GEOLOGICAL SURVEY OF CANADA

OPEN FILE 5702

Monograph on the Norman Wells Pipeline Geotechnical Design and Performance – 2006 Update

Naviq Consulting Inc. and AMEC Earth & Environmental

2007

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Available from
Geological Survey of Canada
601 Booth Street
Ottawa, Ontario K1A 0E8

Naviq Consulting Inc. and AMEC Earth & Environmental
2007: Monograph on the Norman Wells Pipeline Geotechnical Design and Performance -2006 Update. Geological Survey of Canada Open File 5702, 214 p.

Open files are products that have not gone through the GSC formal publication process.

Foreword

The 869 km Norman Wells to Zama Oil Pipeline operated by Enbridge Pipelines (NW) Inc. (formerly Interprovincial Pipe Line (NW) Ltd.) is the first completely buried oil pipeline constructed within the discontinuous permafrost zone of Canada. The pipeline has been in operation since 1985. An extensive program to monitor the performance of both the pipe and the right-of-way has been carried out by Enbridge since construction as per National Energy Board requirements. In addition, a government-industry collaborative Permafrost and Terrain Research and Monitoring Program (PTRM) was implemented under the Environmental Agreement between the pipeline operator and Indian and Northern Affairs Canada (INAC). The Geological Survey of Canada (GSC) of Natural Resources Canada (NRCan) was a key participant in the PTRM and continues to collaborate with Enbridge in research and monitoring along the pipeline corridor.

The results of Enbridge's performance monitoring program are reported annually to the National Energy Board and other regulatory agencies. Results have also been published in a number of scientific papers. In the late 1990s, a monograph on the Norman Wells Pipeline design and performance was completed (GSC open File 3773 by AGRA Earth and Environmental Ltd. and Nixon Geotech Limited, 1999) under contract to GSC/NRCan and in collaboration with Enbridge. The monograph, prepared by the geotechnical engineering consultants involved in the pipeline design, construction and operation, focused on an analysis of the data resulting from Enbridge's monitoring programs and included some of the government's monitoring data. The release of the monograph as a GSC open file ensured that results from the performance monitoring program and lessons learned were synthesized and publicly available.

In 2004-05 and 2006-07, the GSC undertook, in collaboration with Enbridge and with funding from the Program on Energy Research and Development and the Northern Energy Development Memorandum to Cabinet (MC), to update the original monograph. The work was undertaken through contracts to AMEC Earth and Environmental (formerly AGRA) and Naviq Consulting Inc. and their geotechnical engineering consultants involved in the Norman Wells Pipeline. Results are presented up to 2006 in the present Open File including results from more recently established monitoring sites that were not available in the 1999 publication. The latest synthesis focuses largely on slopes but updated results from monitoring sites in overland terrain have also been provided including results from several GSC monitoring network sites.

This Open File presents information that facilitates improved understanding of long-term geotechnical and environmental performance of pipelines and rights-of-way in permafrost environments. In light of proposed hydrocarbon development in the western Canadian Arctic, such information is important to both the design and environmental assessment of future pipelines in order to ensure safe reliable pipelines and sustainable development in northern Canada.

Any interpretations and opinions expressed herein are those of the authors and do not necessarily reflect those of the Geological Survey of Canada, Earth Science Sector, Natural Resources Canada.

Sharon Smith
Geological Survey of Canada, Natural Resources Canada
October 2007

Acknowledgements

Funding support for this project was provided by NRCan, INAC, the Northern Energy Development MC and the Program for Energy Research and Development. The preparation of the synthesis was conducted in collaboration with Enbridge and the support provided by Ann Marie Tout, Darren Skibinsky and Shadie Radmard is appreciated.

Reference

AGRA Earth & Environmental Limited and Nixon Geotech Ltd. 1999. Monograph on Norman Wells pipeline geotechnical design and performance. Final report to Department of Natural Resources, March 1999. Geological Survey of Canada Open File 3773, 120 pp.

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MONOGRAPH ON THE NORMAN WELLS PIPELINE GEOTECHNICAL DESIGN AND PERFORMANCE 2006 Update

Prepared for:

The Geological Survey of Canada
Natural Resources Canada

Prepared by:

Naviq Consulting Inc.

and

AMEC Earth & Environmental

Calgary, Alberta

March 2007

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Background to Monograph

This version of the document represents the third update since its original publication in 1997. The original document was compiled as a collaborative effort between Nixon Geotech Limited and AGRA Earth & Environmental Limited (now operating as AMEC Earth & Environmental). In 1999, AMEC Earth & Environmental undertook an update to include selected information on several pipeline wrinkles that were identified and mitigated. Information for a second update was compiled and analyzed in 2005 by AMEC Earth & Environmental, but this work was not incorporated into a published document. Finally, this update is intended to incorporate the 2005 information as well as additional information and data that came to the public's attention during the public hearing process for the proposed Mackenzie Gas project.

The primary purpose of this monograph is to document 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.).

Since the initial stages of pipeline construction Natural Resources Canada and the Geological Survey of Canada (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). The program was designed to assess the impact on, or interactions with, the terrain and to ensure that lessons learned would be documented and 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 a table of contents initially approved by the GSC. 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 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 was 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 2007 edition of the monograph was prepared by Naviq Consulting Inc. under contract NRCAN-07-03015. The contract administrator is Ms. Marnie Waller.

Acknowledgements

Enbridge Pipelines (NW) Inc. (Enbridge) as owners and operators of the Norman Wells pipeline have generously permitted the authors full access to reports, databases, and photographs as part of the research for this document. The manager of the Norman Wells pipeline is Ms. Anne-Marie Tout. Enbridge staff in Norman Wells and Fort Simpson NT, and Edmonton Alberta have shared information and clarified details where gaps were present. Furthermore, pipeline maintenance staff collected geotechnical instrumentation for over twenty years in all weather conditions to help provide insights into the geotechnical and geothermal responses taking place.

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 from design through construction and operation of the pipeline. Dr. Nixon and Mr. Hanna were lead contributors to the original version of the Monograph. Dr. Jim Oswell, who is currently involved in much of the ongoing monitoring and geotechnical assessment for the pipeline company, was also contributor to the original monograph, and was principal author of all subsequent editions.

AMEC Earth & Environmental provided access to files and databases related to the pipeline. The cooperation and assistance of AMEC staff is gratefully acknowledged.

The technical manager for the Geological Survey of Canada was Dr. Sharon Smith. Dr. Smith provided additional information on terrain monitoring and assisted by providing recent publications and reviewing the document prior to publication.

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1.0 EXECUTIVE SUMMARY

This monograph has been prepared to highlight the history, lessons learned and 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 continue to 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 geotechnical engineering design, 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 databases 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 databases 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 in 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 4000 m^3/day (26,000 bbl/day) over a 20 year period has been transported to southern markets without significant interruption. Current volumes (2007) are in the order of 3000 m^3/day. 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 this document.

PART A: Project Philosophy, Approach

Significant Design or Mitigative Feature	Philosophy, Approach	Lessons Learned
Ambient Pipeline	NW crude oil is light, wax was removed at process facility; feasible to operate as ambient pipeline (product flowing at ground temperature); 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 for pipeline immediately downstream of pump station number one. This was revised in 1993.	Pipe operating temperatures warmed up faster than expected - probably related to warmer than normal climate conditions in the first years, warmer conditions in trench than the adjacent right-of-way, and initial oil cooling difficulties; warmer summer Norman Wells oil temperatures since 1993 have posed no significant concern in the subsequent years.
Routing	Selected overland route on existing cut lines where practical - CNT line, and seismic lines. Many slopes were on undisturbed alignments.	Trench and thaw settlement on the right-of-way is generally as originally estimated, after approximately 20 years of the 25 year design life, except for organic terrain, which has been greater than design predictions.
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 - three spread seasons instead of the four that were planned north of Ft. Simpson. Little snow for snow roads - most of travel in the first construction season was on bare ground without significant impact. Wheel ditching very successful, except in very bouldery soil.
Reclamation	Intended to implement a rapid re-vegetation program in mineral soils 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 decreased with time. Cross drainage erosion on low angle slopes was generally well controlled by the use of sand bag diversion berms and ditch plugs.

Monitoring (by operator and government)	Pipe and ground temperatures monitored to identify 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 and most still operating after 20 years. Bears and fires have been the biggest problem. Should have placed some instrumentation closer to the pipe on slopes; new instrumentation installed below original thermistor depths to monitor deeper thaw. A lack of good survey benchmarks were a problem for measuring pipe settlement. Several survey benchmarks were installed in later years at specific locations, as required.
Operations and Maintenance	Weekly flyovers 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 erosion of a loose backfill mound. Not any threat to pipeline integrity, however, significant remediation effort required in early years. Aerial patrols were reduced in frequency in the 2000s, reflecting stability of the right-of-way.
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 special off-road vehicles to provide better land access to the right-of-way in all seasons.

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 investigations 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 database provided considerable useful information (and drilling program experience) and saved the owner/design team the need for too much more additional 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 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 more historical air photos in less remote areas. Only one case where bank erosion uncovered the pipe (Ochre South). Seasonal flooding and subsequent channel migration at Hodgson Creek necessitated deeper burial.

	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.

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	Expected to respond to adjacent ground temperatures and therefore little impact from pipe expected; immediately downstream of Pump Stations 2 and 3 expected to be warmer and pipe temperature would input heat to ground for about 50 km before running at ambient; chilled oil from Pump Station 1 would actually cool the ground for some 50 km (see details on subsequent pipe temperature 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 initial chilling problems. Also, from 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 give 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 visible 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 temperatures since 1993.	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 in the first years after the temperature change. 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 lsope 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.

		A	B	C	D	E	F	G	H	I
	Impact/Issue	Expected/Predicted Impact	Mitigation Applied	Monitoring Approach	Actual Impact	Effectiveness of: Mitigation	Effectiveness of: Monitoring	Operational Improvements	Lessons Learned	Outstanding Issues
2	Thaw Settlement - ROW Surface	The long term thaw settlement beneath previously cleared right-of-ways estimated to 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-NRCan (GSC) thermal fences at many representative terrain units surveyed by the GSC for settlement since construction.	Estimates of actual right-of-way thaw settlement range from 0.05 m to 0.95 m, with an average of 0.53 m for 15 locations after 12 years of the 25 years design life. Small amounts of additional settlement developed between 12 and 20 years. 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 at a decreasing rate and frequency. Monitoring to continue as long as the pipeline is operating.
3	Thaw Settlement - Pipe	Due to thaw settlement beneath the ROW, 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% for tension and -0.75% for compression.	No instrumentation with respect to pipeline integrity; Gross ROW surface differential settlement to be monitored by line patrol; smart internal inertial tool (GEOPIG) developed 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 to 1997.	Adequate as long as limiting strain not exceeded.	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 internal pipeline tool 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 were obtained.	None	None	None
4b	Near Norman 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. Uplift is likely a combination of increased delta-T effects and localized frost heave.	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 has confirmed the remediation design was adequate to arrest the seasonal delta-T effects but perhaps not the seasonal frost heave.
5	Stability of Ice-Rich Slopes									

		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			
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 modelled as thermal passive, that is, design did not anticipate thermal impact of pipe on slopes.	The width of the ROW was reduced to 13 m on some 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 water 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.	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 developed, many of which appeared several after start of operation - not a particular concern. On some slopes the amount of thawing after 12 years exceeded the predicted 25 year thaw depth in part due the larger than anticipated thermal input from the pipe, and warmer initial ground temperatures. Thaw depth progress can be correlated to initial ground temperature. Sites with initial ground temperatures colder than -1 °C at 5 m depth have experienced less thawing than warmer sites.	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 closer to the pipe for detecting the maximum thaw; some could have been deeper. Physical probing has been very informative.	None	Anticipate more influence from relatively warm pipe temps. beneath the wood chips. Avoid wood species (aspen/ poplar) and/or rotten wood that most likely contributed most to the extended heat generation.	Majority of insulated slopes are performing as well or better than predicted. How-ever, 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).
5b	Pore pressures	As the thaw progressed, 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 of excess pore pressures, that is, when no hot spots and when away from the influence of a warm pipe.	Piezometers installed on representative number of insulated slopes at time of construction	Generally observed pore pressures less than predicted. Long-term monitoring suggests that slopes are equilibrating to a hydrostatic condition. Slopes with high ice contents may be draining while other slopes, possibly with less ice may be saturating.	The insulation reduced the rate of thaw on all slopes.	Piezometers worked well and did detect some excess pore pressures within the thawing zone. However, in several slopes the thaw has now progressed deeper than original piezometer installation and deeper installations were required.	In 1994 vertical slotted drains were installed in three slopes, experiencing high pore pressures	Since piezometers need to be at the actual thaw front to observe maximum pore pressures, anticipate need to install new instruments at deeper depths.	Monitor and assess piezometric pressures in deeper piezometers. Porewater pressures expected to be worse 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 water 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 for ice-rich soils.	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 six slopes in this condition. In 2006, approximately six slopes continued to have evidence of lower than desirable factors of safety.	Generally 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 peizometer instrumentation added since construction.	Anticipate influence of pipe temperatures under wood chips.	Continued close monitoring and stability assessment on approximately 15% of insulated slopes.

		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			
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 ROW, 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 or at the Wrigley and Mackenzie pump stations.	N/A	Adequate		Design assumptions valid to date.	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 ROW and wood chip surfaces following a fire.	Fires in 1994 and 1995 affected 90 km and 53 km of the right-of-way respectively, particularly between Norman Wells and the Ochre River. 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. A forest fire in 2003 along the northwest edge of the Town of Norman Wells came close to the right-of-way. A forest fire in 2004 along the Northwest Territories – Alberta border burned across the right-of-way damaging some GSC thermal fence installations.	N/A	Adequate	Soaking/ wetting wood chips from creeks during fire. Post fire hydro-seeding of burnt terrain adjacent to several slopes (Slopes 29B and 48B) to promote re-vegetation and stability.	Expansion of fire breaks around valve sites and pump stations.	Ongoing monitoring of some “adjacent” consequential instability (e.g. KP 182).
5f	Creeping Slopes	Not explicitly recognized as a design issue.	None	None	Four slopes have been identified as exhibiting creep-like slope movements (Slopes 44/45 (KP 133), Slope 84 (KP 311) and Slope 92 (KP 318)). In two cases slope movements have induced wrinkles in the pipeline necessitating cut-out and replacement of the pipeline.	Cut-out of wrinkled pipe has been successful.	GEOPIG identified wrinkles. Slope indicators monitored slope movements.	None	Need to recognize issue.	Need additional research to help recognize slopes susceptible to creep movements.
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 till slopes (Slopes 8, 13, 18 Vermillion S, Seagrams S)	The six year and 12 year thaw depths are consistent with design predictions. Thaw progression has been found to correlate with initial ground temperatures. Colder initial ground temperatures have thawed less than warmer sites.	N/A	Thaw at some sites have exceeded the deepest thermistor bead.	None	Assumptions adequate	None If on-going monitoring is required deeper thermistor cables will be installed.
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.		N/A	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.	N/A	Good	None	As above	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			
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 that 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 ROW was considered susceptible, rip-rap was placed on the full width of the ROW 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. In 2003 a rock berm constructed along the left bank of Hodgson Creek upstream of the right-of-way failed resulting in significant water flow along the right-of-way for several hundred metres. The pipeline was not exposed. The berm was re-constructed and flow with the creek channel restored.	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. See Column G for remediation taken.	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 sag 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 Environment Canada 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									

		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			
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. 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 ROW 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, ecpecially in organic terrain.	In second winter construction, travel lane was on bare ground too.	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.	None.
9b	Construction	Potential for short and long 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.	N/A	Adequate.	None.	None.	None.
10	Drainage and Eroson									
10a	Drainage and Erosion	The surface of the right-of-way was expected to be erodible on slopes greater 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 ROW. It was recognized that not all locations requiring mound breaks could be identified during design and construction and the maintenance 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 ditch plugs, mound breaks and diversion berms at typical spacing from 10 to 50 m.	Visual observation.	There was notable 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. At a few overland sites cross drainage reduced the cover on the pipe, were dealt with by place weights and granular fill. In the 2000s, off right-of-way erosion has encroached onto the right-of-way at KP 314. This is exacerbated by cross right-of-way surface flow. Remedial work to control the surface drainage and arrest the erosion was undertaken.	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 ramp 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									

		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			
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 revegetation 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.	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, NT, owned by Imperial Oil Resources Limited. 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 were 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, Stuchly and Pick, 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 generally allowed to 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 still develop in buried pipelines because of differential heave or settlement. It is therefore of considerable importance for future developments in this terrain (and other discontinuous permafrost areas) that the amount of frozen ground and the number of thermal interfaces be quantified as well 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 seasonally maintained 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 chart highlighting the location of construction spreads and the season in which 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 710, which corresponded to the design width (7 feet, 2.1 m) and depth of ditch (10 feet, 3.05 m). These machines, which had been developed for proposed, larger diameter gas pipelines in the Canadian Arctic, were capable of excavating a smooth regular trench that made laying the pipe and backfilling much easier than a ditch excavated by backhoe. Smaller, conventional ditchers were used where feasible, particularly in the southern sections. 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 affect 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 significantly compressed 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 needs to be considered 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. Figure 2-3 presents a graph of the flow volumes between 1985 and 2006.

