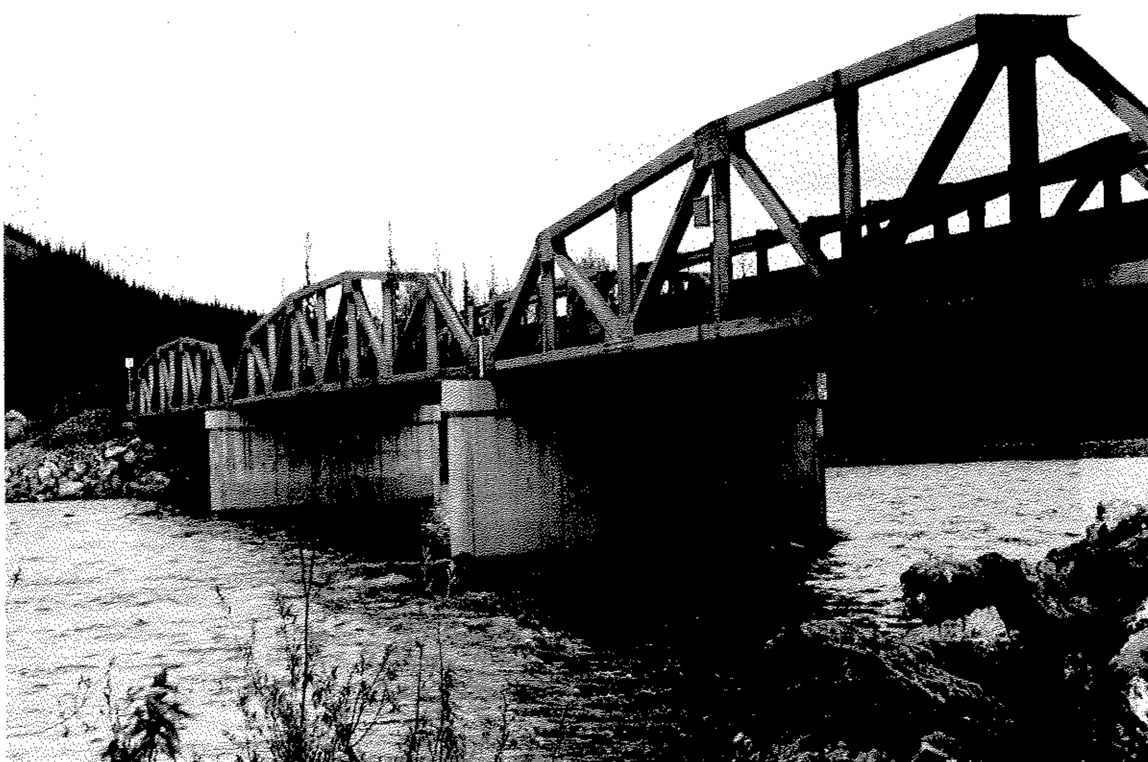


PLATE 8: The Jeckell bridge over the Ogilvie River Bridge comprises 110 m. triple span.
(Km. 196)

PLATE 9: Sideslope 16 m. high fill at 34 degrees to the horizontal. (Built 1976). Culvert in foreground collapsed. "Fluming" is intact (foreground).

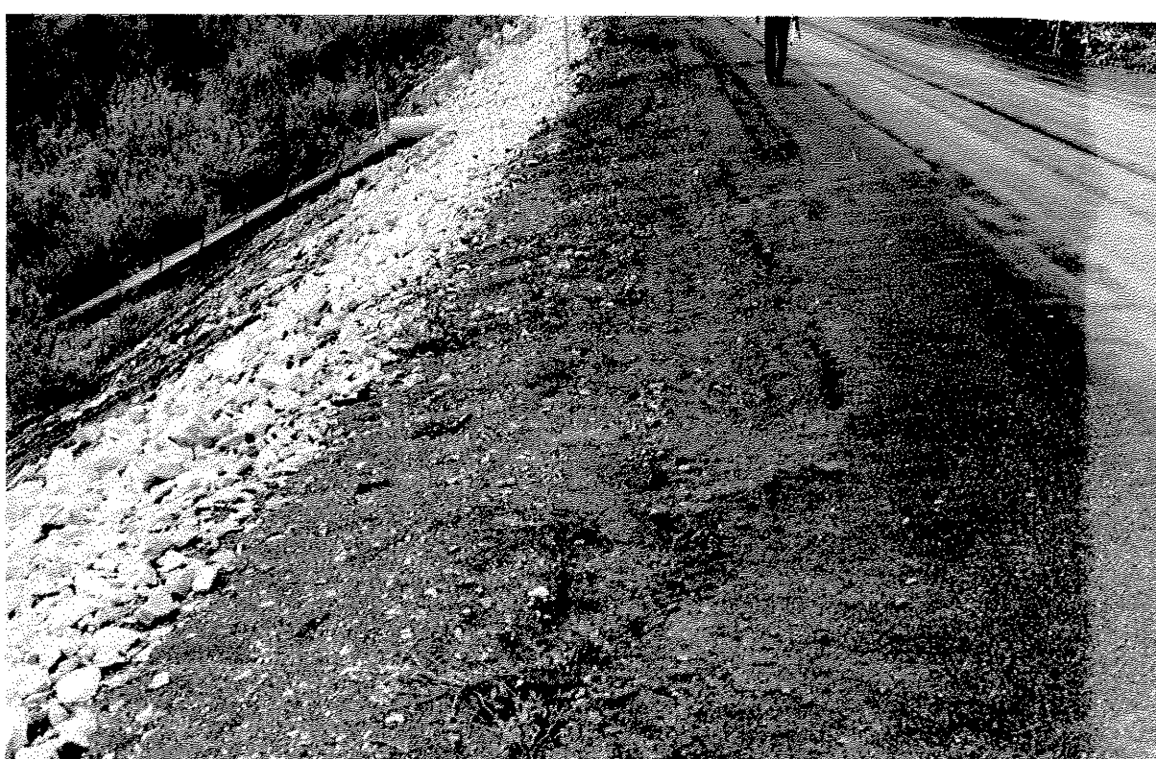


PLATE 10: Close -up view of PLATE 9 showing slumping shoulder. (Km. 352)

PLATE 11: Cut/fill transition at Km. 354. Limestone breccia in rock cut.

PLATE 12: Thaw of polygonal ground ice leaves water filled depressions 2 Km. north of Eagle Plains Lodge. Road alignment follows ridge crests (background).

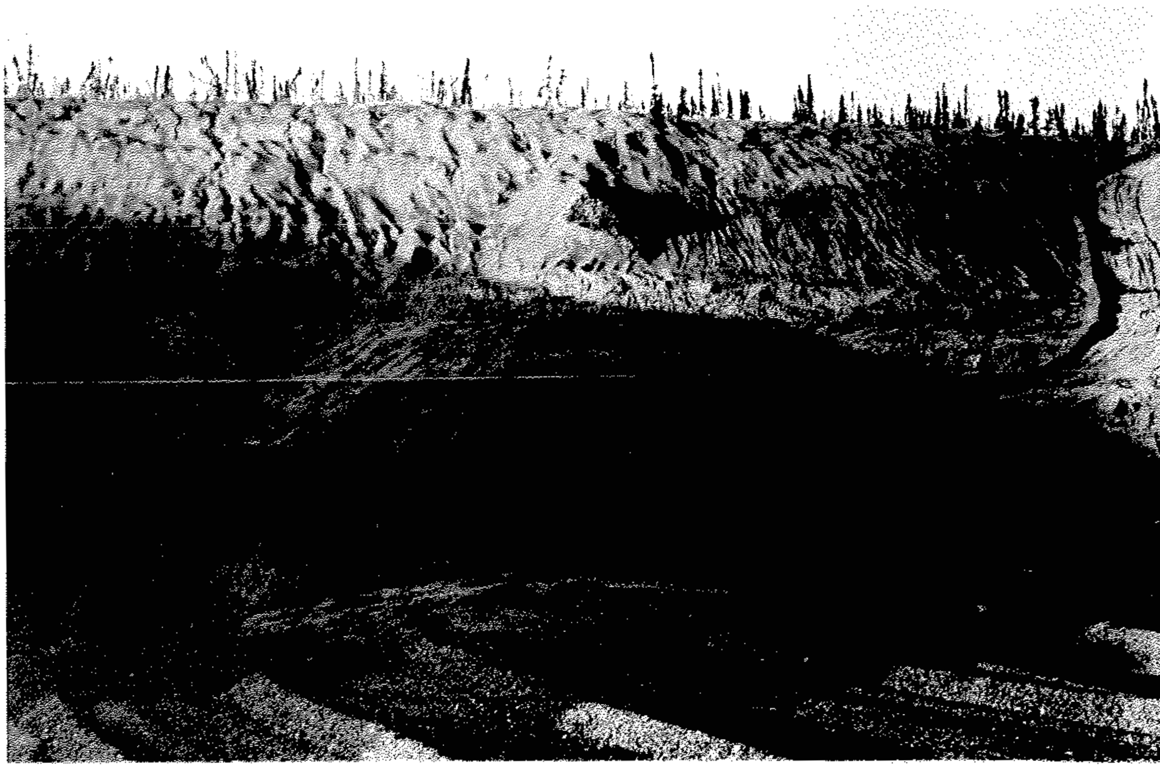


PLATE 13: Shale borrow pit has slight erosion, but is well drained. Trees are about 5 m. in height (Km. 377)

PLATE 14: South abutment of the Eagle River bridge showing 32 degree sideslope. Built in 1977.
(27 Km. south of the Arctic Circle)

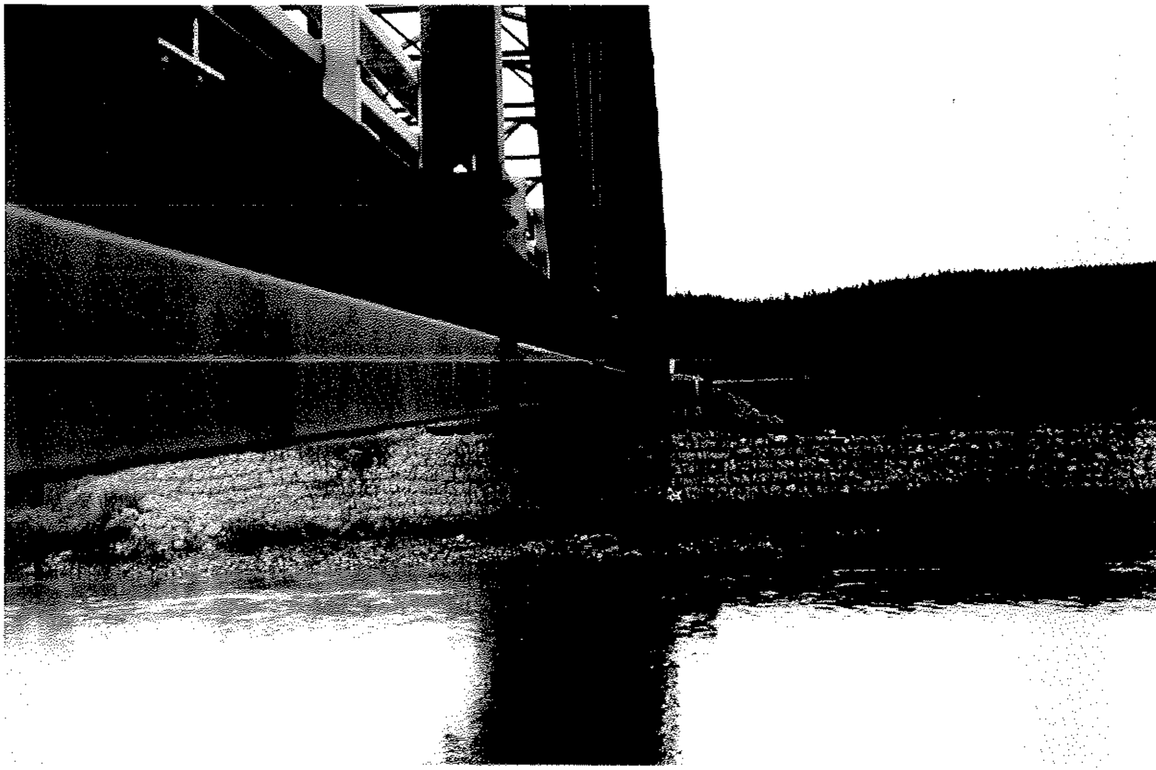


PLATE 15: North abutment of Eagle River bridge with gabion bank armouring in the background. (Km. 373)

PLATE 16: Shale embankment about 6 years old at Km. 412. Snow fence (centre) and large cut and fill (background) are visible. (9 miles north of Arctic Circle).

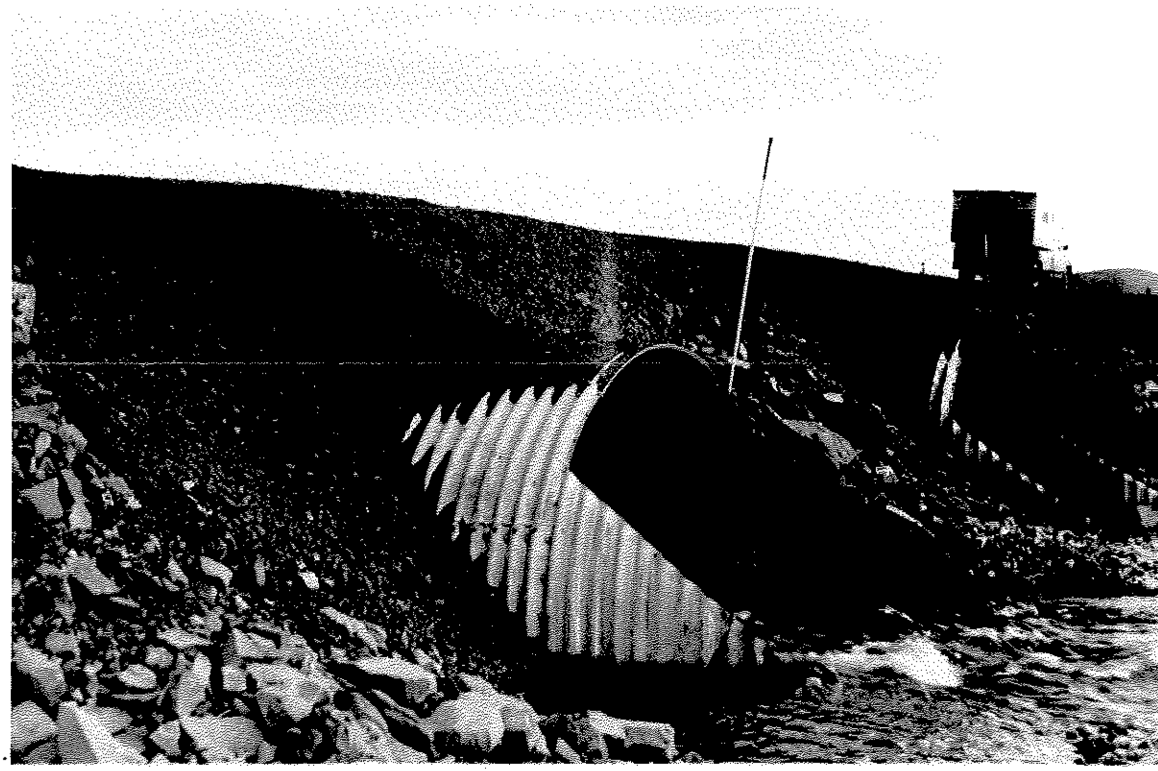


PLATE 17: Embankment erosion at culvert installation.(Km. 423)

PLATE 18: Culverts and ditch blocks (foreground); cut in ice rich soil (background), Yukon.

PLATE 19: Slumping of cut in ice-rich silt.

PLATE 20: Embankment failure 22 Km. west of Peel River, N.W.T.

PLATE 21: No flume on culvert. Consequent erosion is considerable. (Km. 510).

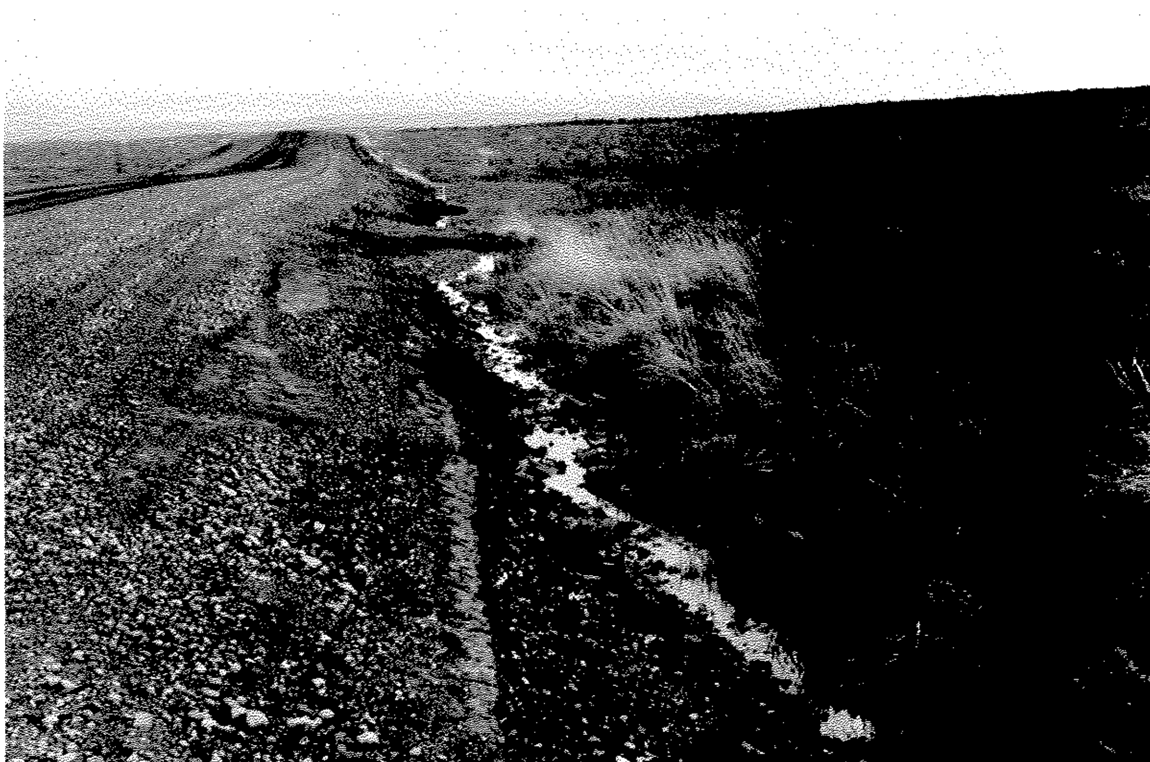


PLATE 22: Soft embankment shoulders at Km. 510.

PLATE 23: Icing area (right centre).
(Km. 571)

PLATE 24: Large cut at Km. 583 (Mile 343). Melt out of ice-rich silty till resulted in uneven backslopes. Leacheate has precipitated from groundwater seepage through a protective blanket of granular material.

PLATE 25: Undulating embankment and subsided culvert (centre) at Km. 633.

PLATE 26: Borrow pit at Km. 720. The sinkholes (about 5m. across) resulted from thaw of ground ice. A polygonal pattern has developed.

C Summary of Observations and Comparisons

Cut/Fill Sloughing

FILLS

The sideslopes of the roadway embankments both north and south of the Yukon/Northwest Territories border, although exhibiting sloughing and soft shoulders in many locations at the time of this study particularly along high fills, were in remarkably good condition for the most part. There was a decrease in inclination of a few degrees relative to design specifications and "as-built" drawings.

The design sideslope of the embankment shown in Figure 8 was specified as 18 degrees to the horizontal, but the actual slope was 15 degrees. In a few cases sloughing and "bulging" of the sideslopes was obvious. (PLATES 9, 10, and 14) Deformations in the embankment -- presumeably resulting from subgrade thermal degradation -- had lead to uncoupling or collapse of only small (CSP) culverts. Collapse and/or uncoupling were observed only in fills greater than about 5 metres high. PLATES 3 and 9 illustrate some cases. Sideslope sloughing was apparently the cause of uncoupling of the culvert in PLATES 9 and 10, which lead to development of a sink hole in the sideslope. Settlement of the embankment near centerline and lateral movement of the fill appeared to overstress and uncouple the culvert shown in PLATE 3.

The deepest fill was encountered at about Kilometer 490. Figure 9 shows a deeply incised valley and braided stream traversed by the fill. Large culverts (4.0 m diameter) are required to accommodate

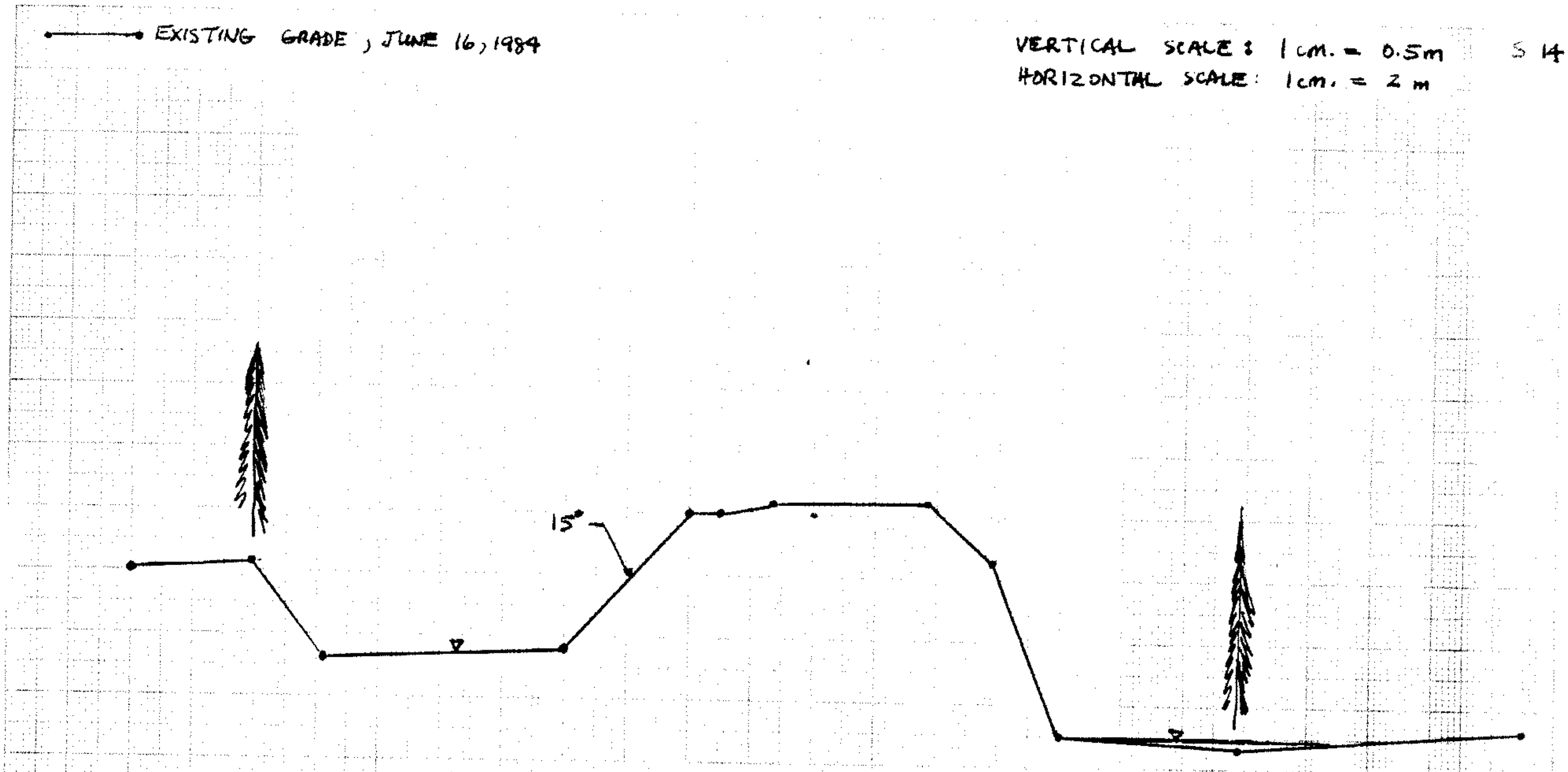


Figure 8 CROSS SECTION, DEMPSTER HIGHWAY (KILOMETER 142)

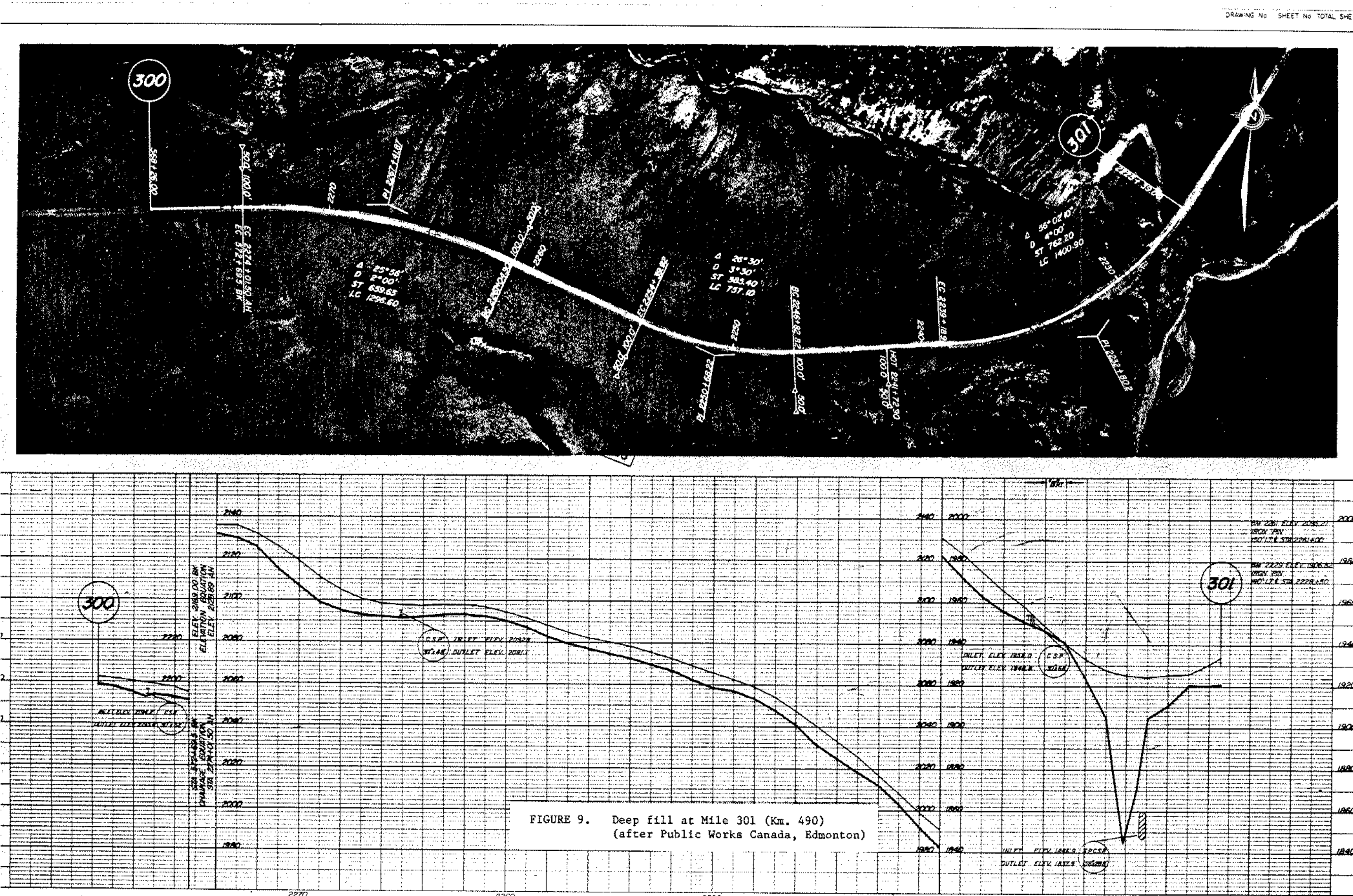


FIGURE 9. Deep fill at Mile 301 (Km. 490)
(after Public Works Canada, Edmonton)

the severe spring runoff. General specifications called for fill sideslopes of 2:1 (26.5 degrees), but the actual slopes were 25.5 and 22.5 degrees. Minor sloughing, erosion and bulging of the slopes had occurred and may account for the discrepancy in slope angle.

Not all embankments employing fine-grained materials either as a core or base have performed entirely satisfactorily. Erosion, warpage, "spreading" and other deformations are some problems which have been encountered as a result of thaw of segregated ice commonly found in fine-grained permafrost materials.

Erosion of sideslopes by runoff was minimal in most cases as a result of judicious choices in mixing fine and coarse grained materials. Serious erosion problems were encountered only at the culvert outfalls lacking flumes or armouring; high spring runoff volumes tended to undermine the ends of those culverts. A variety of factors contributed to erosion: strong eddy currents (PLATE 17); culvert uncoupling; lack of flumes (PLATE 21); and icing of culverts leading to ponding of water above culvert inlet elevations. Proper flume installation eliminated sideslope erosion resulting from culvert discharge. (PLATES 9 and 11)

Only one large failure of an embankment shoulder was encountered along the entire length of highway; at Kilometer 526 (PLATE 20). This is indeed remarkable, when one considers that sideslope angles of 35 degrees were used on some high fills. Standard fill sections in temporate zone highways are typically sloped at about 26 degrees (Thomson, 1984).

Figure 10 shows a profile of the highway along the shoulder near the slide and a profile of the slide itself. An airphoto mosaic of the area prior to the slide (Figure 11) shows gently undulating topography, well established drainage, frost boils in surficial soils, rills and minor slope wash. The original ground surface slopes were approximately 9 degrees to the horizontal, while the embankment was originally sloped at about 25 degrees. A number of factors likely contributed to the failure: (1) the embankment sideslope was long, dark and faced in a southerly direction; (2) the surficial soils consist of frost-susceptible silt which is likely ice-rich ("creep" susceptible and possibly thaw unstable); construction pore pressures may not have dissipated completely; ponded water on the upstream side of the embankment may have created excess pore pressures. The water apparently ponded because the culvert had "iced" shut. The major contributor to instability was likely excess pore pressures generated by thaw consolidation of the subgrade soils.

