GEOLOGICAL SURVEY OF CANADA

OPEN FILE 3773

Monograph on Norman Wells Pipeline geotechnical design and performance

AGRA Earth & Environmental Limited
and
Nixon Geotech Limited

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Monograph on Norman Wells Pipeline geotechnical design and performance

Submitted To:
Department of Natural Resources Canada
Ottawa, Ontario

Submitted By:
AGRA Earth & Environmental Limited
and Nixon Geotech Limited
Calgary, Alberta

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Background to Monograph

Following a meeting in Calgary on January 9, 1997, Ms. Margo Burgess of the Geological Survey of Canada, Natural Resources Canada (NRCan/GSC) requested Nixon Geotech to provide a proposal for a monograph documenting the design, operation and performance of the Norman Wells pipeline from a geotechnical and geoscience perspective. The pipeline is owned and operated by Enbridge Pipelines (NW) Inc. (Enbridge) (formerly Interprovincial Pipe Line (NW) Ltd.).

NRCan/GSC has been, and continues to be a key participant in a government research and monitoring program on permafrost and terrain, through the Permafrost Terrain Research and Monitoring group (known as PTRM), since the initial stages of pipeline construction. The government program was designed to assess the impact on, or interactions with, the terrain and to ensure that lessons learned would be documented and could be applied to future northern pipeline projects.

The relevant history and background data for the geotechnical and geoscience aspects of the Norman Wells pipeline were gathered and collated, according to an approved table of contents. A key element of the presentation is a series of tables or matrix charts contained in the Executive Summary, designed to provide an overall impression of the important lessons learned from the design, construction, and operation of this pipeline.

The project was carried out from February to the end of March, 1997, with revisions made in May and June, 1998 and additions in September 1998 to March 1999. The monograph was prepared jointly by Nixon Geotech Ltd. and AGRA Earth & Environmental Limited, who were a major subcontractor for this project. Dr. Derick Nixon and Mr. Alan Hanna were with the same company (formerly Hardy Associates (1978) Limited) during the design of the project, and have been involved with the follow through from design to construction and operation of the pipeline. Dr. Jim Oswell, who is currently handling much of the ongoing monitoring and geotechnical assessment for the pipeline company, was a principal contributor to the monograph.

The original idea for a review came from the PTRM to document over 15 years of experience on the pipeline. The Department of Indian Affairs and Northern Development transferred funds to the Terrain Science Division (TSD) of NRCan/GSC in 1997 to initiate the work under the Scientific Authority of Ms. Margo Burgess. The TSD has provided many of the references for data gathering and monitoring that has been undertaken by the government over the past 14 years, as well as data included in this monograph. The TSD has also supplied additional funding (through the federal Panel on Energy Research and Development).

The contracts covering this project are Public Works and Government Services # 23397-6-0327/001/SS and # 23397-8-0055/001/SS, administered by Ms. Elaine Metcalfe.

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GLOSSARY

Active Layer The top layer of the ground above the permafrost table that thaws annually. Active layer thickness may vary from a few centimetres to several metres.

Differential Settlement The downward displacement of one point in a structure (such as a pipeline), relative to another, resulting from a localized loss of soil support. Settlements of this nature are of particular concern because of the stresses that can be induced in the structure.

Frost Bulb A bulb-shaped intrusion of the permafrost table into the active layer resulting from localized chilling. The frost bulb will grow until a new thermal equilibrium is reached.

Frost Heave Certain types of soils, under certain conditions of water content and in-situ density exhibit an increase in volume on the soil mass when frozen. This volume increase can impart upard displacement to the ground surface and structures that are shallow buried.

Geophysics Techniques that use electro-magnetic waves to provide subsurface information in a non intrusive manner. Types of geophysical methods include Ground Penetrating Radar, Acoustic methods and Seismic methods.

GEOPIG A device that is inserted into the pipeline to measure the physical characterics of the pipeline. The instrument measures pipe curvature, pipe diameter, and records positions of welds, valves and other features within the pipe. Position is tracked by a Global Positioning System. The data may be used to calculate pipe profile (vertical and horizontal), bending strains, and ovality of the pipe relative to a baseline reading.

Overbend A bend in the pipeline to permit a change in vertical alignment. The overbend is located at the top of the slope, where the pipe alignment changes from a nominally horizontal alignment to a sloping alignment. See also sagbend.

Permafrost A thermal condition of earth materials when their temperatures remain continuously below 0 °C for more than one year.

Piezometer A device to measure porewater pressures in the ground. Two systems are typically used: vibrating wire and pneumatic. Most of the piezometers on the Enbridge pipeline system are pneumatic piezometers.

Sagbend A bend in the pipeline to permit a change in vertical alignment. The sagbend is located at the bottom of the slope, where the pipe returns to a nominally horizontal alignment. See also overbend.

Slope Indicator A device that is installed in the ground to measure mass soil movements. The device consists of a tube that is inserted into a borehole and then grouted into place. The tube should be installed to sufficient depth that the base is well below the suspected zone of movement. A probe is inserted into the tube that measures the deflection of the tube at each depth increment. By integrating the movements over the depth, and comparing the deflections to the baseline readings, movements with time can be determined.

Strain Gauges A device used to measure local strain. Typically the devices are welded or glued directly to the pipe. They measure relative changes in strain at that location.

Thaw Bulb A body of perennially thawed ground caused by localized heat transfer from a warm object at or near the surface. The thaw bulb will grow in size until a new thermal equilibrium is established.

Thaw Sensitive Soils Soils that experience a reduction in volume on thawing. This is usually due to the melting of ice within the soil matrix.

Thaw Settlement A settlement of the ground surface in certain types of soils that results from the melting of excess ice in the soil mass and the consolidation of the thawed soil strata.

Thermal Fence A series of thermistor cables that were installed perpendicular to the pipeline, across the right-of-way to measure the thermal changes in the ground.

Thermistor A device, based on electrical resistance that can be correlated to temperature. A thermistor string is a cable that contains a number of thermistor bead at different positions. Thus, once installed, the individual thermistor beads can be read to give temperature data at different depths.

1.0 EXECUTIVE SUMMARY

This monograph has been prepared to highlight some of the history, lessons learned and unresolved issues resulting from the design, construction and operation of the Norman Wells pipeline project. Being the first fully buried trunk line in Arctic terrain in North America, much can and will be learned from its operation and performance history. The pipeline is owned and operated by Enbridge Pipelines (NW) Inc. (Enbridge) (formerly Interprovincial Pipe Line (NW) Ltd.).

It is important to note that this monograph is written from the perspective of design and geotechnical engineering, and does not cover the environmental, permitting, regulatory and political aspects of the project. It is intended to record for future engineering designers and regulators, some of the differences between expected and actual performance.

This review draws on a great body of data and experience collected over the years by the pipeline operator, their consultants, regulators and government agencies. The monograph can only hope to highlight some of the more important issues, and refer the reader to more detailed references and publications.

The authors have chosen to present the executive summary to this monograph in table form, as being the most efficient and visual method of portraying the more important lessons learned.

Part A of the enclosed tables summarizes the project philosophy, the approach to the design of the pipeline, and lessons learned.

Part B lists and comments on the adequacy of some of the available data bases that were employed in design. The Norman Wells pipeline project, as well as carrying out many detailed field surveys and investigations prior to construction, was also the beneficiary of many valuable data bases from prior projects that were planned, but never constructed. It is therefore important to recognize the contributions from these earlier projects.

The comparisons between design mitigations (the expected impact and the measures selected to minimize such impact) and the actual or observed impacts are summarized on Part C of the enclosed tables. The summary also addresses the effectiveness of the measures employed to monitor the impacts. The issues covered include pipe temperatures, thaw settlement and slope stability. This table also provides a thumbnail sketch of unresolved or outstanding issues that remain to be addressed.

Although a document of this nature tends to highlight the deviations from the expected design performance and problems encountered in the pipeline operation, it should be stated at the outset that the project has generally performed according to expectations. Crude oil averaging about 5000 m^3/day (33,000 bbl/day) has been transported to southern markets over a 14 year period without significant interruption. Environmental issues were identified early and have been monitored and dealt with by the company in a manner acceptable to regulatory authorities. These facts should be borne in mind while reviewing some the more challenging and interesting details outlined in the following.

NORMAN WELLS TO ZAMA PIPELINE PROJECT
PART A: Project Philosophy, Approach

Concern/Issue	Philosophy, Approach	Lessons Learned
Ambient Pipeline	NW crude oil is light, wax was removed at process facility; feasible to operate as ambient pipeline; therefore much less impact on majority of permafrost. Required chilling at Norman Wells to reduce natural oil temperature to closer to the mean ground temperature.	Pipe operating temperatures warmed up somewhat faster than expected to some extent - probably related to warmer than normal climate conditions in the first years; warmer summer Norman Wells oil temperatures since 1993 pose no significant concern.
Routing	Selected overland route on existing cut lines where practical - CNT line, and seismic lines. Many slopes were on undisturbed alignments.	Thaw settlement on the right-of-way is generally at or somewhat less than originally estimated.
Construction Schedule	Winter construction selected to minimize terrain disturbance; winter construction also required because of numerous swampy areas. Wheel ditching planned in all spreads.	Worked well; good progress made - 3 spread seasons instead of the 4 that were planned north of Ft. Simpson. Little snow for snow roads - most of travel on bare ground without significant impact. Wheel ditching very successful, except in very bouldery soil.
Reclamation	Intended to implement a rapid revegetation program combined with physical erosion control measures in highly erodible areas, such as steep slopes.	Surface erosion was quite high at selected locations, particularly in the early years. Erosion decreasing with time.
Monitoring	Pipe - temperatures monitored to know actual ambient temperatures and net relationship between pipe and adjacent ground temperatures; Slopes - monitoring of temperatures and pore pressures considered integral part of design because of potential unknowns and first extensive use of wood chips to mitigate rate of thaw.	Very successful; majority of instrumentation in good condition 12 years later. Bears and fires have been the biggest problem. Should have insisted on some instrumentation closer to the pipe; some new instrumentation now required where thaw exceeds initial depths (typically 5 m). Lack of good benchmarks were a problem for measuring pipe settlement. Several survey benchmarks were installed.
O&M	Weekly flyovers and deal with problems as they arise. Attend to severe erosion with helicopter support in early years as required. For less severe erosion or other problems, plan winter remediation.	Potential of a loose backfill mound. Not any threat to pipeline integrity, however, significant remediation effort in early years.
Contingency Plans	The operator set in place emergency response/oil spill plans. Caches of response materials were located at selected sites.	Emergency training is ongoing. Operator purchased Rollogon vehicles to provide better land access to the right-of-way in all seasons.

NORMAN WELLS TO ZAMA PIPELINE PROJECT
PART B: Design Input Investigations/Databases

DESIGN INPUT	INVESTIGATIONS/SURVEYS/DATABASES	ADEQUACY/LESSONS LEARNED
Terrain Conditions	Terrain interpretation and mapping	Adequacy: Good enough for scoping field investigation and identifying more sensitive terrain areas. Not reviewed or upgraded following field drilling.
	Drilling Investigations	Adequacy: Quite adequate, primarily due to extensive existing Arctic Gas and Mackenzie Highway borehole data. Otherwise the project would have required more extensive drilling.
	Previous Investigations	Adequacy: Valuable data base provided much useful information (and drilling program experience) and saved NWPL the need for too much drilling.
	Laboratory Testing	Adequacy: Again existing database was valuable for thaw settlement parameters. Specific strength data for thawing and unfrozen slopes was adequate.
	Geophysical Surveys	Adequacy: Electromagnetic surveys conducted for frozen/unfrozen interfaces were very useful. Lessons Learned: More interfaces than expected and hence entire pipeline was designed for thaw settlement.
River Crossings	Riverbed Surveys	Adequacy: Adequate cross-section and thalweg surveys conducted on all significant rivers and creeks. Generally adequate information for most significant rivers and creeks. Would normally have air photos in less remote areas. Only one case where bank erosion uncovered the pipe (Ochre South).
	Storm Runoff Predictions	Adequacy: Generally adequate. Some major summer storm events caused extreme flows (Hodgson River discharges exceeded the design 1:100 year predictions; Ochre River discharge was somewhat less than the 1:100 year prediction.) Lessons Learned: Summer storms can be very localized and intense.
Atmospheric Conditions	Meteorological Data	Adequacy: Geothermal analyses conducted based on the Arctic Gas climatic subdivisions for the Mackenzie Valley (Regions 14, 15 and 16). Considered adequate, however, the first five years of pipeline operation experienced warmer than "normal" temperatures. Lessons Learned: Long term records probably adequate for long term design conditions; too early to determine if climate warming will be an issue - nothing significant noted to date.

NORMAN WELLS TO ZAMA PIPELINE PROJECT
PART C: Design Mitigations and Actual Impacts

		A	B	C	D	E	F	G	H	I
	Impact Issue	Expected/Predicted Impact	Mitigation Applied	Monitoring Approach	Actual Impact	Effectiveness of:		Operational Improvements	Lessons Learned	Outstanding Issues
						Mitigation	Monitoring			
1	Pipe/Ground Thermal									
1a	Ground temperature (except slopes)	Warming/thawing resulting from ROW clearing	Nothing specific on overland sections except to minimize surface disturbance	INAC/NRCan (GSC) thermal fences	Temperatures were generally within the 25 years design limits; thawing rates were higher in organic terrain.	Not applicable	Good	None required	Generally within predictions. Site specific response varied widely according to soil conditions.	None
1b	Pipe temperature (ambient line)	Expected to respond to adjacent ground temperatures and therefore little due to pipe expected; immediately downstream of Pump Stations 2 and 3 expected to be warmer. Would input heat to ground for about 50km before running at ambient; chilled oil from Pump Station 1 would actually cool the ground for some 50 km (see details on subsequent pipe temp. excursions, below)	None	Numerous thermistors attached to outside of pipe coating at time of construction. Other thermistors subsequently placed "beside" the pipe by INAC/NRCan (GSC).	Pipe temperatures warmed to predicted range quicker than expected due in large part to disturbed trench and intial chilling problems. Also, in 1985 to 1991 mean annual air temperatures were about 1 to 2°C above normal. Subsequent excursions addressed below.	Not applicable	Good	See below on changes	Nothing specific: see comments on slope section.	None
1c	Bosworth slopes	The natural temperature of the crude oil, combined with the temperature increase across the pump station, would result in 10 to 20°C oil temperatures being input to the pipeline. The stability of the slopes at Bosworth Creek were considered potentially more sensitive because the creek served as the town's water supply at one time.	Decision made that risk of warming the slope too much should be reduced by chilling the oil such that the oil inlet temperature would not exceed -1°C.	Enbridge control centre would given alarm when inlet to Pump Station 1 was warmer than -4°C	For most part temperatures controlled, with only minor, short term excursions, from start to 1992. See 1d below for post 1992.	Reduced thaw in vicinity of pipe (frost bulb developed around the pipe in early years) See 1d for post 1992.	Good, based on visual reviews, and instrumentation installed on slope in later years close to the pipe.	See below for changes	Nothing specific because chilling the oil had not been a technical design requirement	See below
1d	Warm summer pipe temps.	In 1992, Imperial Oil requested relaxation of the drilling requirement during the warmest summer months, when chilling was most difficult and expensive. (Prior to this some random temperature excursions had been allowed for short durations - less than a day). As of August 1993, summer temperatures could go to 12°C for 2 months, however, for 8 months, the inlet temperature would be cooled to -4°C, for a mean annual temperature close to +1°C. In fact, only +10°C allowed by Enbridge and now only +9°C intended for future - with broader shoulders. Close to Norman Wells, a thaw bulb would develop around the pipe in the short warm flow period, however, this would freeze back each winter.	None	Pipe temperature installation to be monitored by operator as prescribed by NEB 1993 order.	Deepening of thaw bulb closer to Norman Wells to about 1.2 m beneath pipe base on a seasonal basis was observed. The effects of seasonal temperature excursions propagate 30-50 km downstream, and then oil temperatures equalize with environment. Seasonal pipe movements of 20 cm at kp 2 have been observed, likely related to these freeze-thaw effects. May accentuate uplift buckling effects at kp 5.2. No negative effects on slope stability observed to date, although thaw bulb increasing at Bosworth and Canyon Creek slopes (These slopes are underlain by relatively shallow bedrock, which controls slope stability).	Not Applicable	Instrumentation adequate; frequency of some readings less than intended	At warmer temperatures (up to +10°C)more wax was entering the pipe and wax buildup required extra scraping pig runs. Latest approach to limit to 9° and with more gradual ramping up and down expected to alleviate problem.	No significant impact on slope stability to date. Important to separate effects of temperature excursions (due to pipe) and warmer summer temperatures (environment)	Determine optimum summer temperatures with respect to waxing problem. Assess impact of excursions, with the much colder than originally intended winter temperatures.
2	Thaw Settlement - ROW Surface	The long term thaw beneath previously cleared right-of-ways estimated to be range from 0.7 m to 1.2 m, for mineral to organic soils.	None on overland (revegetation primarily for aesthetics, erosion control) wood chips on slopes (see (5) below)	INAC-EMR thermal fences at many representative terrain units surveyed by the GSC for settlement since construction.	Estimates of actual right-of-way thaw settlement ranged from 0.05 m to 0.95 m, with an average of 0.53 m for 15 locations after 12 years of the 25 year design life. Settlement in trench greater than adjacent right-of-way and up to 1.5 m in organic terrain.	Not Applicable	Adequate	None	Revegetation excellent - may have reduced actual thaw.	Since thaw still occurring, thaw settlement will continue and must be monitored as long as the pipeline is operating.

		A	B	C	D	E	F	G	H	I
	Impact Issue	Expected/Predicted Impact	Mitigation Applied	Monitoring Approach	Actual Impact	Effectiveness of:		Operational Improvements	Lessons Learned	Outstanding Issues
						Mitigation	Monitoring			
3	Thaw Settlement - Pipe	Due to thaw settlement beneath the right-of-way, the differential settlement of the pipe was predicted in the order of 0.8 m, except in the peat plateaus between Fort Simpson and Zama Lake, where the settlement was predicted to be about 1.2 m.	Pipe wall thickness selected to withstand above differential settlement with a maximum pipe strain of 0.5% (compression)	No instrumentation with respect to pipeline integrity; Gross right-of-way surface differential settlement to be monitored by line patrol; smart internal interial tool (GEOPIG) developed in the late 1980's to measure pipe position in 3-dimensions and permit the calculation of curvatures and strains. GSC conducted level surveys with local benchmarks and probing to the top of pipe at selected sites.	Estimates of actual pipe thaw settlement range from 0.2 m to 1.0 m, with an average of 0.63 m for four locations, based on GSC surveys.	Adequate as long as limiting strain not exceeded; too early for conclusion because thaw settlement will continue.	No means of observing absolute pipe settlement since construction; GEOPIG probably adequate.	None	Absolute thaw settlement data would have provided opportunity to assess the original thaw settlement predictions.	Must continue thaw settlement monitoring and GEOPIG runs as long as pipeline operates.
4	Frost Heave - Pipe									
4a	Sagbends at water crossings	Limited frost heave at sagbends	5.0 cm PU insulation applied to short pipe sections.	Strain gauges were installed on pipe upslope of selected insulated sagbends; GEOPIG monitoring.	No specific large strains detected by GEOPIG.	Appears good, but may not have been required.	Many strain gauges were damaged during clean-up or became unserviceable soon after installation. No conclusive data was obtained.	None	None	None
4b	Near N. Wells (colder pipe)	None	None	GEOPIG profiles and several elevation surveys at Kp 2 and Kp 5.2.	Pipe uplift buckling has occurred at one and perhaps two sites. Frost heave may have been initiating trigger, but uplift buckling has displaced pipe up over 1.1 m.	Mitigation undertaken at Kp 5.2.	Good match between GEOPIG and elevation surveys		Uplift buckling can occur in low density soils near an unfrozen transition.	Monitoring of mitigation at Kp 5.2 is on-going to check on success of remedial work.
5	Stability of Ice-Rich Slopes									
5a	Thaw progression	The thawing stability analyses for the slopes predicted that slopes steeper than 9° in ice-rich clay and 13° in ice-rich till, would have a potential factor of safety less than the targeted 1.5 unless some mitigation was applied. Many slopes were in the range of 12 to 20°, thus requiring insulation and cutting of the slope to a lesser angle. Design Assumption: pipe initially modeled as thermal passive, that is, design did not anticipate thermal impact of pipe on slopes.	The width of the right-of-way was reduced to 13 m on sensitive slopes. For ice-rich clay and ice-rich till slopes, wood chips were placed to reduce the rate and the long term depth of thaw. Thus the pore pressures would be reduced and the thaw stability increased. The thickness of wood chips was increased if the organic mat was significantly disturbed on right-of-way and if right-of-way was cleared wider than 13 m.	Many representative slopes instrumented with thermistor strings in the wood chips and to 5 m into the ground. Physical probing of the thaw bulb was conducted on selected slopes in late fall, beginning in 1992.	On many slopes, the wood chips proved very effective. On about 10% of the insulated slopes, the expected heat generation in the decaying wood chips lasted longer than the majority of slopes (1 - 2 years). Still numerous isolated hot/warm spots, many of which appeared several after start of operation - not a particular concern . On some slopes the amount of thawing after 12 years has exceeded the predicted 25 year thaw depth in part due to the larger than anticipated input from the pipe, and warmer initial ground temperatures.	Generally very good; about 15% of the insulated slopes have been identified for more detailed monitoring and assessment due to excess thaw around the pipe.	Generally good; instrumentation could have been closure to the pipe for detecting the maximum thaw; some could have been deeper. Physical probing has been very informative. GPR (ground penetrating radar) surveys effective in select soils.	None	Avoid harder wood species (aspen/ poplar) and/or rotten wood that most likely contributed most to the extended heat generation. Anticipate more influence from relatively warm pipe temperatures beneath the wood chips.	Majority of insulated slopes are performing as well or better than predicted. However, certain slopes require close monitoring and stability assessment as thaw continues to progress, primarily due to the new warmer pipe inlet temperatures (by NEB order).

		A	B	C	D	E	F	G	H	I
	Impact Issue	Expected/Predicted Impact	Mitigation Applied	Monitoring Approach	Actual Impact	Effectiveness of:		Operational Improvements	Lessons Learned	Outstanding Issues
						Mitigation	Monitoring			
5b	Pore pressures	As the thaw repressed, the water released at the thaw front would not be able to drain efficiently in fine grained ice-rich clay and ice-rich till. Hence, an excess pore pressure would develop at the thaw front, thus reducing the stability of the slope.	Wood chip insulation reduces the rate of thaw and hence reduces the development excess pore pressures, that is, when no hot spots and when away from the influence of the warm pipe.	Piezometers installed on representative number of insulated slopes at time of construction	Generally observed pore pressures less than predicted. However, local areas of concern still exist (eg: Slopes 44, 45, 48B).	The insulation reduced the rate of thaw on all slopes, however, there is additional thaw occurring due to the influence of pipe temperatures.	Piezometers worked well and did detect some excess pore pressures within the thawing zone. However, in several slopes the thaw has progressed deeper than original piezometer installation and deeper installations were required.	In 1994 vertical slotted drains were installed into three slopes, experiencing high pore pressures. Additional piezometers installed deeper and closer to pipe.	Since piezometers need to be at the actual thaw front to observe maximum pore pressures, anticipate need to install new thermistors at later date.	Monitor and assess piezometric pressures in deeper piezometers. Porewater pressures expected to be higher in early years, during initial thawing of near surface soils.
5c	Stability	The stability of the thawing ice-rich slopes is directly related to the depth of thaw, the shape of the thaw bulb and the pore pressures. Recognized that many slopes are underlain by shallow bedrock. Slopes were designed and insulated such that design factor of safety was greater than 1.5.	Cut back and or Insulation to reduce thaw and pore pressures.	As noted above.	Because of deeper thaw than anticipated and some relatively high pore pressures, the estimated factor of safety of about 10 to 15% of the insulated slopes has fallen below the original target of 1.5. A factor of safety of 1.3 would normally be used for unfrozen slopes in southern regions and is considered acceptable when analyzing thawing slopes based on slope specific data. There were portions of approximately 10 slopes in 1992 which indicated safety factors less than 1.3. In a 1996 assessment, there were portions of 6 slopes in this condition.	General good; less effective where pipe temperatures govern, on about one-third of insulated slopes.	See note above concerning thermistor and piezometer locations and depths.	Additional thermistor and piezometer instrumentation added in 1992 and 1997, and 1998 on selected slopes.	Anticipate influence of pipe temperatures under wood chips.	Continued close monitoring and stability assessment on 10% of insulated slopes.
5d	Seismic	The influence of an earthquake event was considered in the design for slope stability. The equivalent of a magnitude 5 event was assumed to occur on the right-of-way, and a ground motion acceleration of 0.12 g was assumed in the slope analyses. A factor of safety of 1.0 was considered acceptable for a seismic event as it was shown that significant displacement on the slopes would occur only when the safety factor was less than 0.85.	None	No instrumentation; observations planned if seismic event occurred.	Three significant seismic events occurred in the general vicinity of the pipeline. The earthquakes (M6.6, October 1985, M6.8, December 23, 1985 and M6.0, March 25, 1988) were near the Design Probable Event . No impact was evident on the right-of-way nor at the Wrigley and Mackenzie pump stations.	Not Applicable	Adequate		None	None
5e	Forest fires	Forest fires were always considered a possibility and expected to have little impact on overland sections. There was obvious concern about the wood chip insulation and the potential for initiating a long term smouldering fire.	None	Inspect all affected right-of-way and wood chip surfaces following a fire.	Fires in 1994 and 1995 affected 90 km and 53 km of the right-of-way respectively. Within these lengths only 20 to 30% of the right-of-way was damaged. Minor wood chip and cribbing damage on slopes. Impact on adjacent slope stability of concern in a few locations.	Not Applicable	Adequate	Soaking/wetting woodchips from creeks during fire. Post fire hydroseeding of burnt terrain adjacent to several slopes (Slopes 29B and 48B) to promote revegetation and stability.	Very limited scorching of surface wood chips. Impact on adjacent areas more serious.	Ongoing monitoring of some "adjacent" consequential instability (e.g. Kp 182).
6	Stability of Ice-Poor Slopes									
6a	Thaw progression	The thaw penetration beneath ice poor slopes was predicted to reach about 4.2 m and about 5.5 m after six and twelve seasons, respectively. Because the ice contents were low, it was not necessary to control the rate or ultimate amount of thaw.	None.	Thermistors installed to 10 m depth in some more significant ice poor fill slopes (Slopes 8, 13, 18 Vermillion S, Seagrams S)	The six year and 12 year thaw depths are consistent within design predictions.	Not Applicable	Good, all thaw still less than thermistor tip depth.	None	Assumptions adequate.	None
6b	Pore pressures	It was a key assumption that there would be no excess pore pressures and that the slopes would drain as thawing occurred.	None	Piezometers were installed to depths of 2 m to 6 m at five ice poor slopes.		Not Applicable	Adequate	None	As above	None
6c	Stability	As long as excess pore pressures did not develop and drainage occurred with time in the thaw bulb the safety factors should be satisfactory.	None	Review and assessment of monitoring data.	Piezometric data indicates no excess pressures and reasonable drainage on instrumented ice poor slopes.	Not Applicable	Good	None	As above	None

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	Impact Issue	Expected/Predicted Impact	Mitigation Applied	Monitoring Approach	Actual Impact	Effectiveness of:		Operational Improvements	Lessons Learned	Outstanding Issues
						Mitigation	Monitoring			
7	Stability of Unfrozen Slopes	The impact of tree clearing and pipeline construction was expected to be minimal on unfrozen slopes. Some slopes were naturally very steep (56 and 81). Slope 81 was marginally stable largely due to high water discharge on the slope.	Very steep slopes were cut back either to improve stability or to provide reasonable construction grade. Major cut at Slope 56.	Significant unfrozen slopes were instrumented with piezometers and standpipes.	Majority of unfrozen slopes experienced no problems; the major cut at Slope 56 resulted in significant side slopes which experienced slumping. Stability along right-of-way remained satisfactory.	Side slopes at Slope 56 could have been cut flatter, however, major excavation costs would have been involved.	Good	Trench drains cut into the side slopes at Slope 56 to improve stability - proved satisfactory.	Consider flatter side slopes on wet slopes.	None
8	River Crossings									
8a	Bank stability	The backfill mound over the typically deep sagbend excavations was expected to settle and be susceptible to erosion. Many river and creek banks were expected to be susceptible to erosion during peak flow conditions.	Coarse rock rip-rap was placed over the face of all backfill mounds. For rivers and creeks where the whole bank at the right-of-way was considered susceptible, rip-rap was placed on the full width of the right-of-way and in extreme cases to some distance either side of the right-of-way.	Visual observation, especially during and following peak flows.	Many sagbend backfill mounds settled considerably, below the surrounding bank profile in many cases. Some banks were severely eroded; Seagrams, Ochre, Hodgson were the most dramatic. Most other banks remained close to original alignment.	Mostly adequate; Sandbags placed at Seagrams instead of specified rip rap was washed away first summer. See 8b for other extreme cases.	Adequate	Placed proper rip-rap at Seagrams; See 8b for other cases.	Do not underestimate rip rap requirements	None
8b	Sagbend protection	The river crossing designs predicted the potential for stream channel migration based on limited time series air photographs. In some cases, especially streams on alluvial fans, the potential migration was expected to be considerable.	Based on potential channel migration predictions, the "sag point" was selected. The "top of pipe" elevation, specified based on the "design scour" under the main channel at the time of construction, was extended into the bank to the sag point before the pipe would rise to follow the adjacent profile. This "sag protection" was intended to accommodate the channel migration rather than restrain any migration. Designs were based on a 1:100 year return period.	Observation only.	In the summer of 1988, a major storm event caused considerable modification of the flow regime at the Hodgson Creek crossing. No pipe was exposed, however, the modified main channel posed a more significant threat to the north bank. A major rock rip rap buttress was placed on this bank in the winter of 1988-89. In the summer of 1988, the south bank of the Ochre River eroded the low terrace back from the original bank. This was largely influenced by an accumulation of timber debris at a sharp bend in the river downstream of the crossing. The water flow was backed up and overflowed onto the terrace. The erosion exposed the pipe for 30 m beyond the sag point, without any serious consequence.	With exception of the extreme events at Ochre and Hodgson, the design for all other water crossings has provided adequate to date. The original design was based on 1:100 year runoff estimates. GNWT fire tower data for the 1988 storm indicates the 24 hour rainfall exceed the pre-1988 1:100 year estimate.	Adequate.	Remediation at Ochre consisted of replacing the exposed pipe and providing for a new sage point beyond the newly eroded south bank. In view of the considerable overflow water which flowed along a portion of the right-of-way, Enbridge lowered the pipe in the area where the 1 m cover had been provided during construction.	Severe summer storms can be very localized and may well exceed the intensity indicated in normal climate station data. Some remote forest fire lookout stations can provide useful additional data for local summer storms.	None
9	Right-of-way Disturbance									
9a	Work Surface/Snow Road	During the planning for construction there was considerable concern that there could be serious disturbance of the surface of the right-of-way: compaction, disturbance or removal of insulating organic layer, removal of vegetation.	It was specified that travel and work surfaces would be made of packed snow to protect the surface of the right-of-way.	Inspectors to monitor snow cover.	In the first winter construction (1983/84), there was insufficient snow on the right-of-way to prepare a thick enough snow pack for traffic in the north end. The snow was used to prepare a work pad for welding crew, sidebooms, etc. Most of the travel lane was on bare ground, with some hummock or rolling surfaces levelled for more efficient travel. Certain cross-slope portions of right-of-way were graded to provide a safer work surface.	Snow pack mitigation was not available.	Not much to monitor in the short term, but longer term effects anticipated, especially in organic terrain.	In second winter construction, travel lane was on bare ground also.	Based on decision in first winter construction to travel on bare surface, it was learned that the impact on the terrain was negligible and the concern was relaxed for the second winter construction in the north end.	None.
9b	Construction	Potential for short term warming of surface; altering the thermal balance at the surface due to removal of vegetation and possibly the organic layers.	None.	Visual observation	See above for discussion on insulated slopes; for impact noted on overland portions of the right-of-way, see 1a and 2, above. Ground temperatures did warm and thaw continues.	Not Applicable	Adequate.	None.	None.	None.

		A	B	C	D	E	F	G	H	I
	Impact Issue	Expected/Predicted Impact	Mitigation Applied	Monitoring Approach	Actual Impact	Effectiveness of:		Operational Improvements	Lessons Learned	Outstanding Issues
						Mitigation	Monitoring			
9c	Forest fires	Potential for short term warming of surface; altering the thermal balance at the surface due to removal of vegetation and possibly the organic layer.	None.	Visual observation.	Clearing of fire breaks near Norman Wells in 1995 led to erosion adjacent to pipeline right-of-way, which necessitated remedial work.	Not Applicable	Adequate		Expand fire breaks around valve sites, remote maintenance bases, and pump stations.	None
10	Drainage and Erosion									
10	Drainage and Erosion	The surface of the right-of-way was expected to be erodible and slopes grater than 3° or 5% gradient, depending the soil type. The greatest concern was in the early years until vegetation was re-established. Steeper slopes were of greatest concern.	Mound breaks were to be provided at all obvious low points and recognizable cross drainages. In addition mound breaks were to be placed at intervals ranging from 25 to 500 m spacing on overland portions of the right-of-way. It was recognized that not all locations requiring mound breaks could be identified during design and construction and the O&M crews would have to create some in the early years following construction. Where mound breaks were relatively close (25 to 100 m), diversion berms were also specified to divert water away from the ditchline and off the right-of-way. On steeper slopes mitigation comprised ditchplugs, mound breaks and diversion berms at typical spacing from 10 to 50 m.	Visual observation.	There was much erosion in the early years following construction. Some spring runoff in May 1984 exposed a length of pipe at the top of Bear Rock. Deep erosion occurred on the "east" side of the gully leading to the north bank of the Great Bear River and along the "west" edge of the Blackwater north slope. Major erosion occurred on the north shoofly at KP 273. Considerable erosion occurred south of Mackenzie station in very low gradient runoff where the right-of-way intercepted flow on the surface of large swampy areas. Subsurface erosion occurred along ditch line at Slope 29B (Kp 79), which produced ditch subsidence and necessitated remedial coarse backfill. At a few overland sites, cross drainage reduced the cover on the pipe, and was dealt with by placing weights and granular fill or sleeves.	Generally good; some backfill erosion could be related to insufficient mound breaks; the potential for erosion at the major erosion sites was only evident in hindsight.	Adequate.	Major remediation works in both summer and winter programs at the more significant erosion sites. Additional stockpiles of sand bags and rip rap deployed in certain areas.	Anticipate significant local erosion in early years. Anticipate erosion along the ditch line and renewed problems following disturbance (for example, forest fires), and anticipate subsurface erosion beneath frozen wood chips.	Close observations and mitigation required in relation to specific events throughout the lifetime of the project.
11	Revegetation									
11a	Revegetation	Design approach was to re-establish ground cover on mineral soils. Organic soils were not remediated.	Imported seed mixtures were used to provide rapid re-growth, with native species to be naturally introduced.		Good vegetation in most areas. Some additional/on-going seeding needed to be done. Vegetation on overland portions of the right-of-way has required maintenance (brushing), particularly around valve sites for helicopter access. Organic terrain slow to regenerate (75 years).	Generally good.	Adequate	None	Approach was generally successful.	None

2.0 PROJECT PHILOSOPHY AND DESIGN APPROACH

2.1 Introduction

The Norman Wells to Zama oil pipeline traverses approximately 869 km of discontinuous permafrost along the Mackenzie River valley, in the northwest of Canada. The pipeline has carried oil continuously since early 1985 from reserves at Norman Wells, NWT, owned by Imperial Oil Resources. The pipeline is unique in that it is the first major pipeline constructed in permafrost in Canada and the first trunk pipeline completely buried in permafrost terrain in North America. Many design details/issues unique to permafrost and cold regions had to be considered throughout the entire project, with some still ongoing. Conditions of frost heave and thaw settlement, which could produce large differential pipe movement or induce excessive pipe stresses, had to be taken into account in the design (See Nixon et al, 1984).

The 324 mm diameter oil pipeline follows the Mackenzie River valley through much of the Northwest Territories as shown in Figure 2.1. The characteristics of the crude oil in the Norman Wells field were such that heating of the oil to facilitate transport was not necessary. Thus, the pipeline would be allowed to generally operate at ambient temperatures. Running a pipeline "chilled" or below freezing is advantageous in areas of continuous permafrost because thaw settlement is minimized. In discontinuous permafrost, it is considered more advantageous to operate the pipeline slightly above freezing to minimize the problems associated with ground freezing and frost heave. However, at thermal interfaces between frozen and unfrozen ground (e.g., Nixon et al, 1984) strains and curvatures can develop in buried pipelines due to differential heave or settlement. It is therefore of considerable importance for future developments in this area (and other discontinuous permafrost areas) that the amount of frozen ground and the number of thermal interfaces be quantified as much as possible.

The pipeline was constructed in the winters of 1983/84 and 1984/85. Winter construction was the only feasible time for construction as there were no all-weather roads to Norman Wells (only a winter road), and much of the right-of-way was only accessible during the winter. Construction of the pipeline was undertaken in segments, which are referred to as construction 'spreads'. The initial construction plan included six spreads. During the first winter, construction at the more northerly spreads exceeded expectations, and Spreads #2 and #3 were actually constructed as a single spread in the second winter. Figure 2.2 is a flow chart highlighting the location of construction spreads and the season they were constructed.

The majority of the pipeline was trenched using large wheel ditching machines, specially designed for arctic work. The machines were custom-built, twin-engine 1200 HP excavators referred to as the Model 7-10, which corresponded to the design width and depth of ditch in feet. These machines, which had been developed for proposed, larger diameter gas pipelines in the Canadian Arctic, were capable of excavating a smooth regular trench which made laying the pipe and backfilling much easier than in the case of a ditch excavated by backhoe. Smaller, conventional ditchers were used where feasible, particularly in the southern parts. The typical trench depth for the pipeline in a normal right-of-way was between 1.1 and 1.2 m. Deeper burial was implemented at all road (1.0 m minimum cover) and stream crossings (1.5 m minimum cover), as well as adjacent to populated regions (1.0 m minimum cover). Backhoes were employed in areas that could not support the weight of the wheel ditchers at horizontal bends, on many slopes, and in areas of boulder tills where the wheel ditchers could not physically excavate the soils.

Right-of-way disturbance was to be minimized as much as possible to preserve the surface organic layer. The presence of this layer has an insulating effect which, in many cases, is the primary reason permafrost remains in discontinuous zones along much of the pipeline route. Mean annual ground temperatures are often near -1°C, and the permafrost is classified as warm. In the 1983/84 construction season, there was insufficient snow to enable preparation of the intended "snow pad" for the construction traffic. Grading of the right of way was kept to a minimum. However, even with care, the organic mat was compressed significantly in some areas, and may have lost some of its insulation value. In certain cross-slope areas, it was necessary to cut into the organic mat to provide a safe construction surface. The impact of this disturbance may be important when long term settlement of the right-of-way due to thaw is evaluated.

The pipeline was designed to transport around 5,000 m^3/day (33,000 bbl/day) of crude oil, although has been flowing up to 5,300 m^3/day in recent years. Aside from one small leak that was contained and repaired with only a minor loss of oil, the pipeline to date has operated successfully for 14 years.

2.2 Ambient Temperature Pipeline

One of the more important and unique features of the project concept was to operate the pipeline at or close to ambient conditions. Previous oil projects either constructed or contemplated (i.e. TAPS in Alaska) had involved larger, hot oil pipelines, that imposed large positive temperature changes on the environment. In certain cases, it is necessary that the oil be maintained above some relatively warm temperature on account of the hydraulic properties. Warming the oil invariably caused a very large thaw zone to develop, with the potential for the attendant problems of thaw settlement, slope instability, etc. In fact, where buried, the TAPS line was predicted to cause thaw zones of at least 15 - 20 m deep to form beneath the pipe. For this reason, the TAPS pipeline was elevated on piles for the most part where icy or fine-grained permafrost was encountered.

The nature of the Norman Wells crude oil, with a pour point of -14°C, is such that the oil can be allowed to cool off. The approach for the present project was to adopt a more passive thermal design for the pipeline. In this way, it was considered that the oil pipeline would not impose large thermal impacts on the terrain, whether initially thawed or frozen. It was acknowledged that because the permafrost was discontinuous, the flowing contents of the line could adapt to one thermal condition (i.e. thawed), before passing into another thermal condition (i.e. frozen). This would cause generally small amounts of thawing or freezing, depending on the situation. However, for this to occur, sufficiently long lengths (i.e. several kilometers) of one thermal condition or the other would have to be available to obtain equalization, and therefore create the conditions for this differential thermal condition to arise.

A slight (3 to 4°C) warming of the pipe contents would occur as the product passed through the pump stations. These warmer temperatures would gradually dissipate over the next 30 to 50 kilometres downstream of each station and the contents would again become ambient.

At Norman Wells, the pipeline crosses Bosworth Creek within a few hundred metres of the inlet. At the time of construction, Bosworth Creek was the water supply for the community. Due to the proximity of the crossing to the pump station, which would provide a temperature spike to the oil, concerns were raised early in the design period about the impact of the "warmer" pipeline on the stability of the frozen Bosworth Creek slopes. To alleviate these concerns, the operator agreed to chill the oil entering the pipeline at Norman Wells. As the oil flowed south from Norman Wells, the pipeline would achieve an ambient condition.