2.2 Ambient Temperature Pipeline

One of the most important and unique features of the project was to operate the pipeline at or close to ambient conditions. Previous oil projects either constructed or contemplated (i.e. the Trans Alaska Pipeline System (TAPS)) 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 bulb 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 to 20 m deep to form beneath the pipe. For this reason, the TAPS was elevated on piles for the most part where ice-rich 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 pumped cool. The approach for the project was to adopt a more passive thermal design for the pipeline compared to the TAPS. 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, at the thermal interfaces along the route, depending on the situation. However, for this to occur, sufficiently long lengths (i.e. several kilometres) 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.

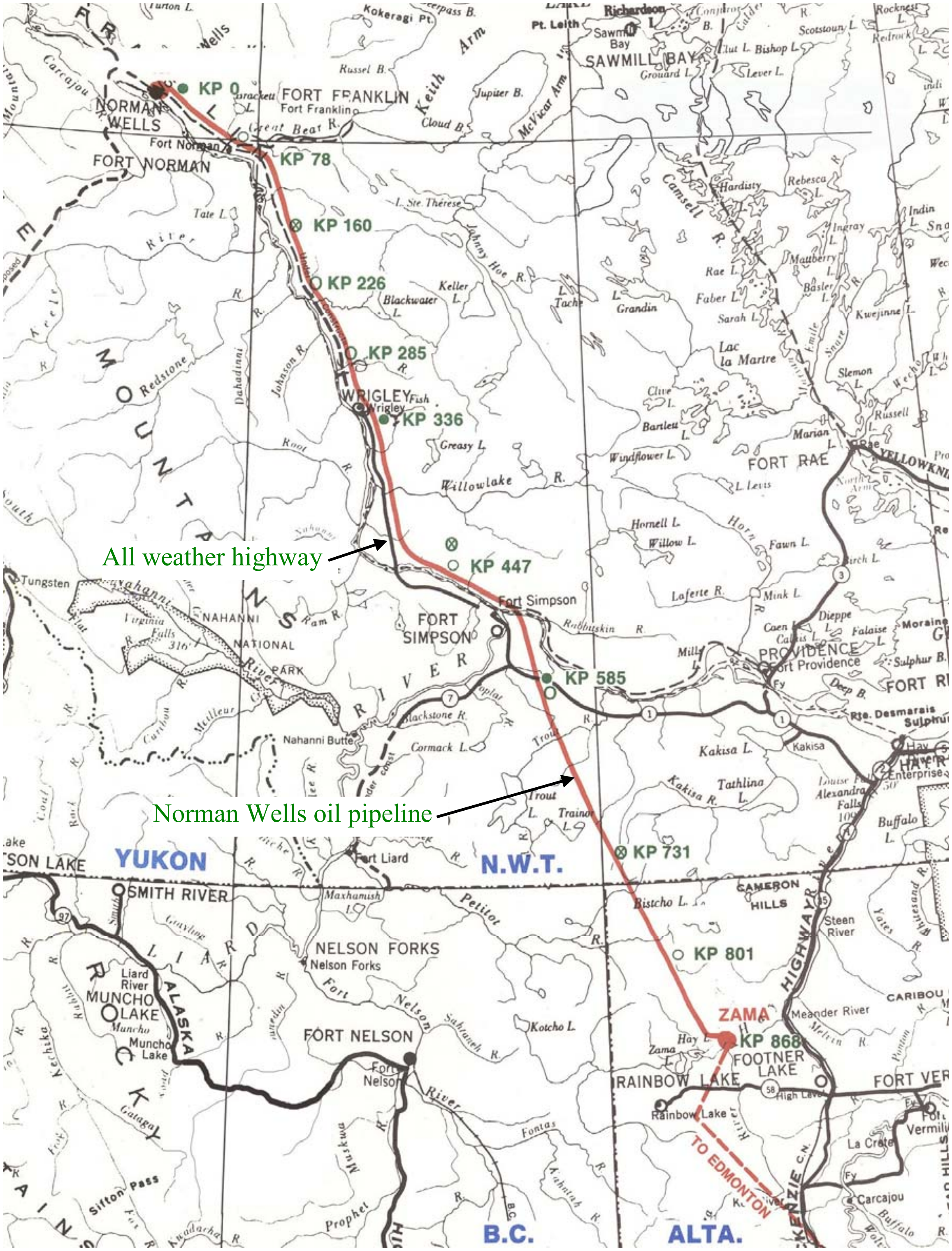


Figure 2-1: **Route map of the Norman Wells pipeline.**
Pump Stations are located at KP 0, KP 336, and KP 585.

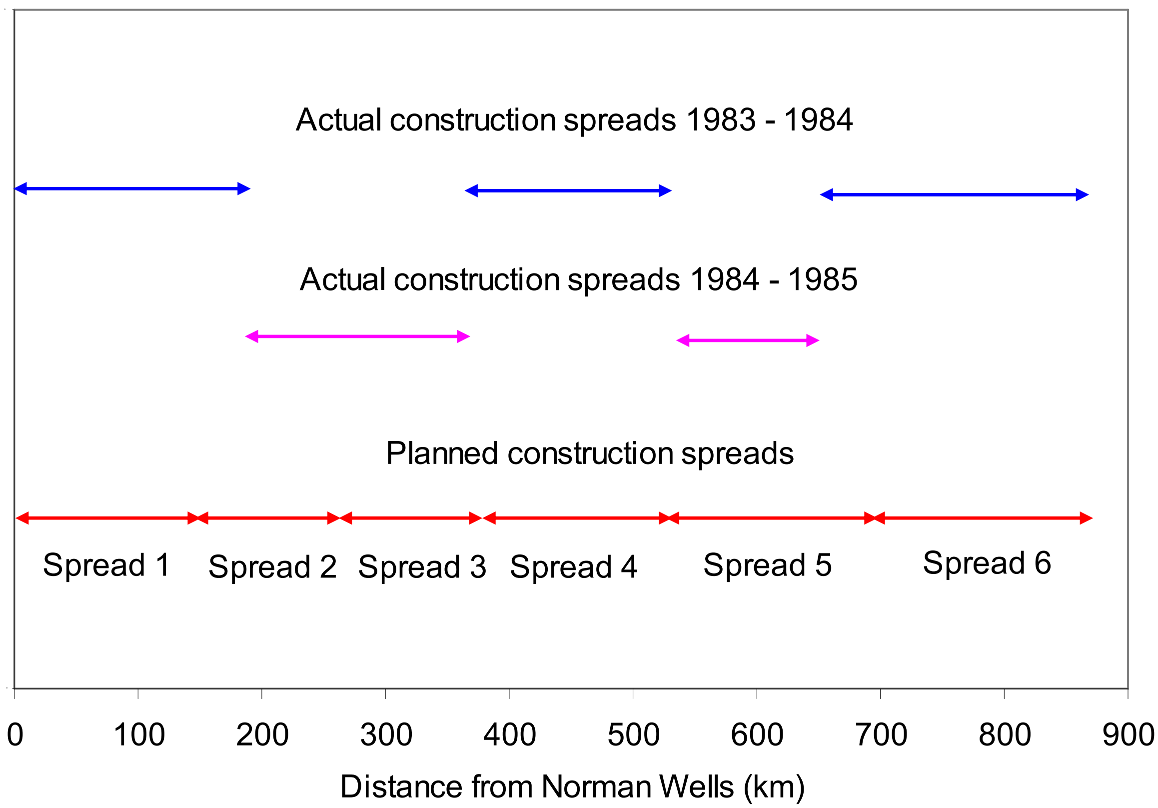


Figure 2-2: Construction schedule for Norman Wells pipeline.

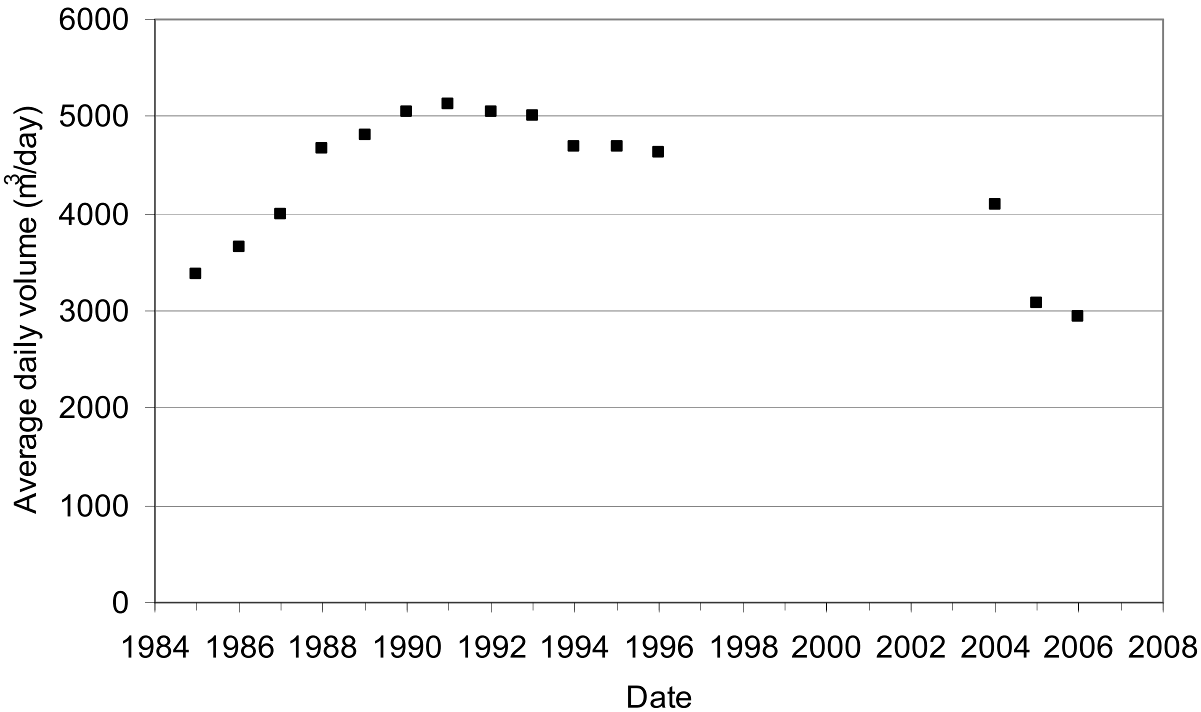


Figure 2-3: Average daily through-put of pipeline.

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. Because of 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)

An important feature of the design approach was to route the pipeline as far as possible along existing lines of disturbance. An existing Canadian National Telegraph (CNT) cut line paralleled the proposed route for much of its length in the northern sections. In some cases the strategy of following existing linear disturbances resulted in increasing the length of the pipeline, 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 was difficult to establish. Therefore, following existing cut lines reduced the possibility of intercepting icy soils in the top few metres beneath the pipeline, but did not always prevent this occurrence completely.

Later, ditchwall logs and geophysical profiles would show 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 undertaken 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 areas, due to insufficient snow fall, snow pad thickness was not sufficient to prevent disturbance to the right-of-way surface organic cover. However, construction was planned so that no permanent (e.g. gravel) work pad remained following construction, and where construction had exposed mineral soils on the right-of-way, these areas were re-seeded.

2.5 Reclamation

The philosophy of construction and 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 on steep slopes. To this end rapidly establishing agronomic species were applied 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 ground 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. Enbridge continues to clear brush from the right-of-way to permit ground observations, to provide helicopter landing areas, and for other maintenance purposes.

2.6 Monitoring and Maintenance

Under the terms of the regulatory approval to proceed with the project, the owner was required to implement a monitoring program. The specific details of the program were left to the owner to develop. The program consisted of line patrols, and the installation of pipe, and geotechnical instrumentation to monitor 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, Burgess, Harry and Baker,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 the active erosion periods in the first few years following construction. The patrols also included regular checks on the controls for automatic block valves. Over the years the type and nature of monitoring has evolved. In the years after beginning operations, monitoring and maintenance focused on issues such as ditchline settlement, right-of-way erosion and warm wood chips. In later years, brushing of the right-of-way, replacing timber cribs, and pipeline dents and wrinkles dominated activities.

Table 2-1, Table 2-2 and Table 2-3 present details of pipeline and right-of-way monitoring and maintenance activities.

Table 2-1: Monitoring and maintenance activities (1985 – 1990) (Doblanko, Oswell and Hanna, 2002, with additions).

Monitoring and Maintenance Activity	85/86	86/87	87/88	88/89	89/90
Visual line patrols – frequency = weekly	√	√	√	√	√
Backfilling ditch subsidence	√	√	√	√	√
Installation of rip rap	√	√	√	√	√
Erosion control activities	√	√	√	√	√
Reseeding/revegetation	√	√	√	√	√
Cooling wood chips		√	√	√	√
Installation of geotechnical instrumentation		√			
Repairs of wood chip retaining timber cribs			√	√	√
Brushing right-of-way					√
Pipeline maintenance			√	√	

Table 2-2: Monitoring and maintenance activities (1990 - 2006) (Doblanko, Oswell and Hanna, 2002, with additions).

Monitoring and Maintenance Activity	90/ 91	91/ 92	92/ 93	93/ 94	94/ 95	95/ 96	96/ 97	97/ 98	98/ 99	99/ 00	00/ 01	01/ 02	02/ 03	03/ 04	04/ 05	05/ 06
Visual line patrols – frequency = weekly	√	√	√	√	√	√	√	√	√	√	√					
Visual line patrols – frequency = 10 days												√	√	√	√	√
In-line-inspection (inertial geometry tool)	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Filling voids in wood chip cover	√															
Installation of rip rap													√	√		
Erosion control activities													√	√	√	
Forest fire impact remediation				√	√											
Installation of geotechnical instrumentation		√			√	√	√	√		√				√		√
Repairs of wood chip retaining timber cribs										√						
Thaw depth probing						√		√		√		√		√		√
Brushing right-of-way												√	√	√	√	√
Pipeline maintenance								√		√		√				√

Table 2-3: Schedule of inertial geometry tool monitoring (1998 – 2006) (Doblanko, Oswell and Hanna, 2002, with additions).

