Proceedings of the Workshops on Subsea Permafrost: November 18, 1985 and Pipelines in Permafrost: November 19, 1985

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PROCEEDINGS OF THE WORKSHOPS ON SUBSEA PERMAFROST NOVEMBER 18, 1985 AND PIPELINES IN PERMAFROST NOVEMBER 19, 1985

PREPARED BY

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FOREWORD

This is a record of Workshops on (a) Subsea Permafrost and (b) Pipelines in Permafrost which were held on November 18 and 19, 1985, respectively, in Edmonton, Alberta. The Workshops were sponsored by the Permafrost Subcommittee, Associate Committee on Geotechnical Research, National Research Council of Canada.

A total of 103 participants attended the two Workshops, 81 from Canada, 21 from the United States and one from the United Kingdom. Each Workshop followed essentially the same format. Papers presented at sessions during the morning and early afternoon each day were followed by a closing session for open discussion and consideration of research needs. Complete texts or extended abstracts of most of the papers presented at the Workshops are included in this volume. Only minor editorial changes were made to the manuscripts submitted. The efforts of the authors in preparing and presenting their contributions is greatly appreciated.

The Permafrost Subcommittee wishes to express its sincere thanks to Mr. D.W. Hayley, EBA Engineering Consultants Ltd., Edmonton, Prof. D.C. Sego, Department of Civil Engineering, University of Alberta, Edmonton, Dr. W.A. Slusarchuk, Hardy Associates (1978) Ltd., Calgary, and Mr. G.N. Lewis, Esso Resources Canada Ltd., Calgary, all members of the Subcommittee, for developing and organizing the programs for the Workshops. Special thanks are also extended to the following individuals for their assistance during the Workshops: Mr. T.H.W. Baker, Institute for Research in Construction, National Research Council of Canada, Ottawa, Mr. P. Collins and Ms. G. Griffin, Department of Civil Engineering, University of Alberta, Edmonton and Mrs. C. Boss and Mrs. M. Cusack, EBA Engineering Consultants, Edmonton.

WORKSHOP ON SUBSEA PERMAFROST

IDENTIFICATION OF SUBSEA PERMAFROST BY GEOPHYSICAL METHODS

M.J. O'Connor*

(Presented at Workshop but manuscript not available for Proceedings)

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DISTRIBUTION OF PERMAFROST IN THE BEAUFORT SEA, CANADA

S.M. Blasco*

(Presented at Workshop but manuscript not available for Proceedings)

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PERMAFROST DISTRIBUTION AND THE QUATERNARY HISTORY OF THE MACKENZIE - BEAUFORT REGION: A GEOTHERMAL PERSPECTIVE

A.S. Judge*

Data Base

The base of permafrost for each of 161 exploratory wells was determined using conventional well-logs as outlined by Walker and Stuart (1976), Hnatiuk and Randall (1977), Hatlelid and Macdonald (1979), and D & S Petrophysics (1983). The values determined are strictly for the base of ice-bearing permafrost as revealed by changes in the physical properties, in particular the electrical and acoustic. In coarse-grained sediments the base may coincide closely with the 0°C isotherm, the difference being dependent on depth and pore water salinity alone. In fine-grained soils the freezing characteristics of the soil predominate and the base of ice-bearing permafrost may be up to 100 m above the 0°C isotherm with a transition layer below (Osterkamp & Payne, 1981; Taylor et. al., 1982). The bottom of the ice-bearing sediments is in fact determined by a very complex set of soil characteristics, both static and dynamic, the inter-relationships of which remain poorly understood.

Deep temperature observations were collected and collated from 172 exploratory wells in the Mackenzie Delta, the Arctic Coastal Plain and the Beaufort Shelf (Geotechnical Resources Ltd., 1983). Three primary sources were used: (1) bottom-hole temperature information from the headers of well-logs, together with relevant information on depth and time since the end of circulation in the well, and mud temperature; (2) drill-stem test determination of formation temperature with information on the depth of measurement and (3) industry-run downhole temperature surveys together with relevant information since the time of last circulation in the well, and the survey accuracy. Probable equilibrium temperatures were calculated for each type of data and plotted as a function of depth. Such information extends to depths of 4 km.

Over the past decade, through the cooperation of industry and government agencies, temperature measurements have been made in 45 wells drilled for hydrocarbon exploration (Taylor et. al., 1982). Precise temperatures have been measured at successive times since completion of the well to depths in excess of 600 m. The base of ice-bearing permafrost, the 0°C isotherm, and indications of zones of high ice content, as derived from this data set, were used to calibrate and interpret the geophysical well-logs. Mean surface temperatures, temperature gradients within and below the permafrost and other characteristics of the temperature curves have been used to determine the nature and extent of the permafrost distribution.

Drilling of about 100 shallow holes on and offshore has enabled an examination of the current near-surface permafrost characteristics

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and active processes through the precise monitoring of temperatures (MacAulay et. al., 1977; Taylor et. al., 1982). The results from offshore temperature measurements have been integrated with seismic reflection and refraction studies, as outlined by Hunter et al. (1976) and Neave et al. (1978).

Thickness of Permafrost

Figure 1 shows a contour of the base of the permafrost at 100 m intervals. The greatest depths, as much as 740 m, to the base of permafrost are found on land in the northern part of the Mackenzie Delta on northern Richards Island, but depths exceed 500 m in the northern half of the entire Tutoyaktuk Peninsula between the East channel of the Delta and the northeastern tip of the Peninsula. Depths to the permafrost base offshore are encountered to 700 m beneath the sea-floor. Almost the entire shelf between longitude 129°W and 136°W is characterized by occurrences of thick permafrost below the sea-bed. The seaward boundaries of permafrost determined from the well-log analysis coincide closely with the boundaries of the permafrost inferred from seismic refraction velocities on the upper permafrost table. Seismic interpretation suggests an additional portion of the northeastern shelf, extending southwesterly in a belt 15-20 km wide, to be without this frozen substrate. Lack of well information in the area prevents confirmation of the total absence of permafrost throughout the belt.

Thickness of permafrost decreases in the southern portion of the Mackenzie Delta, along the Eskimo Lakes to the east of the Delta, east of a N-S line along the west side of the Delta, and possibly on the Yukon Coastal Plain. West of 136°W permafrost is thin or absent offshore. A few limited onshore observations east of the study region again indicate thick permafrost: >300 m at 68 51°N, 126°47'W and 450 m at 67°44'N, 126°50'W (Taylor et al., 1982). Similar increases in permafrost thickness might be expected west of the Mackenzie Delta with a boundary either in or to the south of the Yukon coastal areas. However, well data is not available to confirm this. West of the Mackenzie Trough, offshore permafrost may again thicken, although the available well data is not sufficient to confirm this assumption. Sediment characteristics are akin to those encountered in the Alaskan Beaufort where thick permafrost is encountered offshore (Osterkamp and Payne, 1981).

Deep Temperature Observations

The deep temperatures calculated from industry data were plotted and contoured isotherm maps were generated for depths of 1, 2, 3 and 4 km utilising a 10°C contour interval. As shown in Figure 2 data is often sparse and so the individual contours are speculative. The maximum and minimum temperatures recorded for the given depth zones are 44/14°C at 1 km, 66/33°C at 2 km, 97/46°C at 3 km and 128/83°C at 4 km. Differential temperatures between the maximum and minimum values range from 30° at 1 km to 50° at 4 km, consistent with regions of varying geothermal gradient. The heat sources and sinks are not well-defined because of lack of sufficient data. However, the higher temperature zones consistently occur around the Eskimo Lakes region east of

Tuktoyaktuk, along the west coast of Mackenzie Bay and offshore between latitudes 69°40'N and 70°N around longitude 136°W. A low temperature zone follows a general northeast-southwest trend west of Tuktoyaktuk, centered over northern Richards Island and the western Tuktoyaktuk Peninsula.

Deep geothermal gradients are quite uniform compared with other geological regions and indicate an area of normal heat flow, i.e., gradients are generally in the range of 20 to 30°C/km. A region of gradient 40°C/km, occurs to the northwest of the area around latitude 70°N and longitude 136°W. Gradients appear to decrease towards the eastern edge of the area.

Precise Well Temperature Data

Deep temperature measurements are available at 45 sites in the region, primarily onshore. The coverage is not uniform in distribution; 30 sites are in the old delta where permafrost thickness, as revealed by the 0°C isotherm, ranges from 272 to 670 m with a median value of 370 m. This is four to five times thicker than the permafrost at 9 sites in the modern delta (65 to 175 m; median 85 m). Ground temperatures, extrapolated to the surface from measurements in the upper 100 m at these sites, lie in the range -4.4 to -9.5°C (old delta) and -1.4 to -4.9°C (modern delta). A similar range was found by Mackay (1972) from measurements in seismic shot-holes. Little difference was observed in air temperatures recorded at three stations in the region (Environment Canada, 1975). In most instances the temperatures increase reasonably linearly with depth below 100 m, indicating a quasi-equilibrium state, although above 100 m indications do exist of surface warming in the old delta wells and cooling at the new delta sites. An exception to this distribution of temperature with depth was very pronounced in 5 wells to the west and south of Big Lake in the central delta. Although Mackay (1963) placed the region in the modern delta, thick permafrost in excess of 500 m places it in the old delta while the essentially isothermal nature of the temperatures through the permafrost are most akin to the offshore thermal characteristics.

Shallow Thermal and Seismic Studies in the Offshore

As described by Hunter et al. (1976) and Neave et al. (1978), high seismic velocities encountered on the eastern half of the Beaufort Shelf are interpreted as relic ice-bonded permafrost. The velocity data east of 135°W is divisible into an upper velocity group with a top 60-100 m below sea level and a lower group 130-200 m below sea level. West of 135°W and to the edge of the Mackenzie Trough a more complex structure is present, probably representative of partially ice-bonded or ice-bearing sediments. Shallow drilling to depths of 60 m below the sea floor has revealed the profound nature of the edge of the Mackenzie Trough, north and west of Garry Island, and a very complex shallow thermal regime to the east (MacAuley et al., 1977, 1978). These observations are indicative of relic conditions at depth, possible seasonal aggradation of frost in the upper 3 to 4 m of sediments and non-conductive processes of heat transfer above the top of the main permafrost body.

Permafrost and Quaternary History

The zones of thick and thin permafrost appear to show a strong relationship with the limits of Wisconsin glaciation shown in Figure 1. The relatively thin permafrost of the modern delta and the position of the late Wisconsin glacial limit (Mackay et al., 1972) are in close agreement. The thicker permafrost appears as a wedge encompassing Richards Island, the northwest Tuktoyaktuk Peninsula, and Parga sections of the adjacent shelf again correlates well with the proposed limit of the Wisconsin maxima shown in Figure 1. Following Rampton's position of the early Wisconsin glacial limit, the thickest permafrost might be predicted to be to the northeast of Tuktoyaktuk, north of Rampton's northern limit of glaciation. In fact, values average several hundred metres less than on Richards Islands. In a general sense the existence of ice tongues or ice sheets in the region appears to have insulated the soil from sub-zero air temperatures which characterized the climate during the period of Wisconsin glaciation, thus inhibiting the growth of deep permafrost. In the northern and central part of the Mackenzie Delta, along the Beaufort Crustal zone of the Tuktoyaktuk Peninsula, and on the shelf adjacent to both areas, the presence of deep permafrost suggests an absence of glacial ice or sea-water cover, and a direct exposure of the land to the colder air temperatures occurring in an ice age. The distribution of permafrost may in fact provide a useful additional source of information in determining the position of glacial limits.

Permafrost thickness in the modern delta is relatively shallow although the thickness has been further limited by proximity to seasonal flooding and to major river channels that have shifted over the past few thousand years. Intercept temperatures generally suggest cooling of the land surface over the period, which is consistent with aggradation of permafrost in an aggrading modern river delta.

Observations of permafrost distribution are accompanied by high quality temperature measurements at 10 sites in the Parson's Lake area. In the morainic hills adjacent to the lake, permafrost thickness ranges from 294 to 386 m. Mackay et al. (1972) position the late Wisconsin limit directly through the area; a site 10 km west of the lake has very deep permafrost (550 m), suggesting it may lie outside of the maximum Wisconsin advance. Alternatively, since a recent geophysical survey has shown Parson's Lake is not underlain by permafrost (Geo-Physi-Con, 1983), the lake may be an old feature and the glacial limit may lie further east. Unfortunately, relatively warm sub-glacial conditions could create a similar situation.

The deep permafrost and temperature results add to the definition of the western boundary between the modern and old delta. Mackay (1963) places it at the base of the Caribou Hills and the exposed Pleistocene area of Richards Island further north, towards the sea. This definition results in a loop extending east to include the Taglu gas field in the modern delta. However, the recently obtained deep temperature observations and the well-log interpretations have shown permafrost thicknesses to be in excess of 500 m, values that are similar to permafrost thicknesses in the old delta. Furthermore, permafrost depths in the Taglu field suggest that the area was not glaciated during the late Wisconsin, adding evidence to the ice limit proposed by Mackay et al. (1972). The present data suggest that the boundary between the old and modern deltas follows a linear extension northwest from the Caribou Hills. Several logged wells are located very close to this lineament; the deep permafrost (502 m) at Sun Garry P-04 suggests that the boundary passes to the west of that site and Garry Island further north, as shown in Figure 1.

Contiguity of the permafrost distribution and history onshore and offshore is well illustrated by logs from the Taglu gas field where thick warm onshore permafrost, similar to that found offshore, is found in a region newly emergent in the past 1000 years or less. Consequently the permafrost is degrading at the base and aggrading in the upper 120 m.

Unfortunately, although shallow temperature observations to 60 m below the sea-bottom confirm the degradational character of offshore permafrost and the well-log interpretations reveal the wide regional variations in permafrost thickness, the lack of precise temperature observations taken over a period of time, preclude detailed thermal modelling of the shelf history at the present time.

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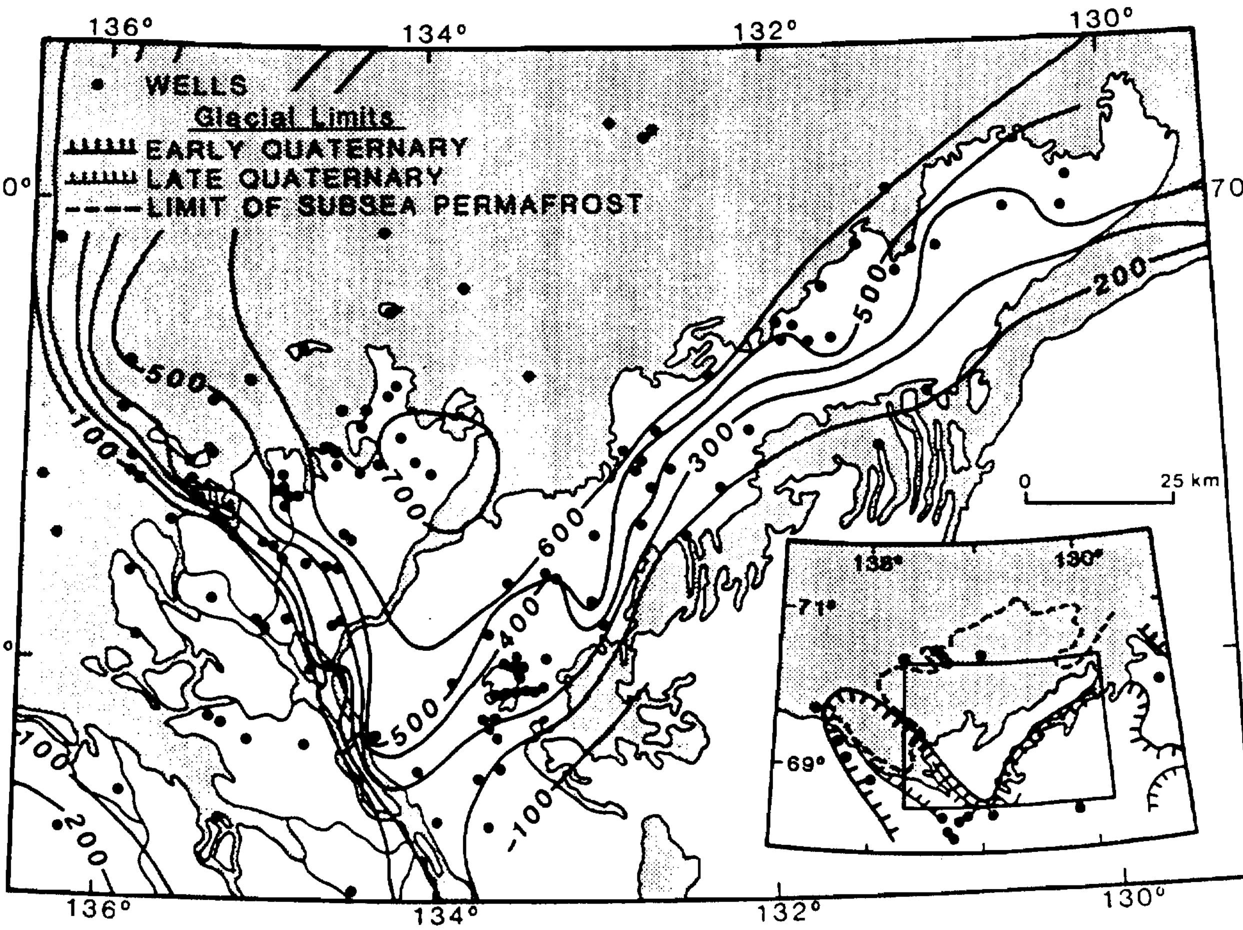

Figure 1. Contour of the Base of Permafrost at 100 m Intervals

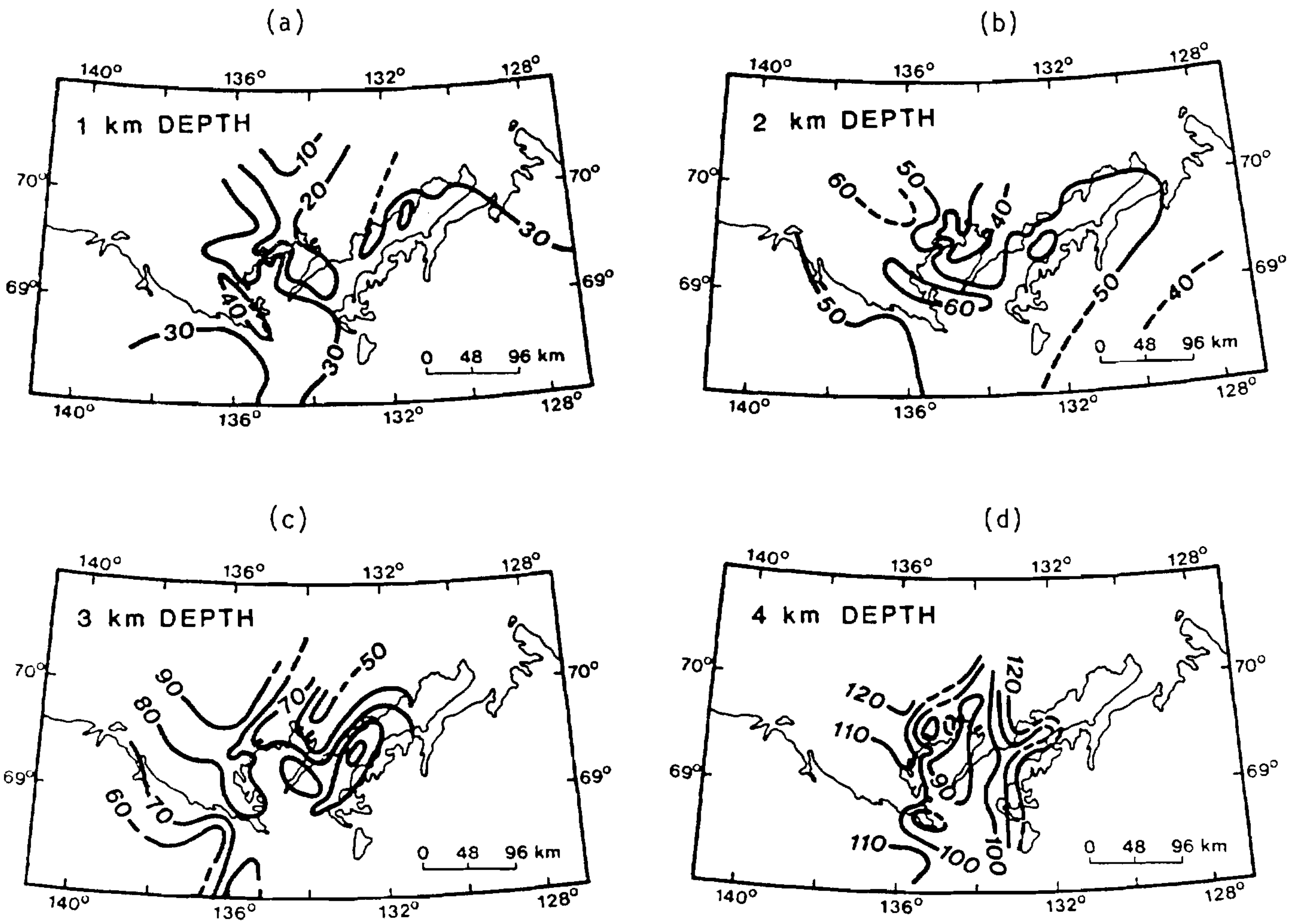

Figure 2(a-d). Contoured Isotherm Maps for Depths of 1, 2, 3 and 4 km respectively

DRILLING AND SAMPLING OFFSHORE PERMAFROST

J.P. Ruffell*

This presentation summarises the collection of diverse data pertinent to the engineering analysis of shallow marine permafrost. It should be noted that, given the large number of boreholes drilled in the Canadian Beaufort Sea to date, very few samples have been taken for this purpose: the primary purpose of most boreholes is the collection of data in the "weaker" unfrozen soils.

Permafrost Distribution in the Beaufort Sea

To briefly reiterate several other papers in this session, permafrost is widespread, thick, and largely undefined across the majority of both the Canadian and Alaskan Beaufort shelves. Significant permafrost bodies are present in all of the areas of interest for hydrocarbon exploration. Exploration of these permafrost zones has largely been confined to their lateral extent: very little investigation of the nature of the body in its vertical dimension has been attempted.

Ice-bonded sediments are widespread across the shelf to depths of between 200 and 600 metres below the seabed. At the Tarsiut N-44 location, the base of permafrost defined by electrical down-hole geophysical logs was close to 400 metres below seabed: this investigation was the only one completed to date, specifically designed to explore the permafrost section.

Permafrost can be encountered within a few metres of the seabed. Locations on the Yukon shelf and in Harrison Bay display partially-bonded soils at or within a few metres of the seabed. The surface relief of the permafrost body is both irregular and complex.

Features of Offshore Permafrost

Some of the more unusual features of offshore permafrost, compared with its terrestrial counterpart, are described below.

No clear variation in the type or quality of ground ice occurs, other than that the variation appears, on the whole, to be lithologically controlled: in warmer zones, for instance, the sands tend to be well-bonded, while the silts and clays are poorly-bonded or unfrozen.

Several occurrences of hyper-saline pore water below bonded intervals have been documented, possibly due to disassociation of salt during slow freezing. Instances of freshwater and brackish water segregated, ice, excess and massive ice are also documented, some being found at considerable depth.

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* See List of Registrants (Appendix A) for affiliation of authors.

Areas of Concern of Exploration Drilling

Exploration drilling for hydrocarbons will be examined to establish the need for adequate sampling of permafrost. Note that hydrocarbon production and related construction activities also present similar problems.

- the permafrost is warm in comparison with terrestrial permafrost, often no colder than -2.5°C for hundreds of metres. Also, because the pore water is saline, the ground is in a delicate state of balance between frozen and nonfrozen, producing a wide variation in the degree of ice-bonding.

- the lithology varies widely, even in small vertical sections. The geologic history of the Canadian Beaufort Sea is one of minor and major delta aggradation and degradation, providing a complex soil profile.

- the consistency of the bonded and unbonded zones varies widely in response to the variable degree of ice-bonding and mixed lithology described above.

All of the above lead to what may conservatively be regarded as uncertain conditions when drilling large-diameter exploration wells.

Problems that may occur are numerous, but more common manifestations of inadequate planning for permafrost conditions are:

- excess erosion and cavity formation when drilling ahead.

- sloughing, caving and collapse of the borehole around the drill bit.

- problems when setting and cementing casing.

- casing settlement, buckling and failure.

These problems are often reported as mechanical abandonments when, in all probability, permafrost is the true root of the problem.

There is a need, therefore, to extend our knowledge of permafrost in its vertical dimension. This may most easily be undertaken by site-specific deep sampling projects, particularly in the case when an oilfield development scenario is being considered.

Current Investigation Techniques

Investigation techniques in the Arctic Ocean are primarily controlled by the season of operation and the water depth at the intended site.

In summer months, conventional geotechnical exploration vessels have been employed. Self-anchoring ships have been, in general, used in water depths greater than 15 metres, while tug-supported barges have been used in shallower waters. Both types of vessel are well-equipped to complete shallow sampling of permafrost, but are susceptible to weather interruptions of drilling, which inevitably leads to loss of the borehole: reentry techniques common in larger offshore programs have not been adopted to date by the geotechnical community, as the major thrust of investigations for exploration scenarios is within 50 metres of the seabed and thus reentry capabilities are of little concern.

Winter off-ice programs are also commonplace, using either terrestrial drilling logistics or helicopters. Again, however, they are susceptible to interruption by movement of the ice platform and consequent borehole loss.

Although these modes of investigation are unsuitable for deep sampling of permafrost, they have provided the proving-ground for techniques that will be employed in such an investigation.

The following sampling methodology has been used on a regular basis over the last five years:

- In poorly-bonded and mixed frozen/unfrozen zones, conventional wireline-deployed, hydraulically-pushed, thin-wall sampling tubes are used.

- In more competent soils, sampling tubes may be driven by a down-hole hammer.

- In well-bonded and competent permafrost, conventional triple-tube core barrels are used.

It is coring that holds the greatest probability of success for deep permafrost sampling. This technique demands great skill on the part of the driller to control down-hole pressure, drilling fluid flow and temperature, and rate of rotation of the bit. Also, the setting of the sampling barrel within the assembly and the selection of the correct bit for the soil/ice combinations encountered is critical. All of these factors must be optimised for each strata if good quality core is to be recovered.

Of all the factors noted above, drilling fluid (mud) temperature control is vital. This aspect of the drilling system has not been seriously developed for permafrost sampling projects. Although chilled mud units have been developed for the exploration drilling rigs, the temperature control is designed solely to keep mud temperatures lower than in situ. For our purposes, we require that a high flow of mud for coring be available to within $\pm 0.1^{\circ}C$ tolerance. Heat exchangers presently used by industry are unlikely to be able to supply these adequately-conditioned volumes of mud, and recoveries will suffer, particularly in zones where the permafrost is susceptible to thaw:

the major zones of interest for engineering purposes.

Also, our ability to provide a chilled mud with good cuttings-lifting and lubrication qualities is in doubt. We have found that our tried and tested mud formulations do not mix well at colder temperatures, and tend to disassociate on chilling. We therefore have had to pump poor-quality muds to the bit and deal with the resulting down-hole problems such as cuttings build-up by breaking the sampling cycle while reconditioning the hole. Consequently, several metres of borehole are lost as the frost-front at the end of the borehole retreats into the formation over the reconditioning period.

Another problem with chilled muds occurs in the extreme case when muds colder than in situ freeze the core barrel into the hole during breaks in rotation. This is a particular problem at depth, where cuttings removal is less efficient.

There is a trade-off to be achieved, therefore, between the cuttings lifting capacity of the mud and our ability to control the temperature at the bit.

Another important aspect of the techniques of collecting permafrost core has also been overlooked. Core that has been diligently collected is often condemned to inadequate storage containers such as insulated core boxes which only delay its inevitable thermal degradation. In better quality investigations, specially-modified chest freezers are provided, but these are often set to an "average" in situ temperature which, in time, changes all core to this temperature.

Individual and highly sensitive temperature of small groups of cores must be implemented immediately the core barrel is retrieved at the surface, if the effort in providing accurate temperature down-hole is to be useful.

An approach to the problems of permafrost temperature degradation in past projects has been to fully define the soil state in situ and to return the soils to this state prior to laboratory testing. This requires an accurate measurement of the soil's in situ temperature, unfrozen moisture content, mineralogy, phase composition and pore water salinity. All of these parameters can be measured accurately, but comparative studies with well-sampled core have yet to be carried out.

Conclusions

Current investigation philosophy

The primary interest of operators in the Arctic is the collection of engineering data in the upper, unfrozen sediment. Precedence is given to the weak engineering soils and the soil in any ice/soil mix. This reflects their concern with the exploration scenario.

Items required for design

The following data is required by a production development scenario:

- continuous sampling of the entire permafrost zone to allow definition of soil conditions.
- in situ temperature information accurate to within $\pm 0.01°C$.
- stress-strain properties of the soils encountered.

Items requiring further development

To aid the would-be operator in this endeavour, we require that the following be developed:

- sample handling procedures from the arrival of the sample at the surface to the time that it is safely jacketed in the triaxial cell, that will prevent significant thermal degradation of the sample.
- a mud supply system that will provide a fluid that has similar temperature and density attributes to that of the soil at the base of the hole, without foregoing the muds ability to ensure stability of the hole and removal of cuttings.
- better relationships between engineering soils properties and the data routinely collected in hydrocarbon exploration, that is down-hole geophysics and measurements made while drilling.

In conclusion, technology is available now which can be easily and efficiently used for the collection of undisturbed subsea permafrost soils in the majority of soil/ice conditions envisioned. The development of the very important topics noted above could best be undertaken during a deep permafrost sampling project.

A REVIEW OF SUBSEA PERMAFROST CONDITIONS ALONG ALASKA'S COASTS

T.E. Osterkamp*

The purpose of this Workshop is to develop a statement of research needs and priorities for research on subsea permafrost. This extended abstract focuses on subsea permafrost along Alaska's coasts and will be divided into three parts: general comments on a research plan, selected comments on the origin and evolution of subsea permafrost, and a short review of subsea permafrost conditions along Alaska's coasts.

A research plan for developing an understanding of subsea permafrost and its associated engineering and environmental problems should have at least three components:

1. A research strategy consisting of a set of objectives and a plan for attaining them,

2. A data base consisting of information on the occurrence, distribution and characteristics of subsea permafrost, and

3. A means for data synthesis and extrapolation of results to other areas (i.e., models).

A research strategy is necessary to provide a philosophy and guidance for the research. Various types of data bases exist. An example of a data base pertinent to the understanding of heat and mass transport processes is shown in Table 1. This data base was generated by considering the heat and mass transport model shown schematically in Figure 1. A more complete data base for subsea permafrost requires consideration of other models (e.g. mechanical, geological).

Subsea permafrost is a product of changing sea levels and past cold climates. It grows, by freezing from the ground surface downwards, in the continental shelves during periods of emergence and thaws from both the base and table during periods of submergence. Sea levels are currently high and fairly stable implying that most subsea permafrost is relic, although some areas may exist where it is aggrading under shallow waters. Figure 2 is one of a number of sea level curves available today. Sea level curves are important since they are helpful in deciding on the initial conditions of temperatures, pore fluid concentrations, moisture content, initial time, and for determining the times of emergence and submergence which control the amount of permafrost that can be formed by freezing and destroyed by thawing. Unfortunately, there is considerable divergence in the sea level curves in the literature. A more reliable curve is badly needed.

Past climate is another important factor for which little information exists. It is important since it defines the thermal surface boundary condition during periods of emergence. By combining sea level

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* See List of Registrants (Appendix A) for affiliation of authors.

curves with past climate curves and information on current seabed temperatures, crude representations of the surface temperature conditions over geological time can be constructed.

An understanding of the response of permafrost to emergence or submergence and its subsequent evolution in time requires an understanding of the physical processes that control its freezing and thawing. The freezing case, in the first approximation, is usually treated as a Stefan problem. It is made complex by the need to account for geothermal heat flow, the presence of saline pore fluids and potential layering produced by rapid changes in sea level. After submergence, the relatively warm and salty seabed boundary condition leads to warming of the permafrost at all depths and thawing from above and below. When heat transfer during warming involves conduction with only ice in the pore spaces, simple models show that isothermal conditions, for the permafrost slab, can be reached in one to two thousand years for permafrost along the Chukchi and Beaufort Sea coasts. When the sediment pores contain brines and ice, then a drastic increase in the time to reach isothermal conditions can be expected. Physically, this is a result of the necessity for thawing ice, with its large latent heat, to raise the permafrost temperature. In this case, relatively cold permafrost can be found at great distances (or times) offshore.

Thawing from above starts immediately after submergence but thawing from below, which is driven by geothermal heat, cannot begin until the temperature profile at the base of the ice-bearing permafrost, changes.

Thawing at the seabed proceeds in the presence of negative mean seabed temperatures. This thawing is a result of the presence and transport of salts from the seabed to the ice-bearing permafrost table. The presence of salts in the permafrost, initially, may also be a contributing factor. However, thawing requires both heat and salts to proceed. Salts are required to depress the freezing point of the pore fluids below the mean seabed temperature and heat is required to melt the ice. Since both heat and salt are needed, then the thawing rate must be controlled by the slower of the processes governing heat and salt transport. This is illustrated in a profile of ice-bearing subsea permafrost near Prudhoe Bay (Figure 3). In the coarse sediments between the coast and Reindeer Island, the salt transport is thought to be convective and the heat transport diffusive. Therefore, thawing at the ice-bearing permafrost table is controlled by the slower heat transport which gives

$$X \approx \sqrt{t}, \qquad (1)$$

where X is the depth of thaw in metres and t is the time offshore in years. Offshore from Reindeer Island, where the sediments are over-consolidated clays, both the salt and heat transport are diffusive. However, the diffusivity for salt is much smaller than for heat. Therefore, thawing at the ice-bearing permafrost table is controlled by the slower salt transport which gives

$$X \approx 0.1 \sqrt{t}. \qquad (2)$$

Thawing from above and below continues for as long as the permafrost is submerged until all the pore ice is melted or until new equilibrium conditions are reached. It should be noted that the above simplified model of subsea permafrost can be changed substantially by variable seabed temperatures and pore fluid concentrations, by changes in the sediments and by other factors.

Subsea permafrost conditions along Alaska's coasts are highly variable, nevertheless, a few generalizations can be made. There is a low potential for encountering ice-bonded subsea permafrost in Norton Sound and in the northern Bering Sea except in areas of recent coastal retreat. In the Chukchi Sea, there is a potential for ice-bonded subsea permafrost in nearshore areas. However, the available data from the Bering and Chukchi Seas is so sparse, that it is difficult to reach any firm conclusions.

From Barrow to Cape Halkett, the sediments are fine-grained and potentially ice-rich. Ice-bonded subsea permafrost has been found near the seabed, twenty kilometres offshore. This area has a very high potential for creating potentially severe engineering and environmental problems. Similar conditions are found in Harrison Bay.

In the area from Oliktok Point to the Sagavanirktok River, relatively coarse-grained sediments are found near the seabed. Deep thawing at the seabed occurs in these sediments except where they are covered by thicker layers of fine and/or overconsolidated sediments. There is a low potential for encountering ice-bonded subsea permafrost near the seabed except nearshore and where the fine and/or over-consolidated sediments exist. Conditions are similar in the area from the Sagavanirktok River to the Canning River except that the occurrence of coarse-grained sediments appears to decrease with distance offshore and to the east. Massive ground ice has been found near the seabed in this area.

There is little data in the area from the Canning River to the U.S.-Canadian border that no assessment of subsea permafrost in this area can be made.

A synopsis of the research needs for subsea permafrost along the coasts of Alaska is given below:

1. Improved past climate and sea level curves.

2. Improved understanding of salt transport processes especially in fine-grained sediments and rocks.

3. Improved physical and numerical models of heat and mass transport processes in subsea permafrost.

4. Selected regional drilling and physical data.

5. A few deep drill holes that penetrate the subsea permafrost.

6. All types of data in the region from the Canning River to the U.S.-Canadian border.

Table 1: A data base pertinent to the understanding of heat and mass transport processes in subsea permafrost.

Type of Data	Specific Needs
Physical	Profiles of temperature, pore water concentration, moisture content and ice content.
	Permafrost thickness and depth to phase boundaries
	Heat flow
	Physical and chemical parameters
Geophysical	Surface and borehole data obtained from exploration geophysical methods and geophysical well logs.
Geological	Sediment types and sedimentation rates.
	Shoreline erosion rates
	Tectonic setting
Oceanological	Sea level history, seabed temperatures and salinities
	Sea ice conditions
	Bathymetry, currents.
Hydrological	Presence of rivers, seasonal flow, freshwater plumes.
	Presence, distribution and characteristics of lakes.
Meteorological	Past and present climate especially mean annual ground temperatures.

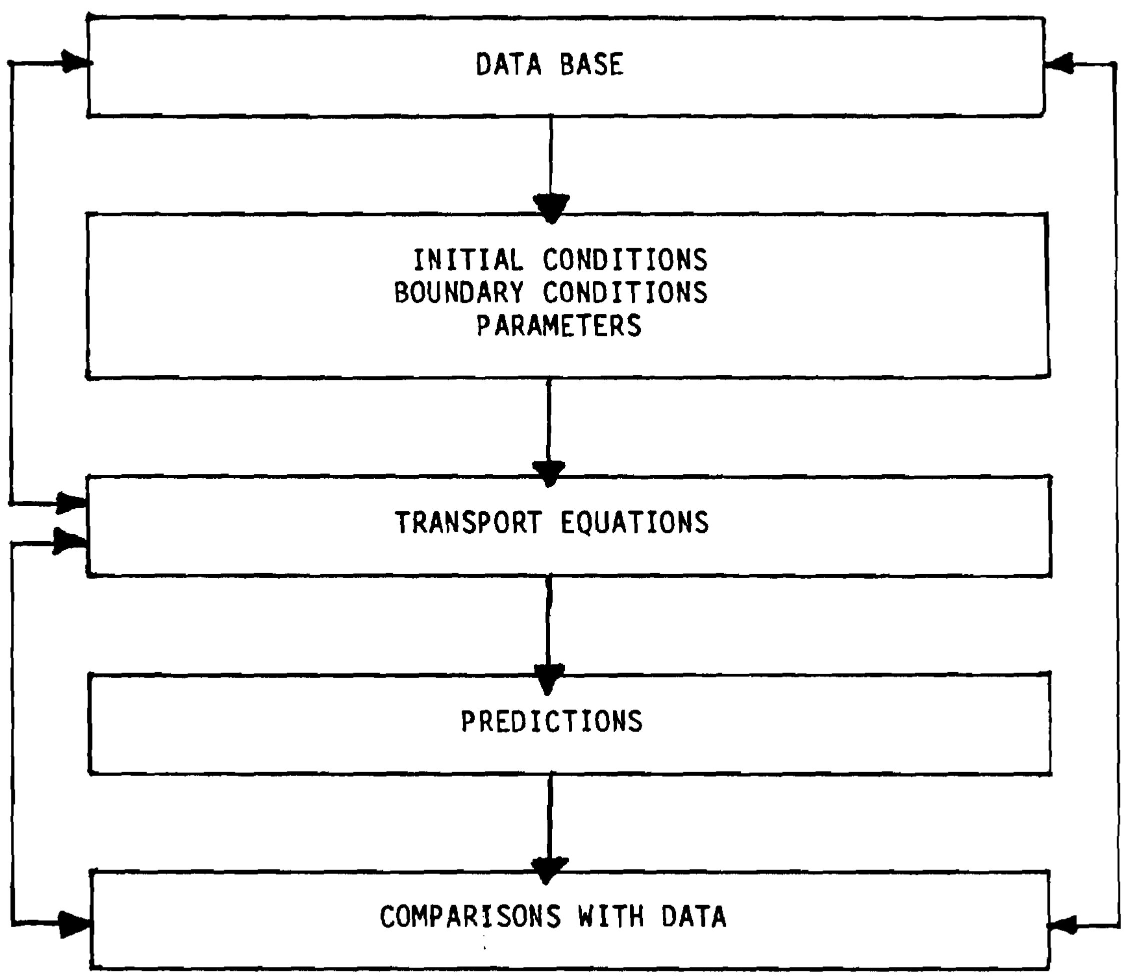

Figure 1. A schematic heat and mass transport model for subsea permafrost

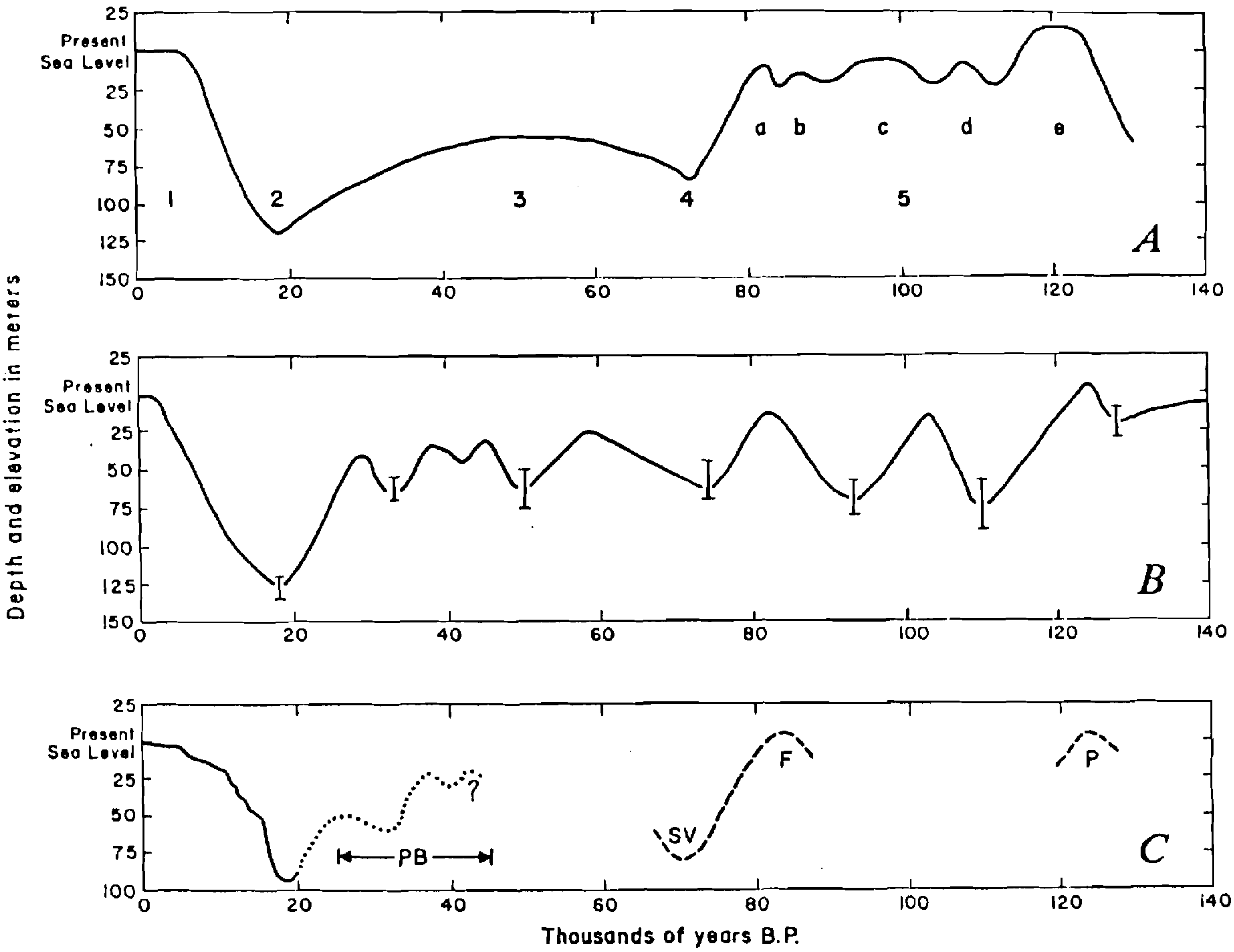

Figure 2. Worldwide sea-level histories postulated on the basis of (a) oxygen-isotope data from benthic foraminifers in a Caribbean deep-sea core (numbered and lettered subdivisions of curve are oxygen-isotope stages (Shackleton and Opdyke, 1973) and (b) raised coral reefs in New Guinea (Bloom et al., 1974) with (c) sea-level history interpretations for Beringia. Portion of Beringian curve younger than 20,000 years after Hopkins (1973) and McManus et al. (in press); a sea-level curve applicable to the Mackenzie Bight area of eastern Beaufort Sea (Forbes, 1980) is not presented here. PB portion is speculative and based on interpretation of radiocarbon-dated stratigraphy of offshore boreholes and onshore gravel pits at Prudhoe Bay (Hartz et al., 1979; Hopkins, Robinson, and Buckley, 1981). Older part of Beringian curve is not yet dated by local evidence. SV represents regression recorded by valleys carved across Beaufort Sea shelf more than 32,000 years ago (Hopkins, et al., 1979). F represents Flaxman transgression, recorded by glaciomarine deposits present at altitudes below 5 m on the coast of Beaufort Sea (Hopkins, 1979b) but absent from coasts of Chukchi and Bering Seas. P represents Pelukian transgression, recorded by shoreline features at an altitude of about 7 m along much of coasts of Bering, Chukchi, and Beaufort Seas (Hopkins, 1967c). Pelukian transgression dates from the last interglacial and no doubt corresponds to isotope stage 5c. (This figure is reproduced from: Hopkins, D.M. 1982 "Aspects of the paleogeography of Beringia during the Late Pleistocene" in PALEOECOLOGY OF BERINGIA, Ed. by Hopkins, D.M., et al., Academic Press.)