2.3 Routing (Existing Cut Lines)

Another important feature of the design approach was to route the line as far as possible along existing cut lines. In fact an existing Canadian National Telegraph (CNT) cutline paralleled the proposed route for much of its length in the northern part of the line. This entailed increasing the length of the pipeline in some cases, over the most direct routing. Experience and some site specific investigations showed that the prior clearing caused deepening of the permafrost table, but only in some cases, and a functional relationship between the two could not be established. Therefore, following existing cut lines reduced the possibility of intercepting icy soils in the top few metres beneath the pipeline, but did not prevent this occurrence completely.

Later, ditchwall logs and geophysical profiles would indicate that the fraction of frozen terrain was certainly reduced within previously cleared areas, but a significant fraction of the terrain still remained frozen below the top several metres.

2.4 Construction Schedule

All construction was completed during two winter seasons, with the exception of pump stations and operations and maintenance facilities. The nature of the terrain precluded overland travel after March 1 south of Latitude 64° North, and after about March 15 north of Latitude 64° North. Overland travel did not normally resume till December or early January.

Snow pads were proposed for use for pipeline construction equipment. In some cases, due to insufficient snow fall, snow pad thickness was not sufficient to prevent some disturbance to the right-of-way surface organic cover. However, construction was planned so that no permanent (eg. gravel) work pad remained following construction, and the right-of-way within mineral soils was re-seeded.

2.5 Reclamation

The philosophy of reclamation was to minimize erosion on the right-of-way by implementing a rapid revegetation program combined with additional physical erosion control measures in highly erodible areas such as steep slopes. To this end rapidly establishing agronomic species were utilized rather than slower establishing native vegetation species. The plan was that eventually the agronomics would die back and succumb to the native species invasion while continually providing stable cover. Immediately following construction, the snow covered right-of-way in mineral soils was to be seeded from truck mounted cyclone seeders, supplemented as necessary in hard to reach areas with hand broadcast cyclone seeders. On organic terrain, no re-seeding was conducted, despite some disturbance to the organic cover.

Special measures such as tree and shrub planting were deemed unnecessary as considerable stock of these species were expected to be found on the right-of-way in the form of roots, sprigs and seeds, which would rapidly re-establish this type of cover. It was considered not desirable to have shrubs growing back on the right-of-way, and in some cases the owner has actually had to cut them down in numerous areas to provide helicopter landing areas, and for other maintenance purposes.

2.6 Monitoring

Under the terms of the regulatory approval to proceed with the project, the owner was required to implement a monitoring program. The details of the program were left to the owner. The program consisted of line patrols, and the installation of pipe, and geotechnical instrumentation to permit monitoring of conditions along the pipe and right-of-way. In addition, the government took an active interest in the project from the perspective of impact on terrain. Various types of instrumentation were also installed by the government for their research purposes (MacInnes et al. 1990).

The main form of pipeline monitoring was the weekly airborne line patrols in which any obvious present or pending problems were noted. The more common observances were related to erosion at river banks or overland sections. These patrols were particularly effective during some active erosion periods in the first few years following construction. The patrols have also included regular checks on the controls for automatic block valves.

Pipe temperatures would be monitored at several key points along the pipeline, with more emphasis on the more northerly parts of the route. Inlet and outlet temperatures at pump stations would also be monitored. A commitment was also made to monitor pipeline movements/curvatures. Although at the time, it may not have been known exactly how this was to be achieved, some interesting and leading edge technology was brought to bear on this project soon after the end of construction.

Pipe curvature was monitored in the early stages by measuring relative elevations of adjacent points along the top of pipe, and differentiating them to obtain a crude measure of curvature.

Some pipe and right-of-way settlements were monitored by the Permafrost Terrain Research and Monitoring (PTRM) group, and these are documented by Burgess (1997). The first permanent site for pipe vertical movements was installed by NRCan at Kp 2.0 in 1994, and has shown interesting trends in seasonal and long term pipe movements. Also, starting about 1993, Enbridge has continuously surveyed an area of pipe uplift at Kp 5.2, and in 1996, a deep bench mark was installed to obtain geodetic elevations for this site.

The most innovative approach to pipe displacement and curvature monitoring was developed early in the pipe operating life. An internal pipeline tool was developed by Enbridge and Pulsearch/

PIGCO using inertial guidance accelerometers to measure pipe curvatures and other characteristics of the pipe (Adams et al, 1989). The instrument is known as a pig tool. The challenge for this project was to downsize equipment and increase data storage and on-board power requirements so that the necessary equipment would fit inside the small diameter pipeline. The equipment has been continually upgraded, and has generally proved very successful in delineating curvatures and changing pipe profiles over time in the pipeline.

Enbridge also retained geotechnical consultants to provide on-going stability assessment of the slopes along the pipeline route. In most years, this assessment has taken the form of a route reconnaissance in late fall to observe the physical state of the right-of-way and the slopes, reading instrumentation at the slopes, and reviewing the current factor of safety in light of the current conditions, relative to the original design assumptions.

Appendix A provides a listing of all instrumentation installed on the pipeline right-of-way to December 1998. Table A-1 lists the both the owner installed instrumentation and some instrumentation installed by Government agencies. Table A-2 provides a listing of NRCan site instrumentation.

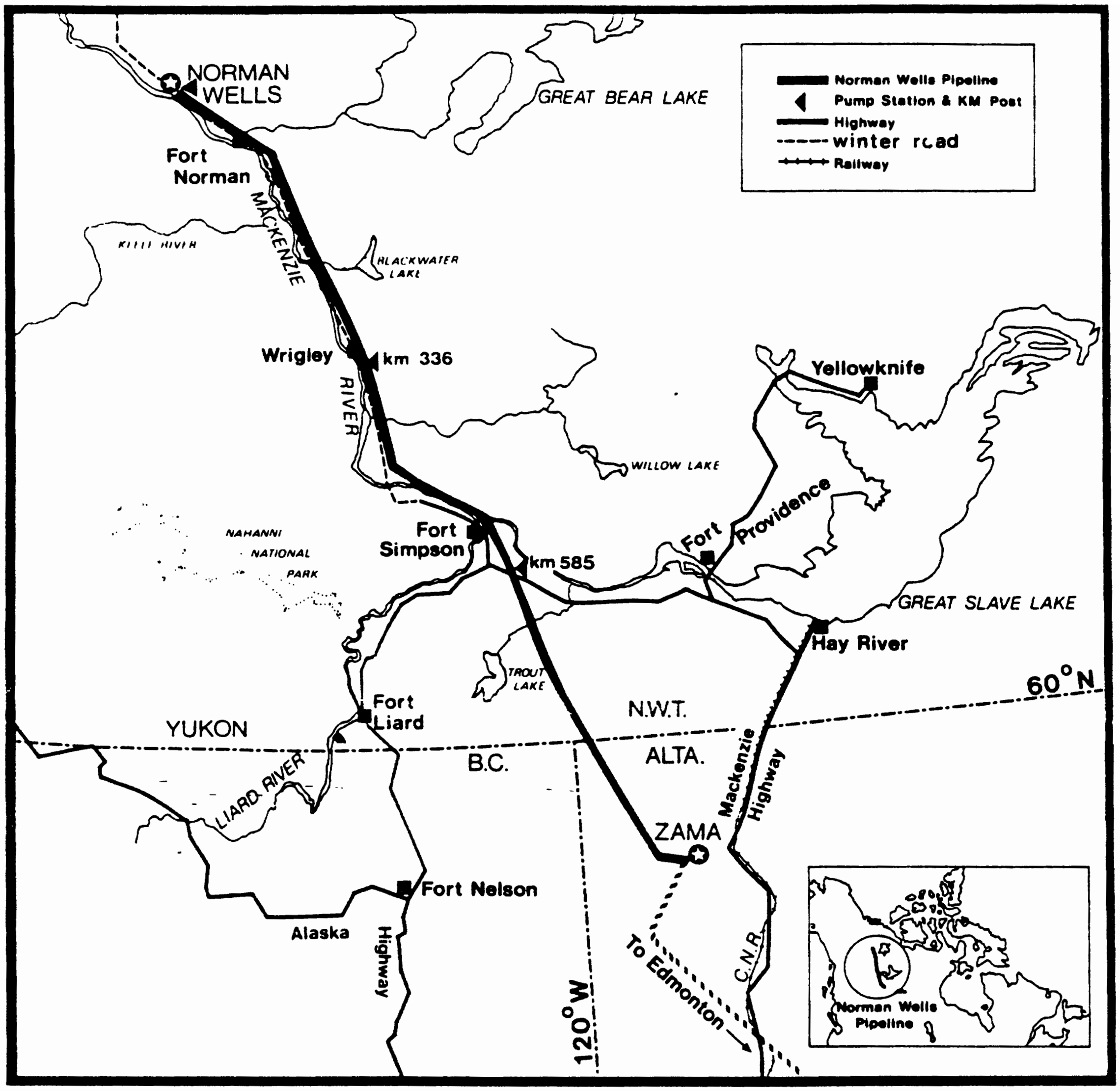
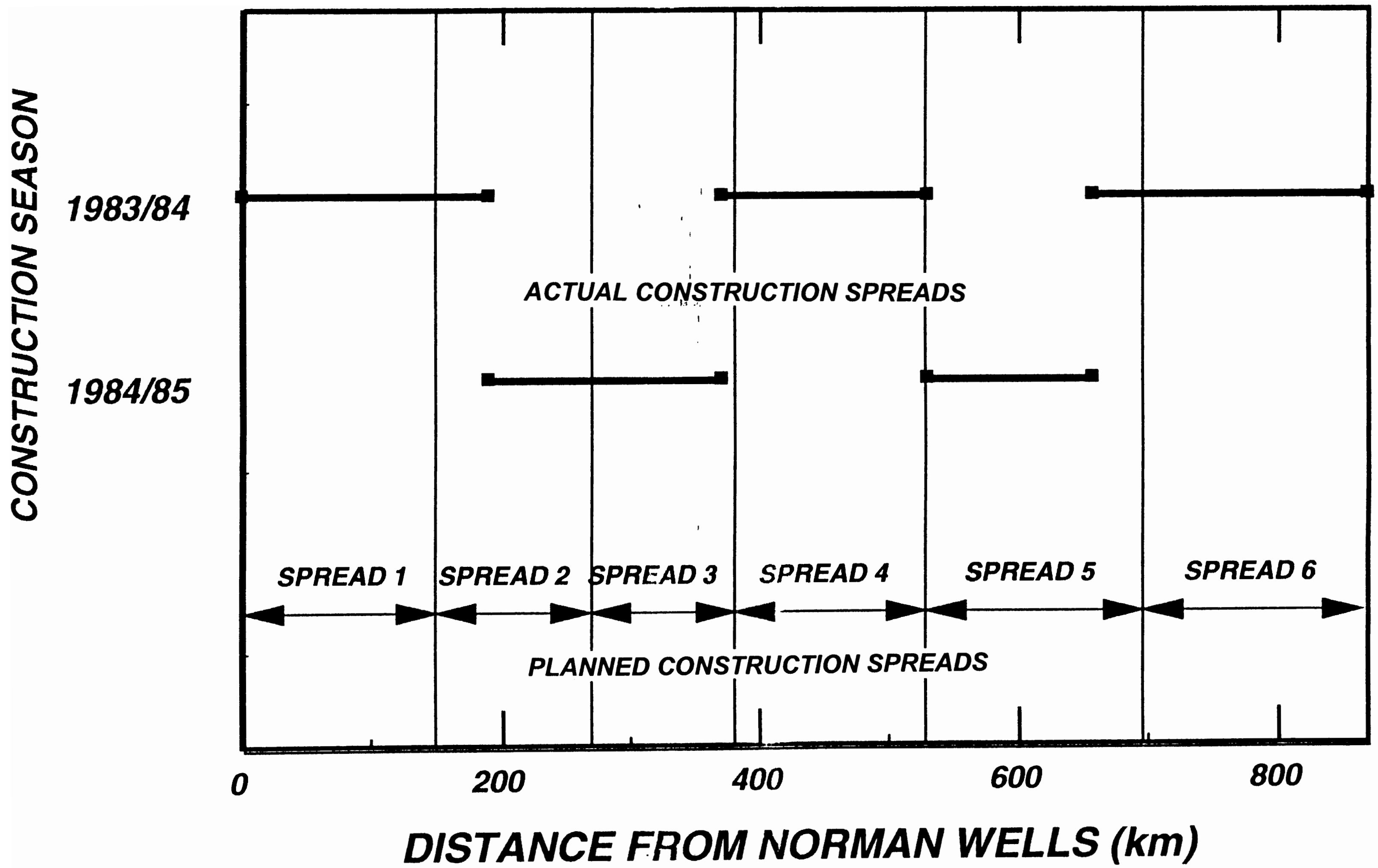


FIGURE 2.1
LOCATION OF NORMAN WELLS PIPELINE AND PUMP STATIONS (KM 0, 336 AND 585)



PLANNED SPREAD LENGTHS
1(149.5); 2(119.8); 3(111.1); 4(148.4);
5(167.1); 6(172.4); ZAMA LAKE AT KMP.869

FILE: CONSEAS2

FIGURE 2.2
CONSTRUCTION SCHEDULE
FOR NORMAN WELLS PIPELINE

3.0 INVESTIGATIONS AND DATA BASES FOR INPUT TO DESIGN

During the fifteen years prior to construction, several pipeline projects were considered in the Canadian Arctic. Engineering studies conducted for these projects recognized the special environment in which the proposed pipelines were to be located. A large volume of information and data describing the specifics of the Canadian Arctic environment had been collected. Different design approaches and concepts had been developed to address the design problems caused by the presence of continuous and discontinuous permafrost. A continuing evolution could be observed in understanding of the northern environment and development of associated design solutions.

3.1 Terrain Investigation and Mapping

The Norman Wells to Zama pipeline alignment traverses terrain consisting of a range of permafrost and soil conditions. The purpose of the terrain investigations was to ensure the integrity of the constructed pipeline as influenced by the various geomorphological and geotechnical conditions along the route.

The methodology of assessment began with an initial alignment location and a field reconnaissance stage in which experienced personnel selected the route in order to avoid potentially unstable terrain. The alignment was chosen on the basis of air photo interpretation and route reconnaissance, and took advantage of existing cleared rights-of-way. In some geographical areas of the general route in the Mackenzie Valley, it was known from previous studies and field experience that poor or potentially unstable terrain units were prevalent. These regions acted as significant control points and in some instances the pipeline alignment was lengthened over the more direct straight line route to avoid potentially troublesome terrain.

The entire routing was subjected to aerial photograph interpretation. The terrain units were identified, with the information being included on the construction alignment sheets. The purpose of the terrain mapping was to identify geomorphological features that were important from either a design or construction perspective. These could include desirable terrain for the pipeline, potentially problematic terrain to be avoided, and the presence of granular borrow sites. Permafrost terrain or frozen-unfrozen interfaces could, in a general manner, be identified from the aerial photographs.

From a design point of view, the potentially most problematic terrain to be dealt with were the slopes. For the overland sections, the main concern was thaw settlement and the influence of the variable frozen/unfrozen conditions as well as varying ice contents within the frozen terrain.

As part of the field investigation program, all significant slopes were catalogued and a list of these slopes was developed. A selected number of representative slopes were subject to geotechnical drilling. In all some 150 slopes were considered to require evaluation for design purposes. Table 3.1 summarizes the category of slopes along the selected route.

In addition to the slope investigations, numerous sites along the route were assessed in terms of thaw settlement potential. Sixty six sites were used in the study between Norman Wells and the

Willow Lake River, where the pipeline alignment followed previously cleared cutlines in permafrost terrain. Comparisons were made between actual field measurements of thaw settlement and thaw settlements as calculated by the thaw settlement model established for pipeline design.

Table 3.1 Slope Categories

Total Number of Slopes Reviewed	Number of Slopes			
	Average Grade Perpendicular to Route			
	Negligible	4 - 7 %	7 - 16 %	>16%
106	47	24	24	11

Seven sites were also instrumented along the most northerly portion of the alignment, between Norman Wells and Fort Norman to monitor and quantify seasonal frost heave at interfaces between undisturbed and previously cleared areas in various types. Most sites were located in glacial till terrain units, although two additional sites were instrumented in lacustrine soils, near Kp 276 and Kp 307.

Table 3.2 summaries the estimated permafrost content along the route in uncleared areas, based on written direct testimony, Canadian Arctic Gas Pipeline Limited, National Energy Board, N-AG-3-178.

Table 3.2 Permafrost Terrain Along Route

Pipeline Kilometrage	Landmarks	Permafrost Terrain (%)
0 - 110	Norman Wells - South of Police Island	93
110 - 376	South of Police Island to Willow Lake River	77
376 - 866	Willow Lake River to Zama Lake	34

3.2 Drilling Investigation

Drilling and sampling of the slopes along the alignment was carried out during the 1981 and 1982 field seasons. In all, 109 boreholes were drilled on, or adjacent to nearly fifty slopes. A representative number of boreholes were instrumented with standpipes or piezometers for monitoring of groundwater conditions.

Similar drilling programs were conducted at the thaw settlement sites to collect information for the prediction of the thaw settlement.

3.3 Previous Investigations

Prior to the start of the project, a number of other linear corridor studies had been undertaken, which included considerable geotechnical review and data collection. These data were reviewed as part of the Enbridge (IPL) project. Previous studies included geotechnical evaluations from the following studies:

- Canadian Arctic Gas Study Limited
- Foothills Pipe Lines Limited
- Beaufort-Delta Oil Pipeline Ltd.
- Mackenzie Valley Research
- Mackenzie Highway

Most of this previous data and the Enbridge (IPL) borehole data were complied into the Enbridge (IPL) Borehole Database.

3.4 Laboratory Testing

As part of the IPL drilling program, samples were collected for laboratory testing purposes. The primary tests that were conducted were: natural ice/water content, Atterberg (Plasticity) Limits, thaw settlement tests and strength parameters by means of direct shear and triaxial compression tests. On the basis of the laboratory tests, and field identification, the frozen soils on slopes were classified into three groups: ice rich clay, ice rich till, and ice poor till.

Additional data for the Mackenzie River Valley soils was also collected from previous studies, noted in Section 3.3 and from other research (Roggensack, 1977).

Table 3.3 presents a summary of the design soil strength parameters used in the design.

Table 3.3 Summary of Soil Strength Parameters

Soil Type	Friction Angle (°)	Effective Cohesion (kPa)	Bulk Density (kg/m^3)
Ice Rich Clay	24.5	3.5	1760
Ice Poor Till	31.5	4	2000
Ice Rich Till - low normal stress - high normal stress	2231.5	12.54	1760

Table 3.4 presents a summary of the geothermal properties of the soils.

Table 3.4 Summary of Geothermal Parameters

Soil	Usual Thickness (m)	Thawed Conductivity (W/m°C)	Frozen Conductivity (W/m°C)	Total Water Content (%)	Unfrozen Water Content (%)	Dry Density (kg/m³)
Peat	0.3	0.46	1.09	200	0	377
Active Layer (fine grained)	0.9	1.55	2	25	5	1600
Icy Subsoil (fine grained)	-	1.38	1.88	50	5	1140
Coarse grained	0.9	2.76	3.8	15	0	1906

3.5 Geophysical Surveys

As part of the investigation to delineate soil and permafrost conditions along the pipeline route, continuous geophysical surveys were undertaken by Enbridge (IPL) and their consultants Hardy Associates between March 1981 and May 1982 (see Kay et al, 1983). The survey was undertaken with the Geonics EM-31 and EM-34. Both instruments measure apparent electrical conductivity of the near-surface soils. The shallowest possible survey with the above equipment was with the EM-31 on its side, which measured conductivity in the top 3.5 m. The deepest measurements were with the EM-34 which had a range up to 9.0 m.

Apparent conductivity is difficult to use to positively identify soil or permafrost conditions even in a homogeneous soil without layering. Quantitatively, there is considerable overlap in apparent conductivities from one soil to another as well as with different geothermal conditions.

The geophysical survey described by Kay et al (1983) was therefore interpreted with the assistance of vegetative indicators and detailed boreholes drilled on the center line of the right of way. Figure 3.1 shows how sharp or well-defined frozen and unfrozen boundaries can be identified in a uniform soil.

The geophysical surveys identified a large number of relatively closely spaced interfaces between frozen and unfrozen terrain. Hence, it was concluded that there would be no realistic basis for selecting different design criteria for frozen and unfrozen segments. The entire pipeline was therefore designed on the basis that significant thaw settlement could occur anywhere.

3.6 Riverbed Surveys

At stream crossing locations requiring site specific designs, three or more cross-sections, thalweg and water surface slopes were surveyed. Generally, a centreline cross-section was surveyed as well as two or more hydraulic sections. The hydraulic sections were usually located both upstream and downstream of the centreline. Observations of high water marks, scour holes, vegetation types and density, bed material and bank material were recorded at locations within the reach investigated. Water velocity and flow discharge were measured using a current meter at surveyed hydraulic cross-sections. Discharge data at the major crossings were obtained from Water Survey of Canada recording stations.

3.7 Storm Runoff Predictions

Fifty seven stream crossings were examined during the pipeline design. The hydrology of the crossing was based on a flood discharge with a return period of 100 years. Assuming a project life span of 25 years, the probability of the 100 year discharge occurring within this life span is approximately 22 percent. Historically, the mid-channel failure of pipelines is rare.

The magnitude of the 100 year flood is best estimated from a frequency analysis of recorded flows. The Water Survey of Canada had recording stations on the following crossings:

- Bosworth Creek
- Great Bear River
- Big Smith Creek
- Willowlake River
- Mackenzie River
- Trout River
- Kakisa River

Only the Great Bear River, Mackenzie River and Kakisa River had sufficient data to estimate the 100 year discharge flow. For the remaining watersheds, the method of area versus discharge envelope curves was used in this study to provide an initial best estimate of the 100 year flood discharge.

3.8 Meteorological Data

Climatic and meteorological data were collected from Canadian Arctic Gas Study Limited (CAGSL) compilations made during 1973-77, based on the 1941 to 1970 records. These are summarized in NES (1974), Applications of Geothermal Analysis. In particular, the climatic data for the Arctic Gas Regions 14, 15 and 16 were used to cover the range of climatic conditions along the route. These CAGSL regions correspond essentially to the following segments of the Norman Wells route:

Region 14:	Norman Wells (Kp 0) to Kp 110
Region 15:	Kp 110 to Kp 376
Region 16:	Kp 376 to Kp 869

Geothermal modeling for the Norman Wells project used mean monthly air temperatures from the Norman Wells, Wrigley and Fort Simpson AES stations. To avoid re-working a lot of the more complex surface energy balance calculations that were carried out previously for the Arctic Gas project, the Norman Wells project used the predicted surface temperatures from earlier modeling, and applied them directly to the soil surface being modeled for the Norman Wells project.

Monthly snow cover values were also taken from the relevant Arctic Gas regions, and used in ground thermal analysis for the Norman Wells project.

3.9 Ground Temperatures

From previous studies in the Norman Wells area, such as the CAGSL project, it was known that the widespread discontinuous permafrost in that area had mean ground temperatures at depth of -1 to -2°C, and active layer depths up to 1.5 m, depending on soil type, surface organic thickness, disturbance and other factors. Some earlier data from CAGSL boreholes were available for ground temperature conditions along the route. In addition, additional holes drilled by Enbridge and NRCan/GSC for this project provided additional coverage for ground temperatures.

Boreholes where thermistor strings were installed for post-construction site investigations and monitoring are found in are listed in Appendix A.

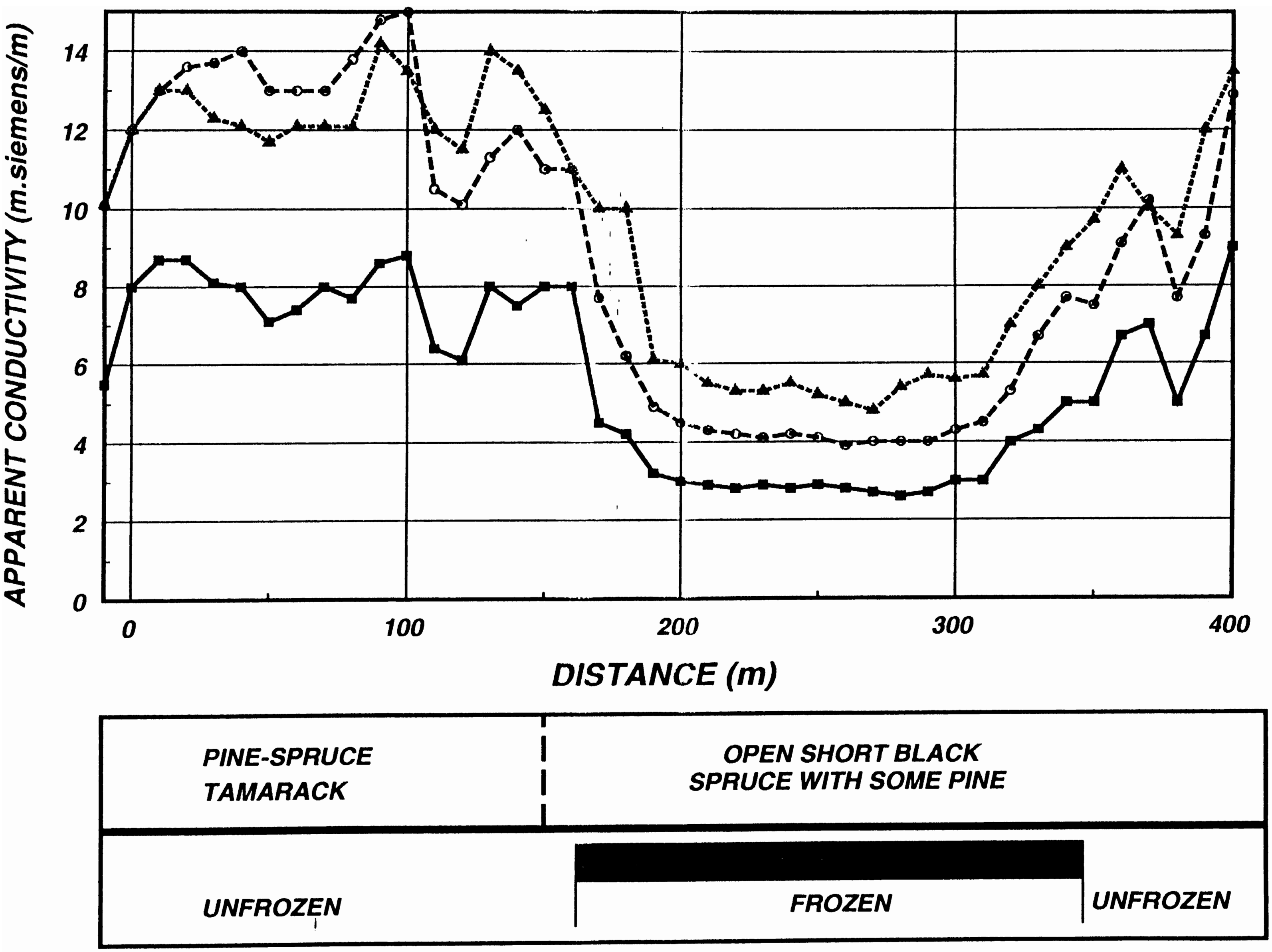


FIGURE 3.1
GEOPHYSICAL PROFILE

4.0 DESIGN AND MITIGATION (EXPECTED IMPACT)

4.1 Pipe/Ground Thermal Regime

The pipe temperatures for different cases, determined by one dimensional geothermal analyses, are shown on Figure 4.1, and illustrate the different (warmest and coldest) pipe temperature conditions that were anticipated for more or less continuously frozen or unfrozen areas. These were obtained assuming the pipe would be fully equalized to the ground temperature at pipe burial depth. These were used in assessment of permafrost and slope stability in the design. They were also to be used by regulators in tracking the actual performance of pipe temperatures in the northern part of the pipeline route. Further warming would be expected further south, but this was not considered as important, due to the lower percentage of permafrost terrain.

Oil chilling at Norman Wells was considered important to permafrost stabilization for the most northern part of the route, although it was recognized that the effects of chilling might only propagate 30-50 km down the pipeline (Hardy Associates (1978) Ltd., 1983b). That is, due to the relatively low energy input of the pipeline to the environment (roughly equivalent to the heat from a 100 W light bulb every 10 m of pipe), the pipe and contents would adapt their temperature to that of the surrounding environment after a relatively short distance along the route. Therefore, although chilling would have some benefit in the early part of the route, the ambient environmental temperatures would tend to control the temperature of the flowing oil after some short distance.

Considerable difficulties were experienced in cooling the oil to a discharge temperature of -1°C or colder during the peak summer periods. Expensive additional compression equipment had to be available on standby for a relatively short period during the summer season, to handle peak cooling demands. This later led to an operational change in 1993 involving seasonal increases in pipe inlet temperature to +9°C to +12°C, and winter cooling to - 4°C to achieve an annual average of about -1°C.

The original criteria for oil cooling involved allowable temperature excursions of greater than 0°C for up to 8 hours. Positive temperature excursions longer than this were to result in a shut-down of the pipeline. In hind-sight, this was a very restrictive criterion, as no thaw for any appreciable distance down the line would occur in such a scenario. (A criterion involving a number of "degree-days" was considered in 1990, being a more reasonable protection against the perceived problems of pipe warming and local thawing. However, this criterion was never formally applied.)

Figure 4.1 indicates that the anticipated pipe temperature for overland thaw settlement design would peak at around + 7.0°C in late summer, and would fall to around 0°C for a lengthy period in winter. This would result after the pipe had crossed through a long stretch of thawed ground. Conversely, the equivalent case for a frost heave design scenario was a peak of about +2°C in summer, falling to -7.5°C or so in late winter. These temperature histories were used in later 2-D thermal simulations for thaw settlement and frost heave.

4.2 Pipeline Design

The maximum allowable operating pressure within the pipeline system is 9929 kPa (1440 psi). Therefore for a selected nominal outside diameter and a specific minimum yield stress, the minimum nominal wall thickness could be determined (about 5.6 mm or 0.22 inches). However, this minimum wall thickness required for a conventional pipeline does not account for additional loadings due to several loading mechanisms.

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, i.e. near ambient air and ground temperature at the time of installation and tie-in. The actual temperature differential used for many of the design studies was 36°C. This corresponds to a reference temperature of approximately -30°C and a maximum operating temperature (at that time) of +6°C.

Novel concepts (for that time) were developed and implemented for the design of the first fully buried oil pipeline in permafrost terrain. The basic design concepts included selection of the pipe diameter to limit the energy input to the environment, and to provide for an 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. Stress criteria, where applicable, were used as defined by existing regulations. Strain criteria for displacement controlled loads were established analytically. 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 also established.

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 0.689 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.

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 were evaluated and defined.

A finite element inelastic computer model SAVFEM was 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. Load cases studied included thaw settlement, frost heave and bend analyses with the inclusion of seismic induced loadings. Significant results of the analyses are discussed as follows.

Design concepts developed for this 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 would result in some degradation of permafrost and would 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 was treated as a structural member designed to withstand deformations caused by differential settlement resulting from construction and operation.
- To the extent possible, the pipeline was located on previously disturbed and cleared rights-of-way (seismic cut lines, and telephone line).

The more important implications of applying the above design criteria to the oil pipeline were to introduce secondary soil loadings on the pipeline as follows. 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.

4.3 Thaw Settlement

As mentioned previously, the low energy input of the pipeline on overland sections into the permafrost meant that the pipe would not directly cause significant thawing of the underlying permafrost. However, even though no work-pad was used, construction disturbance and clearing activities on the overland sections would cause the permafrost to thaw out slowly with time in many locations, because of changed surface thermal conditions. If settlement were to develop uniformly, little or no effects would be felt by the pipeline resulting from this. However, at changes in terrain conditions such as from initially thawed to frozen, or at sudden changes in subsurface ice content, differential thaw settlement may occur across such interfaces. Because of the possibility of stable soil existing close to icy permafrost that could settle to the maximum amount, differential settlement across the transition was conservatively assumed equal to the total settlement that could occur within a terrain unit. This mechanism is illustrated on Figure 4.2. An infinite length of each soil type was generally considered on either side of the interface.

4.3.1 Cover Depth

The minimum depth of cover was 0.76 m, with an additional construction tolerance. Original submissions to regulators had employed a 1.00 m cover depth. However, as the design evolved, it became clear that the design for thaw settlement would benefit from a reduced cover depth. This would reduce the anticipated loads on the pipe, and therefore the resulting pipe strains at a potential settlement transition. A request for cover depth reduction to 0.76 m was considered and granted during the design process.

4.3.2 Borehole Data Base

Based on a very extensive borehole data base of around 10,000 boreholes in the area, thaw settlement analyses were carried out between Norman Wells and the Zama Lake terminal. Computer programs were used to assess the thaw strain of different soil layers, and integrate these to obtain the settlement occurring between the pipe and the maximum anticipated depth of thaw. As the pipe base was located between 1.0 and 1.3 m beneath original ground surface, and the maximum anticipated thaw depth in a 25 year period was about 6.0 m based on long term field observations in approximately similar terrain, the soil depth interval which would thaw could be defined.

Thaw settlement estimates for each borehole were then grouped by geological terrain unit and geographical region. The route from Norman Wells south was sub-divided into 3 geographical regions for convenience. Within each of these, ten or more terrain units could occur, and so a matrix of thaw settlement estimates evolved, based on borehole information alone (similar to Hanna et al., 1983).

4.3.3 Thaw Settlement Test Sites

Seven natural thaw settlement test sites were located along the route to observe thaw settlement based on 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, reported by McRoberts et al (1978) were examined to expand the data base for the pipeline route in this area.

It was concluded that the borehole thaw settlement estimates over-estimated the observed field thaw settlement to some extent (typically by about one-third). The field thaw settlement estimates were therefore used to temper some of the more extreme thaw settlement estimates based on borehole data alone. Later, detailed checking of the borehole data base indicated that some estimates of ice content were on the high side, and the thaw settlement correlations at the low range of moisture content were too high (i.e. overly conservative).

4.3.4 Thaw Settlement Design Summary

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.

4.3.5 Thaw Settlement, Load Transfer and Input to Pipe Structural Analysis

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 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.

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 0.7 to 0.8 m in mineral soils and 1.2 m in organic soils, decreasing slightly with distance from Norman Wells.

The details of the pipe stress analysis are contained in Stresstech (1984). A series of pipe strain simulations were carried out with different loading combinations. The most effective way of accommodating larger ground settlements was to increase the wall thickness, over that thickness required by code to contain the design internal pressure alone. These simulations resulted in design wall thicknesses, as listed in Table 4.1, for a 359 MPa (X-52) grade steel pipe.

Table 4.1 Design Wall Thickness

Location (km)	Design Thaw Settlement (m)	Design Wall Thickness (mm)
0 to 79	0.8	7.16
79 to 440	0.75	6.91
440 to 868	0.7	6.35
Thick organic deposits	1.2	6.35
River crossings	-	9.54

River crossings required heavier walled pipe as dictated by code, and this is normal practice for pipelines elsewhere in Canada.

4.4 Frost Heave

It was not intended to operate the oil pipeline at temperatures significantly below 0°C. However, the possibility existed that the pipe might induce small amounts of frost advance and heave beneath it. If the pipe traversed several kilometers of stable permafrost at temperatures of -1 or -2°C, it was thought that the contents of the oil pipeline would tend to adapt to the surrounding subzero temperatures. The near-surface 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 into unfrozen terrain, the potential for differential frost heave (also shown on Figure 4.2) would exist.

Sag bends were identified as being particularly susceptible to frost action. The compressive strains initially in the pipe due to operating conditions would be accentuated by frost heaving acting upwards at the apex of a sag bend (at the bottom of a slope, for example). This led to the requirement for pipe insulation at a limited number of sag bends, where unfrozen ground and a larger bend angle might coincide. The effectiveness of these insulated pipe joints was to be checked with some thermistor strings.

Geothermal and frost heave analysis using the Konrad-Morgenstern Segregation Potential method (Konrad and Morgenstern, 1981) were carried out to estimate the likely frost depth beneath pipe, and the associated frost heave. The frost depth was estimated to extend 1.5 m or so beneath the pipe, with an estimated heave of 10-12.5 cm. The other important parameter required for an estimate of pipe strains at a frost heave transition is the uplift resistance parameter. A novel method of calculating this input was developed for this project, assuming the frozen soil on either side of the pipe cracked in tension, forming two rectangular blocks of soil. These blocks were then bent upwards in flexure, and the uplift resistance calculated using creep theory. The uplift resistance calculated using this method was in the range of 220 kN/m (see Nixon et al, 1984).

4.5 Seismic Effects and Other Loadings

A buried pipeline is potentially subject to loading conditions from several seismic hazards. The strong ground motions induced by a seismic event are characterized by a series of ground waves that can impose strains on a buried pipeline. No known active faults were identified, and generally the impact of seismic aspects on the pipeline design was very minor. Ground accelerations of 12% and 3% of gravity for the Design Maximum Earthquake were identified for two zones along the route. These translated into small additional compressive axial strains in the pipe wall.

Localized loadings on the pipe such as denting by boulders in direct contact with the pipe were also considered. It was estimated that cobbles or boulders in the range of 0.15 to 0.3 m in diameter would tend to punch into the soil matrix, rather than cause significant denting of the pipe. The potential for denting or ovalling due to larger boulders was present, and the use of over excavation and replacement by loose bedding was considered as a method for reducing local pipe strains to acceptable levels.

Buckling of the pipe due to high compressive axial forces was briefly analyzed, and not considered likely except in areas of organic terrain where transverse soil resistance would be very low. In such organic areas, the terrain would tend to be lower, and the pipe would more likely be roped in with a convex downward profile. Therefore, the pipe would be more likely to buckle downwards, which would be less of a concern for pipe integrity.

4.6 Stability of Slopes

4.6.1 General

The overall approach in designing the slopes along the pipeline right-of-way was based on the following hierarchical process:

- alignment location and field reconnaissance
- slope catalogue of significant slopes and the engineering characteristics
- field investigation and laboratory testing
- design of slopes
- design confirmation during construction phase
- operations, maintenance and monitoring phase

The first three tasks have been discussed in Section 3. Within the design phase, two basic issues were to be addressed. They were:

- determine which slopes will be stable
- establish practical mitigation techniques for potentially unstable slopes

The review of stability was based on potential failure modes that the slopes may experience. The slopes were further classified by the predominant permafrost/geotechnical soil type on the slope. Four soil types were considered: ice rich clay, ice rich till, ice poor till, and unfrozen (Hanna and McRoberts, 1988). Table 4.2 lists the potential failure modes for three geothermal soil conditions.

Table 4.2 Potential Slope Failure Modes

	Thermal Condition		
	Frozen	**Unfrozen**	**Thawing**
Skin/Planar	unlikely condition	possible condition	possible condition
Plug	unlikely condition	unlikely condition	possible condition
Ditch Backfill	unlikely condition	possible condition	possible condition
Deep Seated	possible condition	possible condition	unlikely condition

The following subsections provide specific information on the design of the slopes with the various geothermal conditions, and different geotechnical characteristics.

4.6.2 Mitigation Methods

For those slopes on the right-of-way that were deemed to require mitigative measures, three methods were proposed, depending on the geothermal conditions. In highly ice rich slopes, and on steeper ice rich slopes, a "prevent thaw option" was considered. The intent was to restrict any thawing to the original natural active layer.

In some slopes, depending on the grade, and soil and/or ice conditions, thaw was to be permitted, but at a reduced rate.

Some slopes were sufficiently steep as to require cutting back to ensure long-term stability.

Where prevention or retarding of the thaw was a requirement, the use of insulation was incorporated. During the design process, one dimensional geothermal analyses showed that it was possible to reduce the anticipated depth of thaw by placing an insulating layer on the natural ground surface. The types of insulating materials that were considered were combinations of gravel, synthetic board insulation, and natural insulating materials such as wood chips. It was also considered that some form of gravel/synthetic insulation could retard thaw but could not eliminate thaw penetration.

Wood chips were found to be a good insulator, environmentally neutral, and relatively cost effective. The wood chips were also expected to be more flexible and yielding as thaw settlement occurred, compared to rigid board insulation. Geothermal predictions showed that a sufficient thickness of wood chips could substantially reduce thaw within the 25 year period following construction, compared to a non insulated slope, disturbed by construction.

Table 4.3 lists the design guidelines for cut-off angles for slopes, and backfill materials, based on the soil type and slope surface.

Table 4.3 Design Slope Angle and Backfill Guidelines

Soil Type	Bare Surface	Wood chip Insulation	Gravel Insulation	Backfill Backhoe Spoil	Backfill Wheel Ditcher Spoil
Ice Rich Clay	< 9° stable	> 18° (1)	>14° (1)	> 4° (3)	> 7° (4)
Ice Rich Till	< 13° stable	> 20° (1)	> 18° (1)	> 7° (3)	> 10° (4)
Ice Poor Till	< 18° stable	> 18° - 22° (2)	> 18° - 22° (2)	> 10° (3)	> 14° (4)

Notes: (1) Cut and Insulate or Thermopiles
(2) Cut back depending on height of slope
(3) Improve or Replace
(4) Select

The following table summarizes the mitigation measures carried out on the slopes for the entire length of the pipeline.

Table 4.4 Summary of Mitigation Measures for Slopes

No Mitigative Measures	Select Backfill (only)	Cut Back	Insulate	Cut and Insulate
61 (37%)	33 (20%)	16 (10 %)	46 (28 %)	8 (5 %)

4.6.3 Effects of Thawing

During the design process two primary effects of thawing frozen soils were considered. The first effect was that of “residual stress”. This was the term given to the value of effective stress in a frozen soil that thaws under undrained conditions. In soils where there is considerable ice, and this ice becomes water on thawing and saturates or super-saturates the soil, the residual stress is likely to be zero. On the other hand, when the ice (water) content is low, on thawing the soil may become effectively unsaturated, with a negative effective stress. Such a negative stress would provide an increase in the stability of a thawed slope. It was considered conservative to assume that the residual stress in all soils would be zero.