Year	Norman Wells to Wrigley Pump Station	Norman Wells to Mackenzie Pump Station	Norman Wells to Zama
1998			√
1999	√		
2000		√	
2001	√		
2002			√
2003	√		
2004		√	
2005	√		
2006			√

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 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 issue soon after the end of construction.

Pipe curvatures at several discrete locations were monitored in the 1990s by measuring relative elevations of adjacent points along the top of pipe, and differentiating the readings 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 provide stable elevation data 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, Smith and Pick, 1989). The instrument is known as an “in-line inspection” or “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 2006. Table A-1 lists the both the owner installed instrumentation and some instrumentation installed by Government of Canada agencies. Table A-2 provides a listing of NRCan/GSC site instrumentation.

3.0 INVESTIGATIONS, PREVIOUS STUDIES AND DATA BASES FOR INPUT TO DESIGN

During the fifteen years prior to the Norman Wells pipeline 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 were developed to address the unique or special design problems associated with the presence of continuous and discontinuous permafrost.

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 when 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 to avoid potentially unstable terrain. This work built on the routing studies for other Mackenzie Valley pipelines from the 1970s. This included, for example, the adoption of physiographic and climatic regions that were developed of the Canadian Arctic Gas project in the 1970s.

The alignment was chosen on the basis of air photo interpretation and route reconnaissance, and took advantage of existing cleared rights-of-way, such as the Canadian National Telegraph line to Inuvik and other cut-lines. In some geographical areas along the general route, 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. Furthermore, in light of the small diameter of the proposed pipeline, routing around or between obstacles was easier than for a large diameter pipeline.

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 included desirable terrain for the pipeline, potentially problematic terrain to be avoided, and the presence of granular borrow sites. (Sections of the route that were subject to pre-clearing were also identified on the alignment sheets.) Permafrost terrain or frozen-unfrozen interfaces could, in a general manner, be identified from the aerial photographs. Figure 3-1 shows an example of the construction alignment sheet, containing, among other information, the terrain typed air photo mosaic.

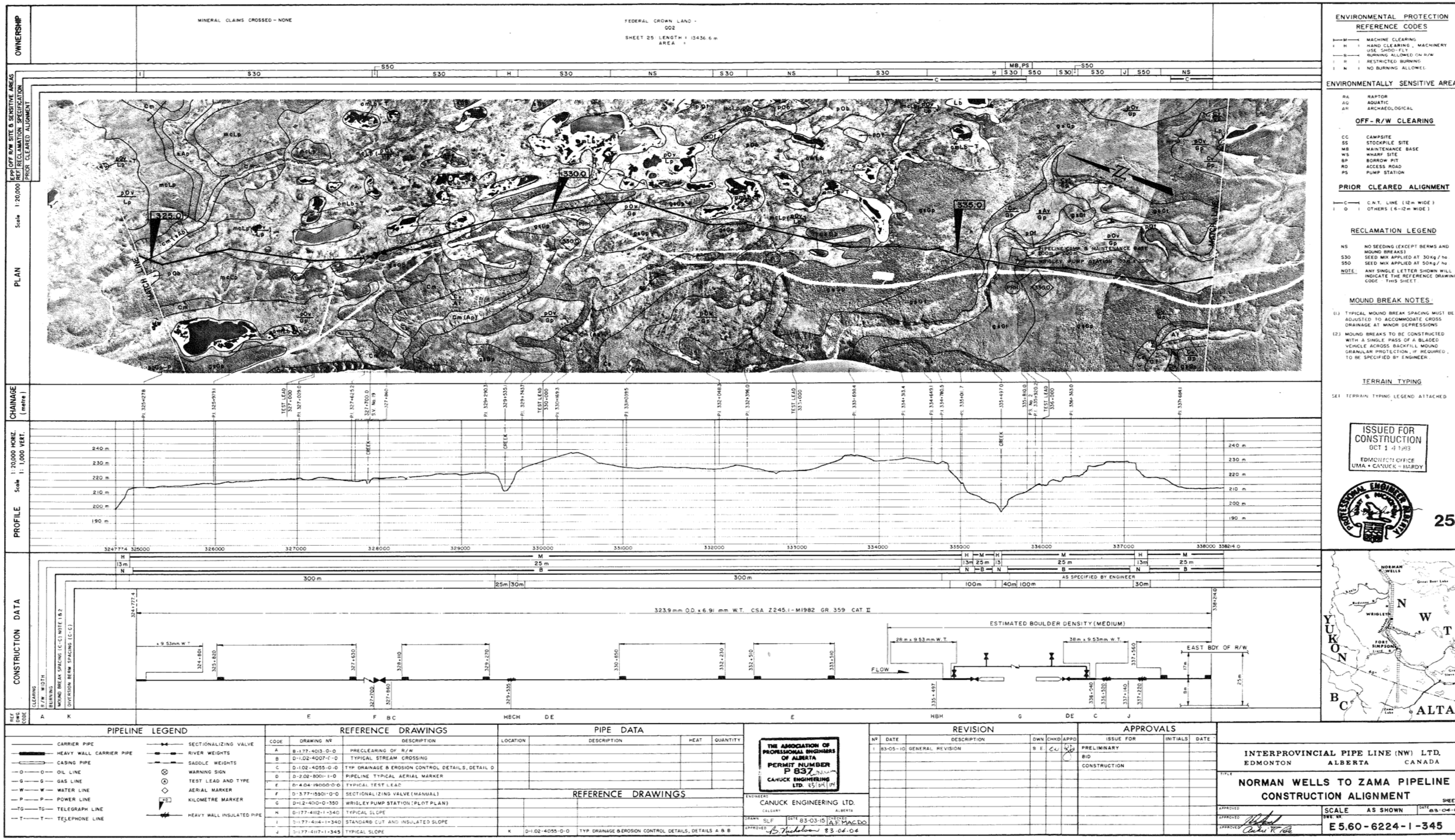


Figure 3-1: Example of construction alignment sheet.

From a design point of view, the potentially most problematic terrain units 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.

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

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 Willowlake River, where the pipeline alignment followed previously cleared cut-lines 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. (see Section 4.3.3.)

Seven sites were also instrumented along the most northerly portion of the alignment, between Norman Wells and Wrigley 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 Willowlake River	77
376 – 866	Willowlake River to Zama Lake	34

3.2 Geotechnical Investigations

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. Thermistors cables to measure ground temperatures were also installed at selected sites.

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 (CAGSL)
- 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. This database has since been expanded and enlarged to include other geotechnical information from other sources (Smith, Burgess, Chartrand, and Lawrence, 2005).

3.4 Laboratory Testing

As part of the Norman Wells pipeline geotechnical 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 2.3 and from other research (for example, 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	22	12.5	1760
- high normal stress	31.5	4	

Table 3-4 presents a summary of the geothermal properties of the soils.

Table 3-4: Summary of geothermal parameters.

Soil	Typical Thickness (m)	Thawed Conductivity (W/m°C)	Frozen Conductivity (W/m°C)	Total Gravimetric Water Content (%)	Unfrozen Water Content (%)	Dry Density (kg/m^3)
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 (Kay, Allison, Botha and Scott, 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 **3-2** presents an example of a 400 m transect showing how sharp and 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. Furthermore, no credit was taken for those sections of the pipeline route that traversed previously disturbed terrain and where some thaw settlement had already occurred.

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.

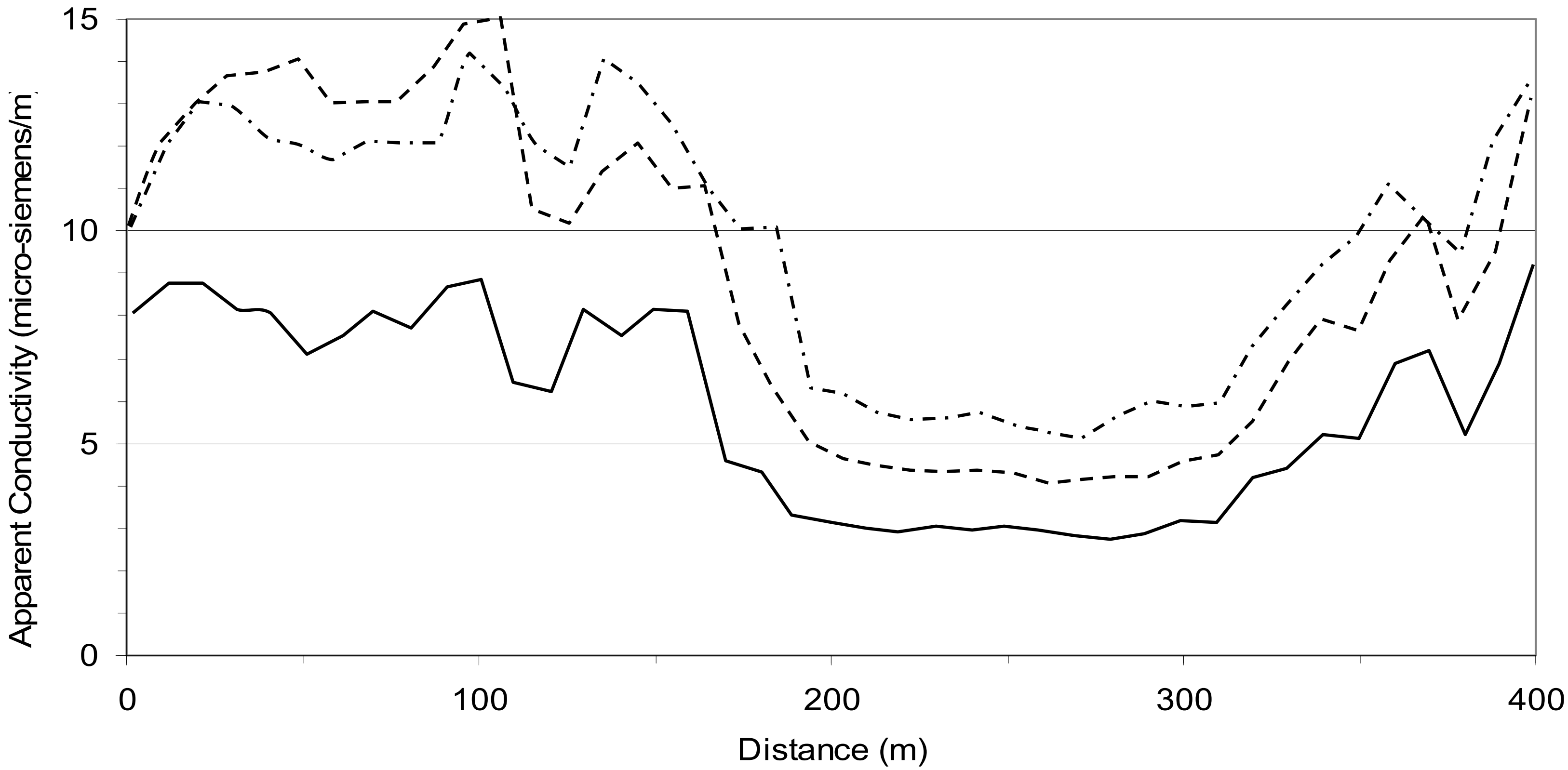
3.7 Storm Runoff Predictions

Fifty-seven stream crossings were investigated during the pipeline design phase. 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 was 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 estimate of the 100-year flood discharge.



Pine - Spruce - Tamarack	Open Short Black Spruce with some Pine	
Unfrozen	Frozen	Unfrozen

Figure 3-2: Geophysical transect showing distinct frozen and unfrozen terrain zones.

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 the report by Northern Engineering Services Company Limited (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 Environment Canada stations. To avoid re-working much 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 along the route 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, boreholes drilled for this project by Enbridge and Government of Canada departments and agencies, such as the Geological Survey of Canada provided additional coverage for ground temperatures.

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

4.0 DESIGN AND MITIGATION STRATEGIES (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 temperature scenarios were obtained assuming the pipe would be fully equalized to the ground temperature at pipe burial depth. These were used in the 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, because of 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 to 50 km down the pipeline (Hardy Associates (1978) Ltd., 1983a). 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 initial part of the route, the ambient environmental temperatures would tend to control the temperature of the flowing oil after some short distance.

In the initial years of operation, considerable difficulties were experienced in cooling the oil to a discharge temperature of -1°C or colder during the peak summer periods. Expensive additional refrigeration equipment had to be available on standby to handle peak cooling demands for a relatively short period during the summer season. This later led to an operational change in 1993 involving seasonal increases in pipe inlet temperature to as high as +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.

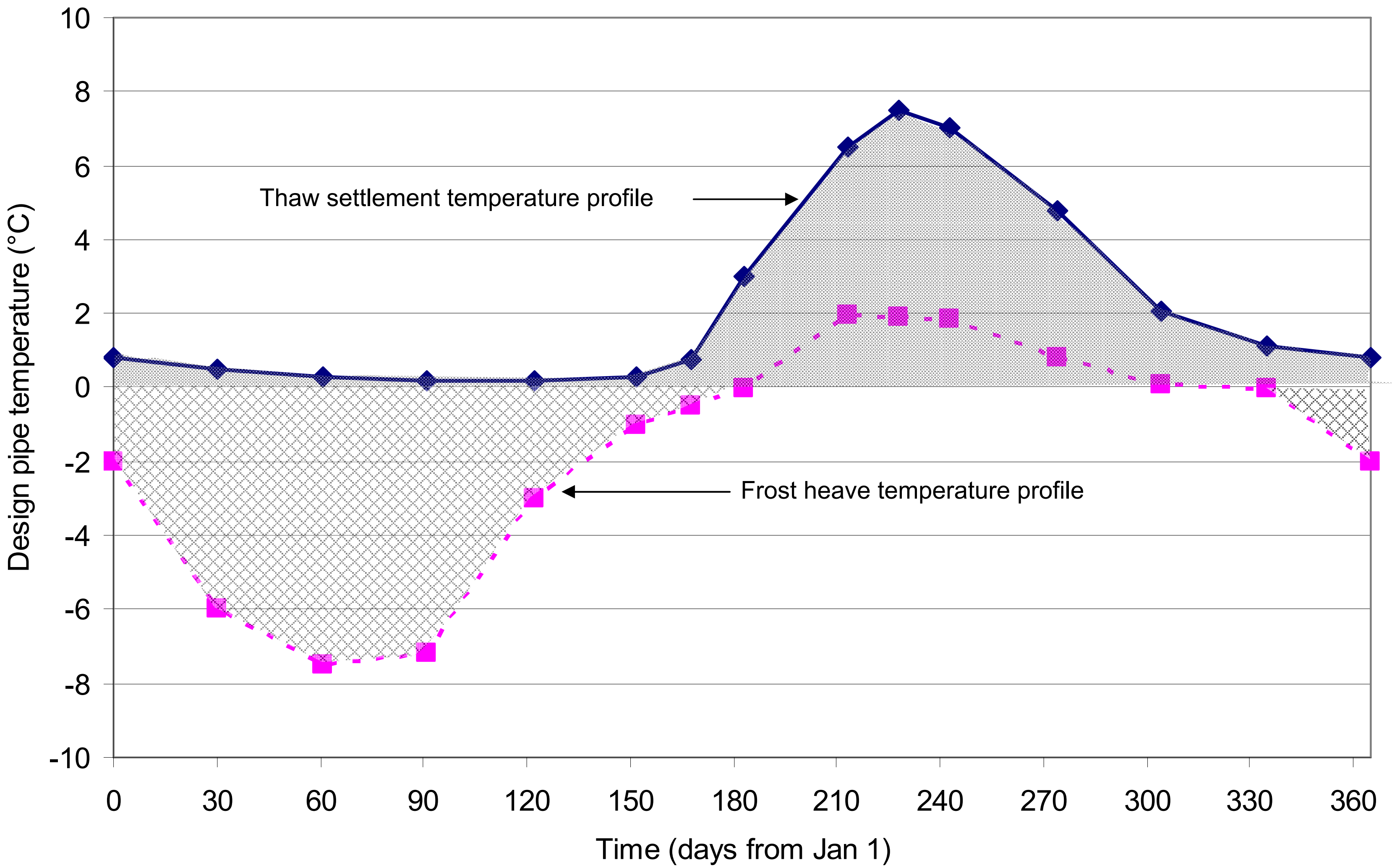


Figure 4-1: Pipe temperature variations used in design for thaw settlement and frost heave.