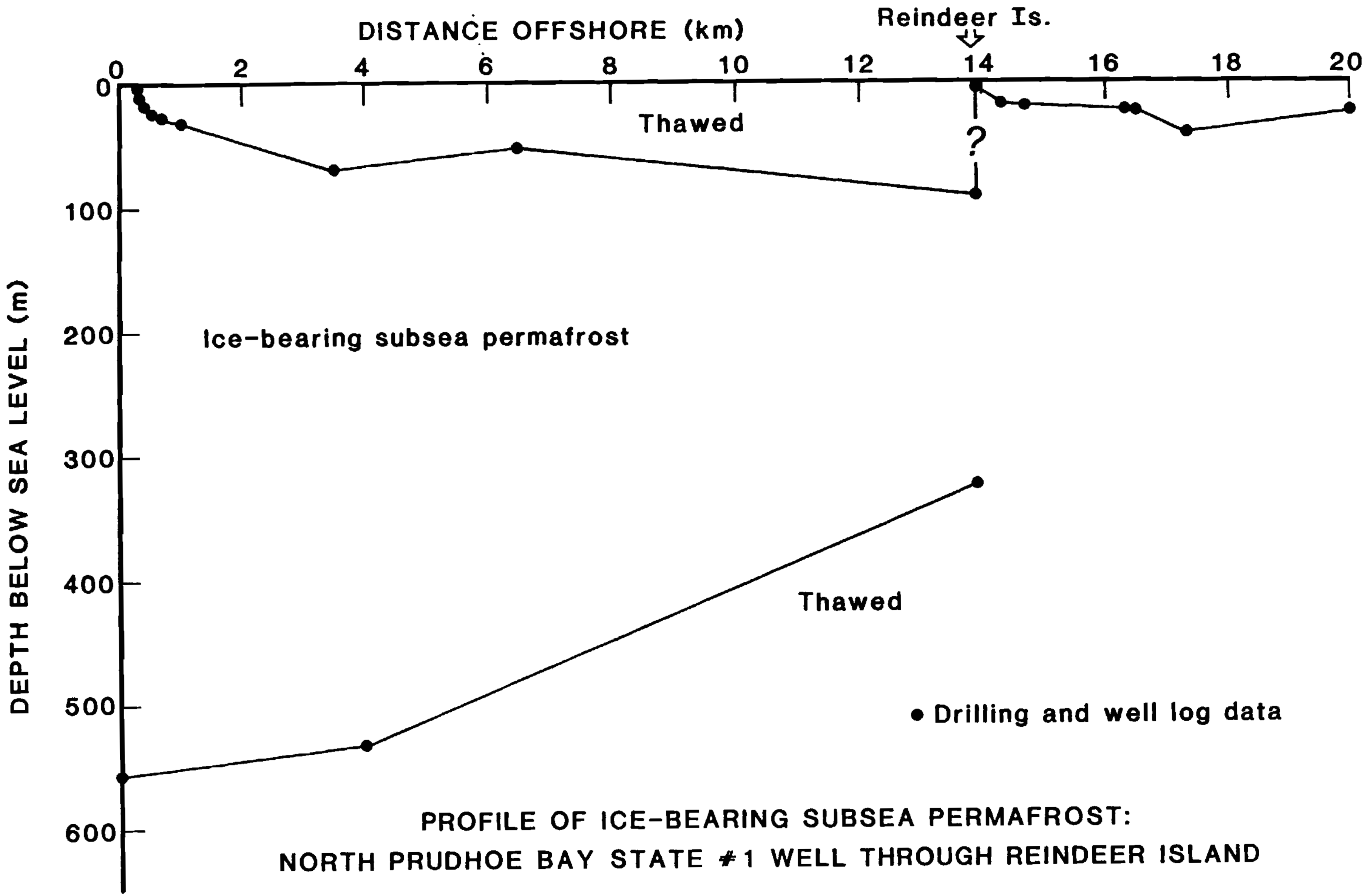
DISTANCE OFFSHORE (km)
Reindeer Is.
0
2
4
6
8
10
12
14
16
18
20
DEPTH BELOW SEA LEVEL (m)
0
100
200
300
400
500
600
Thawed
?
Ice-bearing subsea permafrost
Thawed
● Drilling and well log data
PROFILE OF ICE-BEARING SUBSEA PERMAFROST:
NORTH PRUDHOE BAY STATE #1 WELL THROUGH REINDEER ISLAND

FIGURE 3

DERIVATION OF ENGINEERING PROPERTIES OF PERMAFROST FROM THE CONE PENETRATION TEST

B. Ladanyi*

Introduction

In unfrozen soils the static cone penetration test (CPT) has been used as a logging tool for nearly fifty years, giving valuable information for the design of foundations. Although developed initially mainly for the purpose of pile design, it has through the years been used also for estimating certain short-term parameters of soils, in particular the angle of friction of sands, the undrained shear strength of clays and silts, and, more recently, the pore pressure response of soils during and after cone penetration.

Due to the physical similarity with driven piles, the CPT results can usually be applied directly to the design of such piles, provided proper care is taken of the effects of scale, strain rate, and the proximity of the free surface. On the other hand, if certain soil parameters have to be estimated from CPT data, this is possible only on the basis of a valid deep penetration theory or a well documented correlation with directly measured parameters.

In frozen soils, the CPT has a promising potential application in both of these fields, i.e., for a direct design of driven piles and for the determination of certain, more general frozen soil parameters that can be used for solving various types of geotechnical problems in permafrost.

For offshore use in the Arctic, the CPT has an additional advantage of being able to determine whether or not a soil is ice-bonded, regardless of its temperature and the salinity of its pore water. While in the last few years the test has been used frequently in the Arctic in connection with the design of artificial islands and drilling platforms, its potential in connection with pile design in offshore permafrost has not yet been fully investigated. It is the purpose of this paper to show how the cone penetration test can be used in connection with the design of driven piles in permafrost, and for estimating certain frozen soil parameters needed in permafrost engineering practice.

Behavior of Frozen Soils in Cone Penetration Tests

In unfrozen soils, which show relatively small rate sensitivity of strength, the cone penetration test is usually performed at a standard penetration rate of 2 cm per second. This standard testing procedure cannot be used in frozen soils which are much more rate sensitive, and where one is interested not only in the short-term strength but also in the whole dependence of strength on the rate of strain.

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* See List of Registrants (Appendix A) for affiliation of authors.

In principle, the rate dependence of strength of a given permafrost soil can be evaluated by performing a series of several closely-spaced quasi-static penetration tests, each at a different rate. In practice, however, such a procedure is difficult to realize, because, at very low rates of penetration, it would take a very long time to penetrate the required depth interval. Alternatively, for obtaining the creep information at very low rates of strain, one can perform static, or incremental-loading, cone penetration tests, in which the load is increased in steps and kept constant at each step for a desired period of time.

The static penetration test is similar to an ordinary deep plate loading test carried out at the bottom of a borehole. However, while in an unfrozen soil one is mainly interested in determining from such a test the complete load-settlement curve, in a frozen soil the relationship between the load and the rate of penetration will be of more interest.

The results of field penetration tests in a frozen varved silt (Ladanyi, 1976) and of laboratory penetration tests in a frozen sand (Ladanyi and Paquin, 1978), both performed with an electric Fugro penetrometer, have shown that the dependence of the penetration resistance q_c on the penetration rate $\dot{s}$ can be expressed most conveniently by a power law relationship (Fig. 1)

$$q_c = q_o(\dot{s}/\dot{s}_o)^{1/n} \tag{1}$$

where n is the creep exponent, obtained from the slope of the straight line of q_c vs $\dot{s}$ in a log-log plot, and q_o and $\dot{s}_o$ are co-ordinates of any selected reference point on that line.

Use of CPT in Pile Design

When comparing the point resistance q_p of a pile, penetrating deep into a frozen soil at a rate $\dot{s}_p$, with the point resistance q_c of a penetrometer pushed into the same soil at a rate $\dot{s}_c$, it is found (Ladanyi, 1976, 1982) that, in order to respect the equivalence of resulting strain rates, the following scale effect relationship should be used:

$$q_p = q_c[(\dot{s}_p/\dot{s}_c)/(B_p/B_c)]^{1/n} \tag{2}$$

where B_p and B_c denote the diameters of the pile and the penetrometer respectively.

In principle, Eq. (2) can be used for finding an equivalent point resistance for a pile for which the allowable penetration rate is $\dot{s}_p$.

This is, however, only valid in the interval of rates where the exponent n remains constant. Unfortunately, there is an ample evidence in frozen soil creep literature that the value of n, which is of the order of $n = 10$ to 20 at high rates of strain, shows a sharp decrease at rates rates lower than about $\dot{\varepsilon}_1 = 10^{-5}\ s^{-1}$. Below that rate its value falls to $n = 3$ to 6, with $n = 3$ valid for ice-rich soils and ice of far warmer temperatures, and $n = 5$ to 6 corresponding to lower temperatures, (e.g., Perkins and Ruedrich, 1973; McRoberts, 1975; Ladanyi and Paquin, 1978; Parameswaran, 1980; Bragg and Andersland, 1980). There is some evidence, however, that for fine-grained soils with high unfrozen water content, the opposite may be true, i.e. in the domain of very low strain rates, n tends to increase forwards the value found for unfrozen soils, which is of the order of 25 (Bourbonnais, 1984).

For a standard cone penetrometer, the penetration rate at which a change in n can be expected to occur in ice-rich soils was found to be about 4×10^{-4} cm/s (Fig. 2). In order to find the penetration resistance for a pile, it is recommended therefore to calculate first from the measured q_c at $\dot{s}_c$ the value of $\bar{q}_c$ at, say, $\dot{\bar{s}}_c = 4 \times 10^{4}$ cm/s,

$$q_c = q_c(\dot{\bar{s}}_c/\dot{s}_c)^{1/n_2} \tag{3}$$

where $n_2 = 10$ to 20, and then to determine the pile point resistance from Eq. (2)

$$q_p = \bar{q}_c[(\dot{s}_p/\dot{\bar{s}}_c)/(B_p/B_c)]^{1/n_1} \tag{4}$$

where $n_1 = 3$ to 6.

The temperature correction can be made as follows:

$$q_{p\theta_1} = q_{p\theta_2}[(\theta_c+\theta_1)/(\theta_c+\theta_2)]^k \tag{5}$$

where $0.4 < k < 1.0$, and $\theta_c = 1°C$,

A complete method for CPT-based pile design in permafrost, including the evaluation of adfreeze bond, can be found in Ladanyi, 1982 and 1985.

Determination of Creep Parameters from CPT Data

It is normally assumed in frozen soil mechanics that total strain resulting from a deviatoric stress increment is composed of an instantaneous strain ε_{inst} and a delayed strain ε_{creep},

$$\varepsilon = \varepsilon_{inst} + \varepsilon_{creep}$$

It has been found (Ladanyi and Paquin, 1978) that in cone penetration tests in a saturated frozen soil, the instantaneous strain plays only a minor role compared with the creep strain, so only the latter is usually considered.

In general the creep strain at a constant stress depends on stress σ, time t, and temperature T, and so does also the corresponding creep strain rate, $\dot{\varepsilon} = d\varepsilon/dt$.

A convenient form of a creep law, proposed by the author in 1972 and extended to include the primary creep by Ladanyi and Johnston (1973), is given by

$$\varepsilon_e = (\sigma_e/\sigma_{c\theta})^n(\dot{\varepsilon}_c t/b)^b \tag{6}$$

where subscript e denotes the von Mises equivalent stress and strain, n and b are creep exponents, and $\sigma_{c\theta}$ is the reference stress for a given temperature (where θ is the number of degrees C below zero) at an arbitrary reference strain rate $\dot{\varepsilon}$.

The effect of temperature on creep of a frozen soil can be included in the value of the parameter $\sigma_{c\theta}$ by means of an empirical formula (Ladanyi, 1972)

$$\sigma_{c\theta} = \sigma_{co}(1+\theta/\theta_c)^k \tag{7}$$

where $\theta_c = 1°C$, k is the temperature exponent, usually slightly smaller than one (except for ice where $k \approx 0.4$) and σ_{co} is the value of reference stress $\sigma_{c\theta}$ obtained in unconfined compression creep tests, extrapolated back to 0°C, as shown in Ladanyi (1972).

Introducing the failure strain in uniaxial compression ε_{ef}, or the minimum creep rate $\dot{\varepsilon}_{ef} \approx \varepsilon_{ef}/t_f$ as an additional parameter, the creep strength in compression can be deduced from Eq. (6) and expressed as

$$\sigma_{ef} = \sigma_{c\theta}\varepsilon_{ef}^{1/n}(b/\dot{\varepsilon}_c t)^{b/n} \tag{8}$$

which for b = 1 reduces to the rate-sensitivity-of-strength equation, similar to Eq. (1),

$$\sigma_{ef} = \sigma_{c\theta}(\dot{\varepsilon}_{ef}/\dot{\varepsilon}_c)^{1/n} \tag{9}$$

The effect of normal pressure on creep and strength can be taken into account in several different ways (Ladanyi, 1972), but the effect is most often neglected in the design of straight-shafted piles in permafrost.

As shown in Ladanyi (1985), there are essentially two different methods for deducing from CPT data the values of creep parameters n and in Eq. (9), one assuming the formation of a plastic zone around the cone, and another based only on the highly non-linear creep behaviour of frozen soil.

In the first one, which is a delayed plasticity method, it is assumed, as in unfrozen soils, that the cone resistance is given by:

$$q_c = p_o N_q^o + c(\varepsilon, \theta) N_c^o \tag{10}$$

where p_o is the overall total ground stress at the cone level, and N_q^o and N_c^o are bearing capacity factors for a deep circular punch.

The cohesion c in Eq. (10) is found to be proportional to the strain rate to the power 1/n, which, in turn, is proportional in the penetration rate, i.e.,

$$c \propto \dot{\varepsilon}^{1/n} \propto \dot{s}_c^{1/n} \tag{11}$$

This enables to write from Eq. (10)

$$(q_c - p_o N_p^o) \propto \dot{s}_c^{1/n} \tag{12}$$

leading to the conclusion that, for equivalent strain rates (Ladanyi, 1982), the exponent n in Eq. (9) can be determined from two different cone resistances, q_{c1} and q_{c2}, recorded in a homogeneous soil at two different penetration rates, $\dot{s}_{c1}$ and $\dot{s}_{c2}$. One gets then

$$n = \frac{\log(\dot{s}_{c2}/\dot{s}_{c1})}{\log[(q_{c2}-p_o N_q^o)/(q_{c1}-p_o N_q^o)]} \tag{13}$$

which is the slope of a line passing through the experimental points relating log $\dot{s}$ with log $(q_c - p_o N_q^o)$ in a rate controlled penetration test. At shallow depth the effect of p_o in Eq. (13) may be neglected.

In Eqs. (10) and (13), N_q^o and N_c^o can be calculated from the spherical cavity expansion theory, developed by Ladanyi and Johnston (1974), which takes into account the non-linearity of creep behavior. For $\phi = 0$, that theory gives $N_q^o = 1$, and

$$N_c^o = 1 + \frac{4}{3}\left[n + \ln \frac{2}{3\varepsilon_{1f}}\right] \qquad (14)$$

where ε_{1f} is the failure strain in compression. For $\emptyset > 0$, Ladanyi an Johnston (1974) give the values of N_c^o and N_q^o in a graphical form.

In Ladanyi (1985), it is also shown how the value of $\sigma_{c\theta}$ can be determined by the first method. The method requires, however, the knowledge of the relationship between the penetration rate and a representative strain rate, which is still a rather controversial problem.

This knowledge is not necessary if $\sigma_{c\theta}$ is determined from the deep penetration theory, assuming non-failure conditions. This leads to an approximate relationship for the penetration rate (Ladanyi and Johnston, 1974, Nixon, 1978):

$$\dot{s}_c \approx (3/2n)^n (B/2)\dot{\varepsilon}_c[(q_c - p_o)/\sigma_{c\theta}]^n \qquad (15)$$

valid for ice-rich soils ($\emptyset$=0), and n > 3, from which the value of $\sigma_{c\theta}$ is then

$$\sigma_{c\theta} = (q_c - p_o)(\dot{\varepsilon}_c B_c / 2\dot{s}_c)^{1/n}(3/2n) \qquad (16)$$

Alternatively, if a series of step-loaded static cone tests has been carried out, the resulting set of creep curves can be treated in manner similar to that used for the interpretation of pressuremeter creep tests (e.g., Ladanyi and Johnson, 1973), from which all the three creep parameters in Eq. (6) (b, n, and $\sigma_{c\theta}$) can be determined.

Recapitulation and Recommendations

(1) In the area of determination of engineering properties of permafrost soils, the Cone Penetration Test can be placed somewhere between the geophysical borehole logging and the laboratory testing of samples taken from boreholes. Similarly to the former, the CPT gives a continuous information on certain engineering properties with depth, but this information is in the domain normally covered by laboratory testing.

(2) In offshore use, although the CPT furnishes less detailed data on soil behaviour than a pressuremeter test, its advantage is that it does not require a specially drilled borehole, and that it has a good potential for automatization, i.e., a continuous performance by remote control. In saline frozen soils the use of a piezocone may also be beneficial for studying their consolidation behaviour.

(3) In frozen soils, the CPT can be performed in two different ways: either as a penetration-rate-controlled test, or as a stage-loaded, stress-controlled test. The purpose of the two types of tests is to find the relationship between the penetration rate and the cone resistance, from which certain creep and strength parameters of the soil can be determined. For this to realize, it is necessary to have a loading system which enables an accurate control of either the load applied to the cone, or the cone penetration rate, of both.

(4) Since the test gives information on the soil behaviour only for the ground temperature profile occurring during the test, this temperature should be known from separate sources, to be able to generalize the obtained information.

(5) The CPT is usually performed in the range of strain rates much higher than those needed in the design of foundations. In order to be able to extrapolate the results to very low rates of strain, more information is needed on the whole shape of the rheological curves (σ vs $\dot{\varepsilon}$ - relationships) of frozen soils for a wide range of strain rates, which are found to be affected by the type of frozen soil, its density, salinity, ice saturation, and temperature. The present data base in this area is still rather poor, and much more systematic laboratory work is needed to fill the knowledge gaps.

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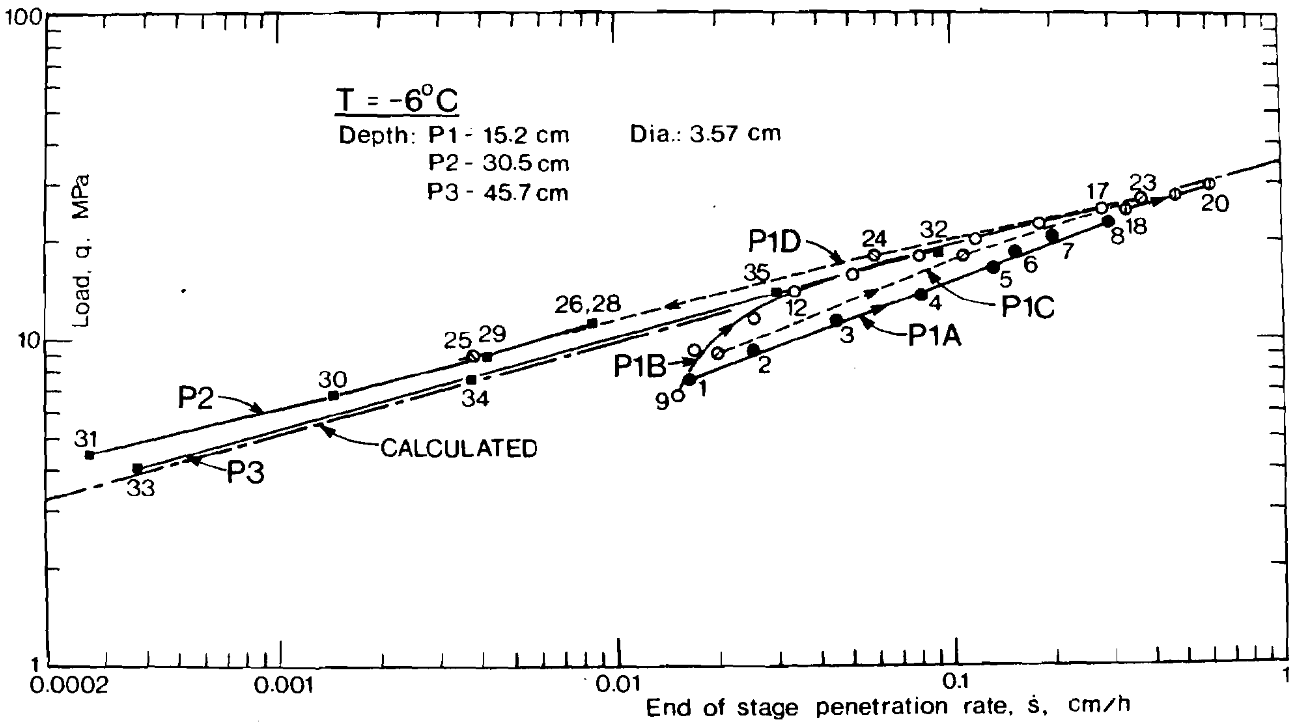

Figure 1. Pressure vs penetration rate relationship, obtained in deep borehole loading tests in frozen sand (After Ladanyi and Paquin, 1978).

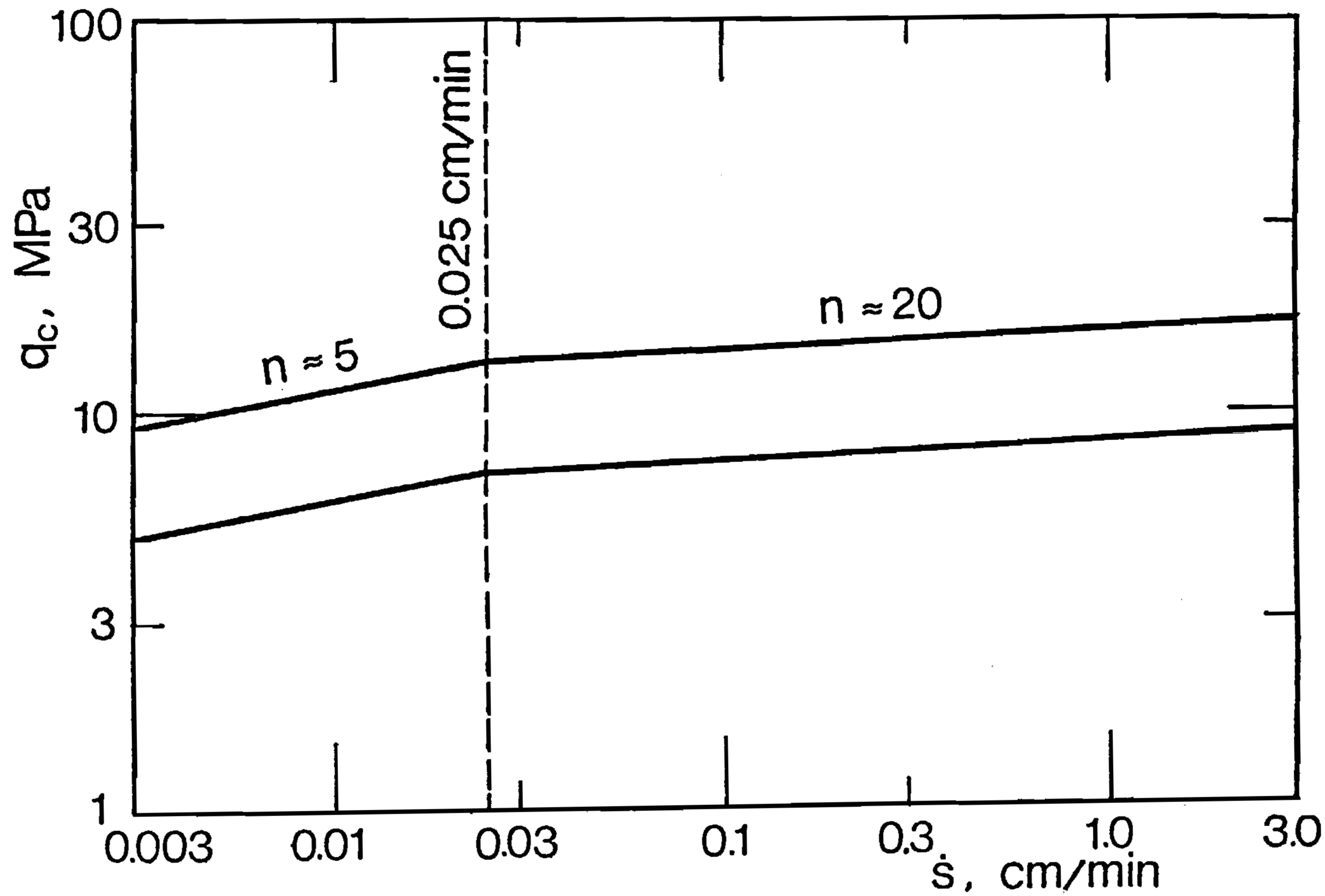

Figure 2. Range of results of cone penetration tests carried out in a frozen varved silt at $-0.3^{o}C$. (After Ladanyi, 1976)

MECHANICAL PROPERTIES OF SUBSEA PERMAFROST

W.D. Roggensack*

(Presented at Workshop but manuscript not available for Proceedings)

* See List of Registrants (Appendix A)
for affiliation of authors.

IMPACT OF OFFSHORE PERMAFROST ON OIL AND GAS PRODUCTION

C.A. Graham*

Introduction

Exploration for oil and gas has been carried out on the Beaufort Continental Shelf during the last 14 years. The distribution and thermal condition of the Beaufort subsea permafrost have been described by Mr. Blasco and Dr. Judge.

This presentation will focus on the impact of subsea permafrost on production development. Due to the limited time, subsea pipelines will not be included.

The following topics will be briefly covered:

(1) an explanation of permafrost thaw subsidence,
(2) a description of the potential effects, or impacts, of permafrost thaw on production system components,
(3) an overview of requirements in order to carry out design engineering,
(4) an evaluation of the present state of knowledge and readiness for permafrost related design,
(5) Gulf's present and projected permafrost studies, and
(6) a summary of recommended areas for industry/government research.

Permafrost Thaw Subsidence

Production of warm oil and gas through permafrost will cause thawing around the well casings. Volumetric changes as the permafrost thaws will result in downward movement at the top of the permafrost, uplift at the permafrost base, and lateral movement towards the wells (Figure 1). As the thaw radius increases, these strains progress inward from the permafrost boundaries. A layered lithology, particularly sand and clay, may result in alternating compressive and tensile strains along the well axis (Figure 2).

Impact on Development Components

With this understanding, we can now consider the effects of permafrost thaw subsidence on the development system components (Figure 3).

A general requirement for offshore production is that numerous wells are drilled at very close spacings. Thaw zones around the individual wells will, in time, coalesce to form one large thaw zone beneath the production structure.

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* See List of Registrants (Appendix A) for affiliation of authors.

The deformations of the permafrost described earlier will induce axial strains in the well casings. Lateral movement of the thawing soil may cause bending strains in the outermost wells of the cluster, particularly at the base of permafrost. At present, designs assume that wells will be deviated below the base of the permafrost. Raising this deviation point up into the permafrost body would allow increased reach from the production structures; however, the impacts of soil movement on casing strains may be more severe.

Subsidence within the permafrost at depth will result in settlements at the seabed, the amount being a function of the permafrost subsidence and the depth to the permafrost. Design impacts will be possible weakening of the foundation, reduction of freeboard in the event of overall structure settlement, partial loss of bearing support below rigid structures, distortion of pipeline tie-ins, differential movement between the wellheads and the structure, and for an island, tilting of the surface-mounted process equipment.

Requirements Prior to Design

The production system components must be able to tolerate the impacts of permafrost thaw. Mitigative measures would be required if these impacts are intolerable for the component designers. To predict the impacts we must firstly know the thickness and properties of the permafrost, and secondly have accurate analytical tools.

The following are data/information, and the basic analytical methods required to evaluate the effects of permafrost thaw subsidence on the production system components.

Data/Information

(1) distribution and thickness of permafrost
(2) geologic model with stress history
(3) ground temperature profile and history
(4) detailed site specific lithology and index properties
(5) pore water salinity
(6) ice content and phase change thaw strain
(7) mechanical and consolidation properties
(8) thermal properties
(9) permafrost creep characteristics.

Analytical Methods

(1) numerical thermal analysis model
(2) thaw subsidence deformation model
(3) soil/casing interaction model.

Appraisal of Present State of Knowledge

The following briefly considers why each of the aforementioned items is important, the adequacy of our present state of knowledge, and what can be done to fill any gaps prior to development.

1. Distribution and thickness of permafrost

Mr. Blasco has described the regional distribution of permafrost. For site specific conceptual design we rely on interpretation of seismic site surveys, geotechnical boreholes, and the logging program from the exploration well. A deep geotechnical corehole would be very beneficial, if not essential, for preliminary design.

2. Geologic model with stress history

It is essential to understand whether the permafrost soils were normally consolidated before freezing, or whether the soils were deposited and subsequently frozen in successive layers. If the latter were true, the thawed soils would be underconsolidated at the present state of stress and would experience greater thaw strains and consolidation strains. Development of the geologic model, under the direction of Mr. Blasco, requires in part laboratory testing of undisturbed core samples and interpretation of exploration well logs. Based upon findings to date, we believe that the soils were normally consolidated before the onset of permafrost; however, this must be confirmed by site specific geotechnical testing at the development location.

3. Ground temperature profile and history

Dr. Judge described thermal conditions in offshore permafrost. Ground temperatures are required to evaluate the fraction of pore water that is frozen, as this affects the thermal properties, thaw strains, and mechanical properties of the permafrost and thawed soil. Ground temperatures are also required as boundary conditions for thermal analysis. To date, we have very good temperature data to 160 metres at Tarsiut N-44 and to shallower depths at geotechnical boreholes, and have limited data from exploration wells.

4. Lithology and index properties

Regional lithology can be approximated from geophysical site surveys. Detail lithology at potential production sites is determined from drill cuttings in the mud returns and the suite of logs run in the exploration well. Sometimes small sidewall samples are taken in the well. Geotechnical boreholes from the exploration site investigation provide information at shallow depths.

It is difficult to accurately determine the index properties of the soils without semi-continuous, representative sampling. This sampling program is very expensive. In general it is justified only after encouraging exploration results and a decision to proceed with production development.

5. Pore water salinity

The warm subsea permafrost is partially or marginally frozen. Pore water salinity not only depresses the freezing/thawing temperature, but causes an unfrozen water content versus temperature distribution for sands similar to that of clays. Recall that unfrozen water content affects thermal properties, thaw strain and mechanical properties. Pore water salinity probably has a greater influence on these properties than ground temperature. At present, we have encountered high pore water salinity to the entire depth of the 160 metre deep Tarsiut N-44 corehole. We are attempting to infer salinity at other locations from the geophysical well logs. Site specific information from a deep corehole will be required prior to preliminary engineering studies.

6. Ice content and phase change thaw strain

The Beaufort Sea operators have, to date, found no indications of significant excess ice within the deep permafrost. Gulf has initiated a laboratory study to evaluate whether excess ice could have formed at depth in saline soils. If excess ice is not encountered, thaw strain would be a function of the phase volume change of the portion of pore water that is frozen, and dissipation of any excess pore pressure.

7. Mechanical and consolidation properties

Mechanical properties, such as the modulus of elasticity and Poisson's ratio, determine the deformation of soil layers in response to volume and stress changes. The previous speaker, Dr. Roggensack, has described the relevant factors affecting mechanical properties. Evaluation by laboratory testing has been limited, having been conducted on Tarsiut N-44 core and reconstituted samples. Gulf's conceptual studies for Amauligak have used mechanical properties determined by the methods described by Dr. Roggensack.

Consolidation properties affect the amount and rate of time dependant deformation resulting from redistribution of pore water. In general, this is in response to negative excess pore pressures in sands and positive excess pore pressures in clay layers.

8. Thermal properties

Thermal properties are a function of lithology, pore water and pore ice contents. As previously noted, the pore ice content is itself a function of temperature, salinity and lithology. Frozen water content distributions have been determined in the laboratory or calculated from published equations.

9. Creep characteristics

We have not yet evaluated the potential effects of permafrost creep in response to the stress changes. Creep may reduce the arching effect at the thawed soil-permafrost boundary, and result in the high strains at the top of the permafrost being experienced to greater depths.

10. Thermal model

An accurate thermal model is necessary to calculate thawing with time around the production wells. The model must be capable of stimulating temperature dependant thermal properties and complex boundary conditions. Gulf and others have conducted thermal analyses for single and multiple well scenarios and are satisfied with the accuracy.

11. Deformation model

Most of the so called "thaw subsidence models" are numerical models which compute deformation in response to volume, stress and soil stiffness changes. The model developed by consultants for Gulf takes both immediate phase change strains and time dependant consolidation strains into account. Because of limited deformation, the soils have been modelled as non-linear elastic elements.

Figure 4 shows predicted soil deformation in response to thaw (scales have been removed for confidentiality). Figure 5 presents predictions of permafrost surface and seabed settlements. Figure 6 shows predictions of vertical soil strains at the centre of the thawed column, for several time intervals after production start-up.

The major modelling limitation is the determination of representative thaw strain and mechanical properties for the soil. For this reason, most studies include parametric analyses to bound the range of predictions.

A second potential limitation is the requirement for verification and acceptance of the thaw subsidence model.

12. Soil/casing interaction model

Soil/casing interaction analyses are carried out to evaluate slip between the casing and subsiding soil, to evaluate lateral soil restraint to buckling, and to predict casing strains. Our results indicate that slip may generally be limited to the upper 100 metres, below which casing strains will equal the predicted soil strains.

Gulf's Present and Projected Permafrost Study Program

Ongoing and future Gulf studies are focussed on improving our knowledge of the soil and permafrost properties at our Amauligak prospect, and improving and verifying the thaw subsidence analytical model. The projects include:

(1) a study of coring, sampling and in-situ testing methods to support a deep geotechnical core through the permafrost,

(2) a study of laboratory testing methods required to obtain representative thaw strain and mechanical properties from deep core samples,

(3) laboratory testing using reconstituted samples to evaluate mechanical and thaw strain properties for input to the thaw subsidence model,

(4) conducting a geotechnical coring program through the entire permafrost sequence, from Gulf's Molikpaq structure, at the Amauligak I-65 site in mid-1986. The execution of this costly program is contingent on successful delineation results this winter,

(5) laboratory testing to determine index, thermal, mechanical and consolidation properties, pore water salinities, porosity, density, etc. using the core,

(6) laboratory evaluation of the potential for excess ice to have formed in saline sediments during the formation of permafrost,

(7) laboratory scale model tests for verification of the thaw subsidence deformation model,

(8) evaluation of raising the well deviation point up into the permafrost,

(9) development of permafrost thaw impact design criteria for all development system components,

(10) assessment of state-of-the-art hardware and software for future well casing instrumentation, and

(11) study of the effects of possible in-situ gas hydrate degradation, particularly within the permafrost.

These projects may be modified, supplemented or deleted in response to perceived study needs or the exploration results at Amauligak.

<u>Recommendations for Government/Industry Research</u>

Gulf's studies are focussed on the Amauligak development, although it is recognized that government research should be of a more generic nature. Several areas of research are suggested which would meet both industry and government requirements for orderly development. These include:

(1) continued development of the soil and permafrost geologic model. This will assist in the understanding of properties contributing to permafrost thaw subsidence,

(2) evaluation of procedures for coring, sampling, sample shipment, sample storage and testing for deep permafrost,

(3) evaluation of in-situ testing methods, particularly the pressuremeter, for measuring modulus properties of permafrost at depths up to 600 metres,

(4) participation in laboratory or field testing programs for verification of the thaw subsidence model. Verification will be essential for industry confidence and government review and approval, if development is to proceed in an orderly manner, and

(5) recognizing that strains in the well casings will exceed the elastic strain at yield, participation in development of a rational strain limit casing design criteria similar to Alaskan North Slope practice.

There are other areas, such as the deep thermistor string proposed by Dr. Judge. Gulf will require site specific information at Amauligak, and would certainly support in principle a thermistor string if it could be combined with our proposed deep corehole.