The second effect of thawing, was the porewater pressure response. In certain soil types, excess porewater pressures could be generated due to thaw consolidation effects. This increase in porewater pressure would have a destabilizing effect on slopes and was the prime issue in the stability analyses.

In ice poor tills, the water contents were generally low enough to permit the assumption to be made that excess porewater pressures would not develop during thawing.

In ice rich soils, water (ice) contents were such that excess porewater pressures could be generated. With time, as the number of thaw cycles increased, the generation of excess porewater pressures would decrease, and become less of a destabilizing influence.

4.6.4 Factor of Safety

The factor of safety applied to the slopes design was a function of many factors, the most important of which was a degree of uncertainty with the mitigation selected for the ice rich slopes. The target factor of safety for the frozen slopes, for static loading conditions, that is not involving earthquake loadings, was 1.5. At the same time, dynamic/earthquake loading conditions could result in a *pseudostatic* factor of safety equal to or greater than unity (Newmark, 1974). It was shown that significant ground movement would not be predicted until the pseudostatic factor of safety fell below about 0.85.

4.6.5 Stability Design

In the 1970's research at the University of Alberta focussed on the stability of slopes in permafrost (McRoberts and Morgenstern, 1974a, 1974b). From this work, the theoretical framework for the slope stability assessment was developed.

For the static analysis of frozen slopes, an extension of the infinite slope theory was developed to include the effects of thawing soils, with a resulting rise in porewater pressure, and the effect of lateral confinement created by the frozen edges of a thaw bulb around the pipeline.

The infinite slope theory was further developed to include the effect of a horizontal ground acceleration, as produced by an earthquake. This theory was termed *pseudostatic*. Both the static and the *pseudostatic* theory are discussed in McRoberts and Nixon (1977), and Hanna and McRoberts (1988).

The static design analyses were conducted on a wide range of parameters encompassing all expected slope conditions, including the soil conditions (ice rich clay, ice rich till, ice poor till), the predicted pore pressures, the slope angle, the predicted depth of thaw, and the predicted shape of the thaw bulb. The pseudostatic analyses considered the similar range of parameters.

For unfrozen slopes conventional slope stability theories and stability analyses were used. It was expected for example that the clearing of trees and other vegetation would have a minimal impact on the stability of the unfrozen slopes. Where it was found that the slopes were considered too steep, they could be cut back, without the potential for long-term instability that may be associated with frozen slopes.

4.7 Right-Of-Way Disturbance

It was recognized during the design process and from previous investigations (Canadian Arctic Gas Study, and Foothills Pipeline Project) that construction effects could lead to significant disturbance of the right-of-way and surrounding lands. Typical disturbances that were recognized included thaw settlement, slope instability, and drainage pattern disruptions. Both pre-construction and construction activities offer opportunities for disturbance. Pre-construction disturbance would be mostly be associated with surveying activities and drilling. Construction activities that would disturb the terrain include site clearing, ditching operations, camp construction, disposal sites and others.

To address these issues, environmental studies were undertaken to identify the sources, causes and effects of disturbance and then to develop plans for mitigation. An Environmental Protection Plan was developed, that in part considered the following:

- environmental specifications and construction guidelines (including site-related specifications and guidelines, environmental inspection and reclamation logistics);
- maintenance and monitoring (including slope stability, thaw settlement, revegetation and erosion);
- contingency plans (including fuel spills, oil spills and forest fires); and
- environmental awareness program (including training of environmental inspectors and contractor environmental awareness training).

4.8 Drainage and Erosion

Two aspects of drainage and erosion were addressed in the design of the pipeline. First, to inhibit excessive groundwater seepage within the pipeline ditch that could lead to the migration of soils, the formation of voids around the pipeline and thermal erosion, ditch plugs were to be constructed on slopes steeper than 4°. These plugs consisted of two types. The standard type, used almost exclusively, was a barrier of sand bags with bentonite placed in and around the sand bags and over the up-slope face. The second type was a plug consisting of sprayed urethane foam, attempted experimentally (without success, due to the development of significant shrinkage cracks).

The second aspect of controlling drainage and erosion was to address overland water movement following construction. Three issues were addressed; slope contouring, drainage and erosion control structures, and control of eroded sediments. One important consideration was that any natural surface drainage entering the right-of-way must be able to be directed off the right-of-way as quickly as possible. Allowances had to be made to permit cross flow and mound breaks were provided at obvious low points.

The usual drainage control structure took the form of drainage berms. Table 4.5 provides that distance between berms for a variety of slope grades.

Table 4.5 Spacing of Diversion Berms on Slopes

Slope Grade (%)	Distance Between Diversion Berms (m)
< 5	100 - 500
5 -10	50
10 - 15	25
15 - 20	17
20 - 25	12

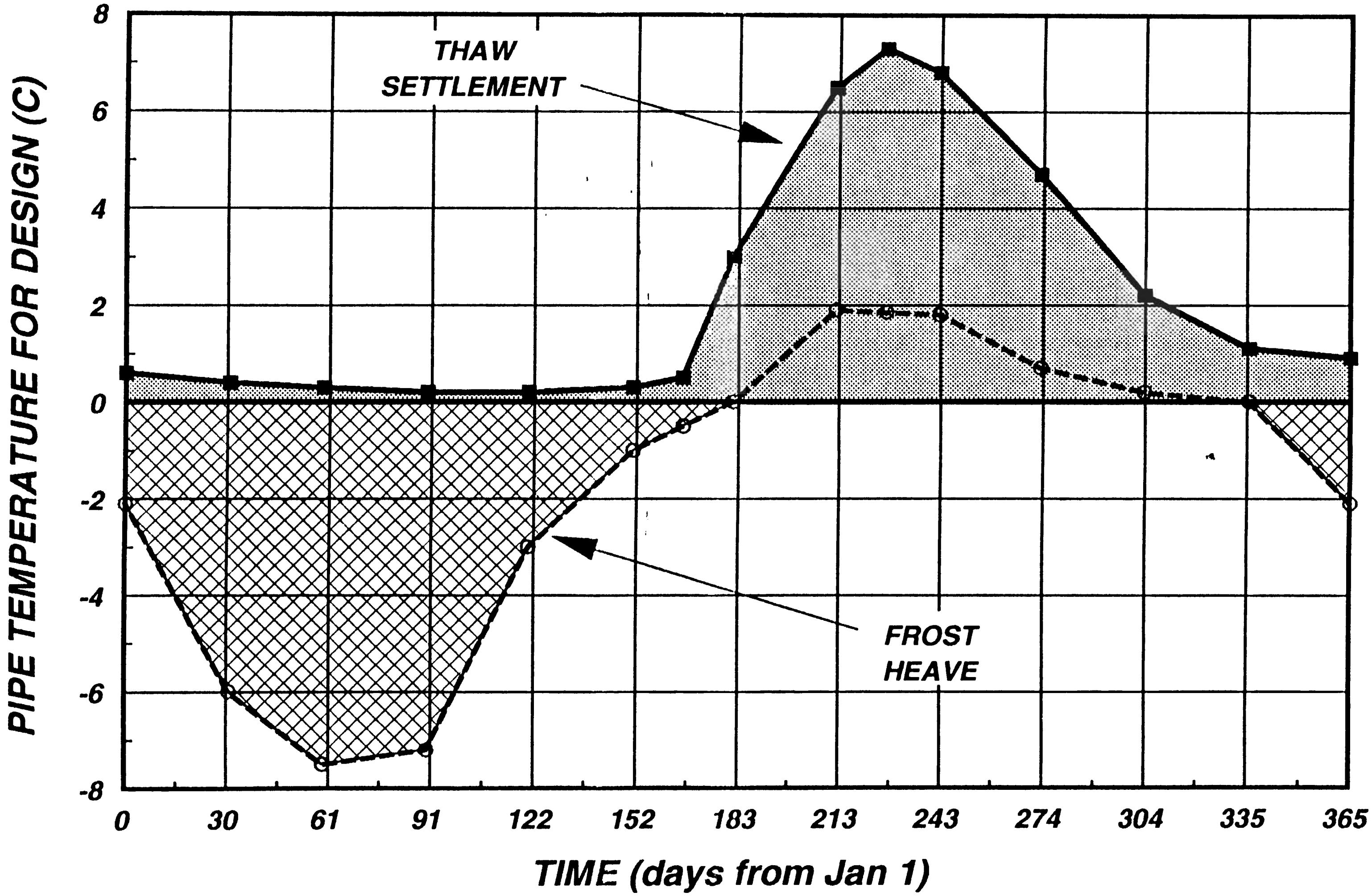
4.9 River Crossings

Each significant river crossing along the pipeline was individually designed. The design process consisted of establishing river bed (thalweg) profiles and river bed and bank cross sections. Based on historical air photos, where available, the migration of the banks were assessed and the "sag-points" were selected. A minimum of 2.5 m cover beneath the thalweg was specified for all design stream crossings.

The hydrotechnical design of the river crossings was based on a flood discharge associated with a return period of 100 years. With the exception of the larger rivers (Great Bear, Mackenzie, and Kakisa Rivers) historical stream flow data was not available and was estimated based on catchment area and environmental data. In the absence of a specific regulatory design criterion, the 1:100 year design return period for oil pipelines was considered a prudent approach to the issue. That criterion has, and continues to be acceptable to the regulatory authorities.

For the construction phase, installation procedures for both winter and summer construction were prepared for the major crossings.

Site specific designs for fifty seven river or stream crossing were undertaken by the designers. In twenty cases, the crossings were of minor concern and a typical design was developed and applied.



PREDICTED FROM 1-D SIMULATIONS
FOR PIPE IN UNFROZEN OR FROZEN TERRAIN

FILE: TP-DES

FIGURE 4.1
PIPE TEMPERATURE VARIATIONS USED IN DESIGN FOR THAW SETTLEMENT AND FROST HEAVE

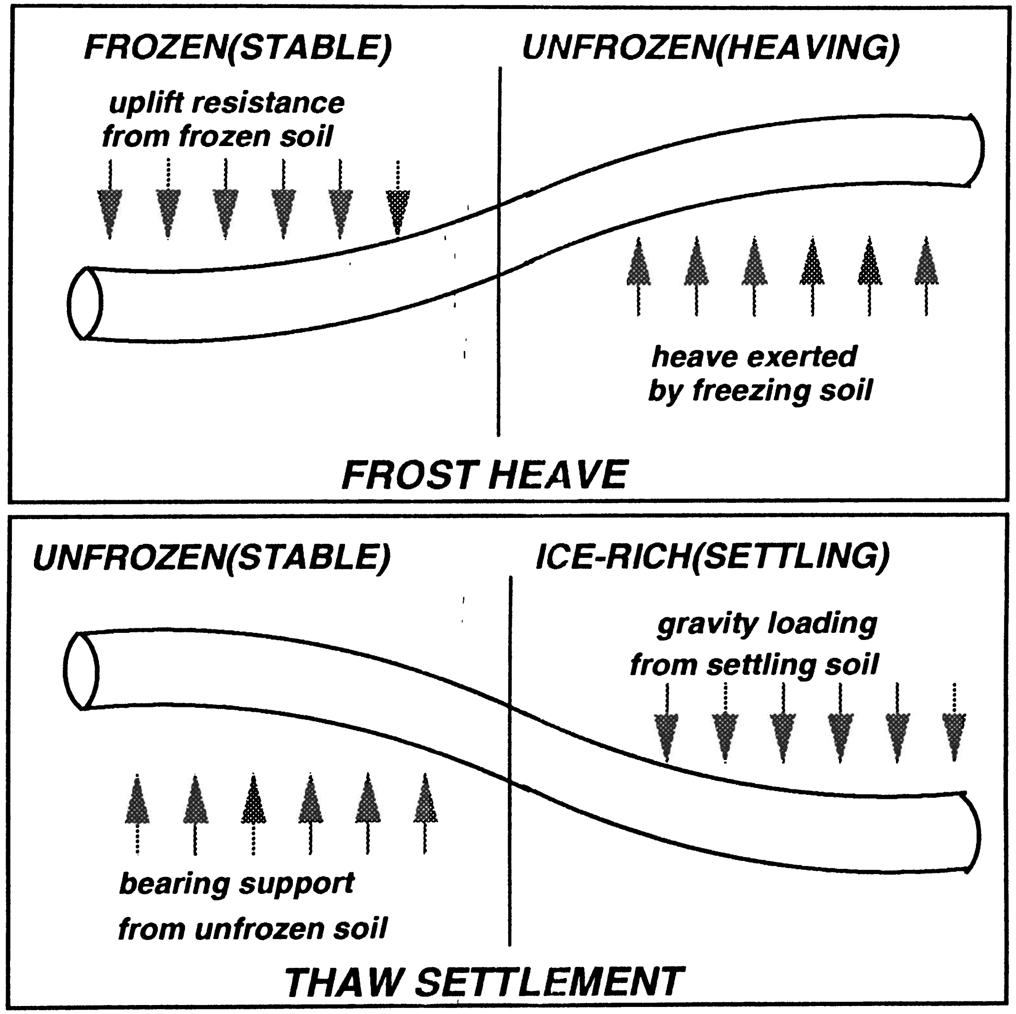


FIGURE 4.2

FREEZING AND THAWING EFFECTS ON PIPELINE IN DISCONTINUOUS PERMAFROST

FILE: DIFF-H-S

5.0 PERFORMANCE MONITORING (ACTUAL IMPACT)

5.1 Pipe/Ground Thermal

5.1.1 General

Observed pipe temperatures have generally fallen within the range of pipe temperatures used for predictive purposes during design, as shown for the first few years of operation on Figure 5.1, from MacInnes et al. (1990). This range of temperatures is quite wide, however, and was originally intended to represent the widest probable range of ground temperatures at pipe depth that might influence operating pipe temperatures. Further, the range of originally anticipated pipe temperatures could not account for the revised temperature excursion conditions that have been in place since 1993.

The delivery of crude oil to the pipeline was hampered in the first number of years by problems with the chillers. The producer, Imperial Oil Resources Canada Limited, was required to make modifications to the chilling equipment in order to improve efficiencies. Waxing of equipment was a particular problem.

Previous methods of predicting temperatures along pipelines in northern regions have assumed a relatively simple approach assuming steady state heat flow from the pipe to the ground, as shown on Figure 5.2. The original line temperature calculations for design for the Norman Wells pipeline required two values for soil thermal conductivity and the ground or ambient temperature surrounding the pipeline. Recognizing that these values could vary throughout the year, monthly or quarterly values for these two geotechnical or environmental parameters were estimated and provided to the designer. A computer based numerical model was used to determine the resulting temperature profiles along the oil pipeline. The profiles themselves were each assumed to be a steady state profile, valid for the instant of time corresponding to the supplied ground temperature and thermal conductivity. The simplified inputs for this idealized model are included on Figure 5.2. Some of the inputs include the thermal and physical properties of the surrounding soils, thermal properties of the oil, and flow of oil.

The simplified method, however, ignores the transient effects of previous temperature excursions imposed by the pipe on the surrounding terrain. In particular, in northern pipelines the pipe may cause freezing or thawing around the pipe at different times of the year, and the simplified exponential solution outlined above cannot account for the heat lost or gained during freezing or thawing cycles. Furthermore, there can be considerable difficulty in selecting which characteristic ground temperature, T_g, to use in the analysis. The ground temperature is typically estimated for different times of the year, for some elevation above the pipe, and these are provided as the characteristic temperature to use in the above calculations. In fact, it is now known (Nixon and MacInnes, 1996) that the pipe does not lose or gain heat to the ground temperature at any one elevation in the soil around or over the pipe. The pattern of heat flow can be much more complex than this, even for uniform soil conditions, particularly during periods of freezing or thawing.

Pipelines can be broadly classified into two categories, namely high energy and low energy input cases. For a high energy input pipeline (e.g. Trans-Alaska pipeline), the thermal regime in the soils around the pipe are strongly dominated by the temperatures imposed by the pipeline itself. These pipelines are typically high flow rate, larger diameter cases, where there is little dissipation of temperature with distance along the pipeline. Low energy pipelines, on the other hand, typically have low flow rates and are smaller in diameter, and adapt relatively quickly to the surrounding temperatures. The latter are also known as "ambient temperature" pipelines.

Once pipe temperature profiles are predicted using the above simplified method, They are subsequently input to a 2-D geothermal model (e.g. Nixon, 1983; Hanna et al, 1995) to predict the effects of the pipe temperatures on thaw depths and geothermal regime.

This analysis has been considered adequate in the past for assessments of thaw depth, thaw settlement, frost heave, etc, considering all of the other uncertainties involved in such analyses.

5.1.2 Detailed Pipe Temperature Simulator

Initially, the incentive for development of a more rigorous pipe line temperature model was related to a study of pipeline temperatures along the Norman Wells pipeline. The temperature regime along this pipeline has been studied for many years, and an extensive data base is available on the temperature conditions along the 868 km pipeline. (MacInnes et al, 1990; Burgess et al, 1992). Originally, the oil was input to the pipe at Norman Wells at a temperature of around -2°C. In particular, concerns relating to slope stability on a few slopes have received attention in recent years (Hanna et al, 1994), and a fuller understanding of the temperature profiles along the pipeline was considered desirable. In 1993, the oil producer (Imperial Oil Resources Ltd) and the pipeline operating company (Enbridge Pipelines (NW) Inc.) requested and obtained permission to operate the pipeline at warmer temperatures at Norman Wells during the summer season. This provided further incentive to more correctly predict and understand the downstream effects of warmer pipe operation on a seasonal basis.

A more rigorous analysis is now available for prediction of downstream pipe temperatures. This more detailed analysis has been undertaken recently (Nixon and MacInnes, 1996). A schematic illustration of the computational process is given on Figure 5.3. The pipe temperature model was validated using the above and other measurements. The initial ground temperatures in the first 80 km were assumed to be -1.5°C, based on previous measurements and experience in undisturbed areas, and temperature data from off- right-of-way locations. Results for the temperature match between predicted and observed pipe temperatures at Kp 20 and Kp 79 are shown in Figures 5.4 and 5.5. The model is capable of reproducing many of the shapes and features of the pipe temperature curve observed in the past. In particular, the warming trend with time is apparent in the predictions, which cannot be reproduced by the simple exponential equation method used previously. The very narrow summer peak in the pipe temperature curve, together with the 0°C curtain during re-freezing of the adjacent ground are also evident from the predictions.

Figure 5.6 plots the results of the predictions using the new line temperature simulator against distance along the pipeline, for the temperature excursion of 1996. Most of the observed impacts of the 1996 excursion were likely detectable for the first 50 km of the pipeline route, but probably not any further. The late fall and early winter pipe temperature profiles are of particular interest. The application of the -4°C operating temperature at Norman Wells has the effect of advancing a freezing front along the pipeline. Until the soils around the pipeline start to freeze back, the flowing oil temperatures tend to remain close to 0°C, and only fall below 0°C once a frost bulb starts to form around the pipe. Once the soils around the pipe start to freeze all the way along the pipe (either induced by the pipe, or later from the ground surface), then the flowing oil temperatures are free to fall below 0°C.

The model may also be used to predict thaw depths at specified locations and results can be used to estimate effects of pipe temperature on thaw settlement, slope stability and frost effects.

5.1.3 Right-Of-Way Pipe Temperatures

Early in the pipe temperature monitoring phase, significant warming of ground temperatures on the right-of-way were noted (MacInnes et al, 1990; Burgess, 1992). Some of these effects were undoubtedly due to clearing, surface disturbance and the presence of disturbed soils or standing or flowing water in the subsided ditch. But it is important to note that some of these effects might be attributed to climatic effects. Figure 5.7 shows a recent plot of mean air temperatures at the AES Norman Wells station. A running mean over a 5-year period is plotted to damp some of the scatter, but not to mask medium term effects. The early 1980's were certainly warmer than average by up to 1.5°C, and this was followed by a cooling period to the late 1980's. It is also of interest to view the larger picture, in which a warming trend of about 1.0°C in 20 years might be interpreted. Future pipeline projects may have to address these apparent climatic warming effects, although it is anticipated they will play a relatively minor role in geothermal design within any 20 year period.

Figure 5.8 shows the running mean pipe temperatures with time for four selected sites along the pipeline. The cooling trend at Pump Station 1 simply reflects the improvements in oil chilling that were achieved in the early years of operation. The sudden increases in 1993 and 1994 reflect the changes in operating limits discussed above. The three remaining sites show a warming trend from 1985 to about 1990, and leveling off or cooling thereafter. This warming is considered to be mainly due to pipe operation and warming of the right-of-way in response to surface clearing, etc. Climate warming was not considered to be a significant influence due to the lack of warming of the right-of-way.

The profiles of average, year-round pipe operating temperature are shown plotted with distance for the entire route on Figure 5.9. After the first 10-20 km, the effects of chilling at Norman Wells diminish with distance, and by 50-80 km they have dissipated completely. The pipe then has a mean annual operating temperature close to +2.0 C. Pump Station 3 seems to cause a significant increase in mean temperature, due to heat input across the pumps. The oil enters the pipe as warm as +6.0°C, and again cools (more quickly than shown on the figure) back to +2.5°C to +3.5°C.

The oil seems to equalize with the environment (soil) temperatures at about +2.0°C in the Northern part of the route, which is close to the anticipated levels for the first 100 km of the pipeline. The equalized annual oil temperature does not warm up with distance, perhaps as much as was expected. The oil temperature increase due to compression at Pump Station 3 seems to be about +3.0°C, and is perhaps somewhat greater than anticipated.

The following conclusions can be made, based on the pipe temperature monitoring and results from the modeling.

- Warming of ground temperatures in the right-of-way took place in the first few years after construction, caused in large part by pipeline construction, and in part by warmer than average ambient or climatic conditions. Warming of ground temperatures on the right-of-way has experienced some continuation into the 1990's.
- The more rigorous pipe simulator provides results to the present that are in close agreement with those observed at downstream locations.
- The effects of a short term temperature excursion (such as that applied in 1993) will be detectable for the first 30 km of the route south of Norman Wells. Thaw depths and temperatures are affected for a distance of up to about 50 km for the larger scale operating temperature fluctuations at Norman Wells during 1994 and later.
- It is important to isolate the effects of warmer than average summer temperatures from pipe inlet temperature excursions in the analysis, otherwise the effects of oil inlet versus the ambient environmental conditions can be confused. This is why the model was upgraded in 1995 to incorporate the effects of monthly variations in ambient temperature, oil flow rate and snow cover.

5.1.4 Ditchwall Monitoring For Ice, Permafrost and Soils

During the construction of the Norman Wells pipeline, a continuous ditchwall log was created during ditching for pipeline burial. The ditch was typically 1.2 m deep, and stretched continuously from Norman Wells to Zama Lake in Northern Alberta, a total distance of 868 km through the discontinuous permafrost zone. The ditch was logged by experienced geotechnical field personnel every 50-100 m, depending on changing conditions. Every transition from unfrozen to frozen soil was logged based on visual criteria such as colour change, visual presence of ice or moisture in the ditchwall, etc. Nixon et al. (1991) carefully studied the ditchwall records and compiled a data file containing all of the relevant data pertaining to permafrost distribution. The digital data file and more information are also provided in Geo-Engineering (M.S.T.) Ltd., (1992a,b,c).

The ditch wall log provided a unique opportunity to study the amount and distribution of permafrost along a continuous transect through discontinuous and sporadic permafrost regions in arctic Canada. The number of thermal interfaces per kilometre is an extremely important input parameter for studies relating to pipeline frost heave and thaw settlement in the arctic. In addition, a knowledge of the percentage of frozen ground is important when deciding whether to operate a gas or oil pipeline above or below freezing.

The sources of data used in creating the ditchwall data base are listed below:

a) Daily Progress Reports (UMA - Canuck - Hardy)
b) Field Diaries of Ditch Inspectors (UMA - Canuck - Hardy)
c) Field Ditch Logs (UMA - Canuck - Hardy)

d) As-Built Alignment Sheets (IPL)
e) Borehole Databank (IPL)
f) Geophysical Survey (Kay et al, 1983)

Field ditch logs were prepared approximately at 100 m stations along the pipeline route. If a significant change in soil conditions or a thermal interface was noted, then the ditch was logged more frequently. The ditch was generally logged from 1 to 12 hours after the passage of the ditcher. The pipeline trench was examined visually and detailed information on ice and soil conditions were recorded.

The number of frozen-unfrozen interfaces have been summarized by Nixon et al (1991) by pipeline spread and geological terrain unit. The overall percentage of frozen ground decreases from up to 95% in the north to a low of around 16% at the south end of the study area, as might be expected. This is shown on Figure 5.10. The number of interfaces typically varies between about one and three per kilometre, with the highest number occurring in some of the organic terrain units in the southern discontinuous zone, as shown on Figure 5.11.

The amount of permafrost as evidenced by the ditchwall records appears to be between 80 and 90 % for the first 200 km from Norman Wells. The geophysics and borehole data support this. The ditchwall logs then indicate a decrease in percentage frozen ground to around 40 % from Kp 250-300, whereas the boreholes and geophysics indicate 60-70% in the same area. Around Kp 350, all three data sets agree on the amount of frozen ground. The ditchwall logs and geophysical data then both indicate a steady decline in the amount of frozen ground to a low of 16-18% around Kp 650. This corresponds to an area of low-lying and wet terrain. The route then rises over the Alberta Plateau, with an increase in permafrost distribution up to 40-50%, before falling to around 22% at Zama Lake in northern Alberta.

The mean or typical unfrozen or frozen segment length is around 200 m, but with a significant percentage exceeding 1,000 m. At the low end, very few lengths are given as less than 20 m, and this simply reflects the fact that the method of ditchwall logging was not capable of providing a resolution consistently less than about 20 m.

The ditchwall logs also provided some information on the shape of the thermal interfaces, which the geophysical surveys or borehole data could not provide. Sketches of the interfaces made by ditchwall inspectors on site indicated that the interfaces tended to be quite well defined, with a sharp contrast in colour, texture and markings left on the ditchwall by the ditcher. Interfaces could be vertical, near-vertical and curved, or near horizontal (dome-shaped).

Finally, comparisons were made with the amount of permafrost and number of interfaces as logged by electrical geophysical surveys carried out and published in advance of pipeline construction. There was reasonable agreement in terms of the overall amount of frozen ground, however the geophysical surveys may have over-estimated the number of thermal interfaces in some areas.

There appears to be some correlation between the amount of frozen terrain, and whether the pipeline right-of-way was previously cleared. Geophysical surveys sometimes delineated a higher amount of permafrost than the ditchwall logs indicated, particularly in one area, and this may be due to the greater depth of investigation. This would result in areas being logged as frozen, where long-term deepening of the permafrost table due to previous clearing activities has taken place. There does not appear to be any clear effect of previous clearing on the number of thermal interfaces logged in the pipeline ditch.

There are other data contained in the ditchwall logs, such as peat thickness, soil type, presence of boulders/cobbles, etc. The computer-based data available in GSC Open file format (Geo-Engineering (M.S.T.) Ltd., 1992a,b,c) containing the ditchwall logs also contains information on the surficial geological terrain unit, and this could be correlated in the future with the soil types logged in the ditchwall, to confirm the accuracy of the terrain mapping for the region. Burgess and Lawrence (in press) have completed such a comparison.

5.2 Pipeline Design

Overall, the pipeline performed well for the first 16 years or operation. One pin-hole leak was detected and fixed in May, 1992, with only minor loss of oil. The cause of the leak, which occurred at a weld, is not established. Stress corrosion cracking, and corrosion due to certain elements have been proposed, but no conclusion was reached on the mechanism. The problem has not reoccurred, and it is considered to be an isolated case.

In the early years following construction, one of the more visible issues with the overland pipeline was subsidence of the ditch line. This in itself did not pose a threat to pipe integrity, but could result in ditchline erosion and eventual pipe exposure in some areas. Wishart (1988) reported that around 200 km of the route had experienced some form of subsided pipeline ditch. Around 80 km had experienced subsidence in excess of 200 mm in depth, and this was considered to require remediation. This took the form re-establishment of the ditch backfill mound by placement of new fill, and also placement of diversion berms to re-direct surface water flow. Seeding was used to resist erosion where the subsidence was less pronounced. These are discussed in more detail later. In later years (1990s), subsidence of the ditchline, in organic terrain has become more prominent.

The GEOPIG monitoring has been used on a regular basis since 1989 to highlight any areas where pipe strains have developed. This has proved to be a valuable tool in providing an indication of pipe strains due to bending. It should be noted that axial pipe strains due to soil sliding past the pipe cannot be measured using the GEOPIG.

The accuracy of GEOPIG monitoring has now been confirmed at several locations. At Kp 2.0, the pipe elevation has been carefully surveyed on several occasions since 1994 by NRCan/GSC (Burgess et al., 1998). The GEOPIG profiles for the same area and roughly the same time of year have been obtained from Enbridge. The comparison is shown on Figure 5.12. The absolute position of either profile in space is not known, so a match point for vertical and horizontal scale must be made (in this case at the peak of the profile). Allowing for this shortcoming, the agreement between the two profiles is extremely good, providing support for the GEOPIG monitoring approach.

Other locations where the GEOPIG has identified issues that were later confirmed by physical excavation and/or inspection include Kp 5.2, Kp 300 and Kp 318. At Kp 5.2 the pipeline profile determined by the GEOPIG was confirmed by physical surveying. At Kp 300 and 318, small wrinkles in the pipe were confirmed by physical inspection. Details of these events are provided in following subsections.

5.3 Thaw Settlement

5.3.1 General

As reviewed earlier, the design thaw settlement was 0.8 m in the northern part of the route (Kp 0 - 78), reducing to 0.75 and 0.70 m further south. This was increased to 1.2 m in thick organic terrain.

Material submitted by NRCan/GSC in the January, 1997 annual geotechnical review, and earlier data from the NRCan/GSC 1995 annual report provided the summary of observed thaw depths and settlement of the right-of-way surface to the present, for the northern part of the route given on Table 5.1. The locations of the sites referenced are given in Table B1, Appendix B.

The surface, or right-of-way settlement observed after about 12 years of operation has been generally less than those estimated for 30 years design purposes. Further, the settlement of the pipe should be somewhat less than the right-of-way settlement, as the soil settlement over the pipe base elevation will be included in the right-of-way settlement in the table.

Some direct measurements of pipe settlement have been made, notably at Kp 2.0 by NRCan/GSC. Pipe settlements can also be inferred from survey measurements at 3 other sites monitored by the PTRM group. Figure 5.13 shows the survey elevations at Kp 2.0 as reported by NRCan/GSC and Nixon Geotech (1997a). Although these do not show the total settlement which might have occurred since start up in 1985, the seasonal heave and settlement associated with the freeze-back and thaw of approximately 1.0 m of soil beneath the pipe can be seen. The seasonal movements appear to be in the range of 0.2 m, or about 20% of the depth of soil frozen and thawed each year. The remaining data sources for inferred pipe settlement are provided by NRCan/GSC, and have been included in Appendix B. Figure 5.14 illustrates the observed right-of-way and pipe settlements for 1995 for selected points along the route.

The interaction of the pipeline with thick peat deposits further south along the route may require special attention. In the peat plateau and fenland areas (Site 12B at kp 608.7 for example), the trench has settled considerably over the pipe. The pipe may have settled up to about 1.0 m at this location, close to the design value of 1.2 m in thick peat terrain. However, there may be less concern for pipe strains at such locations, due to the lower soil loadings imposed by these soft, organic soils on the pipe. It is not known how extensive or widespread the situation as shown on Figure 5.15 is along the southern part of the route. It may also be prudent to inspect the available pipe monitoring results to ensure that the pipe is not becoming over strained.

GEOPIG or precise elevation surveys were not run at selected sites since start-up. This is unfortunate, as much valuable information on pipe settlements, heave and associated mechanisms causing movements could have been obtained. Pipe displacement monitoring stations were recommended by some members of the design team, but there was no agreement or where they should be sited to obtain information of greatest value. The more recent information on pipe strains which can be extracted from the survey elevations at Kp 2.0, coupled with examination of GEOPIG runs indicate that the pipe bending strains are not increasing significantly with time, at least since 1994.

A more rigorous comparison of pipe strain profiles from successive GEOPIG runs might reveal some interesting trends at these or other sites. However, this is beyond the scope of this report. Continued future monitoring of pipe elevations Kp 5.2 by the operator will provide further insight into pipe deformation mechanisms.

Table 5.1 Summary of Selected Pipe and Right-of Way Thaw and Settlement Measurements

SITE	NAME	Kp	THAW DEPTH NEAR PIPE TO 1996 (m)	ROW SETTLEMENT FROM SURVEY (m)	ROW SETTLEMENT FROM PVC PIPES (m)	BENEATH SURFACE EST. THAW STRAIN (%)	PIPE SETTLEMENT FROM SURVEY (m)	BENEATH PIPE EST. THAW STRAIN (%)*	COMMENTS
1	PUMP-1	0.02	2.75	0.6	0.5	21.8	0.35	20	PIPE SURVEY IN 1996
KP-2	FREEZE-THAW	2	2.75				0.2	11.4	SEASONAL SETTLEMENT
2A	CANYON CK	19	4.5	0.2	0.3	4.4			
2B	CANYON CK	19.3	0.75	0.15	0.05	20.0			
2C	CANYON CK	19.6	4.5	0.1	0	2.2			
3A	GT BEAR	79.2	2.25	0.3	0.7	13.3			
3B	GT BEAR	79.4	3.25		0.2	6.2			ROW SETTLEMENT TO 1992 ONLY
7A	TABLE MTN	271.2	4	0.2	0.85	21.3	0.2	0.67	
7B	TABLE MTN	272	5.5	0.5	0.9	16.4			
7C	TABLE MTN	272.3	4.25	0.3	0.3	7.1	0	0	ROW SETTLEMENT TO 1992 ONLY
8A	MANNERS CK	557.8	5.5		0.15	2.7			PIPE SURVEY IN 1996
8B	MANNERS CK	558.2	3.25		0.5	15.4			
12B	JEAN MARIE CK	608.7	4.5	1.2	1.2	26.7	1.1	31.4	SETTLEMENT TO 1992 WAS 0.65 m
5A	PETITOT R. N	783	2.75		0.5	18.2			PIPE SURVEY IN 1992
5B	PETITOT R. N	783.3	3.25	0.5	0.5	15.4	0.5	22.2	PIPE SURVEY IN 1992
6	PETITOT R. S	819.5	5.5	0.8	0.8	14.5	0.6	12.2	ROW SETTLEMENT TO 1992 ONLY; PIPE SURVEY IN 1993

Average Beneath Surface Strain = 15.0 Average Beneath Pipe Strain = 14.9

NOTE: Thaw Depth of 5.5 is actually greater than 5.0 m

Source: Burgess (1995, 1997) and Burgess and Lawrence (1997)

5.3.2 Inferred Thaw Strains

The average thaw strain of the soils at monitored sites can be calculated by dividing the observed settlement by the thaw depth. For right-of-way settlement, the appropriate thaw depth is the full depth from the ground surface to the depth of thaw. For thaw strains associated with pipe settlements, the inferred thaw strain uses the observed thaw depth minus one meter. The thaw strains are calculated on Table 5.1, and are generally in the range 10 to 30%, with an average close to 20%. The thaw strain for the limited number of observed pipe settlements is closer to 30%. The original design in non-organic terrain involved about 0.8 m of settlement for around 5 m of thaw beneath pipe base, for an average thaw strain of 16%. This is similar to the average value observed for thaw strain determined for the larger number of sites where right-of-way settlement was observed, as illustrated on Figure 5.15.

5.3.3 Ground Penetrating Radar

Enbridge, working with the NRCan/GSC have also carried out trials on the use of ground penetrating radar in determining thaw depth on the pipeline right-of-way. This was carried out in 1993/94 under a NRCan/GSC-Enbridge industrial partnership program. (See 1995 Annual Geotechnical Review). Burgess et al. (1995) describe the use of the method in determining thaw beneath insulated wood-chip slopes. There are clearly limitations with the method, relating to soil type, depth of penetration, requirements for control by probing or other physical method, and the timing of the survey. Because of these constraints, it cannot be viewed as a line monitoring tool for obtaining thaw depth inputs for updating slope design and stability calculations. In select cases it does have the potential to extend point source thaw depth information in to a three dimensional framework, in a non-intrusive and rapid fashion.

5.4 Frost Heave and Other Loadings

5.4.1 Frost Heave

There has been little evidence of frost heave in areas where the insulated sag bends were constructed. The GEOPIG monitoring has not indicated any significant curvatures at these locations. However, as noted earlier the GEOPIG will not detect axial strains, which could possibly be occurring if heave was taking place beneath a sag bend and pushing the pipe upwards along a slope.

Within the first 5 to 10 km, however, there is evidence of seasonal frost heave due to re-freezing of the thaw bulb formed around the pipe each year. Figure 5.13 shows the seasonal pipe elevation changes at Kp 2. Seasonal movements of up to 20 cm at Kp 2 have been observed by the NRCan/GSC from 1994-1996 (Burgess et al, 1998), and this is undoubtedly due to seasonal frost heave. This should be compared with slightly lower estimates of frost heave of 10 to 13 cm made during the design.

Secondly, at Kp 5.2 the pipe has shown significant uplift movements of 1.0 m or more. This is likely due to an uplift buckling phenomenon, but may have been initiated by seasonal frost heave as discussed in the following subsection.

Thirdly, frost heave was observed during early route reconnaissance along the first few kilometers of the route, as evidenced by cracking and apparent uplift of the backfill mound over the pipe. (Later pipe temperature monitoring and geothermal analysis showed that the pipe would remain below 0°C for about the first 5 km of the route, in the earlier years of operation, and so this observation seems quite reasonable.)

In Section 4.4 it was noted that some sag bends were considered to be susceptible to frost heaving forces. As a result, some pipe joints were insulated and/or instrumented with thermistor beads or strain gauges. The installed instrumentation became inoperative at an early stage of operation. Notwithstanding this, no pipe movements have been attributed to frost heaving except as noted above.

There have been several locations where dents have developed in the bottom of the pipes due to rocks coming in contact with the pipe. The movement of the rocks may be due to frost heaving forces.

5.4.2 Pipe Uplift at Kp 5.2 and Kp 4.8

As noted above, the pipe up lift at Kp 5.2 has received considerable attention. Ground surface and pipe elevation surveys have been carried out by the NRCan/GSC and Enbridge at this location since 1993. The pipe profile and depth of soil cover are shown on Figure 5.16. By summer 1997, the soil cover had reduced to zero at the apex of the uplift section, and was of the order of 0.5-0.6 m more remote from the apex. Figure 5.17 shows the comparison between GEOPIG and precise survey elevations in mid 1996. Once a match point is selected, the shapes of the two profiles are very similar, providing another independent check on the GEOPIG profiling.

A study (Nixon Geotech, 1997a) concluded that this uplift was likely initiated by seasonal frost heave, which can be up to 20 cm in this terrain. This area has a well defined unfrozen-frozen soil interface, and low density organic soils. During geotechnical drilling by the NRCan/GSC at the site in March, 1997, ground water flowed freely above ground surface from beneath the seasonal frost cap, indicating a plentiful water supply. These factors would combine to provide weak, low density soils that would provide low resistance to upward pipe movement. Due to the high axial loads in the pipe resulting from the cold pipe reference temperatures of -30°C (illustrated in Figure 5.18), the pipe may have buckled towards the ground surface.

This pipe movement had certainly taken place by June, 1993, before the revised pipe temperature excursions had been instituted. Therefore, it is likely that this pipe section displaced upwards to a large extent soon after the start of operation. Natural thawing and/or significant pipe temperature excursions in the years 1984 and 1985 would have resulted in a significant thaw bulb around the pipe in these early years. Natural springs on, or off the right-of-way would also provide a natural cause for greater localized thawing.

The surveyed profile at selected times since 1993 is shown on Figure 5.19. The lack of a proper benchmark until 1996 was a shortcoming in pipe elevation monitoring, and so comparisons between profiles must be made with caution. Relative elevations allow an estimate of bending strains or pipe curvature to be made, but do not allow comparisons of profiles from year to year, or allow study of mechanisms of pipe deformation relative to adjacent soil or right-of-way. If the profiles are matched at the right end of the section as shown on Figure 5.19, there appears to be a tendency for increase in pipe elevations with time. Further, there is a seasonal fluctuation that indicates the pipe is highest in early summer, just before thaw, and then settles somewhat towards fall, when thaw is maximum. The computed bending strains based on the second derivative of these elevations are still below the project limit of 0.5%.

The pipeline operators, in the fall of 1997, undertook a remediation program. Although several options were available (physical cutting and replacement, reburial below existing grade, covering) it was considered that covering the pipe would provide sufficient cover to protect the pipeline integrity. The remediation plan is shown in Figure 5.20.

It was concluded that the uplift was likely initiated by frost heave with further movements due to thermal expansion of the pipe. The remediation was expected to provide sufficient soil cover to reduce movements due to thermal forces, but would not be sufficient to arrest frost heaving forces.

One distinct advantage of the covering option was that the survey rods installed in 1995 and used to monitor the pipe movements remained in place and were monitored through 1998. Post remediation monitoring has shown continued seasonal movement, but by December 1998, it was too early to confirm if the upward trend in pipe movement had slowed or stopped.

A similar uplift section was also developing at Kp 4.8, albeit to a lesser extent. There did not appear to be nearby natural springs that could be contributing surface runoff or groundwater. In the fall of 1998, Enbridge covered the exposed pipeline section using the same remediation design for Kp 5.2. No elevation monitoring of this section has been undertaken.

5.4.3 Seismic Events

There have been at least three large earthquakes in the Nahanni area since pipeline operations began (magnitude 6.6 on October 5, 1985; magnitude 6.8 on December 23, 1985; and magnitude 6.0 on March 25, 1988). The October 1985 event was predicted to have imposed accelerations which were essentially similar to those of the Design Probable Event (McRoberts et al, 1986). Enbridge inspected the right-of-way and especially slopes after the event, and no damage was observed. No damage was reported after the other two events. No negative effects relating to pipe integrity or equipment operation were noted.