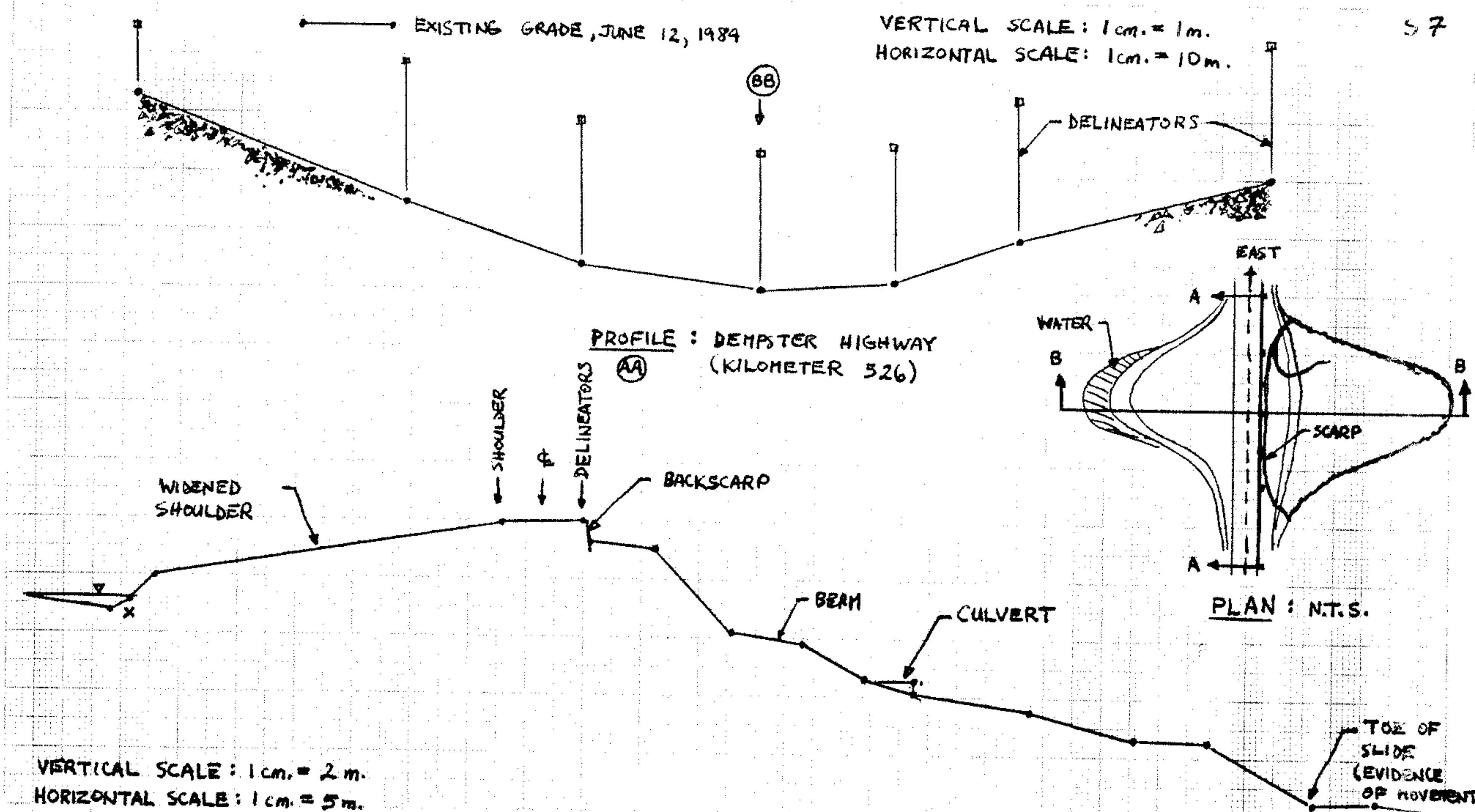


Figure 10. CROSS SECTION (BB): SIDESLOPE FAILURE (KILOMETER 526)

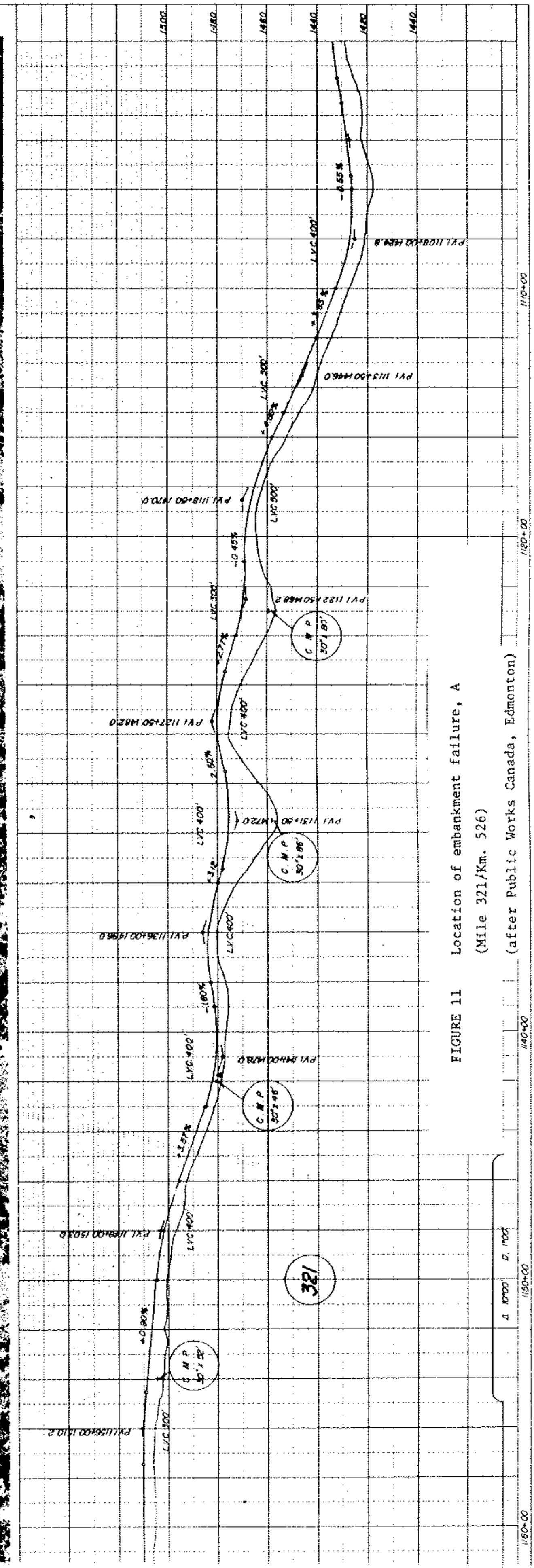


FIGURE 11 Location of embankment failure, A

(Mile 321/Km. 526)

(after Public Works Canada, Edmonton)

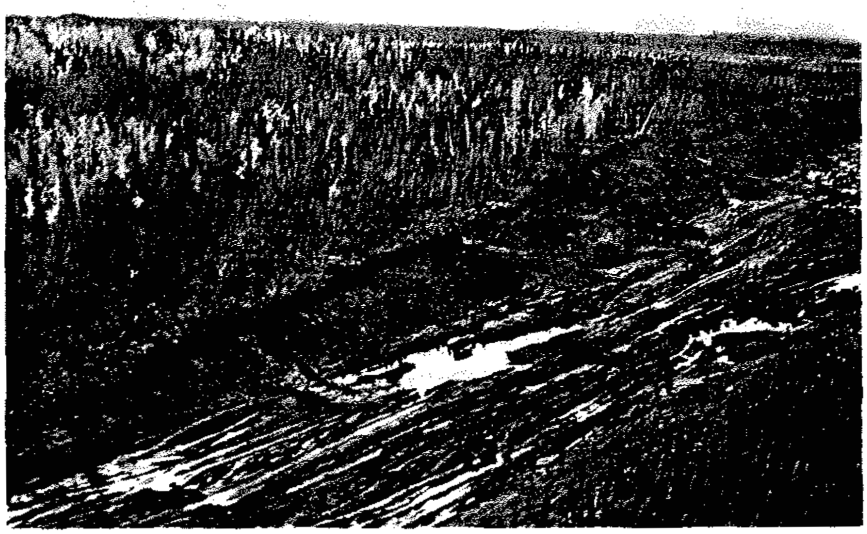
CUTS

The backslopes of cuts made in ice-rich soils in both the Yukon and Northwest Territories have receded in excess of 3 metres in some places, but are now stable after 5 to 12 years. Proper blanketing of exposed ice-rich slopes with adequate thicknesses (over a metre in some cases) of quarried rock or gravels has imparted a degree of stability to the slopes. Given time, nature will provide its own organic "blanket" as the native vegetation above the cut settles downslope. In any case, unsightly erosion channels will develop if the slopes are not protected and subsequent deposition of erodeable materials along the ditches will result.

A large cut in fine grained ice-rich clayey till at Kilometer 583* (Mile 343) is shown in PLATE 27. The excavation was made in April and March of 1972. Backscarps developed at intervals along the backslope where ground ice and ice-rich soil thawed and flowed outward. PLATE 24 shows the backslopes as they were seen in June, 1984. A blanket of granular material was used to protect the slope from further thaw degradation, but subsurface seepage is still vrident. Figure 12 shows thaw penetration in one ditch to be about one metre in June of 1984. Figure 13 shows an "as-built" airphoto mosaic of the cut and the waste material disposal areas. Ground ice areas are not readily apparent, and it is difficult to predict where slumping of a cut is likely to occur solely on the basis of airphoto interpretation.

* Mileages shown on the Airphoto Mosaics are horizontal distances, whereas specified kilometer distances are based on odometer readings.

PLATE 27



ANOTHER VIEW OF THE CUT AT MILE 343.2, DEMPSTER HIGHWAY. THIS PHOTOGRAPH TAKEN SEPTEMBER 7, 1972. OBSERVE THE STANDING WATER IN THE CUT.

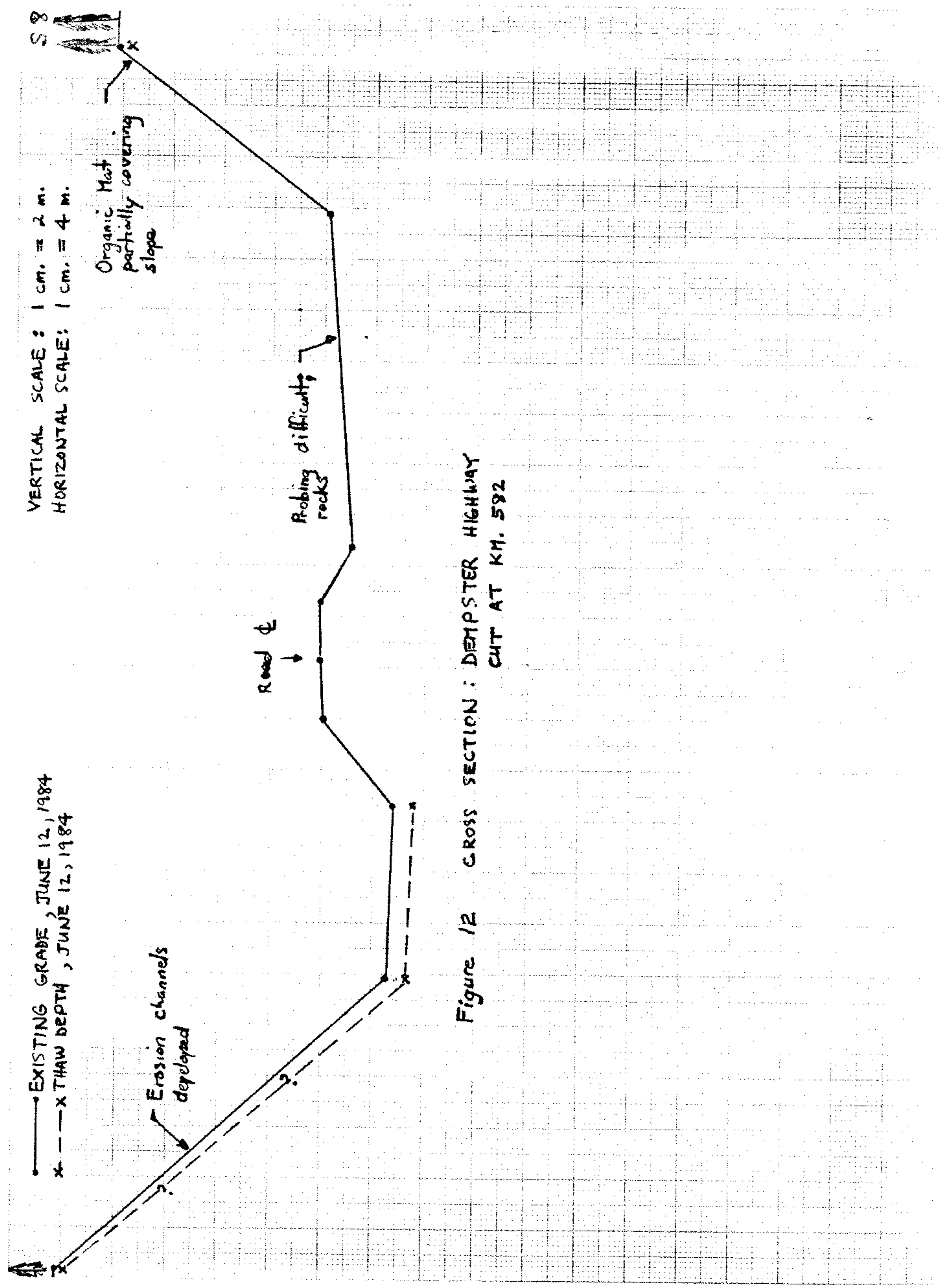
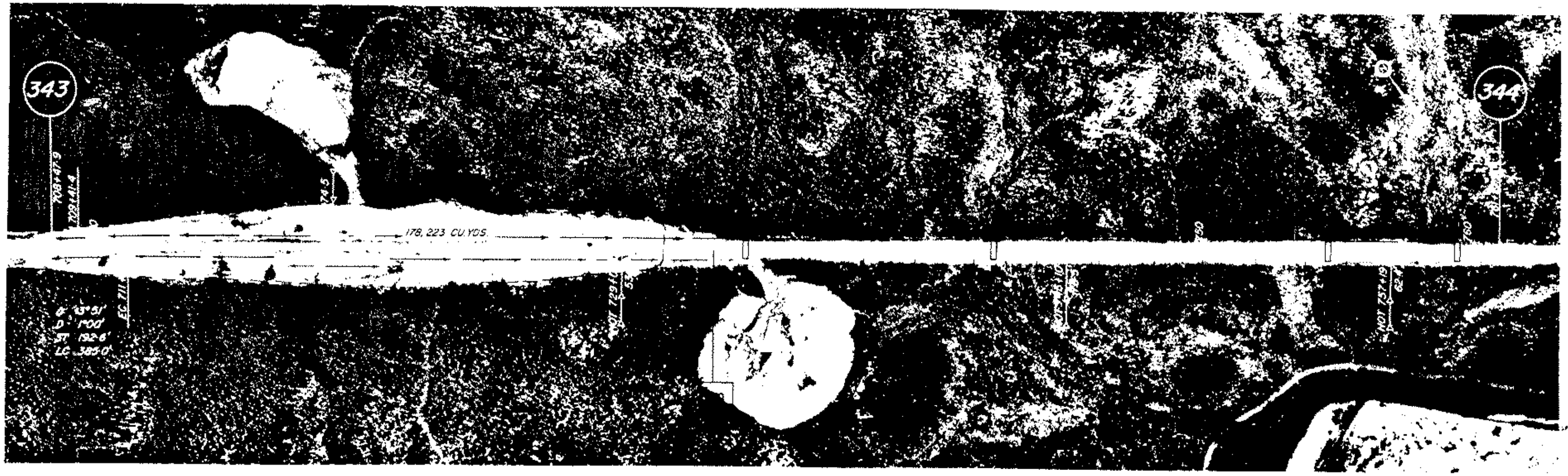


Figure 12 CROSS SECTION: DEMPSTER HIGHWAY CUT AT KM. 592



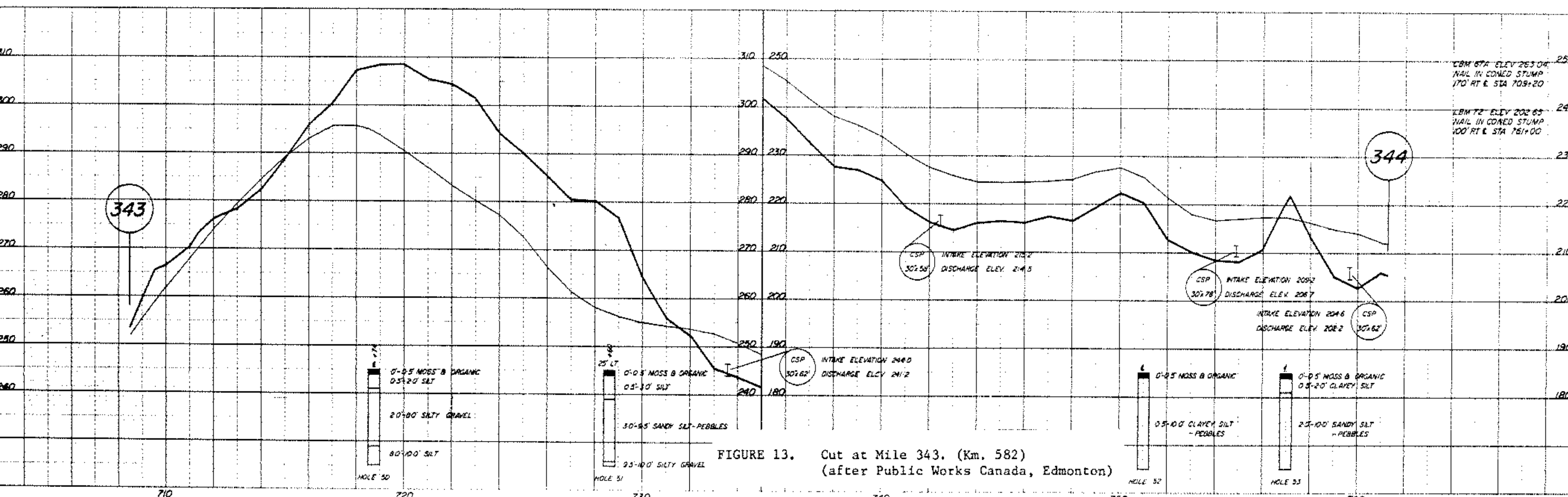


FIGURE 13. Cut at Mile 343. (Km. 582)
(after Public Works Canada, Edmonton)

The borehole logs indicate that the high relief areas have a higher gravel content in the surficial silts than the silts contained in depressions.

PLATES 18 and 19 show a cut made in ice-rich fine-grained soil which has undergone minor slumping due to thaw degradation.

A cut was made in sand and silt at Kilometer 88; tension cracks subsequently formed at the crest, followed by minor slumping. Melt out of ice lenses and interstitial ice is thought to be the cause of instability (Figure 14). A tension crack can be seen in PLATE 6.

Several large and small cuts were made in rock ranging from medium jointed weather resistant sandstone to highly fissured and weathered limestones (PLATE 11), siltstones and shales. All cuts have experienced a certain degree of ravelling over time periods of 5 to 10 years, but in general, design cut slopes have been stable. Figure 15 illustrates a cut in shale which has remained stable at its design slope of 1.5:1. Rock falls were the dominant mode of failure in the harder more competent rocks, while sloughing occurred in softer siltstones and shales. One cut was made in what appeared to be unfavorably jointed rock, but the slope is apparently quite stable.

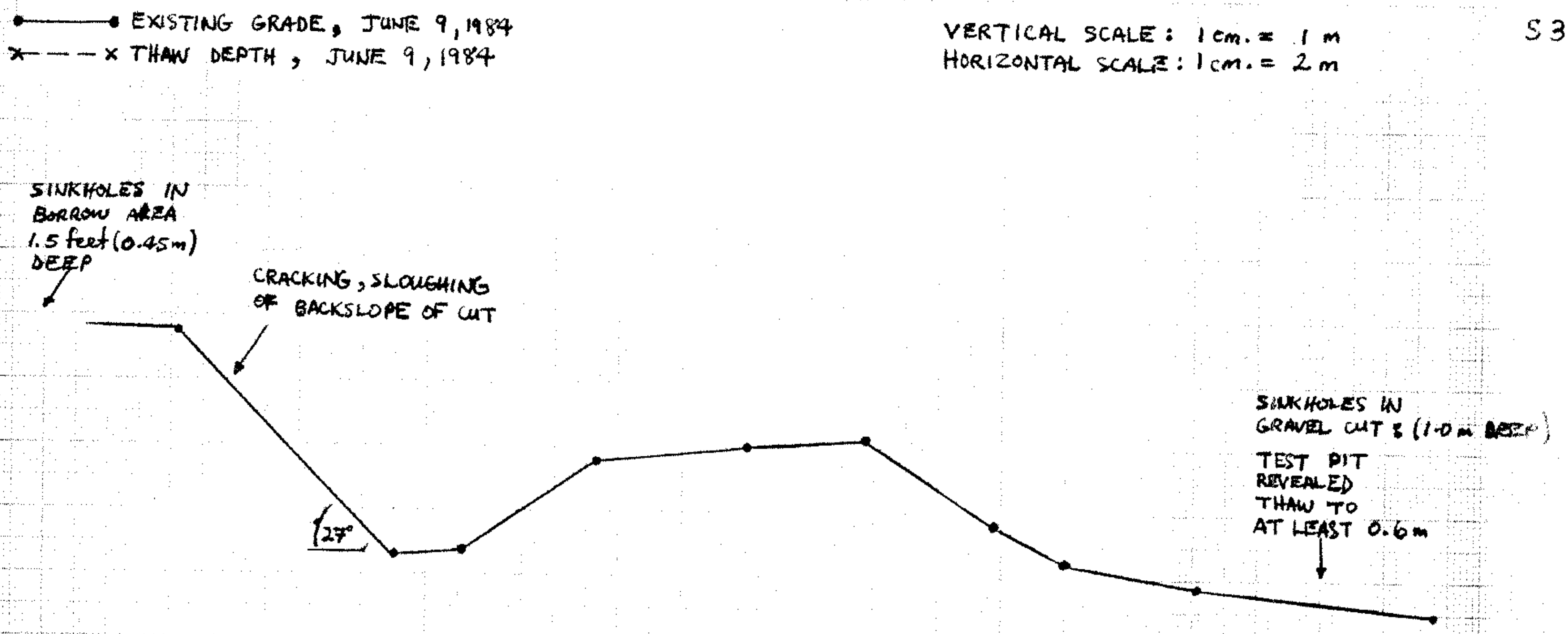
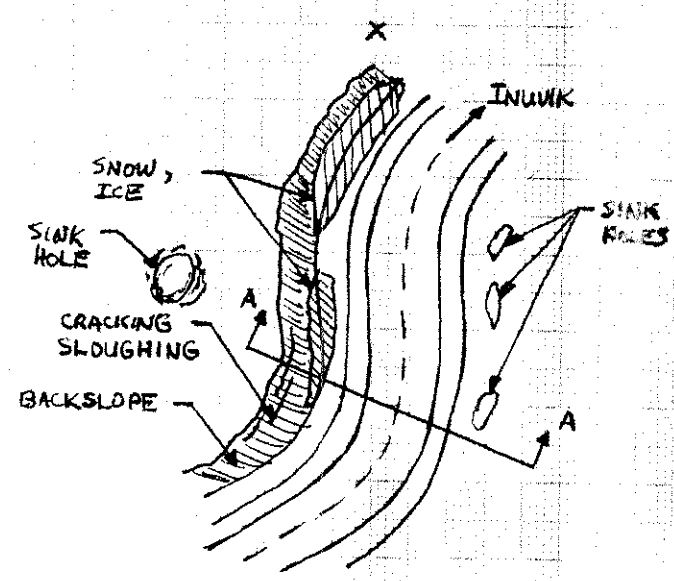


Figure 14 CROSS SECTION (AA) : DEMPSTER HIGHWAY (KILOMETER 88)



PLAN : N.T.S.

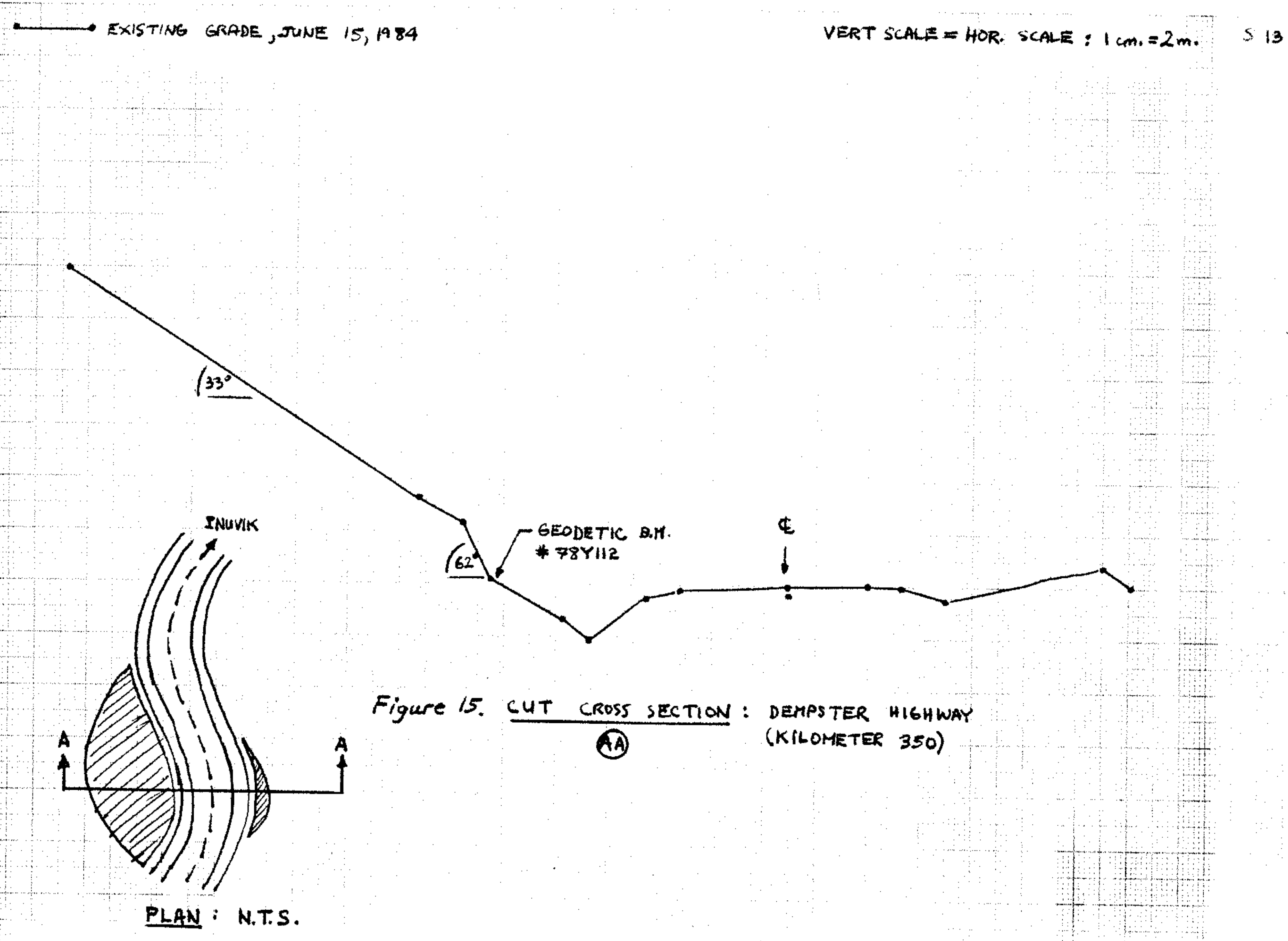


Figure 15. CUT CROSS SECTION (AA) : DEMPSTER HIGHWAY (KILOMETER 350)

Permafrost Degradation

FILLS

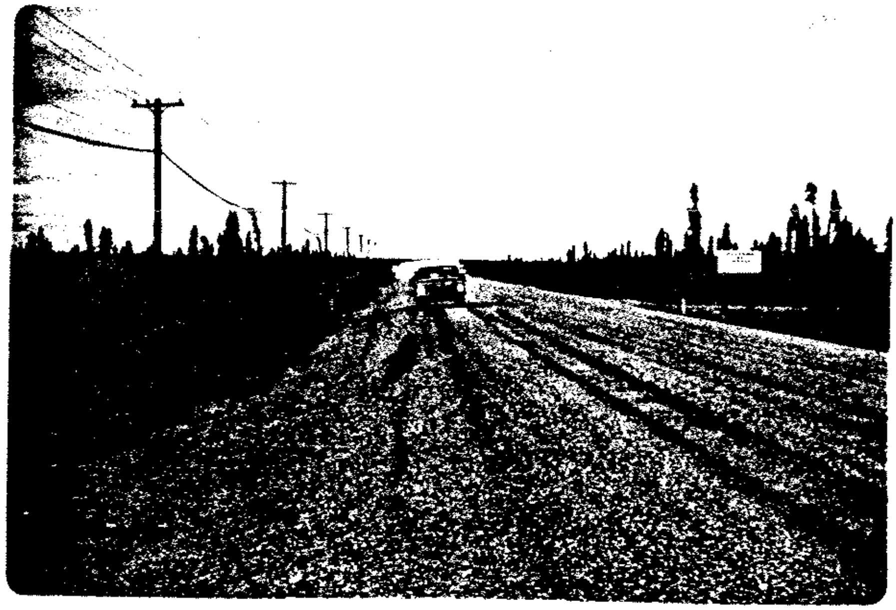
(a) Glaciated Terrain

The extent of permafrost degradation beneath embankments is dependent on several related factors which include: type, height and moisture content of fill material, the degree of destruction or compression of the virgin organic cover, elapsed time since installation, the efficiency of drainage structures and the ice content of the mineral soil. Of the soils we were able to probe, thaw penetration was generally 0.1 to 0.7 metres in mid-June. Ground ice was present in the form of pore ice or segregated ice (small lenses, veins) or massive ice (beds, wedges) in all four physiographic regions (Sections 1 to 4) where glacial activity was evident.

In the spruce forest/sphagnum moss associations outside of the southerly sections of the highway right-of-way (Sections 1, 2, and 4), and in the Northwest Territories (Section 9) thaw penetration of ice-rich mineral soil was only 0.1 metre. (PLATE 2) Similar thaw depths were encountered at the latitudes of the tundra plant communities (PLATE 22) and the altitudes of alpine plant associations (PLATES 5 and 7). The willow, labrador tea and cotton tussock plant community which thrives in the cleared right-of-way ditches was successful in limiting thaw to only 0.4 to 0.7 metre depth in mid-June. In areas of recently ponded water, thaw was generally an additional 0.2 metre deeper. Based on observations of the highway in its present state, and information from previous embankment studies it is estimated that additional thaw caused by clearing of trees and placement of 1.2 metre high embankment is approximately 1 metre after a period of 10 year.

Cook and Huculak (1980) examined thaw penetration under different types of embankments in different terrain areas. Figures 16 and 17 show examples of cross-sections, while the corresponding photographs (PLATES 28 and 29, respectively) illustrate the terrain types. The embankment illustrated in Figure 16 is located along the old airport road south of Inuvik. Settlements in the order of 0.2 metres have occurred since reconstruction (1975). The old road was built in 1960 and comprised silt, sand and gravel. The peat varied in thickness from 0.3 to 1.3 metres, had moisture contents ranging from 100 to 700 percent, and overlies ice-rich silty clays. The old road has settled up to 0.5 metres since construction; compression of the organic mat probably accounts for 50 percent of the settlement. The permafrost table has been lowered by about 1 metre beneath the embankment. The embankment was in good condition in 1980 (PLATE 28) notwithstanding the subsidence. An airphoto mosaic (figure 18) shows that the terrain is relatively flat and poorly drained.