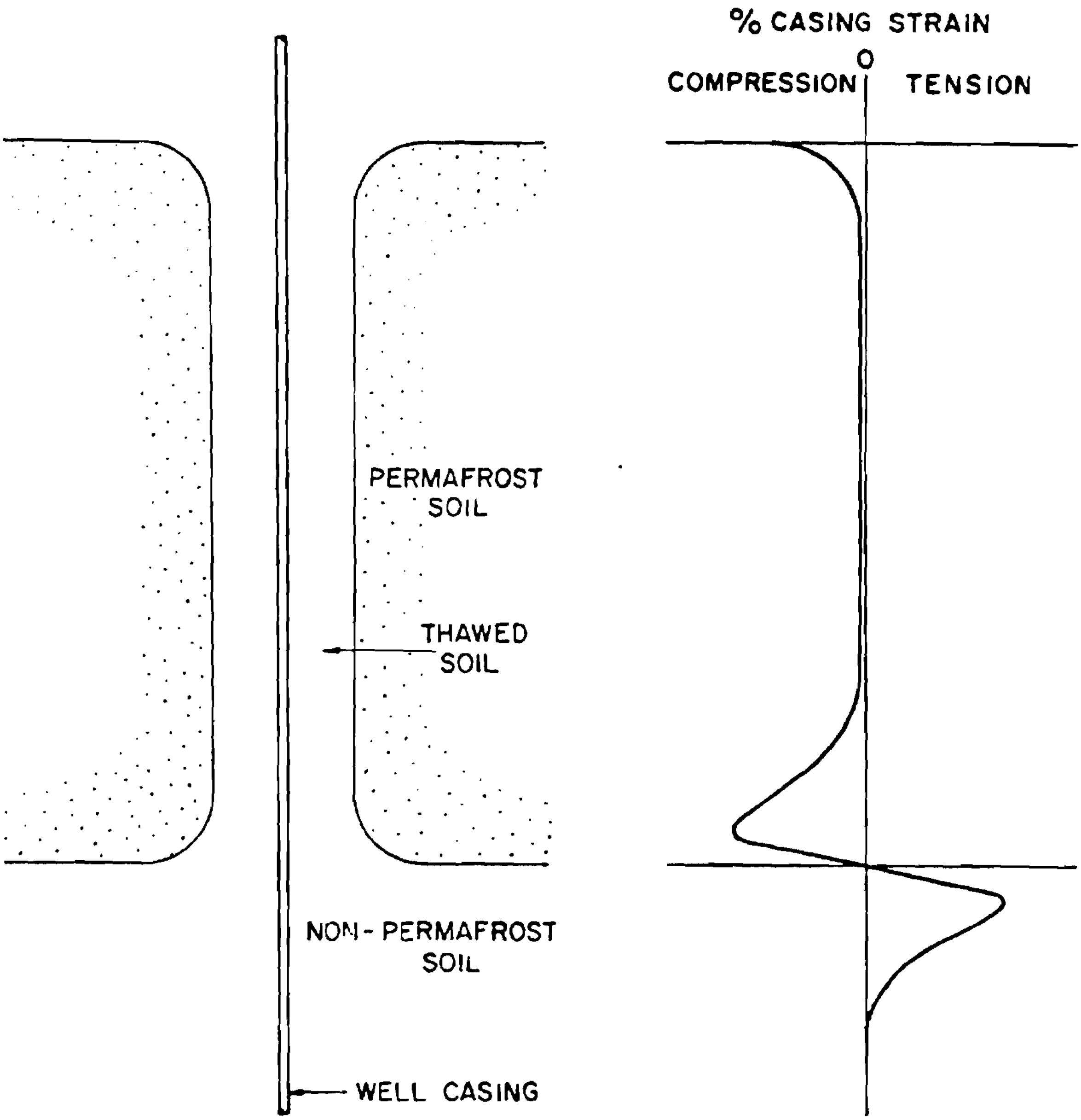


FIGURE 1 AXIAL STRAIN FOR SINGLE WELL THROUGH PERMAFROST, HOMOGENEOUS (NON-LAYERED) SOIL

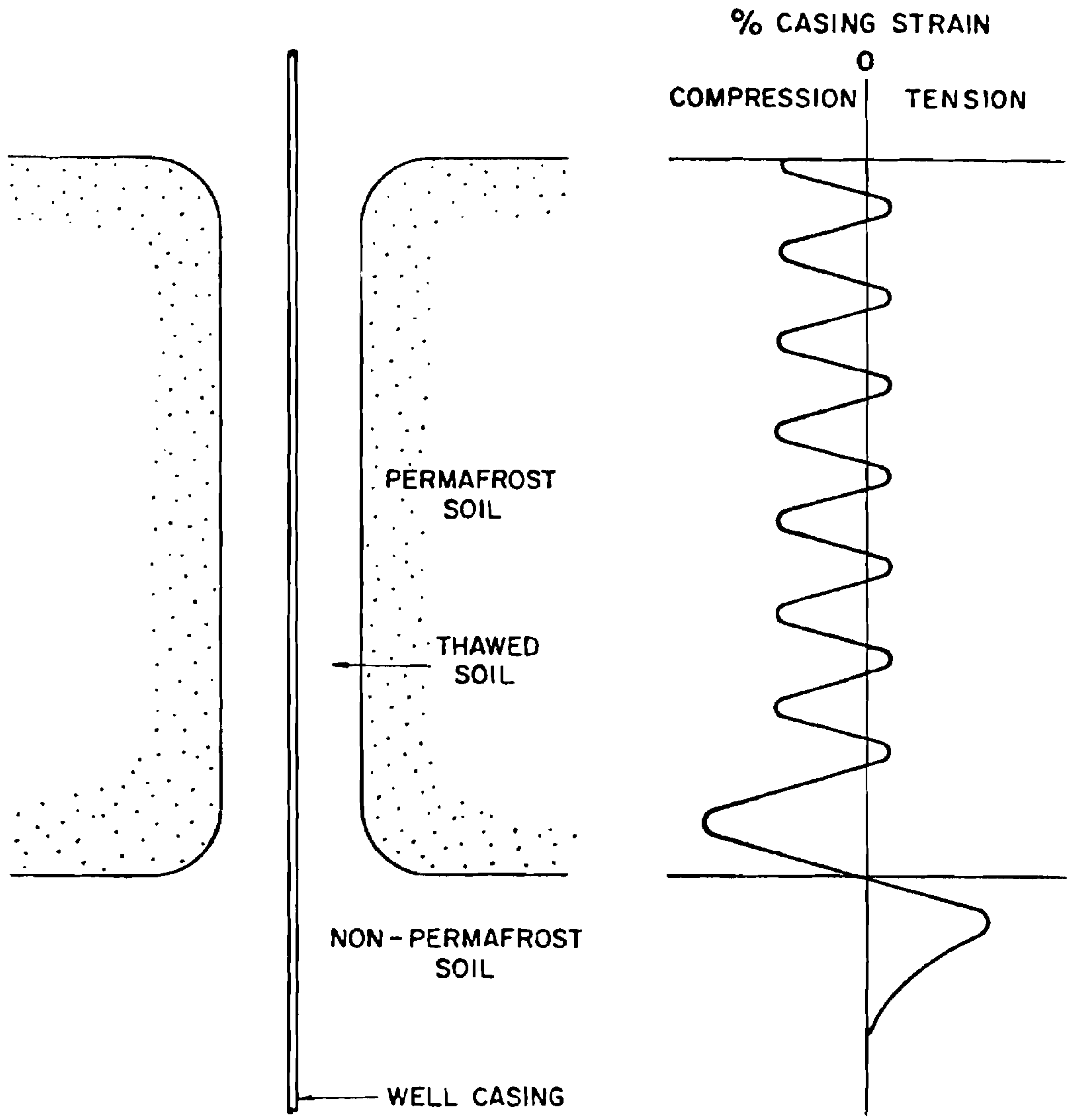


FIGURE 2. AXIAL STRAIN FOR SINGLE WELL THROUGH PERMAFROST, NON-HOMOGENEOUS (LAYERED) SOIL

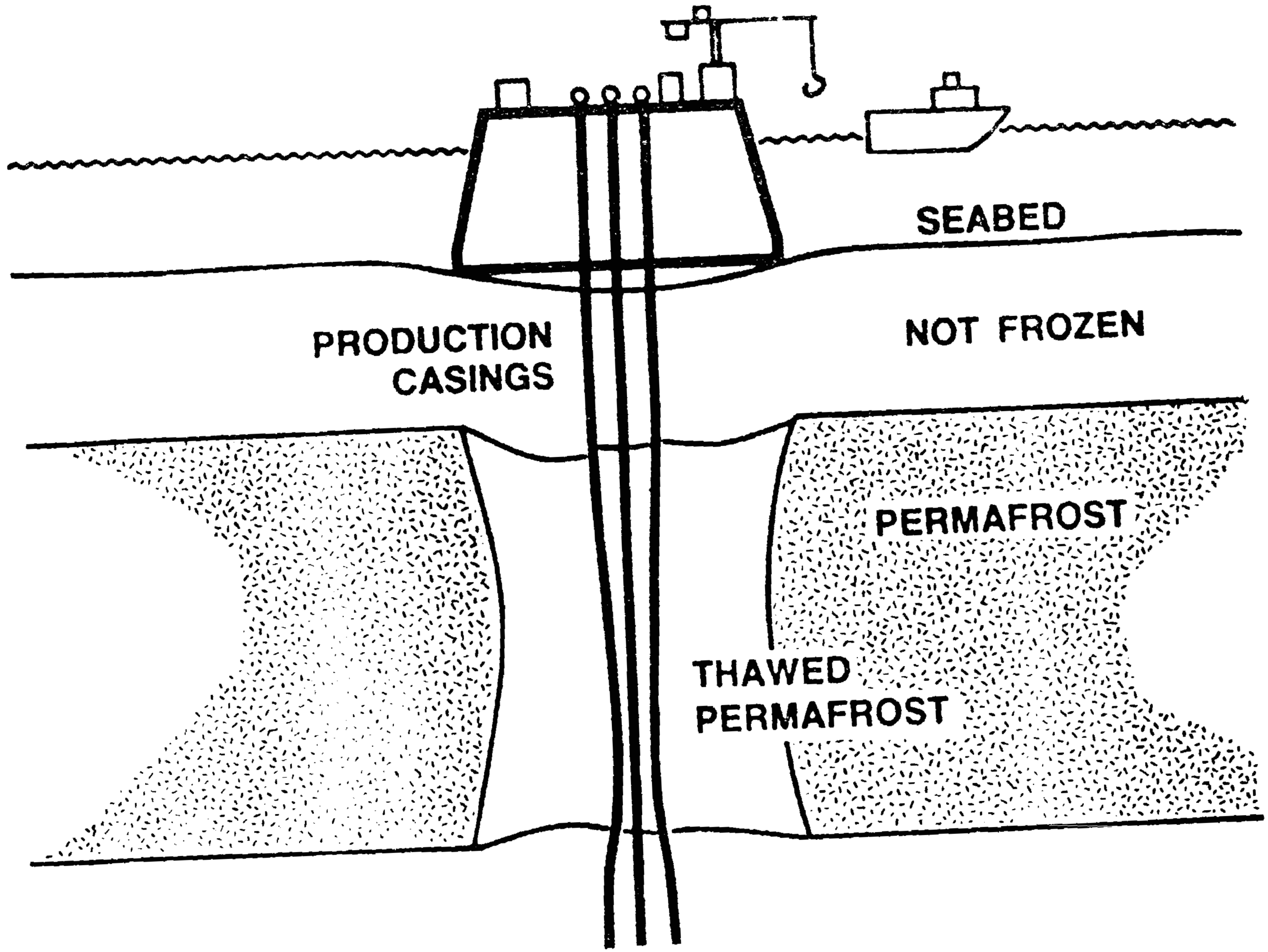
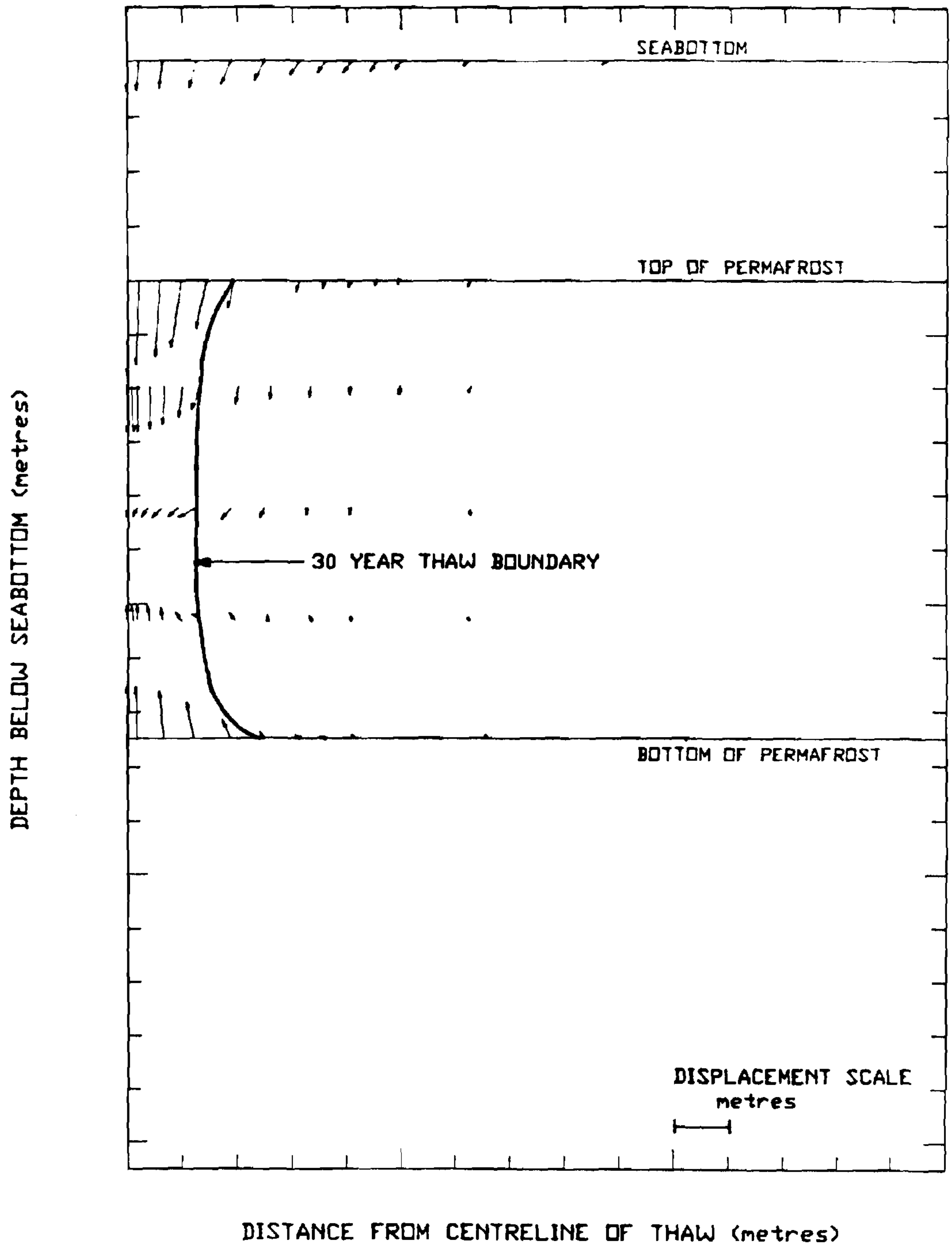
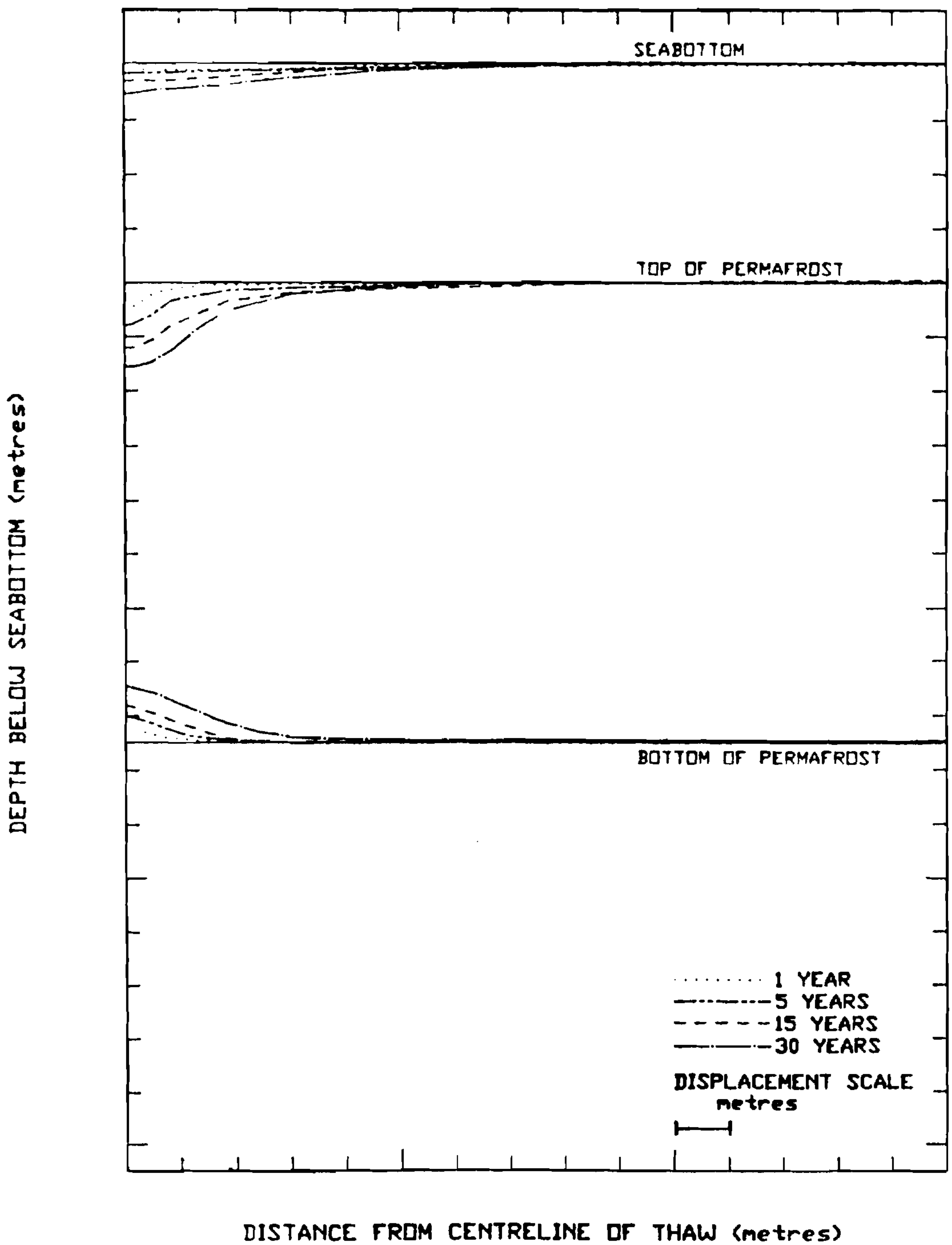


FIGURE 3. EFFECTS OF PERMAFROST THAW SUBSIDENCE



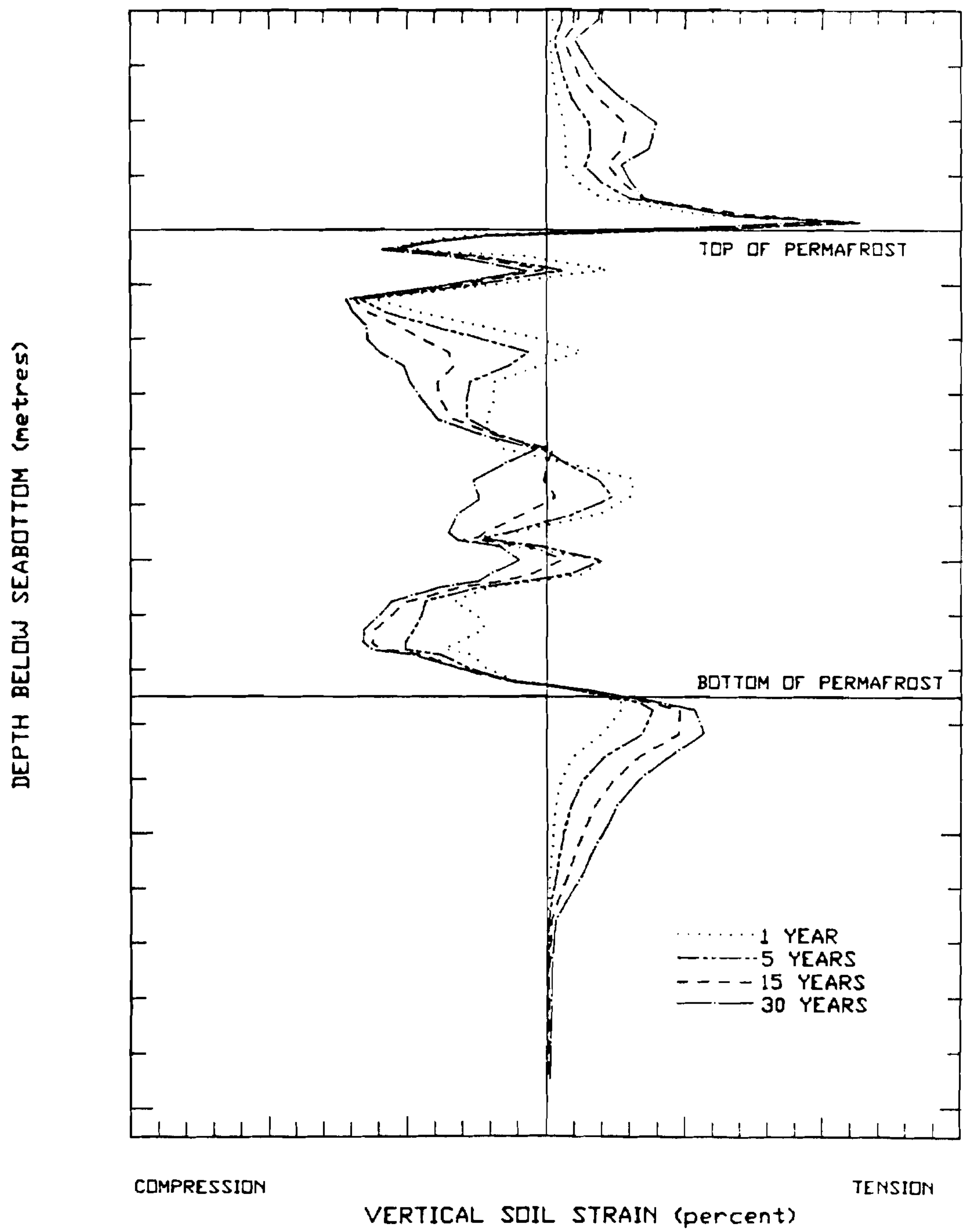
NOTE: Scales have been removed for confidentiality

FIGURE 4. VECTOR DISPLACEMENT PLOT OF SELECTED NODAL POSITIONS AFTER 30 YEARS OF THAW



NOTE: Scales have been removed for confidentiality

FIGURE 5. TRANSIENT SOIL DISPLACEMENTS AT SELECTED STRATIGRAPHIC ELEVATIONS



NOTE: Scales have been removed for confidentiality

FIGURE 6. TRANSIENT VERTICAL SOIL STRAIN PROFILES AT CENTRELINE OR THAW ANNULUS

SUBSEA PERMAFROST

SUMMARY OF DISCUSSION AND RESEARCH NEEDS

Most authors in their presentations at this Workshop addressed the status of information and knowledge available at the present time (with emphasis on the Beaufort Sea area) and also future research needs. Some authors referred to them in a more general way, others were quite specific, depending on the aspects they were covering. Their remarks are included in the preceding papers. Further points were brought out in the open discussion.

In general, it was possible to clearly identify important factors and focus on future research needs and priorities. Some principal highlights are noted in the following paragraphs.

Present acoustic/seismic geophysical methods of delineating permafrost are reasonably satisfactory and permit rough information to be obtained with a minimum number of boreholes. Interpretation may be very difficult, however, particularly when overconsolidated sediments are encountered and the methods are insensitive when ground temperatures are $< -2°C$. Based on all available information there is poor correlation between seismic and electromagnetaic delineation methods. More work needs to be done on correlating geophysical methods with engineering properties of the materials encountered.

Thermal modeling on the geological time scale indicates that insufficient information is presently available on past sea levels and climates and on material properties affecting thermal coefficients. The processes involved in the transport of heat and salt in saline soils are not well understood.

Evaluation of existing technology and further improvements in borehole instrumentation for in situ measurement of moisture, density and mechanical properties are required. More effort should be devoted to methods of evaluating ice contents in situ. The effects of sample handling and storage on laboratory - determined material properties need to be investigated. Improvements in the treatment of pore water salinity are required for modeling thaw subsidence problems around well bores. More information is needed on thermal properties of both frozen and thawed subsea materials (particularly saline soils) as well as laboratory and field work, to properly evaluate thaw subsidence models.

Dr. J.F. Nixon acted as reporter for the day and his remarks on Subsea Permafrost R&D Needs and Priorities, given at the last session, are summarized below under three main headings. For many of the following, it is necessary to distinguish between (a) permafrost, (b) ice-bearing permafrost, (c) ice-bonded permafrost, and (d) those saline soils that might yield excess water upon thawing under their present-day overburden pressures.

Geological

- better understanding of depositional environments in the Beaufort Sea area,

- correlation between engineering properties and results from geophysical techniques,

- instrumented borehole on Beaufort Shelf,

- reliable sea level curves for input to offshore thermal models.

Geotechnical

- application of TDR (and other methods) to delineating ice content in warm saline permafrost,

*- sample storage, temperature control and transportation: (e.g., thermal shock and ice redistribution),

- optimization of drilling fluids., (e.g., temperature control and borehole stability),

*- stability of fresh-water ice in saline soil,

- systematic testing of typical soils; (e.g., creep, strength and thaw settlement, relationship to porosity),

- ice formation in saline soils under appropriate overburden stresses,

- creep testing of saline fine-grained soils,

- evaluation/improvement of downhole geophysical package,

- evaluation and development of in situ testing methods (i.e., CPT and pressuremeter)

Analytical

- better thermal modelling for saline soils to (a) explain past climate, (b) past depositional environments, (c) present-day profiles and (d) future effects of engineering structures on permafrost,

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* were felt by some to have a higher priority

WORKSHOP ON SUBSEA PERMAFROST

NORMAN WELLS EXPANSION PROJECT - OVERVIEW

R.M. Tibbatts*

Abstract

Esso Resources Canada Limited has completed the expansion of the Norman Wells oil field in the Northwest Territories of Canada. Norman Wells is located on the Mackenzie River, about 100 km south of the Arctic Circle. The expansion project involved building artificial islands in the Mackenzie River, the drilling of highly deviated directional wells, the implementation of a field-wide pattern water-flood scheme and the construction of a buried pipeline 866 km long and 323.9 mm in diameter to northern Alberta. The waterflood will result in recovery of more than 40% of the 100 million m^3 of oil-in-place from the highly fractured, tight limestone reservoir. Crude oil production will be increased from 600 m^3 per day to a peak rate of 4000 m^3 per day.

This paper discusses some of the unique engineering and execution challenges that had to be overcome in the development of this field. Included is artificial island design to withstand ice forces, pipeline construction in both continuous and discontinuous permafrost and modular construction techniques.

Introduction

Esso Resources Canada Limited has completed an oil field expansion project that increased production from the Norman Wells oil field in Canada's Northwest Territories, about 100 km south of the Arctic Circle. Although emphasis was placed on using existing oil field technology to minimize risk, the project presented some unique challenges, because most of the oil-bearing reservoir is located directly under the Mackenzie River, and because the project site is in a remote location that has a harsh climate.

This paper provides a brief overview of the Norman Wells Expansion Project and discusses some of the engineering solutions and project execution approaches that were developed to cost effectively overcome these typically Arctic conditions. Included are design considerations for man-made production islands and pipeline construction in permafrost and the use of modular construction techniques.

Development Background

Norman Wells is a settlement of about 400 people located on the banks of the Mackenzie River in Canada's Northwest Territories (Figure 1). The climate is characterized by long, cold, dark winters and short summers. The river provides the major transportation corridor for moving heavy freight into Norman Wells, since there is no winter

* See List of Registrants (Appendix A) for affiliation of authors.

road. However, an all weather modern airport is maintained by the Canadian Government.

Settlements in this sparsely populated region were traditional native hunting, trapping and fishing communities, often located at tributaries flowing into the Mackenzie River. The settlement of Norman Wells did not exist before Imperial Oil Limited (Esso Resources Canada Limited's parent company) discovered oil there in 1920.

About 40% of the oil reserves at Norman Wells are accessible from two natural islands in the Mackenzie River (Bear and Goose Islands) and from the mainland shore. Prior to expansion, about 475 m^3 of oil a day was produced to provide feed stock for the small, distillation-type refinery. Refined product is distributed to regional communities by barge between June and September when the Mackenzie River is ice free.

Esso evaluated the feasibility of expanding the Norman Wells oil field in the late 1970s. The study determined that it was technically feasible and economically attractive to expand the oil field's production by implementing a field-wide, five-spot pattern waterflood scheme, and by developing the previously inaccessible portion of the reservoir which lies under the Mackenzie River.

Expansion Facilities

The facilities required include six man-made production islands and 150 new wells for oil production and water injection (Figure 2). Eighty-six of the wells, most of which will be deviated to some degree, will be drilled from the production islands. To achieve adequate reservoir drainage, horizontal displacements that exceed 550 m and maximum wellbore angles of 70° will be required on many wells. Two drilling rigs are planned to complete the development drilling programme. While reservoir pressure will be maintained using water injection, about 85% of the wells will have artificial gas lift to optimize production.

Surface facilities for gathering and processing the production will consist of flow lines, main gathering lines, satellite testing and control stations and a central processing facility on the mainland consisting of equipment for crude oil treatment and storage, produced water treatment and disposal, gas processing and compression, fresh water treatment and injection and electrical power generation.

These field development facilities cost $530 million and increased production to 4000 m^3 a day.

Pipeline Project

In a separate but dependent project, Interprovincial Pipe Line (NW) Limited constructed and operate a 324 mm outside diameter buried pipeline to ship crude oil 868 km from Norman Wells to join an existing pipeline system at Zama, Alberta. The $360 million pipeline will have a capacity of 5000 m^3 per day.

Schedule

The project was completed in 1985. Applications to proceed with the project were filed with regulatory agencies in March, 1980. During public hearings, conducted in August and October, 1980, considerable public input was provided on such aspects as timing, approval conditions to maximize local benefit, and approaches to mitigate adverse social and environmental impact.

The Government of Canada approved the project in July, 1981. However, to allow time for social planning, substantive construction was not allowed to start until 1983. Consequently, field activity in 1982 was concentrated on installing the infrastructure, such as docks, roads and camps, that were required to support the main construction work force which peaked at 900 in mid 1983 to 1984. A two rig development drilling program also started in 1982.

Production Islands

About 60% of the reservoir lies 400 to 700 m below the Mackenzie River. Although the reservoir could be accessed by directional drilling from natural landforms, man-made production islands were selected because of the ability to waterflood the reservoir more effectively using this approach.

The islands (Figure 3) are designed to provide safe, year-round operation over the reservoir's 30 year lift. Four islands, founded on stiff clay layers, were built along the south side of the main navigation channel and two islands, founded almost directly on shale, will be built along the north side. Although all six sites are different, all the islands were designed to withstand the most critical environmental conditions encountered at the site.

The working surface of each island, one metre above the design flood level and 2.6 m above the highest recorded water level, provides 3600 m^2 of space to allow a full size rig to drill up to 20 wells. The remaining space is taken up by a camp to accommodate 25 people and storage areas.

The islands were constructed of locally available rock and sand fill. A ring of light quarry rock will retain the sand fill which will form each island's core (Figure 4). The core and rock ring were covered with three layers of varying sized armour rock to protect the slopes from waves, currents and ice. Two outer layers of 30 kg to 150 kg and 150 kg to 800 kg limestone overlie a filter fabric that separates the sand core from the armour. The filter fabric prevents sand from seeping out through the armour, while allowing water to pass through it freely, thus preventing excess pore pressures from building up in the fill.

The major design challenge for the islands was determining the design conditions for river current flows, flooding, wave erosion and ice scour.

River currents are the key to the design of the island's slope and toe protection. The most extreme currents occur when the ice jams downstream from Norman Wells are released. Although these extreme currents only occur over a few hours, they can be twice as high as the normal currents. Since mathematical and physical models indicated that once every 250 years the release of ice jams would yield maximum currents of 2.7 ms^{-1}, this rate was used for the design.

The river breakup also provided the basis for the design high water level. When an ice jam forms downstream from Norman Wells, the river backs up causing the water level to rise more than 10 m above the normal summer level in just a few days.

Ice will act on the islands in several ways. It will push against the island's mass, ride up on the slopes, and rubble around the structure. During aerial surveys of the breakup process on the upper Mackenzie, ice was observed rubbling on shoals and being pushing up onto the river bank. These same processes will occur around the islands.

The rapid drop in water level which follows the release of a downstream ice jam was a key design factor. The concern was that water in the island fill might not drain away as quickly as the surrounding river dropped, and the pressure of this water could reduce fill stability. The islands were designed to be stable, even with no drainage of the fill, until the water level dropped to its normal summer level.

Pipeline Systems

One of the greatest challenges in designing the pipeline systems was to overcome the problems associated with permafrost. Buried pipelines were preferred, as they are more economical than pipelines placed in above ground utilidors, and are better protected from natural or human induced hazards. This was of prime importance at the lower land elevations where some overtopping of ice occurs during the Mackenzie River's spring breakup.

Norman Wells is located in a zone of discontinuous permafrost, where permafrost exists together with areas of unfrozen ground (Figure 5). Permafrost occurs below all natural forested upland terrain, but is absent on the river banks and below the river. All permafrost alluvial silts and clays encountered in the region have abundant ground ice and a typical thaw strain potential of 20-40%. The seasonal active layer of permafrost varies from less than 0.5 m in undisturbed peat to more than 2 m in partially disturbed or gravel covered areas.

The two main concerns that permafrost imposed on the pipeline system were the freezing of transported fluids and settlement of the pipelines.

Freeze protection is provided for the individual water injection and multiphase production lines by electrical heat tracing, insulation, and burial 1 m deep. The insulation will play a dual role. It will protect the pipelines from freezing ground temperatures, while reducing the heat transferred from the pipeline to the permafrost.

Thaw settlement of permafrost soils is a major concern. The primary factor influencing the amount of settlement is the amount of excess ice present in the soil. Although the total settlement of the soil is of concern, the main concern with buried pipelines is the strain due to differential settlement occurring at transition zones between frozen and unfrozen ground.

Modular Construction

The Norman Wells Project used modular construction techniques extensively, primarily as a cost saving measure. Previous projects in Western Canada have achieved between 25 and 30% savings of total labour man-hours in prefabrication. Since virtually all Norman Wells facilities, except tankage and offsite pipeways, were modularized or prefabricated, about 65% of the field work can be done at the modularization site. Modularized construction saved more than 10% of the cost of non-modularized construction. These significant savings ensure that the modularization concept will be applied to many other projects in remote locations.

The 1600 km overland route from Edmonton (modularization site) to Norman Wells, involved shipping modules halfway to the site by road to Hay River, Northwest Territories, then by barge on the Mackenzie River the rest of the way.

Limitations on load dimensions determined the size of modules that could be transported by road to Hay River (Figure 6). Loads a maximum of 6 m wide and 5 m high can be shipped on the highway to allow clearances through bridges on the route. Some modules weigh as much as 160 t but shipping weights only pose a problem for about six weeks each spring when the highway subgrade is weakened by thawing frost.

Modular construction alters the sequence of detailed design. Structural steel and piping design are completed simultaneously, because all equipment and line locations must be finalized before finalizing the structural member locations. The design of individual modules is virtually complete before fabrication starts, which helps to minimize changes and rework in the field.

The modules have an open air space underneath them to prevent heat from being transferred from the module to the permafrost and to allow space for transportation equipment to manoeuvre underneath the modules to lift them. The modules are supported on peripheral rows of legs, 1 m long, attached to the tops of foundation piles.

The remaining work of placing and interconnecting the modules at Norman Wells can be done with a much smaller labour force than would have been required at the site with non-modular construction. This small work force reduces field labour overhead costs, provides better productivity through lower manpower densities, is more manageable, and minimizes the socio-economic impact on the town of Norman Wells.

Conclusion

Norman Wells is the first major oil field development to proceed in the Canadian North since the 1940s. The challenges that Esso faced in designing and executing this project dealt with such issues as:

(1) obtaining a better understanding of the project site's physical environment to allow for the design of facilities and to apply and adapt known technology to the specific environment conditions at the site. This problem is common to most remote areas, since their isolation often leads to a lack of the detailed site-specific knowledge required for facility design. At Norman Wells, a good technical knowledge of the Mackenzie River and its breakup phenomenon represented the major data gap, for which programmes were initiated to provide the necessary information.

(2) minimizing the cost impact of the remote location, severe Arctic environment and permafrost by maximizing modularization to allow many of the facilities to be completely pre-assembled in populated areas. Pipeline systems were also designed for the specific conditions encountered at the site to minimize the use of more expensive above ground utilidor systems.

The Norman Wells oil field expansion is important to the development of the Canadian North. It will demonstrate that petroleum development can proceed in an environmentally harsh and culturally sensitive area of the country, in a manner that is technically and economically viable, and is beneficial to all the people of Canada.

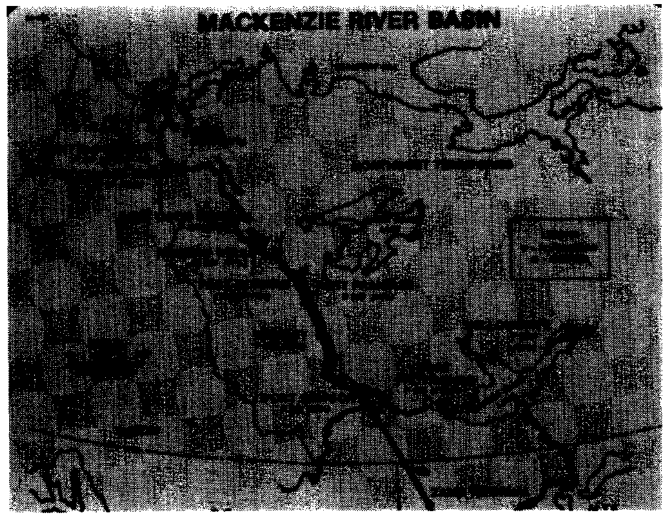

Figure 1. Norman Wells Location

NORMAN WELLS AREA
PROPOSED FACILITIES

Figure 2. Norman Wells Facilities

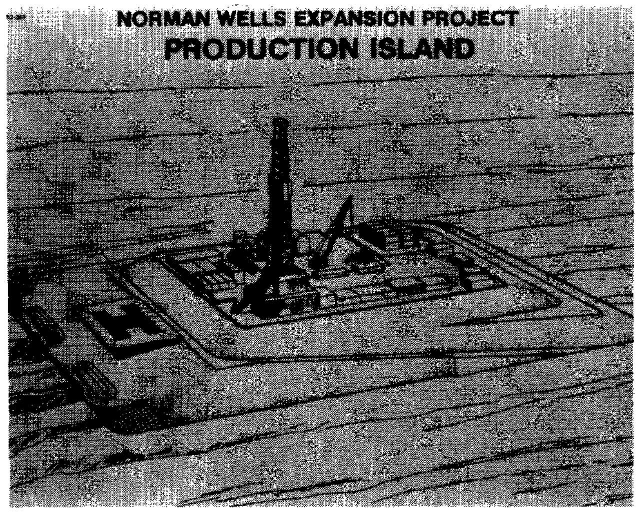

Figure 3. Production Island

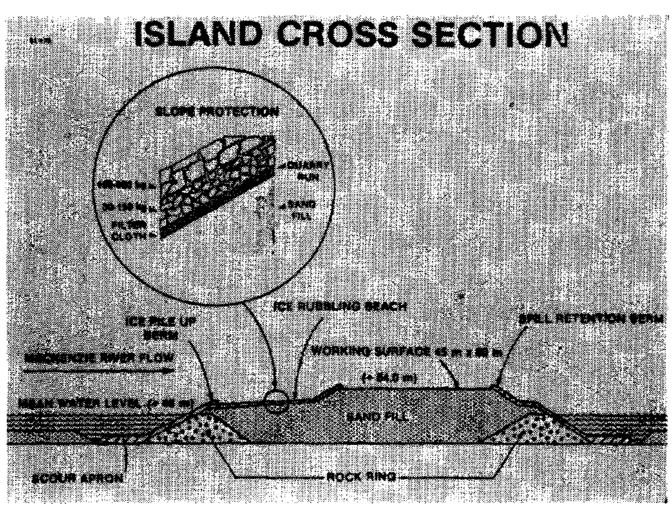

Figure 4. Production Island Cross-section

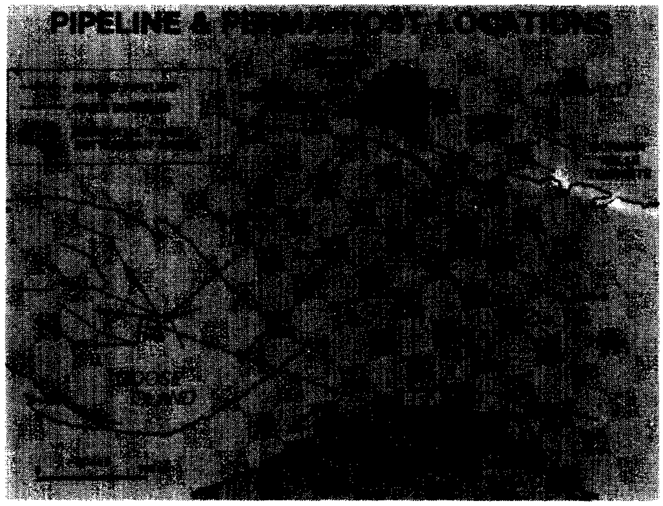

Figure 5. Pipeline and Permafrost Locations

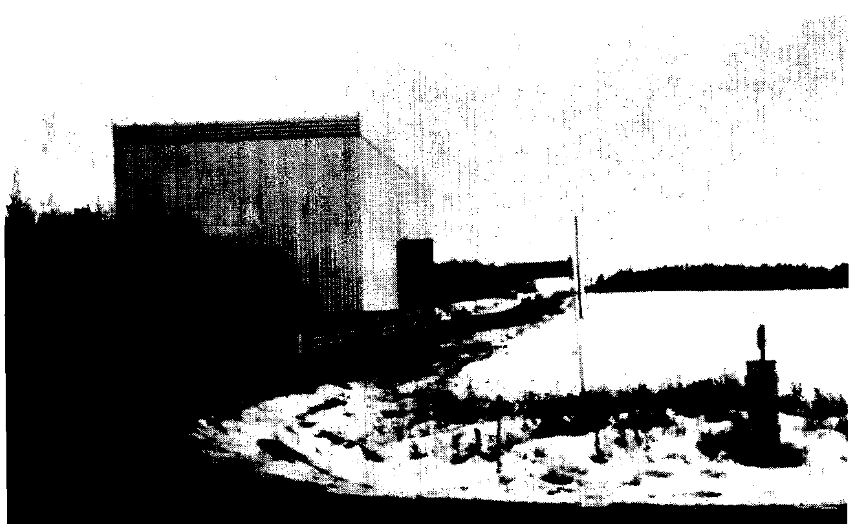

Figure 6. Module in Transit

DESIGN APPROACH TO NORMAN WELLS GATHERING SYSTEM

D.W. Hayley*

(Presented at Workshop but manuscript not available for Proceedings)

* See List of Registrants (Appendix A)
for affiliation of authors.

NORMAN WELLS PIPELINE PROJECT

A.R. Pick*

PART I - OVERVIEW

Introduction

The Norman Wells Pipeline extends from Norman Wells in the Northwest Territories to Zama in northern Alberta. Owned and operated by Interprovincial Pipe Line (NW) Ltd. (IPL), this small diameter (30 cm) buried pipeline carries crude oil 870 km south where it connects to another pipeline system. The pipeline right-of-way extends up the Mackenzie Valley on the east side of the Mackenzie River, crosses the river near Fort Simpson and then continues southeast to Zama, Alberta.

IPL had to build and operate the Norman Wells Pipeline without serious negative effects on the environment or the people in communities along the pipeline route.

In 1979, based on plans by Esso Resources to increase production of the Norman Wells oilfield, an impact assessment examined a proposed pipeline from Norman Wells to Zama. In 1980 an application was submitted to the National Energy Board for authority to construct and operate the "Norman Wells to Zama Pipeline". Public hearings were held by both the Federal Environmental Assessment and Review Panel and the National Energy Board. In 1981 the NEB granted a Certificate of Public Convenience and Necessity allowing IPL to construct and operate the pipeline.

Since 1980, IPL has undertaken numerous engineering, environmental and socio-economic studies to fulfill the terms and conditions of this certificate.

Project Organization

IPL managed all phases of project construction and operation. A Construction Services Manager (CSM) was appointed for material procurement, field management and inspection during construction. Design was managed by IPL using engineering and planning firms.

Project Schedules

Construction of the Norman Wells Pipeline took place over the course of three years, the majority taking place during the winter months of 1983, 1984 and 1985. Construction commenced in January of 1983 with right-of-way clearing. Development of some facility and borrow sites also began at that time and continued throughout 1983. Actual pipeline construction kick-off was in January 1984. The remaining site

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* See List of Registrants (Appendix A) for affiliation of authors.

development and right-of-way clearing was also started at this time. Construction of the pipeline was completed by April, 1985. The pipeline will carry approximately 4,000 cubic metres (25,000 barrels) of oil per day.

The Climate

Although the climate varies considerably in the project area, it is characterized by long, cold winters and short, warm summers.

The Land

From Norman Wells to Fort Simpson the terrain is rolling with numerous valleys and high flat terraces. Well drained areas are covered in tall forests of pine, white spruce, poplar and aspen. Poorly drained areas are covered in black spruce and tamarac. The terrain along the pipeline route is generally flat south of Fort Simpson. Much of this area is poorly drained and dominated by open coniferous forests of dwarf black spruce and tamarack. Stream banks are typically gradual shallow slopes covered in thick willow and alder. In northern Alberta, muskeg is extensive in poorly drained and low lying areas.

Permafrost occurs in over 80 percent of the soils of the route in the Norman Wells area and gradually diminishes to about 30 percent of the route in the Alberta segment.

Planning and Design

Concerns that had to be addressed in the planning/design of the pipeline included:

- public concern regarding social and environmental impacts of construction and operation,
- settlement problems due to permafrost melt,
- frost heave,
- slope stability,
- crossing of two major rivers,
- crossing of 140 water courses of varying sizes,
- frozen, boulder tills,
- shortages of bedding and padding materials,
- very large bogs and fens,
- scarcity of water for hydrostatic testing, and
- problems related to freezing ground temperatures.

A decision was made by IPL to approach construction of the pipeline in such a manner that conventional design and construction procedures would be used to the maximum extent possible. Features resulting from the above decision include:

- location of the route on previously cleared rights of way and clearing one year in advance of construction,
- chilling of oil to near-ground temperature,
- pipeline designed to control maximum compressive and tensile strains,
- reduction of cover to 0.76 m for overland portions of the pipeline,
- low sulphur, high energy steel allowed for ease of welding at low temperature and allowed for fracture arrest,
- certain sections of the pipe were insulated to control frost heave,
- over 150 slopes were designed. Certain critical slopes susceptible to thawing were insulated using a layer of wood chips,
- NEB approved air testing of the line,
- remotely operated valves are designed with radio controlled electro-hydraulic operators,
- above ground pipe, valves and fittings are designed to a -60° criteria, and
- a real time transient model is being developed to allow leaks to be detected.

Conclusion

The Norman Wells Pipeline is viewed by many as a pioneering pilot project for continued resource development in the Canadian North. Our monitoring programs which focus on verification of critical aspects of our design, provide feedback for future development. The Norman Wells Pipeline Project has demonstrated that a pipeline can be built and operated without serious negative effect on the environment or the people.

PART II - THAW SETTLEMENT, DRAINAGE AND EROSION CONTROL

Introduction

Erosion control is an important consideration in the design and maintenance of pipeline right-of-way. Poor drainage control and erosion can contribute to terrain instability and impairment of aquatic habitat. In the north thaw-unstable permafrost soils make drainage control a more important issue.

The design of drainage and erosion control measures had three main objectives: - to maintain pipeline integrity, to maintain right-of-way integrity, and to protect the environment.

The principal factors considered in assessing the rate and severity of erosion due to water action were - vegetation, texture, structure and chemistry of top soil, up-slope topography, rainfall intensity and duration, infiltration rate for soil, and sediment load.

The design approach used for the pipeline was based on consideration of - terrain type, slope, drainage patterns, and design flows.

Terrain along the pipeline was classified into genetic units and within each genetic unit the terrain was further classified on the basis of soil type and genetic origin.

Terrain erodibility indices were derived as a composite of soil erodibility index, thermal erosion susceptibility and topographic index. After classifying each area the erosion control requirements were characterized and design charts were prepared.

Mitigative Measures

Surface Drainage

(a) breaching backfill mound

Ditching results in an expansion of soil volumes. To accommodate subsequent consolidation and settlement, the backfill was mounded over the ditch line to heights of up to 2 m above mean ground level.

In order to maintain cross drainage, the backfill was levelled for short distances. These short breaches in the backfill mound were established wherever cross drainage was evident. Where cross drainage was poorly defined, drainage was trained across the ditch line at intervals up to 500 m apart.

(b) diversion berms

Diversion berms are designed to be a barrier to water movement along the right-of-way, forcing water off the right-of-way at selected locations. Berms were constructed of sand bags. The bags were expected to deteriorate and the surface of the berm would be stabilized by vegetation. Berms were designed to have a 10° slope along the face and the frequency of berm placement was a function of slope angle.

All berm designs worked well on steep or well-defined slopes where drainage direction was easy to read. At locations where the slope angle was low, or general drainage tended to follow the right-of-way, water directed off the right-of-way by berms often flowed right back further down-slope.

Ditch line subsidence during the first thaw season after construction often resulted in a collapse of the berms. The result of this collapse of the berms was that all water collected by the berms above the ditch line was channeled over the collapsed portion of the berm and then directly down the ditch line.

In those locations where thaw-unstable slopes were encountered, berms were placed under an insulating layer of wood chips. During the first thaw season, some water percolated through the berms and under the insulating cover. Over time, the flow of water under the insulating surface might have caused an undesirable amount of thaw. To overcome this problem a "cut-off" berm design was developed to prevent all down slope water movement. A ditch plug was installed immediately upslope of the insulated slope and a berm was keyed to the breaker and into the mineral soil. The upslope faces of the berm and the ditch breaker were sealed with bentonite. The "cut-off" berm design has very low permeability and appears to have worked well.

(c) drainage ditches

Drainage ditches were installed where mineral soils were cut to establish grade and potential for interception of drainage was high. The drainage ditches were designed to ensure that the intercepted flow is collected, taken down the right-of-way in a controlled, non-erosive manner and then directed off the right-of-way.

(d) revegetation

The establishment of stable vegetation along the right-of-way can provide long term control of drainage by reducing surface velocities. Wherever mineral soils were encountered, a seed mix capable of occupying a variety of habitats was broadcast at 30 kg/ha immediately after construction. A nitrogen-phosphorous-potassium fertilizer blend was applied at 250 kg/ha wherever seeding was done. Seed was applied at 50 kg/ha on all slopes.

(e) watercourse designs

Drainage control was intensively applied at all watercourses. All designs discussed above were utilized. Furthermore, in the watercourse, the ditch line was backfilled with select material to the original grade of the stream bed. Rock armouring was placed on the ditch line at the stream banks.