These temperature histories were used in subsequent two-dimensional thermal simulations for thaw settlement and frost heave analysis and design.

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 did not account for additional loadings arising from several loading mechanisms.

The temperature differential (delta-T) 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 this 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 source (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 design 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 (Workman, 1977) was used to perform 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.

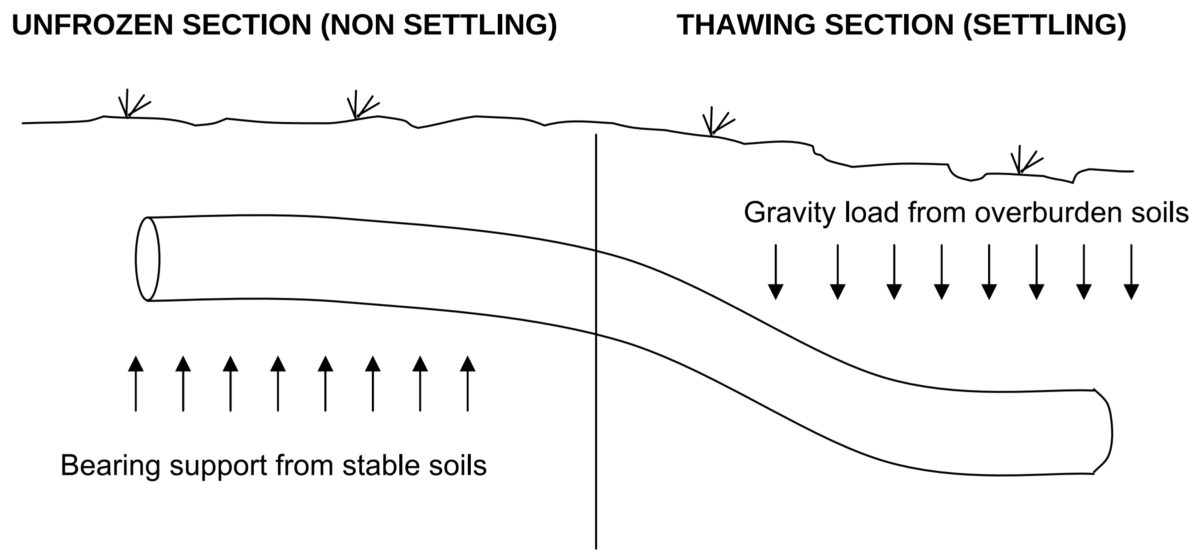
Design concepts developed for this oil pipeline differ significantly from design concepts used for other proposed Arctic pipelines. These differences are summarized as follows:

- The installation of a pipeline buried 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 practical, the pipeline was located on previously disturbed and cleared rights-of-way (seismic cut lines, and telegraph line).

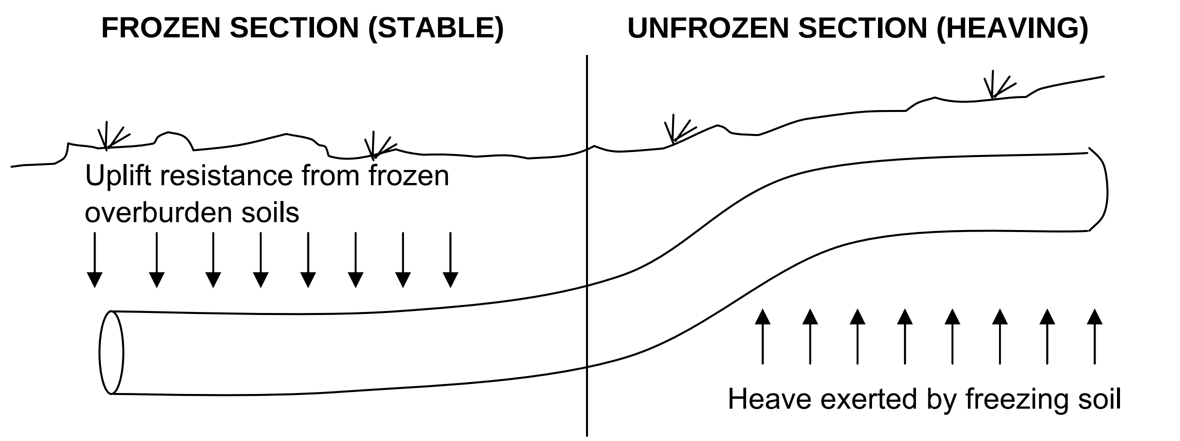
The more important implications of applying the above design criteria to the oil pipeline were to introduce secondary soil loadings on the pipeline, namely those loadings caused by differential settlement, frost heave and seismic activity.

4.3 Thaw Settlement

As mentioned previously, the low energy input of the pipeline into the permafrost on overland sections 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. However, at changes in terrain conditions such as from initially unfrozen to frozen, or at sudden changes in subsurface ice content, differential thaw settlement could 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.



(a) Thaw settlement



(b) Frost heave

Figure 4-2: Freezing and thaw effects on pipelines in discontinuous permafrost.

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. The cover depth for the section of the pipeline within the municipal boundaries of Norman Wells and at road crossings was 1.0 m. The minimum pipe cover at stream crossing was 1.5 m, with some streams requiring deeper burial for scour protection.

4.3.2 Borehole Database

Over 6000 boreholes were drilled throughout the Mackenzie River valley in the 1970s. Of this data-set, approximately 3500 boreholes were located within 5 km of the pipeline centreline. Information in the borehole data base included location, (kilometre post and offset), terrain type, borehole number, a summary of the soil stratigraphy and available laboratory tests data (water content, visible or pure ice, bulk density). Computer programs were used to assess the thaw strain of different soil layers, and integrate the strain to obtain the settlement occurring between the pipe and the maximum anticipated depth of thaw. As the pipe base was located typically 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 that would thaw could be well 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 three 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, Saunders, Lem and Carlson, 1983).

The original borehole database has been upgraded and enlarged. More information is available from Smith, Burgess, Chartrand and Lawrence (2005).

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 cut line 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 cut line between disturbed and undisturbed ground. In addition, several previous studies including sites in the Fort Simpson area, reported by McRoberts, Law and Moniz (1978) were examined to expand the database for the pipeline route in this area. Figure 4-3 presents data on the effect of surface clearing and disturbance on the thaw depth with time (Hardy Associates (1978) Limited, 1983b). For sites where the surface organic mat is not disturbed, thawing to as much as 4 m could naturally occur over a period of 25 to 35 years. The importance of this data is as follows. If a pipeline is buried in a right-of-way that has experienced previous disturbance then the thaw progression and subsequent resulting thaw settlement that the pipeline will experience will be modified by the pre-existing disturbance. The resulting thaw settlement will be less (sometimes significantly less) than the case where a pipeline is constructed over undisturbed terrain.

Work in the 1970s on various pipeline projects in permafrost terrain of North America examined the relationship between thaw strain and total ice-content of various soils (Speer and Watson, 1972; Watson, Slusarchuk and Rowley,1973; Luscher and Afifi, 1973). Initially the work focussed on correlations between bulk density and thaw strain. However, the technical problems associated with measurement of bulk density of fragile specimens rendered the correlations difficult to develop.

Figure 4-4 presents a plot of the thaw strain of soil as a function of bulk density (Hardy Associates (1978) Limited, 1982a, b). Based on work from the proposed Alaska Highway gas pipeline project, Hanna et al. (1983) established relationships between volumetric water content and thaw strain for several soil types. These relationships were based primarily on thaw strain tests of soil samples gathered from pipeline routes in the Yukon and Mackenzie Valleys regions of Canada

Comparison of predicted and actual thaw settlement data found that the laboratory thaw strain correlations tended to over predict the total thaw settlement that would develop. Figure 4-5 presents a comparison of predicted and observed settlements. That is, the correlations, such as those presented by Hanna et al. (1983) are conservative. One reason for the over prediction of thaw strain is because the plots assume that all water is frozen at the initiation of thawing, whereas most soils have some unfrozen water content, even at relatively cold temperatures. This unfrozen water content is therefore not subject to thawing and thaw strain. Second, laboratory tests were conducted on small samples, typically less than 100 mm in height and the drainage path within the thawing specimen was quite short, with two way drainage being available. In reality, the drainage path could be several metres and only in one direction. Furthermore, 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).

For application to the thaw settlement design of the Norman Wells pipeline, the designers applied a "correction factor" of 0.75 to the thaw strains determined by laboratory tests (Hardy Associates (1978) Limited, 1982b).

4.3.4 Thaw Settlement Design Summary

Following the extensive thaw settlement assessment of the route, the designers specified thaw settlement design values listed in Table 4-1 (Hardy Associates (1978) Limited, 1983a).

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.

Table 4-1: Summary of thaw settlement design values.

Kilometer Post Range	Approximate Locations	Design Thaw Settlement (m)
0 to 78	Norman Wells to Great Bear River	0.80
78 to 440	Great Bear River to Willowlake River	0.75
440 to 868	Willowlake River to Zama	0.70
Thick organic deposits		1.20

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 soil prism 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. The transition length over which the pipeline deforms was assumed to be 15 m (Nixon et al. 1983).

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. The design thaw settlement values are provided in Table 4-1.

The details of the pipe stress analyses 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 4-2, for a 359 MPa (X-52) grade steel pipe.

It was determined that the corresponding permissible differential frost heave was 150 mm to 300 mm (Hardy Associates (1978) Limited, 1982c).

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

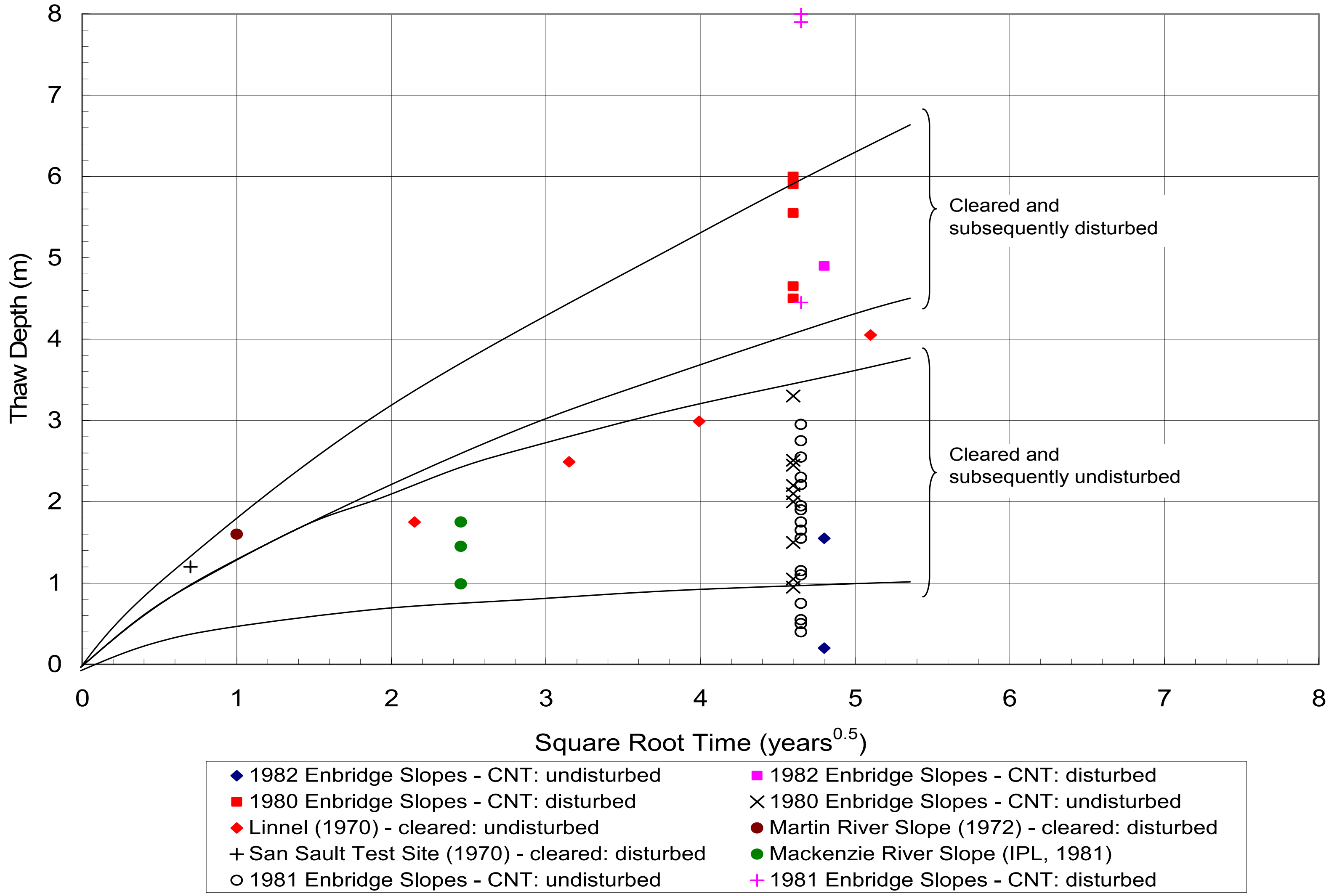


Figure 4-3: Thaw depths in disturbed terrain as a function of the square root of time.
Note: Figure re-plotted from Hardy Associates (1978) Limited, 1983b)