SITE 1 - STATION 182+00 - MILE 966.6



(a)

(b)

PLATE 28 Airport Road about 10 Km. east of Inuvik.
Looking west - note water ponding in ditch on right - some minor loss of section (slumping) on left (south) shoulder.
(after Cook/Huculak)

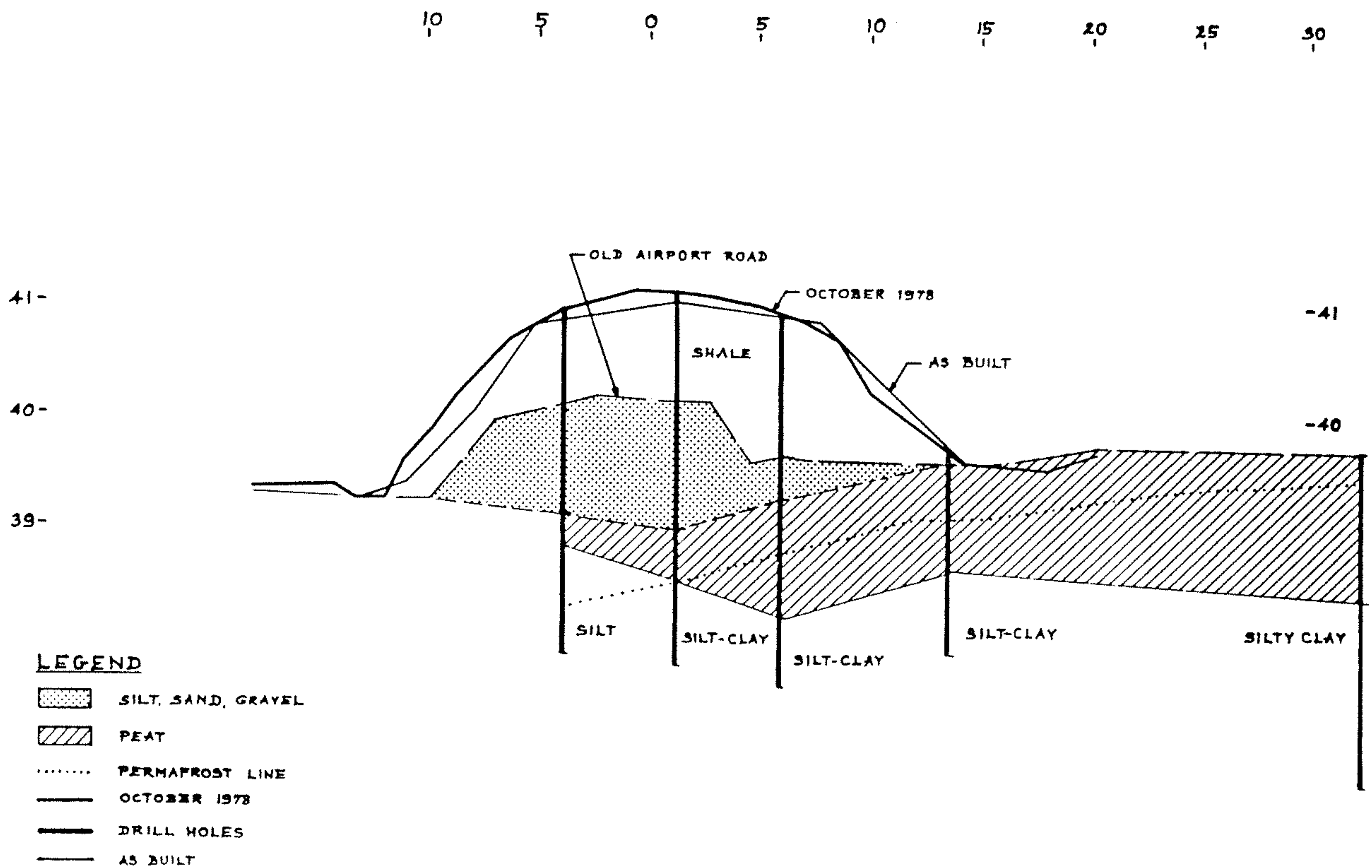


FIGURE 16 Cross section of Airport Road 10 Km. east of Inuvik. (Km. 735)

(after Cook/Huculak)

SITE 1
STATION 182+00
MILE 966.6

DRILL BOOK 2, 3, & 5

SITE 10 - STATION 3265+00 - MILE 902.2

(a)

(b)

PLATE 29 Flat, poorly drained terrain. (Km. 630)

Looking north - water on both sides and running through grade at base - flat, poorly drained terrain. Extensive surface undulations: settlement and loss of section (slumping of shoulders).

(after Cook/Huculak)

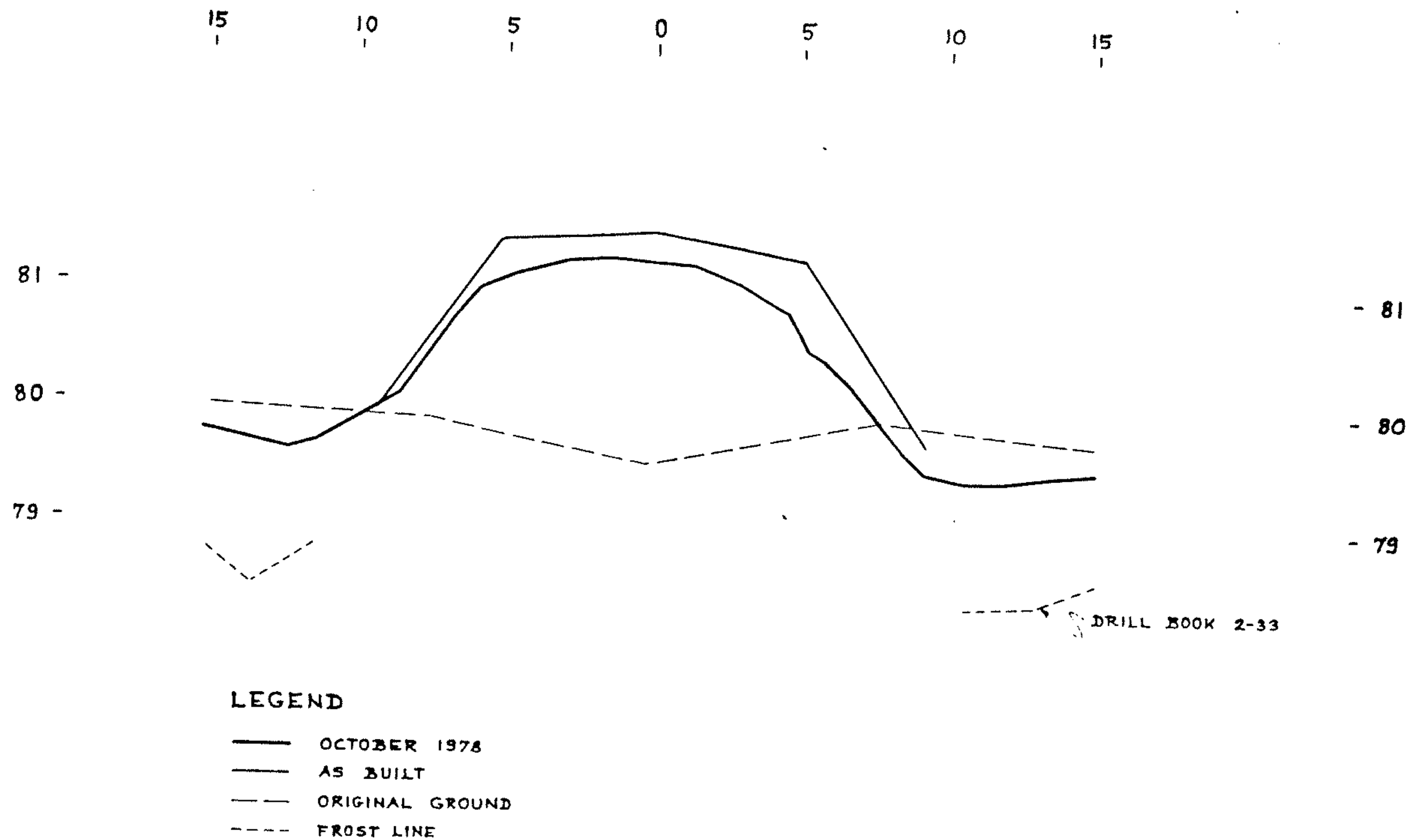


FIGURE 17 Cross section of embankment in flat, poorly drained terrain. (Km. 630)

SITE 10
MILE 902.2
STATION 3265+00

DRILL BOOK 1-17

(after Cook/Huculak)

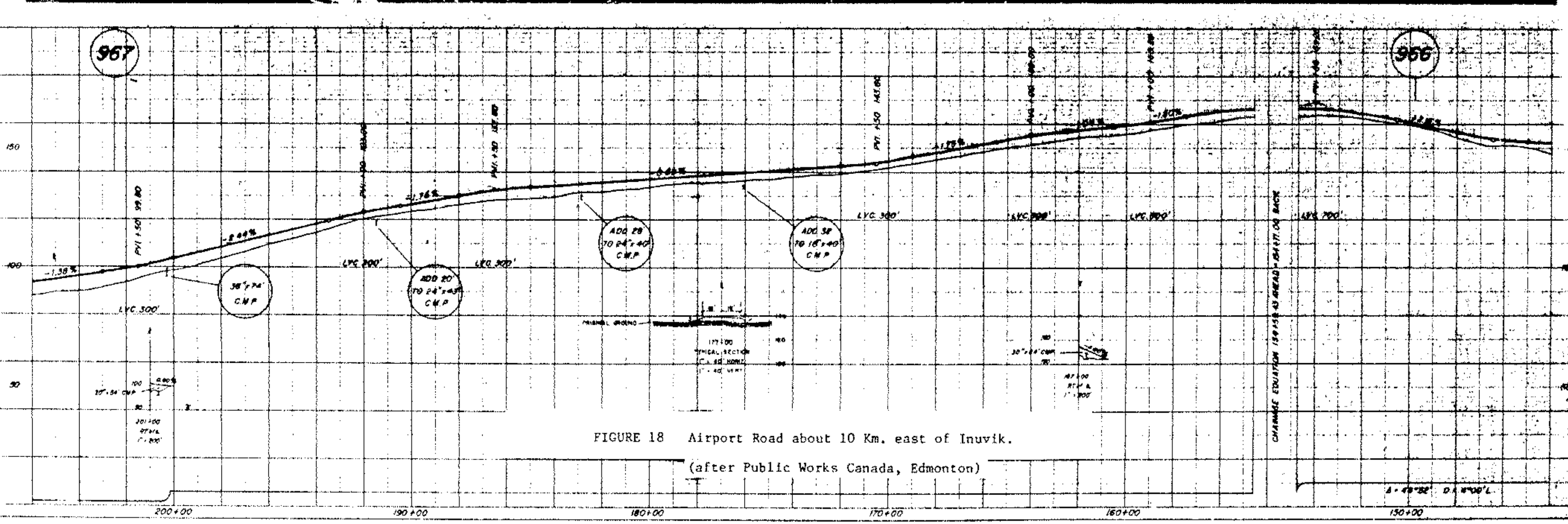


FIGURE 18 Airport Road about 10 Km. east of Inuvik.

(after Public Works Canada, Edmonton)

Undulations and loss of section occurred in areas where low embankments traversed flat, poorly drained terrain. The embankment section shown in Figure 17 is located approximately 10 miles north of Arctic Red River and was constructed with shale in rapid, staged winter construction (September - October, 1973). Slumping of shoulders and settlement of the surface (0.3 metres) were presumably a result of compression of the peat (0.3 metres thick) and thaw consolidation of the underlying ice-rich silt (1.2 metres thick). An airphoto mosaic and highway profile (Figure 19) illustrates the poor local drainage.

Extensive thawing and settlement (0.6 metres) occurred beneath and adjacent to the embankment shown in Figure 21. This was due primarily to the presence of a relatively thin insulating peat cover and previously disturbed terrain. The embankment appears to have subsided in a relatively uniform manner, probably a result of summer thaw consolidition following winter placement. The embankment was brought to grade in the summer of 1971 and the sandstone/limestone core appears to have retained the general design shape in spite of the settlement. Minor surface undulations can be seen in Figure 20.

A fine-grained core with a substantial shale cap (0.5 - 1.0 m thick) was placed in the winter of 1972 south of Campbell Lake near Kilometer 716. (PLATE 30, Figure 22). Minor slumping and loss of section has occurred (0.2 m settlement) over a period of 6 years. The permafrost has degraded relatively uniformly, except in the left ditch where subsurface seepage was apparent from a borehole log.

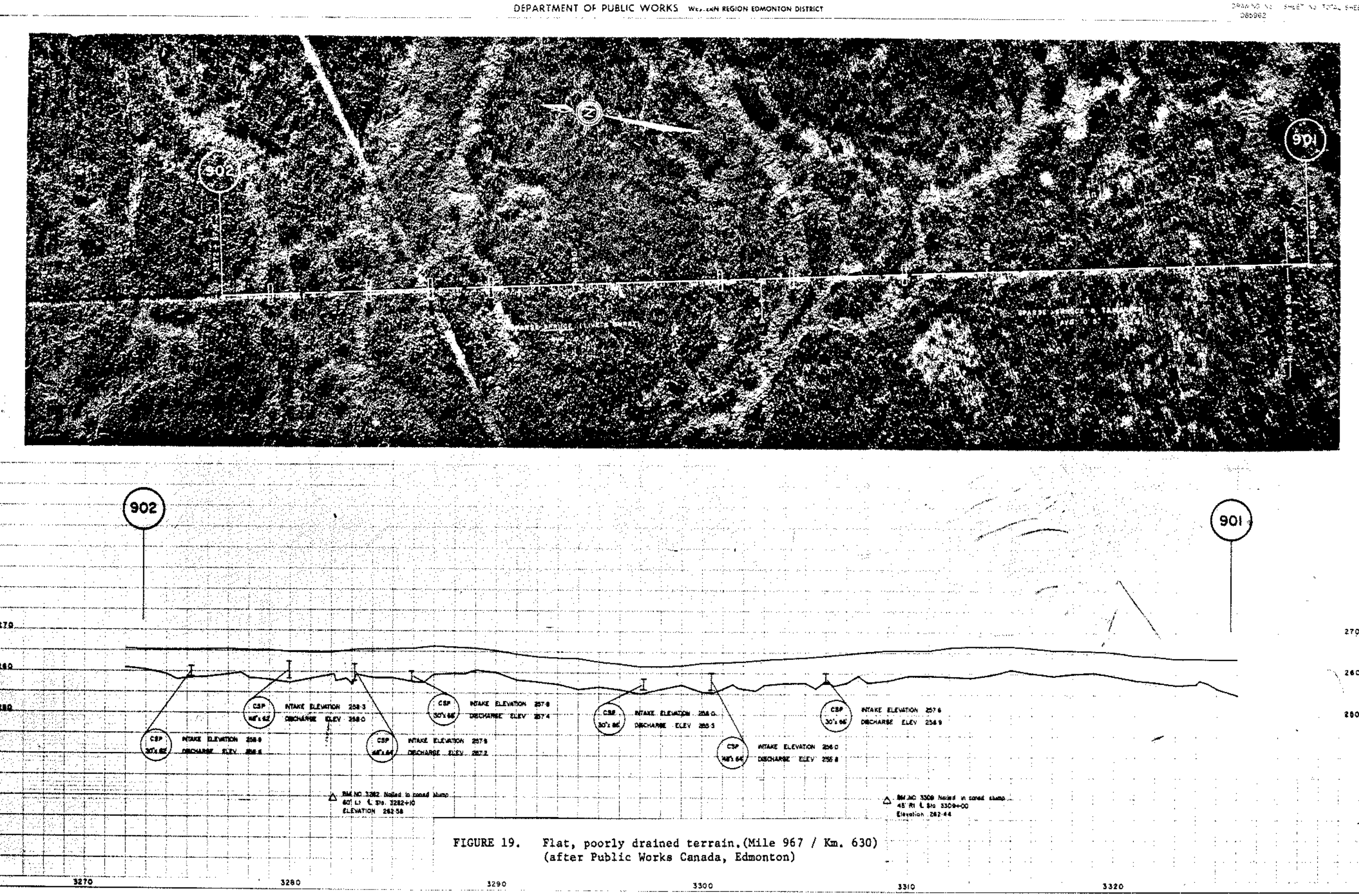


FIGURE 19. Flat, poorly drained terrain.(Mile 967 / Km. 630)
(after Public Works Canada, Edmonton)

SITE 2 - STATION 56+00 - MILE 962.9

(a)

Looking west - embankment has generally retained original x-section but has settled approximately 0.5 to 0.6 m.

(b)

FIGURE 20 Minor surface undulations.(Km. 729)

Looking west - note minor undulations in embankment surface. Old C.N.T. line can be seen to left of small spruce.

(after Cook/Huculak)

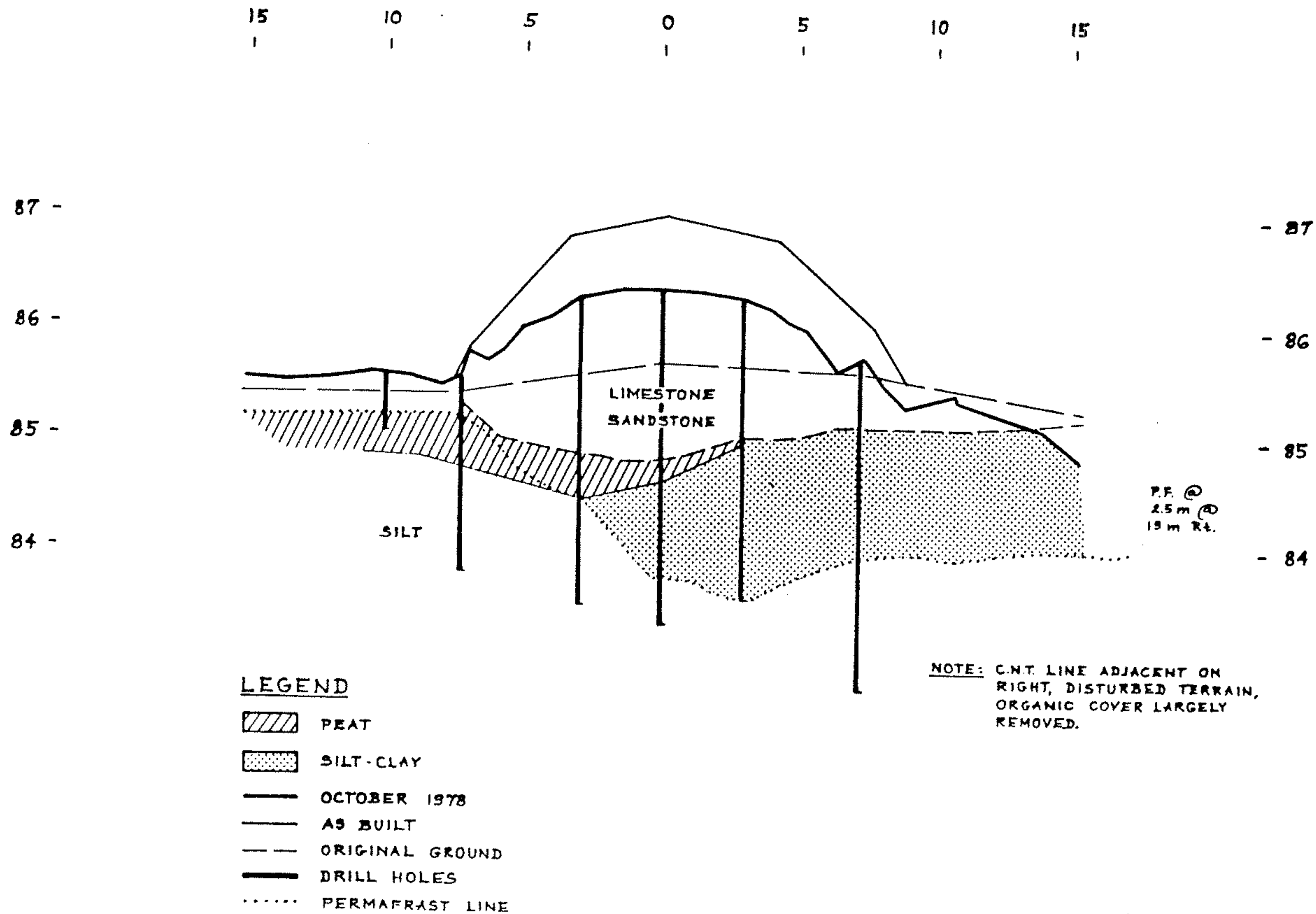
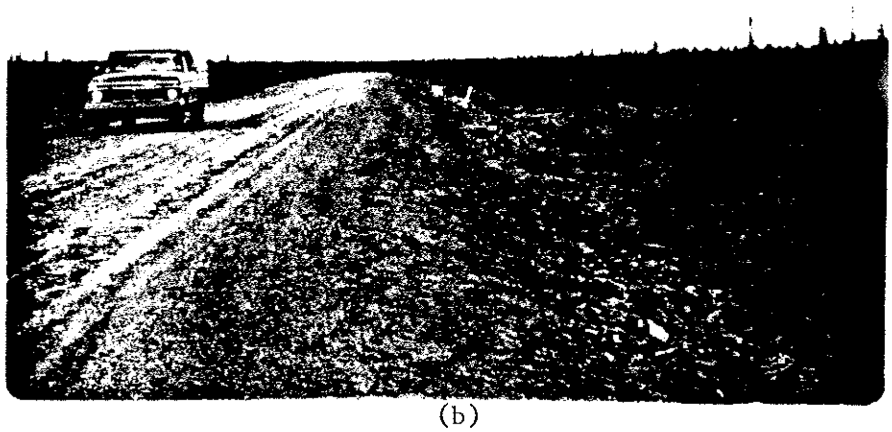


FIGURE 21 Cross section showing considerable subsidence. (Km. 729)

(after Cook/Huculak)

SITE 5 - STATION 587+00 - MILE 953.1

(a)



(b)

PLATE 30 Embankment with a fine grained core capped with shale (Km. 715).

Looking north - site is similar to Site 4 except no water in upslope ditch - settlement is greater than at Site 4

(after Cook/Huculak)

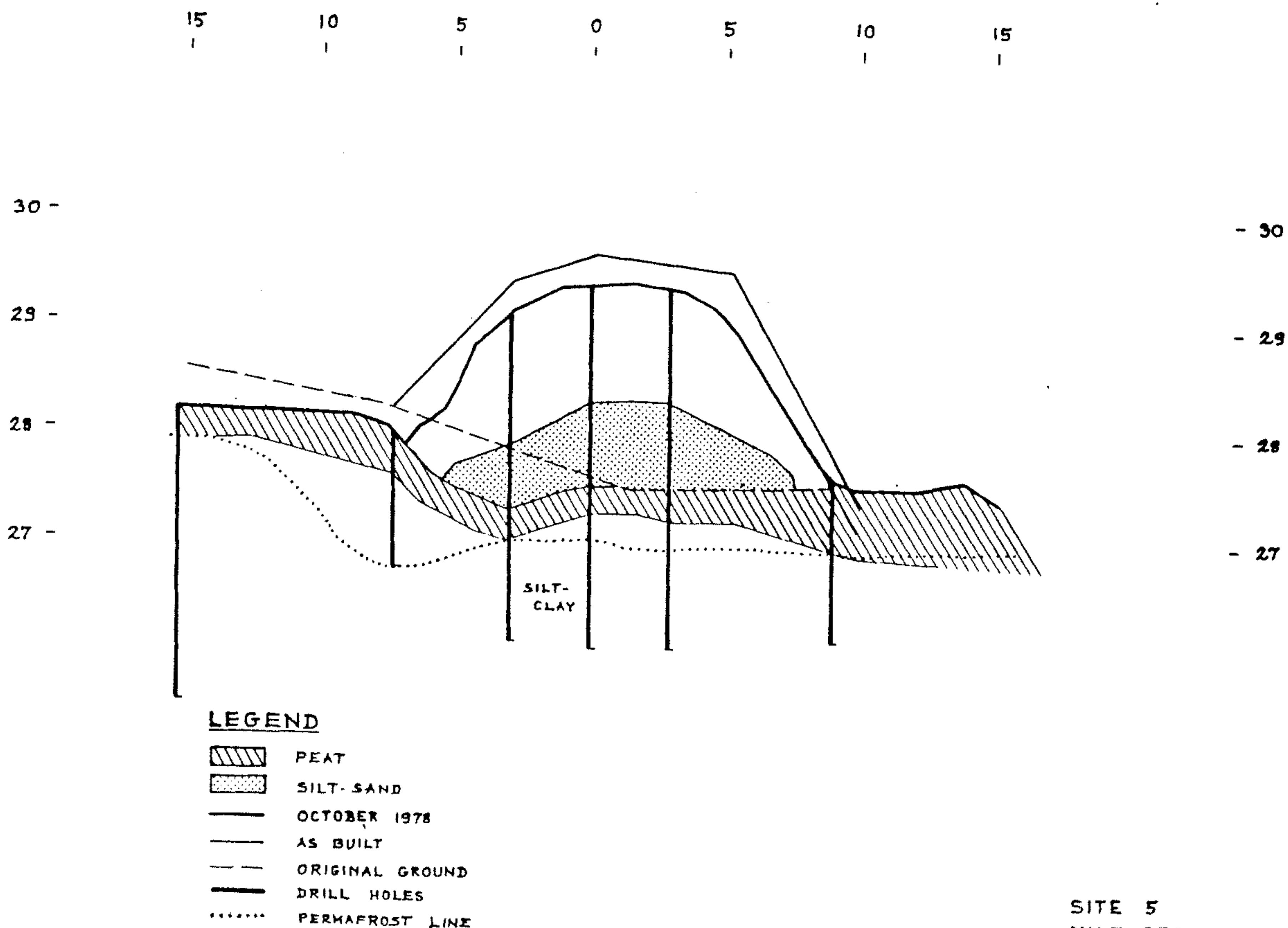


FIGURE 22 Embankment with a fine grained core. (Km. 715)

SITE 5
MILE 953.1
STATION 587+00

DRILL BOOK 1-4

(after Cook/Huculak)

An initial silty sand pad 0.6 metres thick was placed in December of 1971 in the same area. Figure 23 shows that as much as 0.5 metres of settlement occurred after one thaw season. The permafrost degraded by as much as 0.6 m in the ditches, but was preserved (desired behavior) beneath embankments in some locations (Station 548, Figure 23) -- apparently where thicker fills were used. Subsidence in the thicker fills (0.15 m) was less than the thinner fills (Station 515) where as much as 0.4 metres of settlement took place. In 1984 excess undulations or roughness in the riding surface were not observed, indicating that the embankment in this area has stabilized after about 8 years. (The embankment looked the same in 1984 as it did in 1980.)

Figure 24 shows a cross-section obtained during the current investigation (mid-June 1984) about 2 kilometers south of the section in Figure 22. PLATES 31 and 32 (km 718), 1972) show an embankment under construction near the one shown completed in PLATE 30 (1978). Considerable subsidence and thaw degradation was apparent at the time of construction.