In the first thaw season after construction there were instances of ditch line subsidence in the watercourse and on the banks.

(f) stockpiling of materials

During the first year of mainline construction, large stockpiles of sand bags were placed near major slopes for future remedial work. In the second year, the design called for smaller caches of approximately 100 bags at 2 km intervals with provision for two major stock piles in excess of 20,000 bags for later distribution.

Subsurface Drainage

The unconsolidated nature of backfill and the surface of the pipe offer a low friction seam which can facilitate subsurface flow. Uncontrolled, subsurface drainage can wash out the backfill material. The collection and concentration of water in the ditch could also lead to excess pore pressures and slumping, and in the case of ice-rich permafrost soils, rapid thaw degradation.

(a) ditch plugs

To prevent excessive water flow down the ditch line, ditch plugs were designed and installed to reduce velocity and to force subsurface flow up and out of the ditch. Ditch plugs are essentially a wall of low permeability that fills the ditch and surrounds the pipe. Ditch plugs were constructed of sand bags, bentonite and geotextile or polyurethane and bentonite.

(b) insulated slopes

Where thaw unstable soils were encountered on steeper slopes, the design required that these slopes be insulated to retard or prevent thaw. No surface water must be allowed to percolate down slope under the insulating layer. Cut-off berms as described above were used for this purpose.

Remedial Measures

In the post-construction phase, drainage control problems have resulted from: excessive subsidence of ditch, excessive surface flows related to impermeable permafrost soils, unusual precipitation events, or the presence of highly erodible soils. IPL has developed and implemented a remedial measures program designed to either repair or arrest problems as they develop. Remedial measures are implemented on a site basis and the degree of response is controlled by the severity of the problem and the means of access available.

Conclusion

Considerable experience has been gained in the field of drainage control in northern environments since the first construction season. The evolution of designs based on theory to those designs proven effective and practicable in various field situations reflects an on-going process of improvement in northern environmental management.

On the Norman Wells pipeline, drainage control problems will diminish over time as disturbed soils consolidate and the vegetation along the right-of-way becomes better established. Nevertheless, IPL will continue monitoring the right-of-way throughout the operational life of the pipeline and is committed to ensuring rapid implementation of remedial measures.

DESIGN OF NORMAN WELLS PIPELINE FOR FROST HEAVE AND THAW SETTLEMENT

J.F. Nixon and A.R. Pick*

Abstract

Interprovincial Pipe Line (NW) Ltd. has constructed a 323 mm (12.7 inch) diameter buried oil pipeline from Norman Wells, Northwest Territories to Zama Lake, Alberta. The 868 km long pipeline crosses areas of discontinuous and intermittent permafrost resulting in difficult design problems particular to Arctic engineering.

Novel concepts were developed and implemented for the design of the first fully buried oil pipeline in permafrost terrain. The basic design concepts include selection of the pipe diameter to limit energy input to the environment, and to provide for increased structural strength of the pipe to assure its integrity under conditions of loadings and displacement caused by thaw settlement and frost heave.

Loadings acting on the pipe were identified and classified by their origin (pressure, temperature differential, thaw settlement, frost heave) and their type (primary; non-relieved by displacement and secondary; relieved by displacement). Both analyses and field observations were made to enhance the understanding of the loadings acting on the pipe as a result of thaw settlement or frost heave. Relevant models for analytical treatment of these phenomena were developed.

Design criteria for the pipeline were established. Thermal analysis and borehole data were used to define values of thaw settlement and frost heave. Acceptable levels of local pipe deformation caused by a concentrated load (e.g. pipe pressing against a boulder were established.

Analytical approaches supported by field data and laboratory experiments were used to define load displacement relationships for soil interacting with a buried pipe. Both gravity and shear loads were evaluated and defined for different thaw settlement and frost heave values. Maximum forces exerted on a buried pipe by a boulder have been evaluated and defined.

A three dimensional finite element inelastic computer model has been used to carry out the calculations for defining the wall thickness of the pipe required to assure conformance to the design criteria for the most critical loading combinations. Cases studied included thaw settlement, frost heave and bend analyses with the inclusion of seismic induced loadings. Significant results of the analyses are discussed in the paper.

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The analyses confirmed the validity of the design concepts selected for the project. The resulting design proved to be economically viable. A monitoring program has been suggested which will allow for identification of thaw settlement, as the magnitude of the settlement approaches the design values.

Introduction

The Interprovincial Pipe Line (NW) oil pipeline from Norman Wells, N.W.T. to Zama, Alberta differs from the previous projects in its design concepts, size and completion phase. The project consists of 868 km of 323 mm pipeline and has three pump stations and a design capacity of approximately 4800 m^3/day (30,000 bpd) of crude oil. Construction activities on the project were commenced during the winter of 1982-1983, and finished in the spring, 1985. The location of the pipeline route within the Northwest Territories and Northern Alberta is given on Figure 1. Crude oil cooling will only occur at Station 1 at Norman Wells, where the input temperature of the oil is approximately -2°C.

The pipeline was constructed during the winter season, and in general no permanent work pad was employed. The minimum depth of cover was 0.76 m, with an additional construction tolerance.

Design approaches and supporting analytical calculations that formed the basis of the design of this pipeline are discussed in this paper.

Design Concepts

Design concepts developed for the IPL (NW) oil pipeline differ significantly from design concepts used for other Arctic pipelines. These differences can be summarized as follows:

- location of a buried pipeline in permafrost will result in some degradation of permafrost and will cause differential settlement of the terrain. The magnitude of the differential settlement can be controlled and limited to an acceptable level by designing the pipeline in such a way that it will have a low energy input to the environment.

- the pipe is treated as a structural member that is designed to withstand deformation caused by differential settlement resulting from construction and operation.

- to the extent possible, the pipeline is located on previously disturbed and cleared rights-of-way (seismic cut lines, and telephone line).

Loading Mechanisms Acting on a Buried Pipe

For a general case of a buried pipeline, it is convenient to group loadings acting on the pipe in two broad categories:

- primary loadings
- secondary loadings.

By definition, primary loadings are loadings which are not relieved by deformation and/or which are not displacement limited. Conversely, secondary loadings are those which are relieved by deformation or which are caused by forces associated with limited displacement. For the purpose of illustration, internal pressure may be considered as an example of primary loadings, while temperature differential might be considered to cause secondary loadings.

From the above definition, it can be seen that a buried pipeline will be subjected to similar primary loadings independent of its location (in unfrozen terrain or degrading permafrost). A buried pipeline in degrading permafrost and subjected to differential settlement and/or frost heave will experience secondary loadings significantly different from those acting on a pipeline located in stable terrain.

The most important secondary loadings affecting the design of a pipeline located in permafrost are the loadings caused by differential settlement, frost heave and seismic activity. These loading mechanisms are discussed in more detail below.

Internal pressure and temperature differential

Operational loadings such as internal pressure and temperature differential are determined by the pipeline system, the ambient temperature during installation of the pipeline in the ditch, and the applicable codes and regulations. These loads include the specified internal pressure, the temperature differential, the pipe weight and other live and dead loads. The maximum allowable operating pressure within the pipeline system is 9929 kPa (1440 p.s.i.). Therefore for a selected nominal outside diameter and a specific minimum yield stress, the minimum nominal wall thickness may be determined. The hydrostatic test pressure required to prove strength is specified by the applicable regulations. For general cross country buried service, the minimum hydrostatic test pressure is 12,410 kPa (1800 p.s.i.).

The temperature differential is the maximum difference between the extremes of the operating temperature of the flowing oil, and the so-called reference temperature. The reference temperature is defined as the thermally stress-free temperature of the pipeline when laid in the ditch and backfilled. The reference temperature will therefore be at or near the ambient air and ground temperature at the time of installation and tie-in. The temperature differential utilized in this project was associated with the maximum difference between the operational temperature. The actual temperature differential used for many of the design studies was 36°C (65°F). This corresponds to a reference temperature of approximately -30°C and the maximum operating temperature of +6°C. The reference temperature arises from the planned winter construction season.

The pipeline operating pressure would of course decrease from one station to the next, and certain sensitivity analysis were carried out to ensure that the high pressure case (9929 kPa), was indeed the more critical case for thaw settlement or frost heave analysis.

Thaw settlement

As mentioned previously, the low energy input from the pipeline into the permafrost means that the pipeline will not directly cause significant thawing of the underlying permafrost. Construction disturbance and clearing activities will cause the permafrost to thaw out slowly with time in many locations, because of the changed surface thermal conditions. If settlement were to develop uniformly, little or no effects would be felt by the pipeline resulting from this thawing. However, at changes in terrain conditions such as from bedrock to soil, from initially thawed to frozen ground, or at sudden changes in subsurface ice content, differential thaw settlement may occur over short distances. Because of the possibility of completely thawed stable soil existing close to a permafrost soil deposit that could settle to the maximum amount, the differential settlement across the transition was conservatively assumed equal to the total settlement that could occur within a terrain unit. This loading mechanism is illustrated on Figure 2. An "infinite" span length was generally considered, as this usually provided the worst case for the pipe conditions considered.

Based on a very extensive borehole data base, thaw settlement analyses were carried out between Norman Wells and the Zama Lake terminal (kmp 868). Computer programs were used to assess the thaw strain of different soil layers, and integrate these to obtain the thaw settlement occurring between the pipe base and the maximum anticipated depth of thaw. As the pipe base was to be located somewhere between 1.06 m and 1.3 m, and the maximum anticipated thaw depth in a 25 year period based on long term field observations was approximately 6 m below ground surface, the depth of soil which would thaw was defined. The different terrain units encountered along the route included lacustrine soils with or without an organic cover, and glacial till deposits that could exhibit a variable depth of organic cover. In one area of the route to the south of Fort Simpson, thick peat deposits were also present, and these required a special treatment in the design process.

Several natural thaw settlement test sites were located along the route to observe thaw settlement based on changes in surface relief. These test sections were established where a cutline or right-of-way was known to have caused thawing of the permafrost, and the differential elevation in ground surface could be observed across the edge of the cutline between disturbed and undisturbed ground. In addition, several previous studies, including sites in the Fort Simpson area visited by McRoberts et. al. (1978), were examined to expand the data base for the pipeline route in this area.

Based on these thaw settlement studies, a design differential thaw settlement of up to 0.8 m in mineral subsoil deposits was established, depending on the location along the route. In general, thaw settlement was anticipated to decrease from north to south along the route. This is in response to a general decrease in ice content coupled with the general warming trend in mean ground temperatures. In addition, in the thick organic soil deposits between the Mackenzie River and Zama Lake a design differential thaw settlement of 1.2 m was adopted.

The loading mechanism at a thaw settlement transition involves downward loading by the soil within the thaw settling zone, and restraint to pipe movement within the thaw stable zone. In the thaw settling zone, the block of soil over the pipe, causes downward loading arising from two sources, namely (a) the effective weight of the soil block above the pipe; and (b) side shear along the sides of the block due to differential movement between the pipe and the surrounding settling soil. The side shear term was estimated based on the effective strength properties of the backfill soil. Within the thaw stable zone, a relatively conservative value was adopted for upward soil resistance on the thaw stable side of the transition. This could result from an unfrozen, competent till deposition which would provide support for the pipe as it crossed the transition. In addition, downward soil loadings would be exerted on the pipe on the thaw stable side of the transition, due to the soil above the pipe at this location. The downward loading in the thaw settling zone was anticipated to increase with increasing soil density, lower water table, and smaller thicknesses of organic soil cover. Reasonable combinations of soil density, thickness of organic cover and position of water table were used to arrive at representative design downward overburden loadings in the thaw settling zone. Conventional bearing capacity theory was employed to estimate the upward soil resistance in the stable zone.

These downward loadings due to thaw settlement subsequent to construction are unique and novel in pipeline engineering, and are not normally encountered in conventional pipelining practice.

Frost heave and pressure dependency

It is not intended to operate the oil pipeline at temperatures significantly below 0°C. However, a possibility exists that the pipe may induce small amounts of frost advance and heave beneath it. If the pipe traverses several kilometres of stable permafrost at temperatures of 1 or 2°C below freezing on average, the contents of the oil pipeline will tend to adapt to the surrounding subzero temperatures. The ground temperature in a permafrost zone could fall as low as -8 to -10°C in the middle of winter. Should the pipe pass from terrain underlain primarily by permafrost to unfrozen ground, it will tend to form a frost bulb of limited extent around the pipe in the unfrozen soil zone. Studies were carried out to show in general that the frost bulb would form a part of a larger layer of seasonal frost, and that the overall stiffness of the seasonal frost - permafrost transition would be sufficient to prevent excessive curvatures in the ground at the pipe location. This analysis involved studying the curvature of the ground at the transition between the permafrost and frost bulb, and is similar to that described by Nixon et al, (1983).

As stream crossings, however, and for some overland pipe locations, the pipe will be buried deeper in unfrozen soil at sag bends. The frost bulb around the pipe may not form part of the seasonal frost zone in these cases, and separate pipeline stress analyses were required to incorporate these effects into the pipe design. Sag bends were identified as being particularly acute areas for frost heave analysis, as upward pressure at the apex of a sag bend would increase the compressive strains in the pipe. These additional strains increase the compressive strains already present in the pipe due to temperature differential and internal pressure. Consequently, the frost heave potential of the soil beneath the pipeline was evaluated, and input parameters were provided to the stress analysis to indicate the additional strains imposed on the pipe by frost heaving across a frozen-unfrozen transition.

Seismic effects

A buried pipeline is potentially subject to a range of loading conditions from several seismic hazards. The strong ground motions induced by a seismic event are characterized by a series of ground waves which can impose strains on buried pipelines. The additional straining imposed by seismic motions was considered in the design of the Norman Wells pipeline. No known active faults were identified in the Mackenzie Valley, therefore ground rupture due to faulting was not considered as a design issue for this route. Seismic accelerations were also introduced into the stability analysis for sloping terrain using Newmark's (1965) method for incorporating seismic acceleration into slope stability analysis. A review of available information indicated that the soils encountered along this route were not susceptible to liquefaction. The seismic design criteria for the Norman Wells pipeline were based on those laid out by Newmark (1974) and Hall and Newmark (1977). The concept of the design probable earthquake (DPE) and a design maximum earthquake (DME) was considered, as these encompass two levels of earthquake hazard. The lower level of hazard, the DPE, is associated with a return period for the design event of approximately 50 years and the higher event has a longer return period of 100 to 200 years.

In general, the impact of seismic aspects on the pipeline design was very minor, due to the relatively low level of historical seismic activity in the area.

Based on the recommendations of Newmark (1974), and subsequent updating by Hardy Associates (1978) Ltd., ground accelerations for the DME of 12 and 3% respectively were established for the two zones identified along the route, with associated velocities of 15 and 4 cm/sec. Using Newmark's procedure for obtaining pipe strains based on these values, strains of 2.5×10^{-4} and 0.66×10^{-4} were established for unfrozen soils, and 1.25×10^{-4} and 0.33×10^{-4} were established for permafrost soil conditions within the two zones identified. These strains were reduced by 15% for the design probable earthquake event. Strains induced by seismic effects were considered additional to those arising from the other loading mechanisms outlined above.

DESIGN CRITERIA

Structural and geotechnical design criteria have been developed to place limitations on allowable stresses and strains in the pipe and acceptable amounts of differential settlement and frost heave.

Stress and strain criteria

Combined circumferential and longitudinal stresses computed on an elastic basis were limited to prevent excessive ductile yielding within the pipe. The maximum longitudinal compressive strains were limited to prevent local buckling of the pipe wall. The maximum longitudinal tensile strains were limited to prevent excessive tensile yielding in the pipe wall. These strains were computed using an inelastic analysis.

The stress limits used for the project are those dictated by the Oil Pipeline Regulations issued by the National Energy Board of Canada and CSA Standard Z183-M1982.

The maximum longitudinal tensile strain was limited to 0.5%. The maximum longitudinal compressive strain for a pressurized pipe was limited to -0.75%. For the design condition, 0.667 to 0.889 of the allowable strains were used for static loads, and for static plus seismic loads respectively. Local deformation (out of roundness) was limited to 5% of the outside diameter for construction loadings and 15% of the outside diameter for operational loadings.

Thaw settlement

As mentioned above, detailed thaw settlement calculations and field observations were carried out to establish the likely total and differential thaw settlement along the pipeline route. These values were established to be 0.8 m within the first 78 km south of Norman Wells, 0.75 m in the region lying between 78 and 440 km south of Norman Wells, and 0.7 m for permafrost areas along the remainder of the route. In areas of thick peat, the design thaw settlement was taken to be 1.2 m. These are reasonable maxima for much of the terrain covered, but may not include occasional extreme amounts of thaw settlement at isolated locations. As outlined later, these will be identified by monitoring. Soil stress analyses accounting for soil arching and continuity across a thaw settlement transition indicted that the design settlement could occur over a distance in the range of 5 m or less. In view of the general lack of information on the development of differential settlement with distance, a transition length of 1.5 m was selected as a conservative distance over which the design settlement would develop.

Frost heave

The ground temperatures within a stable permafrost deposit were analyzed to obtain the anticipated oil temperature for frost heave analysis. The flowing oil temperature was assumed to be equalized with the ground temperature at burial depth in stable permafrost. When the pipe passed from frozen to unfrozen ground, it was assumed that these temperatures were imposed directly on the outer surface of the pipe causing the growth of a frost bulb beneath the pipe. A two-dimensional geothermal simulation was then carried out to determine the growth of the frost bulb around the pipe using the Hardy Associates (HAL) geothermal simulator, as described by Nixon (1983). Typical results for this analysis are shown on Figure 3.

Knowing the depth of frost advance beneath the pipe, and the thermal gradient at the frost line, an analysis of the amount of frost heave was carried out. This was completed using the frost heave theory of Konrad and Morgenstern (1982) and the integration method described by Nixon (1982). The frost heave properties of Calgary silt were adopted to provide a reasonably conservative estimate of frost heaving in the fine-grained soils along the route. This material is known to be highly frost susceptible, displays a high segregation potential, and has been subject to intensive study by many researchers for several years. In addition, further calculations were carried out to indicate the pressure dependency of frost heaving. This was required later when carrying out stress analysis on the pipeline, to provide the relationship between pipe heave and increased pressure exerted by the pipe.

The predicted frost heave experienced by the pipe at normal overburden pressures varied between 125 mm (4.5 inches) to 97 mm (3.8 inches) depending on the location along the pipeline route. This maximum frost heave was usually experienced during the second or third year of operation, as the pipe usually experienced a lesser amount of frost heave in the first year. In later years, the equilibrium frost depth achieved in the second or third was not exceeded. Further analyses were carried out to indicate the beneficial effects of 5 cm of urethane insulation wrapped around the pipe. This indicated that in the more northerly regions, the frost heave beneath an insulated pipe could be reduced to 79 mm (3.1 inches) and further south could be reduced to 63 mm (2.5 inches).

The shape of the frost heave transition, or alternatively the rate at which the frost heave across the transition develops is an important parameter in the stress analysis. Clearly, if the frost heave is assumed to develop suddenly at a transition, thereby creating a sudden step increase in the elevation of the pipeline trench base, this will provide for the most conservative or extreme pipe curvatures and strains. This is likely too conservative, as frost heave is liable to develop over some short transition length, even though the soil type change is sudden. For these studies, the frost heave was assumed to develop over a transition 1.5 m in length. This was based on geothermal analysis showing that the frost bulb would join gradually onto the vertical permafrost interface, and would not adjoin it suddenly at right angles.

Geotechnical Input to Stress Analysis

Load displacement relationships

Load displacement relationships were required by the pipeline stress analysis to model the interaction between the pipe and the surrounding soil. The pre-yield deformations could be presented by a spring constant, K, and the relationship between load and deformation was assumed to be

$$P = K \cdot y \qquad \text{for less than } y_f,$$

and $$P = P_{ultimate} \text{ for } y \text{ greater than } y_f.$$

where P = load per unit length of pipe,
y = pipe displacement, and
y_f = pipe displacement at soil yielding.

As the edge of the transition was approached, the ultimate yield point of the soil was assumed to decrease to approximately 50% of its nominal value on flat terrain. This is due to the reduction in bearing support resulting from the loss of horizontal ground support in the longitudinal direction.

Gravity loading in thaw settling zone

As mentioned earlier, the downward (gravity loading) on the pipe in the thaw settling zone was computed based on the effective weight of soil over the pipe coupled with the side shear exerted between the soil prism and the surrounding settling R.O.W. This is shown schematically in Figure 4. This is the most important parameter in the thaw settlement analysis, as it is the basic loading mechanism on the pipe. Strains and curvatures in the structure will ultimately be strongly dependent on this value. For a reasonable combination of position of the water table, thickness of organic cover, and density of the mineral soil, downward gravity loadings for different segments of the pipeline route were calculated and input to the stress analysis. Additional loadings due to the weight of pipe, contents and possible buoyancy weighting requirements were then included.

Uplift resistance for frost heave analysis

The resistance of the frozen soil in the stable permafrost is an important input to the frost heave analysis. This was analysed using two different possible modes of yielding of the pipe in the frozen soil, namely (a) viscous creep of the pipe upwards through the frozen soil, and (b) an analysis assuming no tension in the frozen soil and upward flexure of two frozen soil "cantilevers" adjacent to the pipe. These are illustrated on Figure 5. The former method was similar to the procedure outlined by Nixon (1978) for the design of strip footings on permafrost. The second procedure is a new analysis which sums the two components of the uplift resistance, i.e. the weight of the soil "blocks" over the pipe together with their flexural resistance to upward movement. There exists a minimum, or optimum uplift resistance for a certain width of the horizontal cracks extending out from the pipe.

Both analyses converged on a similar answer, and the uplift resistance was calculated to be in the range of 220 kN/m (15,000 lbs/ft), assumed to be mobilized at a yield displacement of 5 mm (0.2 inch).

This elastic-plastic idealization of a rather more complex viscous behaviour of frozen soil was used to represent the resistance of the frozen soil to upward pipe motion.

Localized loadings on the pipe

Should the pipe trench encounter a boulder immediately beneath the trench base, and thaw settlement occurs subsequently adjacent to the boulder location, a concentration of stresses might occur in the pipe supported on the "point load" provided by the boulder. Stress analysis will show that a concentrated load of a large enough magnitude would cause denting of the pipe. A geotechnical analysis was carried out to indicate the magnitude of the loading which could be exerted by a large cobble or boulder embedded in a soil matrix. Details of this analysis are beyond the scope of this paper, but it is sufficient to say that for boulders up to a size of 0.15 to 0.3 m, the cobble or boulder would tend to punch into the soil and not cause significant increases in stress at the base of the pipe.

Stress Analysis

The assessment of the geotechnical information developed as a basis for the stress analysis confirmed the basic design concept that the parameters of the pipe (grade and wall thickness) will be governed by the required structural response of the pipe to the secondary-type loadings caused by differential settlement and frost heave. The stress analysis was carried out using the three-dimensional finite element computer program "SAVFEM" (Structural Analysis Via Finite Element Method), as described by Workman (1977). For example, the thaw settlement problem has been idealized as shown on Figure 6.

Cases studied

The basic reasons used for selection of cases within the scope of the stress analysis were (a) to enhance the understanding of the impact of the different input parameters on the results of the analysis; (b) to develop a representative model or models to be analyzed in detail; (c) to complete a parametric study of variables, and; (d) to complete a series of design-type analyses. The following problem areas have been analyzed in detail: (i) differential pipe settlement; (ii) frost heave; (iii) localized denting caused by large bounders, and (iv) bends.

The following are some conclusions:

- the depth of cover over the pipeline strongly affects the loading on the pipe subjected to differential settlement. Considering the location of the proposed pipeline in a remote area, the minimum depth of cover was reduced to 0.76 m from the 1.0 m required by the oil pipeline regulations of the National Energy Board of Canada (NEB).

- the magnitude of frost heave predicted for the bare pipe at stream crossings cannot be accommodated by reasonable increases in the wall thickness of the pipe. The amount of the frost heave in such areas has to be controlled through the use of thermal insulation.

- the wall thickness of the pipe required to withstand the anticipated amount of differential settlement for the specified downward loads is shown in Table 1. These wall thicknesses should be compared with the wall thickness of 6.22 mm (0.245 inch), based on the elastic circumferential stress limit ignoring secondary loadings such as frost heave or thaw settlement.

- localized denting caused by the presence of large boulders within the transition zone of settling pipe did not exceed 15% of the diameter of the pipe.

- strains at the pipe bends caused by pressure and temperature differential were below the criteria set for the project.

Monitoring During Operation

Ground temperatures are being monitored at several locations along the route both by Interprovincial Pipe Line (NW) Ltd. and the Department of Energy, Mines and Resources Canada. Pipe temperatures are also measured, together with probing to record the pipe elevation.

The effects of thaw settlement will be monitored by visual observations along the route at several times during the initial years of operation. As the design thaw settlement is in the range of 0.7 to 0.8 m, and the pipeline is designed to safely accommodate settlements of this magnitude, it is apparent that settlements will become clearly visible before a problem develops with excessive pipe strain. Settlements of this order can be readily observed by an engineer walking the line, or flying the route at low altitude. Regular observations will constitute the most important aspect of monitoring to anticipate thaw settlement problem areas. Areas delineated by these visual observations will be subject to further scrutiny, and will be flagged as areas for possible maintenance.

ACKNOWLEDGEMENTS

The authors wish to express their sincere thanks to colleagues at Interprovincial Pipe Line (NW) Ltd., Hardy Associates (1978) Ltd., and Canuck Engineering Ltd. In particular the results of many discussions with Prof. N.R. Morgenstern of the University of Alberta, Dr. J. Stuchly of Canuck Engineering Ltd., Dr. G. Workman of Stresstech Ltd. and Dr. W. Slusarchuk of Hardy Associates are reflected in the results given in this paper. Dr. E. McRoberts of Hardy Associates contributed in the areas of seismic design parameters, and Mr. John Tse of Canuck Engineering and Ms. Susan Nelson of Stresstech Ltd. carried out the pipe stress analyses. In addition, the authors wish to sincerely thank Mr. W. Pearce of Interprovincial Pipe Line (NW) Ltd. for permission to publish the results of this paper.

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TABLE 1

DESIGN WALL THICKNESS FOR SPREADS 1, 2 AND 3
GRADE 359 MPa (52 ksi) (0.76 m of cover)

Design Spread	kmp	Seismic Zone	Case	Design Thaw Settlement mm (in)	Soil Loading Thaw Stable/ Settling Side kN/m (lbs/ft)	Design Wall thick mm (in)	Operating Strain of 0.5%	
							Temp. Diff. °C (°F)	Max. Axial Strain (%) (*)
1	0- 78 kmp	A	A	.80 (32)	7.3/3.7 (500/250)	7.16 (.282)	36.1 (65)	.497
2	78-440 kmp		B	.75 (30)	7.3/3.7 (500/250)	6.91 (.272)	36.1 (65)	.496
3	440-478 kmp		C	.70 (28)	6.4/3.2 (440/220)	6.35 (.250)	36.1 (65)	.499
	478-868 kmp	B	D	.70 (28)	6.4/3.2 (440/220)	6.35 (.250)	36.1 (65)	.499

Note: ( ) Max. Allowable Strain
(*) DME and DPE earthquake not included.

Wall thickness ignoring frost heave or thaw settlement was 6.22 mm (0.245 inch), based on the elastic circumferential stress limit of 72% of the specified minimum yield strength.

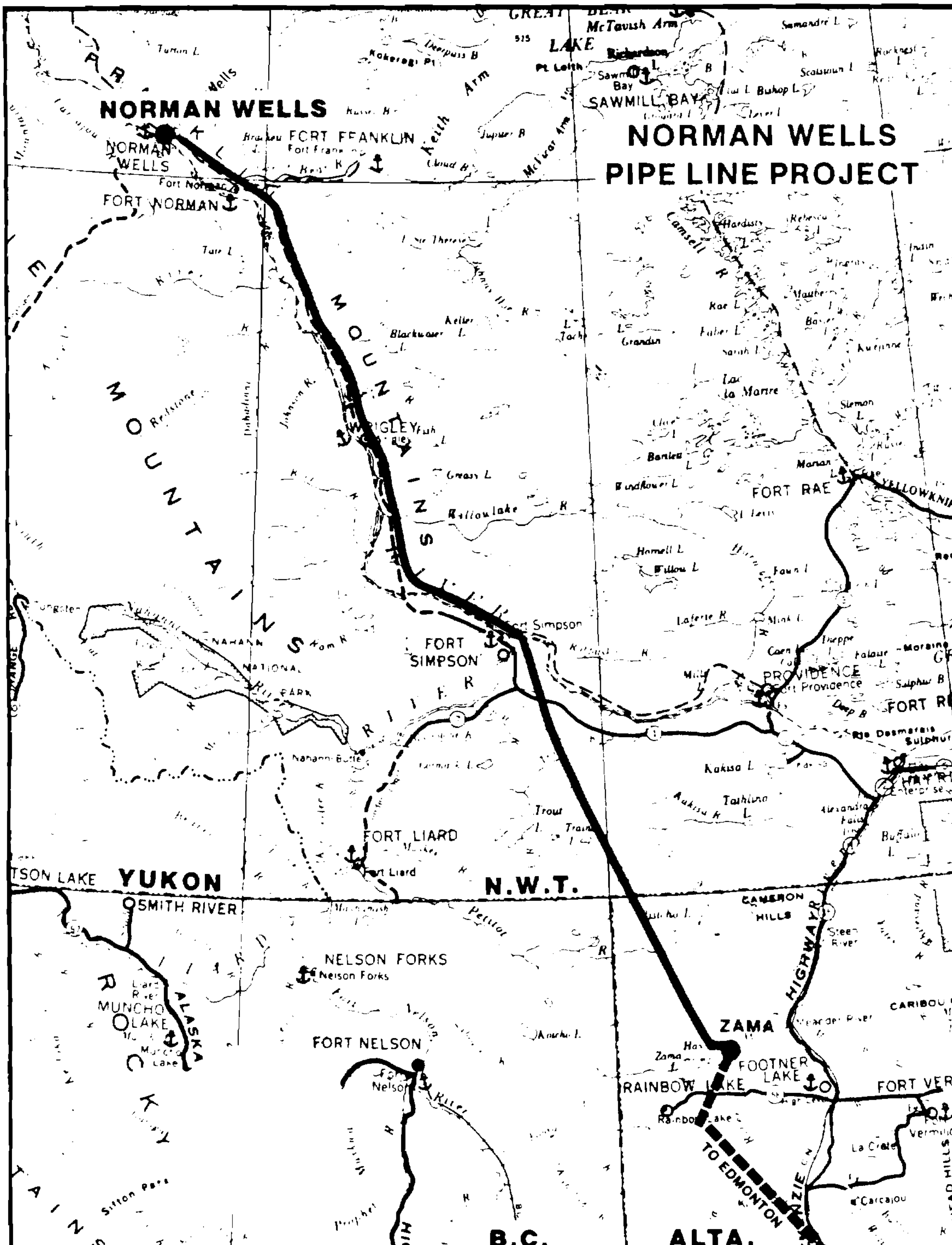


FIGURE 1 LOCATION OF NORMAN WELLS OIL PIPELINE

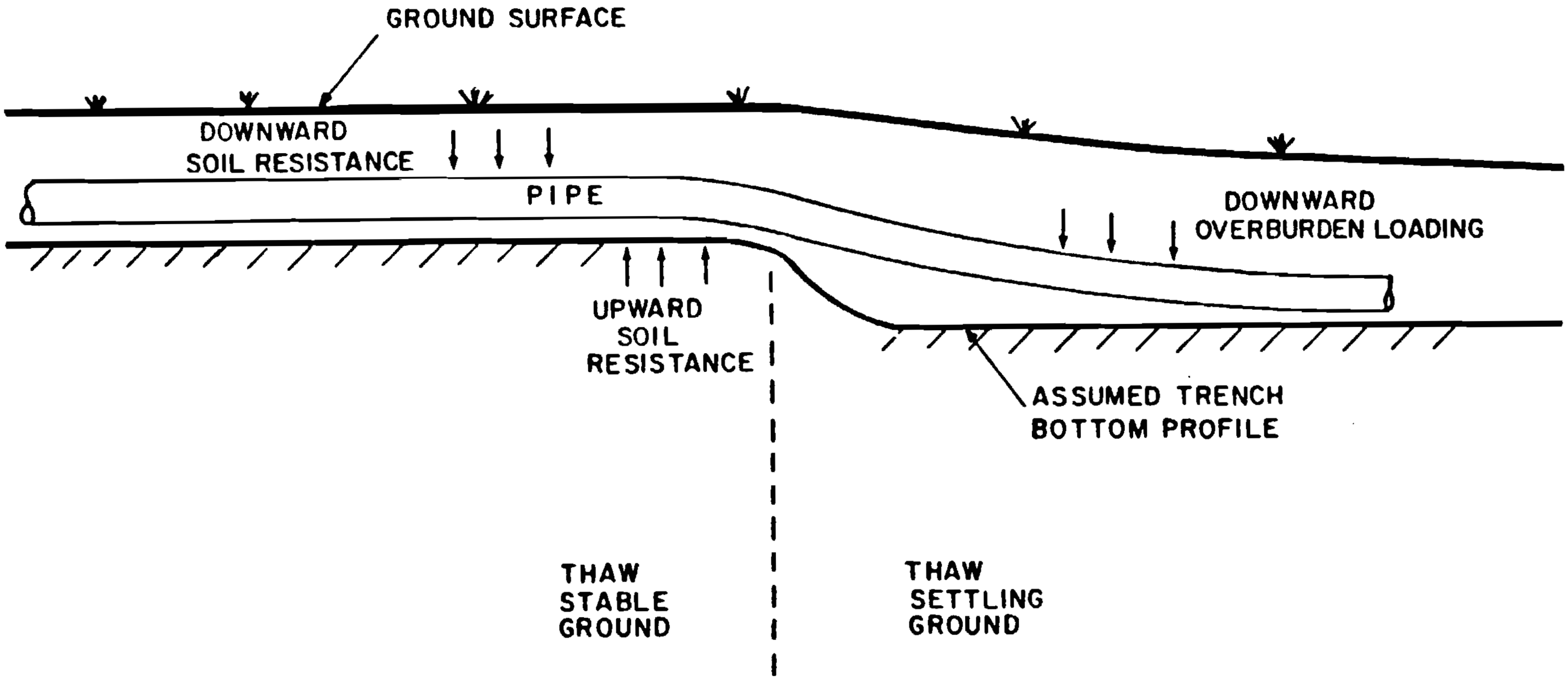


Figure 2. Differential Thaw Settlement at a Transition

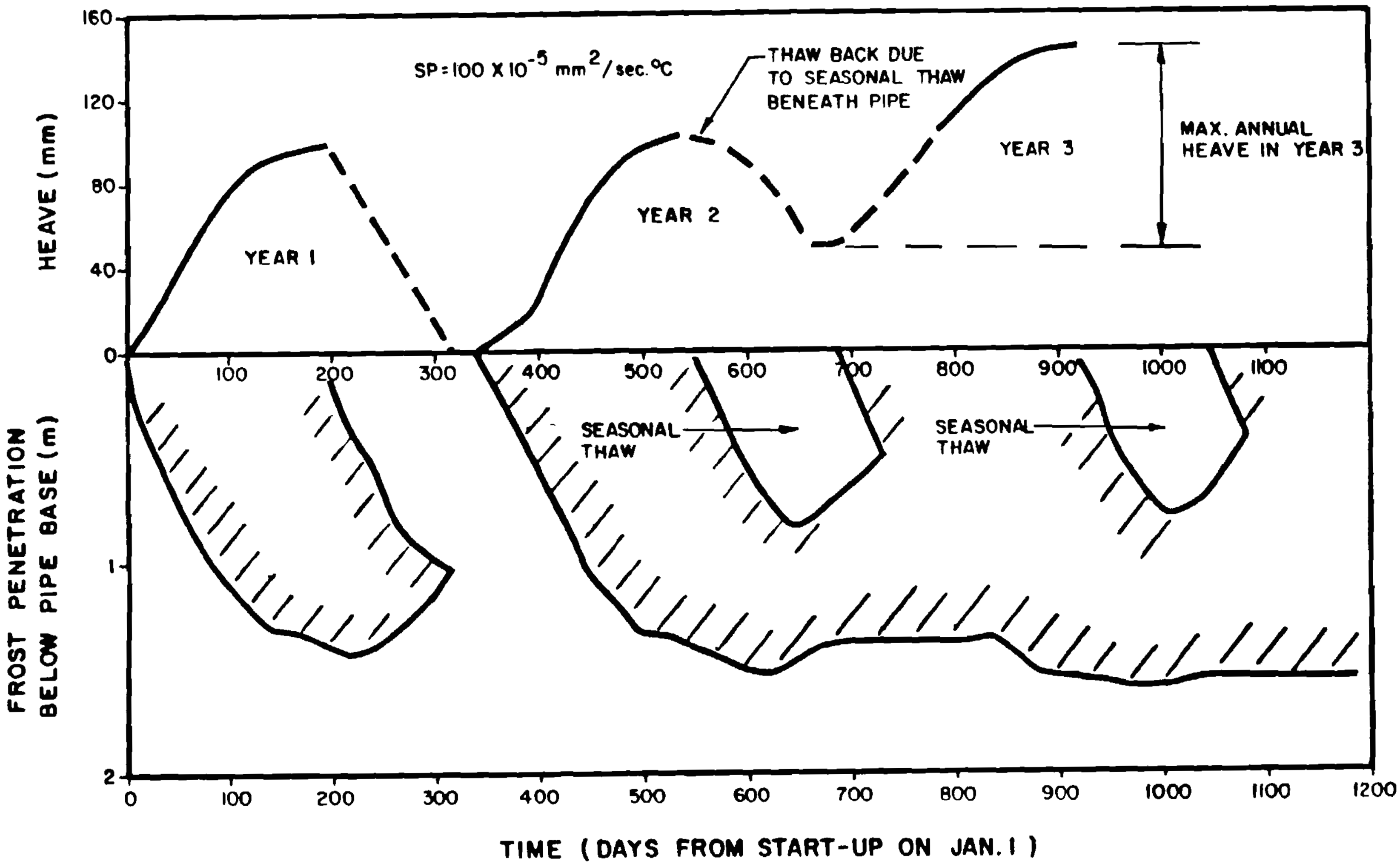


Figure 3. Frost Depth and Heave Prediction

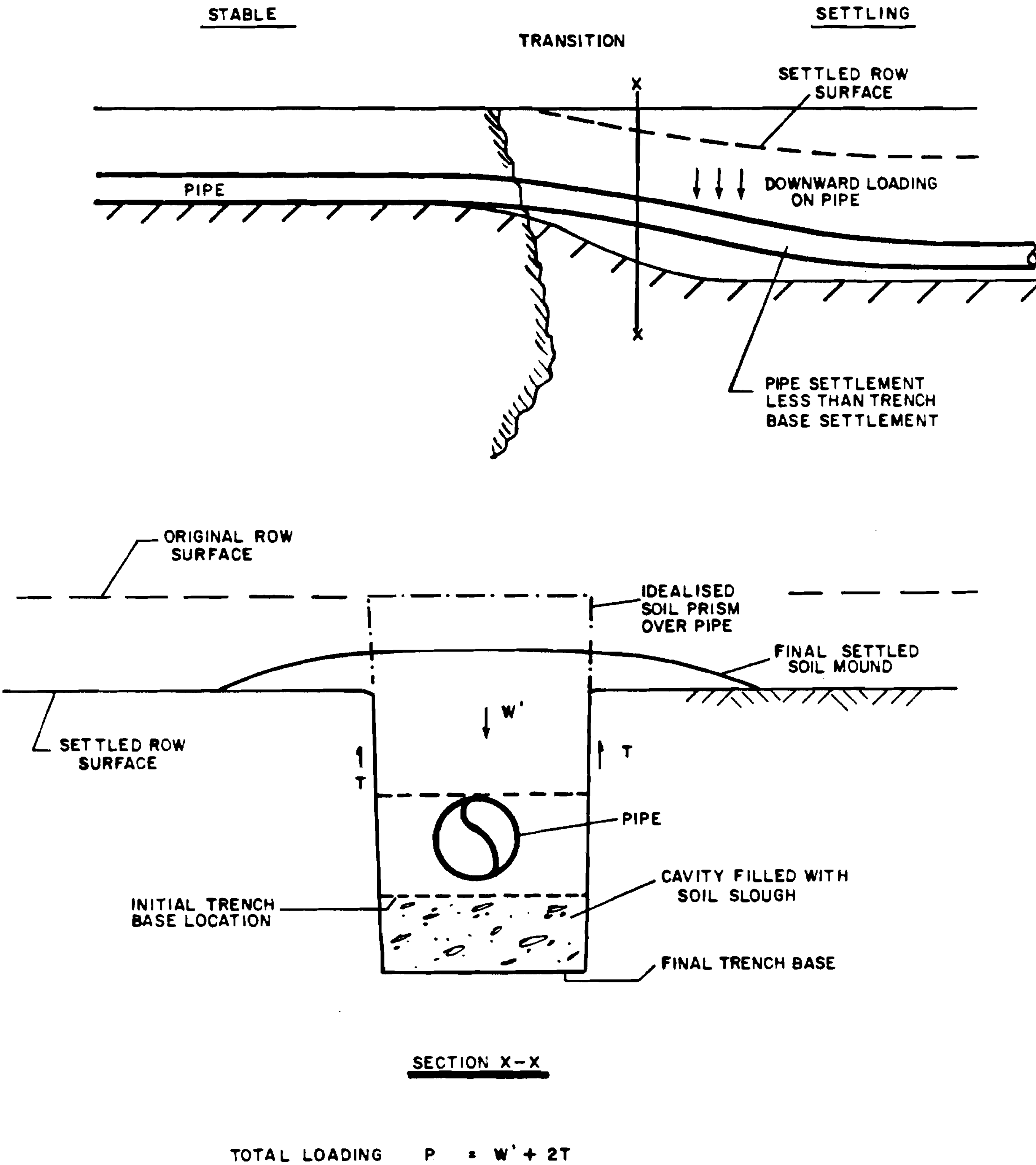
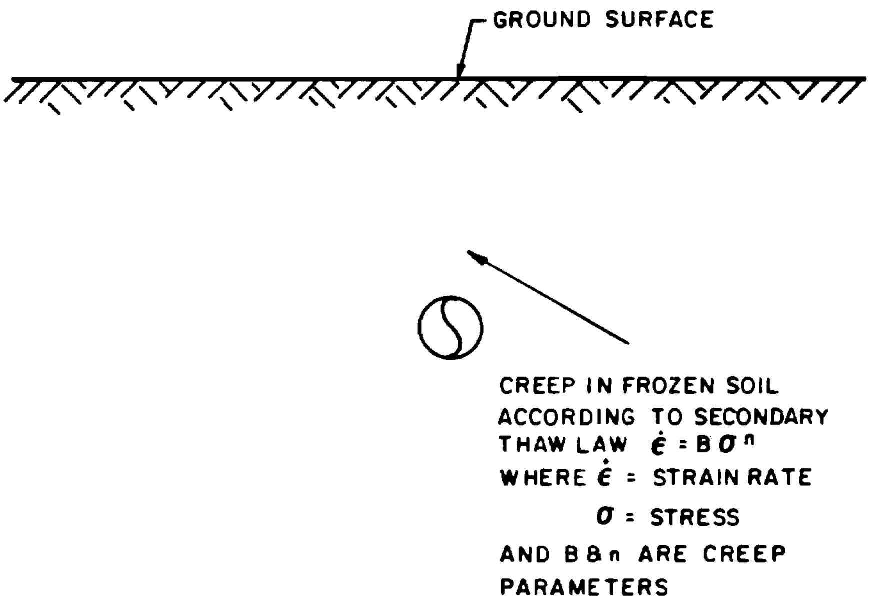
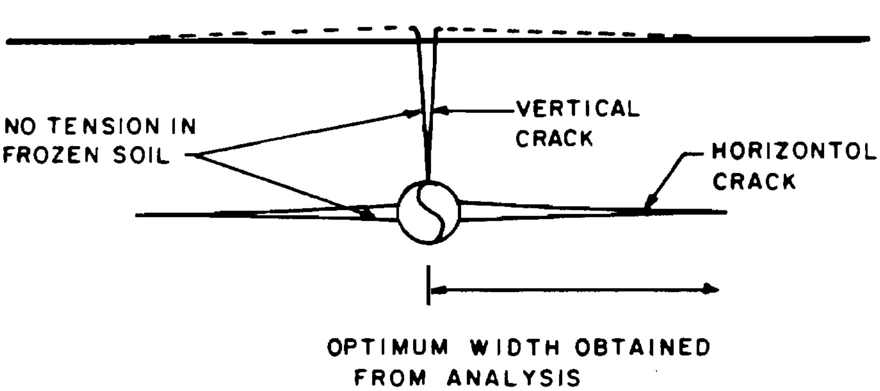


Figure 4. Illustration of Downward (Gravity) Loading Pipe in Thaw Settling Zone



(a) CREEP MECHANISM



(b) NO TENSION / FLEXURAL RESISTANCE MECHANISM

Figure 5. Methods of Computing Uplift Resistance in Frozen Soils

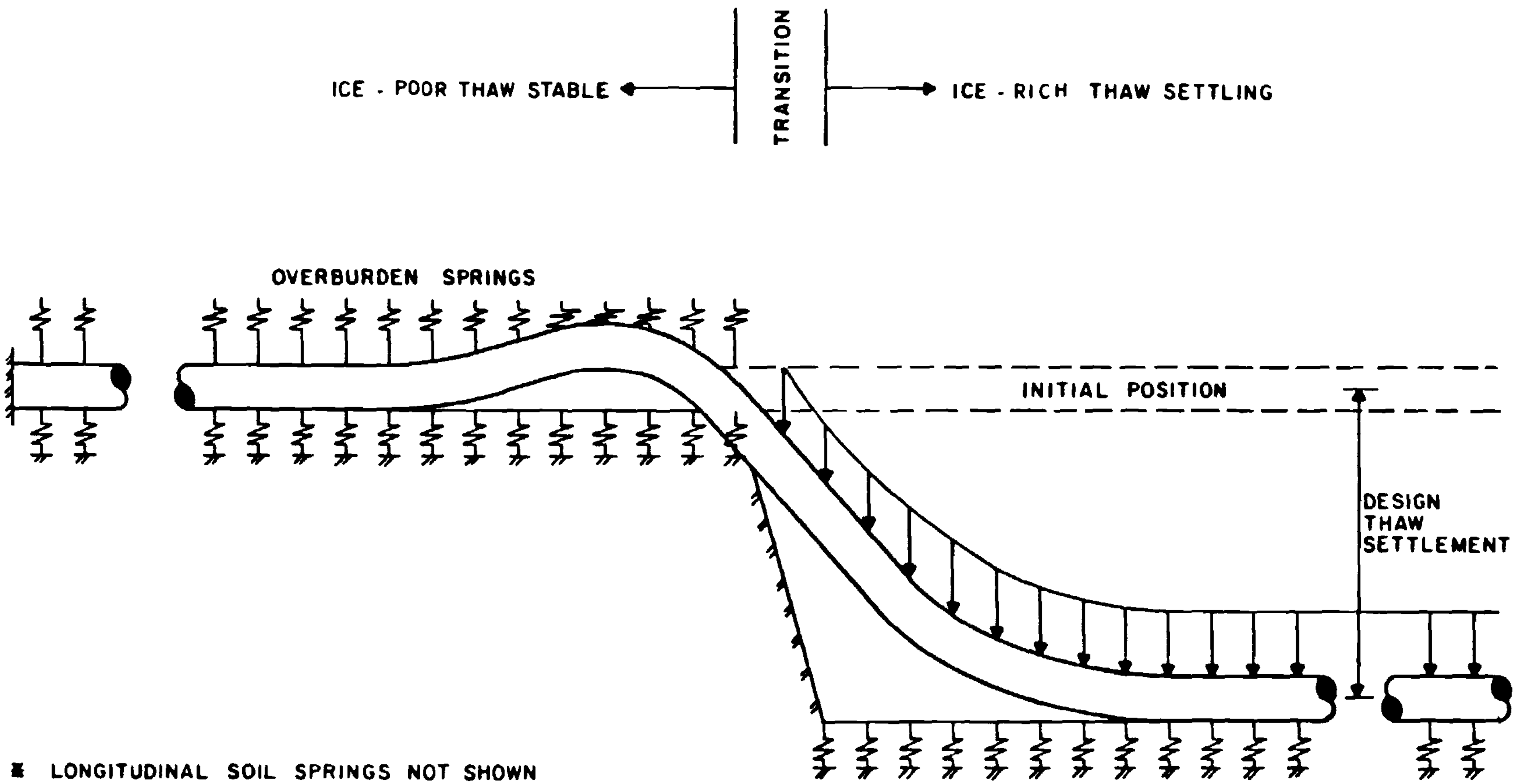


Figure 6. Basic Finite Element Model for Thaw Settlement Calculation

QUILL CREEK TEST FACILITY

D.E. Fielder*

Introduction

The portion of the Alaska Highway Gas Pipeline System (Plate No. 1) that will run through Yukon will encounter ice-rich permafrost. Studies have indicated that the pipeline would be constructed and operated in the most cost efficient manner if the flooring gas temperature is kept below 0°C west of Kluane Lake and above 0°C across and east of Kluane Lake (Plate No. 2). West of Kluane Lake the cold pipeline will encounter non-frozen areas where a frozen annulus will develop around the pipe and where frost heave may therefore occur. Many studies have been undertaken by many people to attempt to determine the amount of heave that may occur and the effect of that heave on the pipelines. Several field test sites have been established by various companies and consortia to determine what will happen with respect to frost heave and to verify predictions. Quill Creek is different. Quill Creek may be considered to be a "Thaw Settlement" test facility, but even that description is not entirely correct as we were testing warm flow designs that we believe would result in zero or acceptable thaw settlement. We were therefore, from a geothermal perspective interested in field testing designs which would be applicable east of Kluane Lake - the warm flow section.