5.4.4 Buoyant Forces

In the first several years, the pipe was lifted up at a location near Kp 500, due to buoyant forces. The pipe was subsequently lowered and covered with select backfill. No other instances of these forces have been reported.

5.5 Performance of Non-Insulated Slopes

In the original design, it was predicted that after six and 12 thaw seasons, the depth of thaw would be in the order of 4.25 m and 5.6 m, respectively assuming an average initial ground temperature at 5 m depth of -1°C. Temperature and thaw depth data for a number of slopes are plotted on Figure 5.21. Also included on this plot are actual post-construction monitoring results for several slopes. It is seen that, by and large, the observed six year and 12 year thaws are consistent with the predicted behaviour.

The thawing of these ice-poor slopes was not expected to generate excess porewater pressures. Measurements taken by the Enbridge maintenance crews and reviewed by geotechnical engineers have shown this to be the case.

5.6 Performance of Insulated Slopes

5.6.1 Wood Chip Performance

As described in Section 4.6.2, wood chips were to be used to reduce the rate of thawing of some the steeper slopes, in ice rich soils. The as-built wood chip thickness ranged from 0.5 to 2.2 m. The thicker wood chip sections were placed in the second winter construction season (Kp 190 to 326). In the first season following placement of the wood chips, all monitored slopes experienced heat generation, as expected, due to fungoidal action. The maximum observed temperatures were as high as 41°C. For the majority of the slopes the wood chips cooled off and effectively froze back the following winter, with no recurrence of heat generation in subsequent years, as summarized in Table 5.2. Apart from some notable exceptions (discussed below), the heat generation on most slopes has been much less than assumed in the design. After the second thaw season, most thermistor installations registered no heat generation.

Table 5.2 Summary of Annual Maximum Wood Chip Temperatures (1984 - 1990)

	Years Since Installation						
	1	2	3	4	5	6	7
No. of Observations	29	27	26	29	27	26	10
Highest Maximum Temp., °C	40.7	34	8	9.5	11.8	8.7	6.5
Lowest Maximum Temp., °C	6	0	-0.2	-0.2	-0.3	-2	-0.3
Average Maximum Temp., °C	27.6	11.5	2.6	2.9	3.7	2.3	2.2

In the spring of 1986, slope reconnaissances showed that hot wood chips had persisted through the winter on eight slopes. An investigation revealed that the likely causes were thicker than specified layers of wood chips, and that greater quantities of aspen, versus spruce chips had been used. It was also found that the heating was often confined to limited areas, sometimes only several metres in diameter.

These areas with higher fungoidal activity were thought to have been triggered by wood rot in the original wood chips.

Several remediation strategies were initiated to attempt to reduce the wood chip heating problem. On one slope, cold creek water was sprayed over the wood chips in the fall. On other slopes the wood chips were removed for 30 to 40 days in mid-winter. This action was decided on for two reasons. First, it permitted the cold winter air to freeze back any of the thawed soil beneath the wood chips. Second, the action of removing the wood chips was specifically designed to provide the maximum cooling, by temporarily spreading the wood chips out in a thin layer at the crest of the slope.

A third method was initiated on the south slope at the Ochre River (Kp 286.7). In 1988, ventilation pipes were laid through two isolated hot spots in the wood chips, with vertical risers to promote air circulation. The risers were opened in the winter to permit passage of cold air, and then closed in the summer. The pipes varied in diameter from 200 mm to 350 mm and ranged in length from 15 m to 30 m. Figure 5.22 shows a photograph of the installed ventilation pipes.

In the early 1990's, several wood chip slopes (23 slopes or 41 percent) continued to experience localized hot spots (Burgess et al, 1993, 1995). Most of these were not of concern to the operator and no action was taken, as the general performance of the insulating layer was considered to be satisfactory.

5.6.2 Thaw Performance

The purpose of the wood chips was to retard the rate of thaw. It was recognized that thaw would occur over time. Figure 5.23 shows the thaw depth as a function of wood chip thickness. The solid line shows the predicted 25 year thaw, and two limited sets of data points are provided to show actual thaw depths in 1990 and 1996. The depth of thaw in 1996 has, on many slopes, significantly exceeded the expected amount of thaw. On some slopes the true depth of thawing is impossible to determine because the thawing has exceeded the depth of the available thermistor beads.

There are a number of reasons for the thaw exceeding expectations. The basic geothermal design had been performed for an assumed slope temperature of -1°C. The actual ground temperature was warmer than this on certain slopes. On other slopes, the ice contents may have been lower than assumed in the geothermal design. Another significant influence on some slopes has been a greater than anticipated influence from relatively warmer than expected pipe temperatures. It is likely that generally warmer air temperatures, water ponding, or subsurface flow near the ditch line are contributing to the higher pipe temperatures and hence the amount of thaw. For example, the thawing on the upper portion of the south slope of the Mackenzie River was confined to the wood chips from 1984 through 1992. However since 1992 this thawing has progressed to about 3 m.

As part of the on-going stability review of the insulated slopes, physical probing was carried out on selected slopes. This probing had the advantage over thermistors of determining the shape and depth of the thaw bulb across the right-of-way, rather than determining a single thaw point. The shape of the thaw bulb has implications to slope stability, as is discussed in Section 5.6.3. Figures 5.24 and 5.25 shows the results of probing on several slopes. For these sections, probing across the same lines were performed in the years 1992 and 1996. The results showed a very consistent pattern between the two sets of data, with some general advancement of the thaw.

Porewater pressures are a response to the thawing action. On some slopes, excess porewater pressures were measured during the year. Of particular concern were those pressures measured in the late fall when the thawing had progressed to the maximum depth.

5.6.3 Slope Stability

The shape of the thaw bulb around the pipeline, and across the right-of-way was known to impact the stability of the slope (Hanna & McRoberts, 1988). In addition, the porewater pressures measured throughout the year, but particularly in the fall, at the point of maximum thaw were reviewed. To assess the stability of the slopes, the thaw and porewater pressure data was used in the original slopes design formulation, subsequently referred to as the Design Base Method (DBM). Where the calculated factor of safety dropped below 1.3, based on the original design soil parameters, a more detailed assessment was undertaken. In 1992, five slopes were found to warrant additional investigation. The primary problems with these slopes were the excess porewater pressures and the shape of the thaw bulb, such that the restraining side shear effect was greatly reduced.

In 1996, 17 slopes out of 55 insulated slopes were assessed. Following that review, 10 slopes are on a "watch" list. In some cases, the issue was one of higher inlet temperatures affecting slopes at the north end of the pipeline, while for some slopes it was a lack of reliable temperature or porewater pressure data. New instrumentation was installed at some of these slopes in February, 1997 to address some of these concerns.

The calculation of the factor of safety has been modified during the monitoring program to include other effects. In addition, other stability methods, including a full, three-dimensional stability program and probability have been used (Hanna et al, 1994). The effect of these refinements has been to increase the confidence in the overall stability of the slopes.

5.6.4 Pipe Wrinkle Study at Kp 318

In the fall of 1997, Enbridge ran an inertial geometry tool (GEOPIG) from Norman Wells to Wrigley Station with the purpose of detecting pipe movement associated with slope stability and thaw settlement. The results of the inspection run, when compared to the 1992 inspection run, indicated a vertical strain of approximately two percent at Slope 92 (Kp 318). The data was further analyzed by various experts and it was concluded, with a high degree of confidence, that a wrinkle existed at this slope.

In February 1998 a team assembled to conduct an investigative dig to verify the existence of the wrinkle. In addition to the investigation, monitoring instrumentation on the pipe and surrounding area was also undertaken. It was decided that a winter dig would allow access to the site that would otherwise be impossible during the summer thaw season. The excavation took place during the week of February 23, 1998.

A variety of monitoring instrumentation was installed at the wrinkle and adjacent pipe section, and in the surrounding slope area. The intention was to monitor subsequent movements of the wrinkle, pipe and soil. A summary of the instrumentation installed and results are as follows: (Figure 5.26) shows the layout of the monitoring instrumentation).

<u>At the Wrinkle</u>

- Strain Gauges - These were installed on the pipe wrinkle to measure longitudinal strains.
- Curvature Measurements by Extensometers - These were installed across the wrinkle in order to measure the overall angular changes at the wrinkled section of the pipe. The extensometers were springs in series with strain gauges that were stretched across the wrinkle between two wooden diaphragms. The diaphragms straddle the pipe on either side of the wrinkle.
- Temperature Measurements - Three thermistors were used to measure the temperature profile just above the wrinkle, at pipe level, and just below the wrinkle.

<u>At Pipe Section Adjacent to Wrinkle</u>

- Pipe Deflection Indicators (PDI) - seven of these devices were installed, five up slope of the buckle and two downslope. These were installed directly to the top of the pipe in order to measure longitudinal tilting of the pipe. The instruments consisted of standard slope indicator casing containing groves, housed in an outer aluminum casing which was welded to an aluminum saddle designed to rest on the pipeline.

<u>At the Surrounding Ground</u>

- Slope indicators (SI) - five standard slope indicator instruments were installed on the slope after backfilling and replacement of the wood chips. The locations of these instruments is shown in Figure 5.26. Three of the instruments were placed within 1.3 m of the pipe centerline in order to monitor the original trench backfill zone. One instrument was placed at 2.1 m from centerline, and one at 15.1 m in relatively undisturbed ground. The installation depths ranged between 8.75 m and 13.1 m. Initial readings were taken several days after installation, which served as a baseline. Subsequent readings were taken on a monthly basis.

- Figures 5.27 and 5.28 show the results for two slope indicators. Slope indicator (SI) 2, was located within 2 m of the pipe. SI 4 was located off the "west" side of the right-of-way.

- Shallow downslope movement (north-south) direction continued throughout the study period, albeit at a reduced amount each month. The maximum amount of surface movement near the lower portion of the slope was about 300 mm (SI 2).

- There was generally progressively less downslope movement from the base of the slope to the crest. The depth of the movement zone was also dependent on the location of the slope indicator on the slope. The depth of movement is deeper for those slope indicators near the toe of the slope.

- The slope indicator plots for the instruments on the right-of-way appear to show two distinct movement zones. The upper zone was occurring at depths of 3.5 m, 3 m, and about 2 m in SI's 1, 2 and 3 respectively. This movement was considered to be associated with the backfill. This movement zone was also readily apparent on the east-west movement plots.

- The second movement zone appears to be at a depth of about 6 - 6.5 m, 7 - 8 m, and 5 - 6 m in SI's 1, 2 and 3 respectively. Such a movement was not observed in the east-west direction. In the case of SI 2, the movement at depth was in the order of 50 mm after approximately 9 months. No deep movement had been observed at the top of the slope (SI 5) or on the west side, off the right-of-way (SI 4). Therefore, it is considered that this creep/straining zone was concentrated on the cleared right-of-way and on the steepest section of the slope.

- A downslope movement of nearly 300 mm at the top of the wood chips as noted in SI 2 would likely apply some traction to the pipe in the vicinity of the wrinkle.

- In SI 4, located off the right-of-way, very little movement was observed through the summer and fall. With the active layer thawing, the downslope movement of about 30 mm presently observed is considered "normal" active layer movement.

- Settlement Plates (SP) - fourteen steel plates with vertical risers were installed adjacent to the pipeline to monitor vertical ground movement.

- Thermistor Strings (TS) - two thermistor strings were installed on the slope in order to provide information on the temperature profile of the soils underlying the pipeline, and off the right-of-way, see Figure 5.26. Figures 5.29 and 5.30 show the thermistor data. Thermistor 97-13 is located about 3 m off the right-of-way on the west side. The temperatures for the entire depth to 8 m shows the ground to be marginally frozen to a depth of 8 m. The ground temperatures on the right-of-way are shown on Figure 5.30. The upper 4 m of soil display the seasonal variations in ground temperatures. Marginally frozen conditions are observed from about 4 m depth to about 7 m depth. Below 7 m the ground temperatures appear to be warming. This suggests that within the right-of-way the permafrost may be degrading and is present over a relatively thin thickness. It is interesting to note that the lower movement zone observed in SI 2 (Figure 5.27) corresponds to the approximate base of permafrost observed in Thermistor 98-B.

A summary report was prepared and submitted to the National Energy Board in August 1998 (Interprovincial Pipe Line (NW) Ltd., 1998).

Plans were prepared by Enbridge to replace the section of pipe at Kp 318 in the February, 1999. The replacement program would require shutting in the pipeline over a distance of about 20 km for several days. As a result of the excavation on the slope, most of the instrumentation installed in 1998 would be destroyed. Some of the slope indicators and thermistors were planned to be replaced after the excavation and cut-out was completed.

5.7 River Crossings

The performance of all river crossings has, for the majority of the sites and years since construction, being highly satisfactory. A few significant events occurred following the construction of the pipeline that required remedial action for pipeline maintenance.

A major storm the occurred in the region of Wrigley to Fort Simpson from June 28 to July 2, 1988. The damage to the pipeline and right-of-way included:

- exposure of about 30 m of the pipeline at the south bank of the Ochre River (Kp 286),
- a washout of a rock armoured dyke at Hodgson Creek (Kp 305) constructed in 1987 to protect the north sagbend area following an earlier storm runoff, and
- washout of diversion berms on the right-of-way south of Hodgson Creek.

As a result of the damage, remediation options were prepared and evaluated by the consultants and the owner. At the Ochre River, it was decided to rebury the pipeline. An additional 0.5 m of cover was provided to counter the impact of potential channel degradation if the subchannel along the south bank develops into a major channel.

At Hodgson Creek, Enbridge also decided to rebury the pipeline, rebuild the original diversion berms, and add several new berms and a channel plug near the point where the overflow commences.

5.8 Right-of-Way Disturbance

Enbridge was required by the National Energy Board to submit aerial photographs of the entire pipeline route, together with an analysis of ground conditions on the right-of-way, following the end of the first and third years of operation. The purpose of the study was to document the vegetation cover and major physical conditions in terms of ditch line subsidence, flooded areas and eroded areas.

Generally, by 1988, most of the pipeline route (88%) had a good vegetation cover, which had increased slightly since 1986. The highest cover was on mineral soil terrain (moraine or lacustrine) while the lowest cover was on organic (bog) terrain.

Two significant forest fires have impacted the right-of-way in the past 12 years. In 1994, a forest fire initiated by an electrical storm, burned an area parallelling approximately 90 km of the pipeline right-of-way. Of this length, only 20 to 30 percent of the right-of-way was damaged. In 1995, a forest fire, initiated by an underground coal seam fire burned an area along 53 km of right-of-way, with about 20 to 30 percent of the right-of-way being damaged (Savigny, Logue and MacInnes, 1995; McNiel, Hanna, Fridel and Babkirk, 1996).

During the actual fires, pumps and sprinkler systems were set up on wood chip slopes in the path of the fires. Water was pumped from nearby creeks to saturate the wood chips. The effect was that only the top 25 mm to 75 mm of wood chips were scorched. The charred wood chips were raked off because of the concern that the now blackened surface would adsorb more solar heat. Some areas adjacent to insulated slopes were hydroseeded to speed the re-vegetation process.

The most significant impact to date has been near Kp 182. The site was burnt in the 1994 fire, and has experienced skin flow slides on the valley wall adjacent to the right-of-way. The route at this section of the pipeline parallels the crest of the valley. Shortly after the fire, helicopter and maintenance patrols noted a number of flow slides developing. It has been hypothesized that one flow slide was initiated in 1994 by water bombing, resulting in the loss of the ground vegetation cover. In 1995, additional flow slides developed. Although the right-of-way and pipeline integrity have not been affected by the fire, a program was initiated to monitor the development of retrogressive slope movement in a number of the slide areas.

Other lessons learned included the need for constructing or expanding fire breaks around valve sites, pump stations and storage areas.

5.9 Drainage and Erosion

As part of the aerial photograph review in 1986 and 1988 to assess re-vegetation an assessment of the physical condition of the right-of-way was also undertaken. The physical conditions were described in four broad categories: no significant features, ditch line subsidence, standing water, and erosion.

By 1988, nearly 700 km (78%) of the route had no significant features. Ditch line subsidence was the most commonly identified (negative) physical condition, but by 1988 represented only 15% of the route length (because of a major winter re-roaching program). The subsidence generally appeared to be shallow, typically less than 25 cm. Other negative physical conditions were relatively minor. Figure 5.31 shows the physical changes in the right-of-way in 1986 and 1988. (No additional specific studies have been conducted in the past ten years.)

As subsidence was found to be most common negative feature, the study also characterized the subsidence on the basis of terrain types. These data are presented on Figure 5.32.

Erosion of the ditch line or right-of-way was a relatively minor problem. The erosion usually occurred where a small stream entered the right-of-way, flowed some distance along the ditch line, and then exits. Most of the erosion features occurred between the Great Bear and Willow Lake rivers on lacustrine, moraine, organic and alluvial terrain types. Where available, sandbags were used to construct flow breakers or berms to attempt direct the surface water (Wishart and Fooks, 1986).

At Slope 29B (Great Bear South) some ditch line erosion of mineral soils occurred in 1987. Piping resulted in the creation of voids under the wood chips, which eventually collapsed (Burton el al, 1995). Remedial work was undertaken in 1992, backfilling the cavity with coarse granular fill. In 1998 additional voids under the wood chips were observed, but no migration of soils was noted. In this latter case ditchline settlement may be occurring.

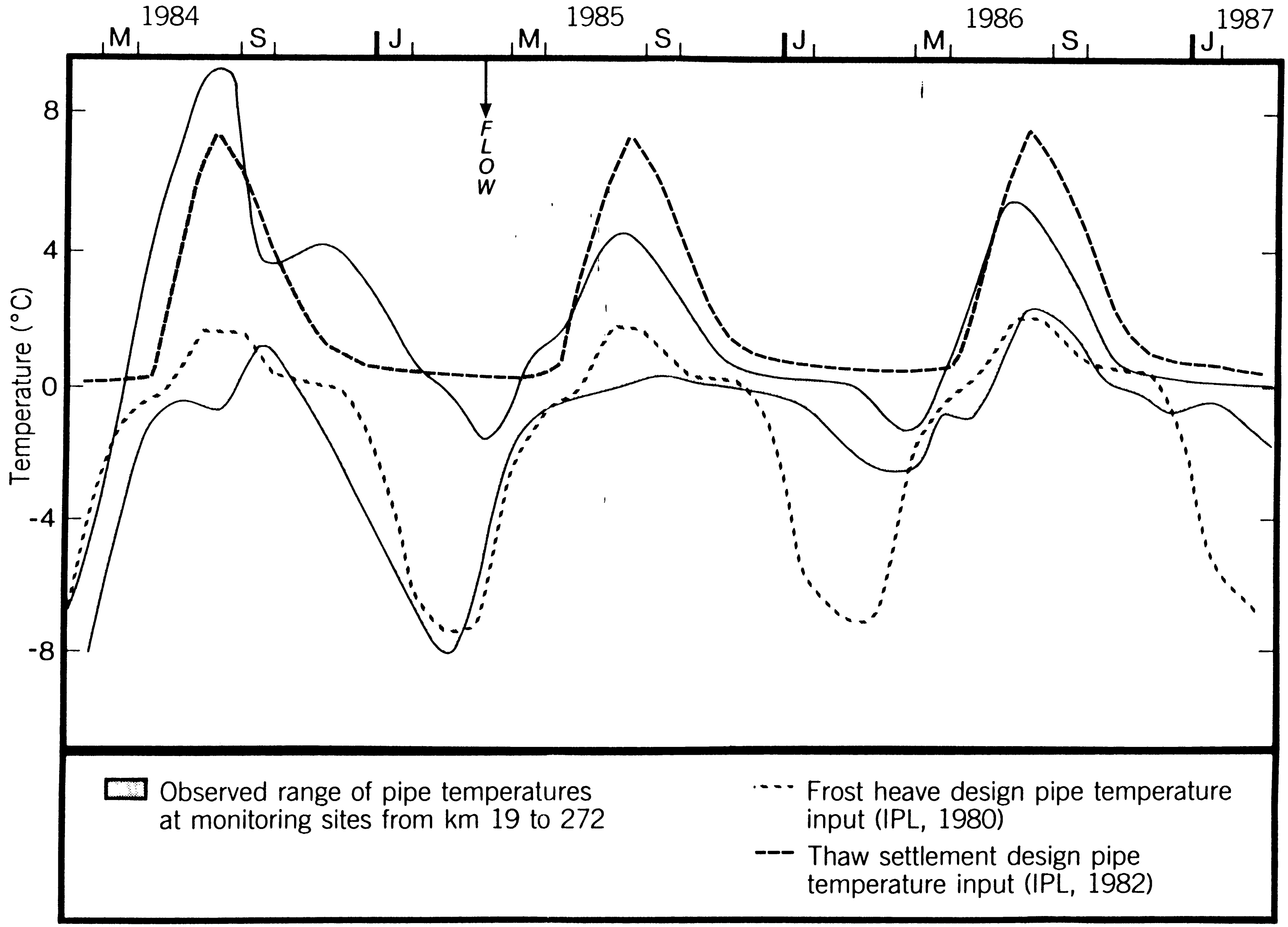


FIGURE 5.1 OBSERVED RANGE OF PIPE TEMPERATURES AT MONITORING SITES from kp 19 to 272 AND IPL DESIGN CURVES (from MacInnes et al, 1990)

FILE: FIG511

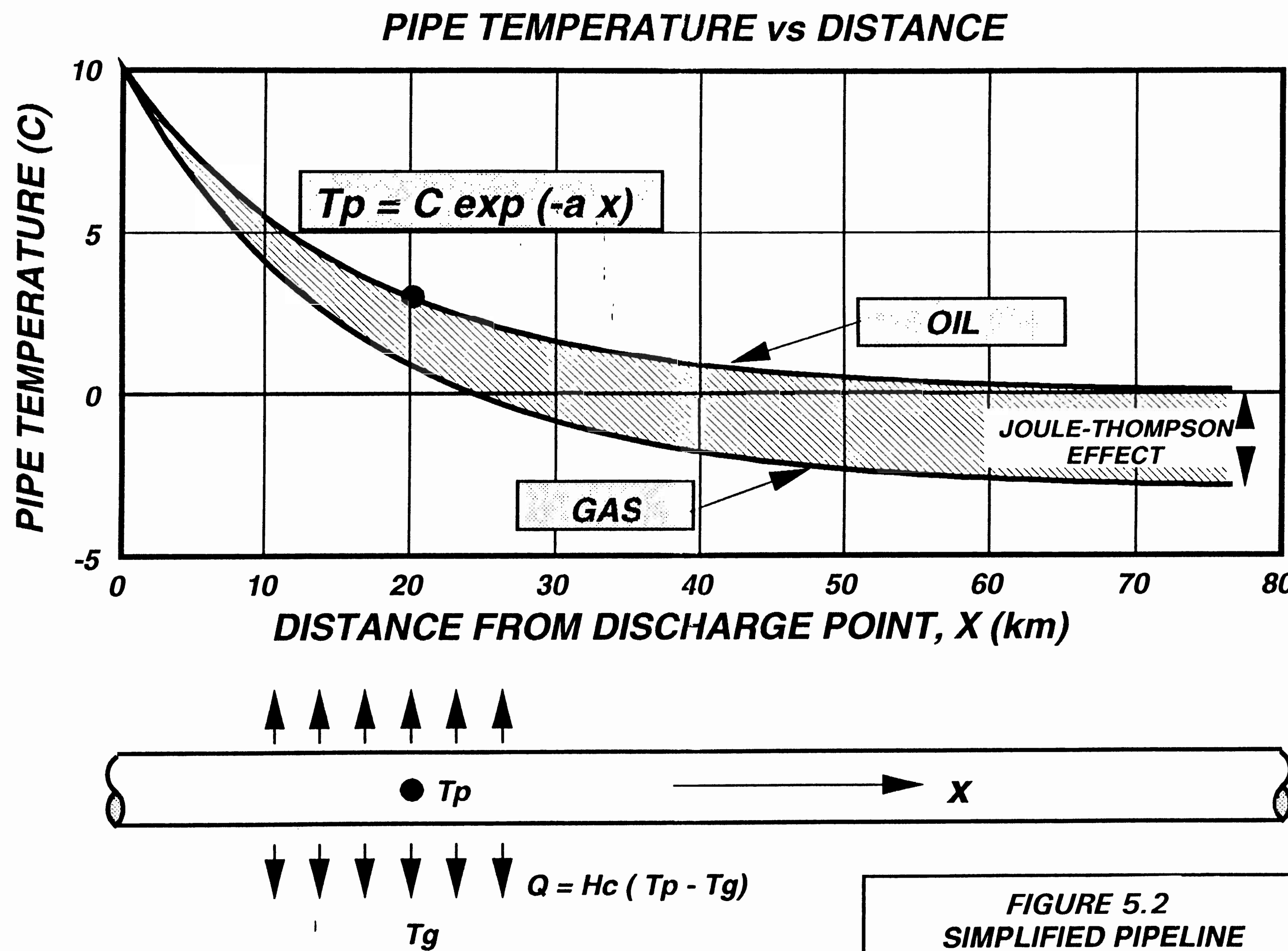


FIGURE 5.2
SIMPLIFIED PIPELINE
TEMPERATURE PREDICTION

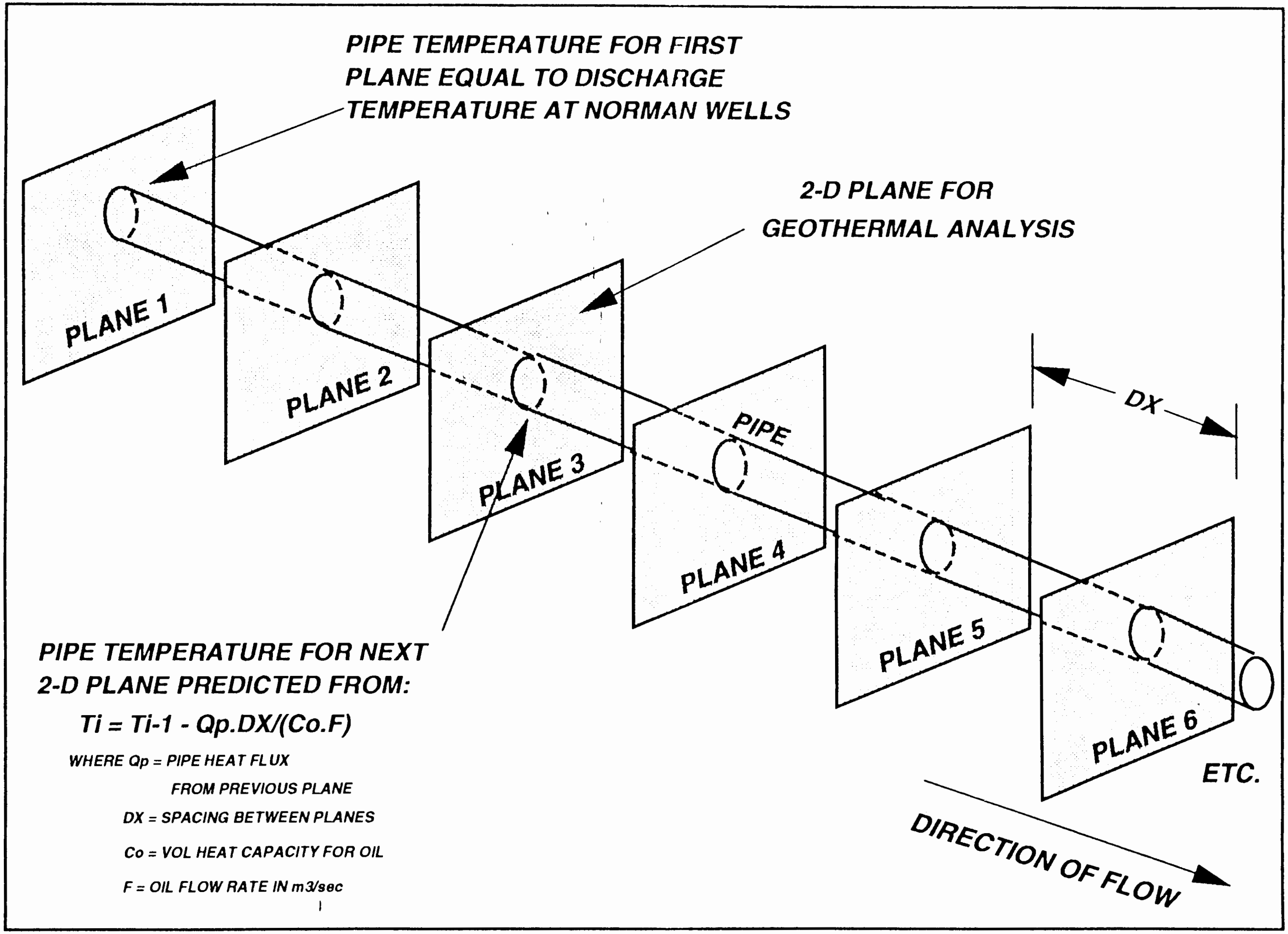


FIGURE 5.3
SIMULATION FOR PIPE TEMPERATURES

FILE: PIP-SIM

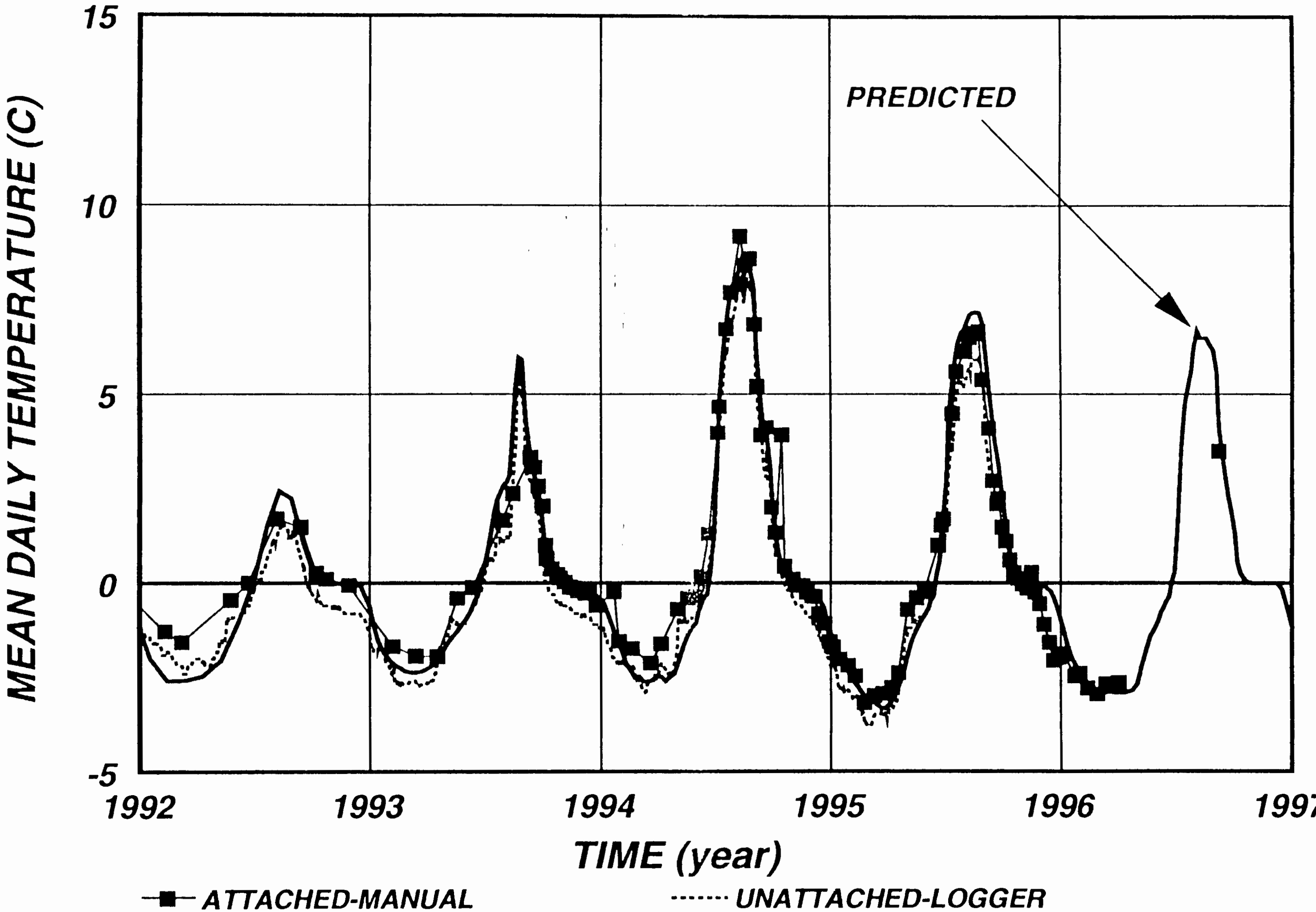


FIGURE 5.4
COMPARISON OF PREDICTED AND OBSERVED
PIPE TEMPERATURE AT CANYON CK KP 19

FILE: COMP-CAN

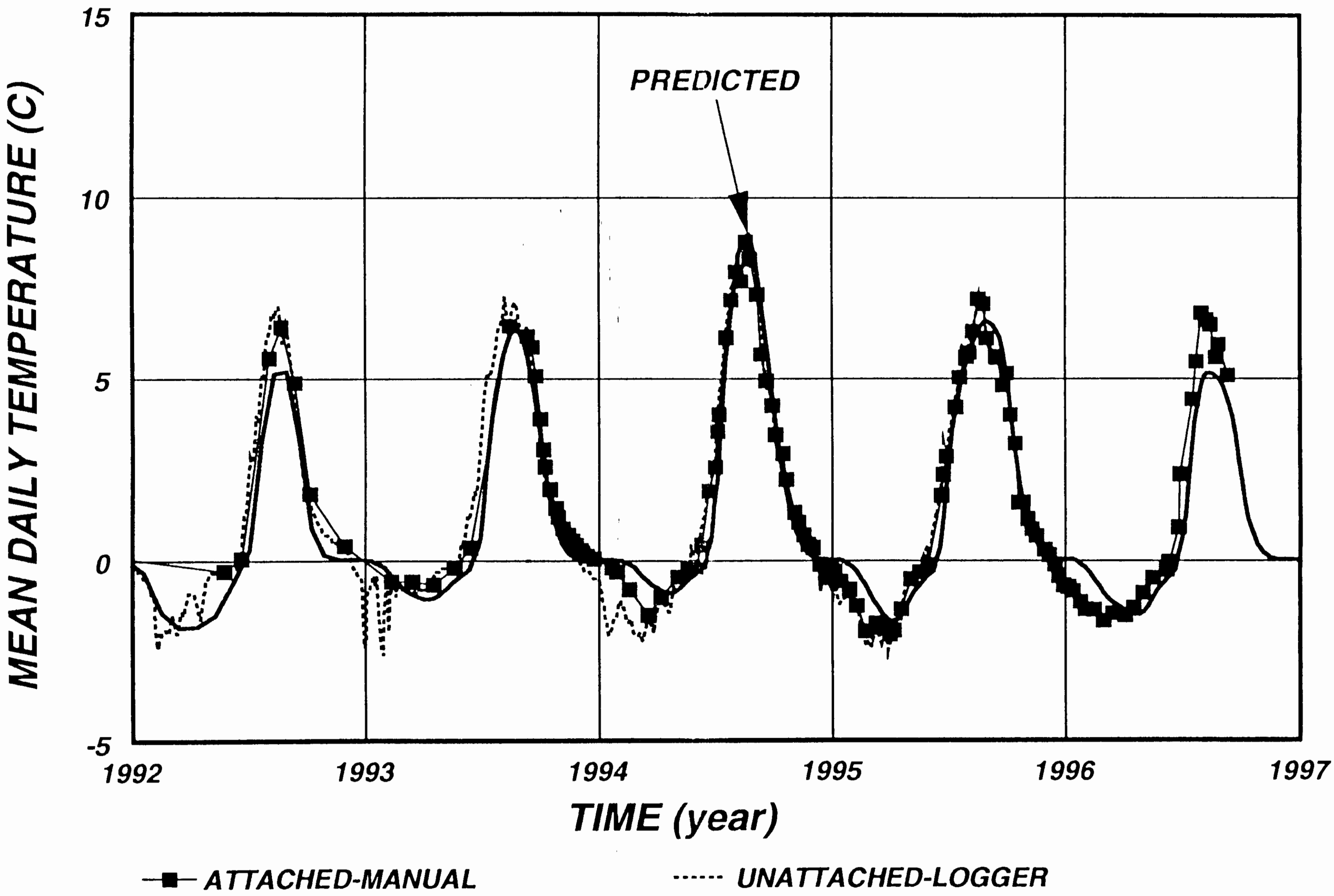


FIGURE 5.5
COMPARISON OF PREDICTED AND OBSERVED
PIPE TEMPERATURES AT GT. BEAR RIVER KP.79

FILE: COMPBEAR

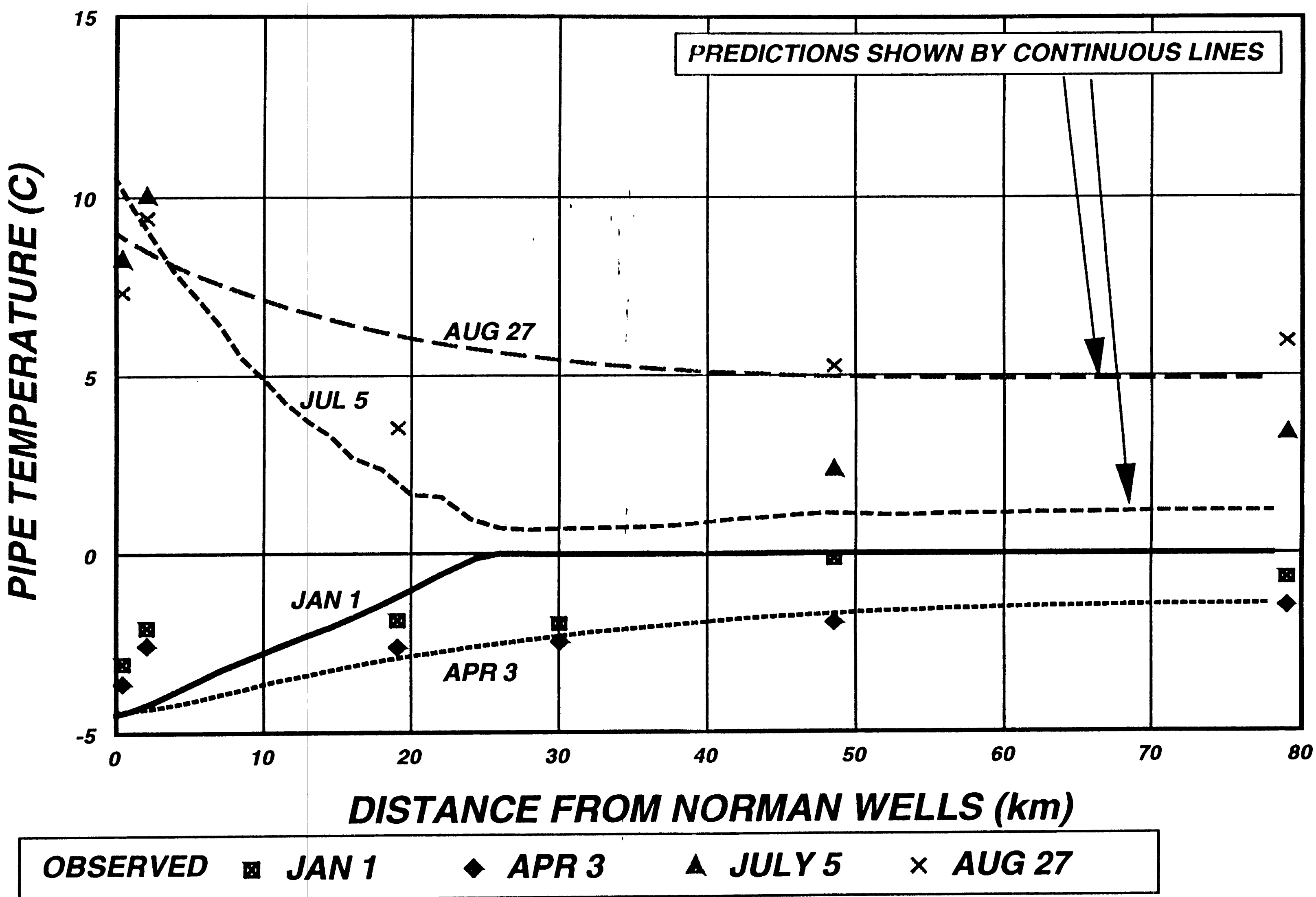


FIGURE 5.6
PREDICTED AND OBSERVED LINE TEMPERATURES IN 1996

FILE: COMPTX96

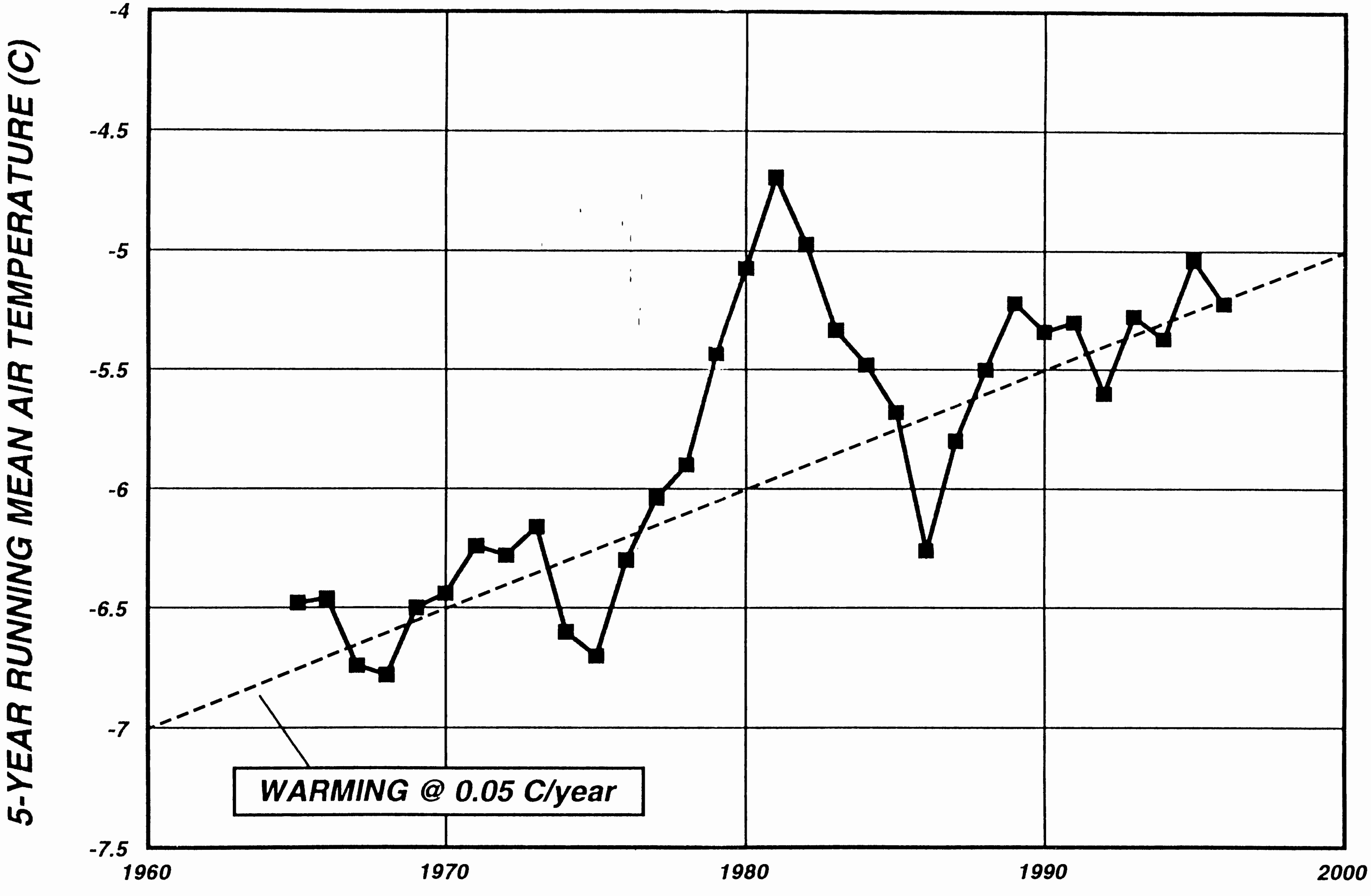


FIGURE 5.7
MEAN AIR TEMPERATURES AT NORMAN WELLS (C)