Thermal regime changes are apparent within the embankment near Km 580, shown in Figure 25. The permafrost degraded in some sections and not in other locations. The airphoto mosaic in Figure 26 shows polygonal ground patterns at stations 518 and 521 and thermokarst features south of the embankment. From borehole logs it is apparent that a minimum of 2.5 m of clayey silt overlies shale bedrock. The embankment was constructed with an initial 1 metre lift of saturated silty clay till in the winter of 1971. A shale cap was placed in the winter of 1972. The embankment shape and quality was very good when

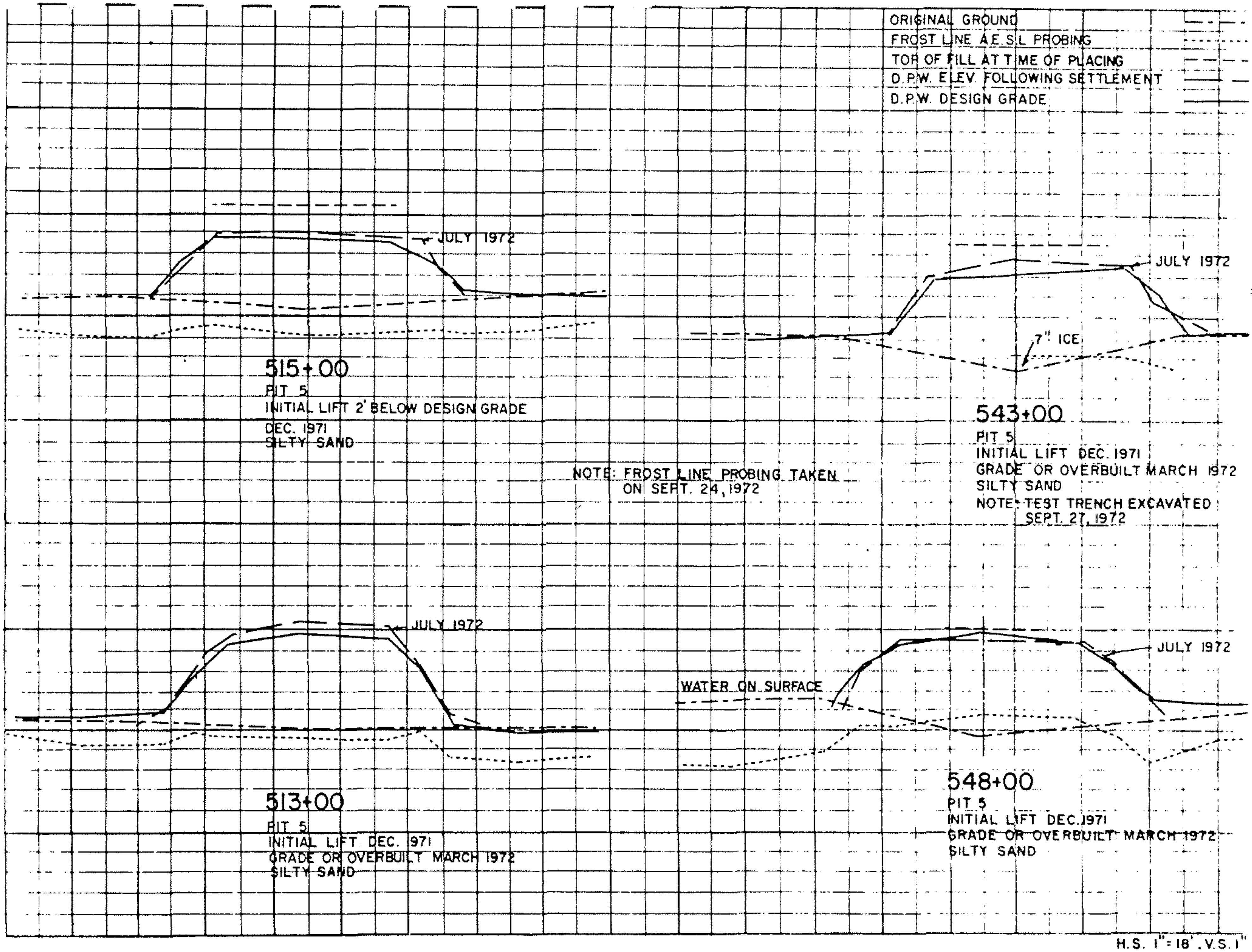


FIGURE 23 Cross Sections of the MacKenzie Highway. (Mile 954 /Km. 716) (after AESL, 1972)

NOV 29 1972

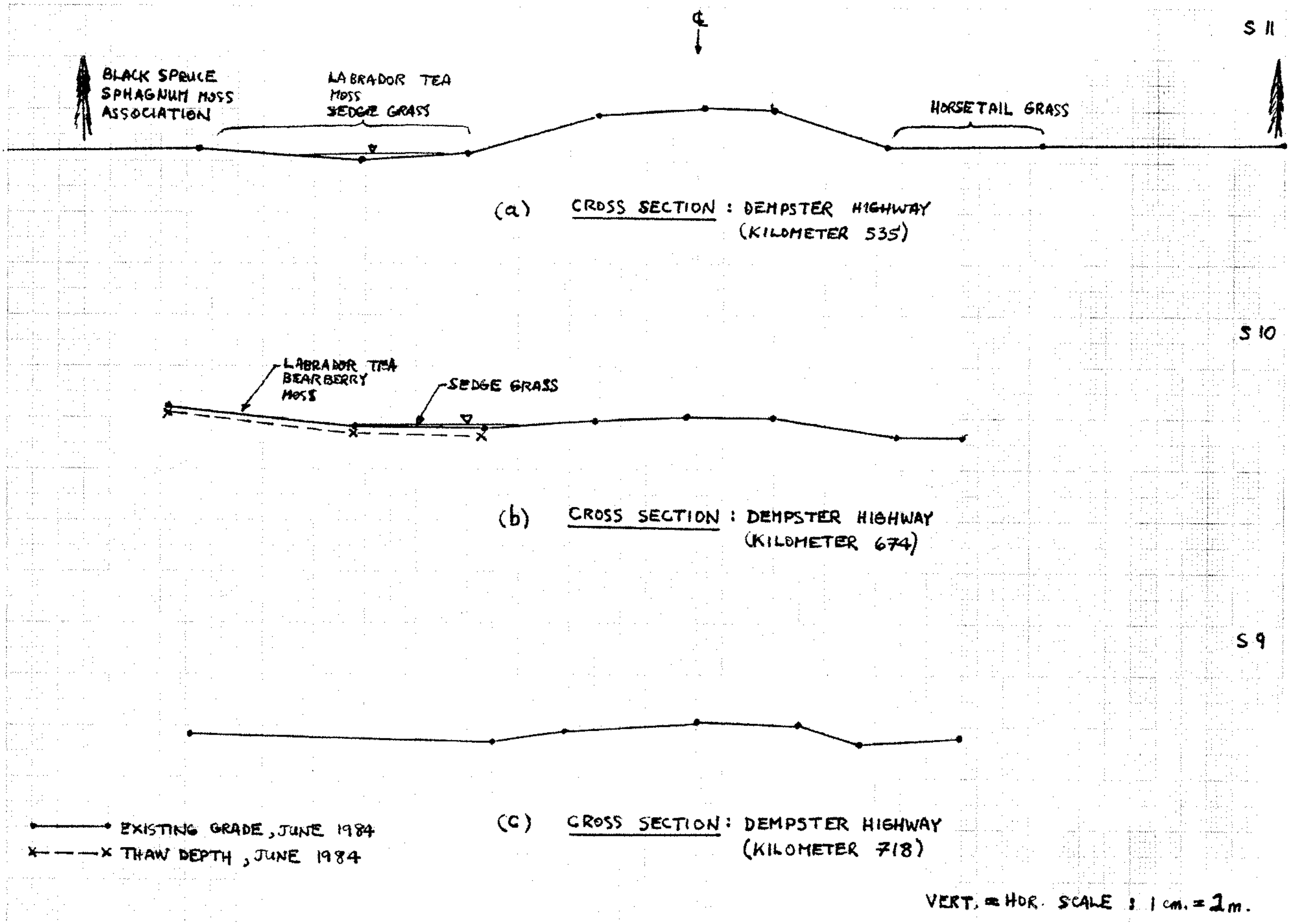
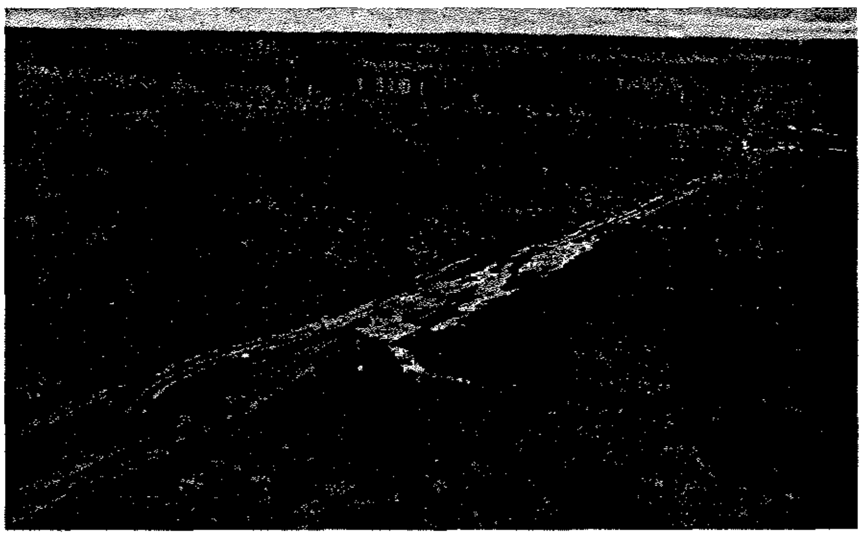

S 11
BLACK SPRUCE
SPHAGNUM MOSS
ASSOCIATION
LABRADOR TEA
MOSS
SEDGE GRASS
HORSETAIL GRASS
(a) CROSS SECTION : DEMPSTER HIGHWAY
(KILOMETER 535)
S 10
LABRADOR TEA
BEARBERRY
MOSS
SEDGE GRASS
(b) CROSS SECTION : DEMPSTER HIGHWAY
(KILOMETER 674)
S 9
EXISTING GRADE, JUNE 1984
THAW DEPTH, JUNE 1984
(C) CROSS SECTION : DEMPSTER HIGHWAY
(KILOMETER 718)
VERT. = HOR. SCALE : 1 cm. = 2 m.

Figure 24.

PLATE 31

MILE 955, MACKENZIE HIGHWAY, SEPTEMBER 7, 1972, LOOKING SOUTHERLY. THIS EMBANKMENT WAS CONSTRUCTED DURING DECEMBER 1971 WITH MATERIAL FROM THE PIT AT MILE 953.3. NOTICE HOW THE EMBANKMENT HAS SUBSIDED AND HOW THE MATERIAL IS IN AN EXTREMELY SOFT CONDITION. AT THIS SECTION, THE LOCATION FOLLOWED ALONG A SEISMIC LINE.

PLATE 32



MILE 954.7, MACKENZIE HIGHWAY, SEPTEMBER 7, 1972. LOOKING SOUTH-WESTERLY. THE CROSSING AT CABIN CREEK CAN BE SEEN IN THE RIGHT BACKGROUND. THIS SECTION OF EMBANKMENT WAS CONSTRUCTED DURING DECEMBER 1971 WITH MATERIAL FROM THE PIT AT MILE 953.3. OBSERVE THE SUBSIDENCE IN THE EMBANKMENT AND THE CONSIDERABLE AMOUNT OF WATER.

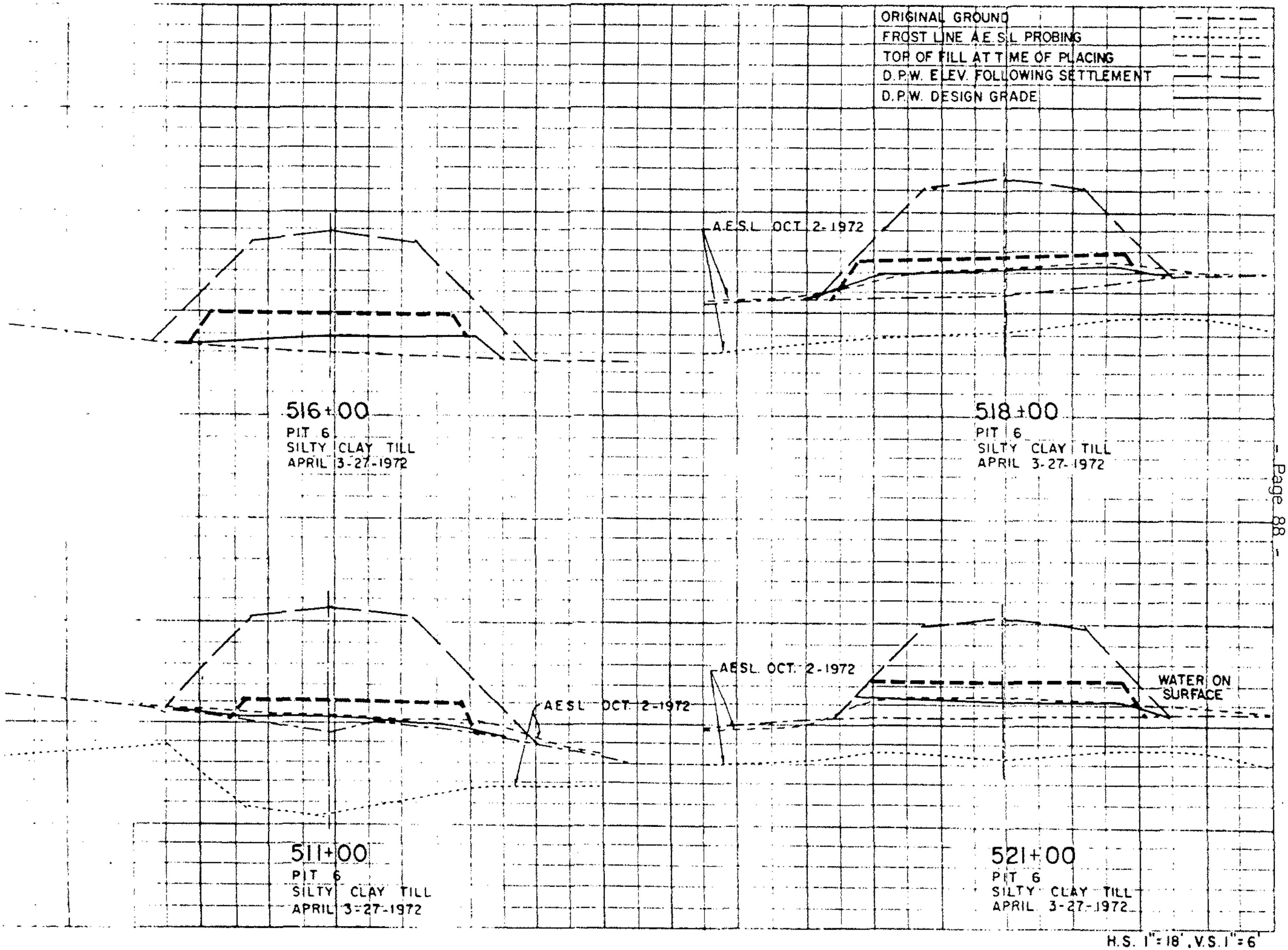


FIGURE 25 Cross Sections of the Dempster Highway (Mile 340.6 / Km. 580) (after AESL, 1972)

NOV 29 1972

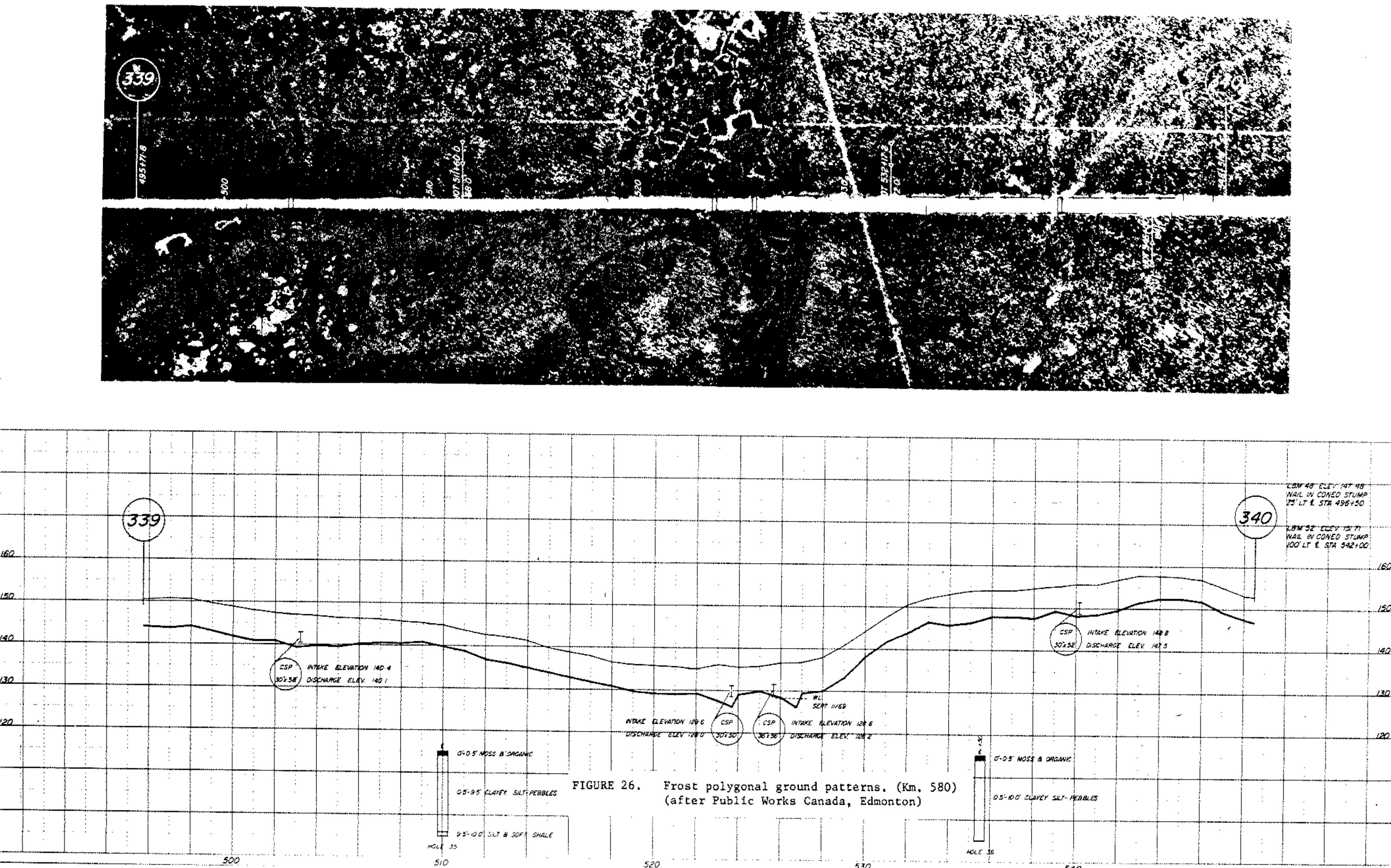


FIGURE 26. Frost polygonal ground patterns. (Km. 580)
(after Public Works Canada, Edmonton)

travelled in June of 1984.

Some high embankments (over 5 m) were constructed in the winters of 1971 and 1972 using initial and secondary lifts of sandy and silty gravels with moderately high ice contents. However, these embankments underwent only 0.2 metres of subsidence in the first 2 years.

One of the oldest sections of the highway comprises kilometer 0 to 126. The first 60 kilometers has been rebuilt, but the remaining portion is in its original state, except where new culverts are being installed, in preparation for up-grading. In several locations the road has subsided below original ground level. Figure 27 (a) shows where the road is almost level with original ground. (Thaw depth increases to one half metre towards the embankment toe.)

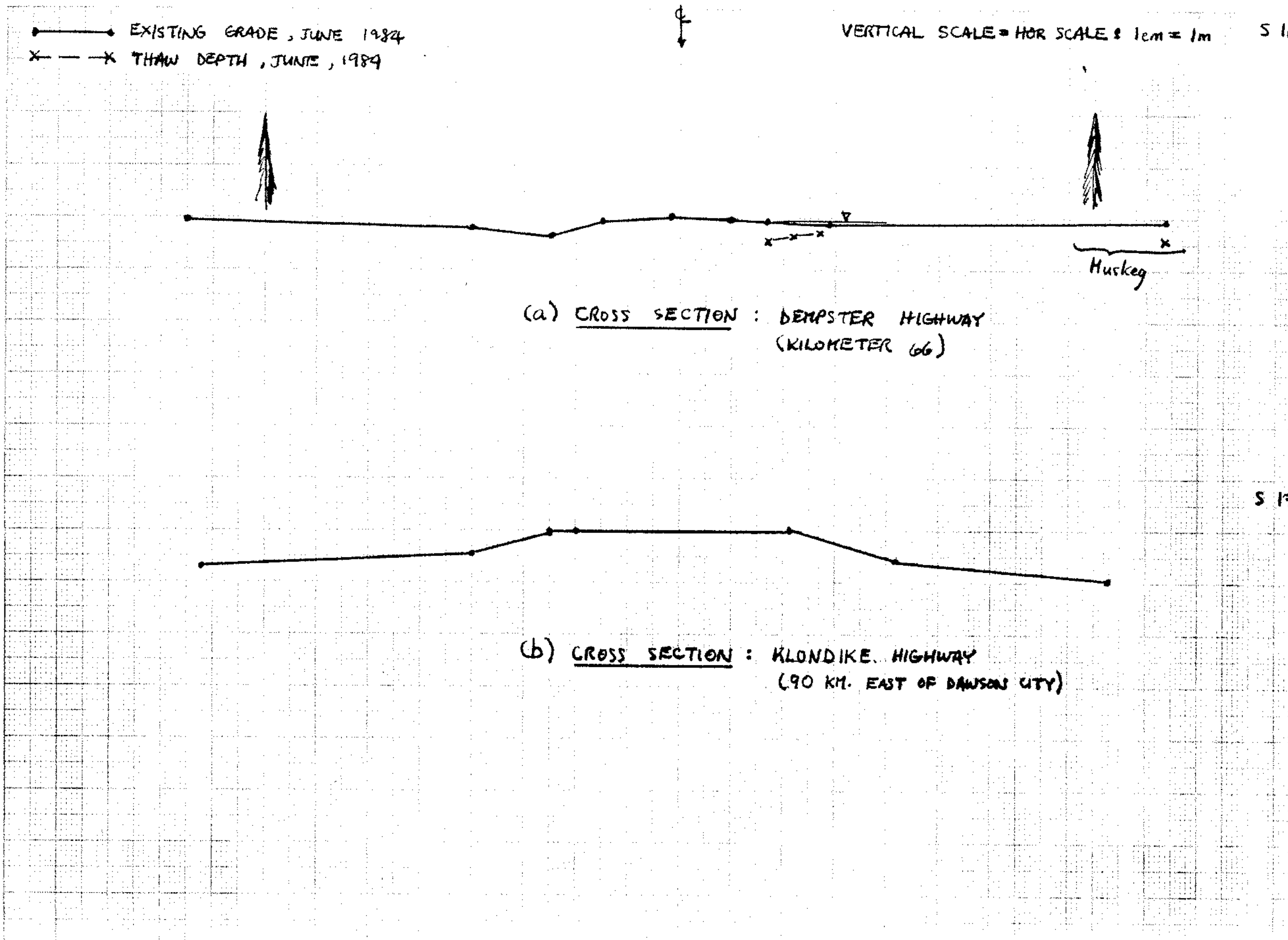

EXISTING GRADE, JUNE 1984
THAW DEPTH, JUNE, 1984
VERTICAL SCALE = HOR SCALE : 1cm = 1m
S 16
Huskeg
(a) CROSS SECTION : DEMPSTER HIGHWAY (KILOMETER 66)
S 17
(b) CROSS SECTION : KLONDIKE HIGHWAY (90 KM. EAST OF DAWSON CITY)

Figure 27.

(b) Unglaciated Terrain

In order to establish limits to the extent of glaciation in the vicinity of the Dempster Highway, photographs were taken at intervals along the highway and examined in conjunction with soil exposures (where available). By determining the presence or absense of glacial features (U-shaped valleys, drumlins, flutings, kames, grooves, roche moutonees, truncated spurs, and moraines) or the presence or absence of non-glacial features (V-shaped valleys, with intercalated spurs, smooth rounded hills with waxing and waning slopes, residual soils) it was possible to estimate the extent of glaciation. The region traversed by the first 110 kilometers of highway appears to have been glaciated. This encompasses the Ogilvie Mountain ranges in which several local advances of isolated valley glaciers are believed to have occurred independent of, but simultaneous with the continental glaciers in temperate zones. (N. Rutter, 1984) From Kilometer 110 up to and including the Richardson Mountains, there was no evidence of glaciation, but there was conclusive evidence of non-glaciation for most of the region.

The embankment section shown in Figure 28 has apparently not subsided significantly. Thaw penetration adjacent to the embankment (0.3 m) was slightly greater than in undisturbed areas (0.1 m); PLATE 7 illustrates the terrain in the vicinity. A section was obtained 20 kilometers to the north (Figure 29) in floodplain terrain. Thaw depth beneath the embankment was about half that of the ditches (1 m). Trees tipping in towards the embankment were good indicators of

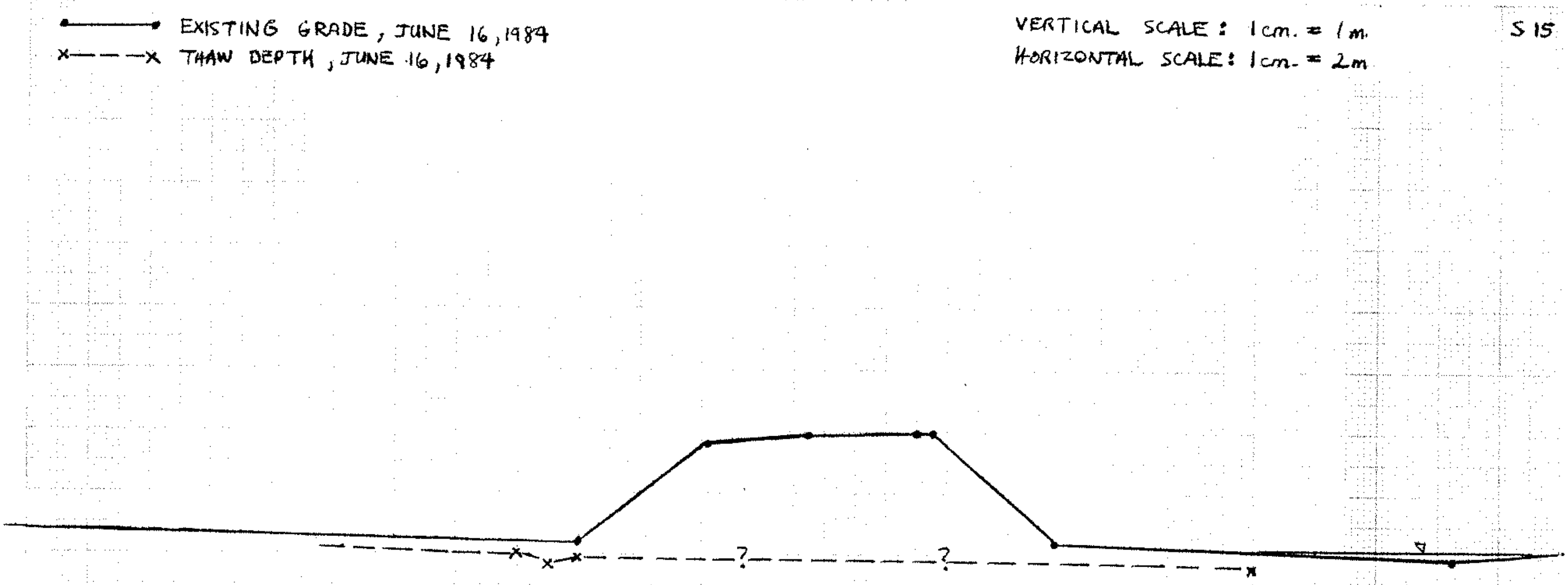
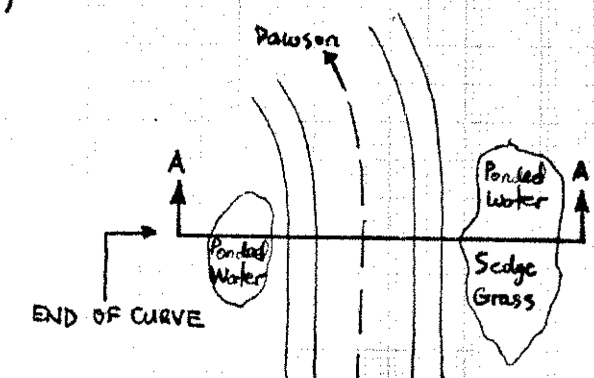


Figure 28. CROSS SECTION (AA) : DEMPSTER HIGHWAY (KILOMETER 122)



PLAN: N.T.S.

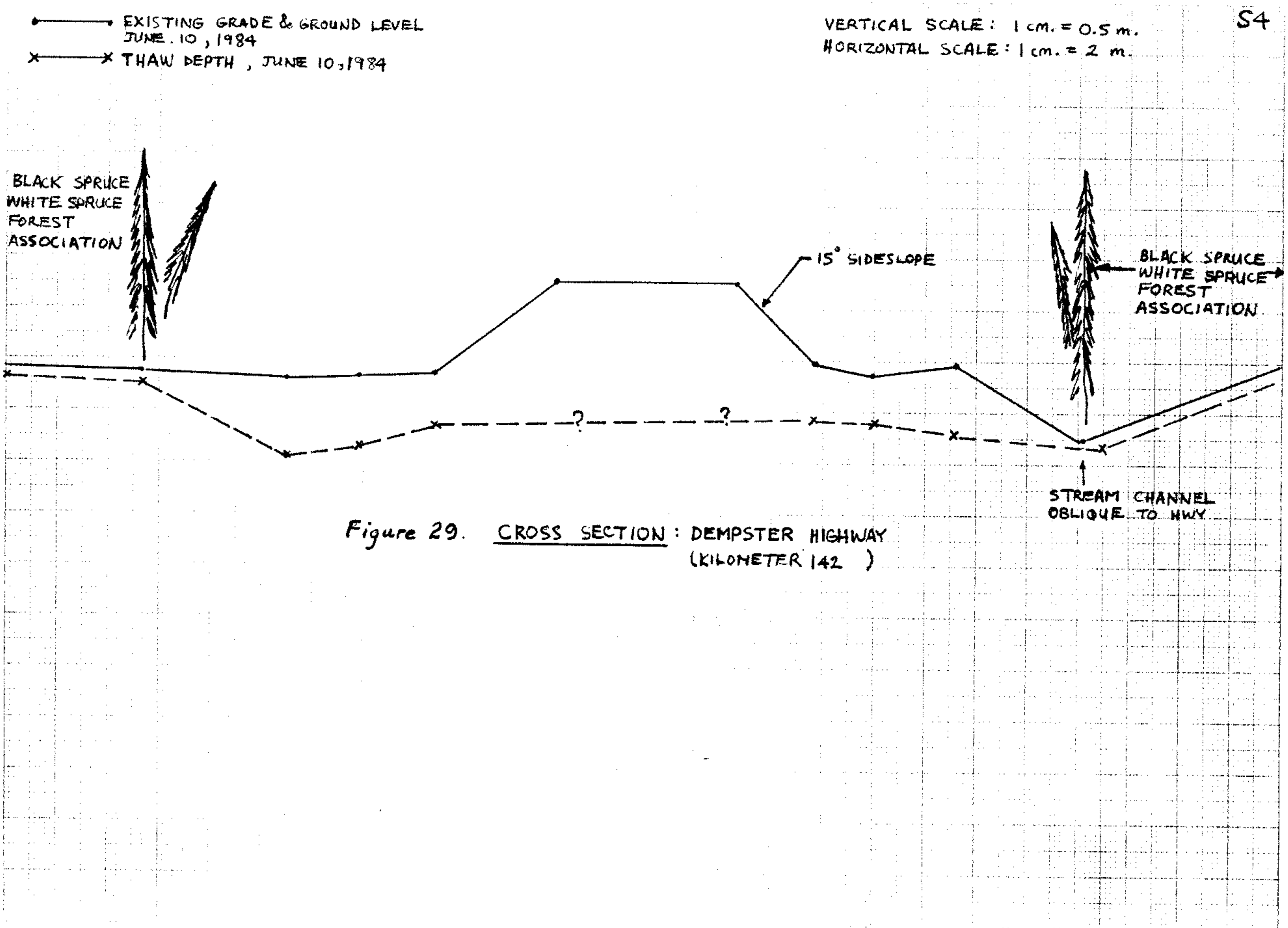


Figure 29. CROSS SECTION: DEMPSTER HIGHWAY (KILOMETER 142)

thermally induced subsidence. The design sideslope for the embankment was 18 degrees, but 15 degrees was measured. Slope flattening such as this was likely a result of thaw of the subgrade and subsequent movement.

Thawing of polygonal ground ice produced the embankment deformations shown in Figure 30. Since the deformations are highly localized, they can be easily repaired. Thaw of the ground ice resulted in the water filled depressions shown in PLATE 12. Trees were tipping in random directions in response to thermally induced subsidence (natural causes). The vegetative mat in this region was up to 0.5 m thick and overlays at least 6 metres of silty clay of low plasticity, which was sandier with depths.

In the higher alpine areas such as the Richardson Mountains surficial soils were thin and supported little or no vegetative cover depending on altitude and slope angles. The thaw depth shown in Figure 31 is deceptive because it actually represents the depth to rock; true thaw depth is presumeable somewhat deeper. Felsenmeer is a common terrain feature of the Richardson Mountains which provides a thermally stable base for embankments.

Embankment performance in unglaciated areas (ie. Eagle Plains Plateau) has been good because throublesome areas (soliflucted slopes, braided streams, failing slopes, thick ice-rich valley soils) were avoided.