Additionally, some construction procedures were tested which are applicable to both warm and cold flow areas and particularly to the installation of large (1219 mm) pipelines in areas containing ice-rich permafrost.

This presentation describes:

- what we set out to do,
- where we did it,
- how we did it, and
- some of the results of our work.

Objectives of the Quill Creek Test Facility:

1. to test mitigative design concepts for warm pipelines in permafrost areas.

2. to test and develop pipeline construction procedures for permafrost areas.

3. to provide additional verification that the geothermal modelling tools being utilized are satisfactory.

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* See List of Registrants (Appendix A) for affiliation of authors.

In order to meet those objectives the site selected had to include:

1. Thaw stable soils.

2. Thaw unstable soils.

3. Areas requiring buoyancy control.

4. Ice-rich side hill areas (slope stability concerns).

5. Areas requiring drilling and blasting for ditch excavation.

6. Areas that could be trenched using a rotary wheel ditcher.

7. Areas requiring a workpad along the right-of-way to prevent excessive deterioration of the ground surface.

These conditions were all found at the Quill Creek site.

Work began in November, 1980, with the processing of gravel (mining and stockpiling of pitrun and 20 mm crushed) at two sites on the Alaska Highway; one at Quill Creek and the other about 12 km from Quill Creek. Field construction began with survey and clearing in January, 1981, with the winter work (construction tests and installation of buried and embankment sections) completed in April. The concrete restrained sections were installed in September of 1981 when the active layer had reached near maximum depth. Warm flow operation began on October 2, 1981, for the embankments and on December 15, 1981, for the concrete restrained sections. The facility was shut down on June 7, 1985 about 4 years after start-up. Costs were about $9 million to construct and about $675,000 to operate for 4 years.

It is not necessary to describe the instrumentation utilized to collect data from the site during the testing period as it was rather standard and most people here today will be familiar with it. However, the types of data collected fall into seven general categories:

1. weather

2. ground temperature

3. pipe operating temperature

4. heat flux

5. pipe and ground settlement

6. slope movement

7. qualitative:

   a. photography
   b. observations.

I will not be able in the time available today, to discuss in detail everything undertaken at Quill Creek during the last 4 years. In summary, however, we did the following:

1. constructed 125 m of buried insulated 1219 mm pipeline.

2. constructed 200 m of insulated 1219 mm pipeline in an above-ground embankment.

3. constructed 50 m of 1219 mm pipeline in above-ground concrete-restrained mode.

4. operated warm (15°C) 1219 mm pipeline in six different configurations for about 3½ years.

5. tested 6 different methods of protecting cut ice-rich slopes.

6. constructed and tested a variety of access road and workpad configurations (including an ice road).

7. made ditch in permafrost utilizing a rotary ditcher, a backhoe with special teeth, and by drilling and blasting.

8. tested six different backfill configurations to prevent buoyancy.

9. tested three different pipeline insulation configurations and tested insulation strength by filling an installed insulated 1219 mm pipeline with water to simulate hydrostatic test conditions.

10. cathodically protected a section of pipeline installed in permafrost to determine:

    - ground bed performance in permafrost.
    - effect of thaw bulb formation on protection current distribution, and
    - characteristics of telluric currents.

Plates No. 3 and 4 show the layout of the test facility.

<u>Construction</u>

The following lists the pipeline construction practices tested at Quill Creek:

1. clearing

2. access road construction

3. grading
4. ditching
5. insulation of pipe
6. installation of pipe in above and below ground configurations
7. buoyancy control measures
8. slope stability measures
9. lower-in and backfill
10. clean up and restoration.

Clearing

Plate No. 5 shows a burning sled being utilized to keep heat and ash as much as practicable away from the ice-rich soil. Slash from clearing is being burned.

Access Roads

Construction of a variety of types of access roads was undertaken for three main reasons:

1. to evaluate trafficability by monitoring the roads through construction and the first thaw period.
2. to determine the mount of remedial work required for the different types of roads.
3. to determine the best type of road based on effectiveness and economics.

Plate No. 6 shows the construction of a road comprising timber rip-rap layed on the ground surface and covered with granular material. An ice-aggregate road was constructed by mining ice from Kluane Lake using an industrial quality roto-tiller (Plate No. 7) with specially designed tines to break the ice. The ice was hauled to Quill Creek and end dumped in place (Plate No. 8) and spread with a dozer (Plate No. 9). The finished ice road in use is illustrated in Plate No. 10.

Ice-Rich Sidehill Grading Tests

These tests had the following objectives:

1. to assess the effectiveness of the various techniques.
2. to monitor the changes in the protected slopes.
3. to assess the stability of cut slopes with no mitigation procedures.

Plate No. 11 shows the side slope being cut to construct a right-of-way. Seven boreholes drilled in this slope show it to be clay and clayed silt overlain with 1-4 m of organics. Visible ice ranges along the slope and with depth from 15% to 70%. Plates 12-16 show cut slopes protected as follows:

Plate 12 - near vertical cut about 2 m high with wire mesh used to hold the peat layer in place.

Plate 13 - 2h:1v cut slope about 8 m high covered with 1.0 m thick peat layer and a jute blanket.

Plate 14 - 2h:1v cut slope about 8 m high covered by a 1.7 m thick gravel blanket.

Plate 15 - ½h:1v cut slope about 4 m high protected by a filter cloth and gravel toe buttress.

Plate 16 - panoramic view of an unprotected vertical cut about 6 m high. The slumping shown is considered to be acceptable as it could easily be dressed up with a dozer. As the rate of slumping decreased significantly after the first year it appears that an unprotected cut, even in ice-rich permafrost, will heal naturally.

Plate 17 shows the whole side slope with (except for the unprotected vertical cut) rather sophisticated and very costly protective measures in place.

Ditching Tests

Objectives:

1. to evaluate various methods of excavation.

2. to assess the suitability of the wheel ditcher to make ditch

3. to assess the effectiveness of backhoe ditching with and without blasting.

4. to compare drilling and blasting techniques.

Insulation Systems

Objectives:

1. to assess alternative insulation systems and application methods.

2. to assess insulation application procedures for field joints (girth weld).

3. to assess the effectiveness of alternative insulation systems in permafrost areas.

4. to determine and assess handling problems associated with insulated pipe.

Three insulation systems were tested:

1. polyurethane foam covered with a 2.5 mm thick outer coating (moisture resistant and mechanical protection) of urethane elastomer.

   a. field applied
   b. shop applied

2. pre-formed quarter segments of 100 mm thick polyurethane foam. These were glued to the pipe and to each other with urethane elastomer. The assembly was then strapped to the pipe using stainless steel bandstraps and the whole assembly then sealed with urethane elastomer.

3. concrete restrained mode - pipe set on pre-formed urethane foam sections coated with urethane elastomer. Concrete blocks internally coated with same insulation and set over the pipe.

Plates No. 19 and 20 show (respectively) pipe sections coated with field and shop applied insulation.

Plates No. 21, 22 and 23 illustrate - segmented insulation, segmented insulation installed on pipe, and insulation segments held in place with steel bandstraps, respectively. Plate No. 24 shows a section of insulated pipe being lowered into a ditch.

<u>Installation Methods - Above-ground and Buried</u>

Objectives:

1. to compare and evaluate installation methods.

2. to monitor alternative methods with respect to the terrain in which they are installed.

3. to assess the feasibility of select backfill for buoyancy control.

We were concerned with the installation of large, insulated pipe. In addition to testing insulation systems we had to be certain that the insulated pipe could be installed without damage to the insulation. 125 m of 1219 mm insulated pipe was installed in a ditch that had been excavated by blasting and hoeing. Installation took place from a granular workpad - partially insulated.

In addition to the insulated buried mode, Foothills developed an above-ground design which we refer to as the "Embankment Mode." The intent is two fold:

1. to attempt to isolate the warm pipe from the underlying frozen soils, and

2. to provide a geometry which better takes advantage of the cold winter weather for freeze-back.

Neither this nor the buried section were operated warm so tend more to represent the pipeline during the dormant period between construction an operation. The following plates show the construction of 200 m of 1219 mm insulated pipe in a granular embankment.

Plate No. 25 - a gravel levelling course is laid on the ground surface, followed by a layer of insulation and more gravel.

Plate No. 26 - placement of insulation.

Plate No. 27 - installation of the pipe in the insulated gravel pad.

Plate No. 28 - the pipe is covered with granular material.

Plate No. 29 - completed embankment.

Warm Flow Tests

In addition to the installation tests just described, we undertook some warm flow tests to provide field data for comparison to geothermal predictions. Plate No. 30 shows the layout of the warm flow test sections:

J1 - Control section - uninsulated pipe - conventional bury. The 4 boreholes drilled show the area to be frozen with ½-1 m of peat and a narrow (0.1 - 0.3 m) layer of silt overlying gravel to at least 5 m. No visible ice is reported.

The rest of the warm flow test sections are located in ice-rich permafrost. The borehole logs show this area to be primarily silts and peat with visible ice ranging from about 10% to as high as 85%.

J2 - Uninsulated pipe in insulated embankment
J3 - Pipe with 200 mm insulation in insulated embankment
J4 - Pipe with 100 mm insulation in insulated embankment
J7/J8 - Concrete restrained mode

Plate numbered 31-35 show the construction of embankment for warm flow testing:

Plate No. 31 - first lift of gravel for pad
Plate No. 32 - placement of insulation in pad
Plate No. 33 - installation of insulated pipe
Plate No. 34 - covering the pipe with granular material
Plate No. 35 - completed embankment

Everything shown to this point was constructed during the winter of 1981 and was therefore constructed on frozen soil.

As an alternative to the "Embankment Mode" which requires significant quantities of granular material, we developed a design we refer to as "Concrete Restrained." After construction of an insulated workpad similar to the one utilized for the "Embankment Mode," the uninsulated pipe is set onto pre-formed insulation segments. Concrete blocks (to provide restraint against movement) internally coated with insulation (to simulate the insulation qualities of the granular embankment) are set over the pipe. Plates numbered 36 and 37 show the Concrete Restrained Mode.

Note in Plate No. 37 the gravel piled against the side of the concrete. This has two purposes:

1. to provide, wherever required, a ramp for wildlife crossings, and

2. to provide buttress-like restraint against horizontal movement at side bends.

The ramp/buttress shown is incomplete.

<u>Some Warm Flow Test Results</u>

<u>Test section J1</u> - conventional buried control section
- thaw stable permafrost

Plate No. 38 shows selected 0°C isotherms during the spring (March) and fall (August) of 1982, 1984, and 1985. As expected, although the thaw bulb has penetrated to about 7 m below ground suface, pipe movement (also illustrated) is insignificant.

<u>Test section J2</u> - insulated embankment
- no insulation on pipe
- thaw unstable permafrost

Plate No. 39 shows selected 0°C isotherms during the spring (March) and fall (August) of 1982, 1984, and 1985. Although the thaw bulb has penetrated only about 1 m, pipe movement is relatively significant; about 1/3 of a metre.

<u>Test section J3</u> - insulated embankment
- 200 mm insulation on pipe
- thaw unstable permafrost

Plate No. 40 shows selected 0°C isotherms during 1982, 1983, 1984, and 1985. Above ground level, the 0°C isotherms retreated into the pipe insulation during the winter months. Pipe movement is considered to be acceptable.

Test section J8 - concrete restrained
- thaw unstable permafrost

Plate No. 41 shows selected 0°C isotherms during 1982, 1983, 1984, and 1985. During the winter months the 0° isotherms retreated into the pipe. The effect of construction during the thaw period (September, 1981) can be seen by the "trapped" thaw section beneath the pad. Although the trapped thaw bulb appears to be increasing, and although the thaw depth during the fall (August and October) is deeper than that shown for Test Section J3, pipe movement is considerably less than that for J3.

Preliminary Conclusions

Although most of the data collected have not yet been analysed, we have seen enough to allow us to conclude that large diameter pipelines can be installed in areas containing ice-rich permafrost without causing unacceptable environmental damage. We do not pretend that this is a new conclusion as other pipelines have, of course, been successfully constructed in such areas. The application of insulation to pipe under both shop and field conditions, and the installation of insulated pipe in both buried and above-ground modes has been shown to be feasible. The rather unique above-ground designs developed by Foothills and tested at Quill Creek have been structurally stable over the test period and are geothermally acceptable. The comparisons made to date between thermal predictions (location of 0°C isotherms) and field data show good correlation.

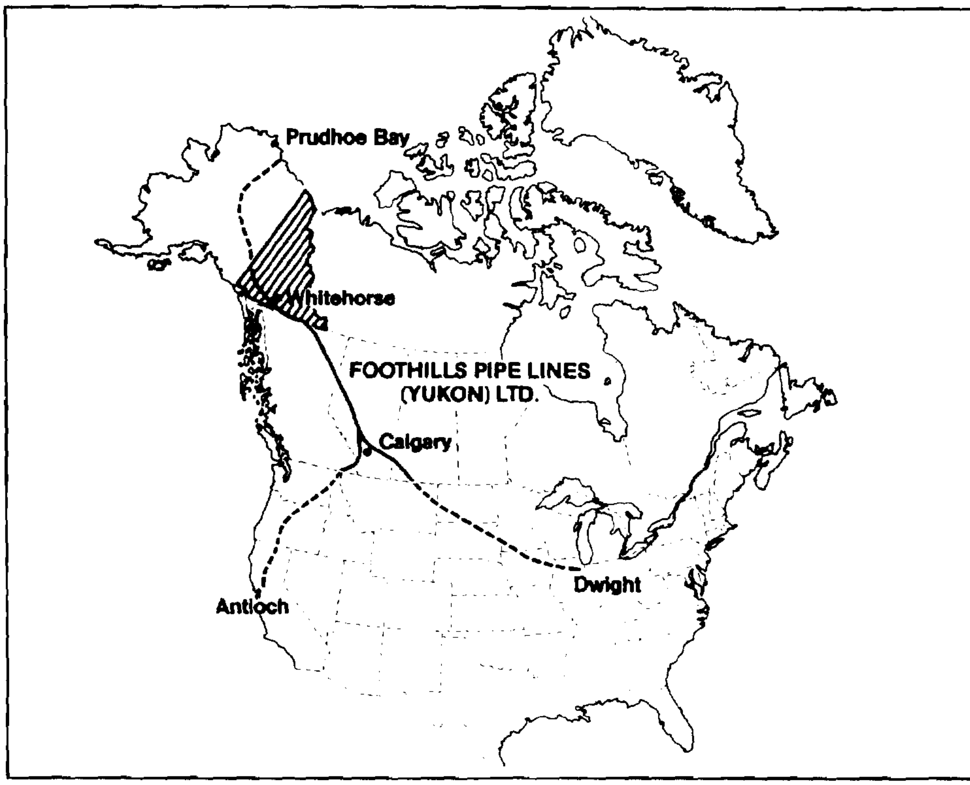

PLATE NO. 1
The Alaska Highway Gas Pipeline Project

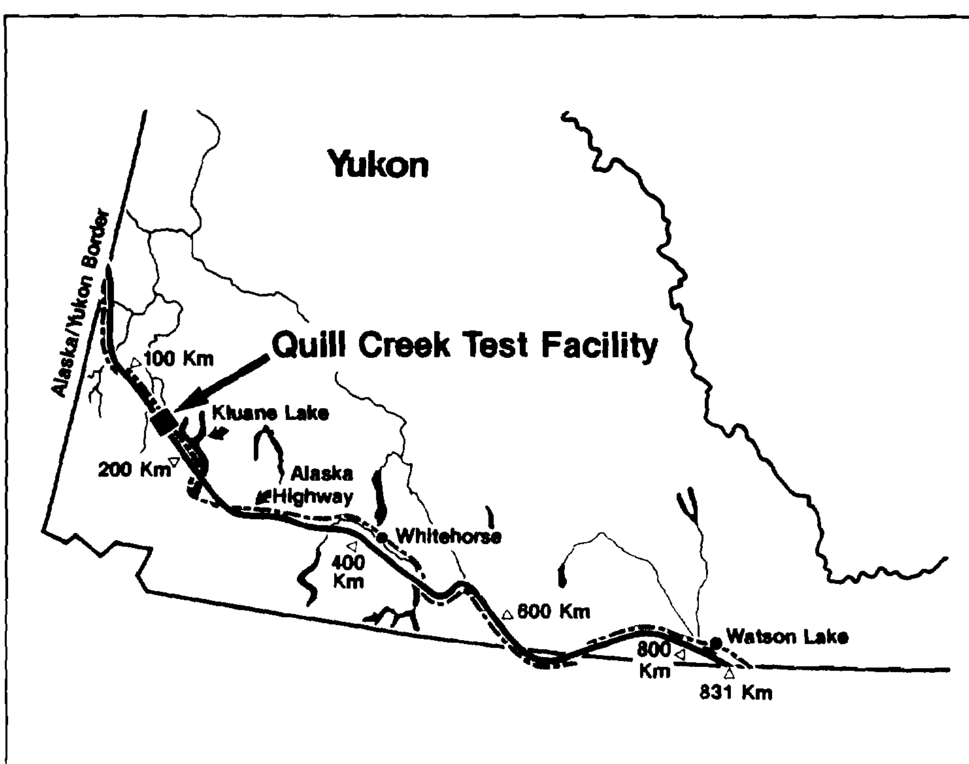

PLATE NO. 2
Location of Quill Creek Test Facility

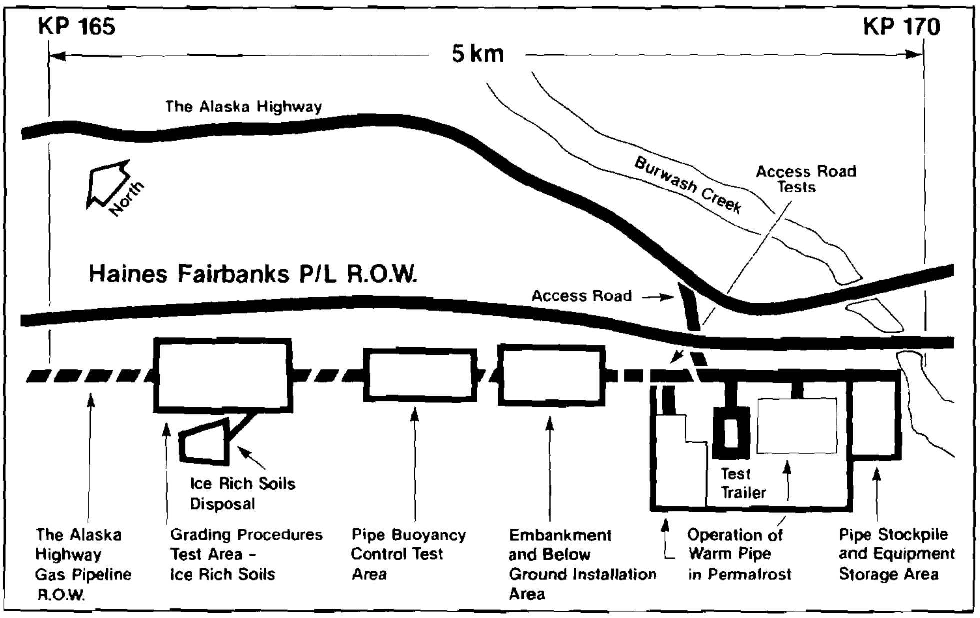

PLATE NO. 3
General Layout - Quill Creek Test Facility

PLATE NO. 4
Aerial View - Quill Creek Test Facility

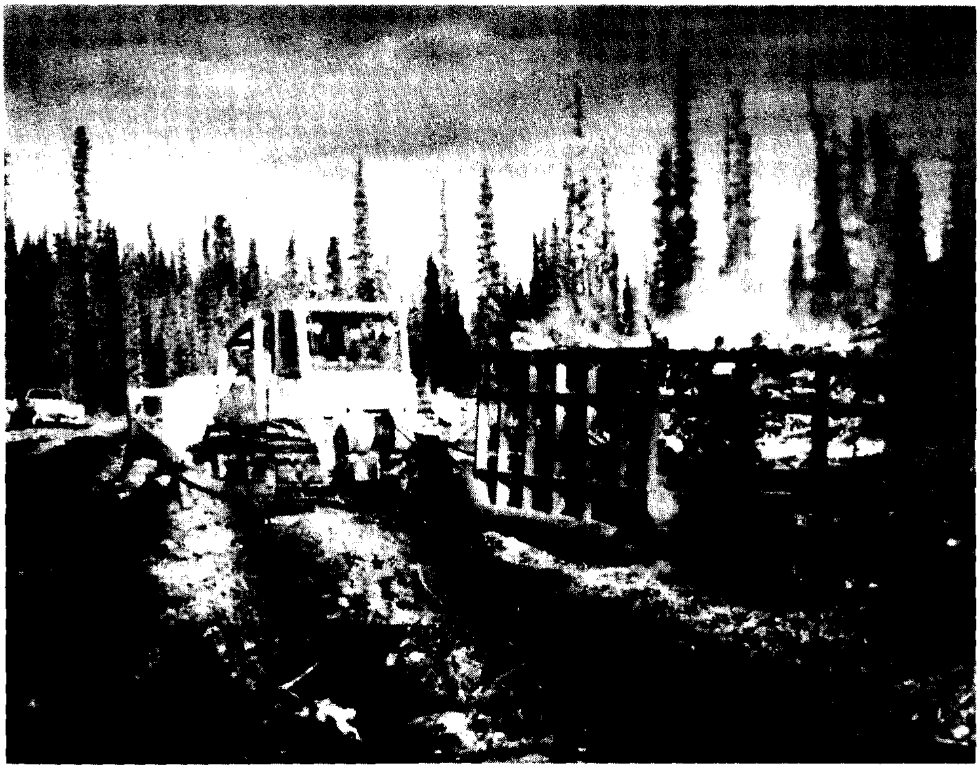

PLATE NO. 5
Burning Sled

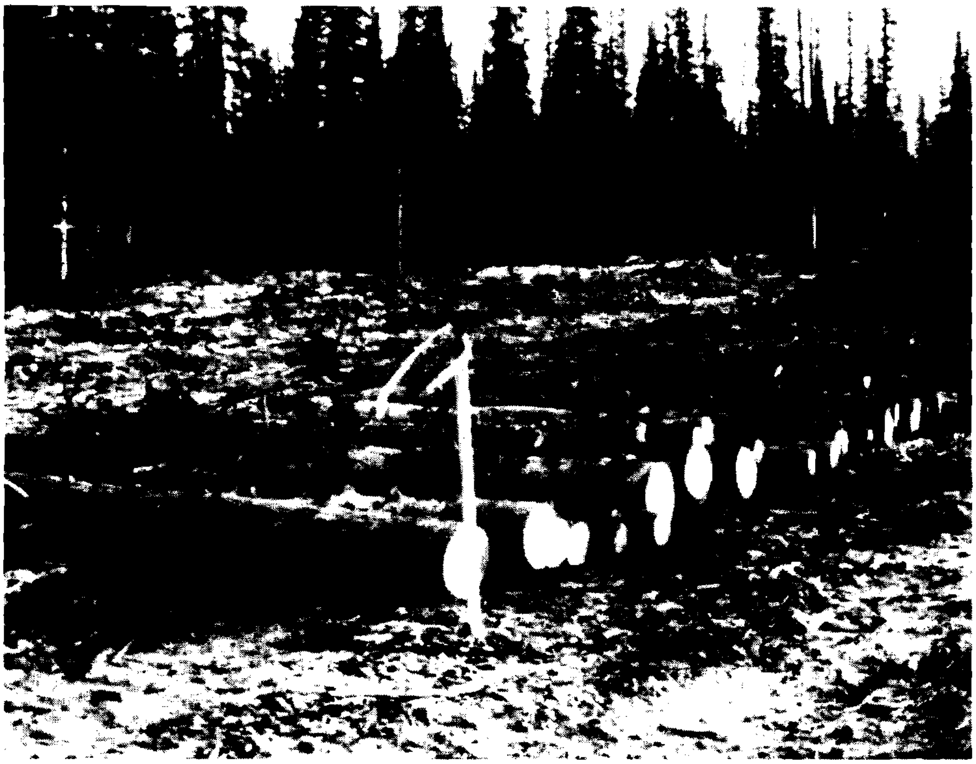

PLATE NO. 6
Access Road Constructed from Timber Rip-Rap
and Granular Material

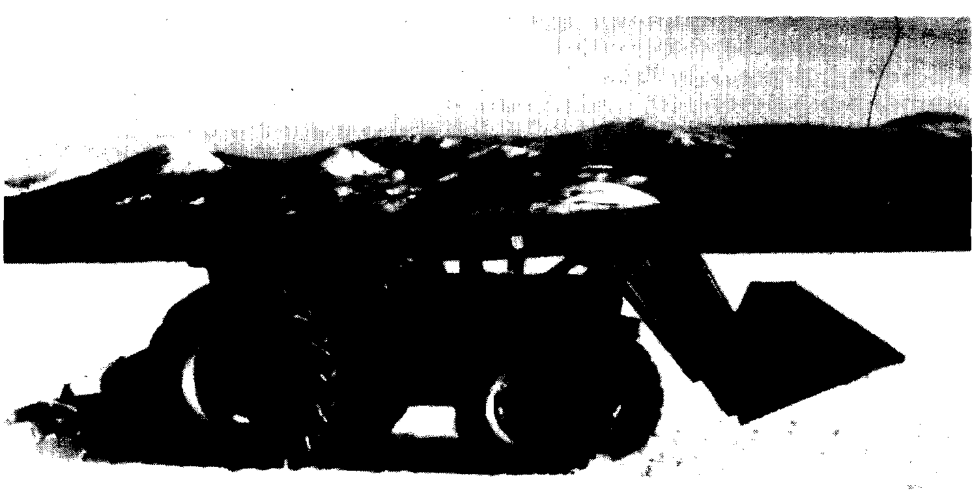

PLATE NO. 7
Mining Ice from Kluane Lake
To Construct Ice Aggregate Road

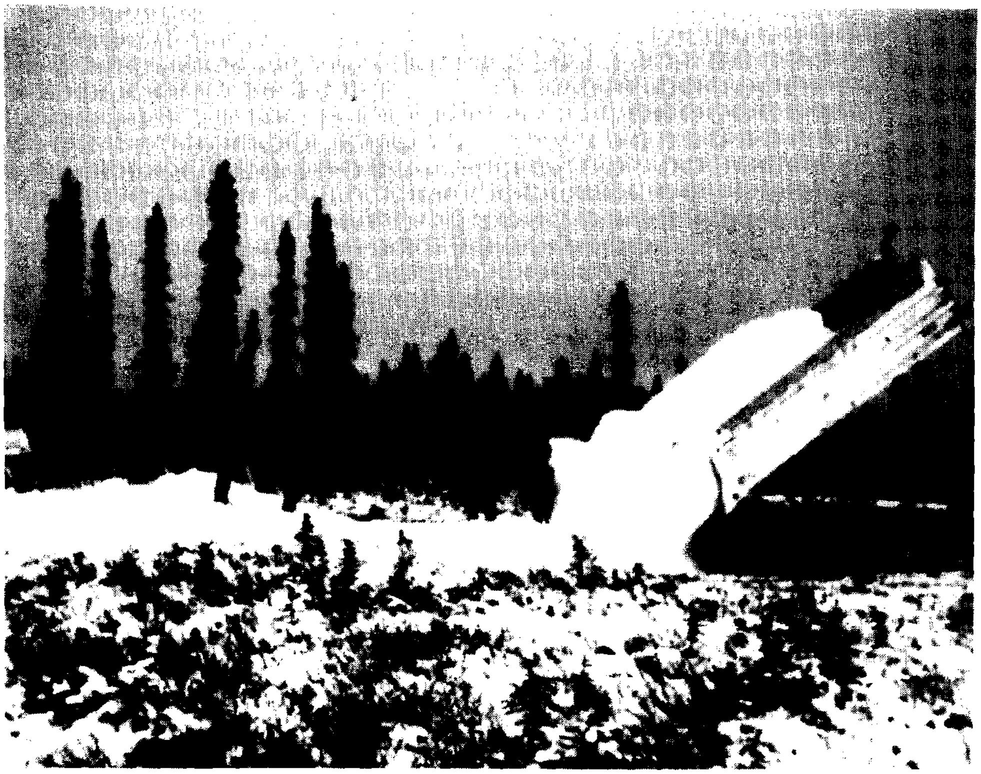

PLATE NO. 8
Ice Mined from Kluane Lake is
End Dumped in Place for Ice Road Construction

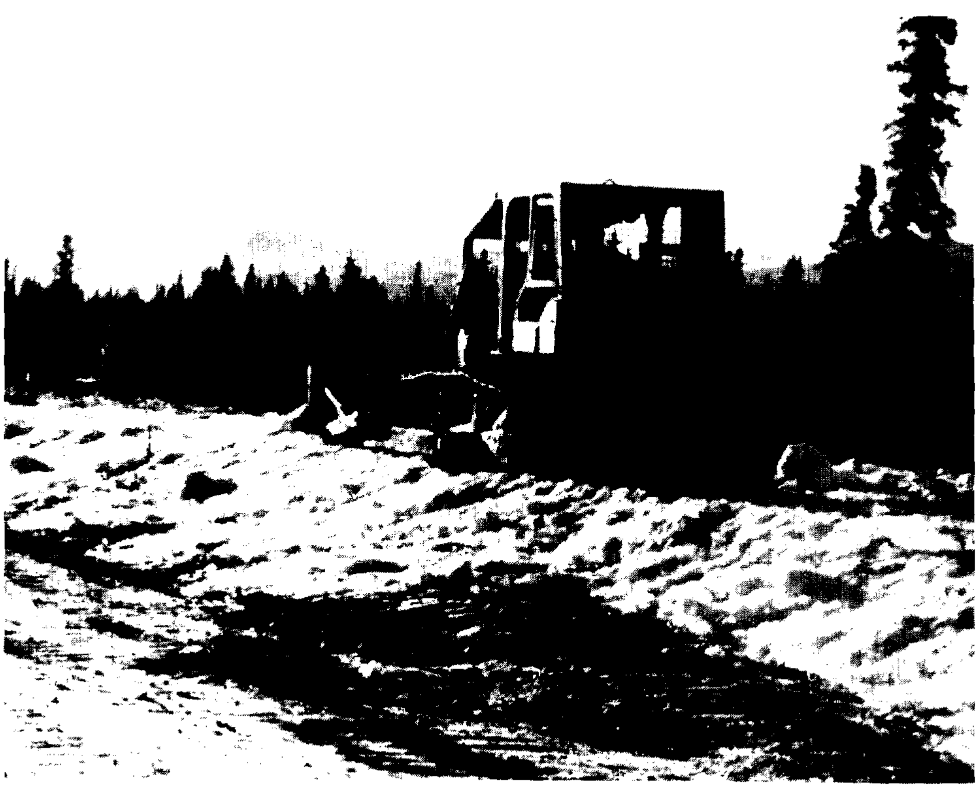

PLATE NO. 9
The Ice is then Spread with a Dozer

PLATE NO. 10
Finished Ice Aggregate Road in Use

PLATE NO. 11
Construction of Right-of-Way on Side Slope

PLATE NO. 12

Protection of Cut Side Slope
Wire Mesh Used to Hold Peat Layer in Place

PLATE NO. 13

Protection of Cut Side Slope
1.0 m Thick Peat Layer Covered by Jute Blanket

PLATE NO. 14
Protection of Cut Side Slope
Protected by 1.7 m Thick Gravel Blanket

PLATE NO. 15
Protection of Cut Side Slope
Protected by Filter Cloth and Gravel Toe Buttress

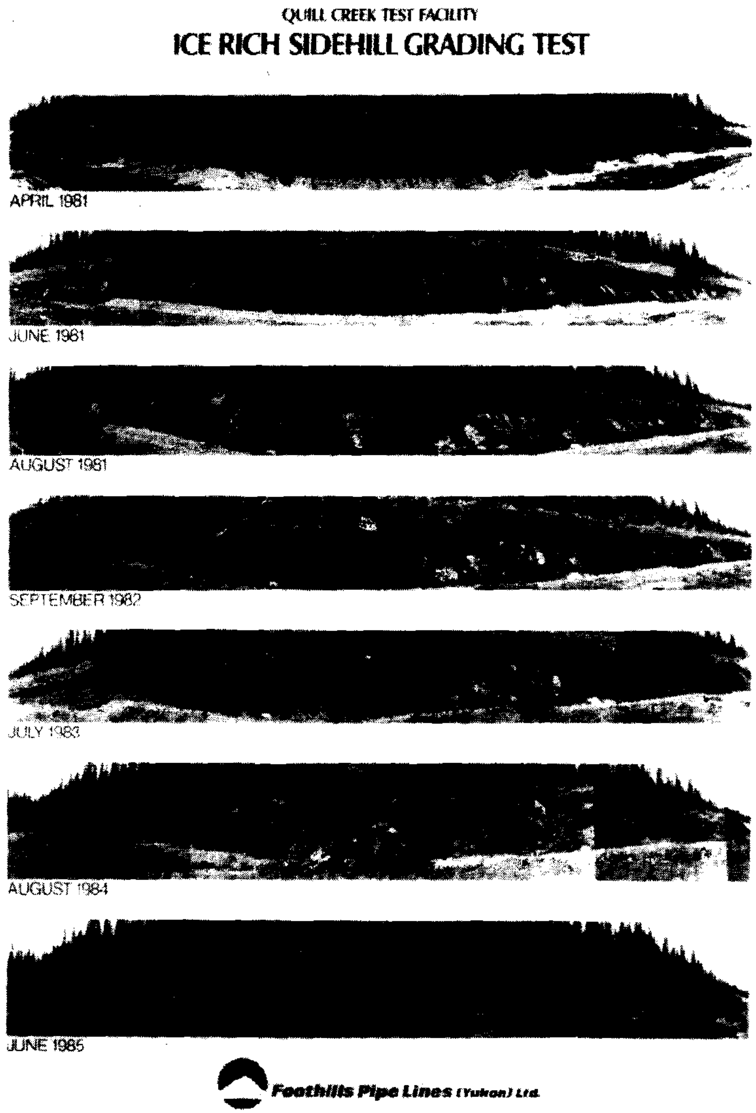

PLATE NO. 16
Unprotected Cut Slope

PLATE NO. 17
Cut Side Slope Test Area

PLATE NO. 18
Massive Ice in Ditchwall

PLATE NO. 19
Field Applied Insulated Pipe

PLATE NO. 20
Shop Applied Insulated Pipe

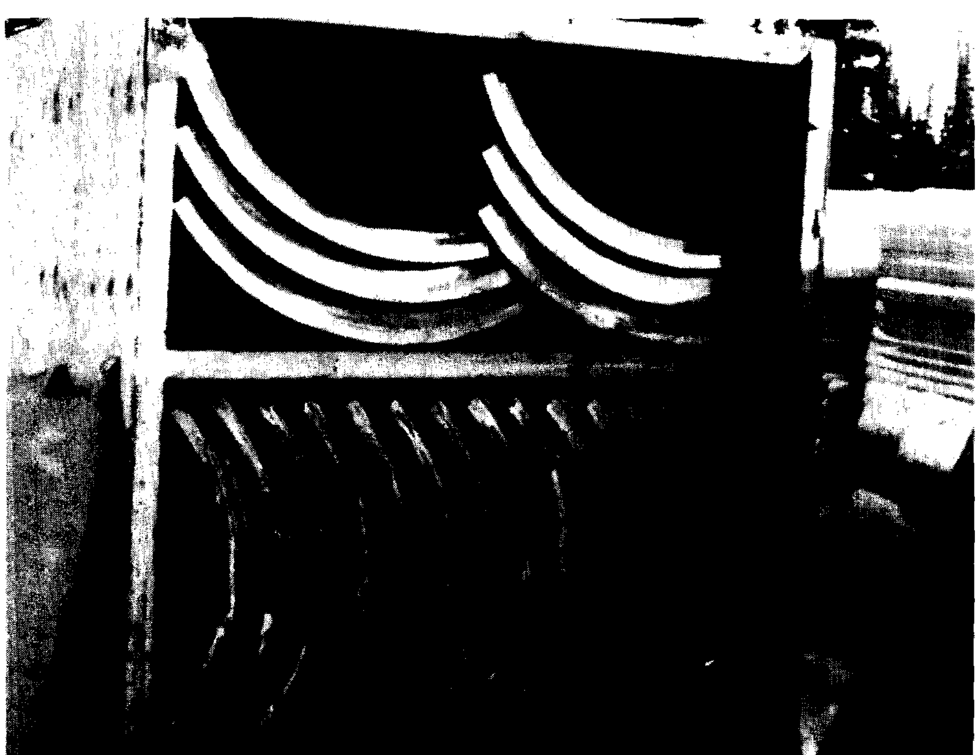

PLATE NO. 21
Segmented Insulation

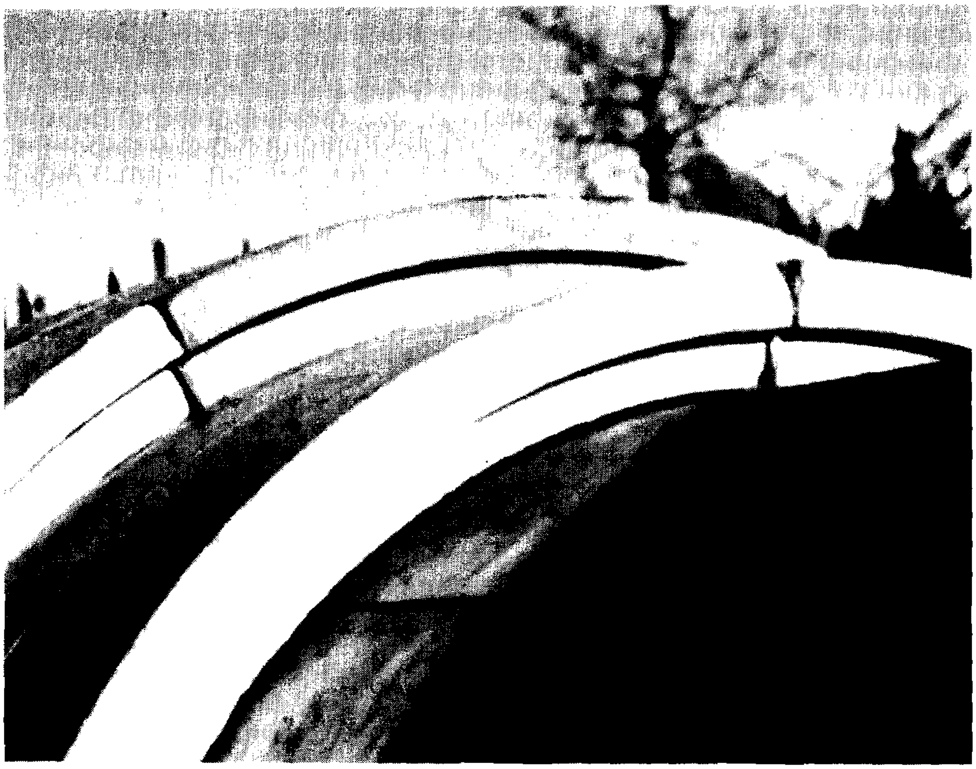

PLATE NO. 22
Segmented Insulation Installed on Pipe

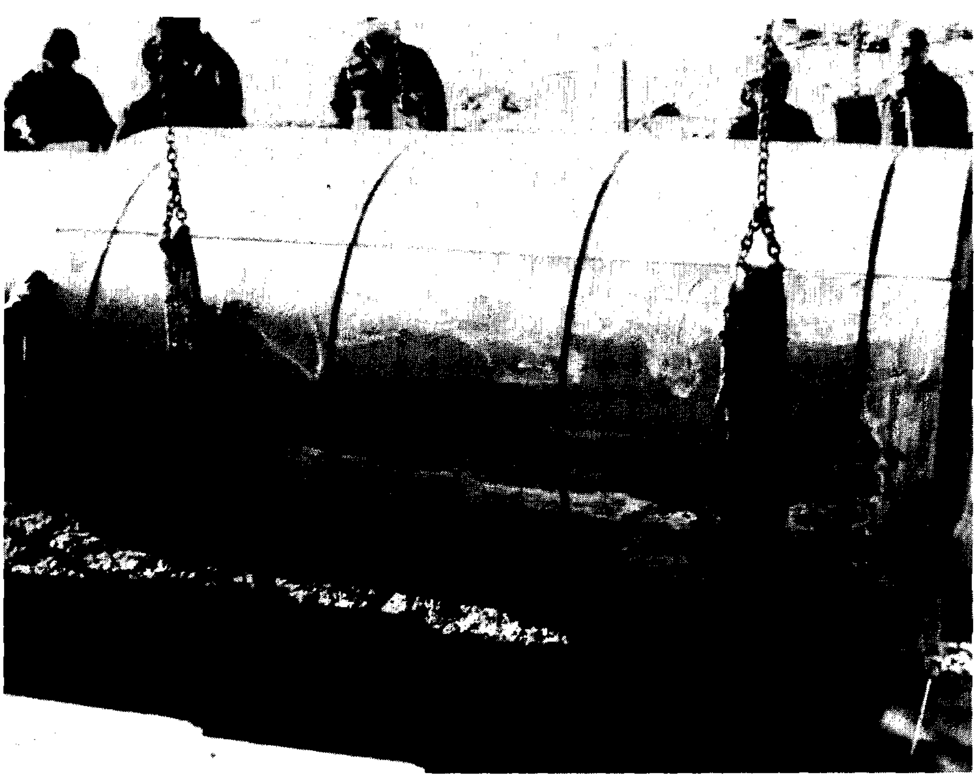

PLATE NO. 23
Steel Band Straps Hold Segmented Insulation
System Together

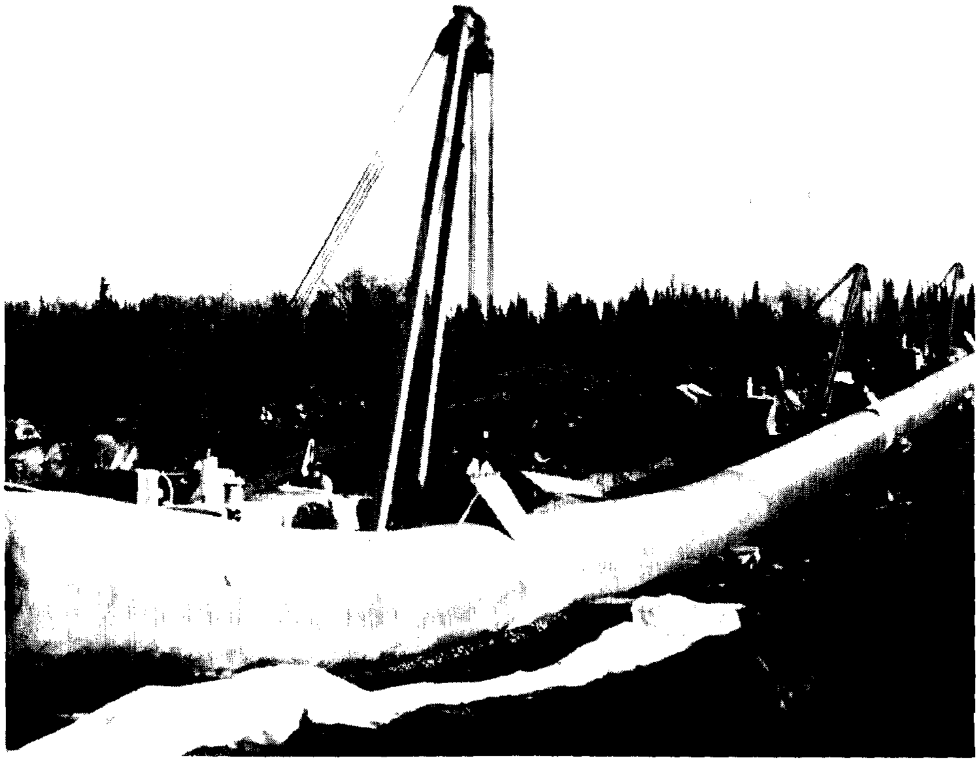

PLATE NO. 24
Lowering-In an Insulated Section of Pipe

PLATE NO. 25
Embankment Mode
Gravel Levelling Course Followed by a Layer of
Insulation and More Gravel