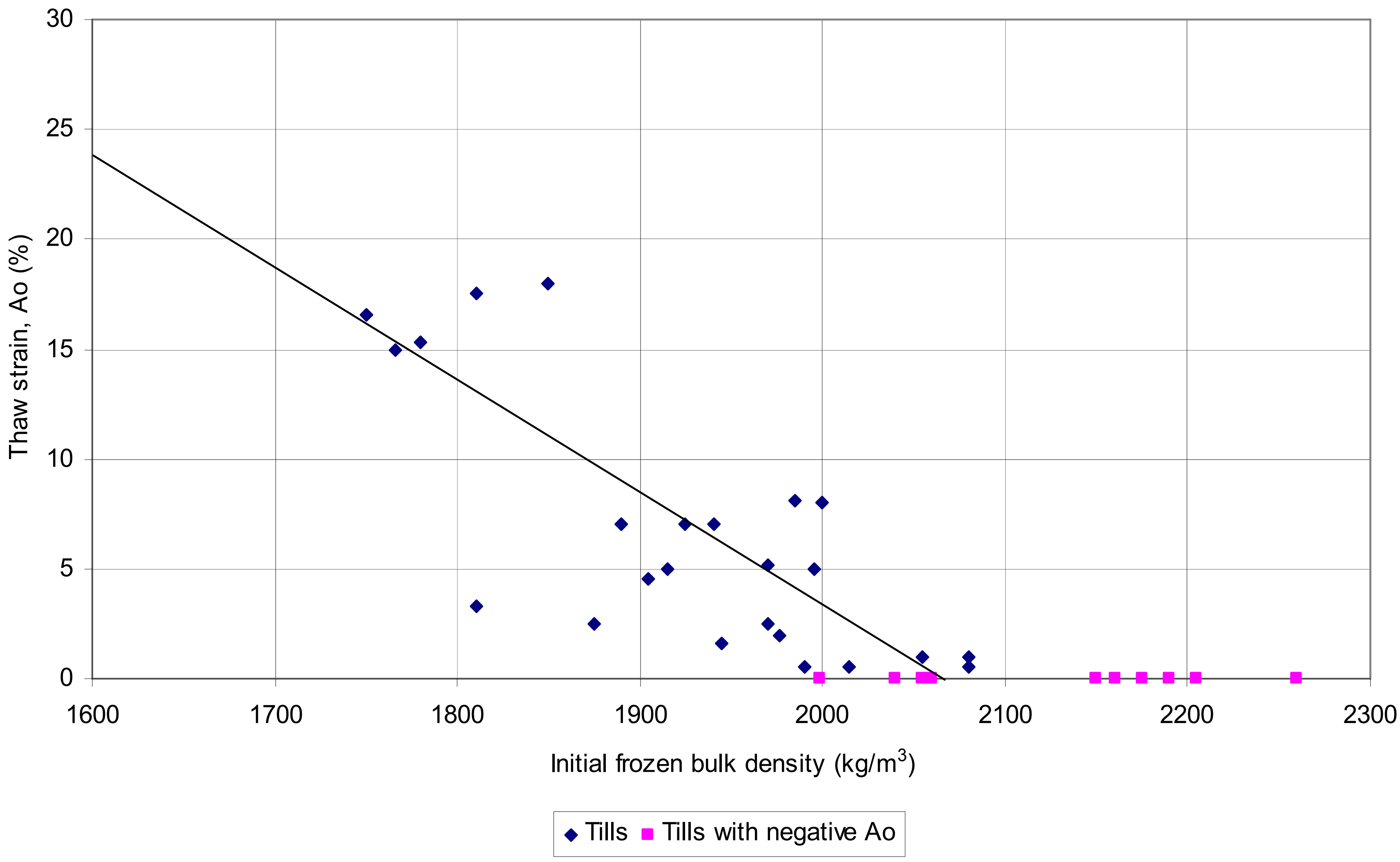


Figure 4-4: Thaw strain versus initial frozen bulk density for tills.
Note: Figure re-plotted from Hardy Associates (1978) Limited, (1982a)

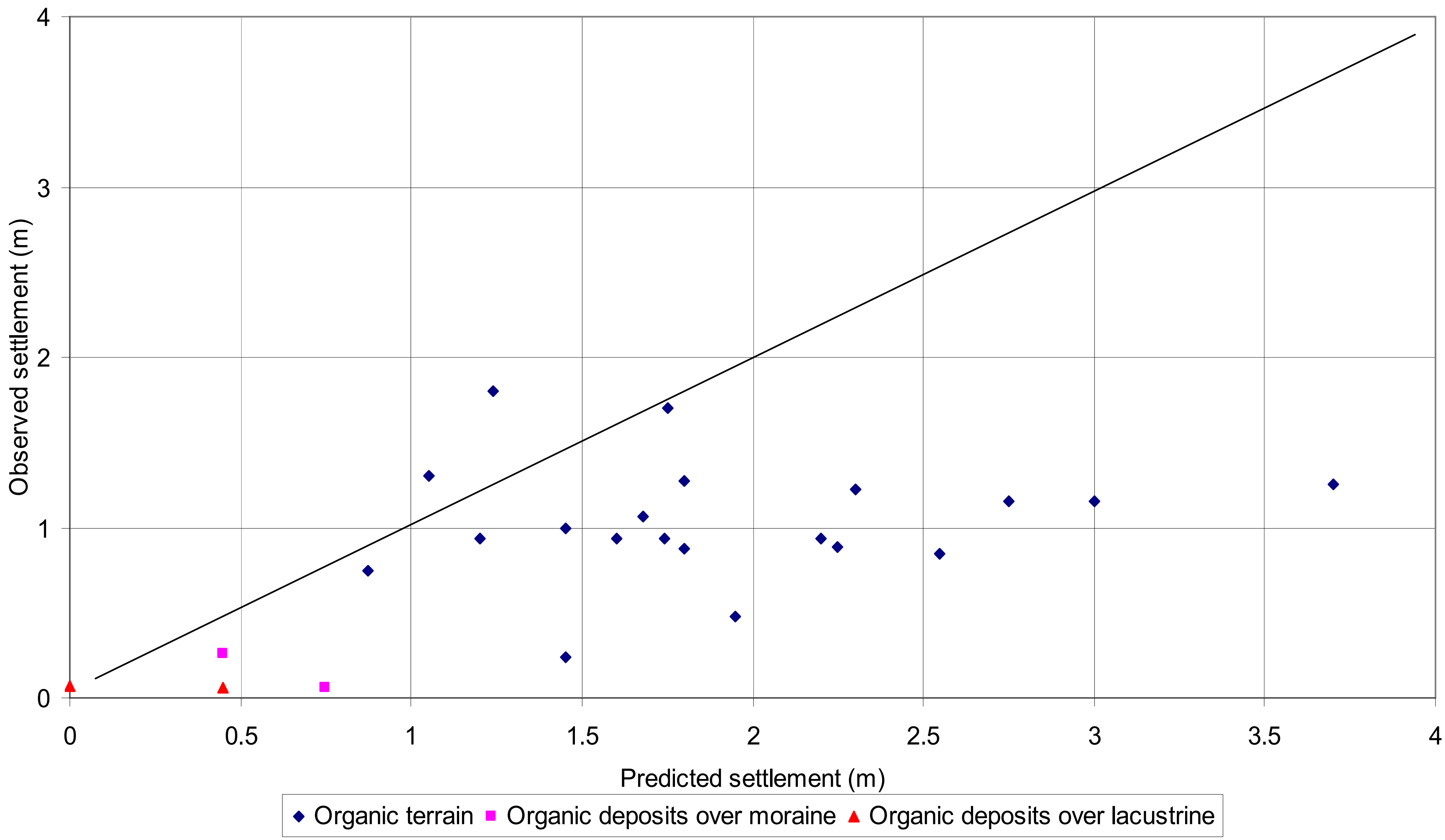


Figure 4-5: Predicted and observed thaw settlements.
Note: Figure re-plotted from Hardy Associates (1978) Limited (1982a)

Table 4-2: Design wall thickness.

Location (km)	Design Thaw Settlement (m)	Design Wall Thickness (mm)
0 to 78	0.8	7.16
78 to 440	0.75	6.91
440 to 868	0.7	6.35
Thick organic deposits	1.2	6.35
River crossings	-	9.54

4.4 Frost Heave

It was not intended to operate the oil pipeline at temperatures significantly below 0°C for extended time periods. However, the possibility existed that the pipe might induce small amounts of frost advance and associated heave beneath it. If the pipe traversed several kilometres 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) exists.

Frost heave along overland sections of the route was investigated during the design phase. Nine frost heave test sites were established between Norman Wells and Wrigley (Hardy Associates (1978) Limited (1982c). These sites were in disturbed terrain where the seasonal active layer was very deep, and hence season re-freezing would occur, potentially giving rise to seasonal frost heave. Most sites were in glacial till soils. Site investigations including geotechnical boreholes, geophysical surveys and installation of instrumentation were conducted. The sites were monitored over at least one winter season. The study concluded that localized frost heave over one winter season would be less than 30 mm.

Sag bends were identified as being particularly susceptible to frost action. The compressive strains initially in the pipe owing 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; however it is understood that little or no data was collected and analyzed.

Geothermal and frost heave analyses 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 100 to 125 mm. For structural modeling of the pipeline, a frost heave transition of 1.5 m was assumed. 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 ground waves that impose strains on a buried pipeline. No known active faults were identified along the route, and generally the impact of seismic aspects on the pipeline design was considered to be 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 (upheaval buckling) 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 4-3 lists the potential failure modes for three geothermal soil conditions.

Table 4-3: Potential slope failure modes.

Slope Failure Mode	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 Effects of Thawing on Permafrost

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. (Nixon and Morgenstern, 1973; Roggensack, 1977)

The second effect of thawing was the pore water pressure response. In certain soil types, excess porewater pressures can be generated arising from thaw consolidation effects (Morgenstern and Nixon, 1971). This increase in pore water pressure could have a destabilizing effect on slopes and was the prime issue in the stability analyses. The normalized pore water pressure is a function of the normalized depth and a coefficient termed the “thaw consolidation ratio”. Equation 4-1 presents the formula:

$$\frac{u(x,t)}{\gamma' d} = \frac{x/d}{\left(1 + \frac{1}{2R^2}\right)} \qquad \textbf{(4-1)}$$

Where u(x,t) is the pore water pressure, being a function of depth (x) and time (t)

γ' is the effective unit weight of the soil

d is the depth to the thaw front

R is the thaw consolidation ratio, defined has $R = \frac{\alpha}{2\sqrt{c_v}}$

α is the thaw rate constant

c_v is the coefficient of soil consolidation

Examination of Equation 4-1 shows that the pore water pressures are proportional to the thaw rate (the faster the thaw rate, the higher the pore water pressures), and inversely proportional to the coefficient of soil consolidation (clays having lower values of coefficient of consolidation will have higher pore water pressures than silts and sands, which have higher values of coefficient of consolidation). As a general rule, values of the thaw consolidation ratio (R) greater than unity can give rise to excess pore water pressures.

The importance of the thaw on any particular permafrost slope is not so much the depth of thaw but rather how much, and at what rate ice is melted and converted to water. The release of water has a significant influence on the stability of the slope. For slopes where the released water cannot rapidly drain away, soil pore water pressures will increase. For thawing that occurs rapidly in an ice-rich soil, the release of water could be sufficient to destabilize the slope. The strategy for the Norman Wells pipeline was to reduce the rate of thawing such that any ice that melted would drain away without generating excess pore water pressures.

Morgenstern and Nixon (1971) showed that the excess pore water pressure during thawing is:

$$u_e = \frac{\gamma_w h_e}{\gamma' d} = \frac{\gamma' d}{(1 + \frac{1}{2R^2})} \qquad \textbf{(4-2)}$$

where, h_e is the excess water head, above the phreatic surface. It can be shown that in the factor of safety equation (Section 4.6.3), the pore water pressure term,

$$\frac{1}{1 + 2R^2} = 1 - \frac{\gamma_w h_e}{\gamma' d} \qquad \textbf{(4-3)}$$

Thus, to determine the factor of safety of a thawing permafrost slope, the ratio of excess water head to thaw depth is necessary. In terms of total height of water above the thaw depth, the pore water pressure ratio, “m,” defined as (Hanna and McRoberts, 1988):

$$m = \frac{h}{d} = 1 + \frac{h_e}{d} \qquad \textbf{(4-4)}$$

where h is the height of water above the thaw depth, d, which is measured from the mineral ground surface (the base of wood chips on insulated slopes). For the case where the groundwater table is coincident with the ground surface, the pore pressure ratio, m = 1. When excess pore water pressures are present, m is greater than unity.

It is seen that if h_e/d is zero, corresponding to no excess pore water pressure then R = 0 and the factor of safety equation effectively reduces to a drained analysis. If h_e/d is 0.5, then the available shearing resistance due to soil weight is only half the value if the groundwater table is at the ground surface.

During the design phase, data was collected to validate the excess pore water pressure predictions, based on the thaw consolidation theory (Morgenstern and Nixon, 1971). As part of the original geotechnical investigations during the design phase, pore water pressure data was collected at several sites with thawing at depths of 1 m to about 5 m. Figure 4-6 presents the field data as the pore water pressure ratio, m, to thaw depth, d. These data indicate that in most cases very low pore water pressures were observed. (This was also confirmed during the drilling programs where groundwater was rarely encountered in the heavy till soils.) A pore water pressure ratio of m = 0.8 was selected for design of unfrozen slopes. Also shown on Figure 4-6 is the assumed behaviour of the pore water pressure as thawing progressed into the slope.

In ice-rich soils, water (ice) contents were such that excess pore water pressures could be generated. With time, as the number of thaw cycles increased, the generation of excess pore water pressures would decrease, and become less of a destabilizing influence. For the design of thawing ice-rich clay slopes a thaw consolidation ratio (R) of 0.47 was assumed for the first thaw season. This corresponds to a h_e/d of about 0.26, and a pore pressure ratio m = 1.26. With time, the pore water pressure would dissipate (McRoberts, Fletcher and Nixon, 1978), as shown Table 4-4.

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

Table 4-4: Predicted dissipation of pore water pressures with time for ice-rich clay.

Year	Percent of first year pore water pressure	R	h_e/d	m
1	100	0.47	0.26	1.26
5	0.55	0.26	0.14	1.14
10	0.30	0.14	0.08	1.08
15	0.15	0.07	0.04	1.04
20	0.05	0.02	0.01	1.01
25	0	0	0	1.00

4.6.3 Factor of Safety Algorithm

In the 1970s, 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 pore water 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 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). Equation 4-2 was used to calculate the static factor of safety, incorporating excess pore water pressures, and side shear developing from a thaw bulb of limited width (Hanna and McRoberts, 1988).

$$FOS = \frac{c'}{\gamma d}\left[\sec\theta \operatorname{cosec}\theta + 2\left(\frac{0.8\ d}{S}\right)\operatorname{cosec}\theta\right] + \frac{\gamma'}{\gamma}\left(\frac{1}{1+2R^2}\right)\frac{\tan\phi'}{\tan\theta}\left(1+\frac{0.8K_o d}{S}\right) \qquad \textbf{(4-5)}$$

Where c' = effective cohesion of the soil
- Φ'= effective friction angle of the soil
- γ = total unit weight of soil
- γ' = effective unit weight of soil
- d = depth of thawing
- θ = slope angle
- S = thaw bulb width
- R = porewater pressure coefficient
- K_o = earth pressure coefficient
- 0.8 = thaw bulb shape factor

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 depths 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.