It is difficult to make meaningful comparisons between embankment performance in glaciated and unglaciated areas because factors other

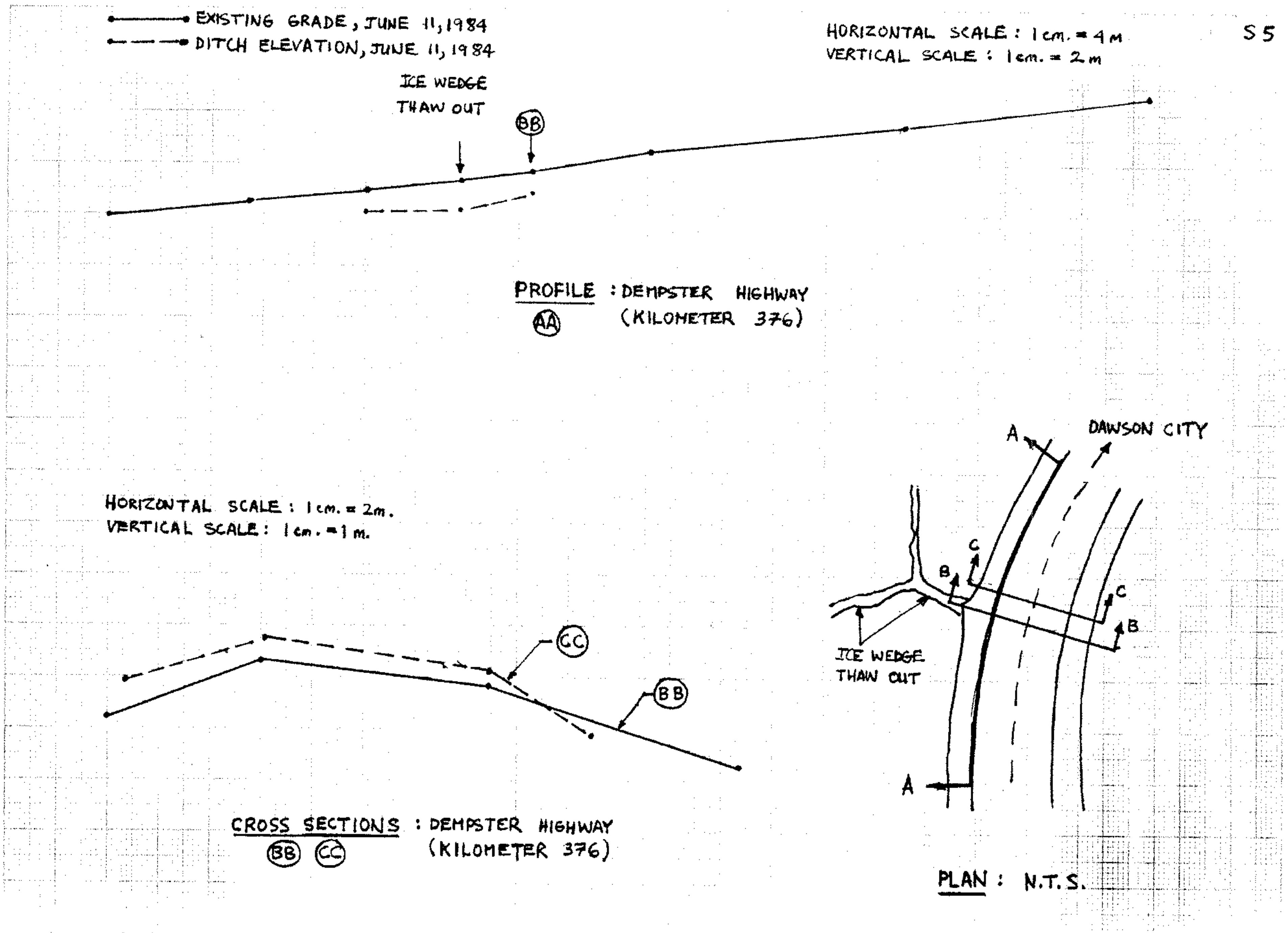

EXISTING GRADE, JUNE 11, 1984
DITCH ELEVATION, JUNE 11, 1984
ICE WEDGE THAW OUT
BB
HORIZONTAL SCALE : 1 cm. = 4 m
VERTICAL SCALE : 1 cm. = 2 m
S5
PROFILE AA : DEMPSTER HIGHWAY (KILOMETER 376)
HORIZONTAL SCALE : 1 cm. = 2 m.
VERTICAL SCALE : 1 cm. = 1 m.
CC
BB
CROSS SECTIONS BB CC : DEMPSTER HIGHWAY (KILOMETER 376)
DAWSON CITY
A
B
C
ICE WEDGE THAW OUT
PLAN : N.T.S.

Figure 30.

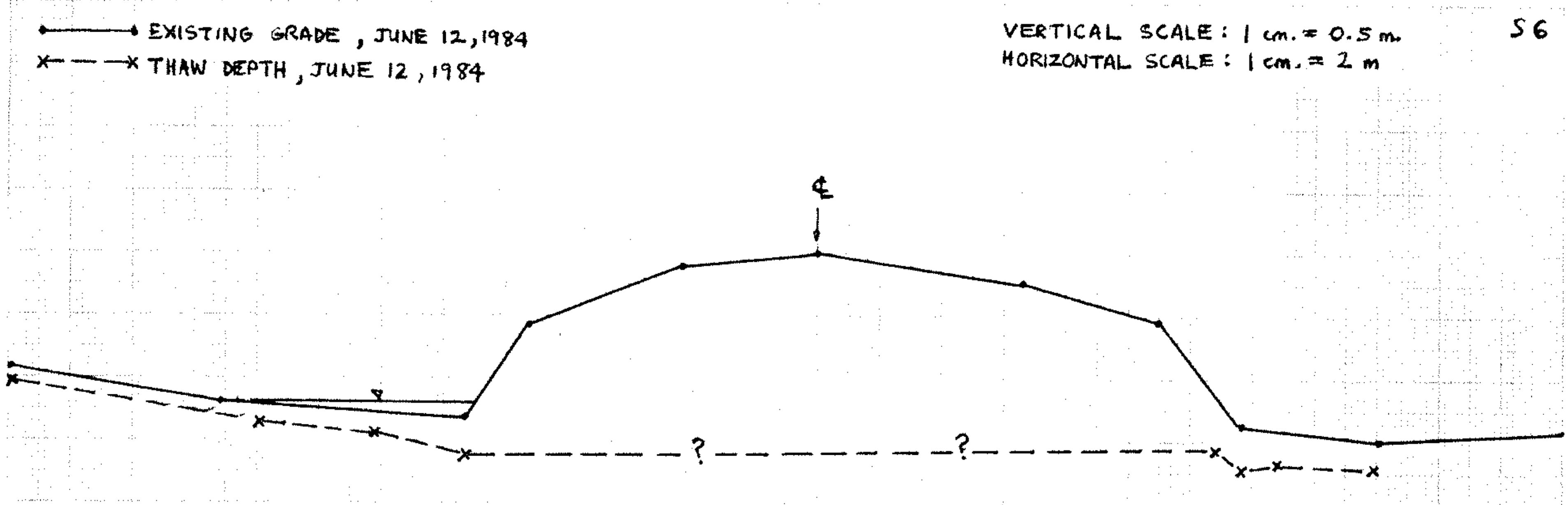
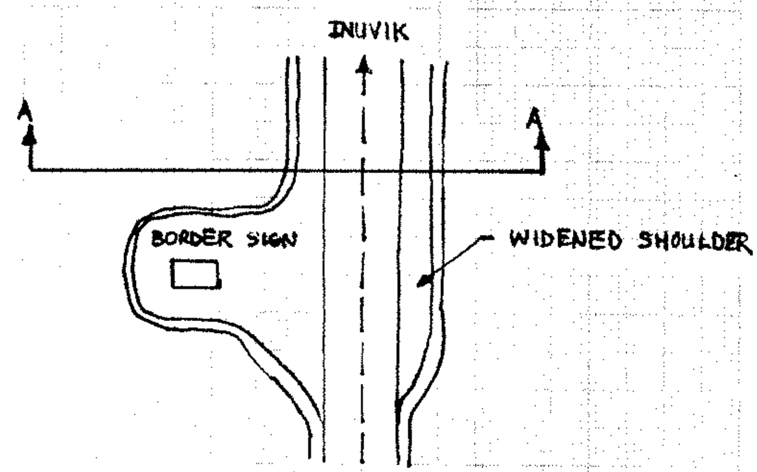


Figure 31. CROSS SECTION (AA) : DEMPSTER HIGHWAY (KILOMETER 0, N.W.T/YUKON BORDER)



PLAN: N.T.S.

than soil type are involved. Fine-grained soils are commonly troublesome due to their ice content, and occur in glaciated and unglaciated terrain alike.

CUTS

As discussed in the previous section entitled "Cut/Fill Sloughing", cuts in ice-rich soils tended to thaw and recede until a new organic covering became established well enough to provide insulation. Cuts in thaw-stable ice-poor soils remained at their original geometries as long as they were protected from the erosional effects of runoff, notwithstanding deep thaw penetration. Thermal degradation of cut backslopes has already been discussed and will not be further examined.

BRIDGES

There are seven bridges along the Dempster Highway ranging in size from small timber crib and span to multiple span steel pony truss structures. In almost all cases subsidence or thaw degradation was not visibly evident. Little or no differential subsidence between the approaches and the bridge decks was apparent. Abutments, gabions, rip-rap and other training structures remained intact and showed little or no sign of thermally induced subsidence, ice scour or fluvial erosion.

An overdeck Bailey bridge crosses the Blackstone River at about Kilometer 78, PLATE 4 illustrates the state of the bridge at the time of this investigation. Minor erosion had occurred on one abutment and "icing" of the river is an annual problem. The "river" is barely more than a creek at this location in the summer.

The small timber-crib bridge crossing Red Creek (Km 172) had no major problems. (No signs of icing.)

A 37 metre Baily bridge crosses Big Creek at Kilometer 198. The abutments showed no sign of distress although rip-rap and river training structures have been employed to protect them from high water conditions during the spring runoff.

About 1 kilometer north of Big Creek crossing is the Jeckell Bridge which crosses the Ogilvie River. PLATE 8 illustrates the bridge and rip-rap used on the abutments. No major problems with the bridge or ancilliary structures were observed at the time. It spans 110 metres and was constructed by the Canadian Armed Forces in the summer of 1971.

The south and north abutments of the Eagle River Bridge are shown in PLATES 14 and 15. This bridge was also constructed by the Armed Forces, (1977). The south abutment is very high and steep (sideslopes of 32 and 34.5 degrees to the horizontal). The ponded water on the left side seems to have contributed to minor slumping of the embankment shoulder. There was no evidence of damage to the bridge supports or gabion training structures. The river was deceptively quite and silt laden at the time.

CULVERTS

The problem of a culvert located within a shifting embankment located above a thermally sensitive foundation is not an easy one to solve in theory or practice. Culverts merit special attention because they act as thermal conduits unless specially insulated. Thermal degradation of their permafrost foundation may result in several problems: (1) differential settlement between the culvert center and extremities (2) differential settlement between the culvert invert and ditch level or (3) instability of the toe of the embankment. All of the above lead to reduced drainage efficiency either through culvert uncoupling, concavity, or section reduction.

To mitigate these efficeincy and erosion problems, a granular foundation was laid, a positive camber placed on the culvert, and the culvert invert was elevated 8 centimeters above ditch level. Many installations proved successful, but some were not (PLATES 2 and 3). Higher invert elevations sometimes caused water to pond adjacent to the embankment (PLATE 23). However, small culverts settled, bringing inlet elevations to ditch level (PLATE 1). The centers of the culverts often maintained their camber, but the flanks were somtimes bent upward if the embankment settled. Large culverts were usually placed above the original stream beds because further thaw settlement was unlikely.

Differential settlement between the embankment and the culverts occasionally occurred during construction due to consolidation of ice-rich backfill, or culvert subsidence from thermal degradation of underlying ice-rich soils. The latter case was recently observed (PLATE 25). Figure 32 was an attempt to illustrate the relationship between culvert location and embankment settlement. Clearly, the correlation is not exact -- nor should it be since several factors are involved: soil type, depth, ice content, location of taliks, and massive ground ice.

Of the culverts observed, only two had failed, but both had distorted and uncoupled as a result of sever distortion of the embankment shoulder. The embankments were apparently too steep; no evidence of ponded water at the toe was visible. The proportionally largest distortions (lateral, radial and longitudinal) were observed in the smaller culverts, where losses of section amounting to 25 percent were observed.

One objective of this study was to determine the amount of settlement at culvert locations. This was not possible since none are instrumented, original elevations are impossible to obtain, some culverts have subsequently been replaced and a thorough knowledge of soil conditions in the vicinity is necessary but not always available. However, rough estimates can be made, based on culvert inverts relative to ditch level, road undulations and ground ice thaw-out. Culverts (in service for about 10 years) overlying silty or clayey soils, such as throughout the MacKenzie Delta and the Taiga, Peel and Ogilvie valleys appear to have typically subsided about 0.2 metres. Settlements were considerably less in areas of alluvial, deltaic or glacial gravel deposits (Ogilvie Mountains) or meager surficial soils (Eagle plains, Richardson Mountains).

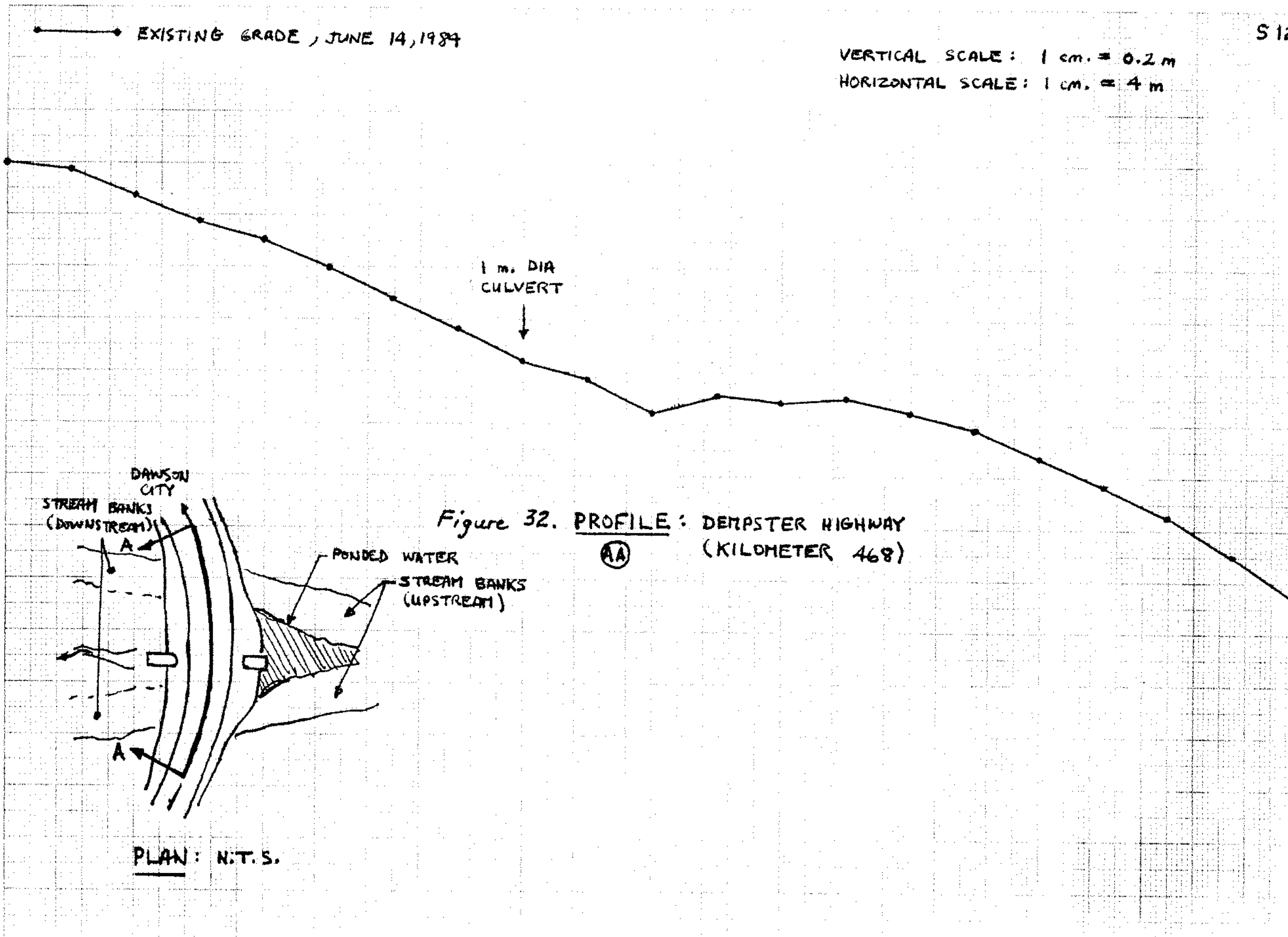


Figure 32. PROFILE (AA): DEMPSTER HIGHWAY (KILOMETER 468)

BORROW PITS

Borrow pit sizes ranged from less than an acre in area to tens of acres and were exploited for gravels, sands, and a variety of rock types (sandstone, shale, limestone, dolomite). The pits are spatially arranged at intervals of about 10 kilometers. Some of the trial pits had to be abandoned because of poor material quality or high ice contents. Most pits were well hidden from view, but in some cases this was not possible due to lack of topographic relief and vegetation.

Of the 30 or so borrow pits examined, virtually all had undergone thaw subsidence to some degree - some (PLATE 26) more than others. Massive ground ice was present even in the more competent rocks.

Some pit materials (PLATE 13) were more susceptible to erosion than others. Some pits were filling with water while vegetation was being re-established in others.

DITCHES

Permafrost degradation is most severe in the right-of-ways cleared by machine. Removal of native trees results in minor subsidence from increased thaw depths ($\sim$ 0.1 - 0.2 m extra), but new vegetation such as willows, labrador tea, sedge grass and horsetail grass has encroached most ditches. In some sections of highway the subsidence was 0.5 metres, with about as much increased thaw depth. Erosion is not a major problem except in areas of ice-rich soils with substantial relief. Local vegetation such as moss and grass tussocks provide micro relief which tends to intercept overland drainage. Tracking of equipment over the permafrost in winter results in some destruction of the organic mat, leading to subsidence and formation of trenches. These trenches eventually become overgrown or provide beneficial drainage channels on shallow gradients, but this practice is not adviseable on steep gradients where erosion is a potential hazard.

Embankment Heave

It was not immediately obvious whether heave of the embankment had indeed occurred or whether subsidence was the sole or major contributor to surface undulations. From studies of other highways both agents appear to be at work, but subsidence is the dominant factor.

Embankment Subsidence

GENERAL

There are numerous interrelated factors which affect the short and long term performance of an embankment placed on permafrost: topography; vegetation; soil type and ice content; thickness of the active permafrost layer; permafrost temperature; embankment height; shape; composition (mineralogy and grain size); homogeneity; and construction scheduling. It is therefore difficult in most cases to attribute subsidence to any single factor. Those factors which are known are presented as follows in an attempt to arrive at some general conclusions.

There are cases where differential subsidence can be attributed to specific factors such as ground ice melt-out as shown in PLATE 12. Distortions in the embankment surface seem to be pronounced at local topographic lows especially in the Northwest Territories and the unglaciated portions of the Yukon. This would imply either local thickening of compressible organic cover or increased ice content in topographic hollows, since compression of organics and thaw settlement of ice-rich ground are the major contributors to subsidence.

The portions of highway consistently in the best overall condition (shape, smoothness, geometrics) were: KM 0 - 88; 410 - 470; and 740 - 743. These were also the newest (410 - 470: PLATE 16) or the most recently reconstructed (0 - 85, 740 - 743) sections (1978 or newer). They all had heights, or average, which were greater than the older sections. Construction materials included sandy gravels, sandstone, competent shales, limestone and dolomite. Subgrade soil conditions varied from sandy gravel floodplain deposits at the south to silty sandy deltaic deposits at the north. Ground ice content varied considerably locally, but was actually seen in massive form at only a few sites. Vegetative cover varied in thickness from 0.1 to 1.0 metres.

The poorest section of highway (Km 88 - 128: PLATE 7) was also the oldest section (1960) and was constructed with a minimal embankment thickness with apparently little concern for geometrics. This section is currently being upgraded.

Other sections of highway are performing well generally but local problems developed as a result of lack of suitable borrow, icing and drainage problems, and thaw of ground ice.

FINE GRAINED MATERIALS AND STAGED CONSTRUCTION

A variety of materials have been employed or tested in the Dempster embankment and other highway trial embankments. Fine grained materials have been used as basal layers or as an enveloped core as a possible means of reducing quarried rock requirements. One such section was built at Km 714 (Mile 953.1) as shown in Figure 22. According to cross-sections presented by Cook and Huculak (1978), settlement and distortion (0.25 m at centerline) were 50 percent worse for the former embankment than an adjacent embankment constructed of gravelly sand with few fines. When observed in June of 1984, these embankments appeared to be performing as well as those constructed of quarried rock (little or no further settlement or distortion). It should be noted, however, that AESL (1972) recommended that fine grained materials are suitable for only special applications such as embankment cores; not for capping embankments, nor erosion control on sideslopes.

One opinion expressed by AESL (1972) regarding the use of fine-grained embankment materials was that best results were obtained when: (1) The first lift or core of fine-grained material was placed during winter; (2) The depth of fill as originally placed was over 4 feet; (3) The permafrost cover had not been disturbed; (4) The moisture content was relatively low. A 2 year staged construction procedure is recommended by AESL, especially for winter constructed highways. A minimal fill is placed (about 1 m thick) over the undisturbed organic terrain and allowed to thaw the following summer. No equipment should be used on the fill during the summer. The following winter additional fill is placed to raise the grade 1.5 m (about 5') above original ground. This fill thaws and subsides somewhat in the second summer. A capping material is placed later in the summer.

Cook and Huculak (1980) noticed poorer performance of embankments with initial 0.6 metre lifts than 1 metre lifts, but they contend that post-construction repairs of a lower final embankment is more cost effective than the extra capital expense of a higher design embankment. In its present state (1984), the highway is performing remarkably well, in spite of the relatively low embankment design height (1.5 m).

Based on observations and discussions with highway engineers at Whitehorse major subsidence in winter constructed embankments (using frozen materials) occurs in the first two or three years following completion and minor distortions persist for an additional two years. For summer construction, the major subsidence occurs in the first year or two, but immediate material quantity requirements are higher than winter constructed embankments.

ARTIFICIAL INSULATION

Studies have been and currently are being performed to measure subsidence of soil and/or rock embankments with and without artificial and natural insulation. Measures such as placing extra basal layers of peat (heat sink) or incorporating artificial insulation such as extruded one-stage expanded polystyrene boards (Esch, 1973, Gandahl, 1978, Johnston, 1983) foamed sulphur (Raymont, 1978), wood chips and/or foamed polyurethane plastics (Smith, Berg and Muller, 1973) with thicknesses varying from 0.10 to 0.28 m have been successful in reducing or preventing thaw penetration beneath embankments in "cold" permafrost areas (Continuous Permafrost) over periods of 5 to 10 years. These methods were less successful in areas of "warm" (Discontinuous) permafrost. Problems such as cracking, separation and moisture absorption in the artificial insulations have yet to be solved. Other aspects such as producing uniform quality extruded foams need to be refined. At present, it seems, artificial insulation is too expensive for major applications, but it is useful for local or specialized applications such as insulation of culverts or bridge abutments.

An insulated test embankment was constructed 22 km east of Inuvik along the Dempster Highway and was monitored by G.H. Johnston (1983) from 1972 to 1978. Four insulated sections and two uninsulated sections were constructed using gravel materials and 3 different thicknesses of extruded polystyrene insulation (0.05, 0.09, 0.115 m). The subgrade soils consisted of 0.15 to 0.6 m of organics overlying 3.0 to 4.5 m of silty clay (50 percent ice content) overlying silty, sandy till with ice lenses up to 0.01 m thick. During a 6 year period permafrost aggraded under the insulated sections but receded by as much as 0.6 m under the control (uninsulated) sections. There was 0.6 m of settlement under the control section, 0.3 m settlement under the section with 0.05 m thick insulation, and no settlement under the section with 0.09 m thick insulation. One control section was constructed during the winter with frozen materials whereas the other was constructed during the summer. Of the total settlement of 0.55 m in the winter constructed section, 65 percent occurred in the first thaw season. The summer constructed section settled a total of 0.30 m following construction, primarily as a result of compression of the organic cover. Thermal equilibrium was established by year 6. Settlement was greater at the shoulders than at the center of the embankment. Reduced embankment thickness (less insulation) and plowing of snow into the ditches during winter (heat removal inhibited) were likely responsible for this behavior.

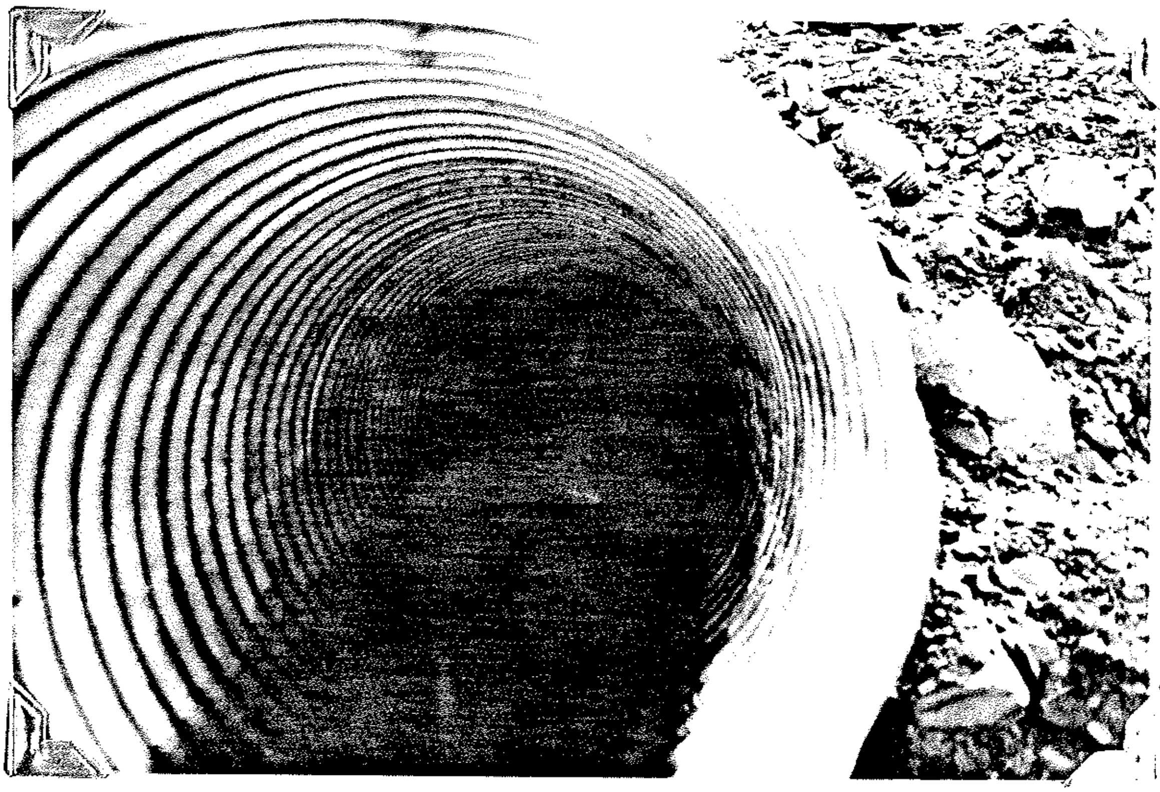


PLATE 33: Culvert Icing.

Drainage Disruption

PONDING

Natural surface drainage in permafrost terrain takes place in established stream and river channels and within the active permafrost layer. Stream flows can be intercepted by culverts installed at the time of embankment construction, but slope wash and subsurface seepage (within the active layer) are more difficult to accommodate since relevant hydrogeologic information is usually sparse or absent. Liberal use of culverts is the approach most often taken and has been moderately successful in the case of the Dempster Highway. Deformation of culverts (ends bent upward) occurred in places and placement of culverts above ditch level has lead to ponding of water adjacent to embankments in some localities (usually flat terrain, such as the MacKenzie delta), but these occurrences have not lead to any major problems with embankment performance.

Ditch blocks were sensibly located in most areas to intercept drainage parallel to the embankment to reduce erosion. Some areas required more ditch blocks than others (PLATE 22).

ICINGS (AUFEIS)

Culvert icings, such as the one shown in PLATE 33 were encountered at stream crossings and in slope wash localities. Indirect and direct methods have been employed to reduce icing problems and have generally met with good success. Steam pipes have been installed in some of the larger culverts. Semi-permeable and impermeable membranes and dykes have been constructed upslope of embankments to keep icing initiation away from the highway. Burlap fences (material hung between posts parallel to the highway) effectively slow down surface water movement enough that the water freezes to the burlap. Successive layers of water pond and freeze behind the burlap fence.

It was observed that round culverts contained the larger proportion of remnant icings than "arch" type culverts. This may be due to the fact that arch type culverts are generally larger and conduct larger spring discharges which remove any traces of icings early in the season.

Staggered culvert elevations for culvert "groups" are a useful innovation but also suffer from icing problems. The middle (elevated) culvert in PLATE 23 was still blocked by ice in June.

EROSION

In several locations equipment had operated along the ditches in the winter and ruts had developed during thaw in subsequent summers forming drainage channels. These channels had become overgrown with grasses, moss, and willow bushes by 1984. The overgrowth was beneficial (reduced erosion) and was not unsightly.

Road Trafficability

The majority of the highway was driveable at or near design speed (90 km/hr) but the older sections which are currently being upgraded (Km 88 to 126) were negotiable only at lower speeds. The surface course was performing well in most places albeit a little thin in several locations (Km 107, 142,158, 170, 255, 260, 272, 380, 626, 669). The black shale used as a running surface near Km 516 has not weathered well, and has resulted in a travel surface which is soft and slippery when wet.

"Dusting" of the running surface was not excessive but was generally associated more with the sandstone and limestone surfaces than other materials. The addition of calcium chloride aids in dust control but is rather expensive according to Macleod (1979) (over $500,000/ year for NWT).

The quality of quarried rock used in embankments varied from location to location, perhaps even within the same geologic formation of source material. For example, the quarried sandstone placed from Km 155 to 178 was more durable (resistant to physical breakdown) than that from Km 178 to 270.

D Conclusions

1. Construction in unglaciated permafrost terrain must take account of the fact that gravel supplies are less abundant than in glaciated terrain and the distribution of ice-rich fine-grained soils is different, but more predictable than in glaciated terrain.
2. There appears to be a benefit in choosing high altitude road alignments (following ridge crests) in unglaciated terrain because (a) borrow material is usually near the surface, (b) subsidence, creep and heaving problems are less than in wet, poorly drained areas (organics are likely thicker and fine-grained ice-rich soils more abundant) at lower altitudes, (c) mass movement areas such as soliflucted terrain, and scree slopes can be avoided, (d) snow tends to blow off the highway in winter, (e) massive beds of ground ice are not as common at high altitude as they are at low altitude, (f) it is not necessary to deal with meandering or braided rivers which erode their banks or switch channels.
3. The upper 1.5 to 2.5 metres of soil in glaciated and unglaciated terrain appears to be the most troublesome with regard to the presence of ground ice (wedges, beds, other forms of segregated ice) which is susceptible to thaw instability. Massive ground ice was found directly beneath the organic mat in glaciated terrain with wet fine-grained soils.
4. Embankment performance in the Yukon and Northwest Territories was generally good, especially considering the steep sideslopes (35°)

used for both low and high embankments. The poorest performance occured: (a) along the oldest road sections (1962) where low embankments were built with little concern for highway geometrics (b) where low embankments (less than 1.4 metre high) were used in flat, poorly drained terrain (c) where initial pads in staged winter construction were 0.6 metres or less in height.

5. Embankment performance was best: (a) along the newest sections (b) in floodplain deposits of sands and gravels (c) in well drained high altitude or rolling terrain (such as Eagle Plain Plateau).