PLATE NO. 26
Embankment Mode
Placement of Insulation in Pad

PLATE NO. 27
Embankment Mode
Pipe Installation

PLATE NO. 28

Embankment Mode
Covering Pipe with Granular Material

PLATE NO. 29

Completed Embankment

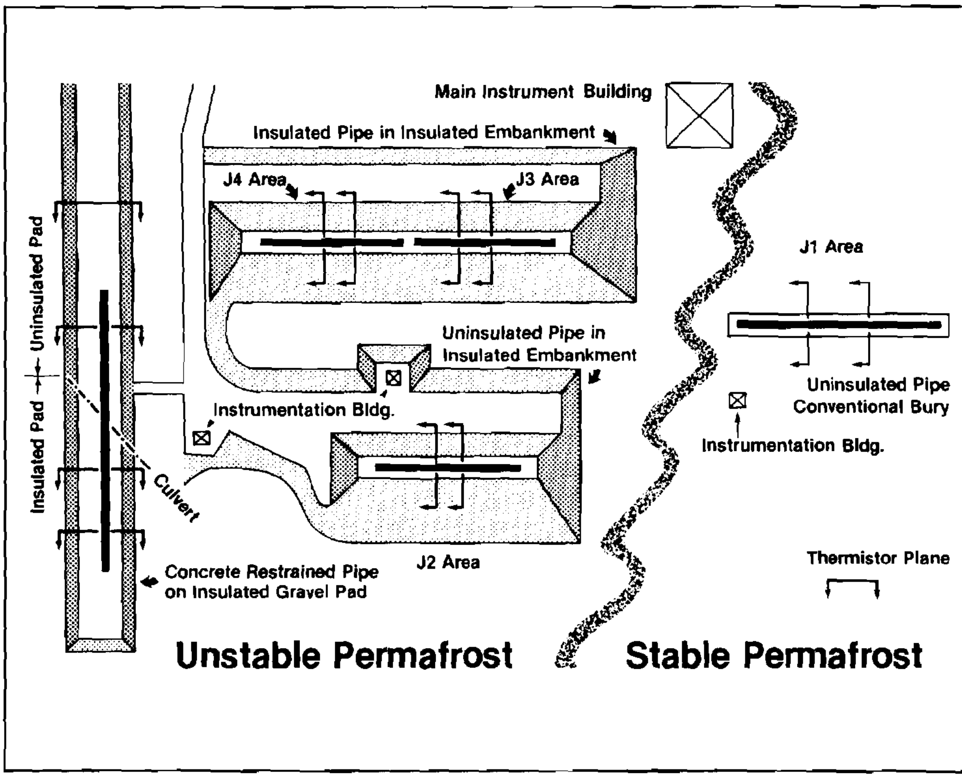

PLATE NO. 30
Warm FLow Test Sections

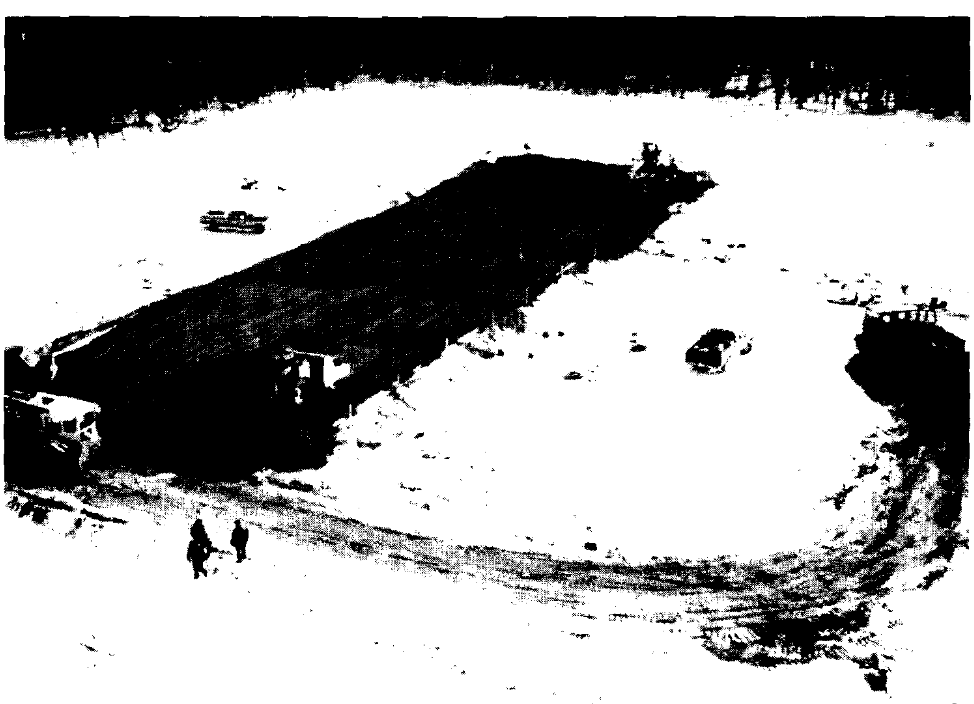

PLATE NO. 31
Warm Flow Testing
Embankment Construction
First Lift of Gravel

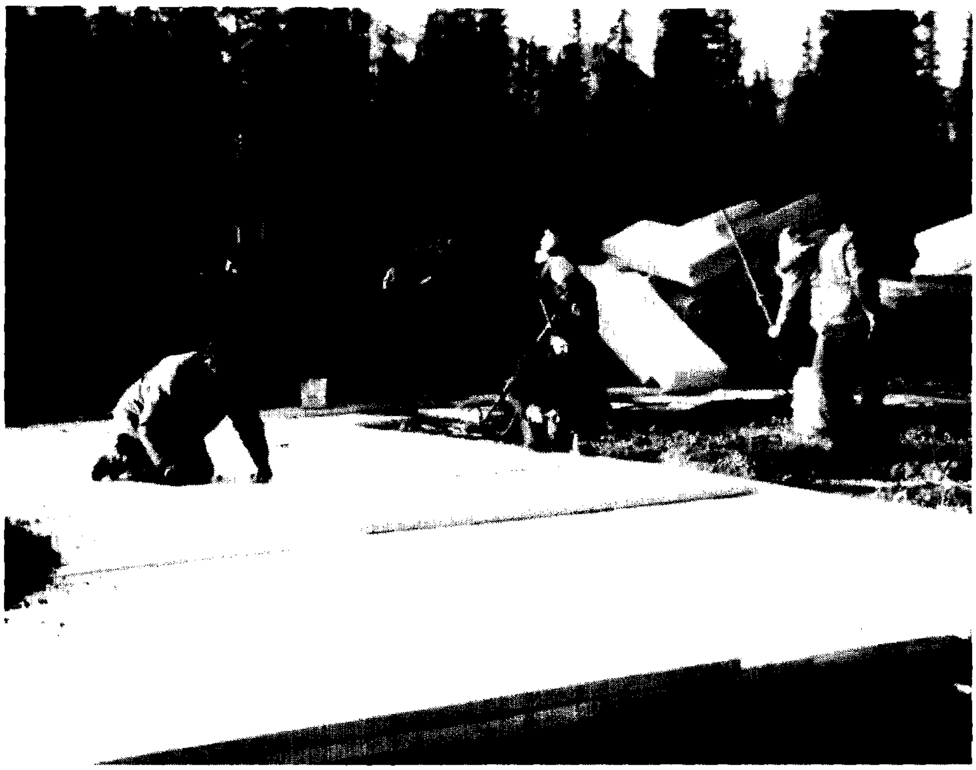

PLATE NO. 32
Warm Flow Testing
Embankment Construction
Placement of Insulation In Pad

PLATE NO. 33
Warm FLow Testing
Embankment Construction
Installation of Insulated Pipe

PLATE NO. 34
Warm Flow Testing
Embankment Construction
Covering Pipe with Granular Material

PLATE NO. 35
Warm Flow Testing
Embankment Construction
Completed Embankment

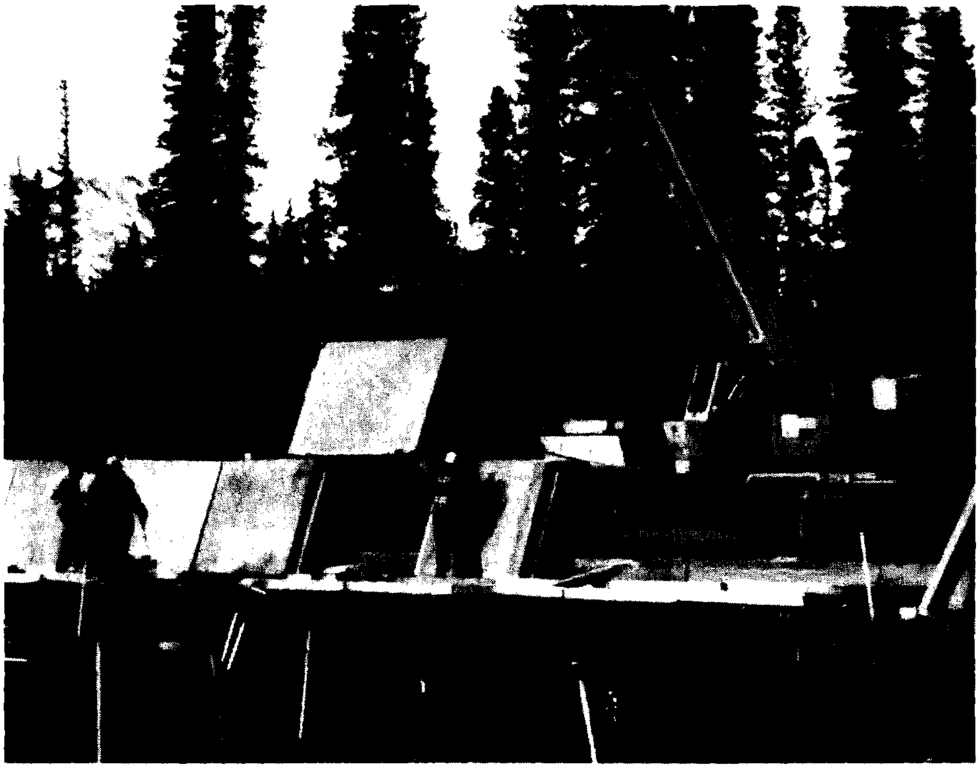

PLATE NO. 36
Construction of Concrete Restrained Mode

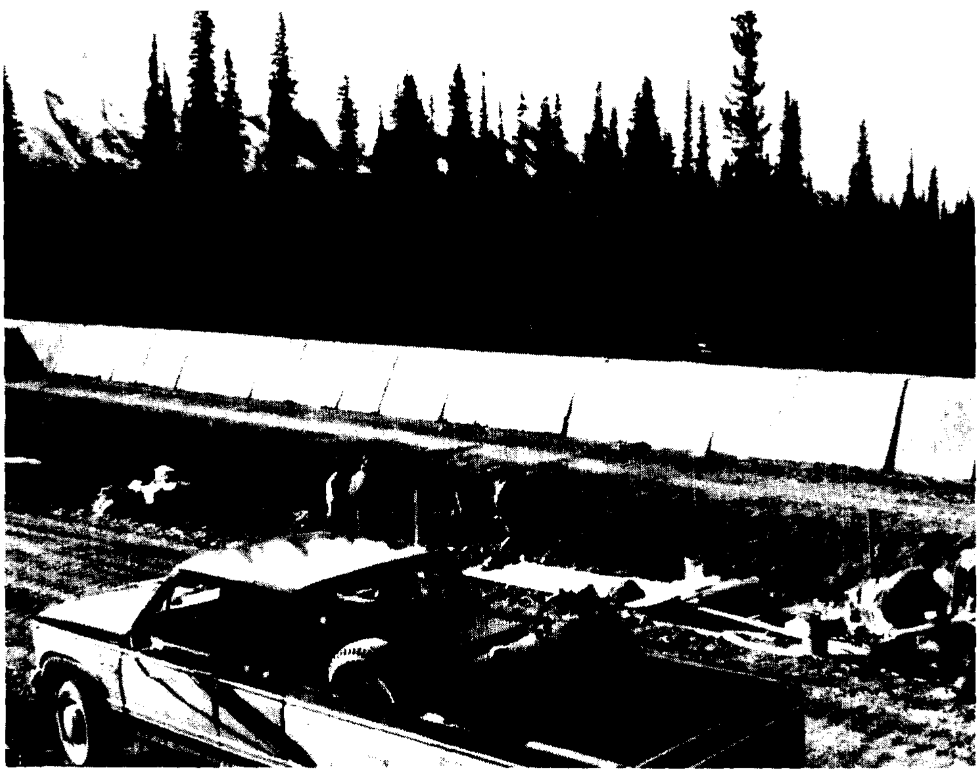

PLATE NO. 37
Completed Concrete Restrained Section

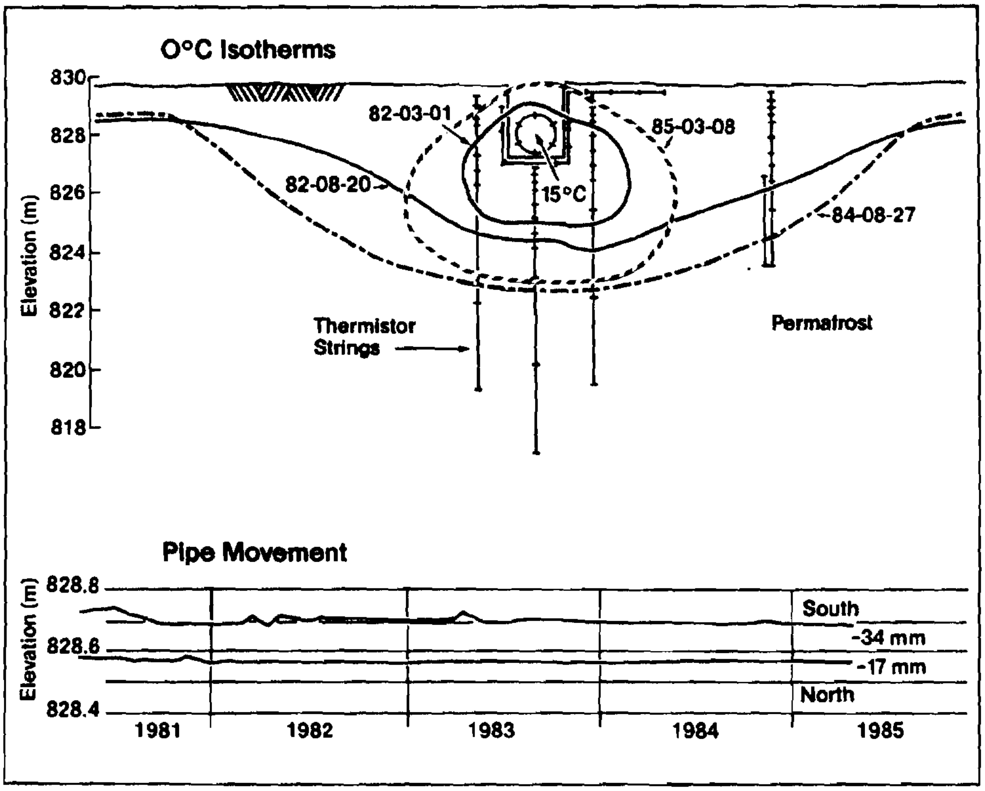

PLATE NO. 38

Test Section J1

Conventional Buried Control Section - Thaw Stable Permafrost

0°C Isotherms and Pipe Movement

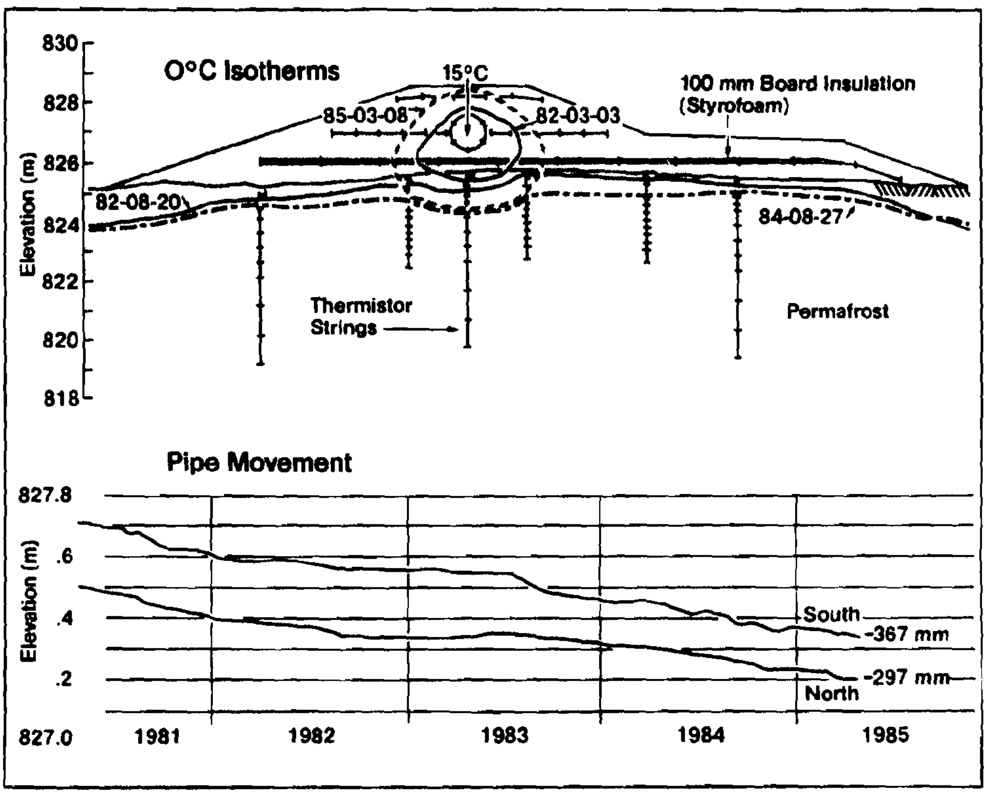

PLATE NO. 39

Test Section J2

Insulated Embankment - No Insulation on Pipe - Thaw Unstable Permafrost

0°C Isotherms and Pipe Movement

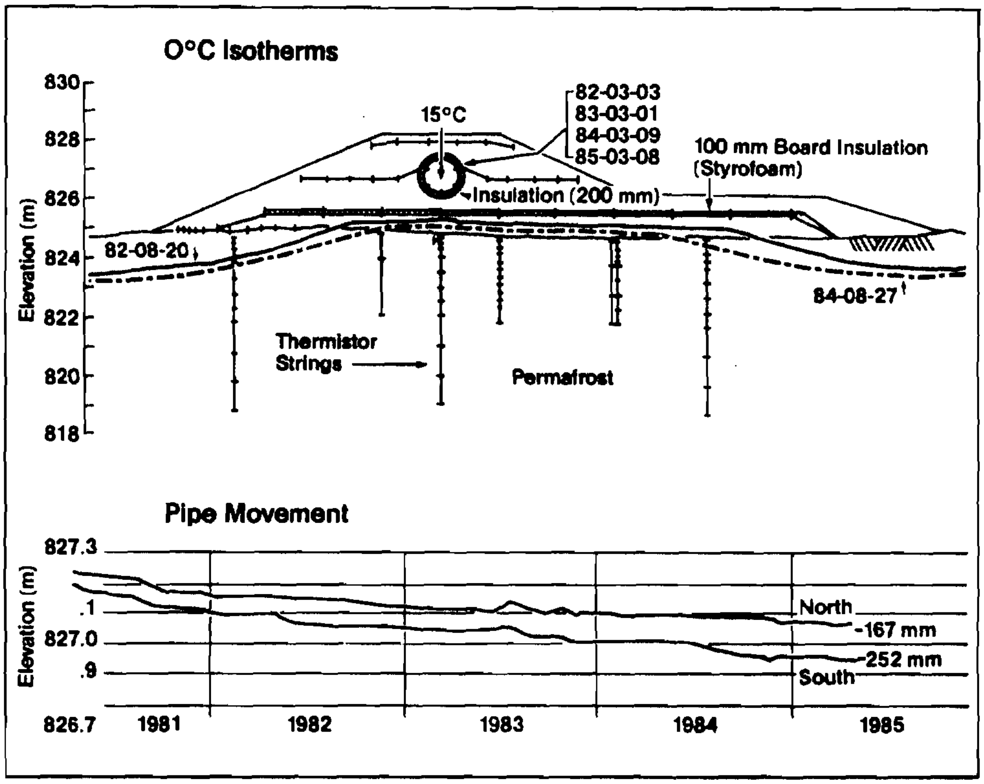

PLATE NO. 40

Test Section J3

Insulated Embankment - Insulated Pipe - Thaw Unstable Permafrost

0°C Isotherms and Pipe Movement

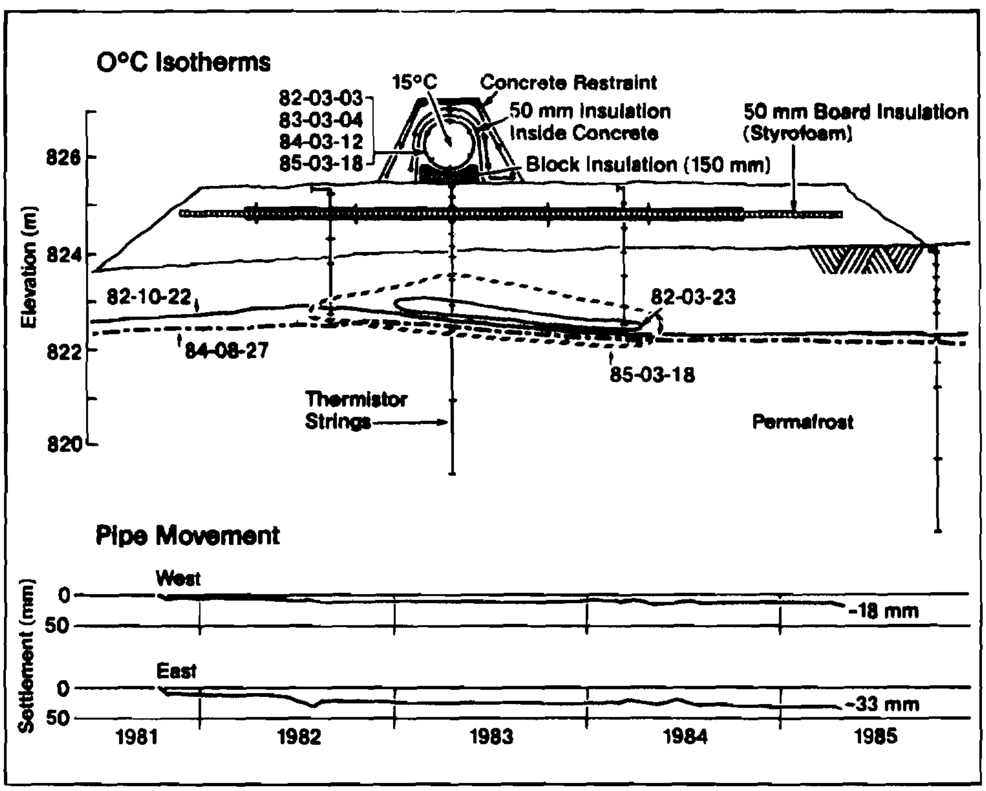

PLATE NO. 41

Test Section J8

Concrete Restrained - Thaw Unstable Permafrost

0°C Isotherms and Pipe Movement

POLAR GAS PROJECT: STRUCTURAL DESIGN ASPECTS

George H. Workman*

Abstract

This paper presents some of the structural reasons for the engineering decision to operate a warm; i.e., above 0°C, buried gas pipeline within the discontinuous permafrost region in northern Canada. Also, a discussion is given on three areas of soil-pipe interaction behavior where research and/or development is currently required to better define the complex response of buried pipelines.

Nomenclature

B = Depth of Pipe Burial
DT = Temperature Differential
D/t = Diameter-to-Wall Thickness Ratio
E = Modulus of Elasticity
OD = Outside Diameter
P = Internal Gas Pressure
SMYS = Specified Minimum Yield Strength
WT = Wall Thickness

Introduction

The right-of-way of any major buried northern gas pipeline may traverse large expanses of three generalized types of northern soil conditions. These generalized soil conditions are continuous permafrost, discontinuous permafrost, and unfrozen soil. The material properties and in-situ behavior of the soil in these different thermal configurations change markedly. Depending on the operating temperature of the buried gas pipeline, two abnormal response situations can occur within the coupled soil-pipe structural system. These abnormal responses are thaw settlement and frost heave. Thaw settlement can occur when a warm, i.e., above 0°C, pipeline passes through initial frozen soil. Frost heave can occur when a cold, i.e., below 0°C, pipeline passes through initial unfrozen soil. The amount of settlement or heave and its consequence on the operational behavior of the buried gas pipeline depends on a number of complex soil-structure parameters. However, a well designed buried gas pipeline is one that minimizes the occurrence of abnormal conditions along the right-of-way. Therefore, it is obvious that, as a general rule, a buried northern gas pipeline should be operated cold within the continuous permafrost and warm within the unfrozen soil. However, the difficult engineering decision is the mode of operation within the discontinuous permafrost. Therefore, this paper briefly outlines some of the structural reasons in favor of operating a warm buried gas pipeline through the discontinuous permafrost, and presents a brief discussion on three areas where research and/or development is currently required to better define the complex soil-pipe interaction response of buried

* See List of Registrants (Appendix A) for affiliation of authors.

northern pipelines. These areas are in the general topical areas of pipe, loads, and soil response.

Pipeline's Response to Differential Soil Movement

The major structural consequences of either thaw settlement or frost heave are derived from the differential vertical movement of the pipeline along its longitudinal length. For a given pipeline configuration, i.e., pipe size and grade, internal gas pressure, operating temperature, etc., the degree of severity of a differential movement depends on the associated constraint of the soil furnished to the pipeline. The soils susceptible to high thaw settlements are normally ice rich soils which, when thawed, have a fairly low stiffness. However, soils susceptible to high frost heaves are normally highly saturated soils which, when frozen, have a fairly high stiffness. Although the localized frozen soil stiffness is not as high as that of the pipe, there is so much more active soil around the pipe that the relative influence of the soil on the pipe is magnified in the frozen state. Therefore, the newly frozen soil which is heaving, constrains the pipeline much more than the unfrozen thawed soil, due to both its higher stiffness and larger active region around the pipe. As a consequence, a given pipeline configuration can tolerate far more differential thaw settlement than frost heave. This is shown graphically within Figure 1 which is a plot showing the design allowable configuration for a given pipe size and grade, internal pressure, and temperature differential subjected to thaw settlement and frost heave. The design allowable thaw settlement is almost five times the design allowable frost heave as the pipe is less constrained by the soil to find its optimum deformed configuration. In structural mechanics the theory of minimum potential energy states that a structural system subjected to external loads and constraints will assume the deformed shape which generates the minimum potential, i.e., strain, energy within the system consistent with the boundary constraints. Therefore, the pipeline wants to obtain an optimum, from a total strain energy point of view, configuration subject to the constraints. Consequently, in general, for a given differential displacement, the more the system is constrained the higher the induced strains within the pipeline.

In addition, due to the relatively high constraint furnished by the frozen soil, the differential frost heave loading is more of a displacement controlled situation than the differential thaw settlement loading. This means that the induced strains within the pipe are governed more by the differential movement itself, and not the pipe's structural response characteristics. Therefore, the influence of changes in pipe parameters are not as effective on the resulting structural response to the soil-pipe interaction problem. Consequently, the effectiveness of pipe wall thickness changes are more dramatic in thaw settlement loadings than frost heave.

Traditional buried pipeline design normally selects the pipe wall thickness based on code and regulatory requirements, which specify the minimum wall thickness based on the maximum operating gas pressure, pipe material, and outside diameter as defined by the circumferential stress requirement. Also, thaw settlement and/or frost heave requirements dictate the generation of specialized structural design criteria not encountered within normal southern buried pipeline design. Historically, the predicted thaw settlements and/or frost heaves when evaluated against these criteria have indicated that mitigative measures are required. These mitigative measures can take a variety of forms, ranging from changing the pipeline's operating conditions to altering the environmental conditions. However, past constructional experience in the arctic gives rise to the realization that any novel field constructional techniques will be difficult, expensive, and generate questionable results due to the general environmental conditions present along the routing and, in particular, winter construction in the arctic and sub-arctic. Therefore, for the analysis of a particular loading case, i.e., thaw settlement, frost heave, etc., if the design criteria are violated, thus indicating that some mitigative measures are required, a design re-evaluation must be conducted. In evaluating the potential solutions to the design problem, heavy reliance should be placed on "shop" methods and proven northern pipeline field construction techniques and not on novel construction gimmicks, i.e., "field" methods. "Shop" methods have the advantage over "field" methods in that the costs are more predictable and controllable. One such "shop" method which proves to be very cost effective is an increase in pipe wall thickness. Besides the economical advantages of a slight increase in pipe wall thickness, as compared to the equivalent field construction techniques, is that the increased structural performance is built into the pipe and not external to it. Therefore, the pipeline does not have to rely on some external agent to strengthen it or alter the loading environment, which in service may not perform as assumed in the design office environment. As described above, thaw settlement designs have proven to be amenable to pipe wall thickness changes in generating practical pipeline designs in the northern climates.

Development Areas

The prediction of soil-pipe interaction response behavior has progressed dramatically in the past two decades. This progress has been fueled by the engineering design and analysis requirements associated with the regulated construction of major northern pipelines and the advances in numerical simulations by the availability of high speed digital computations. However, the numerical simulation of any complex phenomenon, e.g., soil-pipe interaction, requires the continual refinement and development to better predict the actual response behavior. For general soil-pipe interaction problems and thaw settlement response, in particular, three areas of research and/or development have been identified which will result in more adequate numerical simulations. These areas are pipe failure criteria and effective temperature differential and overburden loading.

Major pipeline failures occur when a through wall crack appears within the pipe and the contents leak. This crack growth can only occur as a result of a tensile stress field. For differential soil movements, i.e., thaw settlement, frost heave, etc., the three major strain components within the pipe are a circumferential membrane strain caused by internal pressure, a longitudinal membrane strain caused by temperature differential and the Poisson's effect of the internal pressure, and a gross bending strain caused by the response to the differential soil movement. This type of pipeline response generates a global tensile strain on one side of the pipe and a global compressive strain behavior on the other side due to the high bending moment which can be introduced within the pipeline in response to the differential soil movement. Consequently, on the tension side of the pipe, the required tensile stress field is available for crack growth. However, on the compression side, a local tensile stress field can be generated if local buckling of the pipe wall occurs in response to the high global compressive strains caused by the bending of pipe. Indeed, many pipeline failures can be directly traced to a local buckling of the pipe wall and the subsequent low cycle fatigue failure caused by the local tensile stresses percent in the buckled region of the pipe. In addition to the potential generation of a local tensile stress field, local pipe wall buckling also adversely effects the serviceability of the pipeline. Therefore, two failure criteria are generally defined for the pipeline's response to differential soil movements. These failure criteria are based on local buckling of the pipe wall in compression and fracture mechanics considerations in tension.

In most major buried northern pipeline projects, due to the high positive temperature differential, the compressive behavior tends to govern the design for different soil movements. Consequently, a great amount of work has been performed to define the compressive failure criteria. Figure 2 shows an example of this type of response behavior and failure criteria. For the Polar Gas Project, the compressive local pipe wall buckling criteria was generated by an analytical approach which was first given by Vol'mir (Ref.1). This methodology is based on the deformation theory of plasticity and the double modulus (secant and tangent) approach. The detailed description of this methodology is beyond the scope of this paper, however, Figure 2 compares this analytical methodology against full-scale pipe tests on pressurized axially loaded 48-inch diameter pipes (Ref. 2) subjected to a bending load. Figure 2 shows a number of interesting points. The tests performed on the actual pipe show a great amount of scatter. As an example, for the low pressure tests, the maximum measured critical longitudinal strain is almost twice the minimum measured value. For the higher pressure tests, this scatter has been reduced such that the maximum measured value is 50 percent greater than the minimum value. For the pipe sizes, i.e., diameter-to-wall thickness ratios, utilized in pipeline applications, the local buckling response of the pipe is in the inelastic, i.e., plastic range. The elastic strain associated with the yield strength, i.e., SMYS, of the pipe is shown as the short horizontal line given within the figure. All measured critical strain values are significantly beyond this extended region of elasticity. The theoretical results envelope the measured results in five of the seven tests. The average difference between the measured and predicted values is 4.8 percent, while the maximum single difference is 32.4 percent in the high pressure test. In two of the low pressure tests, the theory over-predicted the critical strain. However, it significantly under-predicted the critical strain for one test at the same low pressure. For the Polar Gas Project, the design allowable strain limits for operational loadings were taken as two-thirds of the theoretical design maximum strain. This corresponding design allowable curve is also plotted on Figure 2 for the tested pipe and it envelopes all of the measured results. Consequently, it is felt that the compressive strain criteria is well defined.

On the tension side of the pipe, fracture mechanics techniques have been applied for many years to predict crack growth phenomena associated with longitudinal cracks subjected primarily to circumferential stresses due to internal pressure. However, the generation of longitudinal tensile strain criteria by the applications of fracture mechanics techniques is in its infancy. The Polar Gas Project has funded a small project to better define longitudinal tensile strain criteria based on fracture mechanics considerations. Therefore, this is one area where further development is required to better understand and predict actual crack growth due to the tensile stress fields, i.e., both longitudinal and circumferential, present within buried pipelines subjected to differential soil movements.

In general soil-pipe interaction response problems the two dominate loadings are internal pressure and temperature differential. The temperature differential is the difference between the operating temperature of pipe and the so-called reference temperature. The operating temperature of the pipe will be at or very near the temperature of the flowing gas. The reference temperature is defined as the thermally stress-free, i.e., zero thermal stress, temperature of the pipe. This is normally assumed to be the temperature of the pipeline when laid in the ditch and backfilled. The assumed reference temperature is, therefore, taken as the ambient air temperature at the time of installation and tie-in. In most major northern pipelines with winter construction and an operating temperature near freezing, this is a conservative assumption. Figure 3 shows the required pipe wall thickness for a given thaw settlement case as a function of temperature differential. Figure 3 demonstrates that a change in temperature differential will have a corresponding change on the required pipe wall thickness. For a minimum winter construction temperature of -40°F and operating temperature of +42°F, the required wall thickness is 0.595 in. for the case shown within Figure 3. However, some proprietary experimental and analytical programs have indicated that the actual reference temperature under winter construction conditions could be 10°F or higher than currently utilized. Also, analytical studies have shown that the effect of the hydrostatic testing of the pipe on the subsequent pipe reference temperature can be significant. Therefore, if it is assumed that the "real" reference temperature is 20°F higher for this situation, then the required wall thickness is 0.527 in. or a reduction in the required pipe wall thickness of over 10 percent. This has a significant economical impact on the pipeline design. Therefore, an industry accepted methodology for more adequately predicting the actual reference temperature within buried pipelines should be developed.

The dominate soil loading for the thaw settlement response is the maximum overburden load within the settling soil zone of the pipeline. Like the reference temperature, the maximum overburden load currently is based on conservative assumptions without a great deal of measured response data. Figure 4 shows the required pipe wall thickness for a given thaw settlement case as a function of maximum overburden load. This figure shows that, at least for the case studied, the overburden load-wall thickness relationship is slightly nonlinear. Also, shown on the figure, is the buried depth to the top of the pipe used in these analyses as a function of overburden load. In actual service, thaw settlements occur over many years in combination with seasonal freeze-thaw cycles on the surrounding soil. Therefore, a better understanding of this phenomenon could change the depth-overburden relationship currently utilized within the design process. Similar to the reference temperature, a reduction in the overburden load based on measured response data, could have a significant cost impact on a major pipeline project.

Conclusion

This paper has given some of the structural reasons in favor of operating a warm buried gas pipeline through the discontinuous permafrost and has briefly discussed some of the important structural parameters where further refinement or development would have a beneficial effect on predicting soil-pipe interaction behavior. In design situations where assumptions must be made, the prudent engineer must rely on historical practices as modified by field experience and judgement to make reasonable yet conservative assumptions. If some of these conservative assumptions can be replaced, through a better understanding of the actual inservice behavior, then the pipeline industry, and the public in general, will benefit.