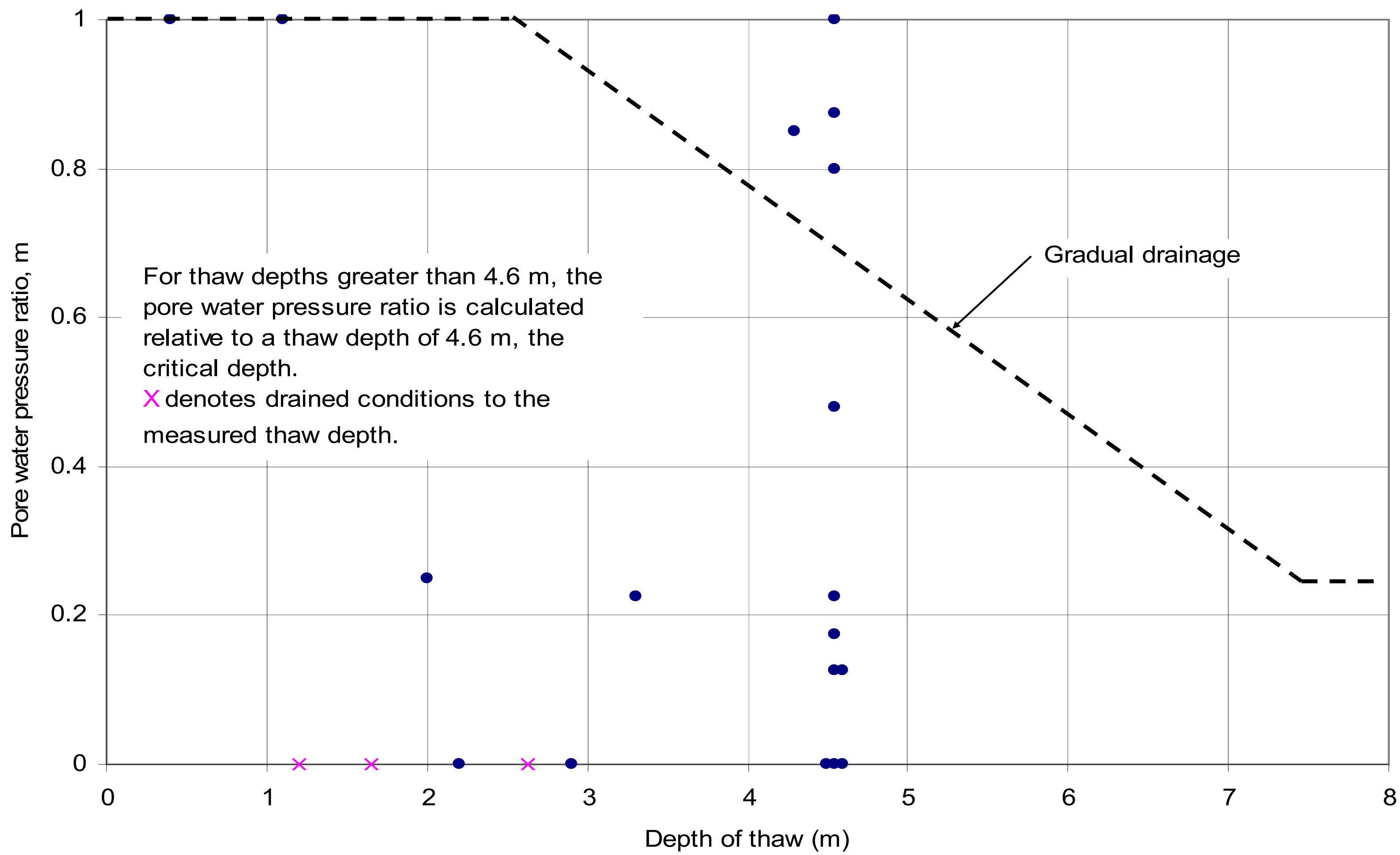


Figure 4-6: Observed pore water pressure ratio as a function of thaw depth for thawing till slopes.
Note: Figure replotted from (Hardy Associates (1978) Limited (1983b).

4.6.4 Target 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 containing ice-rich sediments, for static loading conditions, that is, not involving earthquake loadings, was 1.5. For ice-poor soils, the static factor of safety was 1.3 (Hardy Associates (1978) Limited, 1983a).

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 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, or the rate of thawing 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. Compared to rigid board insulation, wood chips were also expected to be more flexible and yielding as thaw settlement occurred. 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 4-5 lists the design guidelines for cut-off angles for slopes, and backfill materials, based on the soil type and slope surface.

Table 4-5: 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 (Table 4-6) summarizes the mitigation measures carried out on the slopes for the entire length of the pipeline.

Table 4-6: Summary of mitigation measures for slopes (Number of slopes (percentage of total)).

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

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. See Figure 4-3. Typical effects 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 associated with surveying activities and geotechnical investigations. 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)
- 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 4-7 provides the distance between berms for a range of slope grades.

Table 4-7: Spacing of diversion berms on slopes.

Slope Gradient (%)	Slope Angle (°)	Distance Between Diversion Berms (m)
< 5	3	100 - 500
5 -10	3 – 5.7	50
10 - 15	5.7 – 8.5	25
15 - 20	8.5 – 11.3	17
20 - 25	11.3 - 14	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 lateral migration of the banks was assessed and the “sag-points” were selected. A minimum of 1.5 m cover beneath the thalweg was specified for all design stream crossings. Deeper burial was specified for some crossing for scour protection.

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 rivers 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.

5.0 PIPELINE AND RIGHT-OF-WAY PERFORMANCE (ACTUAL IMPACT)

5.1 Pipe/Ground Thermal

5.1.1 Pipeline Temperatures

Figure 5-1 presents the pipeline inlet temperature regime in Norman Wells as mandated by the National Energy Board (NEB). The pipeline inlet temperatures were fixed at -2°C from the initial flow (April 1985) through 1993. The delivery of crude oil to the pipeline was hampered in the first number of years by problems with the oil chillers. The producer, Imperial Oil Resources Canada Limited (IOL), was required to make modifications to the chilling equipment to improve efficiencies. Waxing of equipment was a particular problem. To address this problem, IOL and Enbridge requested from the NEB a revision to the pipeline inlet temperature regime that would permit warmer oil to be pumped in the summer months while balancing the mean annual inlet temperature by pumping colder oil in the winter. The first “excursion” took place in 1993, with the revised temperature regime shown on Figure 5-1. This new temperature regime was later modified to lower the maximum summertime temperature, and broaden the shoulders of the temperature profile. The net result of the regime was to maintain a mean annual temperature of about -1°C. Numerical modeling of the temperature excursions and the resulting pipe and ground temperatures are discussed later in this section.

Observed pipe temperatures have generally fallen within the range used during design for predictive purposes, as shown for the first few years of operation on Figure 5-2, 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 geothermal conditions during operation. Nevertheless, it is observed that prior to the start of oil flow in April 1985, the pipeline temperature ranges exceeded the design temperature profiles for thaw settlement and frost heave, but the operational pipeline temperatures were generally within the design limits.

For the period of 1993 through 2006 pipeline temperatures for the first 79 km (from Norman Wells to near Tulita, on the left bank of the Great Bear River) are presented on Figure 5-3. The air temperature from the Norman Wells airport is also shown. The pipe temperatures downstream of Norman Wells are measured using a set of four or five thermistor beads attached to the pipeline. Examination of the data shows that the pipeline temperature at Great Bear River south (Slope 29B at KP 79.3) are often warmer than locations at Prohibition Creek (KP 32.5). This suggests that the impact of the Norman Wells pipeline inlet temperature is limited in extent to somewhere between 32.5 km and 79.3 km.

The range of pipeline temperatures, both seasonally and with distance between 1994 and 2004 are presented in Figure 5-4 and Figure 5-5a through Figure 5-5d.

Figure 5-4 presents the range and average annual pipeline temperatures for sites from Norman Wells to KP 355 (near River Between Two Mountains). The figure shows that the average pipeline temperature increases with distance from Norman Wells, reflecting a general warming of the ground as the pipeline traverses southward. Figure 5-5a, Figure 5-5b, Figure 5-5c and Figure 5-5d shows the pipe temperature ranges by season. The differentiation by season was made primarily to reflect the current pipeline temperature regime (Figure 5-1) where four distinct periods are evident: winter period of -4°C oil flow, spring shoulder period of increasing oil temperatures, summer period of +8°C oil flow, and autumn period of decreasing oil temperatures.

5.1.2 Detailed Pipe Temperature Simulator

Traditional 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. The original pipeline temperature profile with distance calculations needed for design for the Norman Wells pipeline required two values for soil thermal conductivity (frozen and unfrozen) 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 geothermal or environmental parameters were estimated and provided to the designer. A computer based numerical model was then 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 method, however, ignored the transient effects of previous temperature excursions imposed by the pipe on the surrounding terrain. In particular, in northern pipelines the pipe could cause freezing or thawing around the pipe at different times of the year, and the simplified exponential solution (see Equation 1; Nixon and MacInnes, 1996) cannot account for the heat lost or gained during freezing or thawing cycles. Furthermore, there can be considerable uncertainty in selecting which characteristic ground temperature, T_g, to use in the analysis. The ground temperature would typically vary throughout the year. In fact, it is now known (Nixon and MacInnes, 1996) that the pattern of heat flow between the pipe and surrounding ground can be much more complex, even for uniform soil conditions, particularly during periods of freezing or thawing.

Liquids 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. In contrast, gas pipelines are even more complex in light of the Joule-Thomson effect.

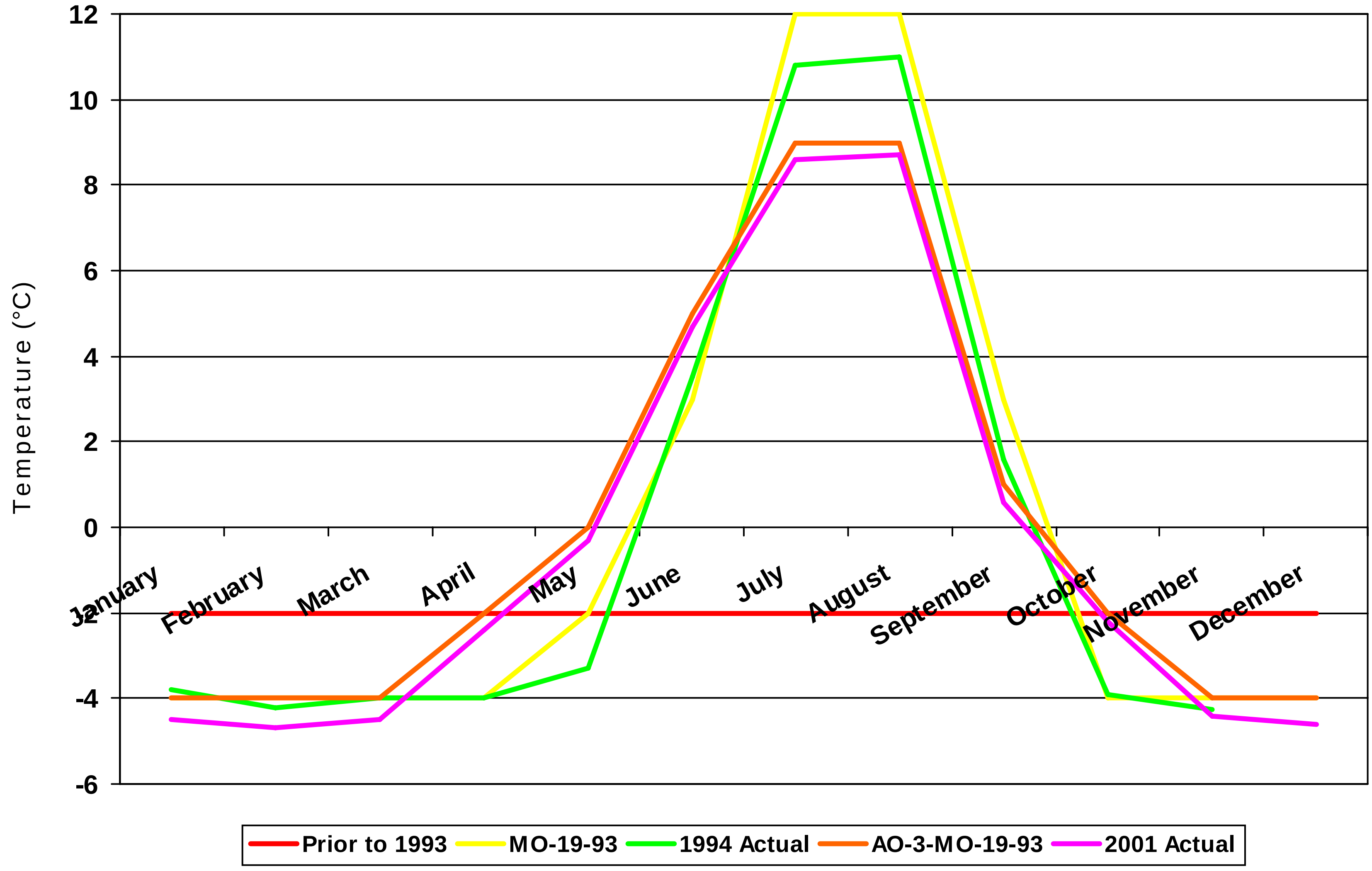


Figure 5-1: Approved and actual pipeline inlet temperatures at Norman Wells pump station.

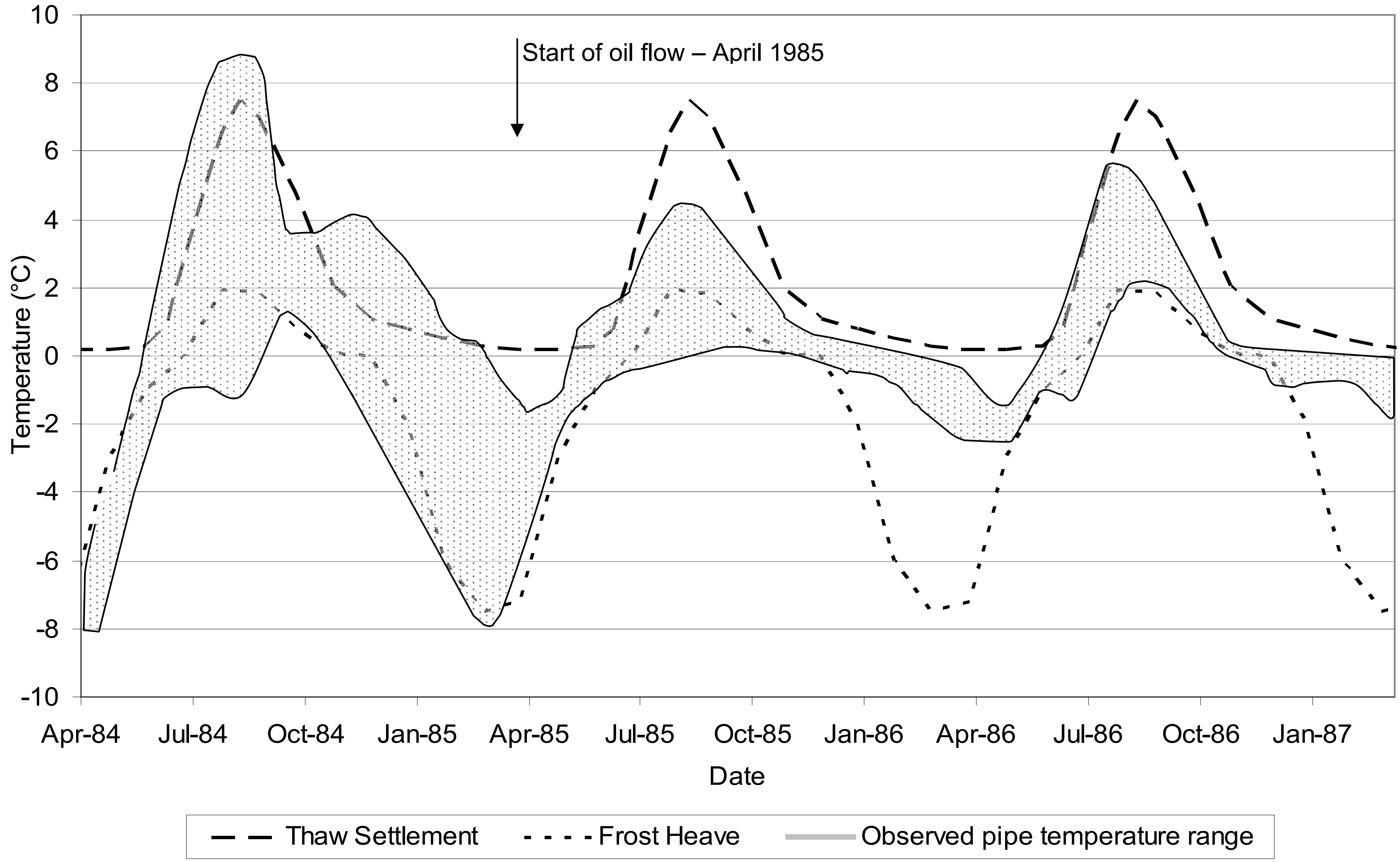


Figure 5-2: Observed range of pipe temperatures at monitoring sites from KP 19 to KP 272, and thaw settlement and frost heave design temperature ranges (from Figure 4-1).
Note: Figure re-plotted from MacInnes, Burgess Harry and Baker, 1990)