6. Major subsidence and distortion of summer constructed embankments occurs in the first 2 or 3 years whereas substantial subsidence and/or distortion may occur up to year 5 in winter constructed embankments. Capital costs and material requirements are higher for the summer constructed embankments.

7. Thaw penetration beneath an average embankment (1.4 m high) 5 years after construction ranged from 0.1 to 1.0 metres while subsidence amounted to as much as 1.0 metre. Compression of the organic cover accounted for as much as 50 percent of observed subsidence.

8. The design embankment height of 1.37 metres (4.5') was not sufficient, in general, to prevent thaw of the subgrade, but provided adequate stability and performance.

9. Undulations occurred in embankments: (a) over 2.44 m (8') high, in some places, (b) at cut/fill transitions, (c) in polygonal ground terrain, (d) flat, poorly drained terrain.

10. Thaw degradation and sideslope movement was slightly greater where water ponded adjacent to the embankment.

11. Most embankment rock materials had good durability characteristics: Dolomite, sandstone, shale, limestone. Some shales broke down rapidly under traffic wear, becoming soft and slippery.

12. Artificial insulation is effective in reducing thaw penetration into "cold" permafrost, but is not cost effective at present.

13. Ancilliary embankment structures such as bridges, culverts, borrow pits and ditches are performing very well in general.

14. Despite these problems, which in most cases, are relatively minor, an all-weather road can be built in northern areas dominated by permafrost.

15. It appears that a period of 5 years after construction is required as a "settling in" period for the type of highway construction observed.

E Engineering Recommendations

Embankments

1. Embankments should not be constructed with steeper sideslopes than 35 degrees to the horizontal, otherwise soft shoulders result from cracking and sloughing.
2. A minimum embankment height of 1.5 metres is recommended for flat, poorly drained terrain.
3. Road widening via "cutting" down of embankments by as much as 0.3 metres is a plausible approach as long as a minimum embankment height of 1.4 metres can be retained and a suitably built and recompacted wear surface can be achieved. However, as much as 0.3 metres of subsidence may be expected in the long term after removal of an equivalent amount of embankment. This procedure is not recommended for fine-grained embankments, nor embankments in flat, poorly drained terrain.
4. Deep fills should be bermed to aid in supporting shoulders and to provide a place for vehicles to come to rest in emergencies. More guard rails should be employed for deep fills, as a safety alternative.
5. Staged 2 year construction is recommended whereby a minimum 1 metre embankment is placed in the winter, allowed to thaw and consolidate the following summer (no traffic), and an additional fill is placed the following winter which is allowed to thaw in the second summer, then capped with a wear surface.

6. A "five cut" grading technique should be employed where it is not presently in use. This will ensure that grade elevation is maintained and the wear surface will not be bladed to the shoulder or off the road (See Figure A1). This procedure is not recommended where soft shoulders occur or where sideslope stability is in question.

Culverts

1. Until a better approach, (or detailed hydrogeologic data) is available, the design philosophy employed by Public Works (Canada) is recommedned for highway construction in permafrost: To err on the greater side regarding number and size of culverts used.
2. Invert elevations of culvert inlets should be within 8 cm of ditch level, except where staggered groupings are used.
3. Where icings occur, culvert ends should be covered with burlap to prevent plugging with ice and oversize culverts should be used.
4. More use should be made of "off-take" ditches to channel water away from embankments in ice-rich terrain, but care should be taken to avoid potential erosion.
5. Flumes should be employed and checked annually where culvert outlets are greater than one metre above ditch level.
6. At stream crossings, culverts should be located away from the apex of concave vertical curves in the alignment. This will reduce the possibility of culvert damage during potential flood or high "runoff" periods. Figure A2 illustrates the concept. The Rock River crossing is an example of application of this technique.

Cuts

1. Cuts in ice-rich and thaw-unstable materials should be avoided if at all possible.
2. Cuts in thaw stable soils should have flat backslopes to reduce potential snow accumulation during winter.
3. Wider ditches should be employed in rock cuts to catch loose or spalling rock.

Borrow Pits

1. Pits should be placed on the lee side of hills if possible (out of sight).
2. If pits are necessary on the visible side of hills, they should be contoured to the lay of the land to make them less visible and vegetation should be used as a screen, where possible.
3. Borrow pits should be asymmetric in shape to give them a more natural appearance.

River Training Works

1. To eliminate embankment erosion problems, at the Ogilvie River, for instance, river training works should be employed to direct the river into its previous channels (Km 248).

Icings

1. The use of burlap or "fibre fences" upslope of road embankments are recommended for control of icing formation.

F Environmental Comments

The practice of tree removal in proposed alignments is a satisfactory approach because revegetation will be relatively quick albeit by new plant communities. Willow bushes, sedge grass and cotton tussocks will occupy areas where black spruce and sphagnum moss previously co-existed.

During the three weeks spent in the Yukon and Northwest Territories our research group encountered 16 animals on or near the highways: one brown bear (near Carmacks); one black bear (Eagle Plain's Lodge); 2 hares; 1 red fox; 1 ptarmigan; 1 grouse; 3 ducks (near Chapman Lake); and a beaver brood. Using binoculars to examine the mountains and surrounding hills at several locations along the Dempster Highway revealled no signs of wild life.

G Economic Considerations

The previously stated recommendations are suggested for premium quality embankment performance. Admittedly, this can be an expensive construction and maintenance alternative. If berming and staged 2 - year winter construction are not economically feasible, then "regrading" and single year winter construction are potential alternatives, but a lower quality and/or higher maintenance embankment will be the result.

VI RESEARCH NEEDS

A Thermal Degradation and Subsidence

Little information is readily available, aside from small test road sections, regarding embankment settlement due to changes in thermal regime in permafrost. Opportunities for such research are at hand. By strategically establishing survey control points and stations along a newly constructed highway, and providing continuous, regular monitoring, useful data could be obtained regarding subsidence and "creep" movements. It would be necessary to keep accurate and up-to-date maintenance records.

B River Training Works

The use of techniques for training rivers and streams should be examined more seriously for use in permafrost regions.

VIII REFERENCES

Associated Engineering Services Ltd. (1972), "Highway Deisgn Concept, Construction Practices and Materials: MacKenzie and Dempster Highways".

Cook, D., Huculak, N.A., (1980), "Evaluation of Design/Construction on Long term Embankment Performance: MacKenzie-Dempster Highway", Public Works Canada, Western Region.

"Dempster Highway Corridor Planning Project (Background Analysis)", (1982), Indian and Northern Affairs Department.

"Dempster Highway Interim Management Plan", (1978), prepared by The Dempster Highway Working Group: Governments of Yukon and N.W.T.

Esch, D.C., (1973), "Control of Permafrost Degradation beneath a roadway by Subgrade Insulation", 2nd Int. Conf. on Permafrost, p. 608 - 622.

Gandahl, R., (1978), "Some aspects of the Design of Roads with Boards of Plastic Foam", 3rd Int. Conf. on Permafrost, p. 791 - 797.

Huculak, N.A., et. al, (1978), "Development of the Dempster Highway North of the Arctic Circle", 3rd Int. Conf. on Permafrost, p. 798 - 805.

Hudson. J., (1984), Personal Communication

Johnston, G.H., (Ed), (1981) "Permafrost Engineering Design and Construction", John Wiley and Sons, Toronto, 540 p.

Johnston, G.H., (1983), "Performance of an Insulated Roadway on Permafrost, Inuvik, N.W.T.", 4th Int. Conf. on Permafrost, p. 548 - 551.

MacLeod, W.G., (1979), "The Dempster Highway", Canadian Arctic Resources Committee.

Prest, V.L., (1969), "Retreat of Wisconsin and Recent ice in North America", Geol. Surv. Can., Map 1257A.

Raymont, M.E.D., (1973), "Foamed Sulphur Insulation for Permafrost Protection", 3rd Int. Conf. on Permafrost, p. 865 - 869.

Richardson, N.W., Sauer, E.K., (1975), "Terrain Evaluation of the Dempster Highway across the Eagle Plain and along the Richardson Mountains, Yukon Territory", Can. Geotech. Journal, Vol. 12 (3), p. 296 - 319.

Rutter, N., (1984), Personal Communication.

Schultz International Ltd., (1972), "Environmental Impact Study of the Dempster Highway: Folio 1 or 2.

Smith, N., Berg, R., Muller, L., (1973), "The use of Polyurethane Foam Plastics in the construction of Expedient Roads on Permafrost in Central Alaska", 2nd Int. Conf. on Permafrost, p. 736 - 745.

"The Milepost Magazine", (1984), 25th Anniversary Edition, p. 464 - 468.

Thomson, R.S., (1984), Personal Communication.

VIII APPENDIX

A1 Geologic Time Chart

B2 Public Works Canada Specifications

C Figures A1, A2

THE GEOLOGIC-TIME SCALE AND THE GEOLOGIC COLUMN

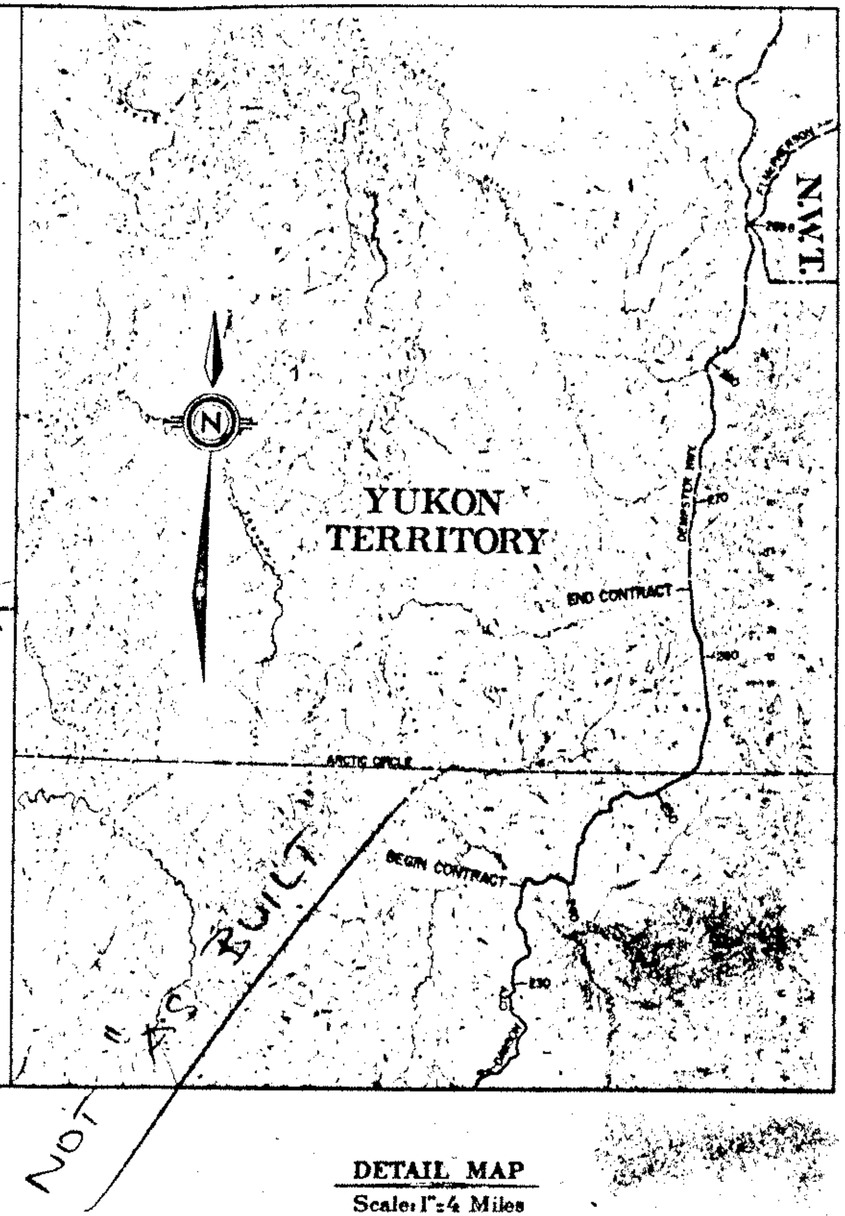
Era[a]	Period	Epoch	Duration (Millions of yr)	Before present (Millions of yr)	Major physical events
Cenozoic	Quaternary	Holocene	0.01		Continental glaciations
		Pleistocene	1.5–2		
	Tertiary	Pliocene	5–5.5		
		Miocene	19		
		Oligocene	11–12		
		Eocene	15–17		Yellowstone volcanics begin
		Paleocene	11–12		
				65	
Mesozoic					Beginning of Rocky Mountains
	Cretaceous		71		
					North America separates from Eurasia
	Jurassic		54–59		
	Triassic		30–35		North America begins to separate from Africa Atlantic basin originates
				225	
Paleozoic	Permian		55		Climax of Appalachian mountain building
	Pennsylvanian[d]		45		
	Mississippian[d]		20		
	Devonian		50		
	Silurian		35–45		
	Ordovician		60–70		Beginning of Appalachian Mountains
	Cambrian		70		
				570	
Precambrian	Precambrian[c]				Oldest dated rocks (±3.8 billion yr ago) Origin of earth (±4.5 billion yr ago)

[a] Duration on approximately uniform time scale.

[b] This column does not give the complete time range of the forms listed. For example, fish are known from pre-Silurian rocks and obviously exist today; but when the Silurian and Devonian rocks were being formed, fish represented the most advanced form of animal life.

The following specifications are for selected portions of the Dempster Highway: Mile 236.8 - 264.7; and Mile 289.77 to 343.88 (Yukon/NWT border to Fort McPherson).

KEY MAP
Scale: 1"=120 Miles



DETAIL MAP
Scale: 1"=4 Miles

Public Works Canada

Pacific Region
Design / Construction
Civil Engineering-Yukon

Project

DEMPSTER HIGHWAY
MILE 236.8-264.7
CONSTRUCTION

Project Manager: S.P. BAKER, P. Eng.

Designed: A. HUDSON, P. Eng. Date: 18 Nov. 1972
Reviewed: S.P. BAKER, P. Eng. Date: 19 Oct 1973

Approved: Project Manager

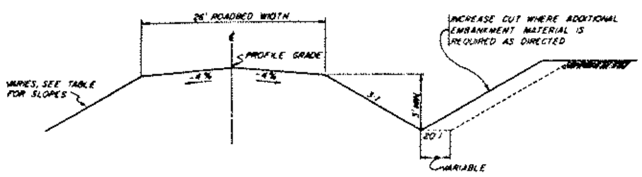
Reduced to 1/2 Scale

Revisions: Sept / Oct 1976

TS No.	Drwg. No. 7106	Sheet 1 of 33

DRAWING NO	VERT SCALE	HORIZ SCALE	SHEET NO	TOTAL
5111	N.T.S.	N.T.S.	61	61

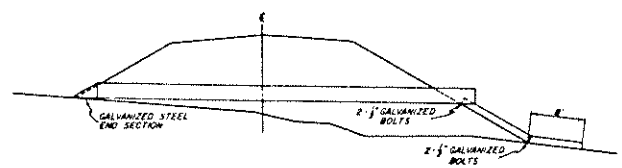
EARTH SECTION



HEIGHT OF CUT OR FILL	SLOPE
0' - 5'	3:1
5' - 10'	2:1
OVER 10'	1½:1

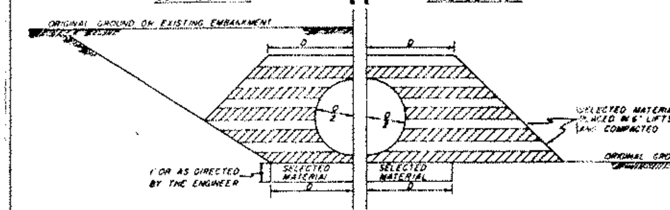
NOTE: 1. SHOULDERS WIDENED 2' WHEN FILLS EXCEED 10'
2. OUTSIDE SHOULDER WIDENED 1' WHEN CURVES EXCEED 3°
3. WHERE CUT EXISTS IN NON-USABLE MATERIAL, A SUBCUT OF 3' OR AS DIRECTED BY THE ENGINEER, SHALL BE MADE BENEATH FINAL GRADE AND BACKFILLED WITH SUITABLE FILL MATERIAL

TYPICAL FLUME INSTALLATION



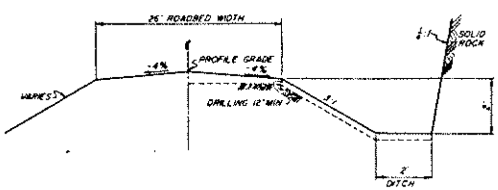
NOTE: 1. FLUME LENGTHS ON PLANS INCLUDE 8' OF FLUME TO BE LAID ON THE GROUND
2. GALVANIZED STEEL END SECTIONS FITTED AT BOTH ENDS WHEN NO FLUMING REQUIRED, CULVERTS 30"-72" DIAM.

INSTALLATION OF PIPE, STRUCTURAL PLATE PIPE & PIPE ARCH CULVERTS

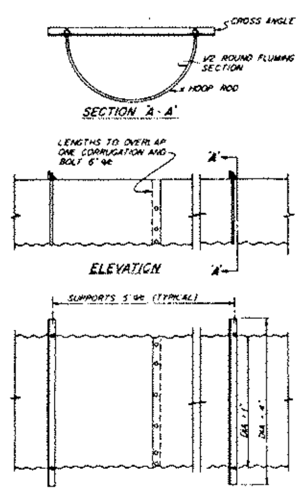


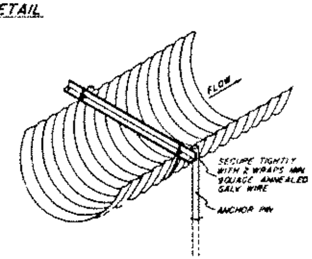
NOTE: MINIMUM FINISHED COVER AND MINIMUM COVER DURING CONSTRUCTION TO BE AS RECOMMENDED BY THE CORRUGATED PIPE INSTITUTE.

ROCK SECTION



METAL FLUME DETAIL





ASSEMBLY - FLUME ON GROUND

NOTE: FLUMING - CORRUGATED METAL HALF ROUND, SIZE AND GUAGE AS SPECIFIED, AASHO M 36
SIZE OF CROSS ANGLE & HOOP ROD

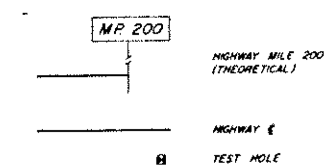
LENGTH	ANGLE	ROD
OVER 30"	2"x2"x¼"	½"

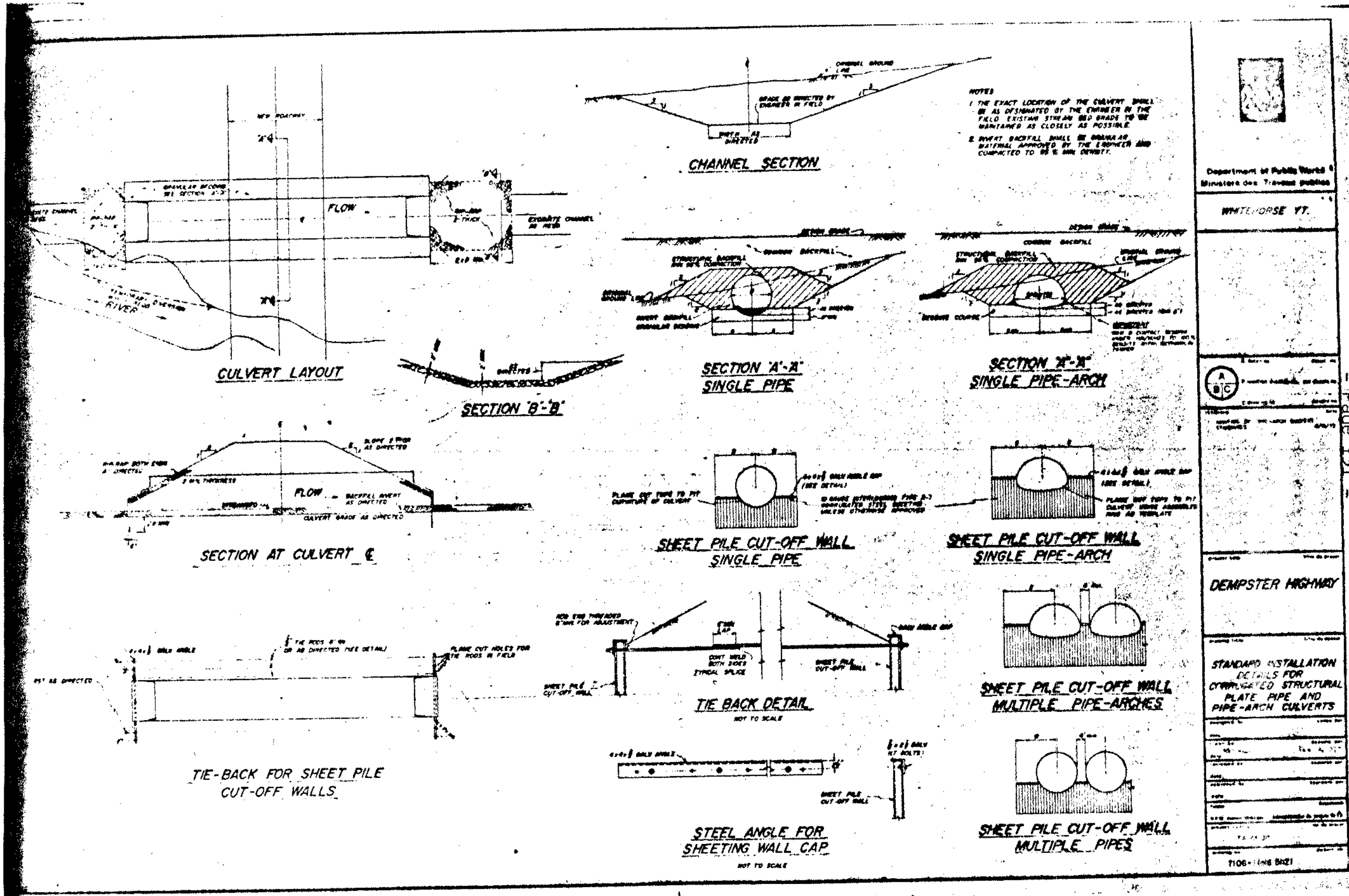
ANCHOR PINS 1" ⌀ x 8' MIN FROM REINFORCING STEEL STOCK
CONSTRUCTION - SEE SPECS

ABBREVIATIONS

℄	CENTRE LINE
PI	POINT OF INTERSECTION OR PLASTICITY INDEX
T	TANGENT DISTANCE (CIRCULAR CURVE)
BC	BEGINNING OF CURVE
EC	END OF CURVE
Δ	DELTA OR DEFLECTION ANGLE
BM	BENCH MARK
D	DEGREE OF CURVATURE
L	LENGTH OF CURVE (CIRCULAR CURVE)
VPI	VERTICAL POINT OF INTERSECTION
LVC	LENGTH OF VERTICAL CURVE
BVC	BEGINNING OF VERTICAL CURVE
EVC	END OF VERTICAL CURVE
CMP	CORRUGATED METAL PIPE
CSPP	CORRUGATED STRUCTURAL PLATE PIPE
MP	MILE POST (THEORETICAL)
TH	TEST HOLE
NP	NIL PLASTICITY
NTS	NOT TO SCALE

LEGEND



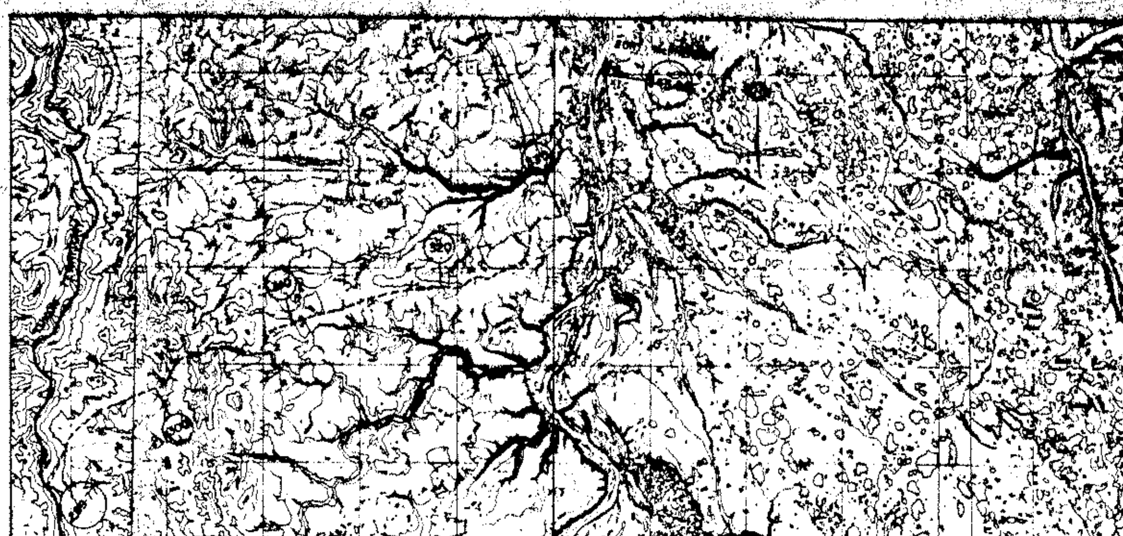

NOTES
1 THE EXACT LOCATION OF THE CULVERT SHALL BE AS DESIGNATED BY THE ENGINEER IN THE FIELD. EXISTING STREAM BED GRADE TO BE MAINTAINED AS CLOSELY AS POSSIBLE.
2 INVERT BACKFILL SHALL BE GRANULAR MATERIAL APPROVED BY THE ENGINEER AND COMPACTED TO
CHANNEL SECTION
CULVERT LAYOUT
FLOW
RIVER
SECTION 'B'-'B'
SECTION 'A'-'A' SINGLE PIPE
SECTION 'A'-'A' SINGLE PIPE-ARCH
SECTION AT CULVERT ℄
SHEET PILE CUT-OFF WALL SINGLE PIPE
SHEET PILE CUT-OFF WALL SINGLE PIPE-ARCH
SHEET PILE CUT-OFF WALL MULTIPLE PIPE-ARCHES
SHEET PILE CUT-OFF WALL MULTIPLE PIPES
TIE-BACK FOR SHEET PILE CUT-OFF WALLS
TIE BACK DETAIL
NOT TO SCALE
STEEL ANGLE FOR SHEETING WALL CAP
NOT TO SCALE
Department of Public Works
Ministère des Travaux publics
WHITEHORSE YT.
DEMPSTER HIGHWAY
STANDARD INSTALLATION DETAILS FOR CORRUGATED STRUCTURAL PLATE PIPE AND PIPE-ARCH CULVERTS

PLAN INDEX

DRAWING No 85925-76 SHEETS 1-59

DRAWING RW 1J3-4-1017 SHEETS 60-76

TOTAL SHEETS 76



SCALE 1" = 4 MILE (APPROX.)