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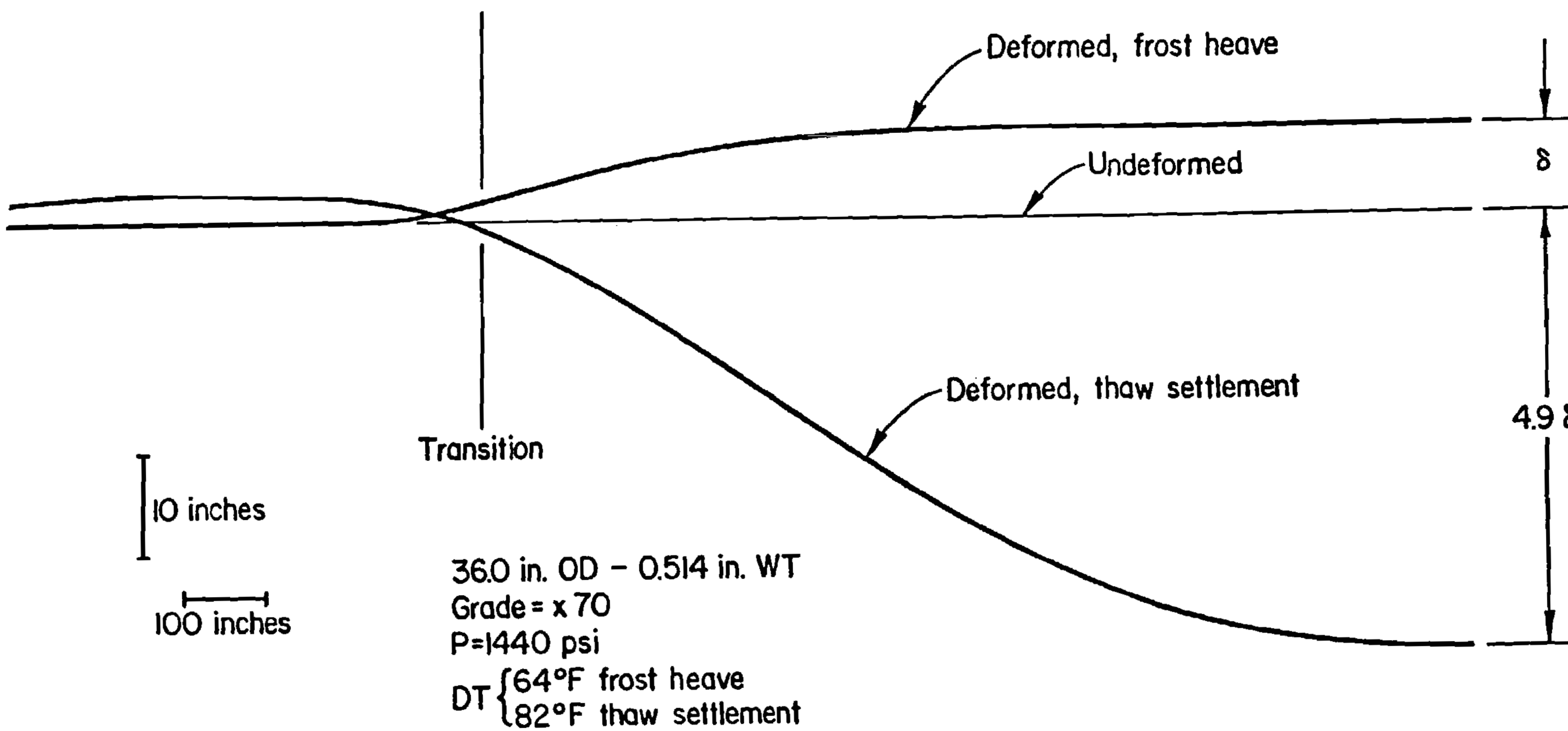

FIGURE 1 – FROST HEAVE – THAW SETTLEMENT COMPARISON EXAMPLE

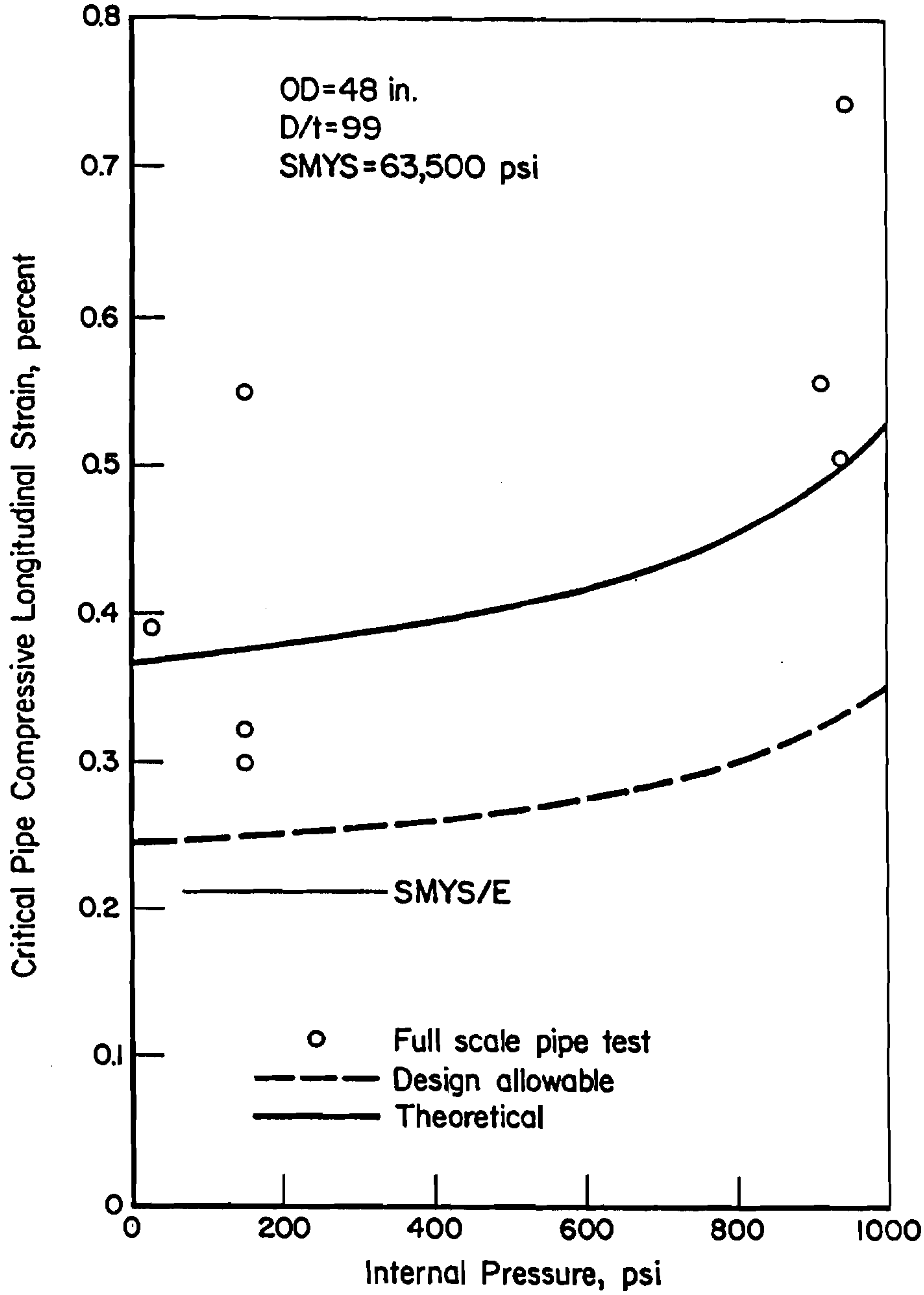

FIGURE 2 - PIPE WALL LOCAL BUCKLING VALIDATION EXAMPLE

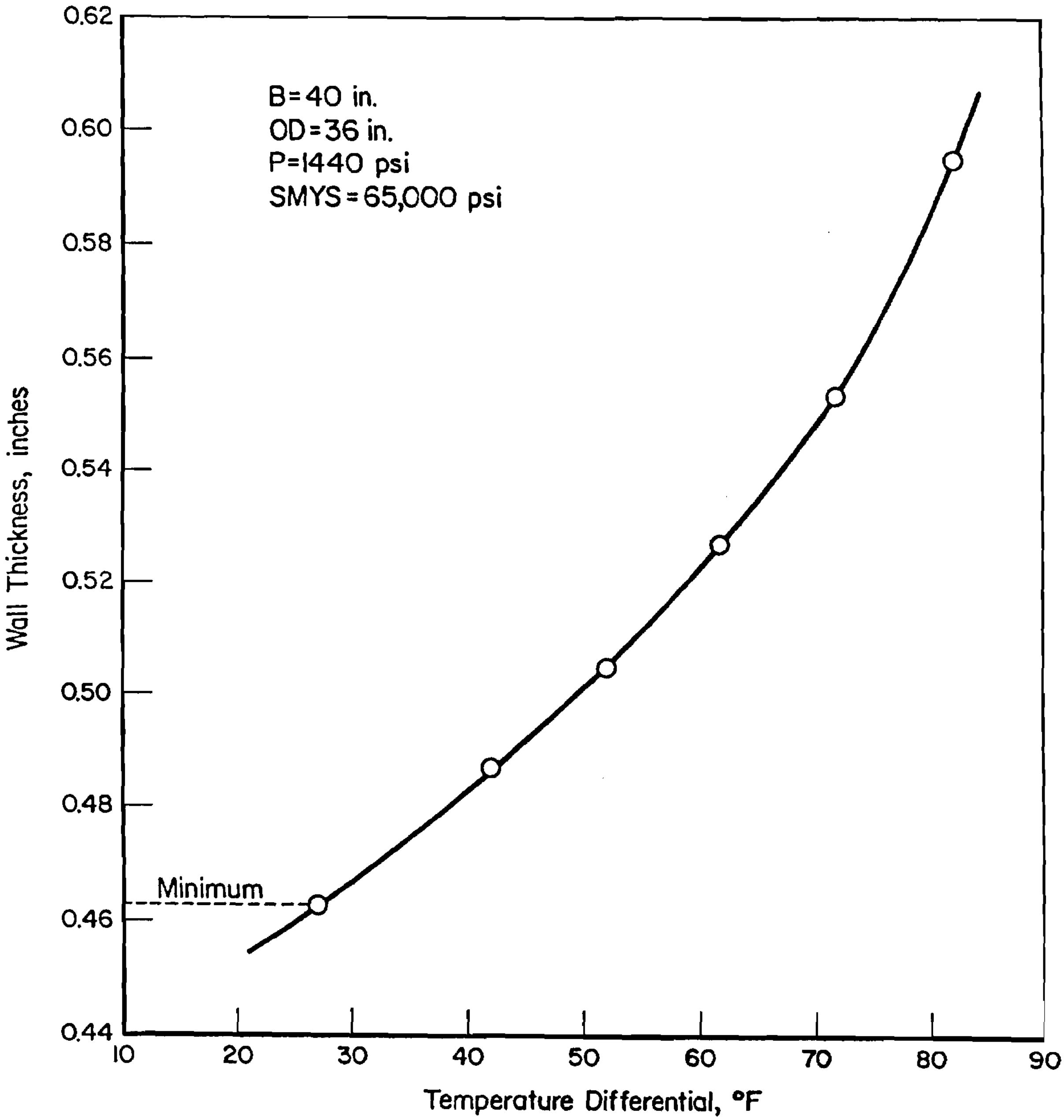

FIGURE 3 - EXAMPLE PIPE WALL THICKNESS - TEMPERATURE DIFFERENTIAL RELATIONSHIP FOR THAW SETTLEMENT

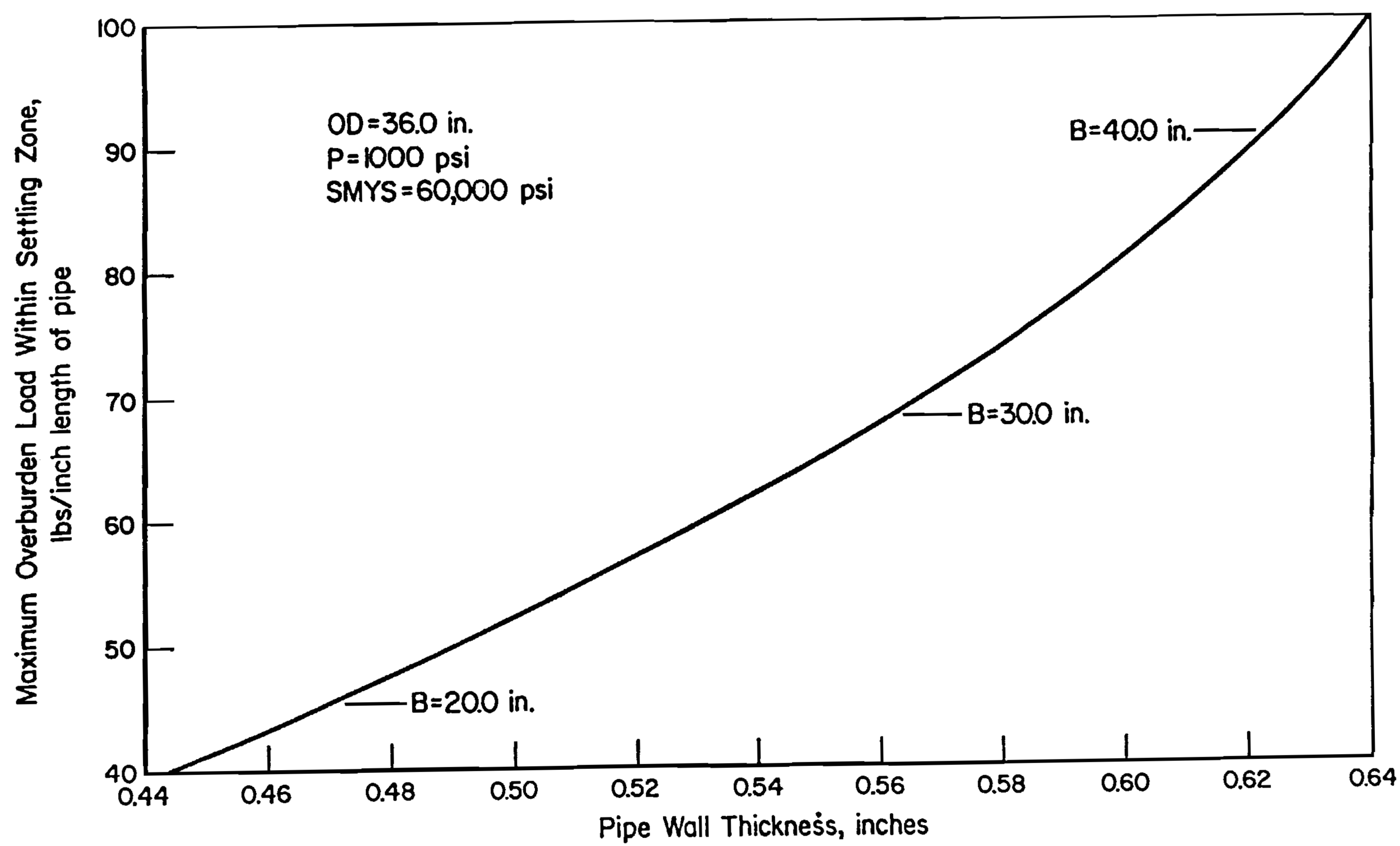

FIGURE 4 - EXAMPLE PIPE WALL THICKNESS - OVERBURDEN LOAD RELATIONSHIP FOR THAW SETTLEMENT

ALYESKA PIPELINE: MONITORING, OPERATIONS AND MAINTENANCE

E.R. Johnson*

Trans Alaska Pipeline (TAPS) traverses permafrost along 70% of the pipeline route. Providing an uninterrupted and environmentally safe operation of the pipeline, is extremely important, to the owners, the producers, the State of Alaska and even the national interest. Significant engineering and financial resources are committed toward maintaining the operating integrity of TAPS due to the special geologic hazards imposed by permafrost foundation materials.

Management of the risk imposed by permafrost is accomplished using (1) a comprehensive surveillance and monitoring program, (2) intensive engineering evaluation, and (3) a preventive maintenance program.

The surveillance and monitoring programs conducted by Alyeska have been very successful in identifying permafrost instability problems and allowing preventive maintenance to be performed. The below ground pipeline has been repaired at 10 different locations since 1979 to prevent excessive pipe strain caused by settlement. The above ground pipeline has performed well, with only 1.6% of the total pile support population (78,000 total piles) exhibiting jacking or settlement in excess of 9 cm (0.3 ft). One problem found with the above ground pipeline supports has been the slow deterioration of heat pipe performance. Heat pipes are used to remove heat from the ground during winter and hence maintain the soils in a permanently frozen condition. Approximately 30% of the heat pipe population (122,000 total heat pipes) has exhibited some level of cold top condition caused by accumulation of noncondensable gas on the inside of the heat pipes. As for steep slopes in permafrost, only one major repair has been required since pipeline start-up.

A vital lesson is taught from the Alyeska experience. Pipelines in permafrost; and especially those with high operating temperatures, are subject to a significant risk of foundation instability. This risk must be viewed objectively and a strategy of risk-management developed which includes the feasibility, design, construction and operating stages of the project. Monitoring of foundation performance during operation is essential to verify that the theory used by engineers is compatible with the reality imposed by Mother Nature.

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* See List of Registrants (Appendix A) for affiliation of authors.

TABLE I

PIPELINE STABILITY OVERVIEW

A. Major distinguishing design factor - permafrost on 70% of line.

B. Many important facilities subject to risk of permafrost instability.

		Total	Permafrost
1.	conventional buried pipeline	380 miles (613 km)	230 miles (371 km)
2.	aboveground pipeline	416 miles (671 km) (78,000 VSM)	332 miles (535 km) (61,000 VSM)
3.	refrigerated below-ground pipeline	4 miles (6.5 km)	4 miles (6.5 km)
4.	refrigerated pump station piping	1.8 miles (2.9 km)	1.8 miles (2.9 km)
5.	pump station foundations	11	5
6.	steep slopes	53	32

C. The three biggest pipeline stability concerns to date.

1. below ground pipeline settlement
2. progressive failure of heat pipes in thermal VSM
3. failure of permafrost slopes

TABLE II

BELOWGROUND PIPELINE STABILITY

A. BACKGROUND

- Two pipeline failures occurred during same week in June 1979 - pipe had been in operation only 2 years.

- Failure at Atigun resulted from 4.0 ft (1.2 m) of settlement - pipe buried in ice-rich bedrock.

- Failure at MP 734 resulted from 5.5 ft (1.7 m) of settlement - pipe buried in undetected permafrost.

- A major pipeline monitoring program was instituted to detect other potential problem areas.

- Have detected 8 other areas requiring repair since 1979.

B. MONITORING PROGRAM

- Objective of stability monitoring program is to find all potentially damaging pipe settlement areas to allow effective and efficient repair <u>before</u> pipe failure.

- Program consists of many different interrelated techniques:

  1) surface surveillance
  2) monitoring rods - 2345 now in place
  3) routine monitoring - 1930 rod readings/year
  4) survey control - 685 bench marks now in place
  5) soil borings - 500 since startup
  6) thermistor strings - 250 now in place
  7) CD pig - run 2 times/year
  8) TSI survey - will survey most of buried line in 1985
  9) data base management - 865,000 pieces of information
  10) priority system - focuses attention on areas of greatest concern.

C. PRIORITY SYSTEM

	Priority I	Priority II	Priority III	Priority IV
- Probable repair time frame	Immediate	2-5 yrs	>5 yrs	Unlikely

- Priority criteria

Curvature % CR	>85	60-85	40-60	<40
Settlement rate ft/yr	>0.3	.15-.30	.08-.15	0-.08
Total settlement ft	>3	2-3	1-2	<1
Expected future	settlement considered on site specific bases			
CD pig anomaly	considered on site specific bases			

- Current results

Number of areas	none	7	114	--
Number of miles	none	1.5	29.7	348.9

- Response

Monitoring frequency per yr	2-3	1-2	1/2-1	1/3-1/2
Repair response	construction	contingency design	observe	none

D. MONITORING PROGRAM RESULTS

- No pipeline leaks from pipe settlement since 1979.
- Have identified 8 separate repair areas after 1979 costing 58 MM$ to fix.
- No false alarms.
- Annual monitoring costs systematically reduced with time.

Year	Annual Cost, MM$
80-82	6.3
83	2.8
84	1.7
85	1.7
Total	31.7

E. STABILITY REPAIR

Repair	Used When	Historical Cost
Underpinning	Large future settlement expected, stable zone at depth.	2-7 MM$

Thermal stabilization	Favorable environmental conditions.	6-8 MM$
Remode in place	Favorable terrain conditions.	2 MM$
Releveling	Small future settlements expected.	0.2-1.0 MM$
Reroute	High risk of future settlement. Inaccessible location High risk during repair	28 MM$

TABLE III

ABOVEGROUND PIPELINE MONITORING

A. BACKGROUND

- 78,000 VSM used to support 420 miles of above ground pipeline.

- 80% or 61,000 of VSM utilize 122,000 heat pipes to provide thermal protection.

- Random survey sampling shows generally good VSM performance.

  1) 1.0% jacked over 0.3 ft.
  2) 0.6% settled over 0.3 ft.
  3) Approximately 200 VSM have required adjustments.
  4) Repairs (over 100M$) have been required at 3 locations.

- FLIR monitoring of heat pipes has shown deterioration of thermal performance.

	1981-82	1982-83	1983-84	1984-85
Number of cold tops	5,100	12,400	23,400	36,600
% of population	4.2	10.1	19.0	30.0

B. HEAT PIPE REPAIR

- VSM failures are not anticipated because of highly conservative design and slow failure sequence.

- Presently only 250 VSM require repair.

- Repairs will involve with the use of "getter" devices which absorb and isolate noncondensable hydrogen.

- Long term repairs are projected as follows:

	1985	1986	1987	1988
No. of failures	250	400	1000	2100
Cost @ 150$/repair	38M	60M	150M	315M

TABLE IV

SLOPE STABILITY

- Have identified 53 steep slopes subject to stability concern, 32 in permafrost.

- Annual surveillance conducted to inspect slopes.

- Special monitoring conducted where instability is indicated.

  1) Survey monitoring
  2) Piezometer monitoring
  3) Slope indicator monitoring.

- Only one slope has required repair

  1) Above ground pipeline subject to deep thaw
  2) Repair utilized heat pipes - wood chip insulation
  3) Cost of repair approximately 1.0 MM$.

MONITORING OF THAWING PERMAFROST SLOPES: INTERPROVINCIAL PIPE LINE

E.C. McRoberts, A.J. Hanna and J. Smith*

Abstract

The Interprovincial Pipe Line (NW) from Norman Wells, N.W.T. to Zama, Alberta is the first fully buried oil pipeline in permafrost. The pipeline crossed 165 slopes, many of which were frozen. This paper reviews the basis for choosing wood chips to provide insulation against thawing of sensitive permafrost slopes. Performance data primarily for heat generation effects in the wood chips and on observed thaw depths and ground temperatures is provided.

Introduction

Interprovincial Pipe Line (NW) Ltd. (IPL) recently completed construction of the first fully buried oil pipeline in permafrost. The pipeline runs from Norman Wells, N.W.T. to Zama, Alberta, a distance of 868 km. Details of the pipeline have recently been summarized by Pick and Smith (1985). From Norman Wells south to kmp 400, the pipeline parallels the east bank of the Mackenzie River, crossing the valley walls of many tributary creeks and rivers. Many of these slopes along this portion of the route are frozen. South of about kmp 400 the pipeline is located along the Great Slave Plain and, with the exception of the Mackenzie River crossing, avoids permafrost slopes.

While the route was located so as to avoid major areas of known instability, permafrost slopes were unavoidable. In total, over 165 slopes were identified as requiring geotechnical examination and site specific design response. Of the 165 slopes catalogued a total of 26 slopes were instrumented (see Table 1). Because construction required two winter seasons to complete, some slopes were instrumented in the winter of 1983/84 and underwent a complete thaw season before the pipeline went into operation. The purpose of this paper is to review the results of slope monitoring for the summers of 1984 and 1985. However, before doing so it is useful to make the following introductory comments.

A chiller at Norman Wells ensures an initial oil flow temperature of between +0.5 to -3.0°C. During the summer of 1985, pipe skin temperatures rose to peak values of near +3°C south to Kmp 300 and to +5°C south of Kmp 400. During the winter months it is expected that pipe skin temperatures will fall below freezing such that on average there will be no net thermal input into the surrounding soil.

Therefore while the pipeline, being more or less at ground temperature, does not influence permafrost slope stability, a fundamental component of geothermal disturbance results from right-of-way clearing

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* See List of Registrants (Appendix A)
for affiliation of authors.

and construction disturbance effects. Previous studies (for example McRoberts et al, 1978) of thawing along cleared rights-of-way in the vicinity of the IPL line established that past terrain disturbance on cleared trails and roads initiates permafrost thawing. A review of available data indicated that complete removal of vegetation and severe damage to the organic surface cover could be expected to result in about 4 to 6 m of thaw over a 25 year period. Less thaw in the order of 1 to 4 m in 25 years, and in many instances no degradation at all, might be anticipated if construction disturbance was restricted to removal of trees alone. However, as this was viewed as being impractical it was recognized that pipeline construction would result in slope thawing.

Experience in the Mackenzie Valley (McRoberts 1978), as well as a variety of unpublished studies of failed slopes has established that thawing of permafrost slopes can cause failures. These studies supported by theoretical analyses of the type reported by McRoberts et al, (1978) lead to the conclusion that slopes associated with the IPL project greater than 9°, 13° and 18°, in ice-rich clay, ice-rich till and ice-poor till soils, respectively, would not be sufficiently stable during thawing or long-term degradation. Two major mitigation concepts were evolved to develop a sufficiently high safety factor. In highly ice-rich slopes and/or for steeper inclinations it was specified that thaw had to be virtually prevented by being restricted to the natural active layer. In other slopes, depending upon angle, soil type and ice conditions thaw was permitted but at controlled rates.

In assessing potential mitigation measures, a variety of candidates were considered. Among these were a gravel/synthetic insulation sandwich, wood chips, wood chip/synthetic insulation sandwich, and a thermopile or cryo-anchor/insulation mode. Wood chips were selected as the primary design mode for several reasons. Firstly, properties of wood chips provide a better overall geothermal solution. While the thermal conductivity is greater than artificial insulation (boards, or foamed-in-place) the wood chips retain moisture and the high latent heat effect is important in retarding seasonal thaw. Secondly, wood chips can be placed directly on the slope during winter without high quality specification and expensive construction procedures. Because some thaw is inevitable, the rigidity of artificial insulation modes were considered to be a major disadvantage because of subsequent settlement. Finally, it was anticipated that some failure may occur due either to slope movements or other effects, and wood chips were viewed as being easy to rehabilitate. An environmental perspective, ultimately supported by regulatory agencies, was that a renewable resource was being used (compared with gravel, often in short supply) and wood chips were the best choice.

There were, of course, certain disadvantages of wood chips. While the pipeline is situated in the boreal forest, tree harvesting would require development of wood lots off the right-of-way. Wood chips do not revegetate easily and in addition, there was the possibility of polluting leachates entering the surface water regime. Wood chips were also known to generate heat, due to micro-biological action, and this effect as well as possible deterioration of thermal properties with time required study.

The wood chips exert only a minor surcharge on the sloping soil and little benefit in improving effective stresses at the thaw line could be expected. Finally there was some concern that the wood chips might not be stable themselves and could be eroded or become buoyant during a heavy rainfall.

While it is generally held that wood chips might serve as an insulative material, there is a general lack of well documented precedence. Certainly sawdust and shavings have been used in Canada for many generations as an insulative material in houses, and prior to mechanical refrigeration to store ice during summer months. Wood chips and sawdust have also been used as lightweight road fills in muskeg terrain. It was known that wood chips have been used in the Mackenzie Delta as expedient pads for transportation of drill rigs and the Canadian National Railways have used them south of Churchill, Manitoba in conjunction with cryo-anchors to reduce thaw in railroad embankments (Anon, 1983).

Wood chips are a common product of the pulp and paper industry in North American and some experience is available concerning the behaviour of large piles of wood chips and sawdust. Wood chip piles can generate heat due to both respiration of still living wood cells and, primarily, to the action of a variety of micro-organisms, primarily fungi (Hajny, 1966). The action of such fungi is to reduce the amount of wood substance. Brown rots primarily associated with the types of soft woods available (pine, spruce) preferentially utilize cellulose leaving the lignen untouched and therefore leave the essential structure of the chip intact (Shields, 1977). This bacterial action gives rise to heat generation which promotes further fungoidal action. Temperatures up to 40-60°C can be found in large wood chip piles in the order of 20-25 m high. Experience in the forest industry is that chip deterioration is sharply reduced in smaller piles where the surface area to volume ratio is greater. Chemicals such as borax and sodium carbonate have been used by the forest industry to reduce wood loss and heat buildup (Shields, 1977). However, such applications raised environmental issues and in the long-term would likely require reapplication of chemicals.

In summary, as the design of the Norman Wells pipeline evolved, it became clear that measures were required to either essentially eliminate or to retard thaw in permafrost slopes. Wood chips offered several cost advantages and were considered the best solution from several technical perspectives. A range of studies were undertaken to define the appropriate thermal properties of chips, to account for the possible influence of heat generation, and to allow for the likely reduction of wood substance with time (McRoberts et al, 1985). More recently, Alto (1986) has reported on the use of a wood chip and cryo-anchor solution for remedial measures on a 11° to 17° slope of thaw unstable silts and clays at MP698 of the Alyeska Pipeline.

<u>Performance Data</u>

Monitoring of thawing slopes has the following basic components:

- visual inspection during line patrol flyover supplemented by on-ground inspection by geotechnical engineers.

- monitoring of instrumentation installed during construction. Subsequent site investigation as required is planned, and a program of insitu shear vane testing was undertaken in the fall of 1985.

Visual inspection

Weekly line patrol flights along the entire route by IPL maintenance personnel forms the main visual inspection. During the summer months there have been frequent aerial and ground observations by maintenance crews and other interested parties. In 1984 and 1985 there were three summer inspection trips along most of the route by geotechnical engineers involved in the design. Generally, the performance of the slopes has been excellent. Some typical observations on slope performance are summarized below.

Erosion control measures in the form of ditch plugs and diversion berms, constructed with sand bags, were placed at 10 to 50 m spacing on design slopes depending on slope angle and soil type. It was generally observed that this treatment was adequate and in some cases more than adequate. On some slopes, however, the ditch plugs settled and the berm needed rebuilding over the ditch line. On other slopes the berms required some realignment once the actual runoff conditions were observed.

There was considerable erosion on three design slopes and these were related to unsuspected or underestimated concentrated cross flow from off the ROW in two cases, and to inadequate erosion protection for a known runoff condition in the other.

Wherever there was a cross slope, or a design cut on a slope, there was a resultant side slope. The design specifically ruled out restoring or refilling the cut areas. Side cuts were excavated to about 1.5H:1V to 2.5H:1V. Some ice-rich side cuts were left vertical and performed satisfactorily. Most of the side slopes have performed well. There have been three side slope failures, one of which was an unfrozen, saturated slope. This has slumped onto the ROW to some extent but is no present threat to the pipeline. It is expected that it may self-stabilize.

The two other failures were in ice-rich slopes where wood chip insulation was required. Because of the wood chips, the side slopes could not be left as an uncovered vertical cut. The wood chips were placed against the side cuts which were cut close to vertical. The specifications had called for wood chips to extend over and beyond the crest of the side slope. In both cases thaw progressed into the side slope faster than intended and local slumping occurred. On one of the slopes this initiated a skin-flow failure in the sloping active layer behind the side slope, which required considerable restoration work.

The wood chips proved stable in terms of wind and rain erosion. The chips had been "slippery" during placement, being chips of frozen wood.

Upon thawing, however, the wood chips set into a firm, stable cover and there has been no problem with surficial instability of the wood chips themselves.

Instrumentation

The instrumentation installed on potentially thaw unstable slopes was primarily directed towards monitoring of the wood chip insulation mode but in addition some uninsulated slopes were also monitored. Instrumentation used consisted of thermistor strings and piezometers. Settlement plates were installed on some slopes. Thermistors were Atkins units and all thermistors were calibrated at the freezing point. At wood chip insulated slopes a thermistor string was set in the wood chips and an additional string placed in the underlying soil, usually to a depth of 5 m. The wood chip string was arranged such that the one bead was in the air, one at the wood chip surface and usually one at the wood chip/slope interface. Piezometers were primarily the Petur pneumatic type and some standpipes were installed. Pneumatic piezometers were installed at varying depths below the slope surface, usually 2 or 4 m. All site instrumentation is read by hand and there was no facility for automated readings built into the program. Instruments were read 5 times during 1984 and 4 times during 1985 thaw seasons.

As summarized in Table 1, some 26 slopes were instrumented. Figure 1 is a typical site layout for Slope No. 1 at KP 0.36 Bosworth Creek at Norman Wells and Figure 2 is a summary of major observations at this slope. Several trends are apparent in the thermal data at this slope. Wood chip thickness was 1.0 m and during the first thaw season (1984) maximum wood chip temperatures of 37°C were observed and thaw extended about 0.5 m below the wood chips. During the winter of 1984/85 the wood chips froze back completely but there was a thin zone under the wood chips which did not freeze. The subsequent summer there was no evidence of heat generation in the wood chips and a more or less linear temperature gradient was established. Thaw extended to about 1.0 m below the wood chips, based on interpretation of thermistor data.

Heat generation

Heat generation has been experienced in all wood chip layers and a summary of maximum observed temperature versus wood chip thickness is provided in Figure 3. In all cases wood chips placed in the 1983/84 winter construction season showed heat generation similar to the response in Figure 2 following the 1984 summer seasons, but no heat generation the subsequent summer season. Wood chips placed in the 1984/85 winter showed an identical first summer response to those placed during the 1983/84 winter. It is of interest to note that this response is singularily different than the response observed at a wood chip test undertaken at Grande Prairie and previously reported by McRoberts et al (1985). This test site was for 1.1 to 1.2 m of wood chips placed in July 1983. Within days wood chip temperatures reached 42 to 51°C maximum and gradually cooled but did not freeze back the following winter with temperatures of 3 to 4°C sustained in the middle of the layer. During the summer of 1984, wood chips again generated heat to at least 29 to 39°C at the Grande Prairie site.

Wood chip performance to date in the Mackenzie Valley is certainly favourable compared with the test site as maximum heat generation was less and occurred for only one thaw season. However, the reasons for the difference in performance between the test site and right-of-way and the variation of heat generation within the right-of-way piles are not clear. Several speculations are as follows:

- species types may influence heat generation. However in most cases spruce was the major timber source. The Alberta test site was 75% black spruce, 25% tamarack. Some pine and poplar was used in the N.W.T. but no trends were noted based on qualitative records of the type of wood chip and observed heat generation.

- winter placement may affect subsequent fungoidal activity. Placement of wood chips in the summer certainly appears to create a more rapid or different fungoidal action (McRoberts et al 1985).

- the slow rate of thaw in the wood chips due to latent heat effects (in the second thaw season) resulted in a limited amount of fungal activity thus precluding a chain reaction leading to a significant heat buildup.

- maximum heat rise may not have been measured.

Thaw depths

A summary of thaw depths below the wood chip insulative layer, Figure 4, provides data for both the 1984 and 1985 thaw seasons. In some cases thaw has not proceeded through the wood chips. In six slopes thaw has progressed deeper in the second year while in three slopes, the second year thaw has been less. In two slopes there was no noticeable trend. For comparative purposes thaw beneath uninsulated slopes is also shown and it can be seen that wood chips have retarded thaw.

Factors controlling variation of thaw depth below wood chips have been considered. There is no significant correlation noted between thaw depth and wood chip temperature (Figure 5). There certainly is a tendency for thaw to be less in ice-rich slopes reflecting the retarding influence of a higher latent heat of fusion component on thaw rate. The most important factor appears to be ground temperature, see Figure 6 which correlates thaw depth below wood chips and Figure 7 for thaw below ground surface at uninsulated slopes with temperature. One difficulty in such a correlation is determining undisturbed ground temperatures. All thermistor strings were installed after construction and depending upon the progression of construction work, site specific disturbance factors and time of placement of wood chips, a wide range of ground temperatures are possible. The temperature selected for the correlation was the warmest temperature at the 5 m or 10 m depth (depending on string length) during the first thaw season.

Assessment of wood chip performance

Design thaw predictions for the IPL wood chip insulation mode were finalized in 1983. The details of these predictions have been presented by McRoberts et al (1985) and the long term thaw prediction reproduced on Figure 3. These original predictions also indicated that the long term thaw depth was only modestly influenced by heat generation. The performance data indicates, in some instances, greater initial thaw than was predicted.

The major elements of the design thaw predictions were a heat generation function, time dependent properties for water content, thermal conductivity, and heat capacity varying exponentially with time and the initial ground temperature. The heat generation function initiated at 10°C and was terminated at 60°C. Performance data on wood chip temperature rise indicates that this component was realistically modelled, although performance data suggests that fungoidal action, and therefore heat generation, may initiate at 5°C.

The computer simulation for time dependent moisture content and conductivity were conditioned by the rate at which wood chips gained in water content. Laboratory freeze/thaw cycle tests reported by McRoberts et al (1985) suggested that wood chip moisture contents reached stable levels after 7 freeze/thaw cycles. In these tests water contents increased from fresh chip values of 80% to 190%. Samples of 7 year old wood chips were obtained from a pile near Norman Wells left by Canadian Arctic Gas Study Ltd. These samples had a water content for bulk samples in the order of 210%. Based on this data an exponential time decay of thermal properties over 7 years was used in design. During the summer of 1985 wood chip samples were obtained from slope sites along the IPL right-of-way and bulk density and water contents determined. It was found that water contents in these samples had already increased to around 200 to 240% in the first 0.4 m of wood chip depth. These data indicate that the time for deterioration of thermal conductivity was overestimated. Certainly the temperature data suggests a higher thermal conductivity in the wood chips as evidenced by greater heat penetration and faster cooling of the chips in the early winter. Preliminary thermal analyses, not reported here, which used fully deteriorated thermal properties lead to simulations which more closely match observed performance.

The design thermal analyses were executed for an initial permafrost temperature of -1.0°C and a no heat flow boundary condition at computer grid base. In most cases ground temperatures are colder than -1°C and many thaw depths are small, see Figures 6 and 7. Low insitu ground temperatures will offset thermal conductivity effects, but at higher ground temperatures the combined effects can lead to greater than predicted thaw.

It is beyond the scope of this paper to explore the implications of thermal underprediction on overall slope design, however it can be noted that thermal response was only one component of the design analyses. Where thaw is exceeding prediction, the implications of this thaw on stability is less critical as the slopes most affected are not generally ice-rich.

Piezometer readings

The design calculations assumed that fully saturated conditions would be maintained in the thawed zone and depending on the thaw rate, excess pore pressures were accounted for in the design process. A limited amount of pore pressure data has been collected in those slopes where thaw proceeded to the piezometer tip. Interpretation of piezometer readings has been complicated by the observation of piezometric response prior to the indication by the adjacent thermistor string (1 to 3 m offset) that the tip had thawed. Whether this reflects nonuniform thawing, or the measurement of a piezometric response due to unfrozen water content effects is not known. To date, observed pore pressures are generally less than full hydrostatic.

Seismic Considerations

Because the route traversed a seismically active portion of the NWT, seismic considerations were part of the thaw slope design. On October 5, 1985 an earthquake, of M6.6 occurred some 70 km west of the pipeline, see Figure 8, with an epicenter at 62.3°N, 124.3°W.

Seismic design parameters for pipelines down the Mackenzie River Valley were initially recommended by Newmark (1974). In view of the more recent seismological studies in the Canadian North, Newmark's work was reviewed and all known epicenters located as shown on Figure 8. The pattern of epicenters correlated well with a tectonic feature referred to by Douglas (1970) as the Mackenzie Fold Belt. This fold belt relatively closely correlates with the MKZ source zone delineated by Basham et al (1982) as a zone of background seismicity in the western Yukon Cordillera, surrounding the active Richardson Mountains and bounded on the northeast by the Mackenzie River.

It was suggested by Basham et al (1982) that the seismicity in the MKZ region is spatially correlated with areas of most severe geologically mapped faulting. An upper bound magnitude of 6.0 was set for this region, although it was commented that probability of occurrence for the larger events was poorly defined. The previously highest recorded event in the MKZ zone was M5.2.

The existing seismicity for the region, the observation that M4 to M5 events had occurred historically all through the region, and the fact that an M4.7 event occurred on February 20, 1981 just north of Wrigley, N.W.T., lead to the design recommendation that the IPL line be designed for an M5 event in the near field, that is very close to the pipeline.

Newmark's original recommendations were found to be entirely consistent with the ground motions expected in the near field of a M5 event and were therefore adopted, see Table 2. The recommendations provided by Newmark involve the concept of a dual earthquake design, that is, the design maximum event (DME) and the design probable event (DPE) set at 50% of the DME. Newmark considered that a pipeline system should be able to operate and continue operation after a DPE event while the large DME event may produce damage.

The October 5, 1985 event of M6.6 at 70 km from the pipeline would be predicted to impose sustained accelerations in the order of 4 to 6% gravity based on attenuation relationship for western Canada published by Hasagawa et al (1981) or a mean peak acceleration of 5% g based on Donovan and Bornstein (1975). Their motions are essentially equal to the DPE event for the pipeline.

Immediately after the event IPL initiated a line patrol which was followed up by an inspection of slopes by a geotechnical engineer. No damage was observed and the pipeline can be said to have survived, a DPE event - as it was designed to.

Conclusions

To date the performance of the slopes on IPL have been satisfactory, with the exception of minor erosion problems and the stability of side cuts aggravated by non-adherence to design specifications. The performance of the primary mitigative measure, a layer of wood chips, has been satisfactory although in some instances, especially in ice-poor and warmer slopes, thaw has been greater than predicted.