FILE: TAIR-NW

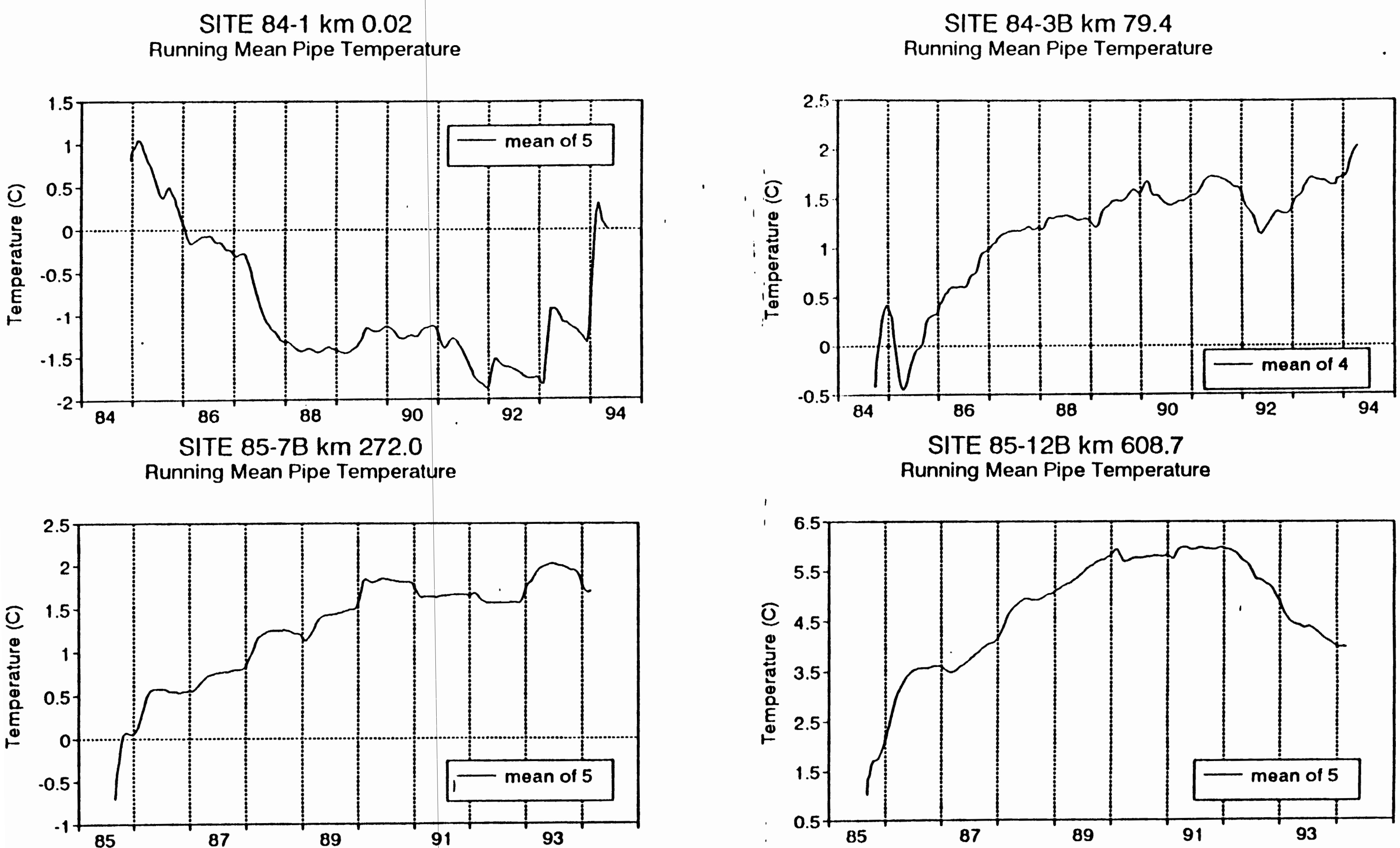


FIGURE 5.8

RUNNING MEAN PIPE TEMPERATURES FROM BURGESS/PTRM (1995)

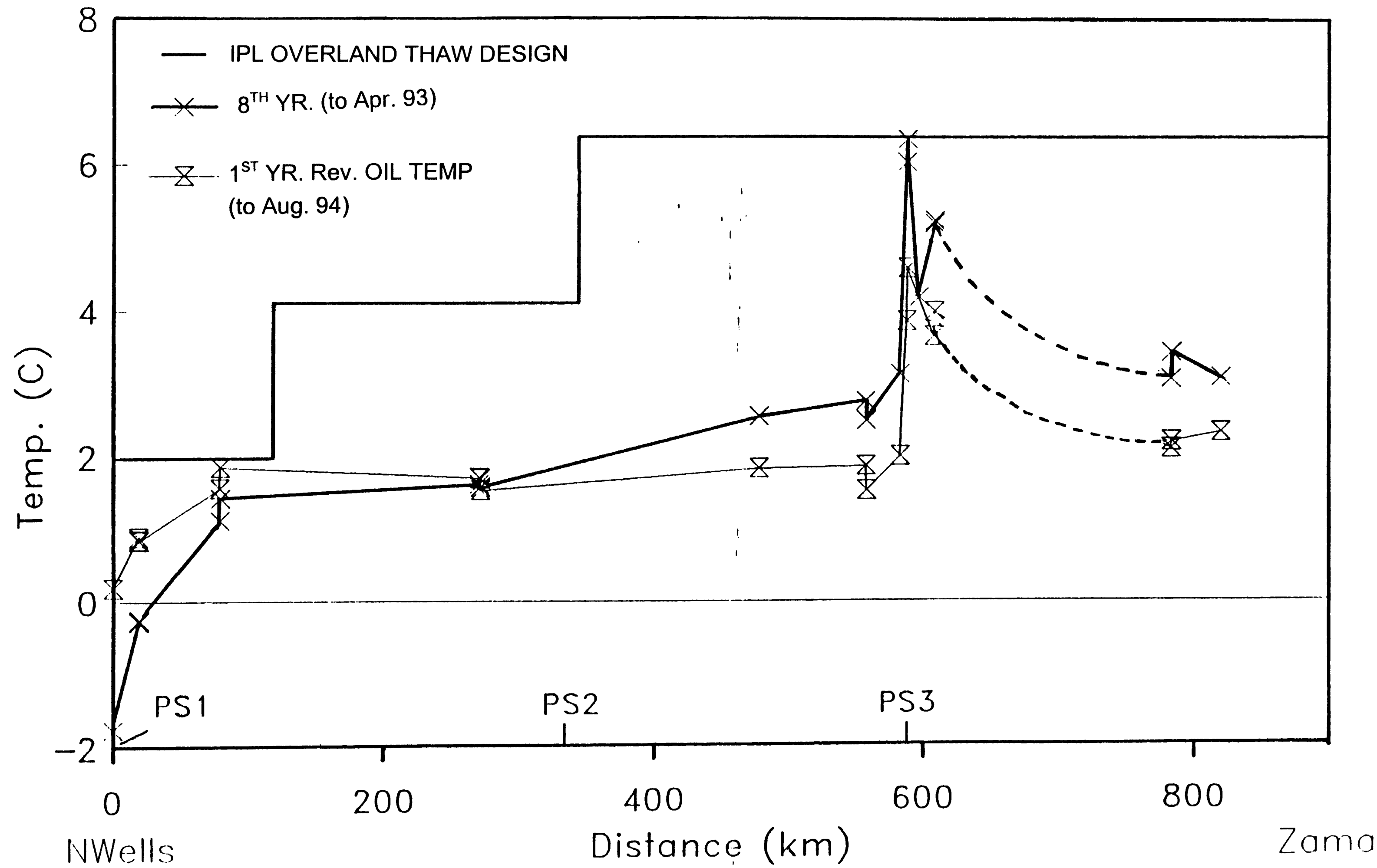
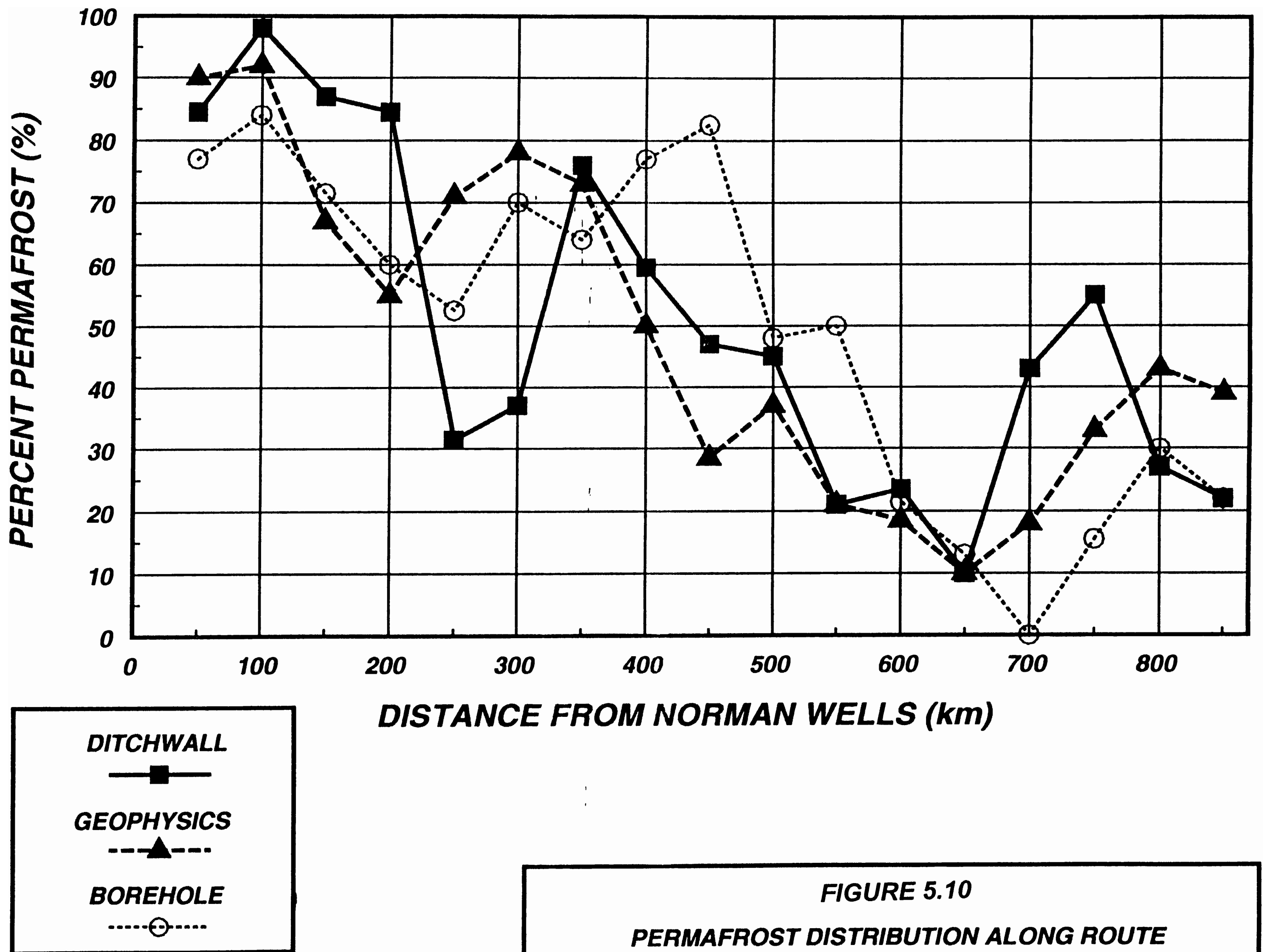


FIGURE 5.9

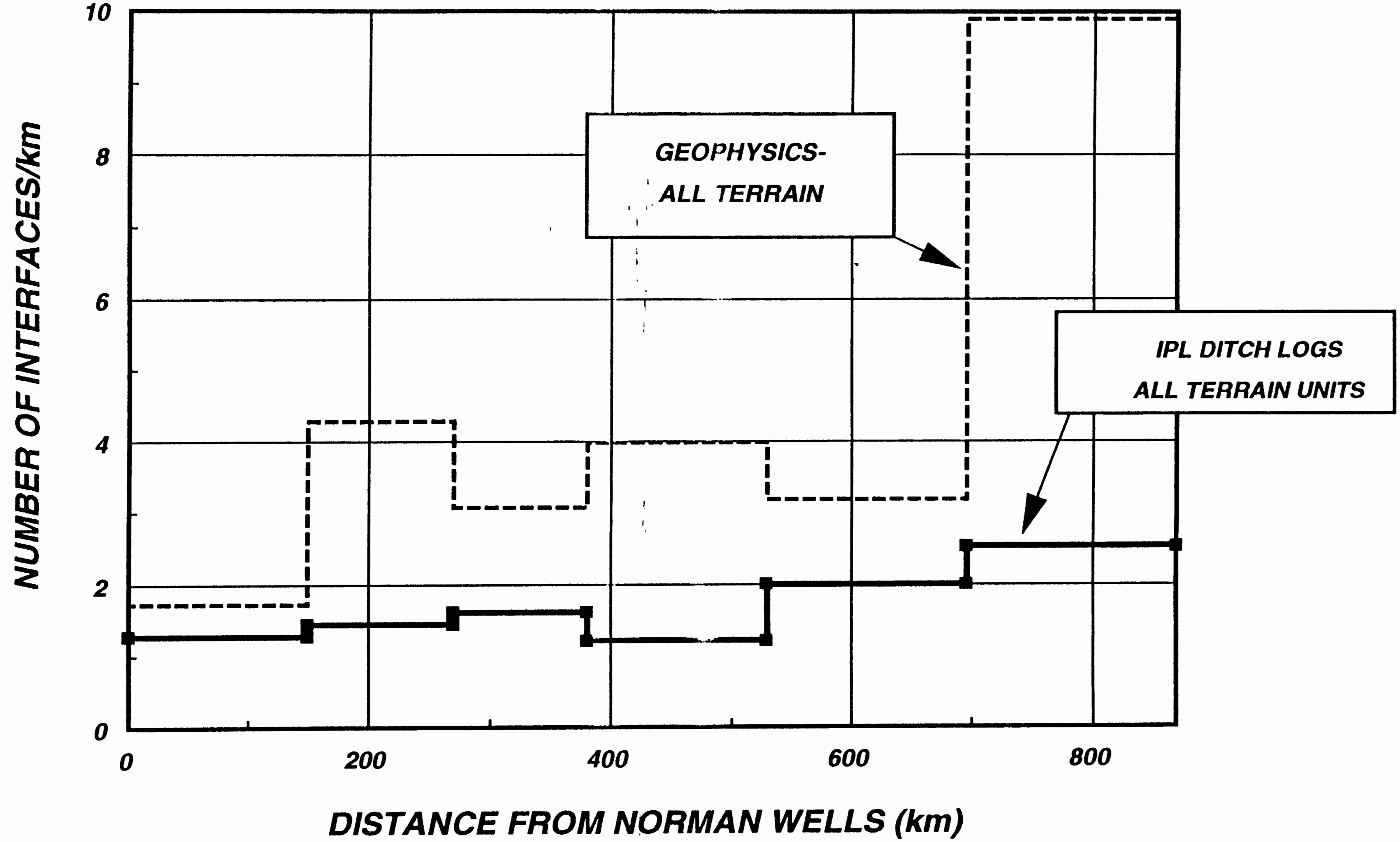
MEAN ANNUAL PIPE TEMPERATURE WITH DISTANCE FROM BURGESS/PTRM (1995)



FILE: PERMDIST

FIGURE 5.10

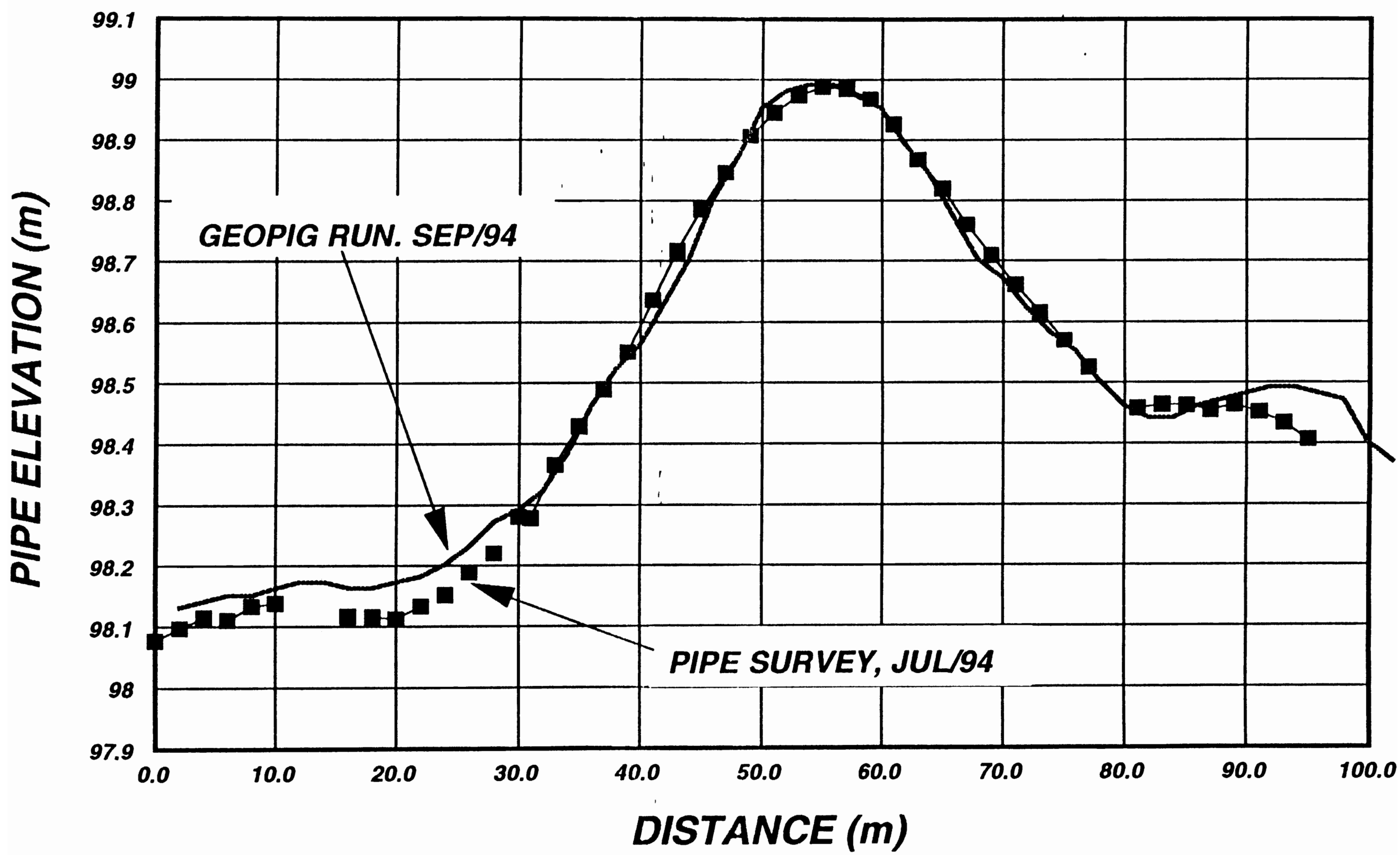
PERMAFROST DISTRIBUTION ALONG ROUTE



IPL SPREAD LENGTHS
1(149.5); 2(119.8); 3(111.1); 4(148.4);
5(167.1); 6(172.4); ZAMA LAKE AT KMP.869

FILE: FRINT-7

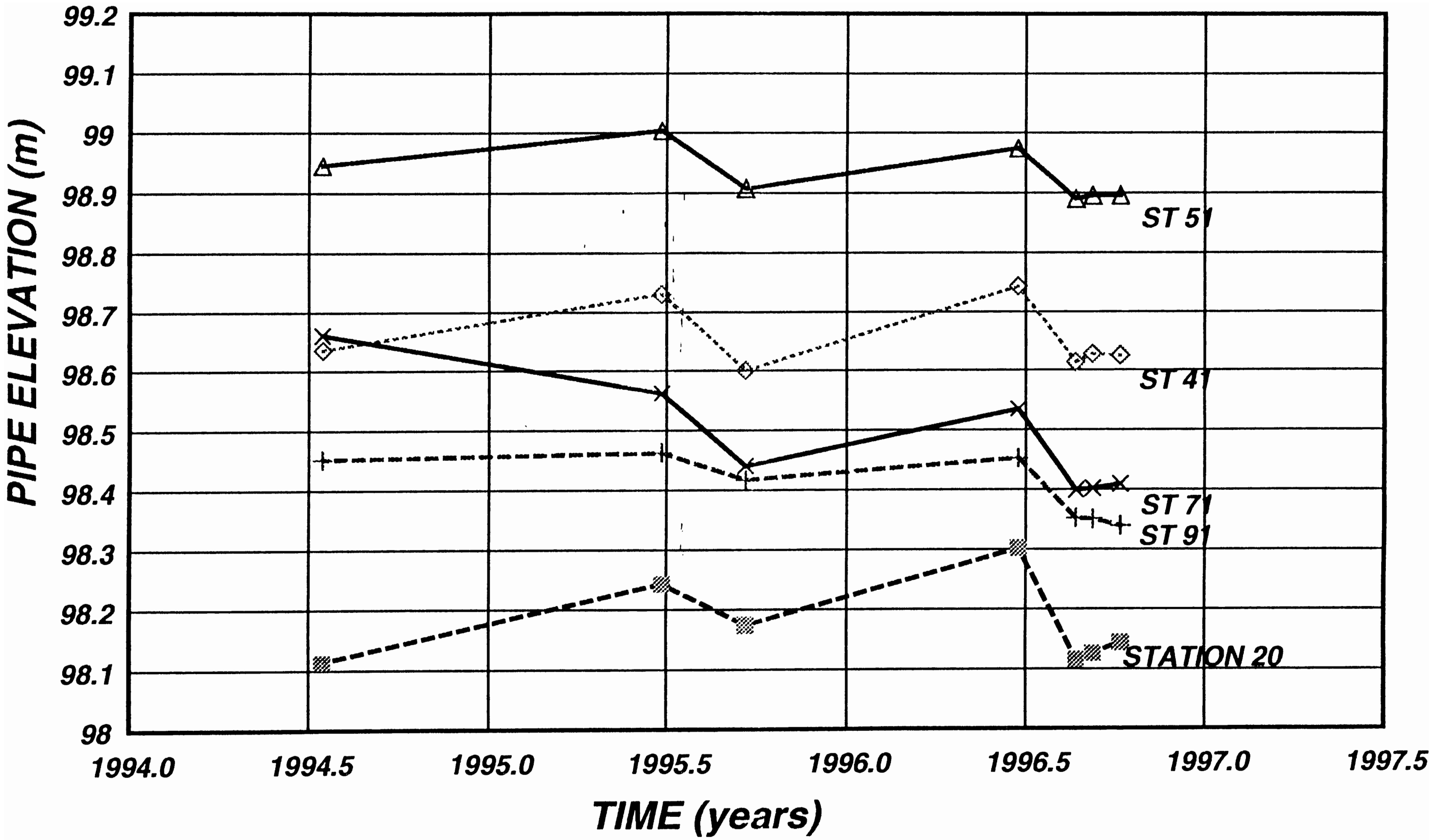
FIGURE 5.11
NUMBER OF THERMAL INTERFACES ON PIPELINE ROUTE
BETWEEN NORMAN WELLS AND ZAMA LAKE



LOCATION AT NRCan TEST SITE AT KP 2.0

FILE: NWEL-COMP

FIGURE 5.12
PIPE ELEVATION WITH DISTANCE IN JUL/SEP, 1994

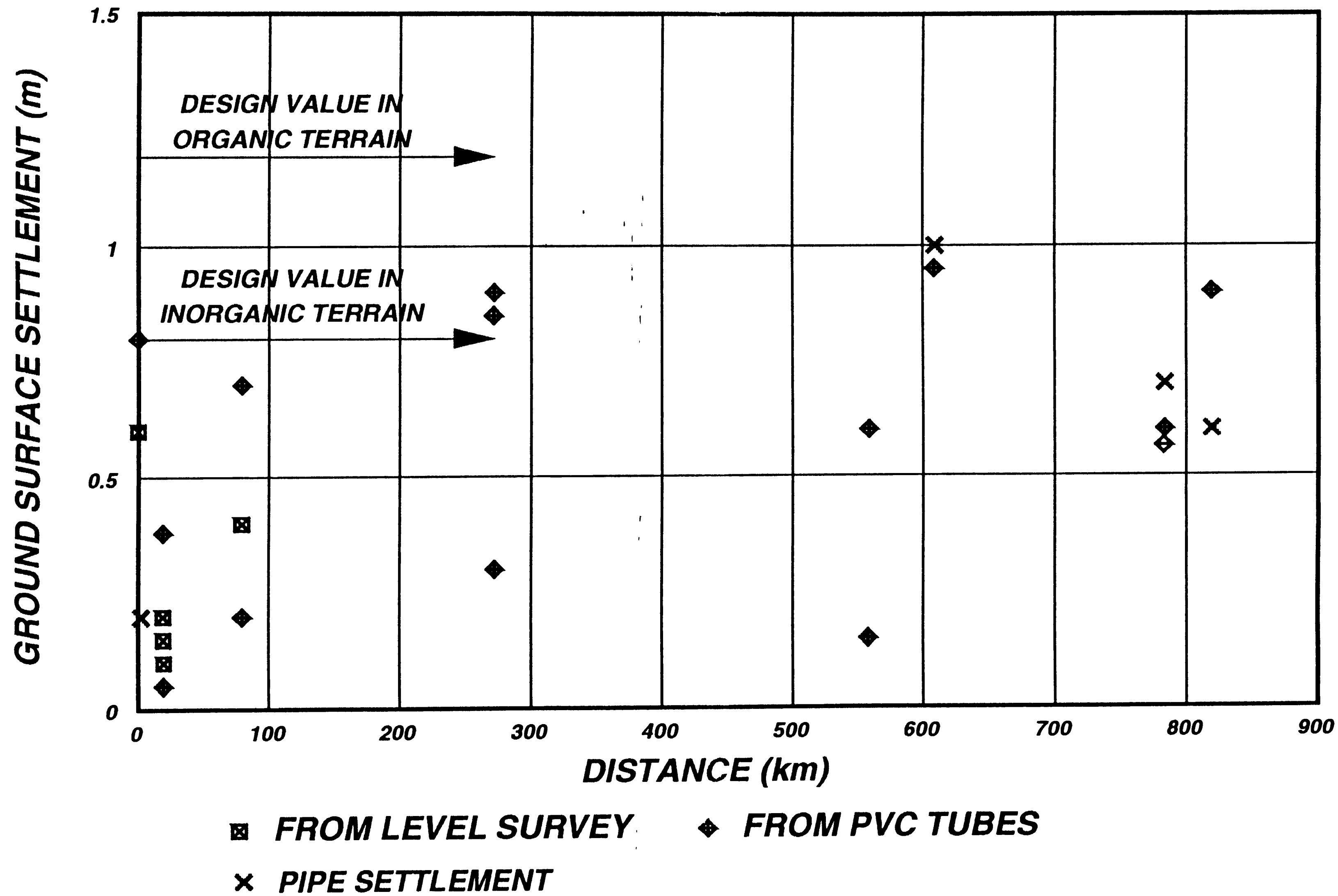


LOCATION AT NRCan TEST SITE AT KP 2.0

EL BENCHMARK = 100.00 m

FILE: NWEL-T

FIGURE 5.13
PIPE ELEVATION WITH TIME

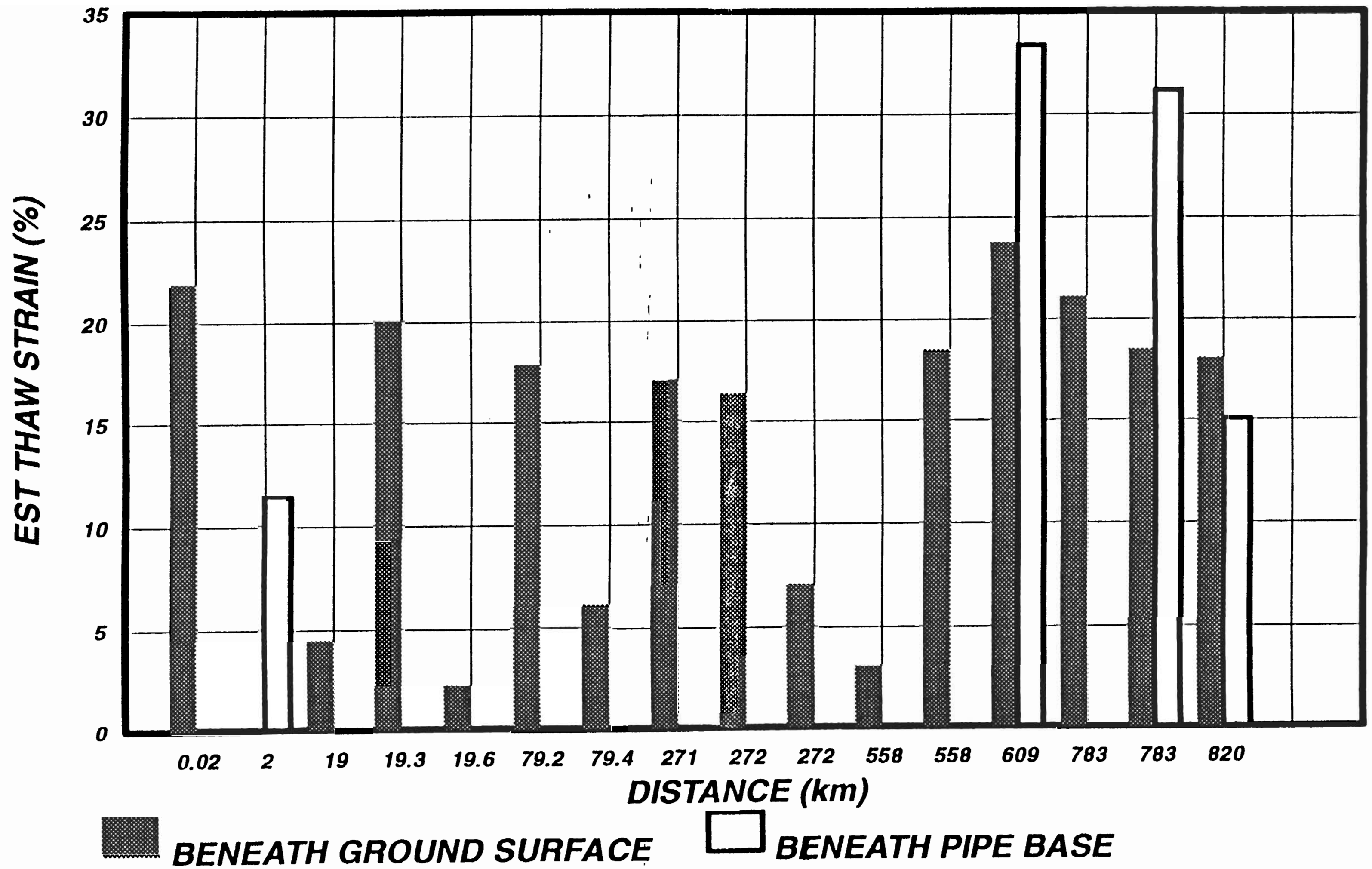


SOURCES: BURGESS/PTRM 1995 REPORT

FILE: ROW-SETT

FIGURE 5.14

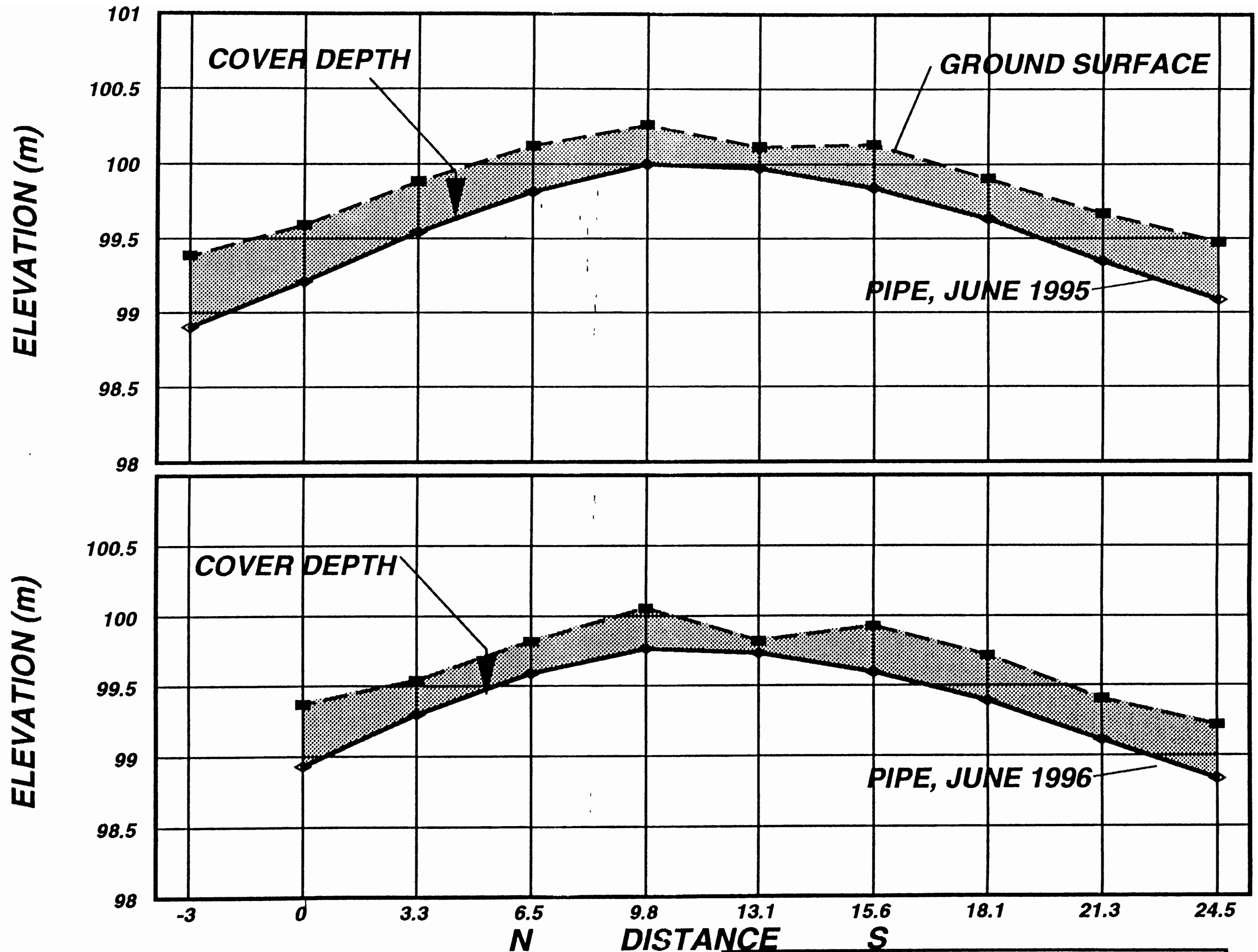
OBSERVED ROW AND PIPE SETTLEMENTS TO 1995