Department of Public Works
Ministère des Travaux publics

WESTERN REGION

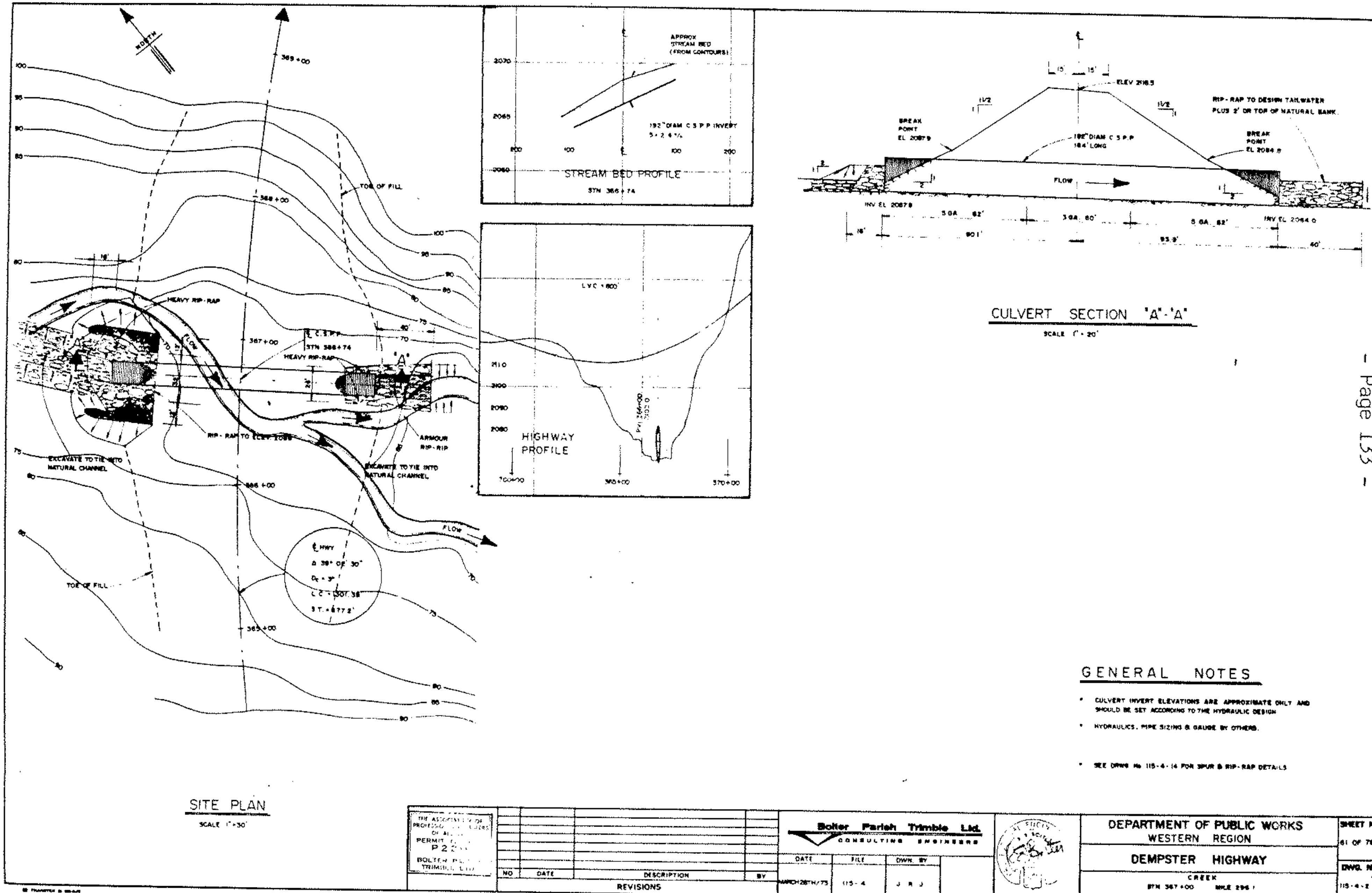
DEMPSTER HIGHWAY
MILE 259-77-343-88
CLEARING, GRADING & DRAINAGE

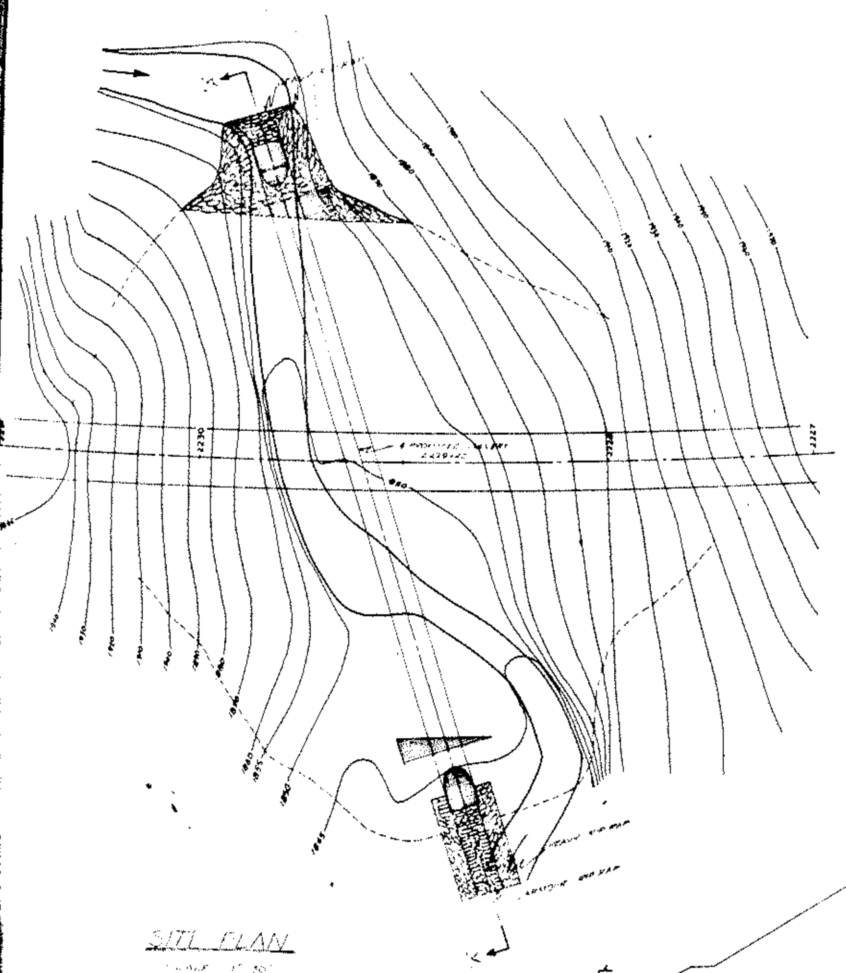
R. STRONSON

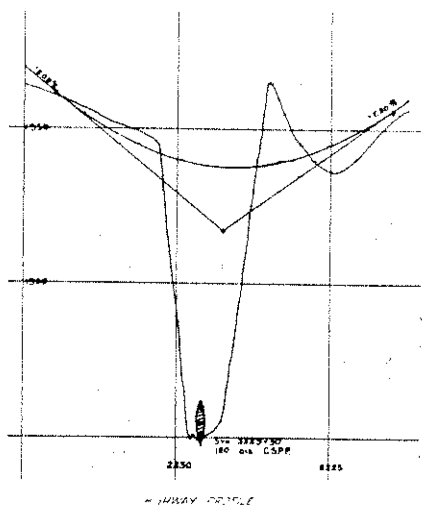
APRIL 1974

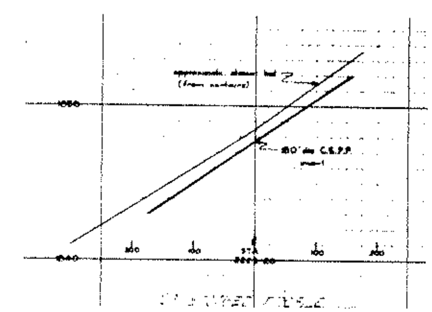
85925

85925-76 1 OF 76


SITE PLAN
SCALE 1"=30'
NORTH
TOE OF FILL
HEAVY RIP-RAP
FLOW
℄ C.S.P.P.
STN 366+74
RIP-RAP TO ELEV. 2088
ARMOUR RIP-RIP
EXCAVATE TO TIE INTO NATURAL CHANNEL
℄ HWY
Δ 39° 08' 30"
Dc = 3°
STREAM BED PROFILE
STN 366+74
APPROX STREAM BED (FROM CONTOURS)
192" DIAM C.S.P.P. INVERT
HIGHWAY PROFILE
LVC = 800'
CULVERT SECTION "A"-"A"
SCALE 1" = 20'
ELEV 2118.5
BREAK POINT EL 2087.9
BREAK POINT EL 2084.8
192" DIAM C.S.P.P. 184' LONG
RIP-RAP TO DESIGN TAILWATER PLUS 2' OR TOP OF NATURAL BANK
INV EL 2087.9
INV EL 2084.0
FLOW
GENERAL NOTES
• CULVERT INVERT ELEVATIONS ARE APPROXIMATE ONLY AND SHOULD BE SET ACCORDING TO THE HYDRAULIC DESIGN
• HYDRAULICS, PIPE SIZING & GAUGE BY OTHERS.
• SEE DRWG № 115-4-14 FOR SPUR & RIP-RAP DETAILS
REVISIONS
NO
DATE
DESCRIPTION
BY
Bolter Parish Trimble Ltd.
CONSULTING ENGINEERS
DATE
FILE
DWN. BY
MARCH 28TH/73
115-4
J.R.J.
DEPARTMENT OF PUBLIC WORKS
WESTERN REGION
DEMPSTER HIGHWAY
CREEK
STN 367+00
SHEET №
61 OF 78
DWG. №
115-4-2

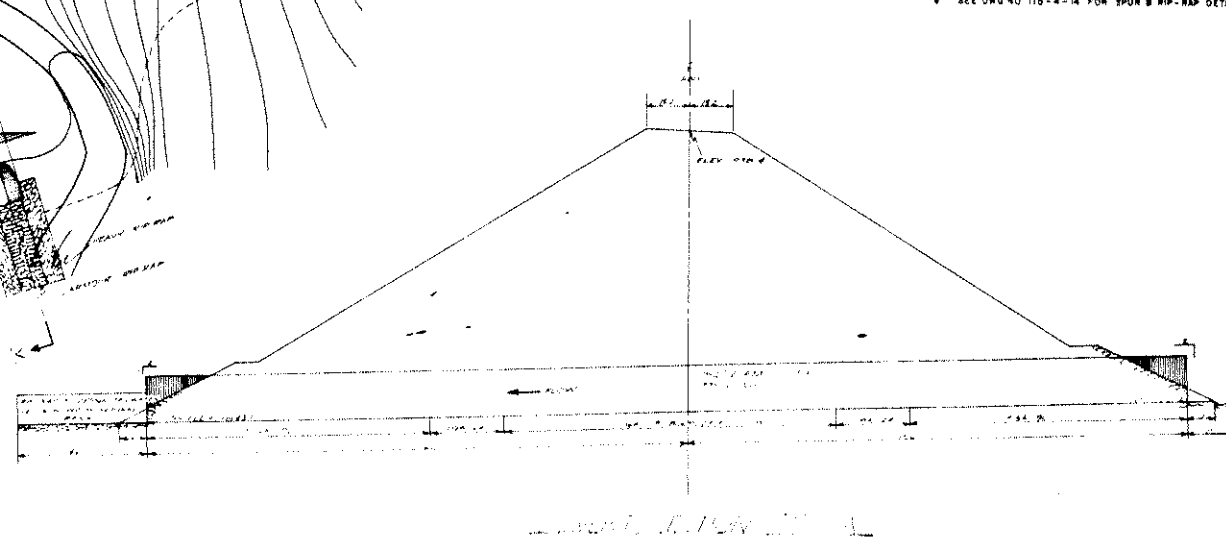






GENERAL NOTES

- SEE DWG NO. 115-4-18 FOR HOLD DOWN DETAILS
- SEE DWG NO 115-4-14 FOR SPUR & RIP-RAP DETAILS



Department of Public Works
Ministère des Travaux publics

WESTERN REGION

project title: CSPP SUPPLY
MILE 290-344

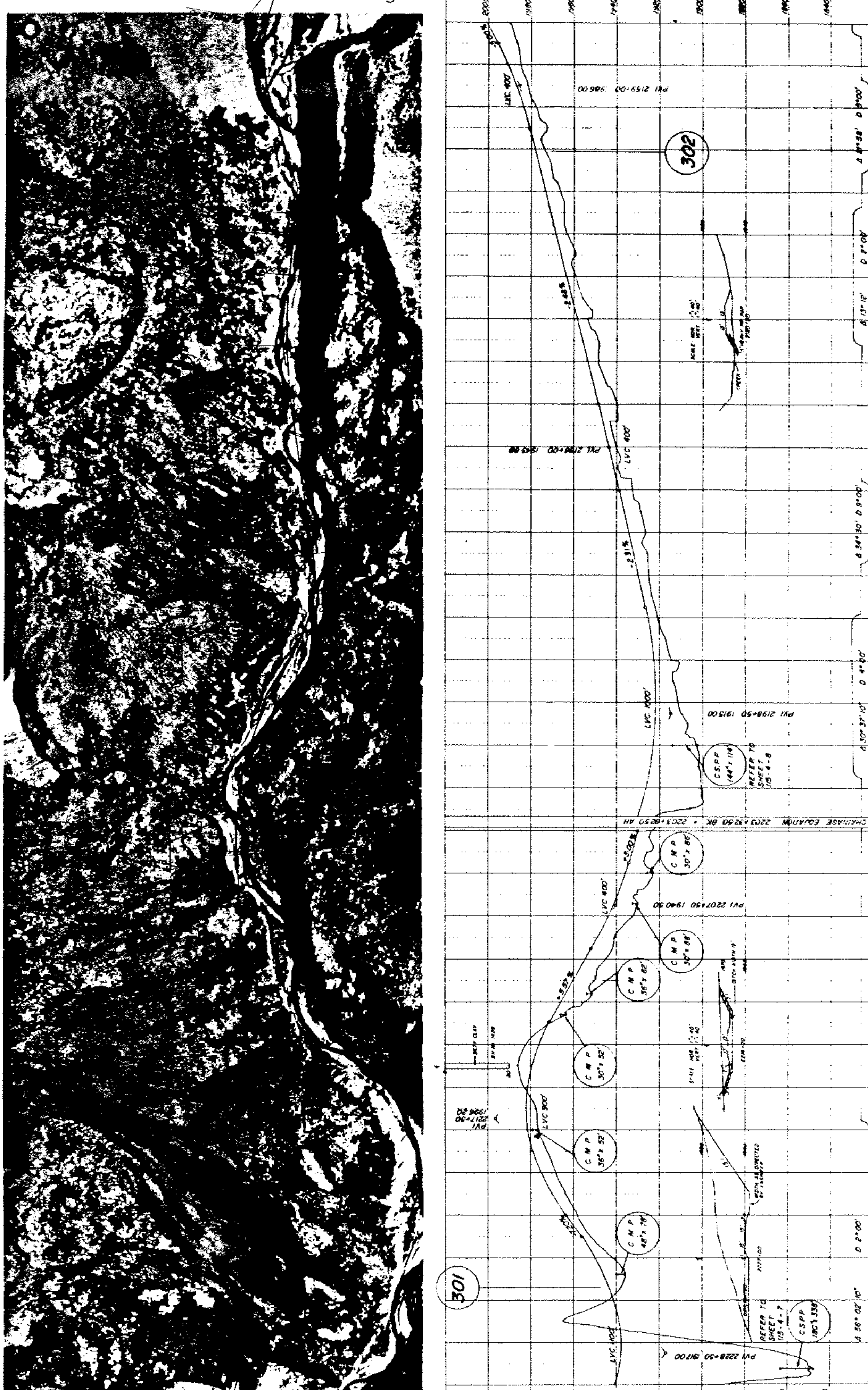
drawing title: CREEK STA 2229+20
MILE 309.9

date: 22/10/73
date: 22/10/73

project number: 85925

drawing no. 115-4-7

SHEET No.66 OF 76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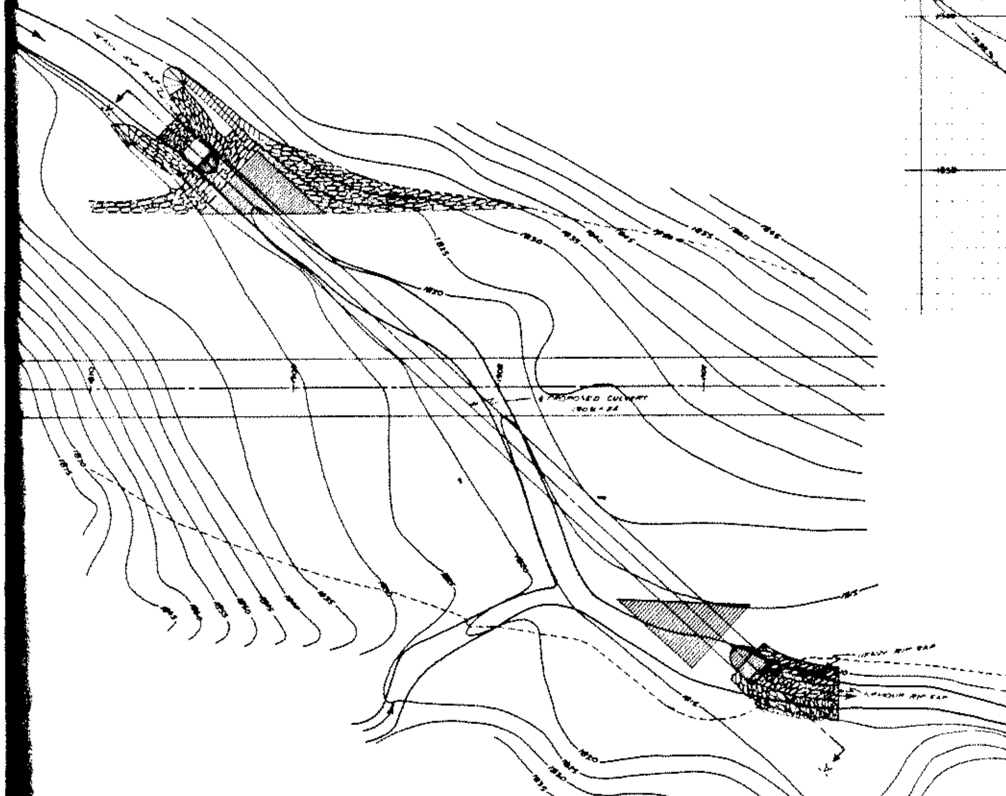
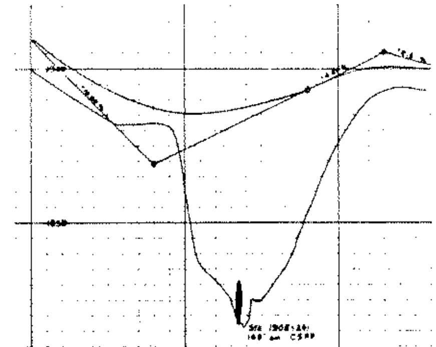
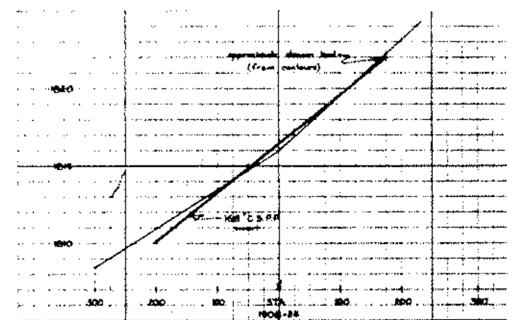


SITE PLAN

SCALE 1"=30'



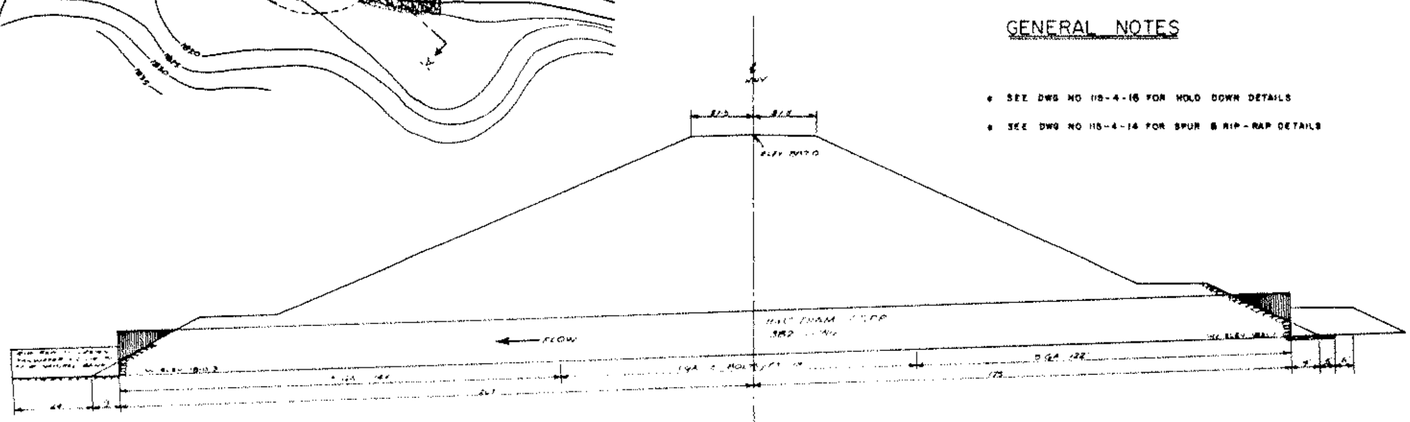
HIGHWAY PROFILE



STREAMBED PROFILE

GENERAL NOTES

- SEE DWG NO 118-4-16 FOR HOLD DOWN DETAILS
- SEE DWG NO 118-4-14 FOR SPUR & RIP-RAP DETAILS



CULVERT SECTION 'A'-'A'

Department of Public Works
Ministère des Travaux publics

WESTERN REGION

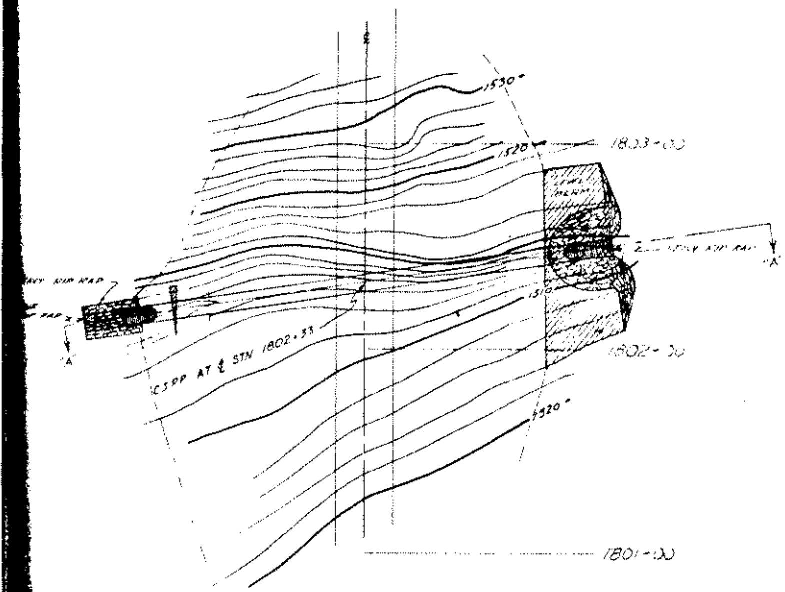
CSPP SUPPLY
MILE 290-344

CREEK STN 1908+24
MILE 308.7

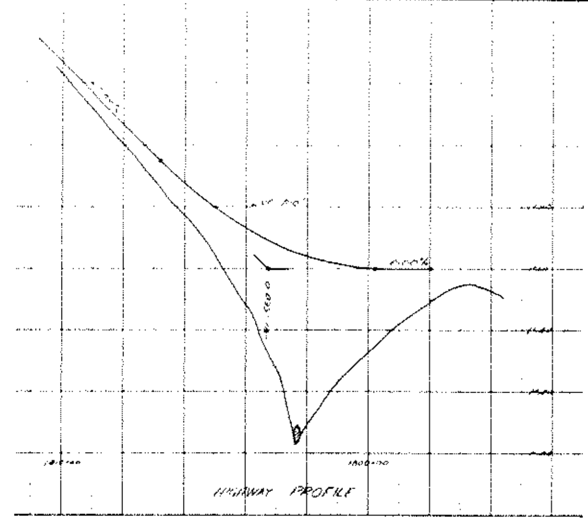
designed by DRF & DRK
date 22/10/73
drawn by DRF
date 22/10/73

project number 85925

drawing no 118-4-10
SHEET No. 59 OF 76



SITE PLAN
SCALE 1"=30



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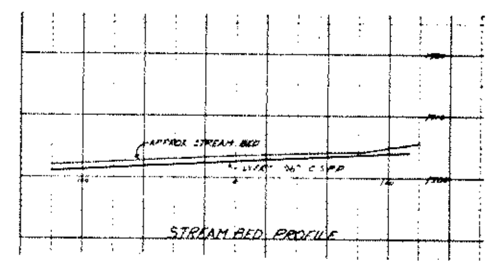

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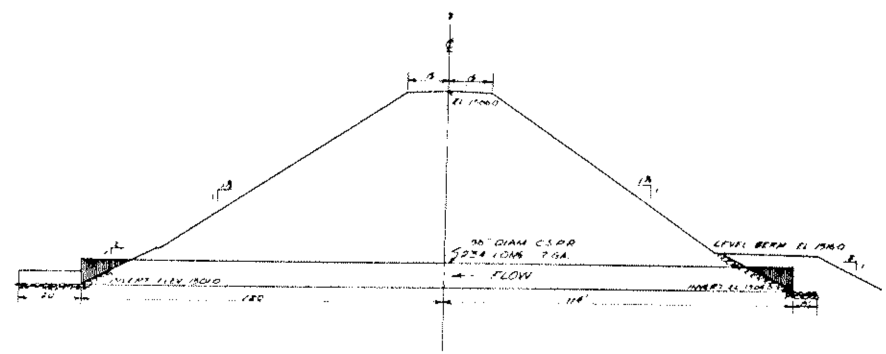

CULVERT SECTION 'A' - 'A'
SCALE 1"=20'

GENERAL NOTES

SEE DWG No 115-4-14 FOR SPUR & RIP RAP DETAILS

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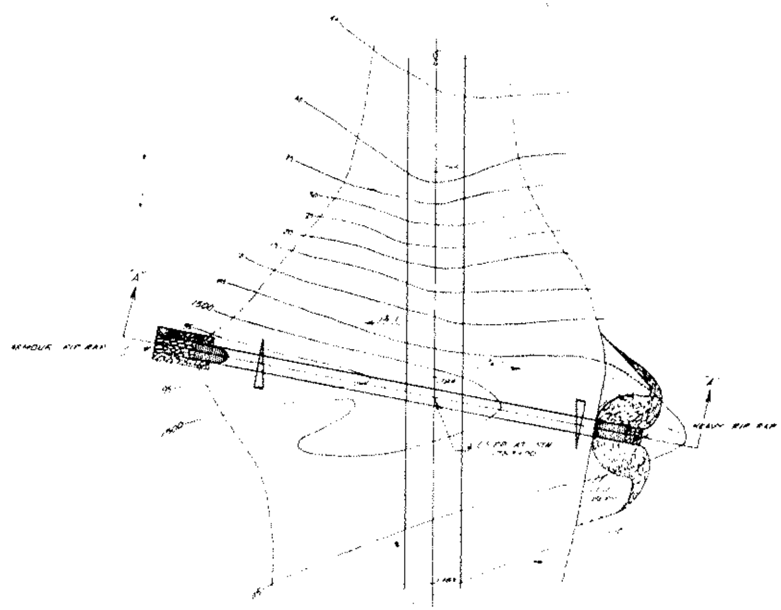
WESTERN REGION

- Page 137

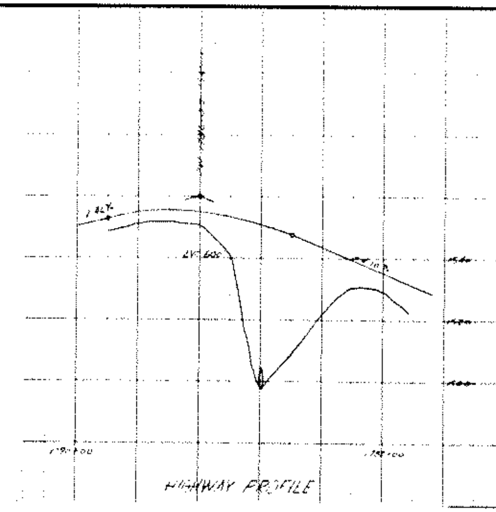
CSPP SUPPLY
MILE 290-344

STN 1802+30
MILE 308.8

designed by DRF & GRA
date 2/4/74
drawn by WTA
date 2/4/74
85925
drawing no. 115-4-11
SHEET No. 70 OF 78



SITE PLAN
SCALE 1:30



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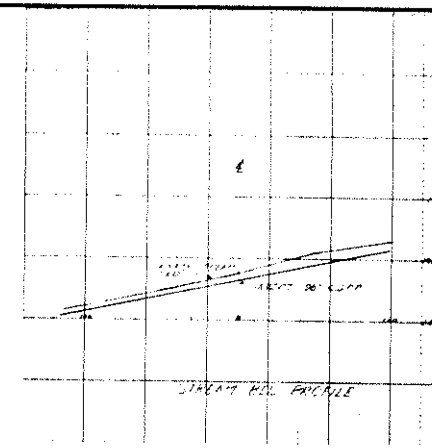

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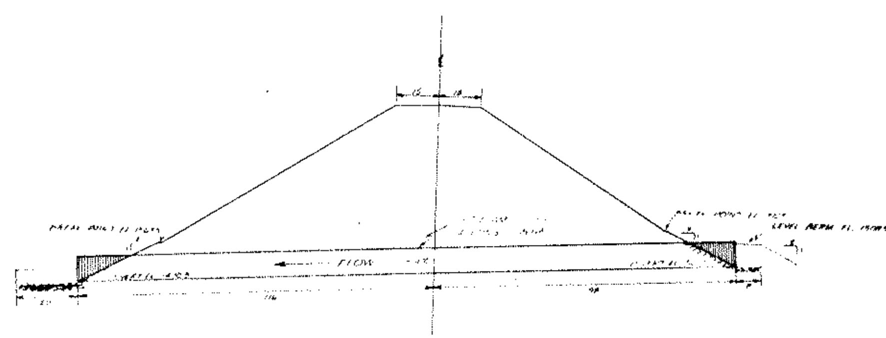


CULVERT SECTION 'A-A'

GENERAL NOTES

SEE DWG No 115-4-14 FOR SPUR & RIP RAP DETAILS

Public Works Canada / Travaux publics Canada

WESTERN REGION

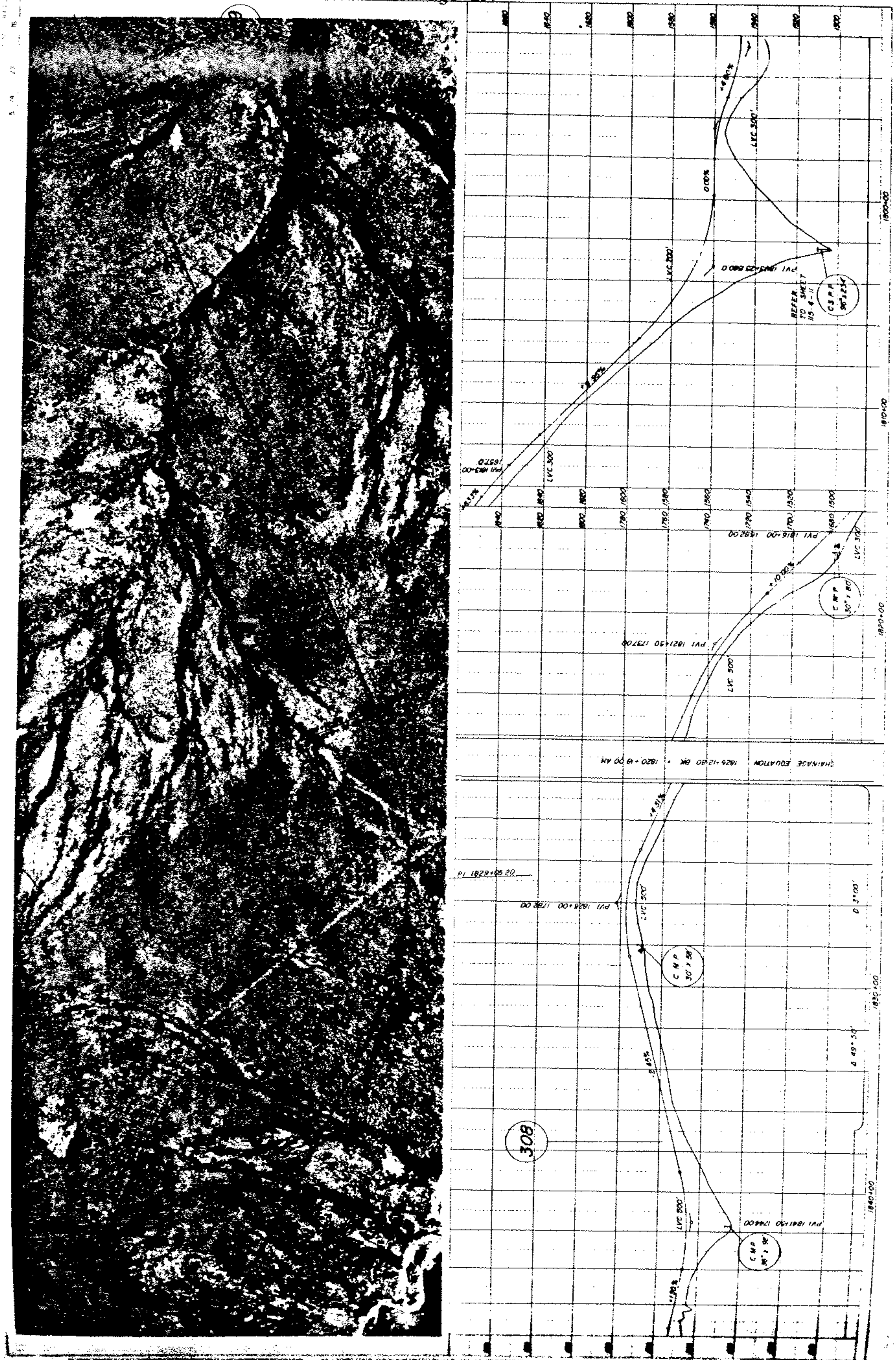
CSPP SUPPLY
MILE 290-344

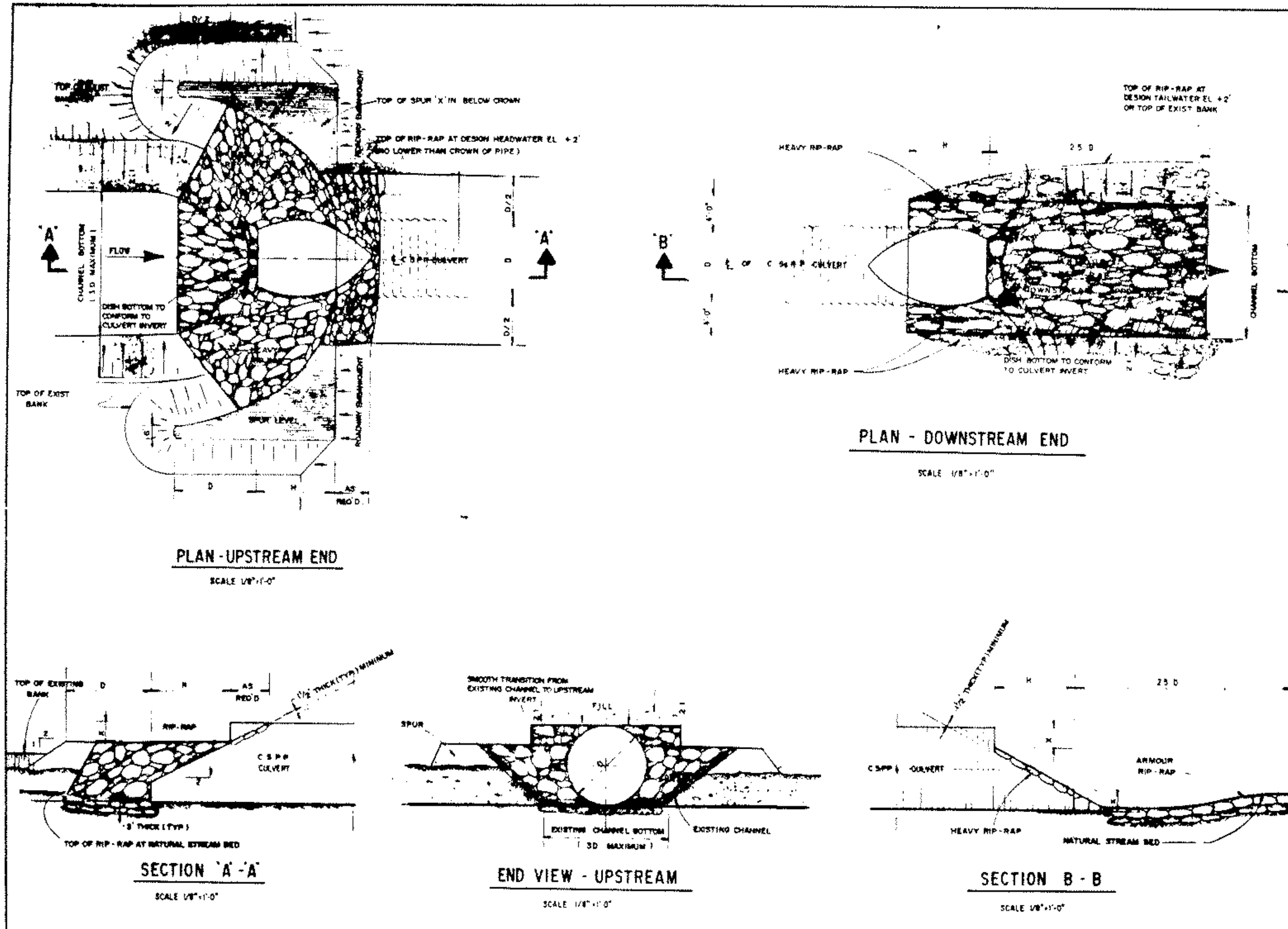
CREEK STN 1783+90
MILE 309.1

date 2/4/76

drawing no. 115-4-12

SHEET No 71 OF 78





GENERAL NOTES

- RIP-RAP DIMENSIONS ARE SCHEMATIC ONLY. ACTUAL DIMENSIONS DEPEND ON CONDITIONS AND TOPOGRAPHY AT EACH INDIVIDUAL SITE AND ARE TO BE DETERMINED IN THE FIELD

STN.	MILE	'D' FT.	'H' FT.	'X' IN
505+80	294.8	14	12	54

Boller Parish Trimble Ltd. Consulting Engineers

DATE	FILE	DWN. BY
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NO	DATE	DESCRIPTION	BY

REVISIONS

THE ASSOCIATION OF PROFESSIONAL ENGINEERS OF ALBERTA PERMIT NUMBER P220 BOLTER PARISH TRIMBLE LTD.