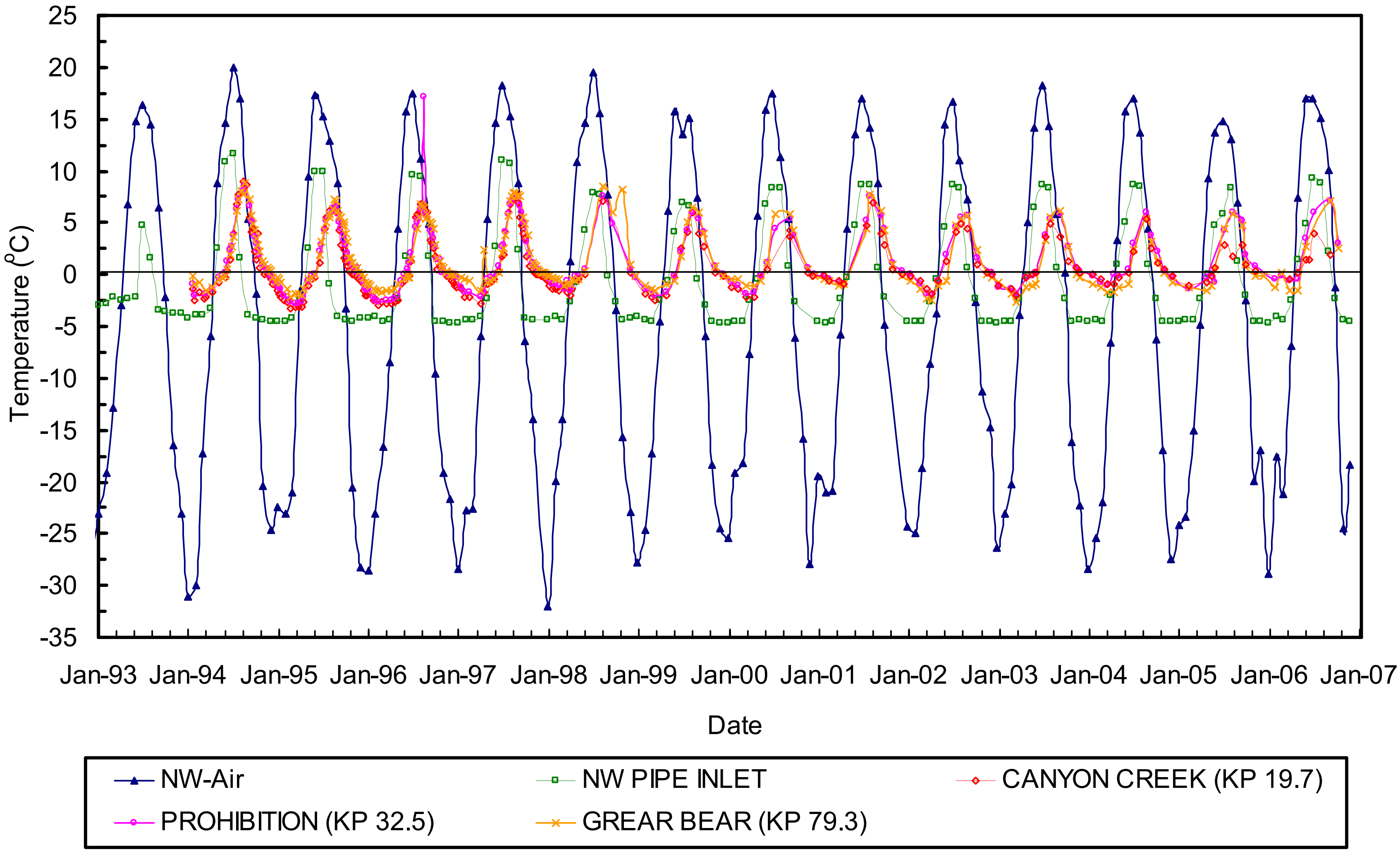


Figure 5-3: Pipeline and Norman Wells air temperatures for the period 1993 to 2006.

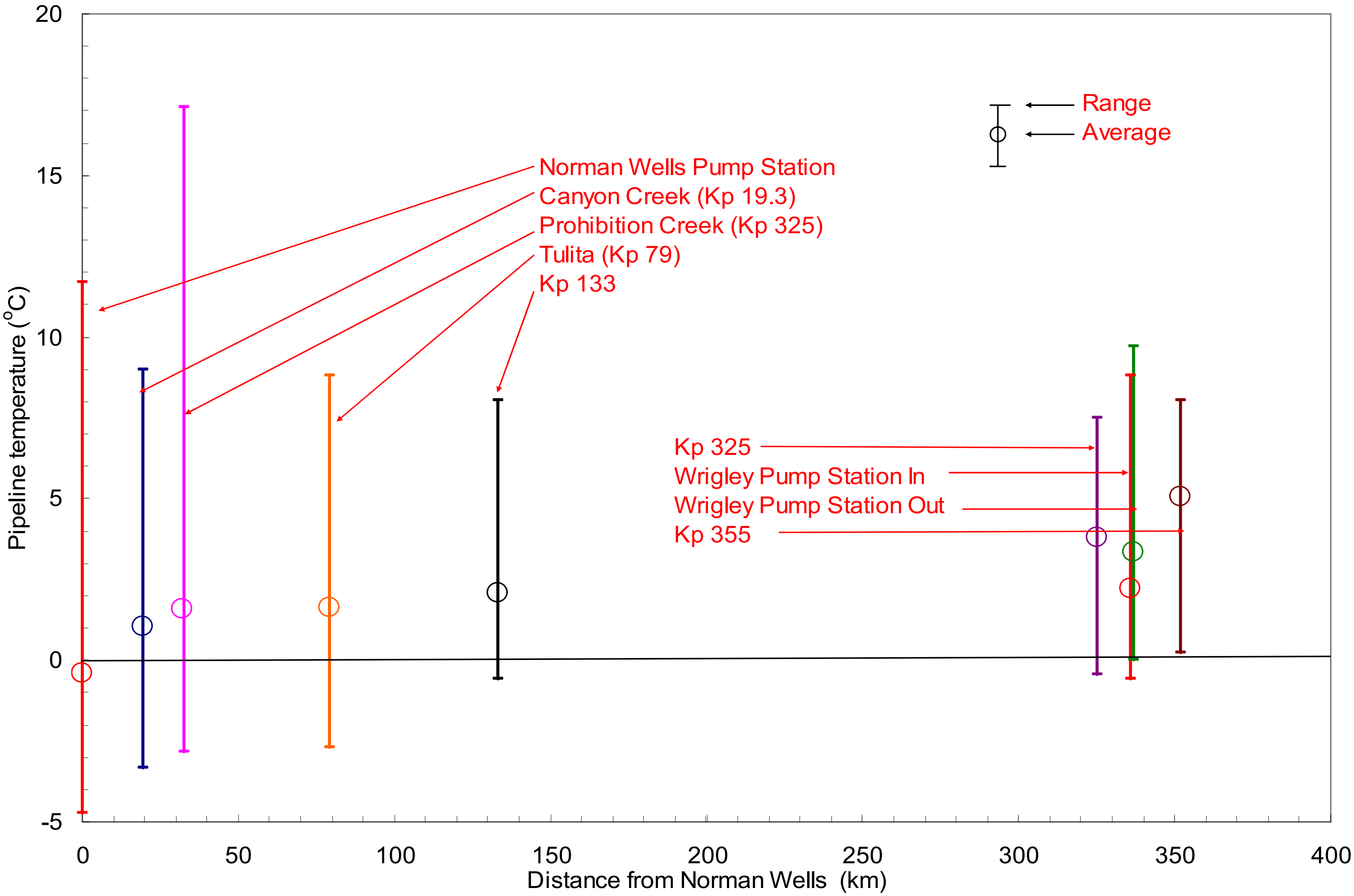


Figure 5-4: Range and average of maximum annual pipeline temperatures (1994 - 2006).
Note: the temperature record is not continuous for all locations.

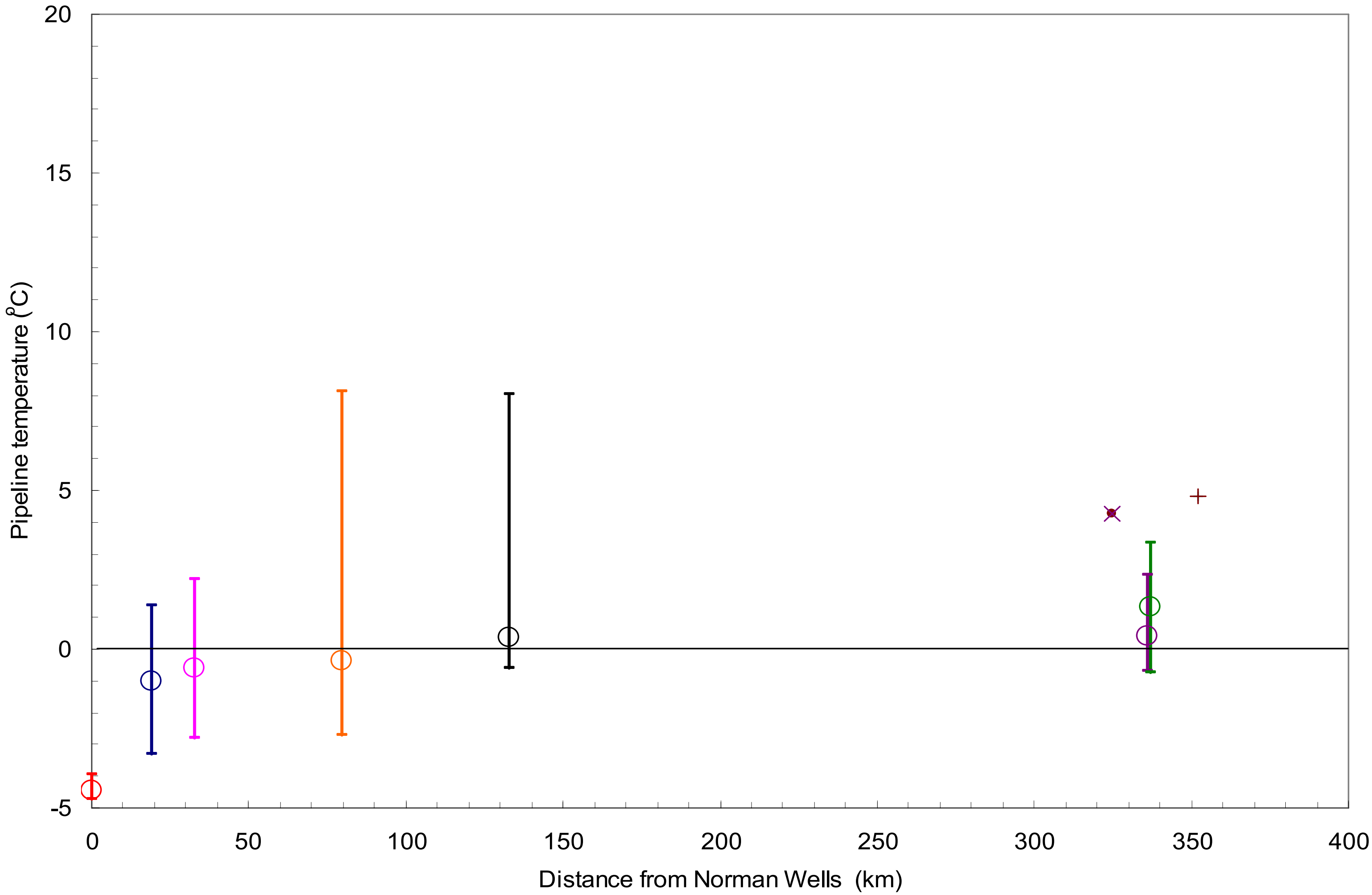


Figure 5-5a: Range and average of maximum seasonal (November to March) pipeline temperatures (1994 - 2006).
Note: the temperature record is not continuous for all locations. Pipeline temperature sites are defined on Figure 5-4.

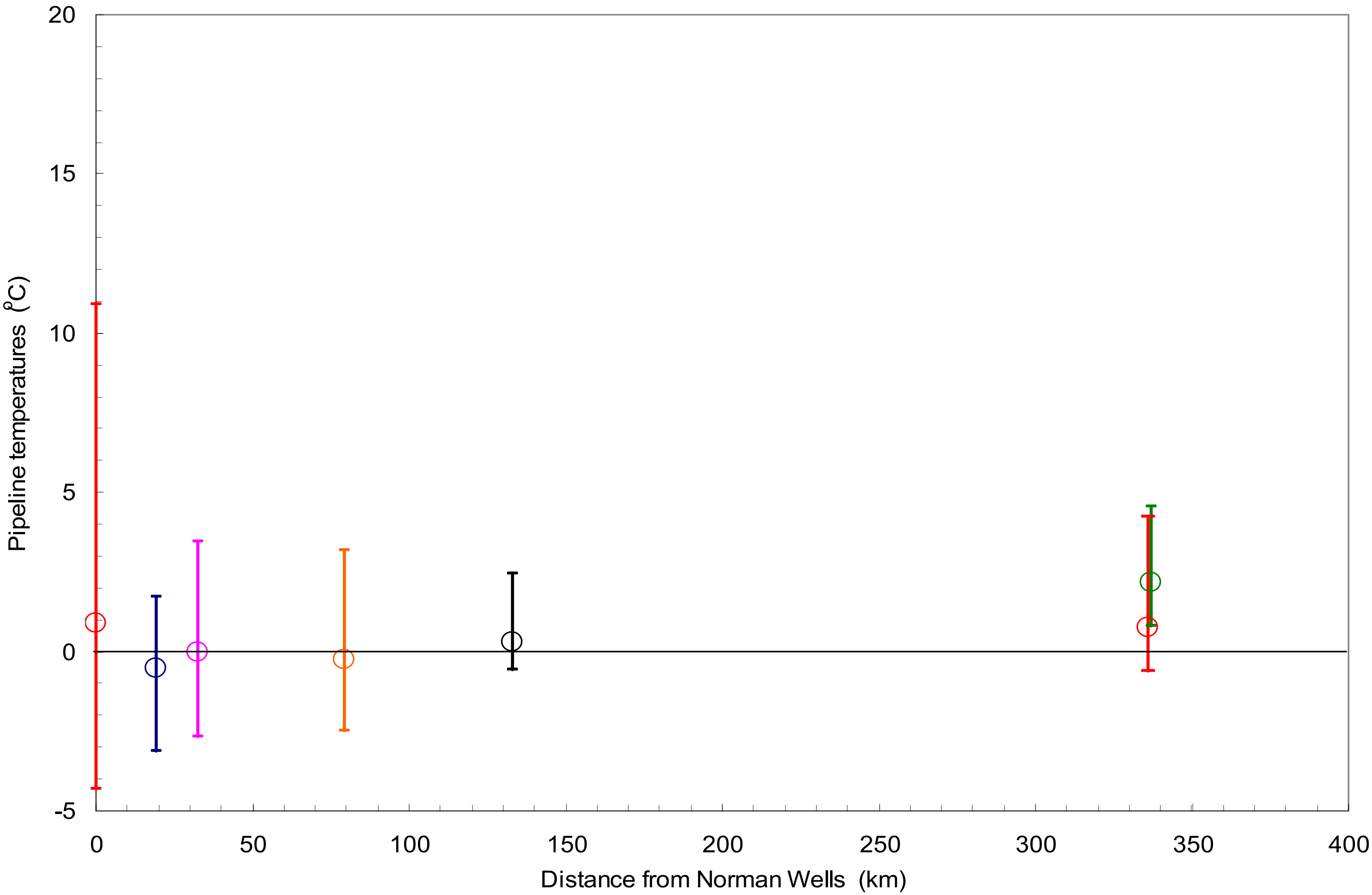


Figure 5-5b: Range and average of maximum seasonal (April to June) pipeline temperatures (1994 - 2006).
Note: the temperature record is not continuous for all locations. Pipeline temperature sites are defined on Figure 5-4.