SOURCES: BURGESS/PTRM 1995 REPORT

FIGURE 5.15
OBSERVED ROW AND PIPE THAW STRAINS

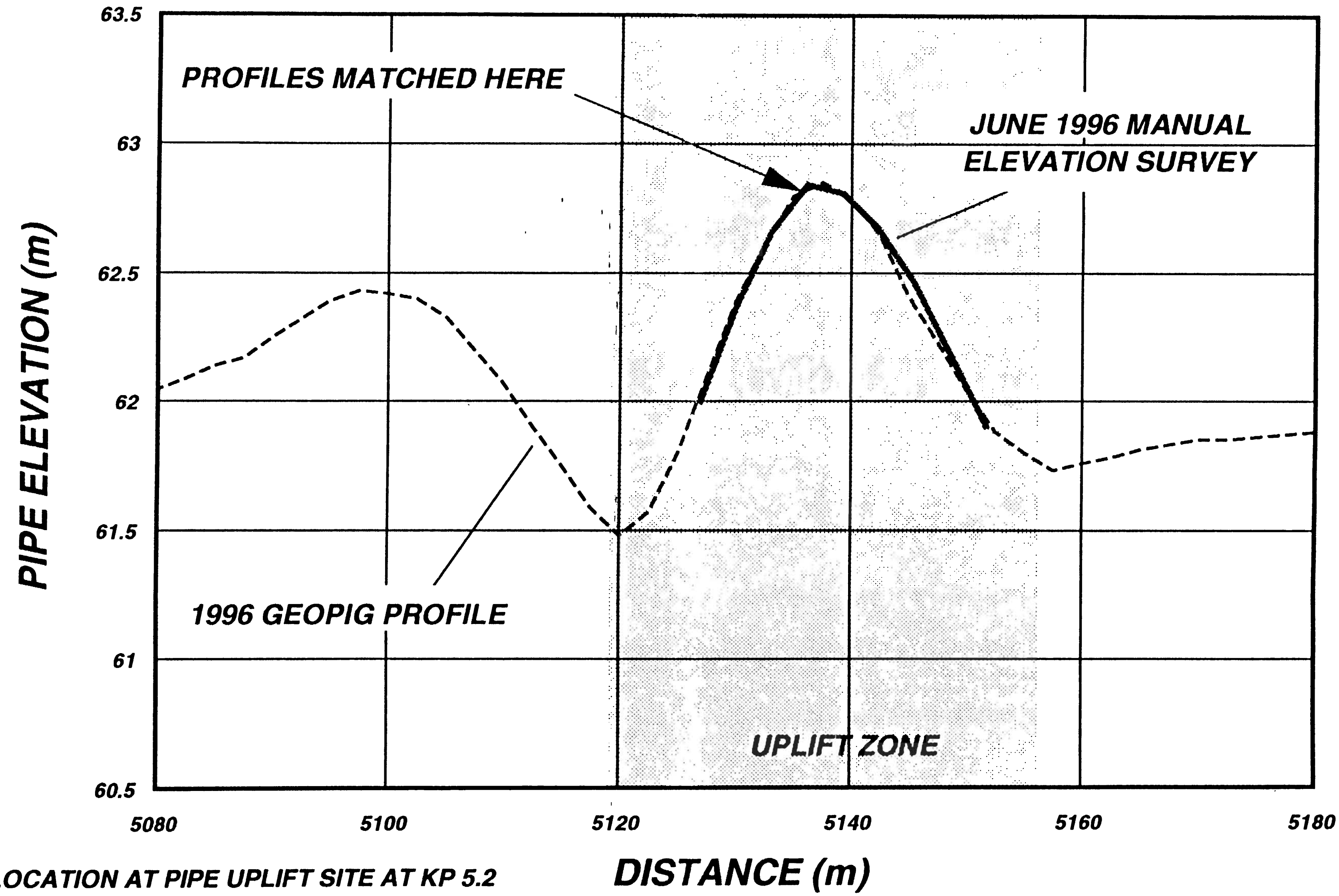
FILE: ROW-STR



ELEVATIONS RELATIVE TO BENCHMARK ON TREE

FILE: COV-9596

FIGURE 5.16
PIPE SURVEY ELEVATIONS AT KP 5.2



LOCATION AT PIPE UPLIFT SITE AT KP 5.2

FILE: ELCOMP96

FIGURE 5.17
SURVEYED PIPE ELEVATIONS AND 1996 GEOPIG PROFILE

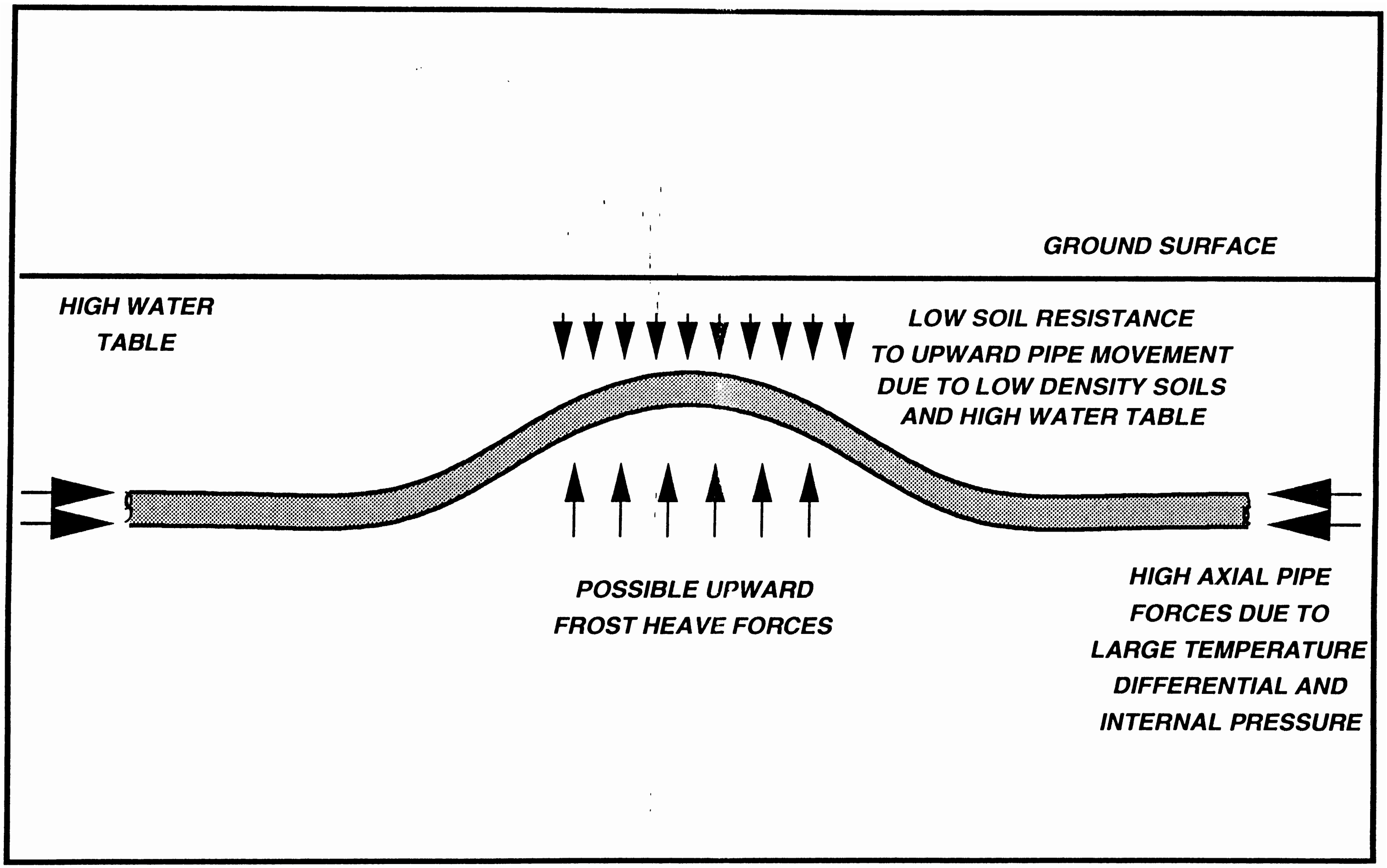
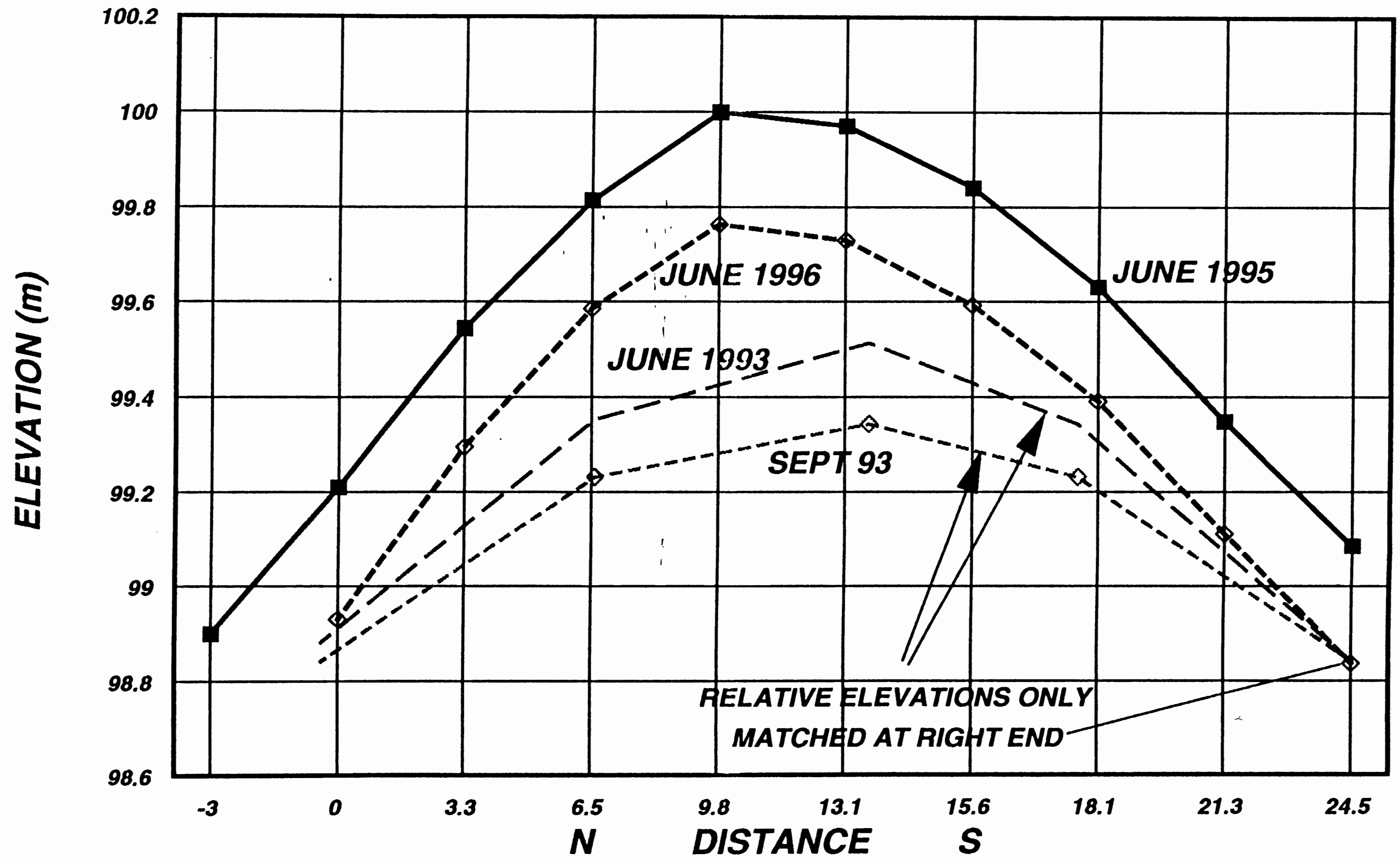


FIGURE 5.18
FORCES ACTING ON BURIED PIPE TO CAUSE UPWARD BUCKLING

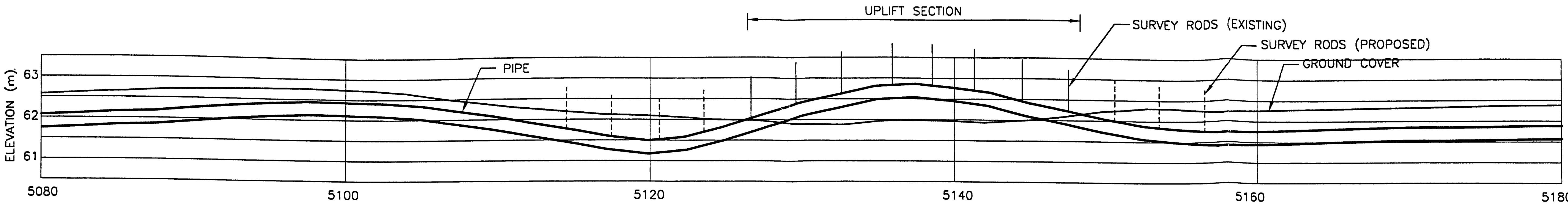
FILE: FDT-PIPE



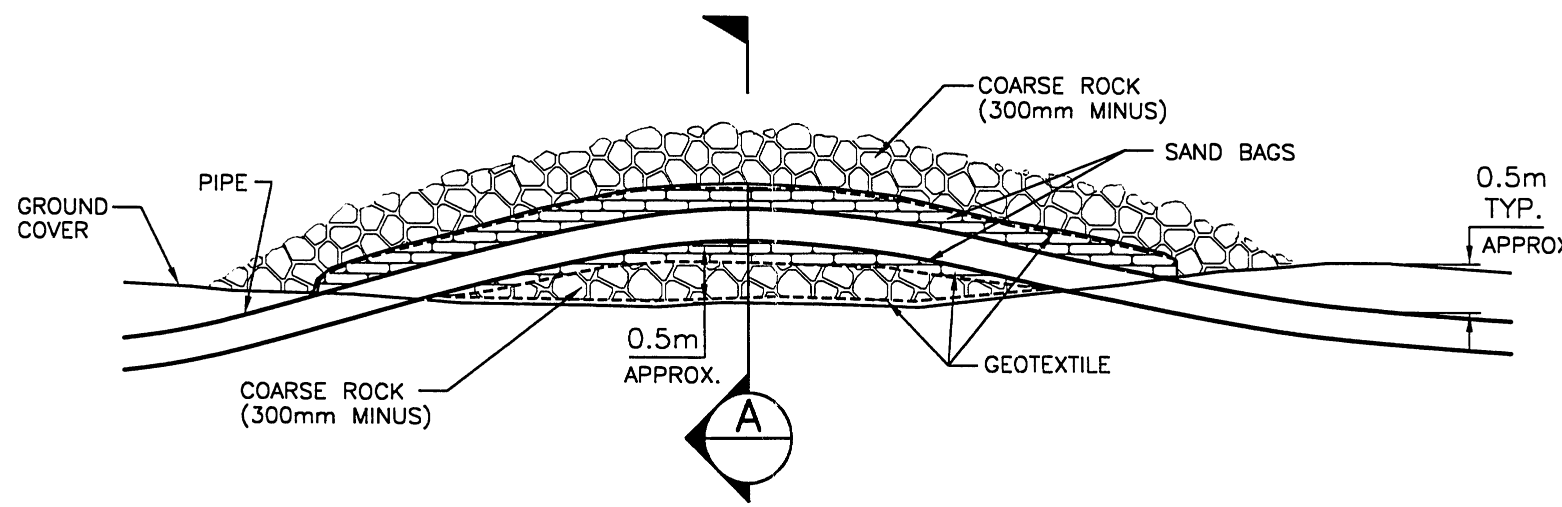
JUNE 1995 AND JUNE 1996
ELEVATIONS RELATIVE TO BENCHMARK ON TREE

FIGURE 5.19
SUCCESSIVE ANNUAL PIPE SURVEY ELEVATIONS AT KP 5.2

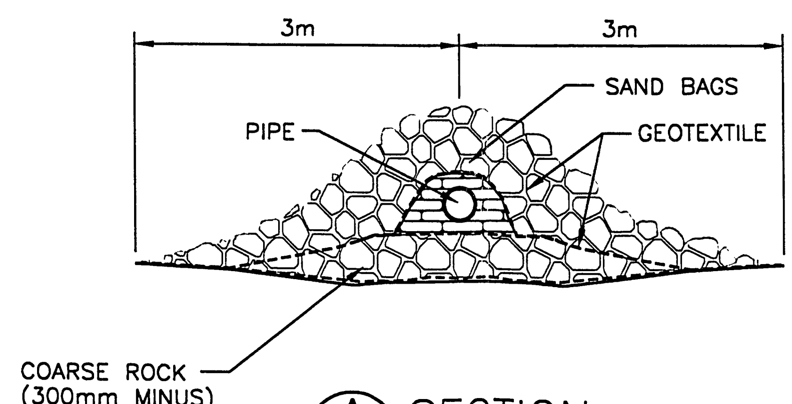
FILE: ELUPLIFT



PROFILE
N.T.S.



DETAIL OF UPLIFT SECTION
N.T.S.



A SECTION
N.T.S.

NOTE:
- SAND BAG TYPICALLY 175mm DIAMETER
- GEOTEXTILE MATERIAL AVAILABLE FROM IMPERIAL OIL RESOURCES LIMITED
- GEOTEXTILE MATERIAL TO BE REVIEWED PRIOR TO USE

FIGURE 5.20

REMEDIATION DETAILS: Kp 5.2

PLOT 1:1

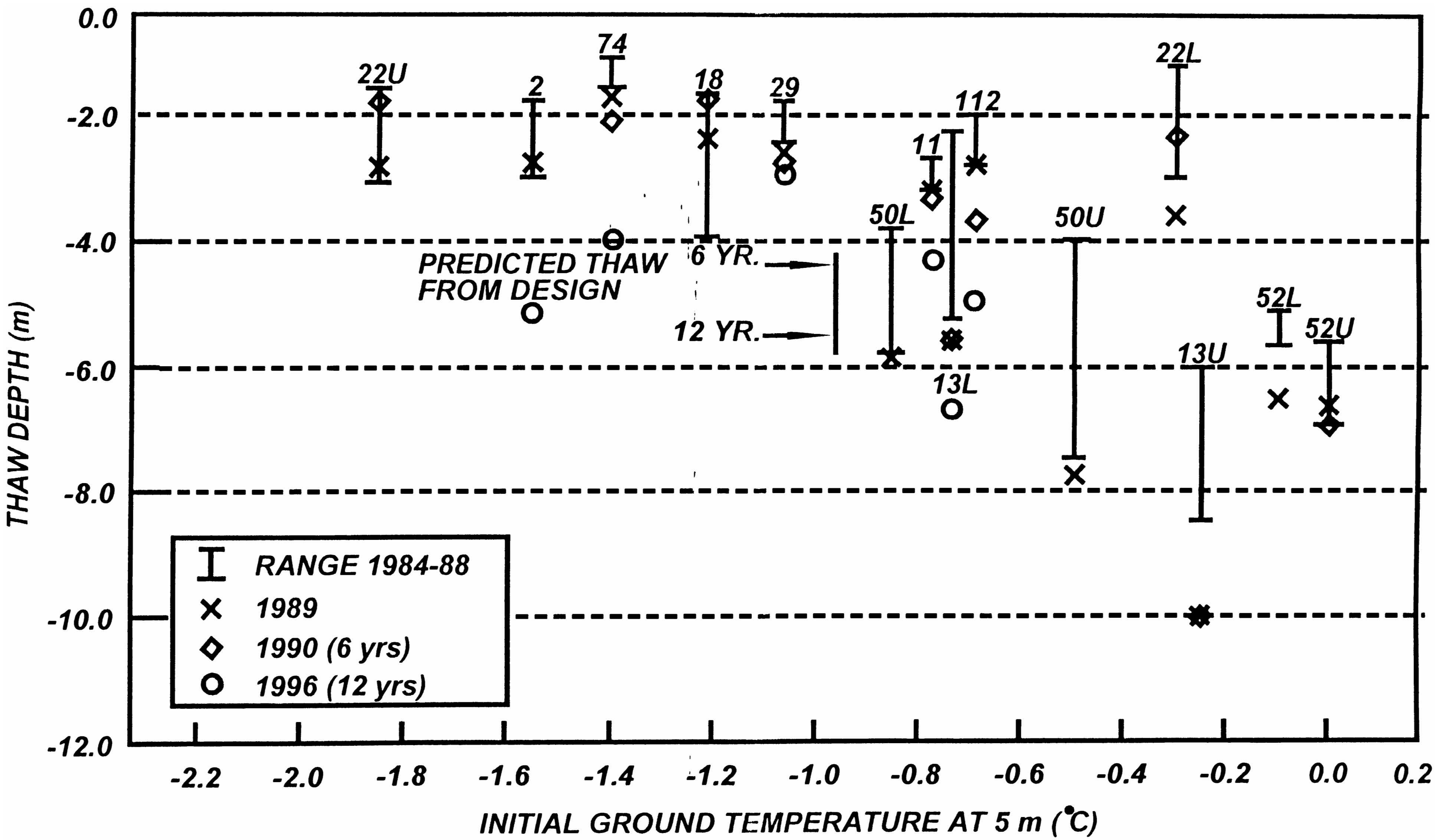


FIGURE 5.21
THAW DEPTHS FOR SELECTED
NON INSULATED SLOPES

FIGURE 5.22
OCHRE RIVER SOUTH VIEW OF VENTILATION PIPES

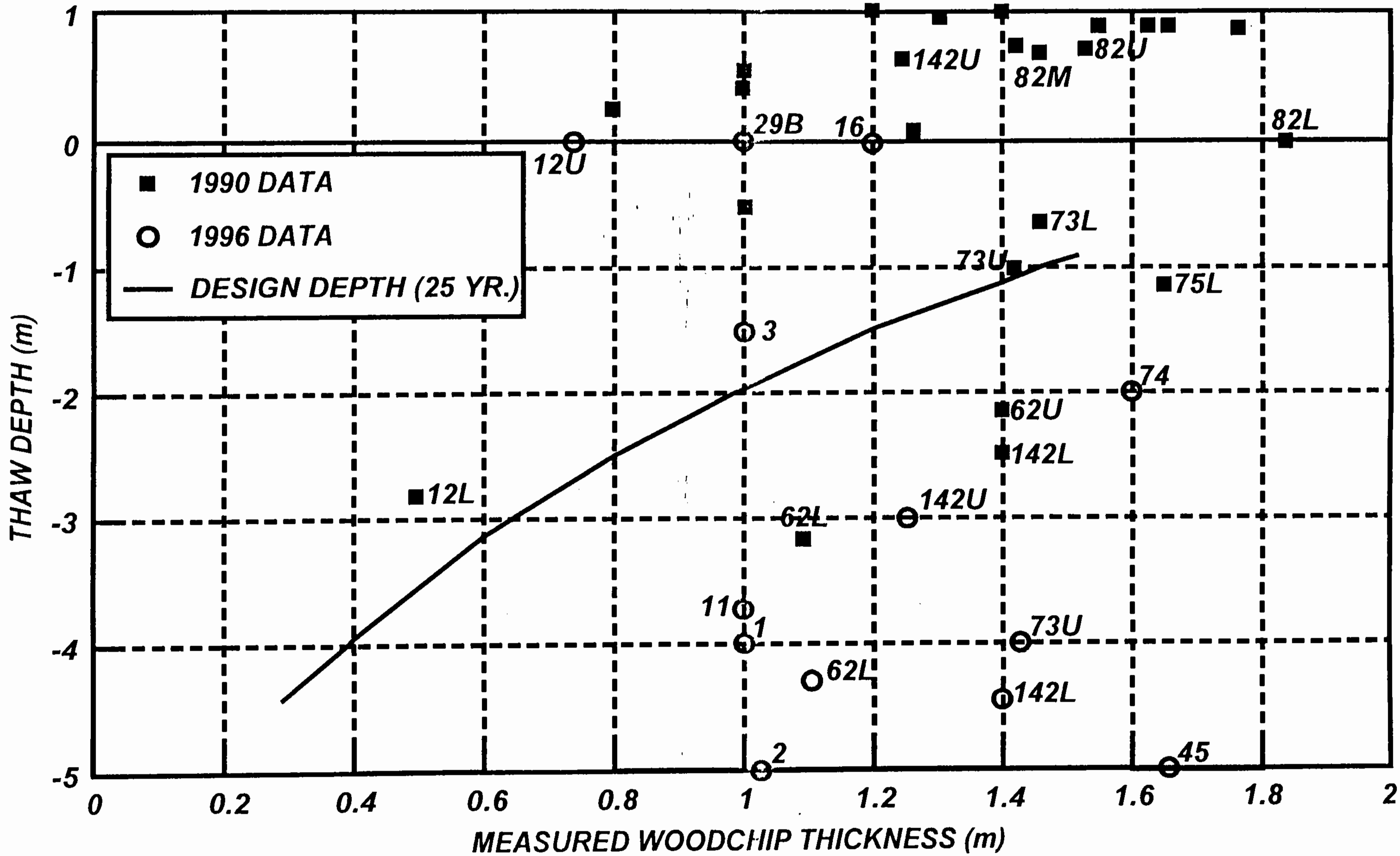


FIGURE 5.23
THAW DEPTH FOR SELECTED WOODCHIP INSULATED SLOPES

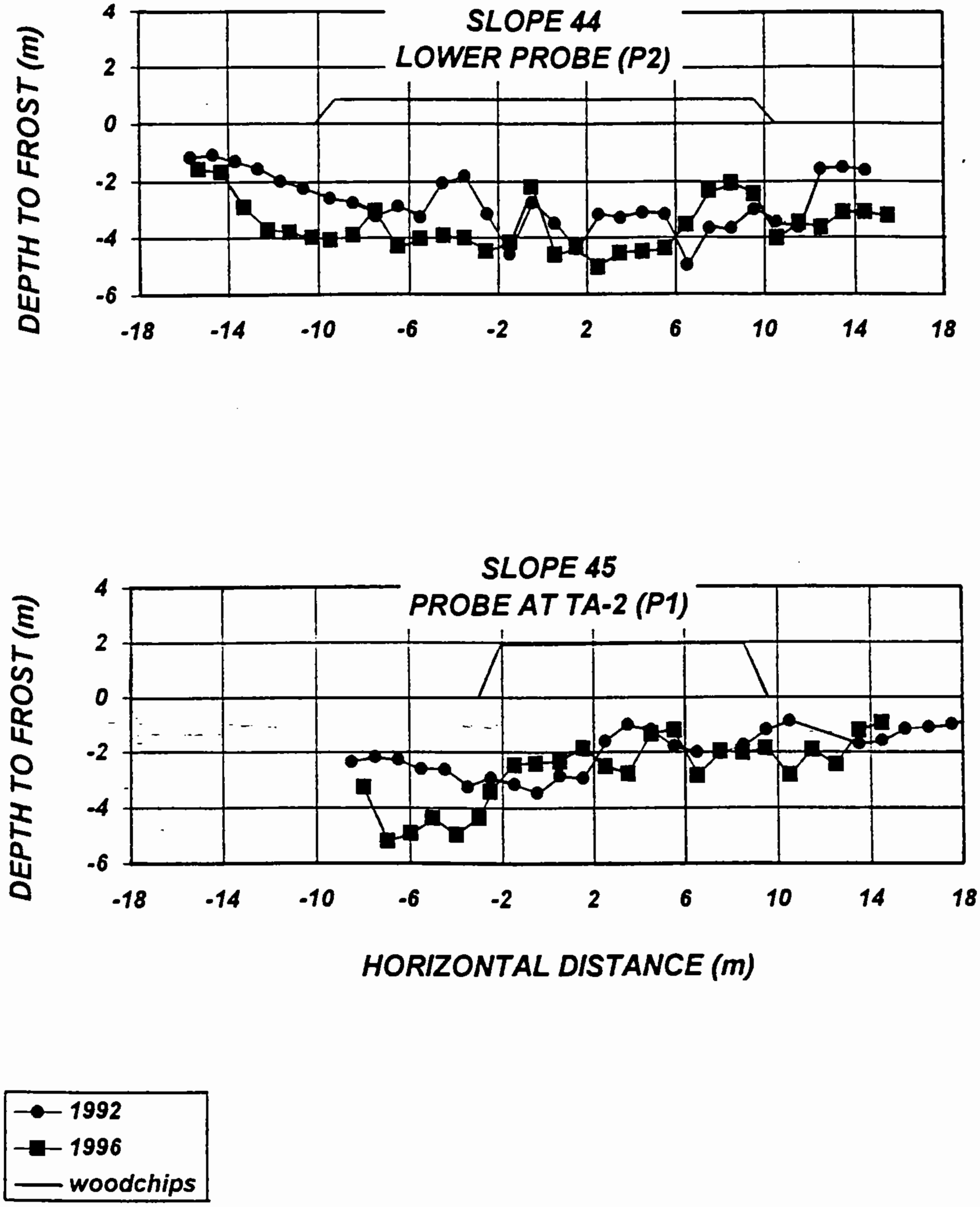


FIGURE 5.24
THAW PROBING DATA FOR 1992 AND 1996
SLOPES 44 AND 45

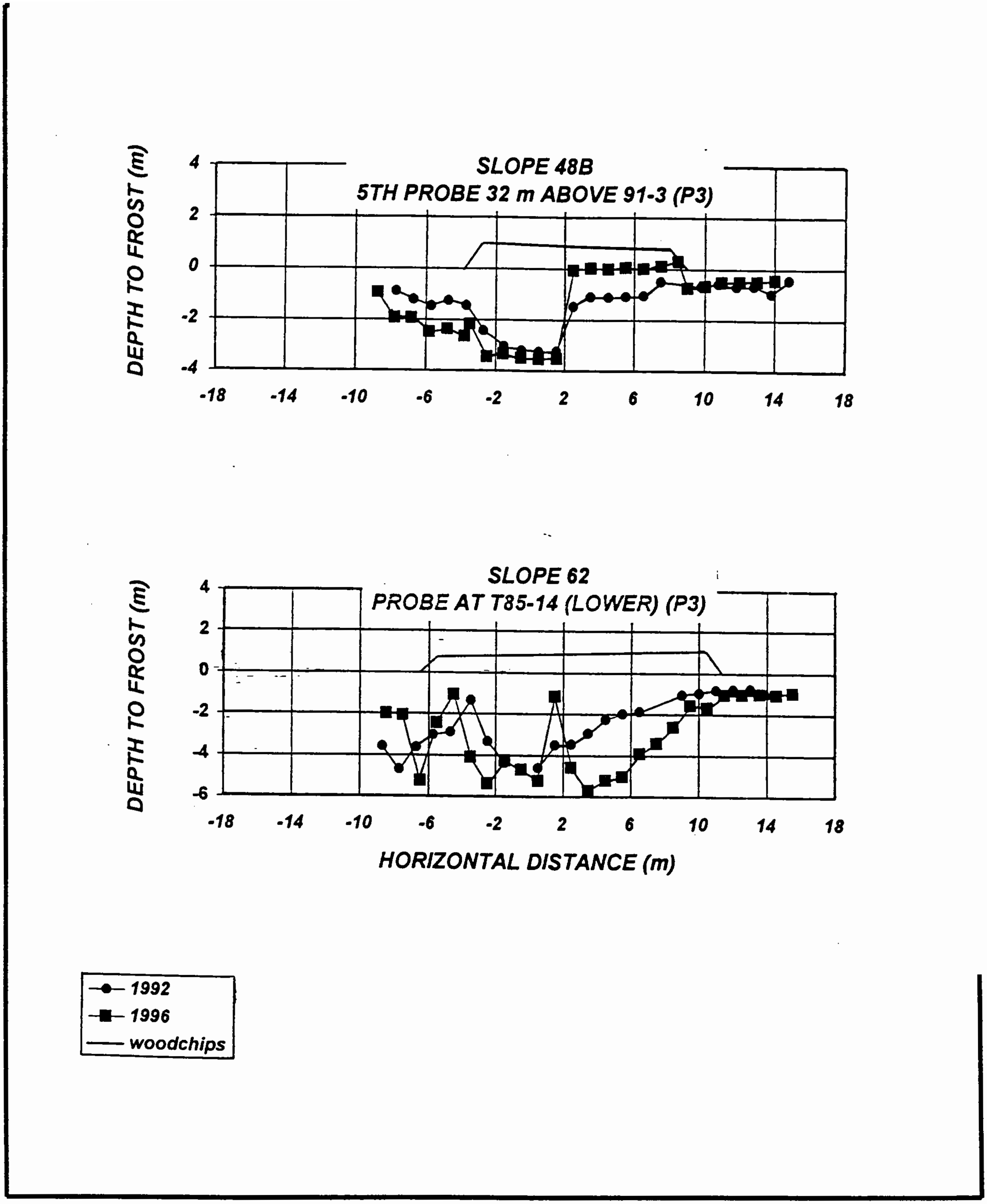


FIGURE 5.25
THAW PROBING DATA FOR 1992 AND 1996
SLOPES 48B AND 62

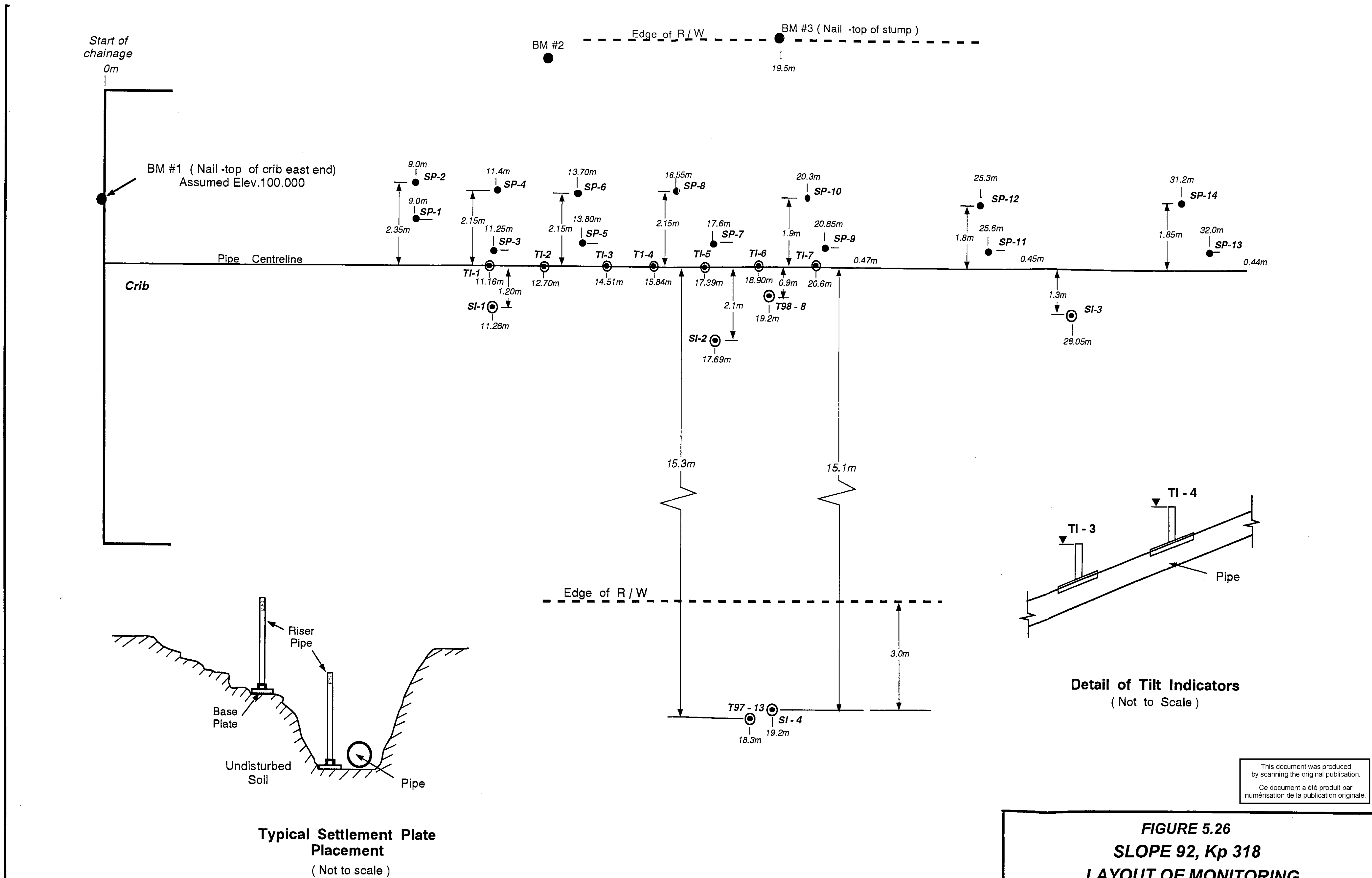


FIGURE 5.26
SLOPE 92, Kp 318
LAYOUT OF MONITORING INSTRUMENTATION

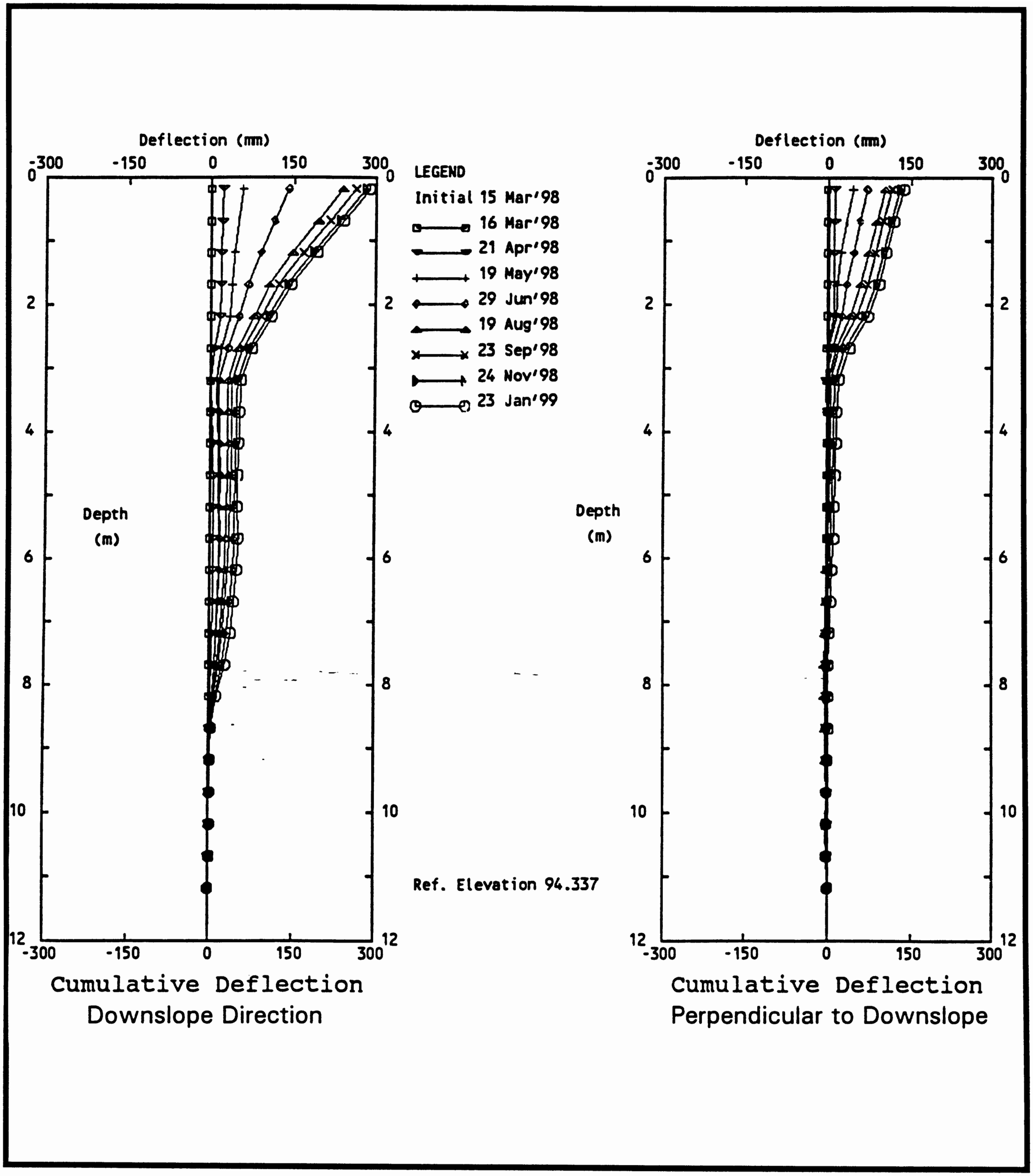


FIGURE 5.27

SLOPE INDICATOR SI - 2 AT Kp 318 (SLOPE 92)

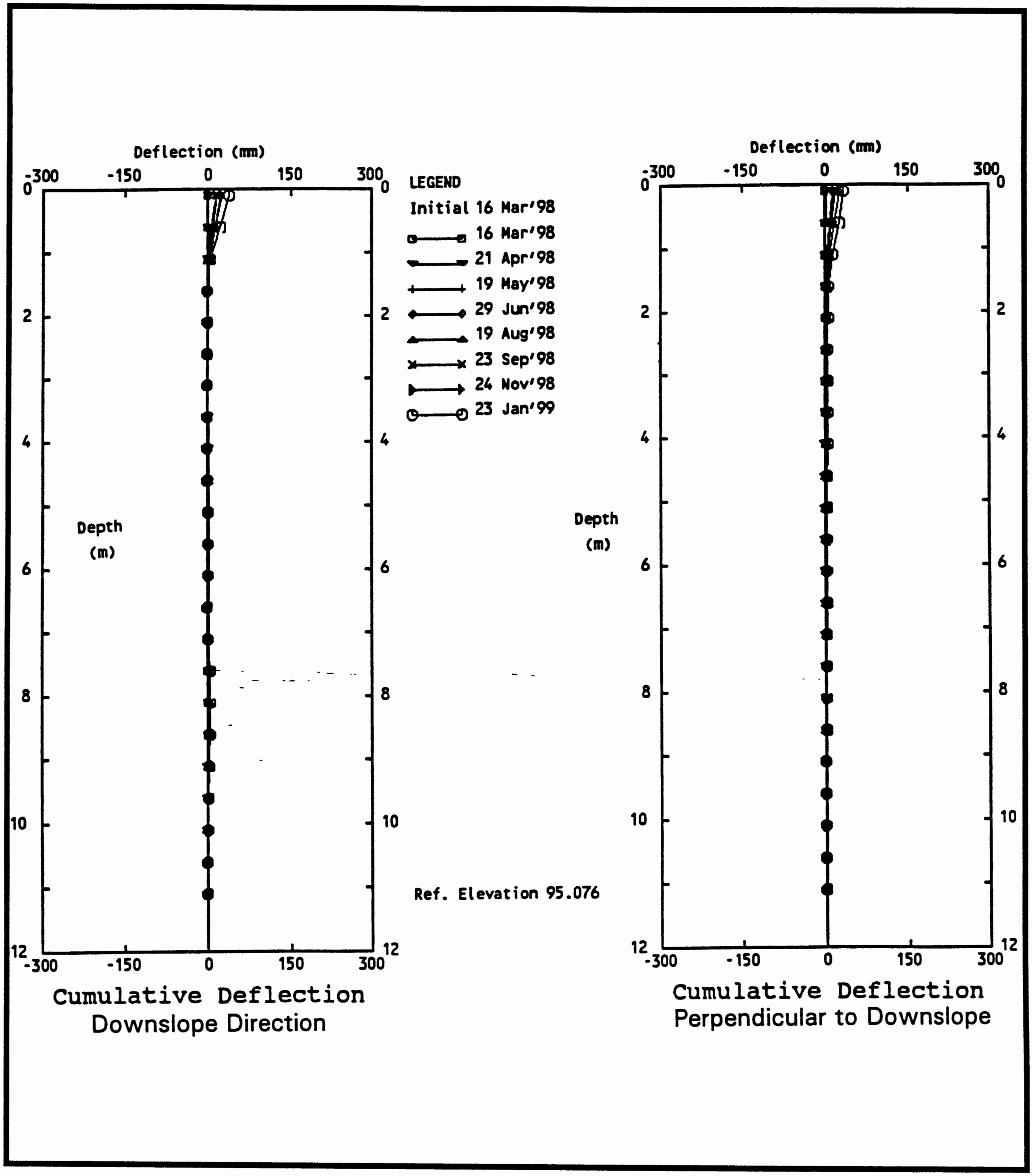


FIGURE 5.28

SLOPE INDICATOR SI - 4 AT Kp 318 (SLOPE 92)