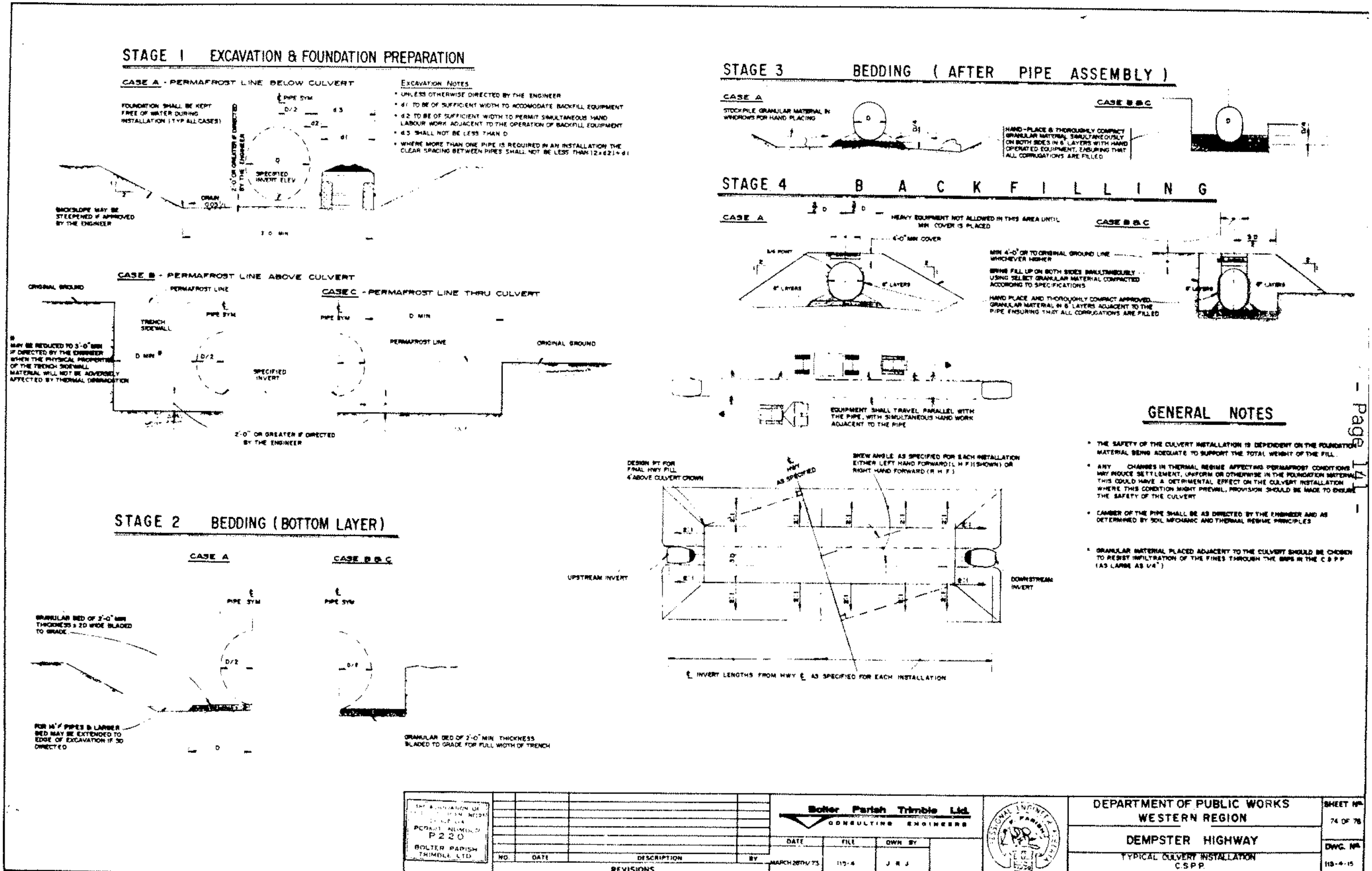
DEPARTMENT OF PUBLIC WORKS
WESTERN REGION

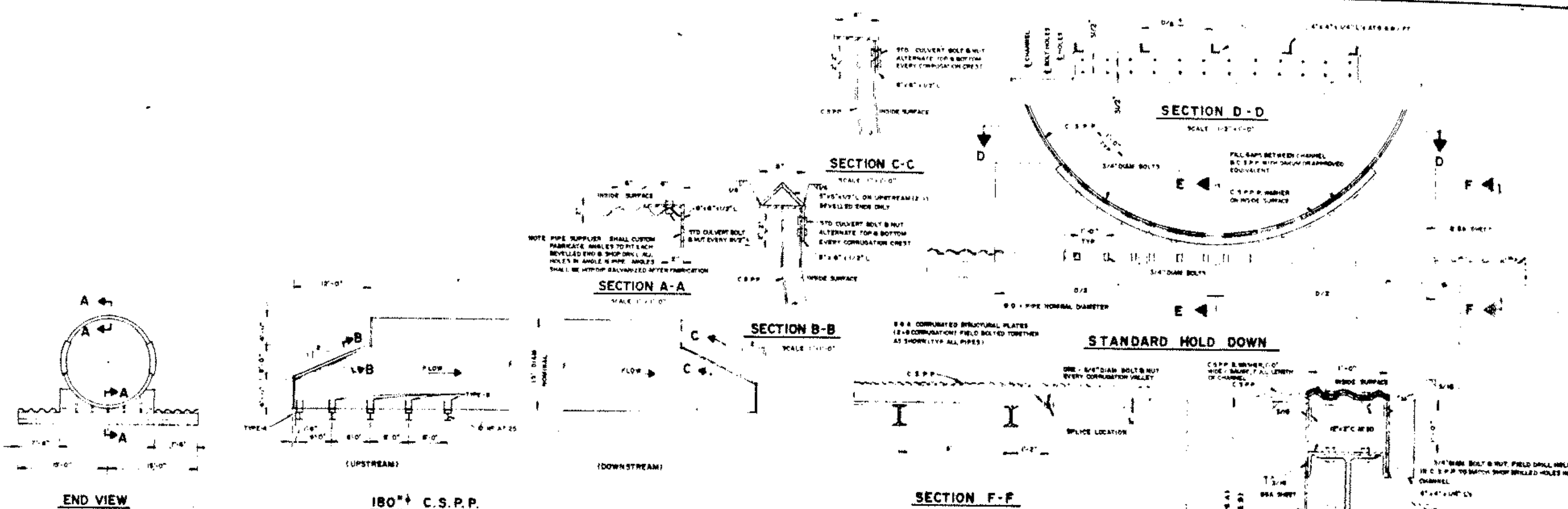
DEMPSTER HIGHWAY

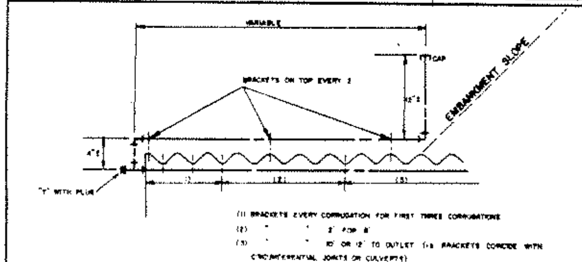
TYPICAL RIP-RAP END TREATMENT

SHEET NO. 73 OF 75

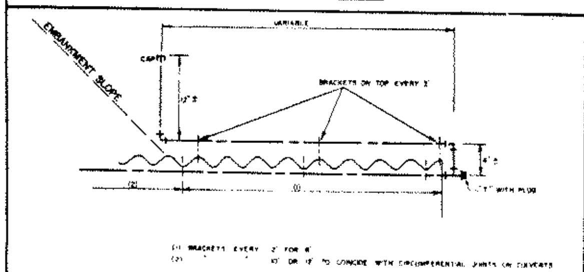
DWG. NO. 119-4-14


STAGE 1 EXCAVATION & FOUNDATION PREPARATION
CASE A - PERMAFROST LINE BELOW CULVERT
FOUNDATION SHALL BE KEPT FREE OF WATER DURING INSTALLATION (TYP ALL CASES)
BACKSLOPE MAY BE STEEPENED IF APPROVED BY THE ENGINEER
2'-0" OR GREATER IF DIRECTED BY THE ENGINEER
SPECIFIED INVERT ELEV
DRAIN
PIPE SYM
D/2
d3
d2
d1
D
Excavation Notes
• UNLESS OTHERWISE DIRECTED BY THE ENGINEER
• d1 TO BE OF SUFFICIENT WIDTH TO ACCOMODATE BACKFILL EQUIPMENT
• d2 TO BE OF SUFFICIENT WIDTH TO PERMIT SIMULTANEOUS HAND LABOUR WORK ADJACENT TO THE OPERATION OF BACKFILL EQUIPMENT
• d3 SHALL NOT BE LESS THAN D
• WHERE MORE THAN ONE PIPE IS REQUIRED IN AN INSTALLATION THE CLEAR SPACING BETWEEN PIPES SHALL NOT BE LESS THAN (2×d2)+d1
CASE B - PERMAFROST LINE ABOVE CULVERT
ORIGINAL GROUND
PERMAFROST LINE
TRENCH SIDEWALL
PIPE SYM
D MIN
D/2
SPECIFIED INVERT
* MAY BE REDUCED TO 3'-0" MIN IF DIRECTED BY THE ENGINEER WHEN THE PHYSICAL PROPERTIES OF THE TRENCH SIDEWALL MATERIAL WILL NOT BE ADVERSELY AFFECTED BY THERMAL DEGRADATION
2'-0" OR GREATER IF DIRECTED BY THE ENGINEER
CASE C - PERMAFROST LINE THRU CULVERT
PIPE SYM
D MIN
PERMAFROST LINE
ORIGINAL GROUND
STAGE 2 BEDDING (BOTTOM LAYER)
CASE A
PIPE SYM
D/2
GRANULAR BED OF 2'-0" MIN THICKNESS x 2D WIDE BLADED TO GRADE
FOR 14'Ø PIPES & LARGER BED MAY BE EXTENDED TO EDGE OF EXCAVATION IF SO DIRECTED
D
CASE B & C
PIPE SYM
D/2
GRANULAR BED OF 2'-0" MIN THICKNESS BLADED TO GRADE FOR FULL WIDTH OF TRENCH
STAGE 3 BEDDING (AFTER PIPE ASSEMBLY)
CASE A
STOCKPILE GRANULAR MATERIAL IN WINDROWS FOR HAND PLACING
D
D/4
CASE B & C
HAND-PLACE & THOROUGHLY COMPACT GRANULAR MATERIAL SIMULTANEOUSLY ON BOTH SIDES IN 6" LAYERS WITH HAND OPERATED EQUIPMENT, ENSURING THAT ALL CORRUGATIONS ARE FILLED
STAGE 4 BACKFILLING
CASE A
HEAVY EQUIPMENT NOT ALLOWED IN THIS AREA UNTIL MIN. COVER IS PLACED
4'-0" MIN COVER
6" LAYERS
EQUIPMENT SHALL TRAVEL PARALLEL WITH THE PIPE, WITH SIMULTANEOUS HAND WORK ADJACENT TO THE PIPE
CASE B & C
MIN 4'-0" OR TO ORIGINAL GROUND LINE WHICHEVER HIGHER
BRING FILL UP ON BOTH SIDES SIMULTANEOUSLY USING SELECT GRANULAR MATERIAL COMPACTED ACCORDING TO SPECIFICATIONS
HAND PLACE AND THOROUGHLY COMPACT APPROVED GRANULAR MATERIAL IN 6" LAYERS ADJACENT TO THE PIPE ENSURING THAT ALL CORRUGATIONS ARE FILLED
6" LAYERS
DESIGN PT FOR FINAL HWY FILL 4' ABOVE CULVERT CROWN
HWY AS SPECIFIED
SKEW ANGLE AS SPECIFIED FOR EACH INSTALLATION EITHER LEFT HAND FORWARD (L H F) (SHOWN) OR RIGHT HAND FORWARD (R H F)
UPSTREAM INVERT
DOWNSTREAM INVERT
INVERT LENGTHS FROM HWY ℄ AS SPECIFIED FOR EACH INSTALLATION
GENERAL NOTES
• THE SAFETY OF THE CULVERT INSTALLATION IS DEPENDENT ON THE FOUNDATION MATERIAL BEING ADEQUATE TO SUPPORT THE TOTAL WEIGHT OF THE FILL.
• ANY CHANGES IN THERMAL REGIME AFFECTING PERMAFROST CONDITIONS MAY INDUCE SETTLEMENT, UNIFORM OR OTHERWISE IN THE FOUNDATION MATERIAL. THIS COULD HAVE A DETRIMENTAL EFFECT ON THE CULVERT INSTALLATION WHERE THIS CONDITION MIGHT PREVAIL, PROVISION SHOULD BE MADE TO ENSURE THE SAFETY OF THE CULVERT
• CAMBER OF THE PIPE SHALL BE AS DIRECTED BY THE ENGINEER AND AS DETERMINED BY SOIL MECHANIC AND THERMAL REGIME PRINCIPLES
• GRANULAR MATERIAL PLACED ADJACENT TO THE CULVERT SHOULD BE CHOSEN TO RESIST INFILTRATION OF THE FINES THROUGH THE GAPS IN THE C S P P (AS LARGE AS 1/4")
PERMIT NUMBER P220
BOLTER PARISH TRIMBLE LTD
NO. DATE DESCRIPTION BY
REVISIONS
Bolter Parish Trimble Ltd.
CONSULTING ENGINEERS
DATE MARCH 28TH/73
FILE 115-4
DWN BY J R J
DEPARTMENT OF PUBLIC WORKS
WESTERN REGION
DEMPSTER HIGHWAY
TYPICAL CULVERT INSTALLATION C S P P
SHEET No 74 OF 78
DWG. No 115-4-15

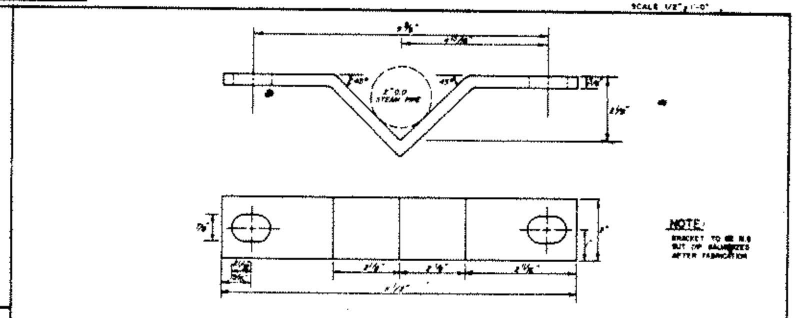




STEAM PIPE – INLET END



STEAM PIPE – OUTLET END



BRACKET DETAIL

NOTES:

1. PIPE & COUPLINGS TO BE 2" CLASS 40 GALVANIZED STEEL
2. HOLES REQUIRED FOR BRACKETS, OTHER THAN THOSE ON CIRCUMFERENTIAL CULVERT JOINTS, SHALL BE FIELD DRILLED
3. RISERS AT INLET TO BE BUTTED TO END OF CULVERT

GENERAL NOTES

- DESIGN – HOLD DOWNS HAVE BEEN DESIGNED TO RESIST A BUOYANT FORCE EQUAL TO ONE-HALF PIPE VOLUME BUOYANCY
- ALL STEEL EXCEPT C.S.P.P. SHALL CONFORM TO C.S.A. G40.12
- ALL NON-GALVANIZED STEEL SHALL RECEIVE ONE SHOP COAT OF RED LEAD IRON OXIDE PAINT CONFORMING TO C.G.S.B. 1 GP-106(A) TYPE 8
- BOLTS SHALL CONFORM TO A.S.T.M. A 325, GALVANIZED TO CONFORM TO A.S.T.M. A153

NO.	DATE	DESCRIPTION	BY
2	OCT. 19/73	MYLAR FROM 115-4-11	
1	OCT. 4/73		

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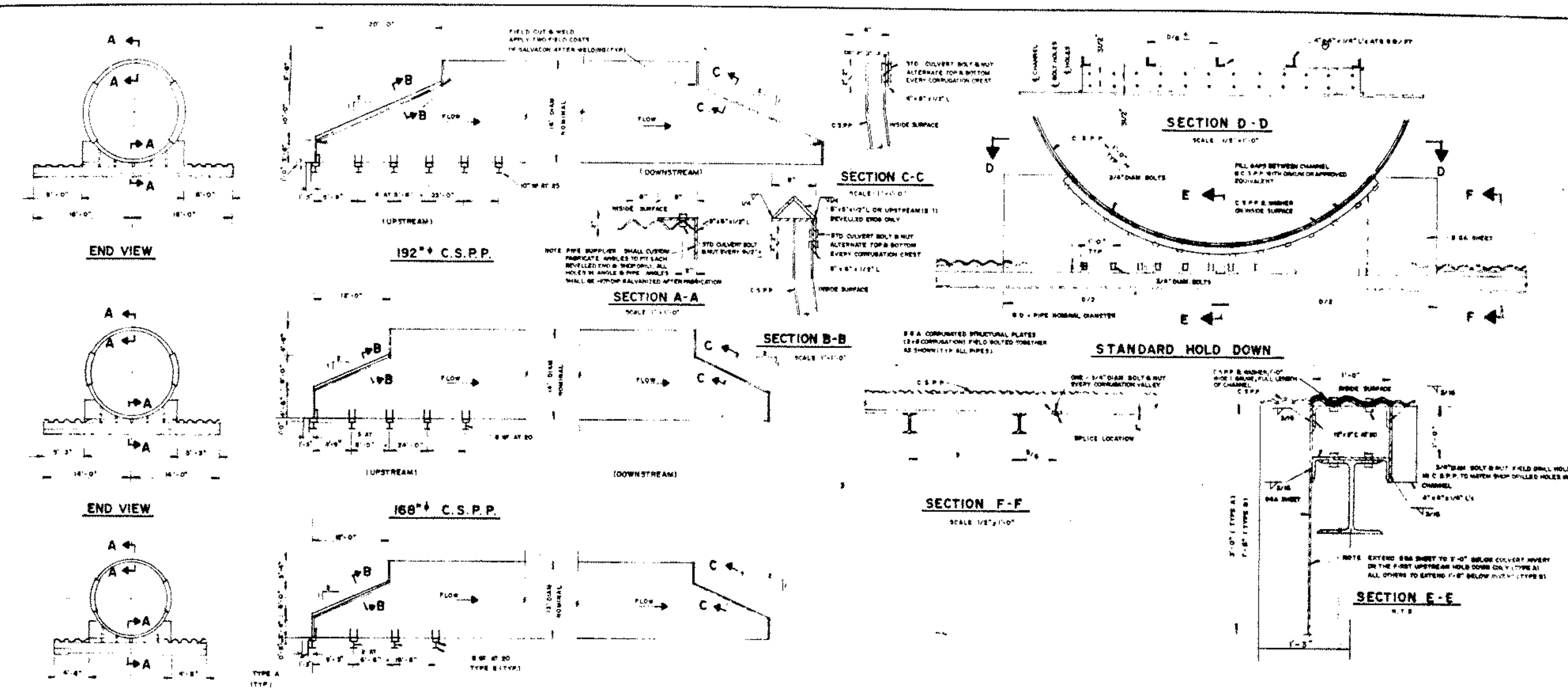
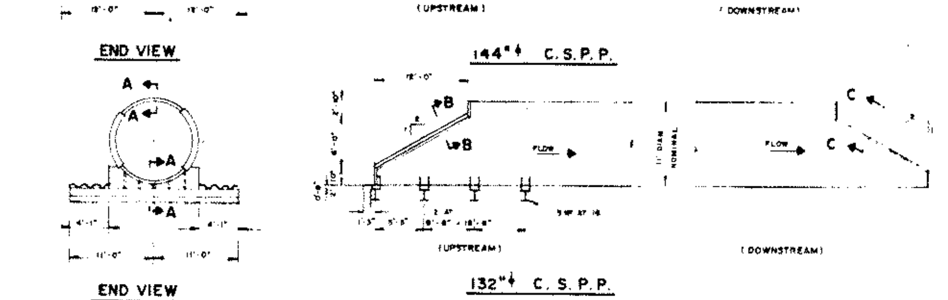
DEPARTMENT OF PUBLIC WORKS
WESTERN REGION

DEMPSTER HIGHWAY

15' DIA. C.S.P.P.
END TREATMENT DETAILS (ALL STEEL)
STEP-BEVELLED ENDS

SHEET No. 78 OF 78
DWG. No. 115-4-11





GENERAL NOTES

- DESIGN — HOLD DOWNS HAVE BEEN DESIGNED TO RESIST A BUOYANT FORCE EQUAL TO ONE-HALF PIPE VOLUME BUOYANCY
- ALL STEEL EXCEPT C.S.P.P. SHALL CONFORM TO C.S.A. G 40 12
- ALL NON-GALVANIZED STEEL SHALL RECEIVE ONE SHOP COAT OF RED LEAD IRON OXIDE PAINT CONFORMING TO C.G.S.B. 1 GP-166(A) TYPE 3.
- BOLTS SHALL CONFORM TO A.S.T.M. A 325, GALVANIZED TO CONFORM TO A.S.T.M. A153

HOLD DOWN DETAILS

STEP-BEVELLED ENDS

NO.	DATE	DESCRIPTION	BY
		REVISIONS	

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CONSULTING ENGINEERS

DATE	FILE	DWN BY
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DEPARTMENT OF PUBLIC WORKS
WESTERN REGION

DEMPSTER HIGHWAY

END TREATMENT DETAILS (ALL STEEL)
STEP-BEVELLED ENDS

SHEET No 75 OF 78

DWG. No 113-4-16

16

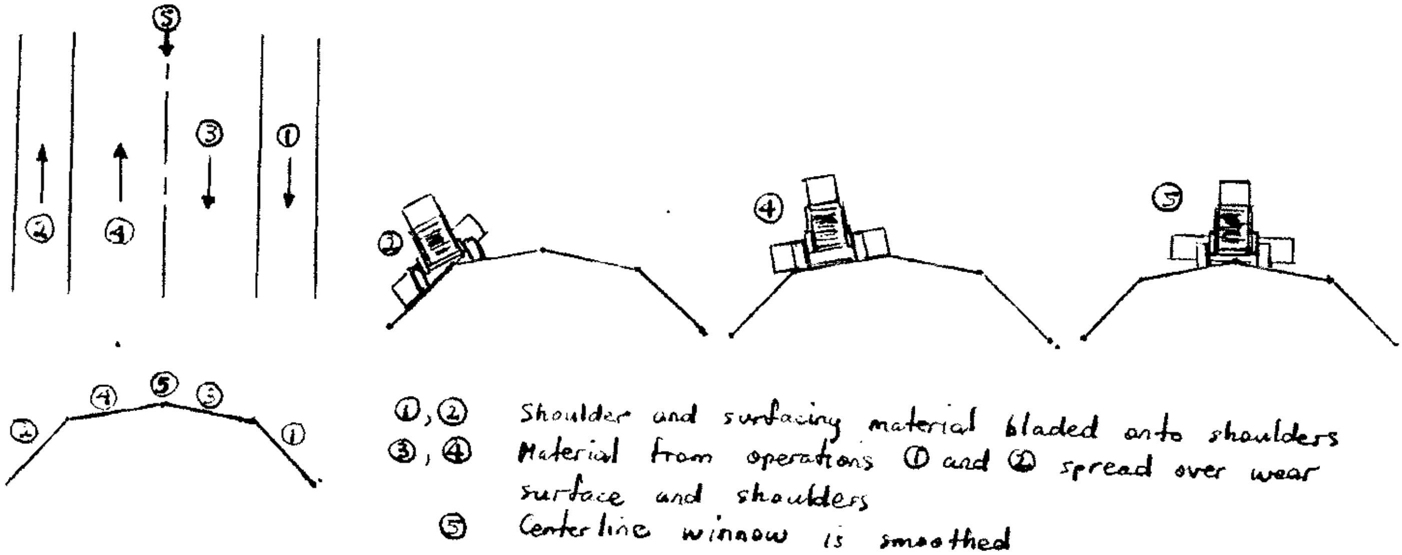

FIGURE A1 : "Five Cut" Grading Technique

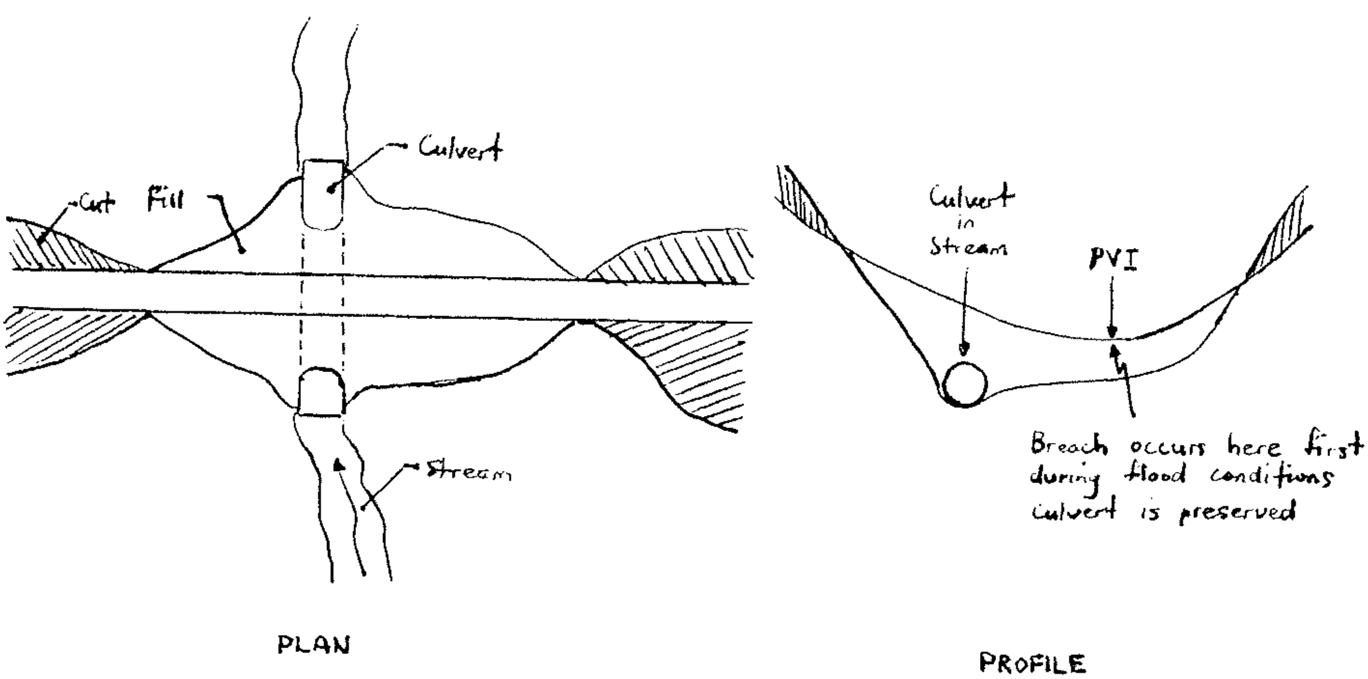

FIGURE A2 : Embankment Design