Acknowledgements

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TABLE 1

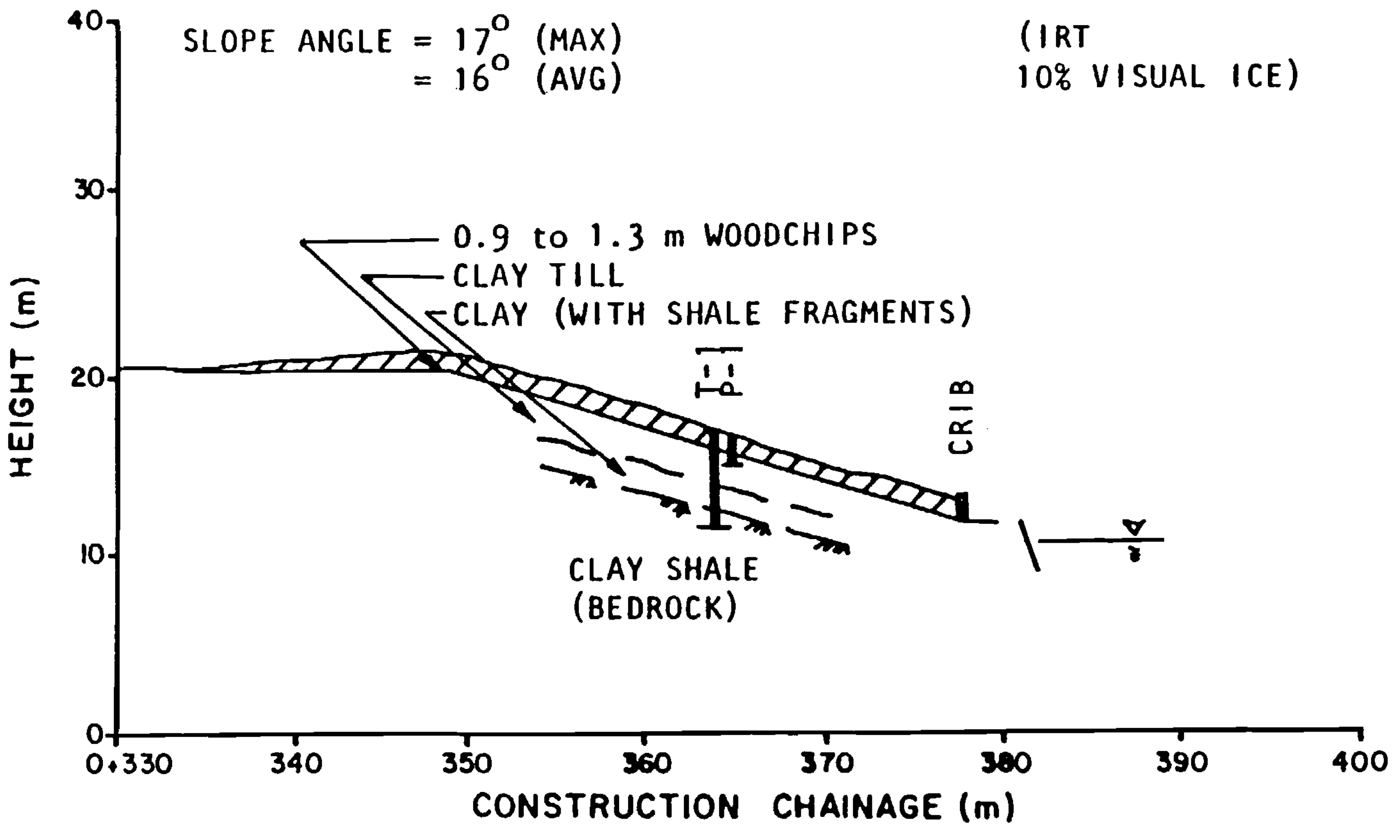
Summary of Slopes Design and Monitoring Program

	No. of Slopes	Design Mode: No Mitigation	Select Backfill Only	Cut	Insulate	Cut & Insulate
Total Catalogued	165	61	33	17	46	8
Slopes Instrumented	26	--	4	5	13	4

TABLE 2

Recommended Seismic Accelerations - IPL

REGION	ACCELERATION	
Fold Belt	12% g	Design Maximum Event
KPO to KP 487	6% g	Design Probable Event

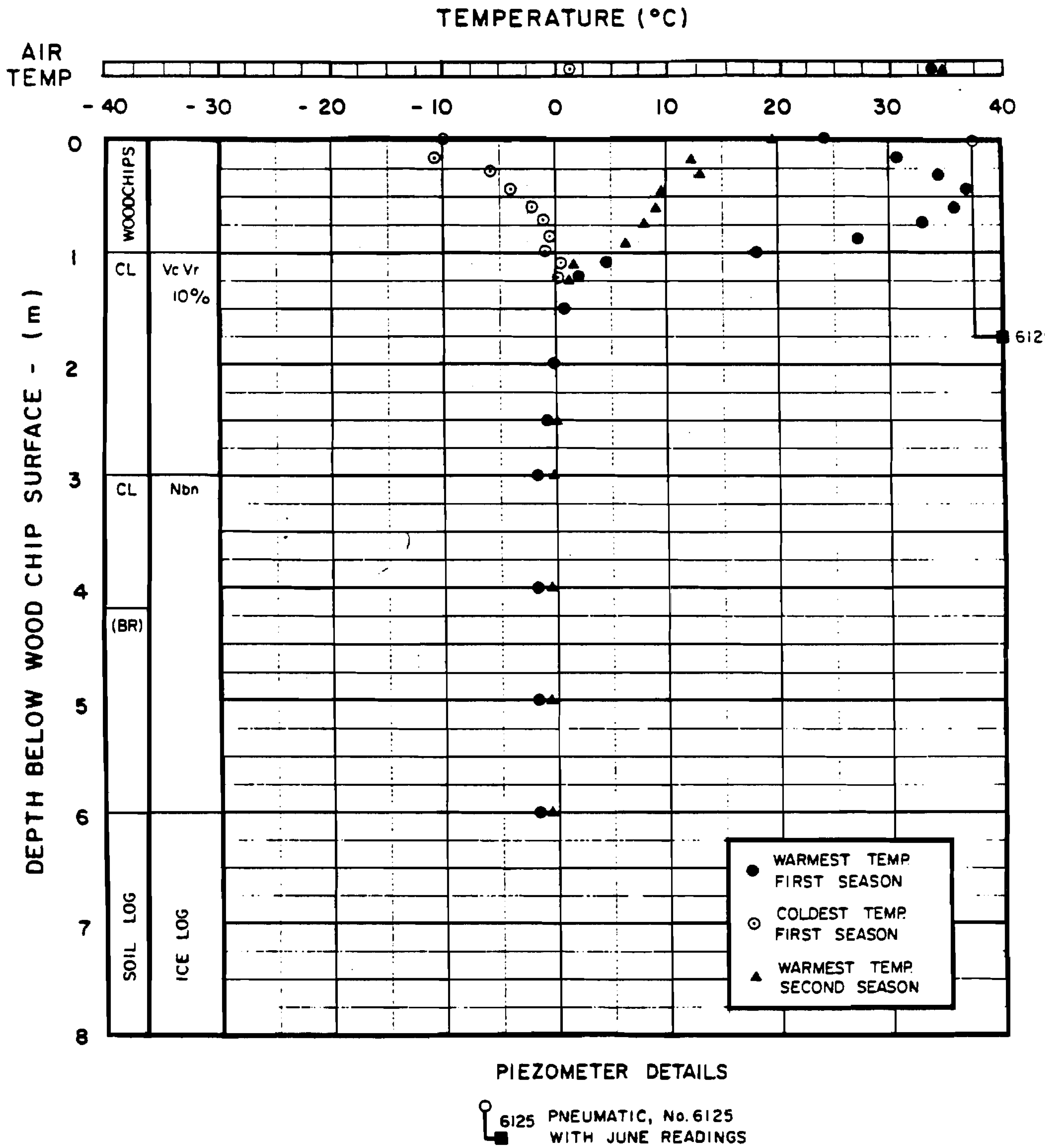


STRATIGRAPHIC SECTION

IPL INSTRUMENTATION PROGRAM
NORMAN WELLS TO ZAMA PIPELINE

SLOPE #1/BOSWORTH N. AT KP 0.36

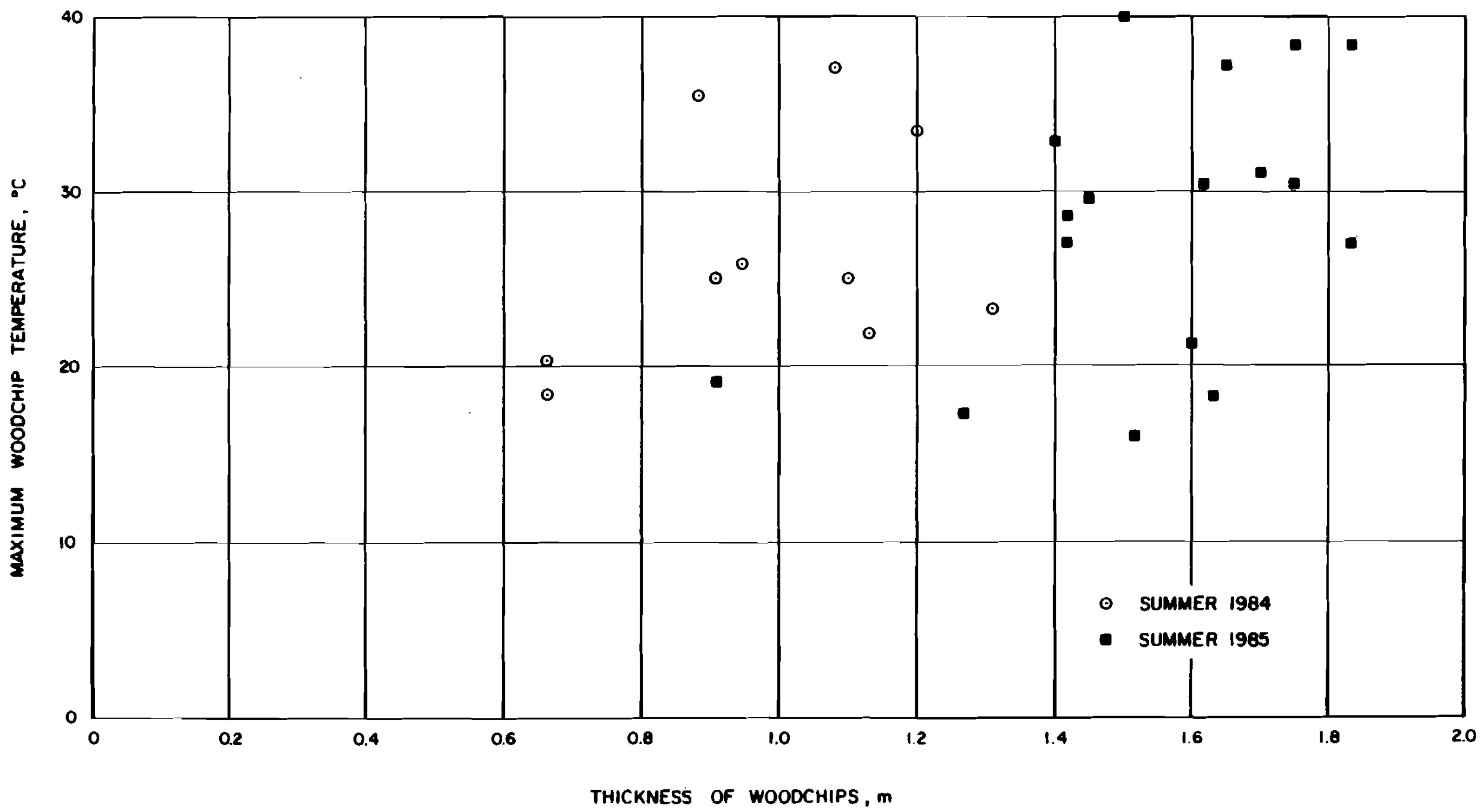
FIGURE 1



OBSERVED TEMPERATURES - 1985

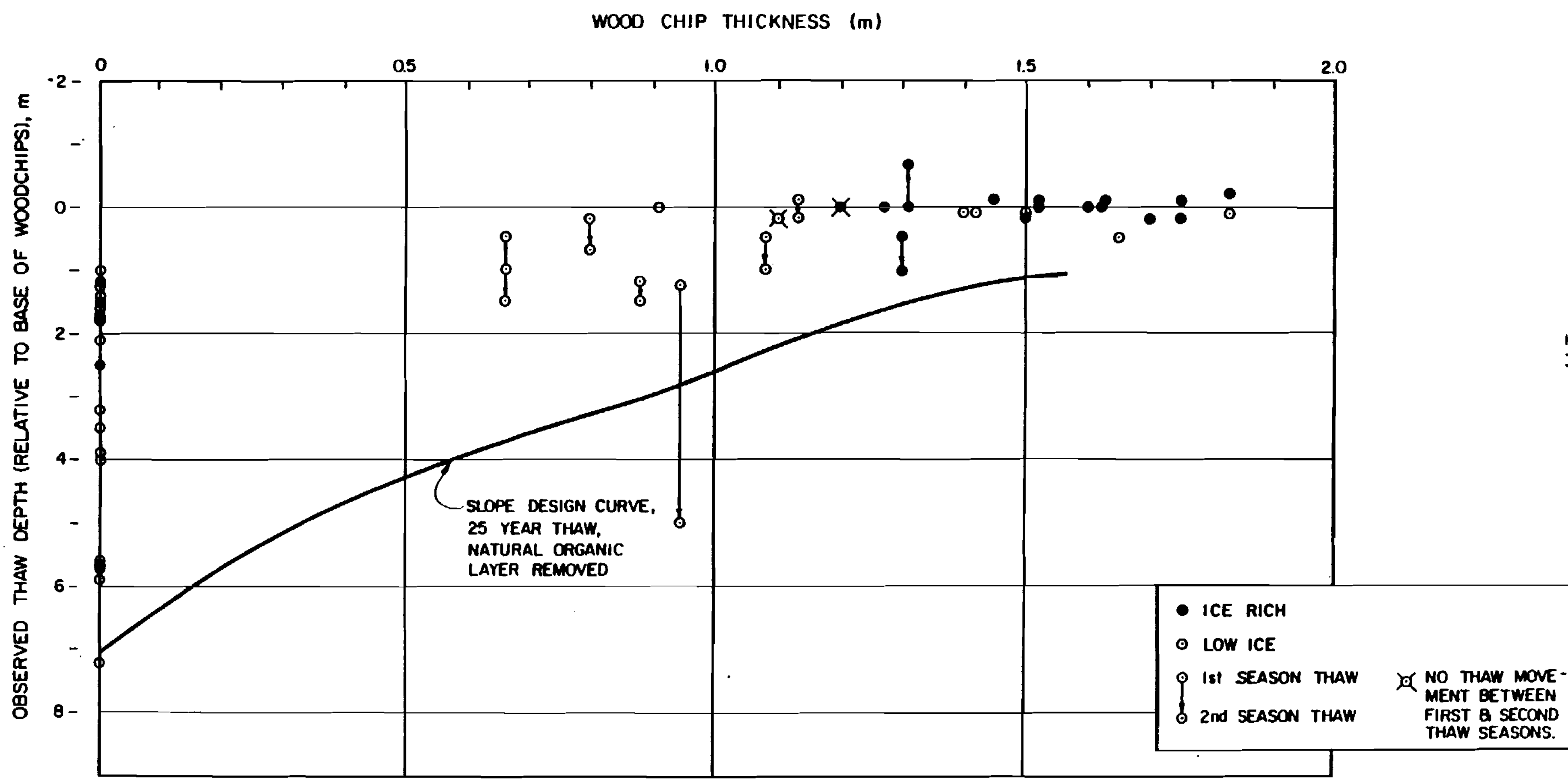
BOSWORTH - NORTH KP - 0 - 36
SLOPE = 1 INSTALLATION T-1

FIGURE 2



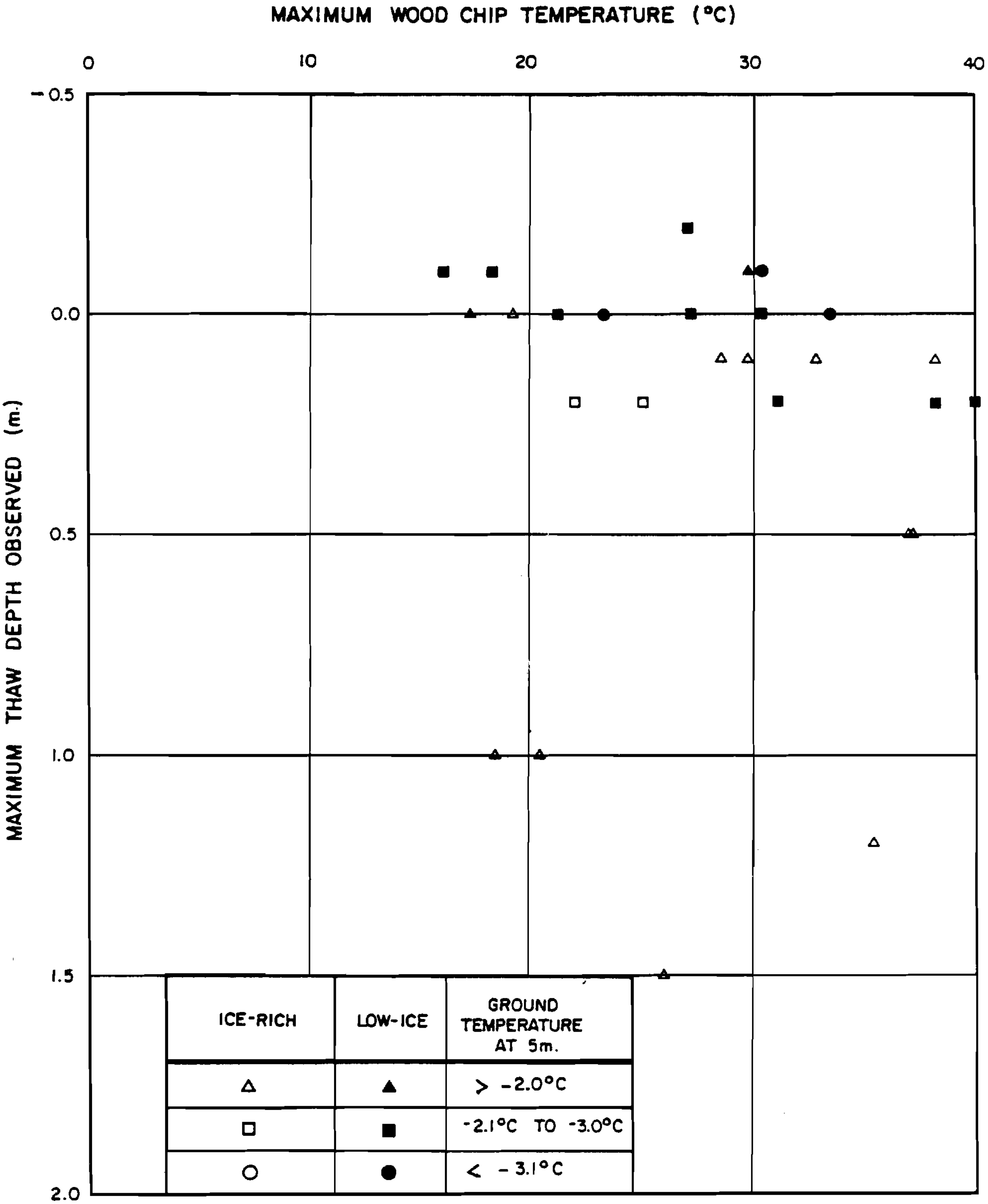
MAXIMUM WOODCHIP TEMPERATURE IN FIRST THAW SEASON SINCE CONSTRUCTION, VERSUS WOODCHIP THICKNESS

FIGURE 3



OBSERVED THAW DEPTH VERSUS THICKNESS OF WOODCHIP INSULATION

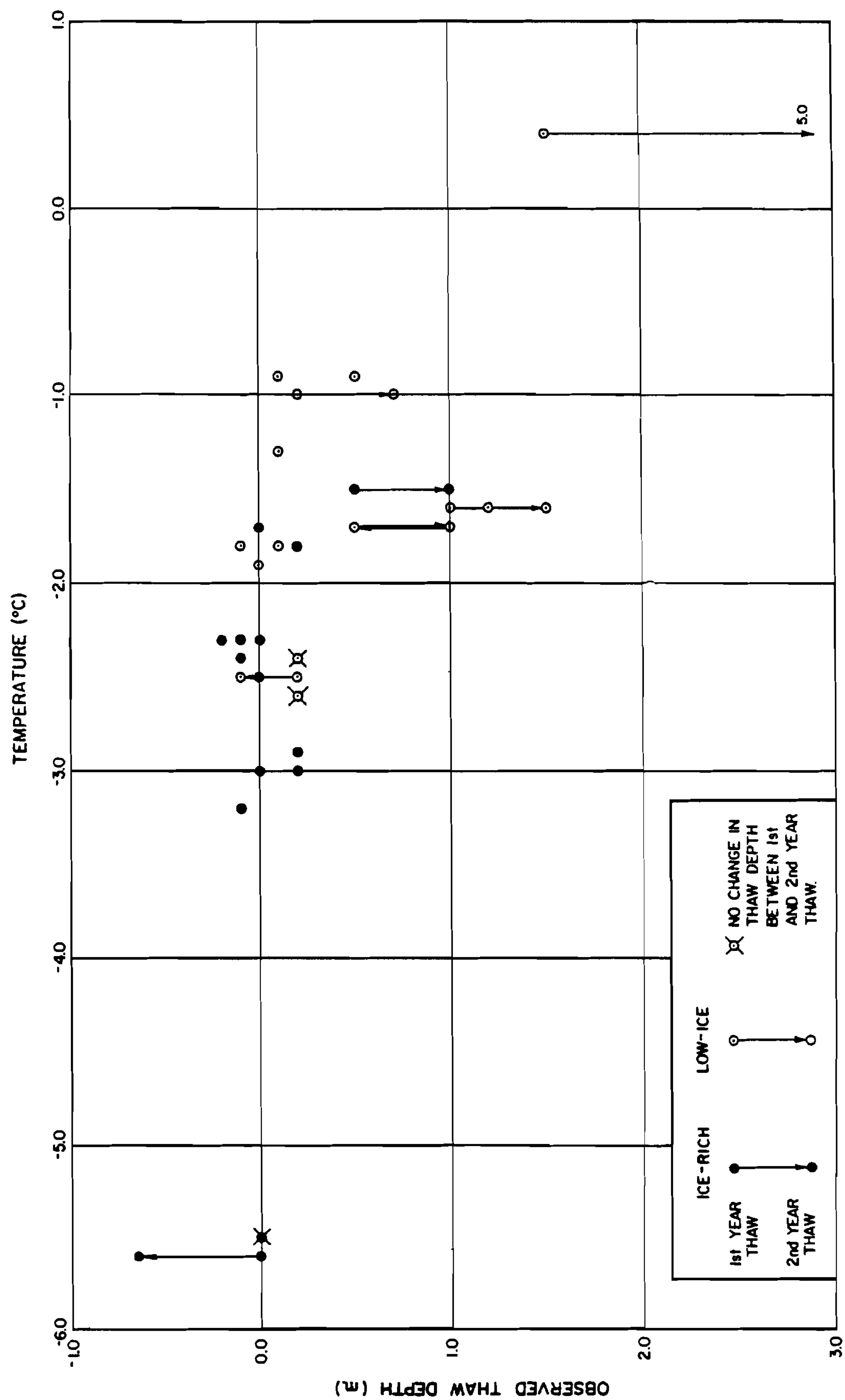
FIGURE 4



MAXIMUM WOOD CHIP TEMPERATURE VERSUS MAXIMUM THAW DEPTH OBSERVED DURING FIRST YEAR OF THAW

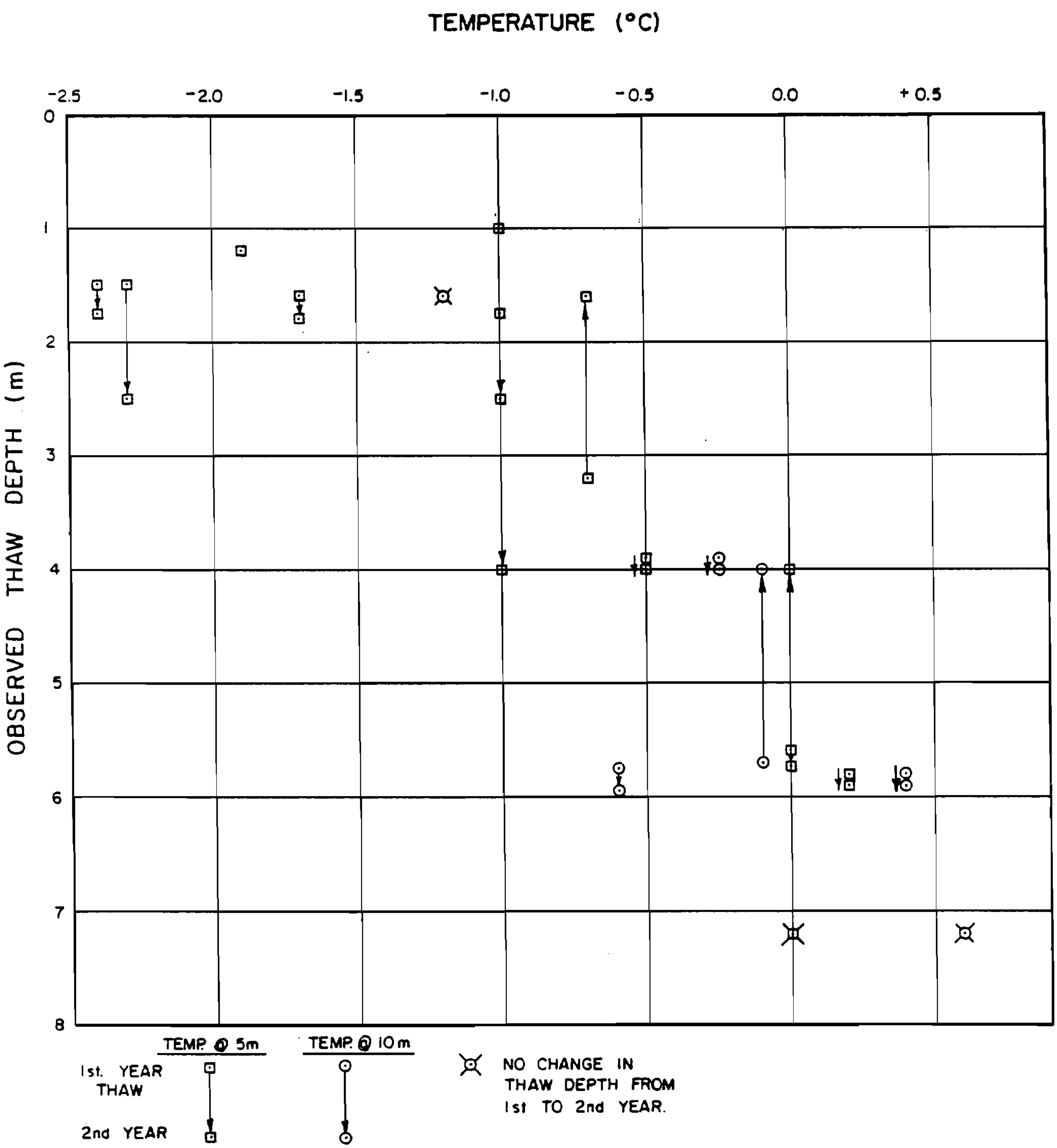
FIGURE 5

HT83-82/C



GROUND TEMPERATURE AT 5m. BELOW GROUND SURFACE FOR OBSERVED THAW DEPTHS ON INSULATED SLOPES.

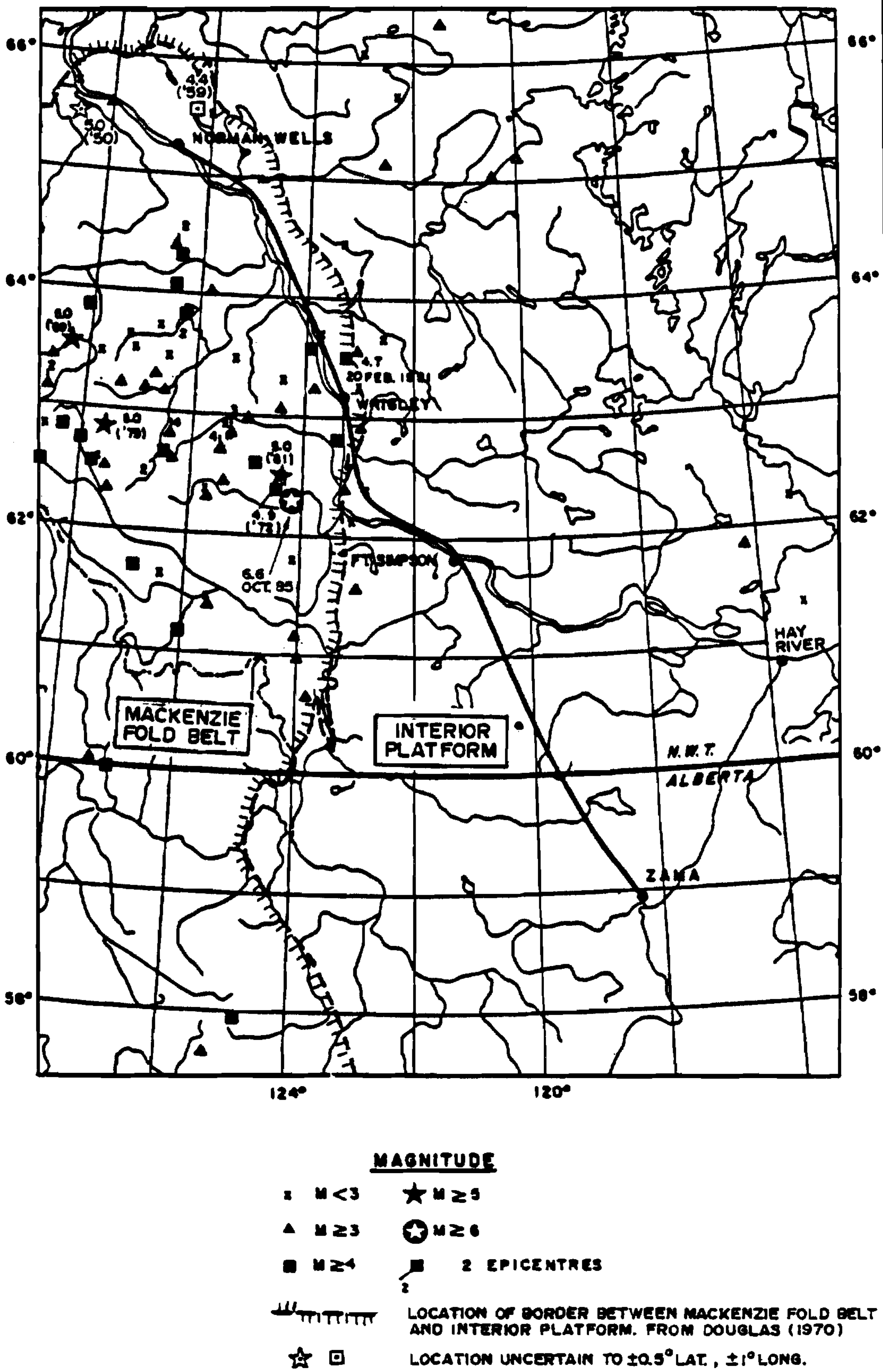
FIGURE 6



GROUND TEMPERATURES AT 5.0m AND 10.0m BELOW GROUND SURFACE FOR OBSERVED THAW DEPTHS ON UNINSULATED SLOPES

FIGURE 7

HT83-82/C



LOCATION OF EARTHQUAKE EPICENTRES

FIGURE 8

THE CANADA-FRANCE BURIED CHILLED PIPELINE EXPERIMENT

P.J. Williams*

The Canada-France Pipeline Ground Freezing experiment is being carried out at Caen in Normandy in a controlled environment facility where a 16 metre length of 273 millimetre diameter pipe is buried, and through which air at -2°C in the first experiment and subsequently at -5° passes. The effects of freezing around the pipe are being closely monitored. The project is funded, on the Canadian side, by Energy, Mines and Resources Canada, who have placed contracts for the work with the Geotechnical Science Laboratories of Carleton University. The EMR funds are part of the Canadian Panel on Energy, Research and Development (PERD) funding which has been mentioned earlier during today's meeting. The Geotechnical Science Laboratories has a number of scientists, graduate students and other personnel involved and we also have working with us Professor Ladanyi and his students at Ecole Polytechnique in Montreal. On the French side the work is funded by two French government institutions, the Laboratoire Central des Ponts et Chaussees and by the Centre National de la Recherche Scientifique. My counterpart in France is Michel Fremond of LCPC, who specializes in numerical modelling. Michael Smith (who should have given this talk today) is a co-principal investigator on the Canadian side.

The controlled environment facility consists essentially of a large garage-like building; the soils were specially prepared and placed in a basal trough. The air temperature in the building is controlled for the experiment at -0.75°C. I would emphasize that this is a scientific study. This pipeline placed in the ground is not a prototype for a field design. The pipe lies half in a silt soil and half in a sandy soil (Figure 1). The two soils were prepared with great care and are extremely uniform. The object of the study as a scientific experiment is to reduce the number of factors influencing the behaviour of the pipe to the fewest possible. The air temperature and the temperature of the gas in the pipe are maintained constant throughout the freezing cycles and thus one avoids the vagaries of test sites under natural conditions, which are exposed to the weather and the ever-present variations of soils. Furthermore we control the water table which is maintained at 30 cm below the pipe. We are seeking the fundamental behaviour of the frozen ground around the pipe.

The deformation of the pipe which has taken place due to the differential frost heave of the two soils is shown in Figure 2. The deformation is known very accurately, being monitored by four methods: optical surveying of rods which are mounted on the pipe and protrude above the ground surface; readings of electrical resistance strain gauges, some forty of which are attached to the pipe; measurements of curvature over small distances using a special instrument devised by Professor Bowes, Carleton University, which spans three of the rods mentioned, to enable very precise measurement of the displacement of the middle rod; and finally a levelling procedure which reveals any

* See List of Registrants (Appendix A)
for affiliation of authors.

inclination of a particular rod from a measurement of the inclination of a plate on the top of the rod.

Some of the other scientific equipment includes time domain reflectometry apparatus, installed in a number of profiles, which shows the unfrozen water content and ice conditions in the freezing annulus and in the unfrozen soil. There are numerous tensiometers and thermistors such that we know the temperature distribution around the pipe. Gloetzl cells which measure total stress in the ground are placed in profiles below and to the side of the pipe. Telescopic heave tubes of the kind originally devised by Professor Mackay of the University of British Columbia, are mounted both through the pipe and to the side of it. These are particularly relevant for the measuring of secondary frost heave, that is, the strain of already frozen ground. The 'telescopes' are progressively extended by heave as the frost line penetrates downwards. The continuation of heave in a layer after a succeeding layer has started to heave is indicative of that strain of the already frozen ground which we call secondary frost heave.

As shown in Figure 2, the section of the pipe in the more frost susceptible soil has been raised more than 20 cm relative to that in the sandy soil. I do not want to get deeply into the question of pipe deformation and stress because this experiment centers on the geotechnical or soil freezing problems, but I should mention that the calculated longitudinal stresses developed in the pipe are substantially above values permitted under the appropriate CSA standard for operating lines. It is recognized that operating pipelines may occasionally be exposed to stresses above those defined by that standard but clearly the question raised by the amount of deformation which occurred in only a little over a year, is: how often and for how long will such deformation occur in a real pipeline?

To turn again to the behaviour of the soil we are particularly interested in the thermodynamics of the behaviour of the frozen ground. The phenomenon of ice lens formation which is responsible for frost heave is well-known. Figure 3 shows a sample from beneath the pipeline and you can see the lens development. As it happens, I have the actual sample with me and it is not thawing either, because it has been preserved by a special new technique developed in France. In effect it is in the frozen state exactly as in the ground but at room temperature. This remarkable technique which enables us to preserve frozen specimens and also to examine them in microscopic sections, is to be brought to Canada very shortly under the France-Canada scientific exchange agreement. We can probably make arrangements for seminars for those wishing to learn of the technique.

It has been known for some years that when soils freeze not all the water turns to ice at least not at temperatures within a few degrees of 0°C. Consequently, there is unfrozen water along with the ice. It is absorbed on the particle surfaces and experiments in various countries have demonstrated quite clearly that, at least at temperatures near to 0°C, the unfrozen water can move. We know from fundamental thermodynamics that water tends to move from warmer ground to freezing ground; that is, it moves to freezing ground because of the suctions developed by freezing soils. Within the frozen soils the unfrozen water has a greater suction, the lower the temperature. Consequently, one can predict that unfrozen water slowly moves towards lower temperatures which would be towards the pipe. This occurs slowly because the permeability of frozen soil is rather small. Furthermore, when the water moves to a colder part of the soil, and freezes there, it produces the secondary heave and at the same time higher pressures are generated than were the case if it froze at or about 0°C. On this very important matter, we have some evidence from our telescopic heave tubes of secondary heave occurring around the pipe. We intend to get more data during the next freezing period. In addition we have been experimenting in Ottawa at the Geotechnical Science Laboratories, on soil shipped from Caen and have measured the internal pressures which are due to the formation of ice within the frozen soil.

There is substantial work being carried out in Japan especially, on frozen ground, as well as in other countries. There is a difference, or range, of opinion between specialists as to the actual amount and rate of secondary frost heave and also at how low a temperature it occurs. Chinese workers, some Russians and indeed Professor Mackay in Canada have suggested that the secondary heave may occur at temperatures as low as -2 or -3°C. Others maintain that the secondary frost heave will be limited to a rather narrow band of soil - that near the frost line and which is not colder than a few tenths of a degree below 0°C. If this is the case, it would, of course, be fortunate, although it would still be very important to know the pressures that secondary heaving is able to develop in this way.

Dr. Ladanyi has been carrying out, together with his students, experimental studies of the creep properties of the frozen ground because it appears that it is the creep resistance of the frozen soil which restrains the expanding ice and thus determines the pressure the ice has, and, according to thermodynamic considerations, the rate of ice accumulation. The behaviour of frozen ground has to be understood in such a way as to meet the laws of thermodynamics as they relate to freezing and the laws of mechanics as evidenced by the creep behaviour of the soil.

We are currently in a period of thawing the ground around the pipe and observing settlement. A further freezing period will commence after that. This seminar is centered on research needs and I have simply, and briefly, described a particular experiment. But to conclude, it seems to me that an experiment like this which is unique in its controlled nature, represents at the very least a good investment in basic knowledge, for the various challenges which major gas pipeline construction will pose for the permafrost regions. A number of countries have controlled environment facilities of a similar kind, for example, Sweden, Japan, Switzerland, and the United States, but the country which has the most frozen ground of all the western, developed nations, Canada, currently has no such facility of its own. One hopes that this is something that will be rectified in the near future.

Note: Numerous reports, reprints, etc. on the project are available from the Geotechnical Science Laboratories, Carleton University, Ottawa, K1S 5B6.

Research Needs: A University Viewpoint

Quite generally I think that the crucial research needs can be grouped into three of four main categories: Firstly the material itself, frozen ground. A pipeline specialist with some experience of frozen ground described it to me as like a "living material". The moduli and coefficients that engineers use and which are normally available to them in table books are enormously variable and little understood in the case of frozen ground. The material properties are highly temperature-dependent and history-dependent and the behaviour of frozen ground shows remarkable feedbacks. The creep properties of frozen ground are both controlled by, and are the result of, the thermodynamics of phase change, that is, of freezing.

These scientific complexities are made worse by the inherent variability of natural soil materials and this is another area of needed research. It extends to the variability of the ground surface, its vegetation, relief and so on which control the thermal conditions in the ground. Indeed this leads to a third very important field - that relating to microclimate and changing climates. Clearly thermal behaviour is at the root of the special challenges for northern pipelines. Overlooking the abundant evidence from elsewhere, of small changes of climate, would appear risky indeed for such thermally sensitive foundations as those for northern pipelines.

A somewhat different but equally important "research" concerns the knowledge gained in other countries. There is a limited number of specialists worldwide in this field of frozen ground research and it is especially important that we do not overlook what is already known, or what others may be finding that, with our limited resources, we have not been able to consider.

You may note that I have not said anything about the actual pipelines. This is because, demonstrably, there is a great deal of expertise about pipelines in Canada and the world. What is lacking is knowledge about how the cold northern environment will affect pipelines.

Where is the role of universities? Firstly, universities are places where there is a great diversity of expertise. Geologists, chemists, engineers, geographers may all consult with each other with ease, linguists too are available and the analysis of their translations is greatly aided by their specialist colleagues. Secondly, universities are especially appropriate places for "basic" research. There is often confusion about what "basic" research is. It gives that highly significant knowledge which is the foundation from which we can define and attack the practical problems as they occur.

Our knowledge of the strength and creep behaviour derives from the work of glaciologists during the 30's and 40's and on, mainly in the universities. It was Kersten at the University of Minnesota who first produced values for thermal properties of frozen soils still widely used by engineers. Professor Lovell in Civil Engineering at Purdue was perhaps the first outside the Soviet Union to demonstrate clearly the existence of the unfrozen water content of frozen soils, which is so basic to understanding their behaviour. Our knowledge of the thermodynamics of frozen ground derives from the studies of soil physicists and agronomists, especially at the University of California some 30 or 40 years ago. Our knowledge today of climatic change is based on the studies over several decades of almost exclusively, university scientists.

A knowledge of surface conditions, geomorphology and surface geology has largely derived from university workers - a tradition carried on by, for example, Hugh French. The extensive field studies of Ross Mackay have done much to tell us how the northern environment behaves. Universities of course are not the only institutions that have carried out basic research and the role of the Geological Survey for example in Northern studies has been great; nor should one forget the major role played by the National Research Council's Division of Building Research, especially during the early days when few others were looking at geotechnical problems. In recent years the need for Canadian trained students and more obvious research needs have led to a relatively much greater involvement by universities.

Another role of universities, of course, is training and here I refer to education at the highest level, making sure that practitioners are up to date in their subjects and well placed to tackle non-conventional problems.

Universities have weaknesses too. Currently the main one is a shortage of funding which in many respects is a "political" difficulty. Governments fund universities mainly for their basic education role of teaching people how to read and write - and rightly so, but in times of restraint what are seen as less immediate problems, basic research and other 'academic' studies, are left to others to fund. There seems to be a trend in western countries for governments to at least contain their expenditures on science and technology and (also perhaps rightly) to expect industry to play a larger role.

Some perceive a different weakness - that universities are hot beds of radicalism, environmentalism and other 'isms'. Apart from the fact that such generalization is ridiculous, it is worth remembering that it is in the nature of the university that opinion be expressed freely and openly. A more certain weakness relates to the status of universities in Canada and indeed the status of research and advanced technology. In general, there is not a well-established association of university research with industrial needs. Canada has the most frozen ground of any of the western nations. Yet, as I pointed out earlier, other nations have invested considerably more in research facilities for frozen ground studies. It may be worth noting that our pipeline ground freezing project has attracted considerable newspaper publicity in France, with large articles in major papers. French oil and gas industry people have taken substantial interest in the work and have donated materials and advice. We have not been surprised by the modest, perhaps reluctant, interest shown by Canadian industry - but rather we have been struck by how such a project, which clearly relates to permafrost, has excited so much interest in France.

Let me conclude by saying that, at least for my own group, the Geotechnical Science Laboratories at Carleton, we are extremely keen to pursue our research in close collaboration with Canadian industry and we at Carleton do not intend to be accused later of having been unwilling to address 'real' practical problems in our scientific studies.

Successful hydrocarbon development in the North of our country must not be seen too narrowly as a matter of petroleum engineering. The northern environment, if not properly recognized, understood and engineered for, constitutes a giant threat to such development.

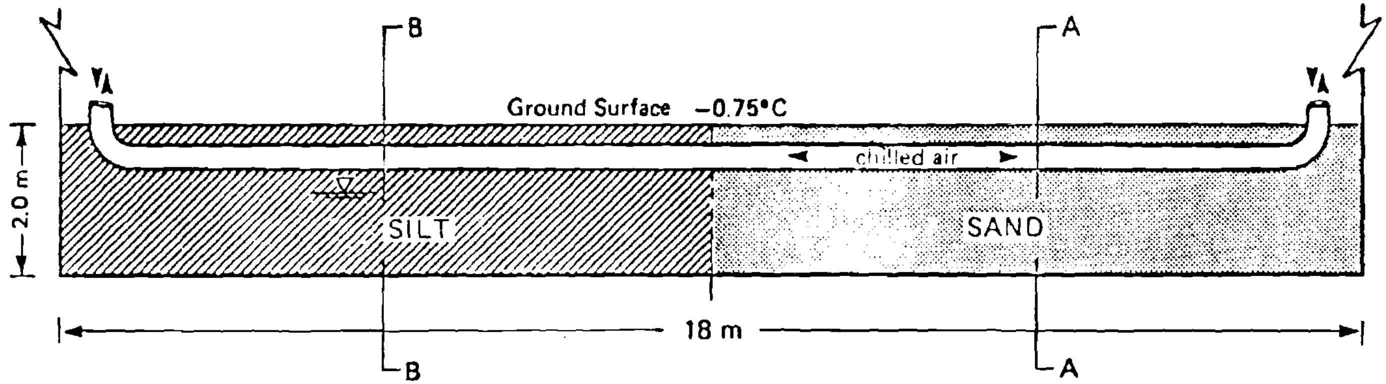


Figure 1. Arrangement of Pipe in Facility

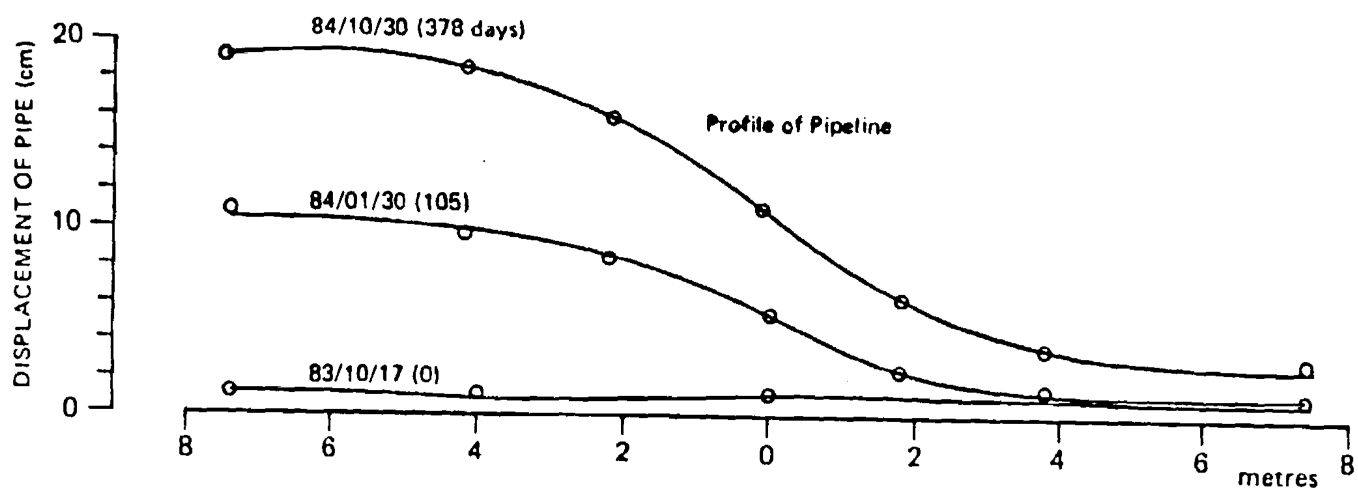


Figure 2. Progressive Deformation of Pipe by Frost Heave

Figure 3. Sample of soil showing ice lensing responsible for deformation of pipe. This sample is permanently preserved and the photo taken at room temperature. Knowledge of the lens and soil structure is important for interpretation of the soil thermodynamic and mechanical behaviour.

PIPELINES IN PERMAFROST

SUMMARY OF DISCUSSION AND RESEARCH NEEDS

Most of the papers presented at this Workshop were slanted towards design, construction and operating (engineering) aspects of oil and gas pipelines and covered the "state-of-the-art" for this topic. Some papers included comments on areas in which more information was needed. Performance monitoring programs on operating pipelines and field experiments associated with yet-to-be-built pipelines, were noted as being necessary and useful sources of important information. In general, industry representatives implied that much of the information is available now and many of the research and analytical techniques required to successfully design, construct and operate pipelines in permafrost, are in place. It was noted that much of the information obtained from field, laboratory and office studies by industry is proprietary and not available for evaluation or use by others.

During the final session, however, which consisted of presentations by members of a panel and open discussion from the floor, some research needs and priorities emerged. Most of the concerns expressed in the papers and the discussion dealt with problems related to slope stability, frost heave and thaw settlement. Dr. Williams' comments on the role of universities and research needs are given in an addendum to his paper.

It was generally agreed that there was a need for on-going field and laboratory research, even and particularly during lulls in northern resource development activities, to provide a basis for environmentally sound designs. Some studies that were considered important and that should be continued are noted below:

- basic frost heave testing of different types of soils - at different pressures to increase the existing data base and compare the variability in frost heave properties,

- pipe/soil interaction with respect to both thaw settlement and frost heave; magnitude of stresses and strains induced

- compare with present allowable values - what is acceptable,

- effect of cycling temperatures on frost heave and/or the effects of cyclic heave on pipelines (cycling of temperature may be proposed as a method of controlling frost heave in future projects),

- influence (relationship) of degree of saturation on thaw strains,

- effect of soil arching on thaw settlement/pipeline interaction.

A good deal of discussion centered around somewhat opposing views concerning stresses and strains in the pipe - how were they induced, their magnitude, rates at which frost heave (and thaw settlement) occurred and over what distance, effect of secondary heave - if any, magnitude and influence of temperature differentials in a pipeline, etc. Views expressed ranged from the lack of and need for information to the suggestion that the results of some studies seemed to indicate that present designs were overly conservative. Some thought should be given as to how the experience gained (from the monitoring program) on the Norman Wells pipeline could be usefully extrapolated to future larger pipelines proposed for more northerly areas in Canada.

It was the consensus that another workshop on pipelines in permafrost would be useful, possibly within the next year or two. At such a meeting the views of active researchers, as well as industry, would be presented. It was suggested that representatives from regulatory bodies must also be present.

APPENDIX "A"

ATTENDANCE LIST

WORKSHOPS ON SUBSEA PERMAFROST AND PIPELINES IN PERMAFROST

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