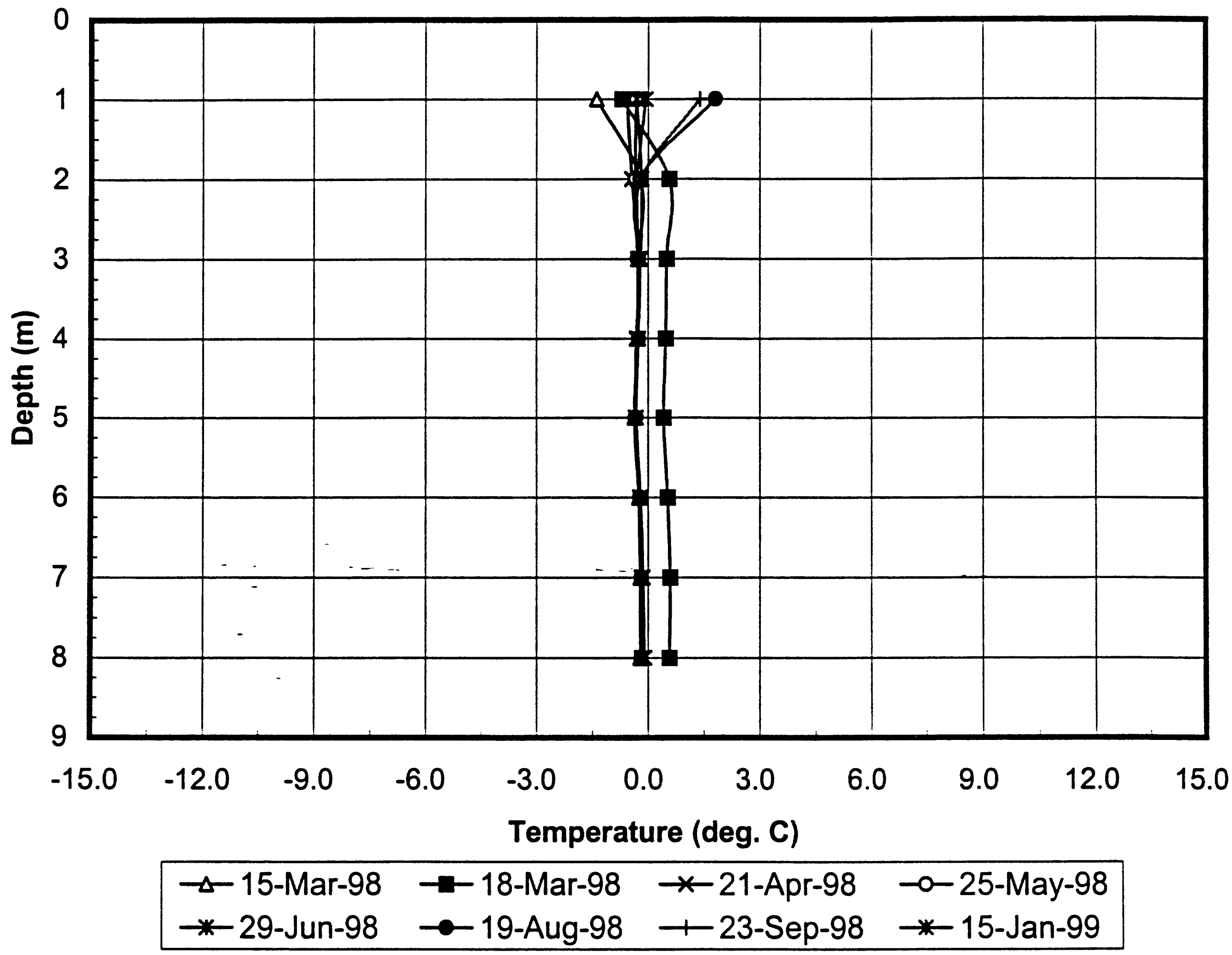


FIGURE 5.29

THERMISTOR 97-13 (OFF RIGHT-OF-WAY)

Kp 318 (SLOPE 92)

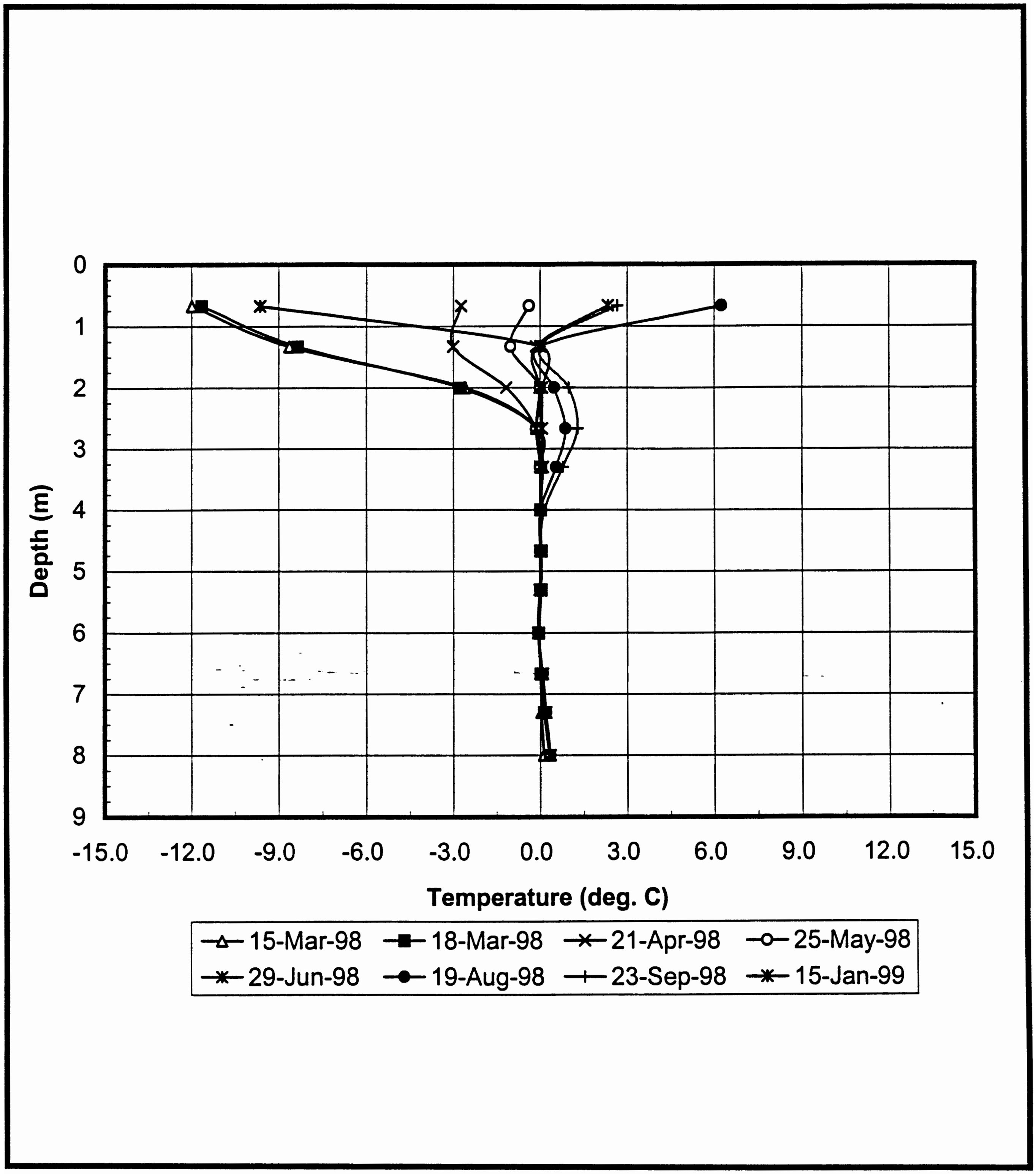
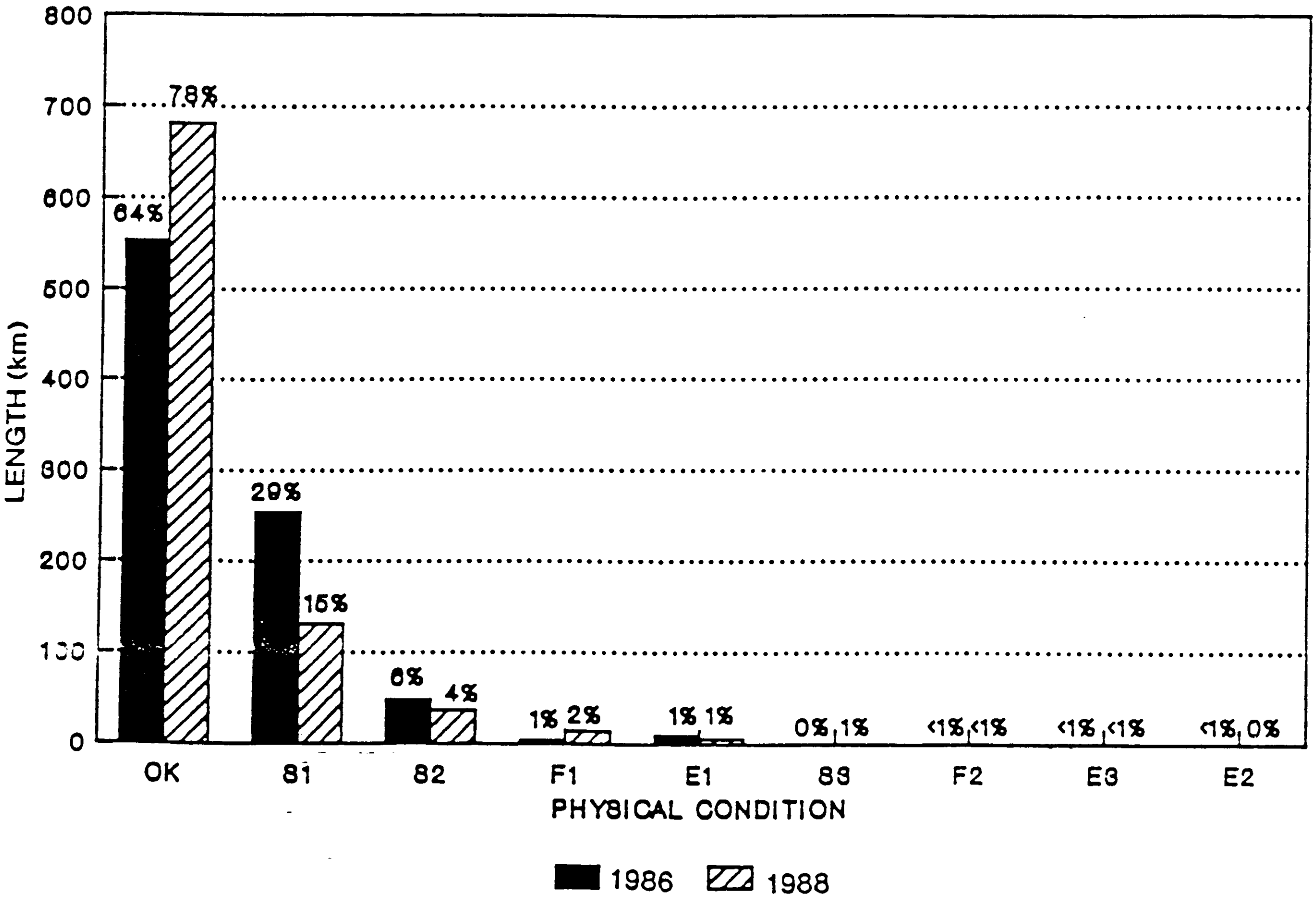


FIGURE 5.30

THERMISTOR 98-B (ADJACENT TO SI-2)

Kp 318 (SLOPE 92)

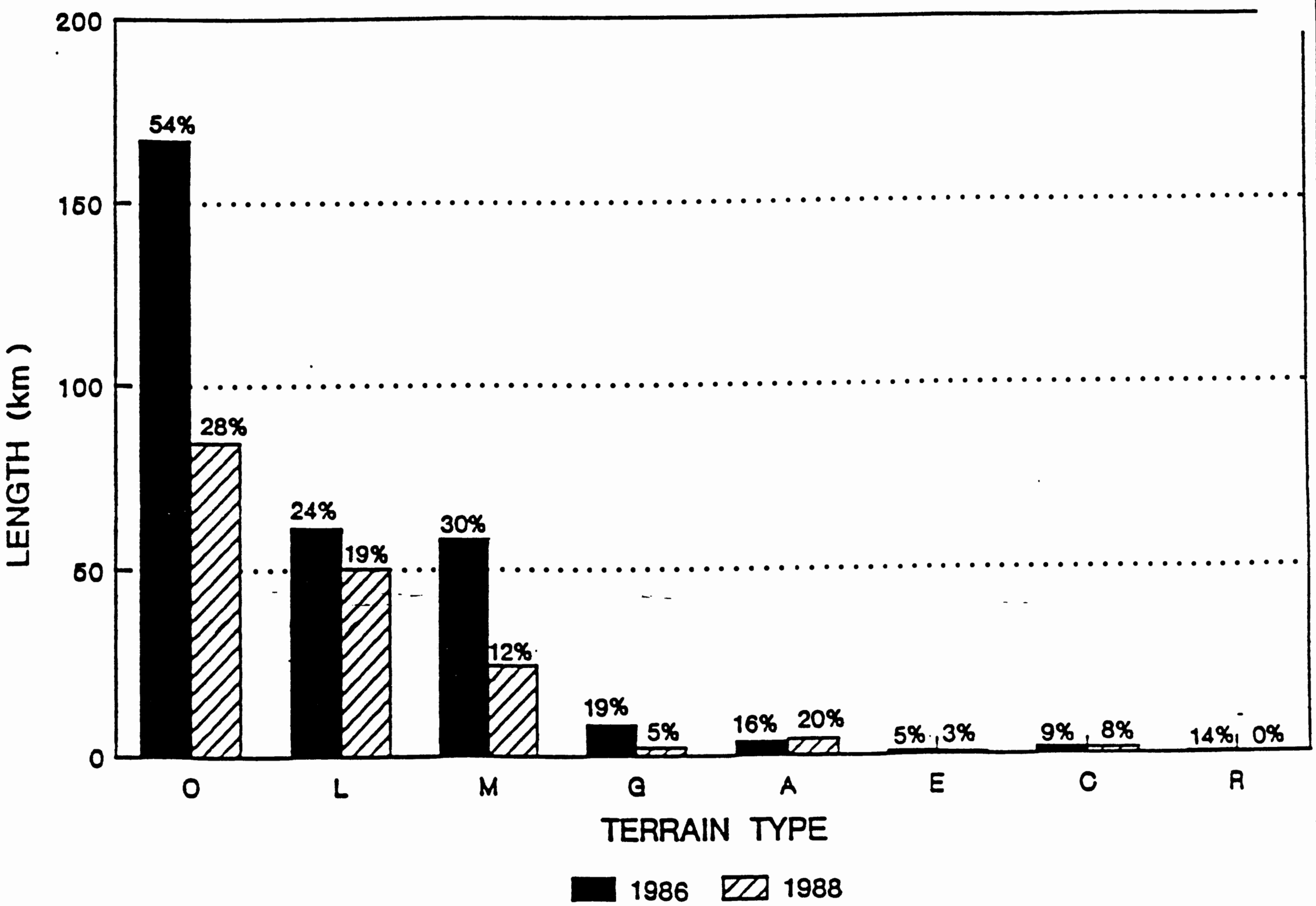


LEGEND

OK - No Significant Features
S1 - Ditchline Subsidence
S2 - Ditchline With New Backfill
S3 - Ditchline Subsidence Under Wood Chip Insulated Slope
E1 - Erosion of Right-of-Way
E2 - Slope Failure
E3 - Erosion or Failure of Wood Chip Insulated Slope
F1 - Standing Water
F2 - Beaver Pond

* Number of top of bar is percentage of the total route affected by each physical condition

FIGURE 5.31
CHANGE IN PHYSICAL CONDITIONS FROM 1986 TO 1988



LEGEND
O - Organic
L - Lacustrine
M - Moraine
G - Glaciofluvial
A - Alluvial
E - Eolian
C - Colluvial
R - Bedrock

* Number on top of bar is percentage of the total route occupied by the terrain type that is affected by ditchline subsidence

FIGURE 5.32

EXTENT OF DITCHLINE SUBSIDENCE ON EACH OF THE MAJOR TERRAIN TYPES

6.0 ECONOMIC ASPECTS

Construction of the 868 km pipeline and three associated pump stations was completed three months ahead of schedule, and started operation on April 3, 1985. The final cost reported by Pick and Smith (1985) was $360 million. This is in contrast to the original estimated project cost of $576 million. The lower than estimated costs were likely due to lower labor costs, and lack of other large projects at the time.

The above cost can be translated into a rough guide for pipeline cost estimators at about $55,000 "per diameter-inch-mile", including pump station costs.

Maintenance/monitoring costs for 1994 and 1995 were 2.92 and 0.32 million dollars respectively.

7.0 SUMMARY OF IMPORTANT LESSONS LEARNED

7.1 Construction Approach

There was less snow than anticipated during construction, and snow pad thicknesses were less than envisaged. However, the negative effects of this may not have been severe, and in fact more initially cooling of the right-of-way may have occurred.

Rates of ditching production through different terrain units was quite dependent on soil type. Glacial tills with cobbles slowed ditching down, whereas ditching was faster than anticipated in finer, lacustrine soils.

Costs were less than originally estimated, primarily due to lower labor costs, and an absence of other large construction projects at the time.

7.2 Pipe and Ground Thermal

A combination of warmer than average climatic conditions and greater thermal disturbance effects to the right-of-way surface, including the ditchline, resulted in greater warming to the right-of-way soils than anticipated. This in turn led to warmer pipe operating temperatures remote from Norman Wells. A study in 1986 addressed the issue of pipe temperatures (Hardy Associates (1978) Ltd., 1986).

Difficulties in chilling the oil at Norman Wells in the first few years required modifications and efficiencies to chilling equipment.

The requirement to continually chill oil to -1 or -2°C at Norman Wells is somewhat unrealistic, and likely unnecessary. The ambient or prevailing ground temperatures at pipe burial depth in this area warm to +6°C or so in summer, and cool below 0°C in winter. The imposition of negative temperatures at Norman Wells in summer resulted in a year-round local frost bulb for the first few kilometers, and the pipe eventually warmed up to the ambient conditions as dictated by the disturbed ground in any case. The chilling required a large and unnecessary expenditure of energy in summer. This requirement has now been replaced with a permissible warmer summer temperature excursion since 1993, provided the average year-round temperature is maintained at or below 0°C.

Pipe temperatures beyond 50 km or so from Norman Wells (or other pump stations) have no memory of the conditions on exiting the pumps, and adapt completely to the surrounding soils and environment. Warmer than average years result in warmer pipe and oil temperatures in summer that are not related to temperature excursions at Norman Wells.

7.3 Pipeline Design

There is a need to distinguish between design and operational limits for pipe strain. Even though the pipeline was designed to a compressive strain limit of 0.5%, this should not necessarily imply or require that mitigation or repair be carried out when the strain reaches this limit. The danger of rupture or loss of service may occur at a different strain than the strain limit set during the design.

There is a common misconception that bending strain, as evidenced by pipe curvature, is the same as compressive (or tensile) strain. Because there may be large axial stresses initially present in the pipe prior to bending (due to temperature differential and internal pressure effects), the compressive strain limit may be reached before the bending strain, as evidenced by pipe curvature measurements. Further, testing by the University of Alberta in Edmonton (Souza and Murray, 1994) and elsewhere has established that pipes such as the Norman Wells pipe can be strained in compression to levels much higher than 0.5%. This has been cited as a reason for higher bending strain limits. However, this argument should be followed with caution, as the tensile strain limits are governed by defect size in the welds, and may not permit significantly higher bending strains in an existing pipeline.

Where a thermal interface, low density soils and a high water table combine, uplift buckling of the pipeline can occur. This may require remediation depending on the strains and displacements interpreted from ongoing monitoring.

The lack of stable survey benchmarks has been a recurring problem with determining absolute pipe movements, strains and soil-pipe loading mechanisms. Pipe sections showing signs of ongoing thaw settlements should have a deep benchmark installed to make pipe monitoring more meaningful.

Several pipe thaw settlement test sites should have been installed at the outset of construction or operation, or soon after start-up, when sites with thaw settlement could be delineated. This has been a major impediment to understanding the processes of thaw, settlement and interaction between the pipe and surrounding soils. The recent NRCan/GSC thaw settlement test site installation at Kp 2.0 will assist in overcoming some of these deficiencies.

7.4 Thaw Settlement and Frost Heave

Thaw settlements after 12 years have generally been less than those calculated for the 25 to 30 year design life. There is only limited evidence for the sudden, step differential ground settlement profiles assumed in design. Consequently, pipe bending strains as evidenced by the GEOPIG or level surveys have generally not approached the design limits.

Frost heave has generally not been a significant issue for pipeline design, as pipe temperatures have been warmer than anticipated. Exceptions to this may exist in the first few kilometers from Norman Wells, where colder pipe temperatures due to pipe chilling may have resulted in some frost heave, and may have initiated the uplift buckling currently observed at Kp 5.2.

7.5 Seismic Effects

Seismic effects from at least three significant earthquakes in the area have apparently not caused any distress to the pipe, or surrounding soils on sloping terrain.

7.6 Slope Stability

The original slope monitoring instrumentation was installed after the pipeline was constructed, with one purpose of assessing right-of-way conditions. The instruments were installed at some distance (up to 4 m) from the pipe. This has lead to some problems interpreting the conditions close to the pipe. In the future, it would be recommended that the pipeline be staked at several points on the slope, so that the precise location can be determined after backfilling and application of slope mitigation. In this way it should be possible to install the instrumentation closer to the pipe.

Some of the instruments installed shortly after construction are now too shallow to provide data at the thaw front. In future projects, some instrumentations should be installed deeper than initially required, on the assumption that circumstances may change and that the deeper installed instruments may be needed.

7.7 Drainage and Erosion

Significant right-of-way erosion occurred at some locations in the first several years following construction. However, over time the problems associated with erosion have reduced. Some localized problems have continued to develop as the pipeline matures, due to meteorological or hydrological events.

For future projects, stockpiles of sandbags and other diversion berm construction materials should be placed at selected locations, to be used as needed. For the present project, this has already been addressed by the pipeline maintenance group.

8.0 UNRESOLVED ISSUES

By and large there are no significant unresolved issues relative to the geotechnical design and operation of the pipeline. The recent formation of wrinkles at a few locations is worth investigation to determine if there is a geotechnical influence.

Some temperature monitoring cables have not been maintained, and are not currently being read. These need maintenance, and a resumption of regular readings, based on recommendations from the consultants.

One small leak was discovered and repaired with only minor fluid loss from the pipeline. The potential risk of this type leak re-occurring should be considered. It should be noted that this leak was not due to any geotechnical factor.

The effectiveness of the insulated pipe sections at sag bends is not known. It would be of interest to (a) determine the quality and integrity of the thermal insulation after 14 years of service, and (b) examine GEOPIG records to determine if any movements have occurred due to seasonal frost heave at these or other similar locations.

The current method of calculating the gravity loading on pipes in thaw settling terrain may be too conservative. It would be of great interest to re-examine the values used in design, in the light of the observed pipeline performance in the field.

There is a need to determine the optimum seasonal temperature operating cycle to minimize both waxing and the geothermal effects on surrounding terrain.

Thaw and settlement are still developing, and will require monitoring with GEOPIG and level surveys over time. Closer examination of sequential GEOPIG profiles at selected sites should be carried out to monitor development of pipe settlement. The mechanism of the development of some small wrinkles also requires some additional investigation.

Certain slopes will require ongoing monitoring and stability assessment as thaw continues. The same is true for pore pressures in deeper piezometers.

Some slopes near fire-affected areas require further inspection, e.g. the insulated slope at Kp 182.

9.0 RELATED LITERATURE

The listing at the end of this document provides a good starting point for references to this project. In particular, the 2-volume set by MacInnes et al, 1990 provides an excellent background to the terrain monitoring that took place in the first 5 years of the project. A large number of excellent photographs are included, and links to other reference material are provided.

In addition NRCan/GSC are currently preparing a full bibliography of documents related to the project. The bibliography reference is:

> Burgess, M.M., and Naufal, J. (in prep.). Bibliography of Norman Wells pipeline permafrost, terrain and geotechnical reports. Geological Survey of Canada. Open File.

The Annual Geotechnical Reports for each year of operation are a good source of reference for specific details of studies, mitigation and monitoring that took place. These reports include contributions, by EMR, IPL, DIAND and their consultants.

The NEB "Reasons for Decision" (National Energy Board, 1981) relating to the project, documents the review process behind the granting of the permit to construct the pipeline.

The report of the Environmental Assessment panel on the project in January 1981 (Duffy et al.) provides an environmental perspective of the project prior to final design and construction. Some of the research deficiencies perceived at that time are reviewed.

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APPENDIX A

LISTING OF INSTRUMENTATION ON OR NEAR THE PIPELINE RIGHT-OF-WAY

NORMAN WELLS PIPELINE PROJECT
Enbridge-EMR INSTRUMENTATION PROGRAM
(UPDATED December 1998)

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
EMR-84-1	0+020	T-1	84-1-T1(5.1)Y1[3]	IRT/BR	
		T-2	84-1-T2(5.0)Y1		
		T-3	84-1-T3(10.4)Y1		
		T-4	84-1-T4(13.6)Y1		
		T-5	84-1-T5(19.6)Y1		
		G-1	76 mm PVC (12.9 m)		
		G-2	76 mm PVC (19.7 m)		
		PT	[PT1-1A] may be broken(?)		0.90 m depth of cover
Slope 1, Bosworth N	0+361		T91-5(4.19)	IRT	Below top of wood chips
			97-10(8.0)		1.0 m wood chips
			P21327 (3.8)		
	0+364	T-1	T3(6.0)A, [TA11(1.0)A], SP(1.0)		
	0+364	P-1	6125(0.75)		
Slope 2, Bosworth S	0+423	T-2	T1(6.1)A,[TA7 (1.1)A], SP(1.1)		
	0+423	P-1	6106(1.0)	IRT	1.1 m wood chips
	0+465	T-1	T2(5.0)A		
			P21337(2.8)		
FH 1	16+420	PT	[PT1-2A]	-	2.6 m depth of cover
	16+426	PT	PT2-1A		2.6 m depth of cover
EMR-84-2A	18+972	T-1	84-2A-T1(5.3)Y1	IPT/BR	
		T-2	84-2A-T2(5.1)Y1		
		T-3	84-2A-T3(19.6)Y1		
		T-4	84-2A-T4(13.0)Y1		
		G-1	76 mm PVC(19.3)		
		PT	PT1-3A		0.95 m depth of cover
EMR-84-2B	19+266	T-1	84-2B-T1(5.8)Y1	IPT/BR	1.0 m wood chips
		T-2	84-2B-T2(5.9)Y1		1.0 m wood chips
		T-3	84-2B-T3(20.5)Y1		-
		T-4	84-2B-T4(20.6)Y1		1.0 m wood chips
		G-1	76 mm PVC(20.5)		-
		PT	PT1-4A		1.0 m depth of cover
Canyon Creek North	19+700		T97-8(8)		
			P21336(3.0)		

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Canyon Creek South	20+		T97-9(8.0) P21330(3.0)		
EMR-84-2C	19+551	T-1 T-2 T-3 T-4 G-1 PT DT PT	84-2C-T1(5.5)Y1[3] 84-2C-T2(5.5)Y1 84-2C-T3(19.4)Y1 84-2C-T4(20.0)Y1 76 mm PVC PT1-5A [113A] PT-2B	 - 	 0.95 m depth of cover Ditch thermistor possibly malfunctioning
Slope 8, Francis S	23+230 23+235	P-2 P-1	6119(6.8) 6133(16.0)	 UFT/UFC	
IPL-PT 1	25+612	PT	PT1-6A	-	0.9 m depth of cover
Slope 11 Helava N	25+670 25+705 25+713 25+724 25+728 25+730	T-1 T-2 T-3 P-1	T8(5.0)A T19(6.0)A, TA9(1.0)A T97-5(2.8) P2133(3.5) T5(6.0)A, [TA19(1.0)A], SP(1.0) 6101(2.0)	IRT	 1.0 m wood chips 1.0 m wood chips 1.0 m wood chips
Slope 12 Helava S	25+778 25+778 25+780 25+780 25+794 25+796 25+827	T-1 P-1 T-2	T17(5.5)A, TA10(0.5)A T97-6(8.0) P21334(4.0) 6123(2.0) P97-7(8.0) P21333(4.0) T14(5.8)A, [TA6(0.75)A], SP(0.75)	IRT	0.5 m wood chips 0.5 m wood chips 0.75 m wood chips
Slope 13 Christina N	26+600 26+625 26+625 26+625 26+625 26+625 26+648 26+648	T-1 T-2 P-1 P-2 P-3 SI-1 SI-2 T-3	[DT5(10.0)A] DT2(10.0)A 6118(6.1), S(3.0) 6129(3.6) [6132(16.5)] SI(20.7) SI(12.2) HT212(3.0)Y1	IRT	 1.5 m fill (berm) 1.2 m fill + 0.1 m snow 1.2 m fill 1.5 m fill 1.5 m fill

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 16	32+450	T-1	T4(6.4)A, TA4(1.2)A, SP(1.4)	IRT/IRC	1.4 m wood chips
Prohibition S	32+468		P21329(5.0)		1.2 m wood chips
	32+471		T97-4(8.0)		Side cut string
	32+500	T-2	T23(6.2)A, [TA1(1.2)A]		
	32+500	T-2	CT-3A		
IPL-PT 2	32+609	PT	PT1-7A[3]	-	0.8 m of cover
Slope 18	43+698	T-1	DT4(10.0)A, S(4.5)	UFT/IRT	
Vermilion S		81-S19A	2325(4.3)		Preconstruction installation
Slope 22	66+050	T-1	T7(5.0)A	IRT	
Norman Range	66+080	P-1	6126(3.9)		
	66+122	T-2	T21(4.0)A, S(4.0)		
IPL-PT 3	76+000	PT	EMR-8A	-	0.8 m depth of cover. Possibly inoperative.
EMR-84-3A	79+155	PT	EMR-11A	-	0.9 m depth of cover
	79+180	T-1	84-3A-T1(4.7)Y1		
		T-2	84-3A-T2(4.7)Y1		
		T-3	84-3A-T3(22.1)Y1		
		T-4	84-3A-T4(8.0)Y1		
		G-1	76 mm PVC(21.2)		
Slope 29B	79+310	T-1	[T16(6.0)A,][TA15(1.0)A], SP(1.0)	IRC	1.0 m wood chips
Gt. Bear S	79+312	P-1	[6124(2.0)]		1.0 m wood chips
	79+316		T97-3(8.0)		1.0 m wood chips
	79+319		P21335(4.0)		1.0 m wood chips
	79+355	T-2	T12(6.0)A, TA17(1.0)A		
	79+357	P-2	[6100(2.0)]		
	79+405	T-3	T20(5.0)A		
EMR-84-3B	79+395	T-1	84-3B-T1(6.3)Y1	IRS/IPC	
		T-2	84-3B-T2(6.3)Y1		
		T-3	84-3B-T2(6.3)Y1		
		T-4	84-3B-T4(20.9)Y1		
		G-1	76 mm PVC(20.8)		
		PT	PT1-10A		1.15 m depth of cover
					0.3 m wood chips
		DT	[117A]		Ditch thermistor
IPL-PSS	95+150	1	84-4B-T2(5.7)Y2	IRC	F hole at pipe settlement site (PSS)

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 44	133+600	P	17614		No confirmed location or depth of tip
	(approx)				
	133+604		P21332(4.0)		
	133+605	P	17613 (4.57)		No confirmed location
	(approx)				
	133+607		T97-1(8.0)		
Slope 45	133+747	-	17611	IRC	Depth not available
Unnamed S	133+744	-	17612		Depth of tip not available, Enbridge reports instrument to be out of order
	133+758	T-1	P21328(5.0)		1.65 m wood chips
	133+760	P-1	T97-2(7.0)		1.65 m wood chips
	133+760	-	T13(6.3)A[TA2(1.2)A][3]		
	133+762	-	6107(2.0) S(2.2)		Site cut strong
		-	CT-2A		
IPL-PT4	133+900	PT	PT1-11A	-	1.15 m depth of cover
IPL-PSS	135+125	2	HA128(10.0)Y1	UFS/IPC	UF hole at interface
	135+160	3	HA127(10.0)Y1	IPC	F hole at interface
Slope 48B	160+174.5	P	17610(3.55)	IRC/T	
	160+181.5	P	17609(3.66)		
	160+206	T	T91-3(6.0)Y1		Below top of wood chips
	160+212	P	14008(4.5)		Below top of wood chips
	160+215	P	[16116(4.6)]		
	160+221	P	6281(4.7), 16115(4.7)		
	160+223	T	T92-5(7.2)		
	160+254	T	T92-3(7.2)		
	160+253	P	16113(2.4), 61176(2.4)		
Slope 50,	168+230	T-1	DT6(10.0)A	IPT	
Seagram S	168+232	P-1	6111(6.0),S(3.0)		
	168+233	P-2	6108(3.0), SP(1.0)		
	168+270	T-2	DT3(10.0)A		
IPL-PT 5	179+775	PT	PT1-12		0.9 m depth of cover
Slope 52,	179+790	T-1	DT7(10.0)A	IPT/UFT	
Saline N	179+870	T-2	DT1(10.0)A, SP(1.0)		
	179+870	P-1	6116(6.0), 6131(3.0), S(2.35)		

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information	
Fire Burn Area	182+	T	182-T1(6.0)	IPT/UFT?		
		T	182-T2(3.7)			
IPL-PT 6	194+351	PT	85PT1-1A	-	0.90 m depth of cover	
Slope 62,						Previous
Steep N	194+600	P	P14009(4.47)	IPT	Below top of wood chips	
	194+601	T-2	[85T15(4.6)A, TA12(1.2)A]		1.05 m wood chips	1.40 m wood chips
	194+626	T	T91-4(6.0)Y1		Below top of wood chips	
	194+631	P	P14013(3.0)			
	194+649	T	T97-11(8.0)			
	194+650	P	P22850(3.7)		Below top of wood chips	
	194+655	P-1	6128(2.6), S(2.3), SP(1.65)		1.10 m wood chips	1.65 m wood chips
	194+656	T-2	85T14(6.15)A, [TA14(1.2)], 4T146(1.15)A		1.10 m wood chips	1.65 m wood chips
Slope 63, Steep S	195+010 (approx)	P	P21367 (6)			
	195+011 (pprox)	T	T98-63 (11)			
Slope 64,	197+022	P-2A	6122(5.80)S(4.0)	UFT		
Unnamed N	197+023	P-2	6134(16.15), C(16.15)			
	197+049	P-1	6143(14.02), [6109(6.86)], S(5.0)			
Slope 65,	197+132	P-1	[6112(8.08)], S(5.5)	UFC/UFT		
Unnamed S	197+159	P-2	6138(16.00), S(8.3)			
	197+161	P-2A	6117(10.67), S(5.2)			
Slope 68,	230+949	-	S(1.2)	IPT/UFT	2 standpipes in ditch	
Slope S	231+019	-	S(1.2)		2 standpipes in ditch	
	231+089	-	S(1.2)		2 standpipes in ditch	
EMR-85-7A	271+231	T-1	85-7A-T1(5.0)Y1[3]	IRC/IRT		
		T-2	85-7A-T2(5.0)Y1			
		T-3	85-7A-T3(20.0)Y1			
		T-4	85-7A-T4(20.0)Y1			
		-	HA108(20)Y1, HA111(100)Y1			
		G-1	76 mm PVC(20.1)			
		PT	85EPT1Y1		0.9 m depth of cover	

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information	
Slope 73,						Previous
Unnamed N	271+442	T-3	85T6(5.9)A, TA16(0.9)A	IRC	0.91 m wood chips	
	271+458	P-1	6105(2.90), S(2.3), SP(1.40)		1.42 m wood chips	
	271+459	T-2	85T3(6.4)A, TA3(1.2)A		1.42 m wood chips	
	271+459	-	CT1(1.42)A, 85CT2(1.42)A		Horizontal strings	
	271+490	-	P14011 (2.2)		2.2 m below top of wood chips	
	271+491	T-1	85T1(6.15)A, TA18(1.2)A		1.10 m wood chips	1.45 m wood chips
	271+491	-	EMR-91-3536(1.8)		0.5 m west of pipe;	1.8 m below ground surface
			EMR-91-3539(1.8)		0.25 m west of pipe; to base of wood chips	
			EMR-91-3540(1.8)		0.25 m west of pipe;	1.8 m below ground surface
	271+519	-	T91-2 (4.5)		4.5 m below top of wood chips	
	271+524	-	P14012 (3.8)		3.8 m below top of wood chips	
	271+540	-	119(0.90)A		1.00 m wood chips	
Slope 74,						Previous
Unnamed S	271+779	T-1	85T9(6.45)A, TA13(1.31)A	IRC/IRT	1.4 m wood chips	1.75 m wood chips
	271+780	P-1	6103(2.75)S(2.5),SP(1.75)		1.4 m wood chips	1.75 m wood chips
	271+802	P	P22849 (3.8)			
	271+803	T	T97-12(8)			
	271+819	T-2	85T11(6.3)A, TA5+85TA16(1.55)A		1.6 m wood chips	1.60 m wood chips
	271+819	-	85T2(0.6)A		Horizontal string	
		-	T97-12(8.0), P22849(3.8)		No wood chips	
	271+909	T-3	85T13(5.0)A			
EMR-85-7B/ IPL-PSS	271+986	-	HA132(10)Y1		UF hole at interface	
		T-1	[5-7B-T1(5.0)Y1]	IRC/IRT	Removed by GSC/INAC - Sept 1996	
		T-2	[85-7B-T2(5.0)Y1]	UFT (in cleared area)	Removed by GSC/INAC - Sept 1996	
		T-3	85-7B-T3(20.0)Y1			
		T-4	85-7B-T4(20.0)			
		-	HA110(20)Y1			
		-	HA129(10)Y1		F hole at interface	
		G-1	76 mm PVC (20.3)		Abandoned by GSC/INAC - Sept 1996	
		PT	85EPT3Y1		0.9 m depth of cover	

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
EMR-85-7C	272+306 272+311	G-1A - T-1 T-2 T-3 T-4 - PT	76 mm PVC (20.3) 114A [85-7C-T1(5.0)Y1] [85-7C-T2(5.0)Y1] 85-7C-T3(20.0)Y1 85-7C-T4(20.0)Y1 HA109(20)Y1 85EPT2Y1	IRC/IRT	Abandoned by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 0.9 m depth
Slope 75, Unnamed N	273+622 273+633 273+634 273+659	- P-1 T-2 T-1	85CT3A 6102(2.65), S(2.25), SP(1.65) 85T16(6.35)A, 85TA4(1.2)A 85T12(6.3)A, 85TA5(1.2)A[3]	IRC/IRT	Previous At taper wood chips 1.4 m wood chips — 1.65 m wood chips 1.4 m wood chips — 1.65 m wood chips 1.4 m wood chips — 1.61 m wood chips
Slope 76, Unnamed S	273+714 273+715 273+734	T-1 P-1 T-2	85T4(6.2)A, 85TA12(1.3)A 6104(2.50), S(2.1), SP(1.50) 85T5(6.45)A, 85TA7(1.3)A	IRC/IRT	Previous 1.00 m wood chips — 1.50 m wood chips 1.00 m wood chips — 1.50 m wood chips 1.20 m wood chips — 1.75 m wood chips 85T4 possibly malfunctioning
Slope 79, Whitesands N	279+089 279+120 279+129 279+145 279+169 279+170	- T-1 - T-2 - T-3	85T8A 85TA14(1.29)A [85TA3(1.8)] [85T17(1.0)] HT147(1.15)A [85TA10+TA1(1.64)] [85PT1-4(0.6)] 85TA13(1.34)A	IRC	Side cut string 1.10 m wood chips 1.8 m wood chips Horizontal string 1.00 m wood chips 1.65 m wood chips Horizontal string 1.00 m wood chips
Slope 81, Ochre N	285+878 285+898 285+979 285+929	P-3 P-4 P-1 P-2	6115(8.84), S(6.1) 6139(16.76), 6121(8.84), S(5.5) 6145(20.73), 6110(7.62), S(5.3) 6140(23.47), 6114(9.75), S(7.6)	UFT	

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 82, Ochre S	286+731	-	82-21(0.8)A	IRC	Small hot area
	286+738	T-1	85T7(4.7)A, 85TA2+85TA8(1.8)A		1.83 m wood chips
	286+739	P-1	6120(2.83), S(2.4), SP(1.83)		1.83 m wood chips
	286+740	-	81-16(1.0)A		Small hot area
	286+740	-	82-4(2.8)A		Small hot area
	286+746	-	82-8(1.2)A, 82-23(0.8)A		Small hot area
	286+756	-	82-24(0.8)A		Main hot area
	286+757	-	82-20(0.8)A		Main hot area
	286+763	-	82-15(0.9)A		Main hot area
	286+764	-	82-1(4.0)A		Main hot area
	286+764.5	-	82-2(4.0)A		Main hot area
	286+765	-	82-3(2.4)A		Main hot area
	286+772	-	82-18(0.8)A, 82-10(1.1)		Main hot area
	286+773	-	82-22(0.8)A, 82-9(1.1)		Main hot area
	286+788	T-2	T9(4.75)A, 85TA6(1.3)A		1.42 m wood chips
	286+804	-	82-26(0.2)A, 82-12(1.3)A[3]		Experimental area
	286+804	-	82-17(0.6)A		Experimental area
	286+819	-	82-14(1.2)A		Experimental area
	286+820	-	82-131(1.2)A		Experimental area
	286+821	-	82-5(3.0)A		Experimental area
	286+822	-	82-7(0.9)A		Experimental area
	286+822	-	82-6(1.2)A		Experimental area
	286+822	-	82-27(1.0)A		Experimental area
	286+835	-	82-25(0.6)A		Experimental area
	286+836	-	82-19(0.2)A, 82-11(1.2)A		Experimental area
	286+858	T-3	T18(4.7)A, 85TA15(1.3)A		1.52 m wood chips
FH 8	311+739	PT	PT2-4A	-	0.95 m depth of cover
Slope 92, Unnamed Creek	318+	T SI	T97-13(8.0) SI-4 (11.2)	-	Placed 3 m off west side of ROW. In February 1998, five slope indicators, two thermistor strings, 14 settlement plates, seven pipe deflection indicators, and strain gauges on the pipeline were installed. All instrumentation except that noted here will be removed in February 1999 as part of a scheduled pipe replacement program.

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 99, Smith S	325+338	T-1C	T11(4.2)A, [85TA9(1.29)A]	IRC	1.63 m wood chips
	325+388	T-2	T6(5.0)A, 85TA11(1.27)A		1.27 m wood chips
	325+389	P-1	6113(2.80), S(2.2), SP(1.27)		1.27 m wood chips
IPL PT7	325+583	PT	85PT1-2A	-	0.95 m depth of cover
Slope 109	352+010	-	P14049	-	2.7 m below top of wood chips
	351+014	-	T91-6 Y1		6.0 m below top of wood chips
	352+014	-	EMR-91-3628(1.8)		0.5 m west of pipe; 0.3 m in wood chips
					1.5 m below ground surface
			EMR-91-3629(1.8)		0.25 m west of pipe;/ installed from 1.9 m to 3.7 m below ground surface
			EMR-91-3653(1.8)		0.2 m west of pipe; installed 0.3 m in wood chips
					1.5 m below ground surface
IPL-PT 8	352+466	PT	EMR-3A	-	1.0 m depth of cover
Slope 112, RBTM N	352+560	T-2	T15(5.0)A	IRC	
	352+560	P-2	6130(2.0)		
	352+613	T-1	T10(5.8)A		0.8 m wood chips
	352+613	P-1	6127(2.8), SP(0.8)		0.8 m wood chips
	352+615	-	T91-1Y1		6.0 m below top; of wood chips
	352+621	-	P14010		4.5 m below top of wood chips
FH 9	359+538	PT	PT1-2A	-	2.0 m depth of cover
	359+398	PT	PT2-8A	-	0.76 m depth of cover
FH 10	403+823	PT	PT1-2A	-	0.9 m depth of cover
	403+988	PT	PT2-8A	-	0.9 m depth of cover
IPL-PSS	469+961	5	HA131(10.0)Y1	UFS/UFC	UF hole at interface
	469+988	6	HA130(10.0)Y1	IPC	F hole at interface
EMR-84-4A	477+988	T-1	84-4A-T1(20.0)Y1[3]	UFS/UFC	
		T-2	84-4A-T2(20.0)Y1		
		T-3	[84-4A-T3(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-4	[84-4A-T4(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		G-1	[76 mm PVC (5.6)]		Abandoned by GSC/INAC - Sept 1996
		PT	EMR-1A		0.9 m depth of cover
		-	118A	-	Ditch thermistor

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
EMR-84-4B	478+116	T-1	[84-4B-T1(20.0)Y1]	UFS	Removed by GSC/INAC - Sept 1996
		T-2	[84-4B-T2(5.5)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	84-4B-T3(5.5)Y1		
		T-4	84-4B-T4(20.0)Y1		
		G-1	[76 mm PVC(13.5)]		Abandoned by GSC/INAC - Sept 1996
	478+838	PT	PT1-9A		0.9 m depth of cover
Slope 142, Mackenzie S	529+727	-	P14048	IPC	5.2 m below top of wood chips
	529+743	T-1	T22(4.5)A, [85TA13(1.3)]		1.40 m wood chips
			HT144(0.95)A		
		P-2	6135(2.83), S(2.4)		
	529+753	-	85CT1A		Side cut string
	529+760	-	P14052		2.89 m below top of wood chips
	529+764	-	T91-7Y1		6.0 m below top of wood chips
	529+776	T-2	85T10(4.75)A, [85TA14(1.3)]		1.25 m wood chips
			HT145(0.95)A		
	529+777	P-1	6141(2.45), 23070(2.45)		1.25 m wood chips
			S(2.1)SP(1.45)		
	529+778	-	85CT4A		Side cut string
Slope 146 Unnamed S	541+798	-	-	IRS	Required regular inspection of performance. Slope had previously failed.
EMR-85-8A	557+828	T-1	[85ET2(5.0)Y1]	IPS/IRC/UFC	Removed by GSC/INAC - Sept 1996
		T-2	[85ET1(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	85EDT8(20.0)Y1		
		T-4	85EDT5(20.0)Y1		
		G-1	[76 mm PVC(20.3)]		Abandoned by GSC/INAC - Sept 1996
		PT	85EPT8Y1		0.9 m depth of cover
		DT	[115A]		Ditch thermistor. Abandoned by GSC/INAC Sept 1996
EMR-85-8B	588+158	T-1	[85ET4(5.0)Y1]	PT/UFT	Removed by GSC/INAC - Sept 1996
		T-2	[85ET5(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	85EDT1(20.0)Y1		
		T-4	85EDT6(20.0)Y1		
		G-1	[76 mm PVC (20.4)]		Abandoned by GSC/INAC - Sept 1996
		PT	85EPT7Y1		0.9 m depth of cover

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
EMR85-8C	558+333	T-1	85ET6(5.0)Y1[3]	IRC/UFC	Site abandoned by GSC/INAC - Sept 1996. Site too wet to access and remove instrumentation.
		T-2	85-8C-T2(5.0)Y1		
		T-3	85EDT3(20.0)Y1		
		T-4	85EDT2(20.0)Y1		
		G-1	76 mm PVC(20.3)		
		PT	85EPT12Y1		0.9 m depth of cover
EMR-85-9	583+339	T-1	85-9-T1(5.0)Y1	BR	
		T-2	[85-9-T2(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	85EDT9(20.0)Y1		
		T-4	85EDT4(20.0)Y1		
		PT	85IPT9Y1		0.9 m depth of cover
		DT	[116A]	-	Ditch thermistor. Abandoned by GSC/INAC Sept 1996
EMR-85-10A	588+276	T-1	[85-10A-T1(5.0)Y1]	UFT/BR	Removed by GSC/INAC - Sept 1996
		T-2	[85-10A-T2(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	[85EDT10(20.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-4	[85EDT11(20.0)Y1]		Removed by GSC/INAC - Sept 1996
		G-1	[76 mm PVC (5.6)]		Removed by GSC/INAC - Sept 1996
		PT	85EPT4Y1		0.95 m depth of cover
EMR-85-10B	588+680	G-1A	[76 mm PVC(8.8)]	PT/UFT/BR	Abandoned by GSC/INAC - Sept 1996
	588+686	T-1	[85-10B-T1(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-2	[85-10B-T2(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	85-10B-T3(10.5)Y1		
		T-4	85-10B-T4(10.5)Y1		
	588+686	PT	85EPT5Y1		0.95 m depth of cover
EMR-85-11	597+396	T-1	[85-11-T1(5.0)Y1]	IPS/UFT	Removed by GSC/INAC - Sept 1996
		T-2	(85-11-T2(5.0)Y1)		Removed by GSC/INAC - Sept 1996
		T-3	85-11-T3(12.0)Y1		
		T-4	85-11-T4(12.0)Y1		
		G-1	(76 mm PVC(14.2))		Abandoned by GSC/INAC - 1996
		PT	85EPT11Y1		0.95 m depth of cover
EMR-85-12A	608+534	T-3A	(85-12A-T3A(16.4)Y1)	UFC/UFT	Removed by GSC/INAC - Sept 1996
	608+562	T-1	[85-12A-T1(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-2	[85-12A-T2(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	HA135(7.5)Y1		
	608+562	T-4	85-12A-T4(12.0)Y1		
		G-1	[76 mm PVC(10.9)]		Abandoned by GSC/INAC - 1996
		PT	85EPT6Y1		0.95 m depth of cover

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
IPL-PSS	608+672	1	HA133(6.7)Y1[3]	UFC	UF hole at inteface
	608+694	3	HA134(7.35)Y1	IRC	F hole at interface
EMR-85-12B	608+715	T-1	[85-12B-T1(5.0)Y1[3]]	PT/UFT	Removed by GSC/INAC - Sept 1996
		T-2	[85-12B-T2(5.0)Y1]		Removed by GSC/INAC - Sept 1996
		T-3	85-12B-T3(17.2)Y1		
	608+715	G-1	76 mm PVC (12.5)		
	608+729	T-4	85-12B-T4(9.7)Y1		
		PT	85EPT10Y1		0.95 m depth of cover
EMR-85- 13A	682+233	T-1	[85EDT7(20.0)Y1]	PT/IRC/IPT	Removed by GSC/INAC - Sept 1996
13B	682+422	T-1	[85-13B-T1(10.5)Y1]	PT/IPT	Removed by GSC/INAC - Sept 1996
13C	682+633	T-1	[85ET3(4.4)Y1]	PT/UFT	Removed by GSC/INAC - Sept 1996
EMR-84-5A	782+953	G-2	[76 mm PVC(20.6)]	PT/IRT	Abandoned by GSC/INAC - Sept 1996
	782+963	T-1	[84-5A-T1(5.2)Y2]		Removed by GSC/INAC - Sept 1996
		T-2	[84-5A-T2(5.6)Y2]		Removed by GSC/INAC - Sept 1996
		T-3	84-5A-T3(20.6)Y2		
		T-4	84-5A-T4(20.6)Y2		
	782+973	G-1	[76 mm PVC(20.6)]		Abandoned by GSC/INAC - Sept 1996
	782+963	PT	EMR-4A		0.77 m depth of cover
EMR-84-5B	783+253	T-1	HA123(5.5)Y1 [84-5B-T1(5.5)]	PT/IPT	
		T-2	HA124(5.7)Y1 [84-5B-T2(5.7)]		
		T-3	HA125(20.5)Y1 [84-5B-T3(20.5)]		
		T-4	HA126(20.5)Y1 [84-5B-T4(20.5)]		
	783+253	PT	EMR-5A		0.85 m depth of cover
	783+263	G-1	76 mm PVC (20.4)		
EMR84-6	819+488	T-5	[84-6T5(10.1)Y2]	PT/IRC/IPT	Removed by GSC/INAC - Sept 1996
	819+508	PT	EMR-6A		0.8 m depth of cover
	829+508	T-1	84-6-T1(5.5)Y2		
		T-2	84-6-T2(5.4)Y2		
		T-3	84-6-T3(20.6)Y2		
		T-4	84-6-T4(20.7)Y2		
	819+518	G1	[76 mm PVC(20.4)]		Abandoned by GSC/INAC - Sept 1996

LEGEND

1. **Boreholes:**

- T = Thermistor string boreholes
- P = Piezometer boreholes
- PT = Pipe thermistor string location
- SI = Slope Indicator

2. **Instrumentation:**

- 85T14(6.15) - thermistor string 85T14 installed in natural soils at 6.15 m depth. Where wood chips are in place, depth of thermistor string below ground surface is the difference between depth noted and the thickness of wood chips given in "other information" column.
- 85TA14(1.2) - thermistor string 85TA14 installed in wood chips to depth of 1.2 m.
- 85TA5+85TA16(1.55) - thermistor string 85TA16 placed in wood chips to depth of 1.55 mm.
- 85PT-1 - pipe thermistor string 85PT-1 attached to pipe with depths of cover noted in "other information" column.
- Remaining thermistor strings not included in the above categories (e.g. 85CT series and few 85T and 85PT1 series) were installed in wood chips horizontally and at ground surface - wood chips interface at sidecuts or taper section as noted in "other information" column.
- 6128(2.6) - piezometer 6128, tip at 2.6 m depth; where wood chips are in place, depth of tip below ground surface is the difference between depth noted and the thickness of wood chips.
- SP 1.65 - settlement plate set at 1.65 m depth and usually at the ground surface - wood chips interface.
- [85TA3(1.8)] - instrumentation that has been destroyed/damaged.

3. **Thermistor Types:**

- A = thermistor string fabricated with Atkins thermistor beads, 0°C at 16.325 k
- Y1 = thermistor string fabricated with YSI 44033 thermistor beads, 0°C at 7.355 k
- Y2 = thermistor string fabricated with YSI 44032 thermistor beads, 0°C at 94.98 k

4. **Soil/Ice Conditions:**

IPC - ice-poor clay	UFC - unfrozen clay	BR - shallow bedrock
IPT - ice-poor till	UFS - unfrozen sand	PT - thick peat layer
IRC - ice-rich clay	UFT - unfrozen till	

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NWZ PIPELINE CORRIDOR GOVERNMENT (GSC) RESEARCH SITES AND THERMAL INSTRUMENTATION - STATUS AS OF OCTOBER 1998

SITE	KP	CABLES / SENSORS	SOIL PROBES	LOGGERS	COMMENTS / RATIONALE
Gibson Gap		HT221, HT222			long term pf/climate studies - GSC; 100 m hole
Kee Scarp		HT137, HT139, HT152		AES logger on shallow cable; 2 grnd surf temp loggers (Stowaways)	GSC long term pf/climate studies, effect of elevation and 1954 fire; 130 m hole; AES automated climate station
Norman Wells	0	Water Treatment Plant, Arena		2 XLs; 2 Stowaway gr. surf. temp. loggers; 1 Vemco air temp. logger	town site; 1 cable at water treatment plant and 2 at arena
84-1	0.1	T1, T2, T3, T4, T5, PT, air	off-ROW	T4, T5 on XLs, Stowaway with ext. sensor on top pipe; XL on soil probe	easy access for reading and later removal
Bosworth N	0.36	pipe			leads left, no logger attached
Pipe/Soil, Freeze/Thaw	2	pipe, ground, air	ground on-ROW	7 XLs, 1 RDL100, 3 Stowaways, 4 Vemcos	pipe/soil interaction in response to new operating temperatures
Sewage Rd	2.9	pipe			leads left by PVC tube, no loggers
Pipe uplift	5	5 cables on-ROW; 1 off-ROW; pipe		1 pipe Stowaway; 1 pipe Vemco; 2 XL loggers on ROW	
Quarry Road	7	side of pipe		XL (3 sensors)	alkaline batteries, 24 pin connector
kp 12 - firebreak	12	3 cables		2 cables in break on XLs	2 cables are in firebreak north of ROW 1 cable is on-ROW, on west side of firebreak

SITE	KP	CABLES / SENSORS	SOIL PROBES	LOGGERS	COMMENTS / RATIONALE
84-2A	19	T1, T2, T3, T4, pipe, air, HT140, HT138, HT153	off-ROW	3 XLs on HT140, pipe sensor, soil probe off-ROW; 2 Stowaways	off-ROW deep hole (130 m) for long term pf/climate studies; AES automated climate station, w-T4 on AES logger, T1 cable needs replacing
84-2B	19.3	T1, T2, T3, T4, pipe		T1, T3; T4 on XLs	w.c. slope response to new pipe temps regime
84-2C	19.6	T1, T2, T3, T4, pipe			T3/T4 enable monitoring of long term response of slopes and off-ROW
Prohibition N	30	pipe			two leads from external Stowaways; no loggers
Jungle Ridge	48.5	pipe		XL	alkaline batteries, 24 pin connector; no logger--malfunctioned October 98, had no spare to replace it
84-3A	79.2	T1, T2, T3, pipe, air	off-ROW	T1, T3 on XLs	impacts of burn off-ROW; thawing still on ROW; T4 cables chewed off, frozen in could not install new one
84-3B	79.4	T1, T2, T3, T4, pipe		T4 on XL; ext. sensor Stowaway @ pipe	impacts of burn off-ROW; thawing still on ROW
Saline North	176.1		off-ROW	XLs (Tarnocai)	2 probes one each in burnt and unburnt terrain
kp 182	182	5 short cables		XLs on cables; gr and air VEMCO loggers	impact of burn on slope; n-factors studies; 4 set-ups on burnt slope, one on unburnt , 2 grnd surf. loggers and one air temp. loggers at each location
85-7A	271.2	T1, T2, T3, T4, HA108, HA111, pipe, air	off-ROW	T2, T3, T4 on XLs	warm ice-rich fine grained soils; ROW and off-ROW response to pipeline and climate change (90 m hole)
85-7B	272.0	T3, T4, T5(HA110), pipe			T3, T4 & T5 contribute to documenting longer term response of terrain to pipeline clearing and climate changes

85-7C	272.3	T3, T4, GH, T5(HA109), pipe			T3, GH, T4 & T5 contribute to documenting longer term response of terrain to pipeline clearing and climate changes
SITE	**KP**	**CABLES / SENSORS**	**SOIL PROBES**	**LOGGERS**	**COMMENTS / RATIONALE**
84-4A	478.0	T1, T2, pipe		T2 on XL logger	response of uf terrain to clearing and climate change
84-4B	478.1	T4, pipe			response of unfrozen terrain to clearing and climate variability and change
85-8A	557.8	T3, T4, pipe, air	off-ROW	T4 on XL logger; one Stowaway	response of frozen mineral soil to pipeline clearing / climate
85-8B	558.2	T3, T4, pipe			permafrost degraded near pipe; peatland (palsa) response to pipeline/ climate changes
85-8C	558.3	Too Wet to Access			Site abandoned; all cables and PVC tubes still in place
85-9	583.3	T1, T3, T4, pipe			response of unfrozen mineral terrain to clearing and climate change
85-10A	588.3	pipe			cap left over GH and T4 at ground surface
85-10B	588.7	T3, T4, pipe			response of peatland to pipeline clearing / climate changes
85-11	597.4	T3, T4			off-ROW (thin pf) climate change studies; on-ROW unfrozen
85-12A	608.6	T4, pipe			response of unfrozen organic terrain to clearing and climate change
85-12B	608.7	T3, T4, GH(HT196) T1, T2 still here but not accessible		T3, T4 on XLs; ground and air temp. Vemcos off ROW	T1 and pipe sensors are no longer accessible due to collapse and water filled trench; response of frozen peat plateau to pipeline and climate change; thermal response of organic terrain slower than that of mineral

84-5A	783	T3, T4, pipe T1, T2			response of frozen peat plateau with thick permafrost (~10m) to pipeline and climate change; thermal response of organic terrain slower than that of mineral
SITE	**KP**	**CABLES / SENSORS**	**SOIL PROBES**	**LOGGERS**	**COMMENTS / RATIONALE**
84-5B	783.3	T1, T2, T3, T4, GH (HT197), pipe (kaput), air	off-ROW	T2, T3, T4 on XLs	response of frozen peat plateau with thick permafrost (~10m) and thick peat (~7m) to pipeline and climate change; thermal response of organic terrain slower than that of mineral
84-6	819.5	T1, T2, T3, T4, pipe			response of frozen peat plateau with thick peat (5-6 m) ; thermal response of organic terrain slower than that of mineral
"Thaw settlement sites" :	kp95 kp135 kp477	1 cable 2 cables 2 cables			cables all remain in place

NOTES:

1. Kp 7 pipe sensors are located just south of intersection of pipeline and Quarry Road. Three sensors were installed here (taped to pipe with duct tape); 1st and 3rd sensors are kaput.
2. Prohibition N pipe sensor leads are just south of KP marker post.
3. Saline sites are located south of sleeved section of pipe and north of cathodic protection site (orange pipe/riser), on west side of ROW.
4. IN Sept 1996 PVC tubes of geophysical access holes were cut off at ground surface at the following sites: 7A, 7B, 8A
5. Unless otherwise indicated the two shallow on-ROW cables at thermal fences were removed and PVC tubes cut-off at ground surface in Sept 1996. At several sites a cap was left over the casing at the ground level, so that re-entry into silicone casing can be possible at a later date.
6. In Sept. 1996: TDR rods all removed; all Tarnocai ROW soil probes removed.

APPENDIX B

SITE MONITORING LOCATIONS AND PIPE SETTLEMENT SOURCES

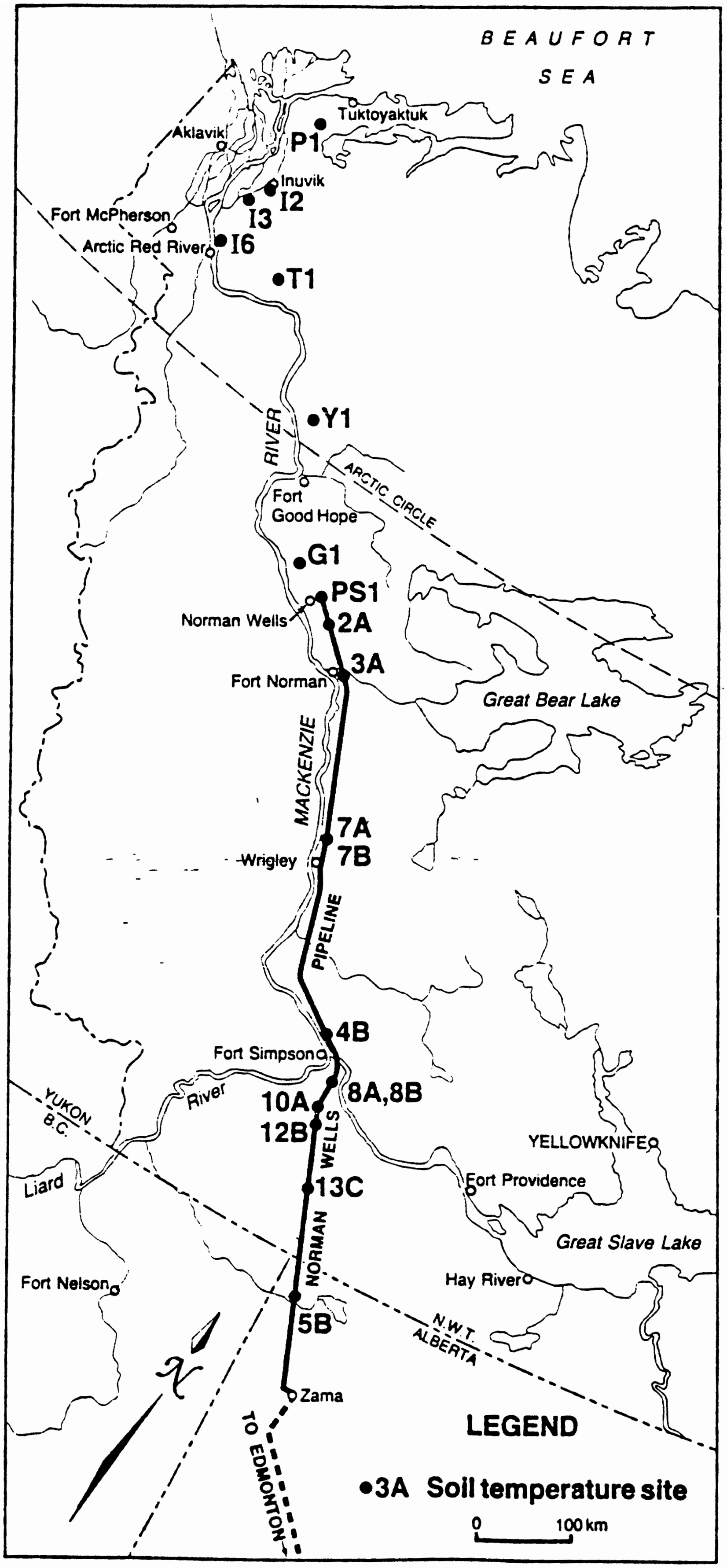
TABLE B1 SITE DESCRIPTIONS

No.	NAME	KM	DESCRIPTION (at time of establishment)
84-1	**Pump Station 1**		**Widespread permafrost**
		0.02	Ice-rich silty clay; widespread permafrost
84-2	**Canyon Creek**		**Previously cleared alignment, thaw sensitive slopes, widespread permafrost.**
	A	19.0	Level location, frozen till with low ice content
	B	19.3	East-facing slope with a 1 m insulating wood chip cover
	C	19.6	Uninsulated section of west-facing slope
84-3	**Great Bear River**		**Joint IPL site with thaw sensitive slope**
	A	79.2	Stratigraphically complex ice-rich alluvial terrace deposits in widespread permafrost; cliff-base
	B	79.4	Cliff-top lacustrine deposits with aeolian veneer
85-7	**Table Mountain**		**Joint IPL site with thaw sensitive slopes**
	A	271.2	Ice-rich lacustrine plain(old seismic line)
	B	272.0	Drillpad clearing at bend on top of north facing slope, ice-rich lacustrine plain
	C	272.3	New clearing on ice-rich lacustrine plain
84-4	**Trail River**		**Pipeline previously traversed frozen ground**
	A	478.0	Unfrozen saturated sands/silts in dune hollow
	B	478.1	Dry sands and silts in dune crest
85-8	**Manner's Creek**		**Rapidly changing permafrost conditions**
	A	557.8	Thin peat with thick (10 m) permafrost
	B	558.2	Thick(2.7 m) peat with thin(4 m) permafrost
	C	558.3	Thin peat (1 m) with thin (1 m) permafrost
85-9	**Pump Station 3**		**Pipe previously traversed frozen section**
		583.3	Unfrozen granular soils
85-10	**Mackenzie Highway South**		**Unfrozen/frozen interface**
	A	588.3	Helipad clearing in unfrozen terrain
	B	588.7	Thin (3 m) permafrost with 2 m peat cover
85-11	**Moraine South**	597.4	Thin (<4 m) permafrost in helipad clearing
85-12	**Jean Marie Creek**		**Unfrozen/frozen interface**
	A	608.6	Thin unfrozen peat
	B	608.7	Thick ice-rich peat plateau; 4 m permafrost
85-13	**Redknife Hills**		**Frozen/unfrozen interface; single cables only**
	A	682.2	Frozen (6 m) terrain surrounding large fen
	B	682.4	Frozen (6 m) terrain at fen border
	C	682.6	Unfrozen terrain in fen
84-5	**Petitot River North**		**Degrading peat plateau**
	A	783.0	Ice-rich peat (3.5 m); (15-18 m) permafrost
	B	783.3	Very thick icy peat (7 m); 12 m permafrost
84-6	**Petitot River South**		**Peat plateau preceded by unfrozen fen**
		819.5	Thick (5 m) ice-rich peat; 7 m permafrost

The above are the principal study sites established in 1984 and 9185 during pipeline construction. Additional key sites instrumented since then are:

- Freeze-Thaw /Pipe Soil study site at Kp2 established in July 1994
- Kp182 burn area slope thermal investigations established in August 1995
- Short term studies of hot spot were conducted in 93 to 95 on select slopes

From: Burgess, M. 1995. PTRM contribution to 1995 annual report



Prepared by the Information Systems and Cartography Unit, Land Resource Research Centre, Agriculture Canada, 1989.

FIGURE B1 LOCATION OF SITE CLIMATIC SITES ALONG PIPELINE ROUTE

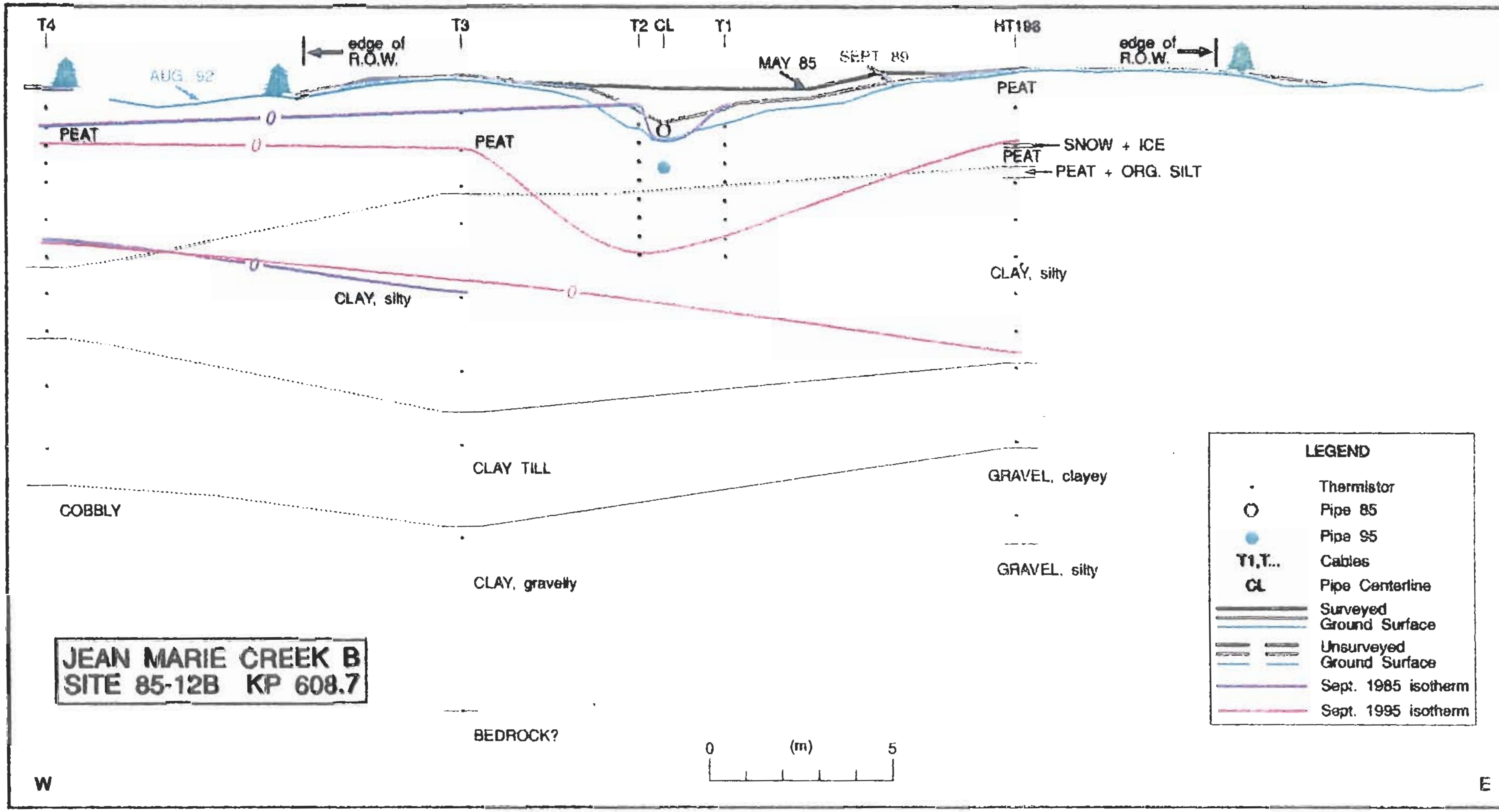


FIGURE B2 ISOTHERM DATA AT SITE 85-12B

From: Burgess, M. 1995. PTRM contribution to 1995 annual report

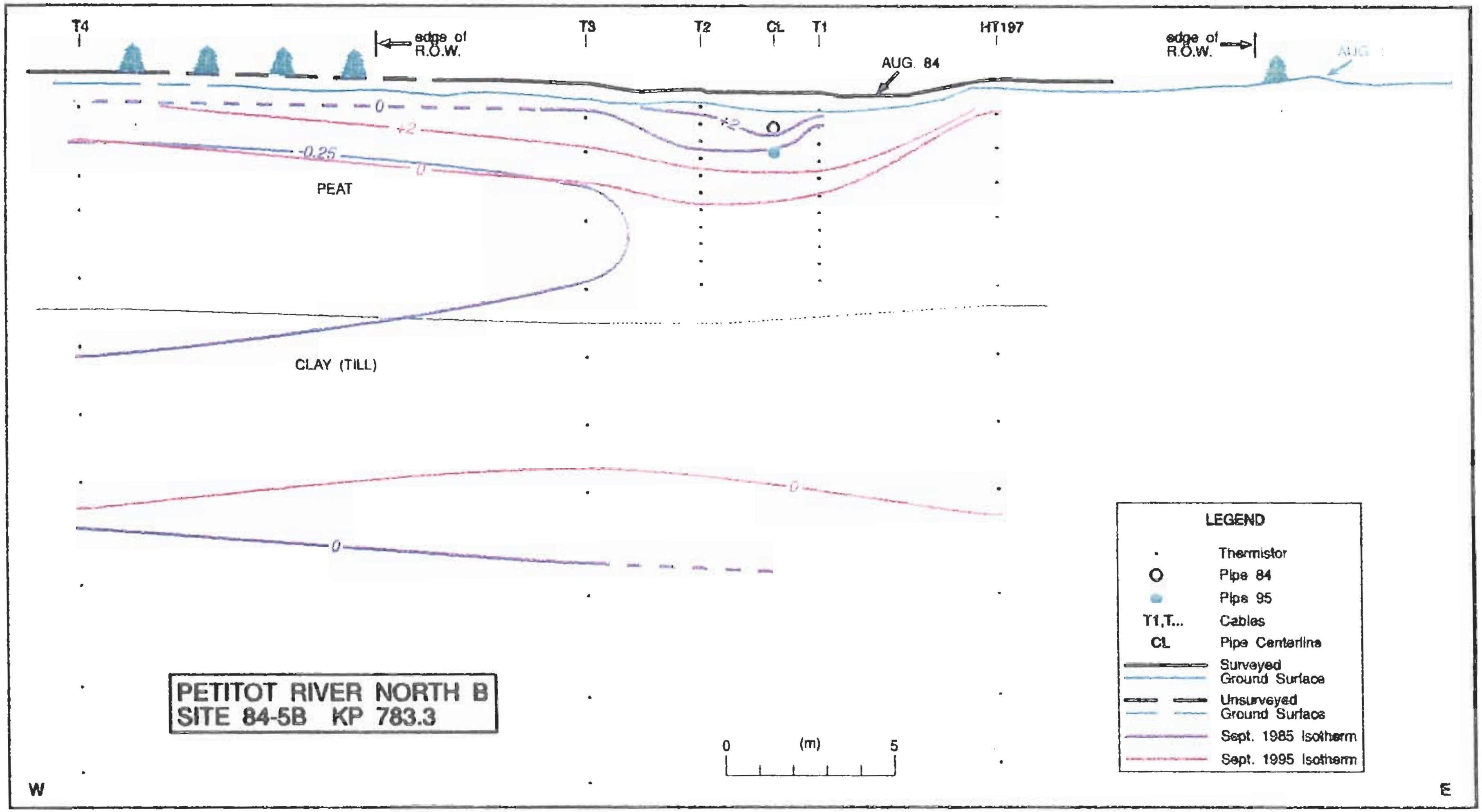


FIGURE B3 ISOTHERM DATA AT SITE 84-5B

From: Burgess, M. 1995. PTRM contribution to 1995 annual report

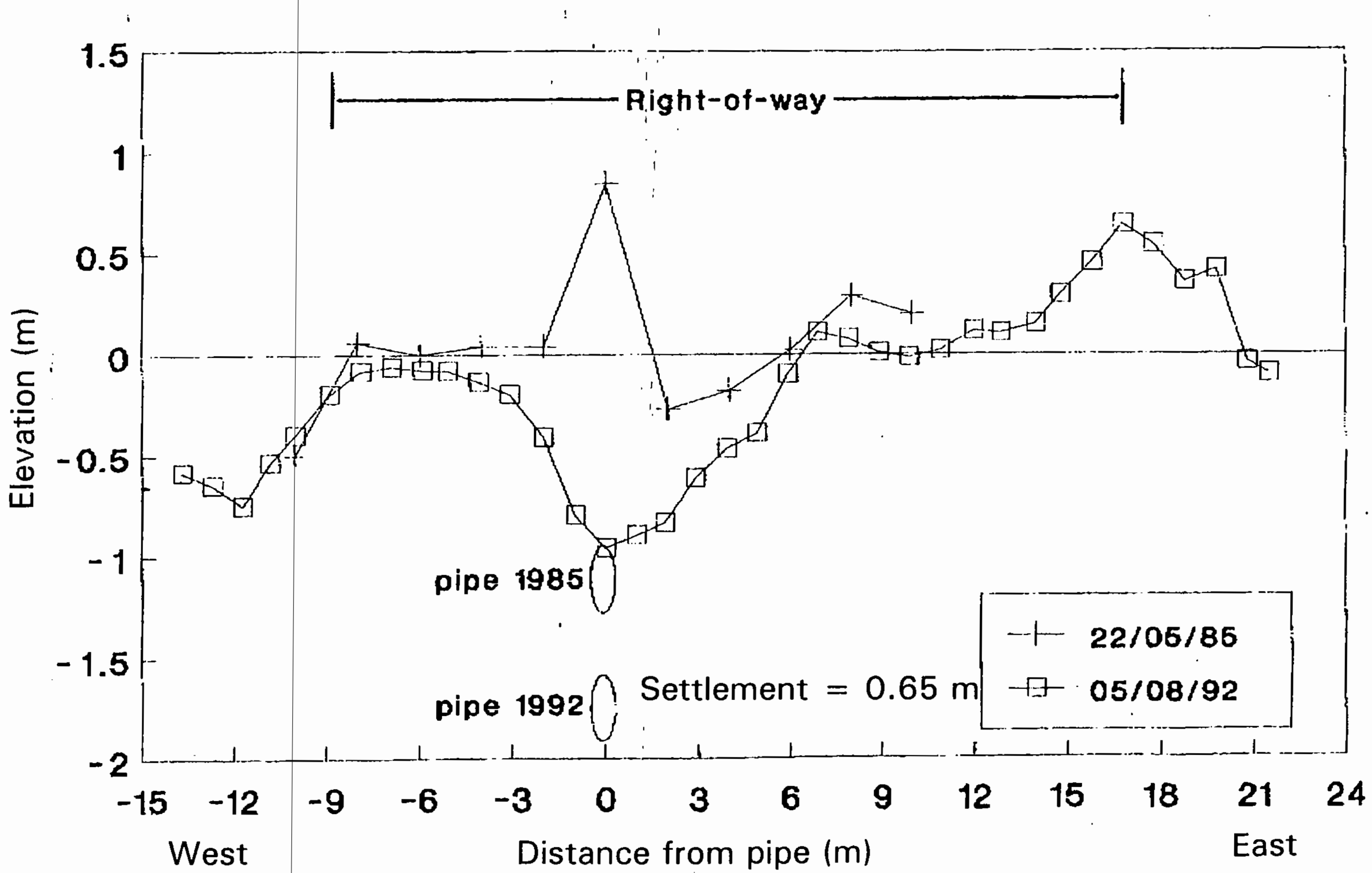


FIGURE B4 TOPOGRAPHIC SURVEY ACROSS RIGHT-OF-WAY AT SITE 85-12B

From: Burgess, M. 1997. Unpublished report

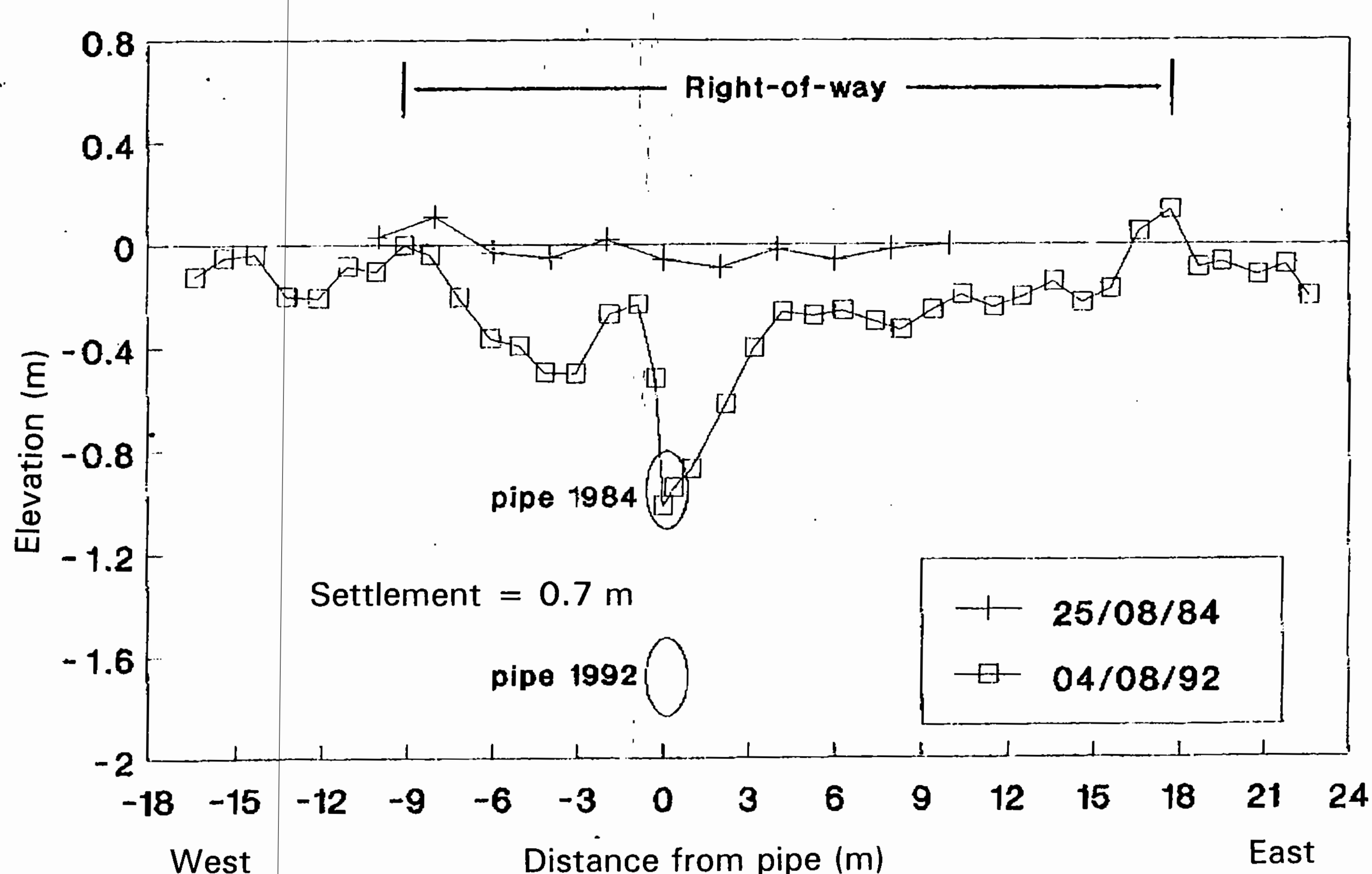


FIGURE B5 TOPOGRAPHIC SURVEY ACROSS RIGHT-OF-WAY AT SITE 84-6

From: Burgess, M. 1997. Unpublished report