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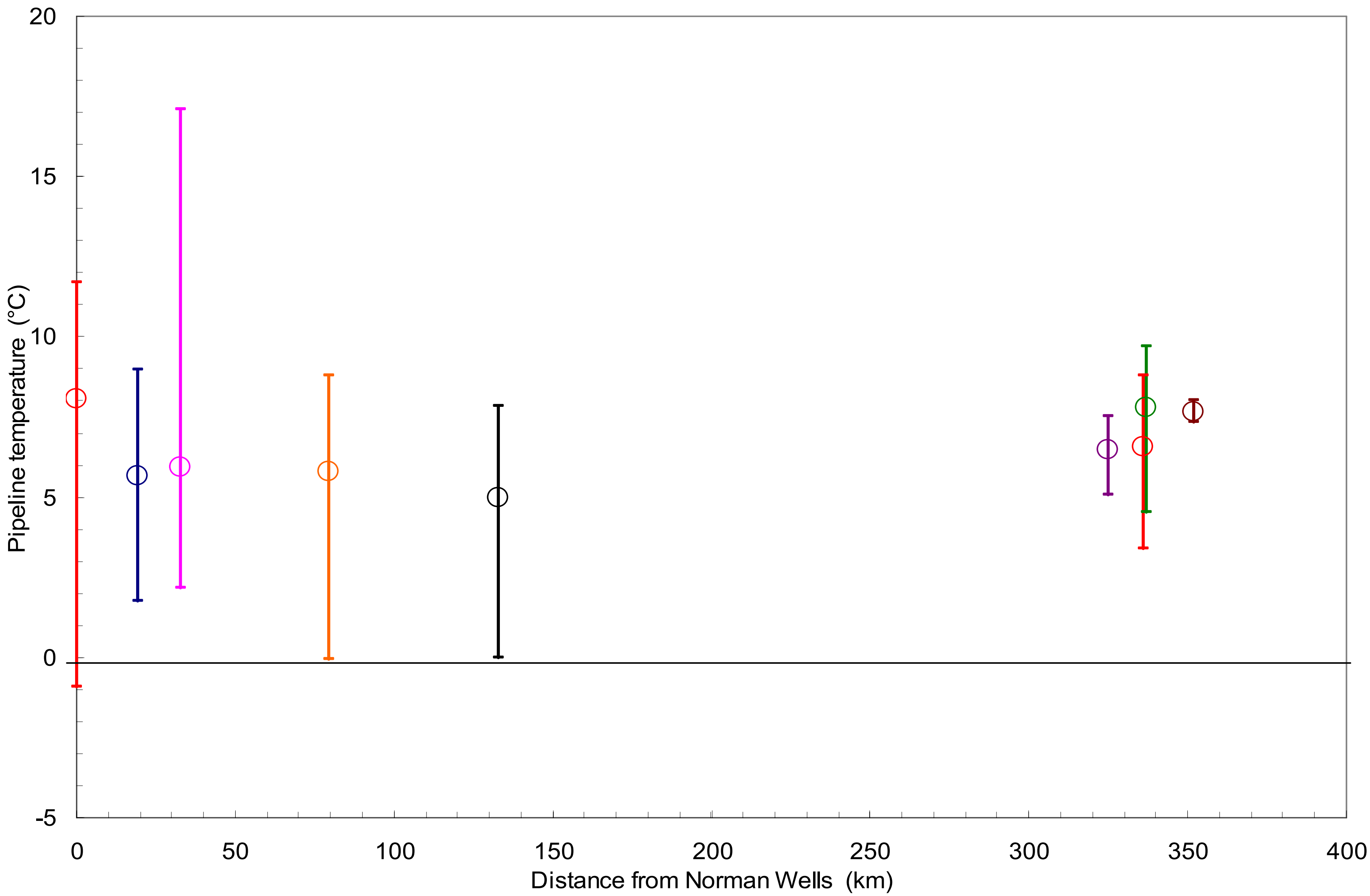

Figure 5-5c: Range and average of maximum seasonal (July and August) pipeline temperatures (1994 - 2006).
Note: the temperature record is not continuous for all locations. Pipeline temperature sites are defined on Figure 5-4.

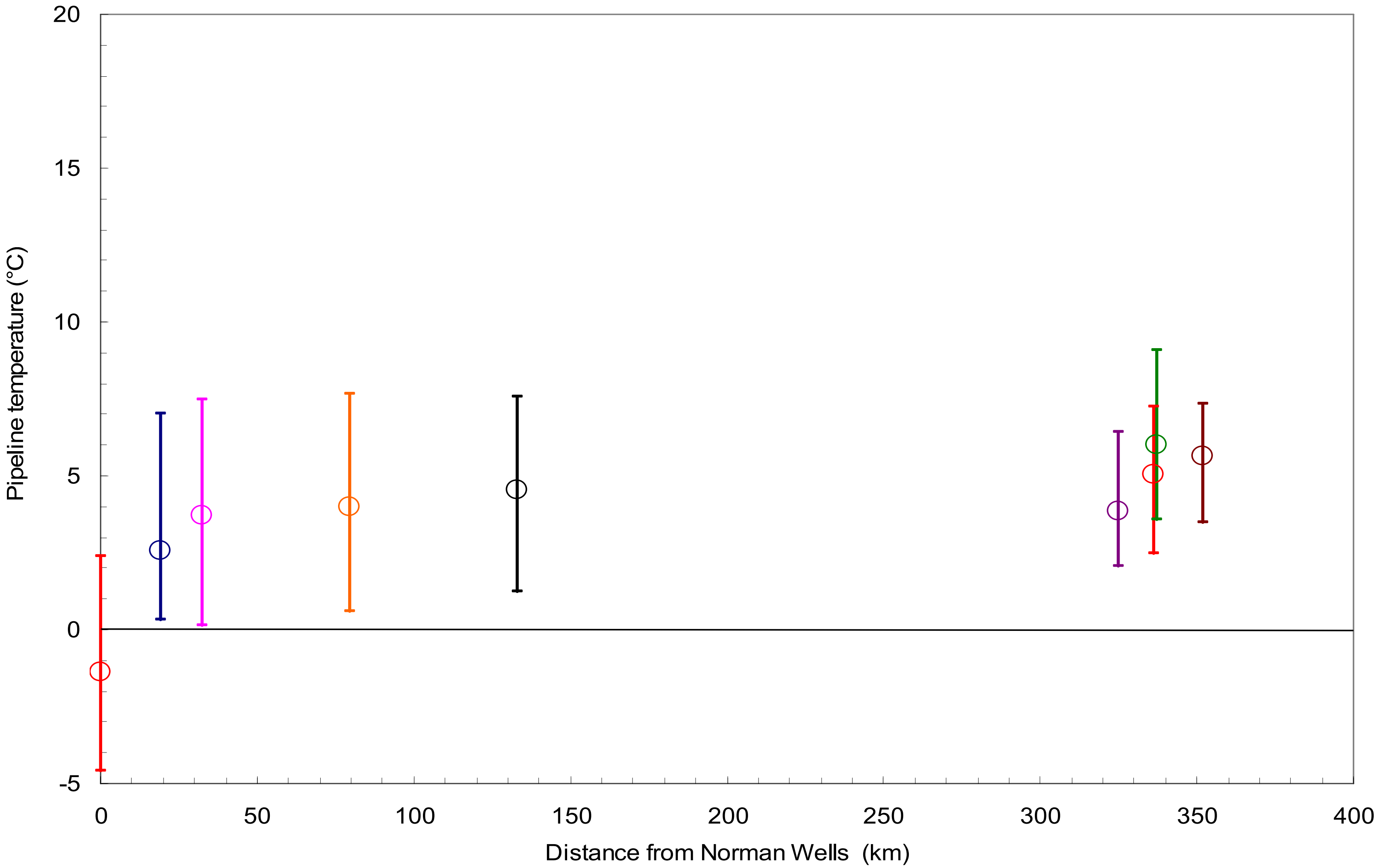

Figure 5-5d: Range and average of maximum seasonal (September and October) pipeline temperatures (1994 - 2006).
Note: the temperature record is not continuous for all locations. Pipeline temperature sites are defined on Figure 5-4.

The simplified analysis had 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. However, advances in numerical modeling and the speed of computing have now allowed more complicated problems to be addressed in greater detail.

The original incentive to develop a more rigorous pipeline temperature model arose from a study of pipeline temperatures along the Norman Wells pipeline. The temperature regime along this pipeline had been studied for many years, and an extensive database of ground and pipeline temperatures was available along its 868 km length (MacInnes et al., 1990; Burgess, 1992). Originally, the oil was input to the pipe at Norman Wells at a near constant temperature of around -2°C (Figure 5-1). In particular, concerns relating to the stability of a few slopes have received attention in the early 1990's (Hanna, Oswell, McRoberts, Smith and Fridel, 1994), and a fuller understanding of the temperature profiles along the pipeline was considered desirable. In 1993, the oil producer and the pipeline operating company requested and obtained permission to operate the pipeline at warmer temperatures at Norman Wells during the summer season. This provided further incentive to more accurately predict and understand the downstream effects of warmer pipe operation on a seasonal basis.

A more rigorous analysis was undertaken and reported by Nixon and MacInnes (1996). A schematic illustration of the computational process is given on Figure 5-6. The pipe temperature model was validated using actual ground and pipeline temperature 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 Figure 5-7. This improved 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.

5.1.3 <u>Right-Of-Way Temperatures</u>

5.1.3.1 Air 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 examine if some of these effects might be attributed to climatic effects. Figure 5-8 shows mean annual air temperatures at Norman Wells and Fort Simpson from the 1940s through 2006. Both annual data and a 5-year running average are plotted. The running average is useful to dampen some of the scatter, but not to mask medium term effects. After an apparent cooling period through the 1940s and

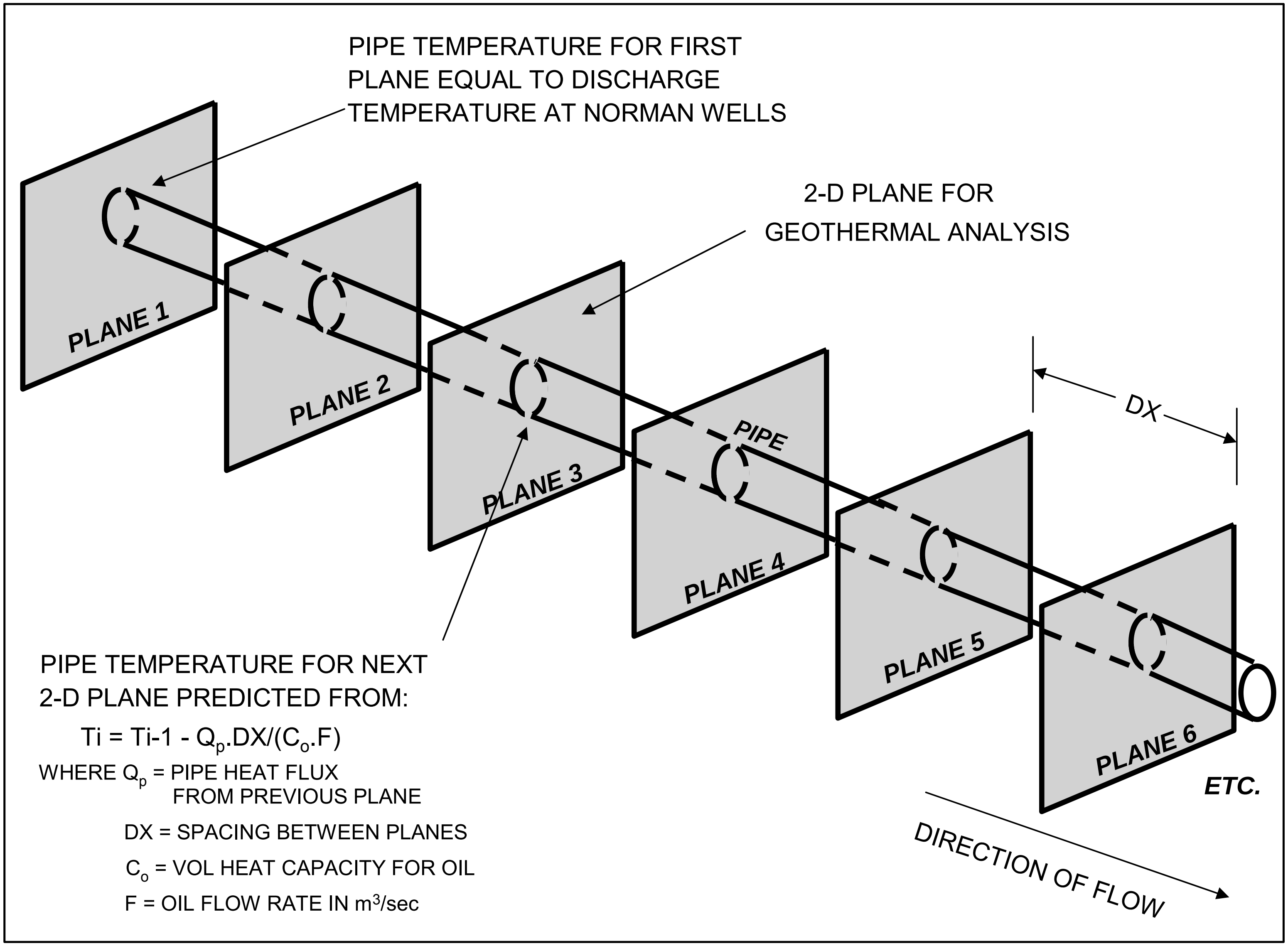

Figure 5-6: Simulation schematic for pipe temperatures.

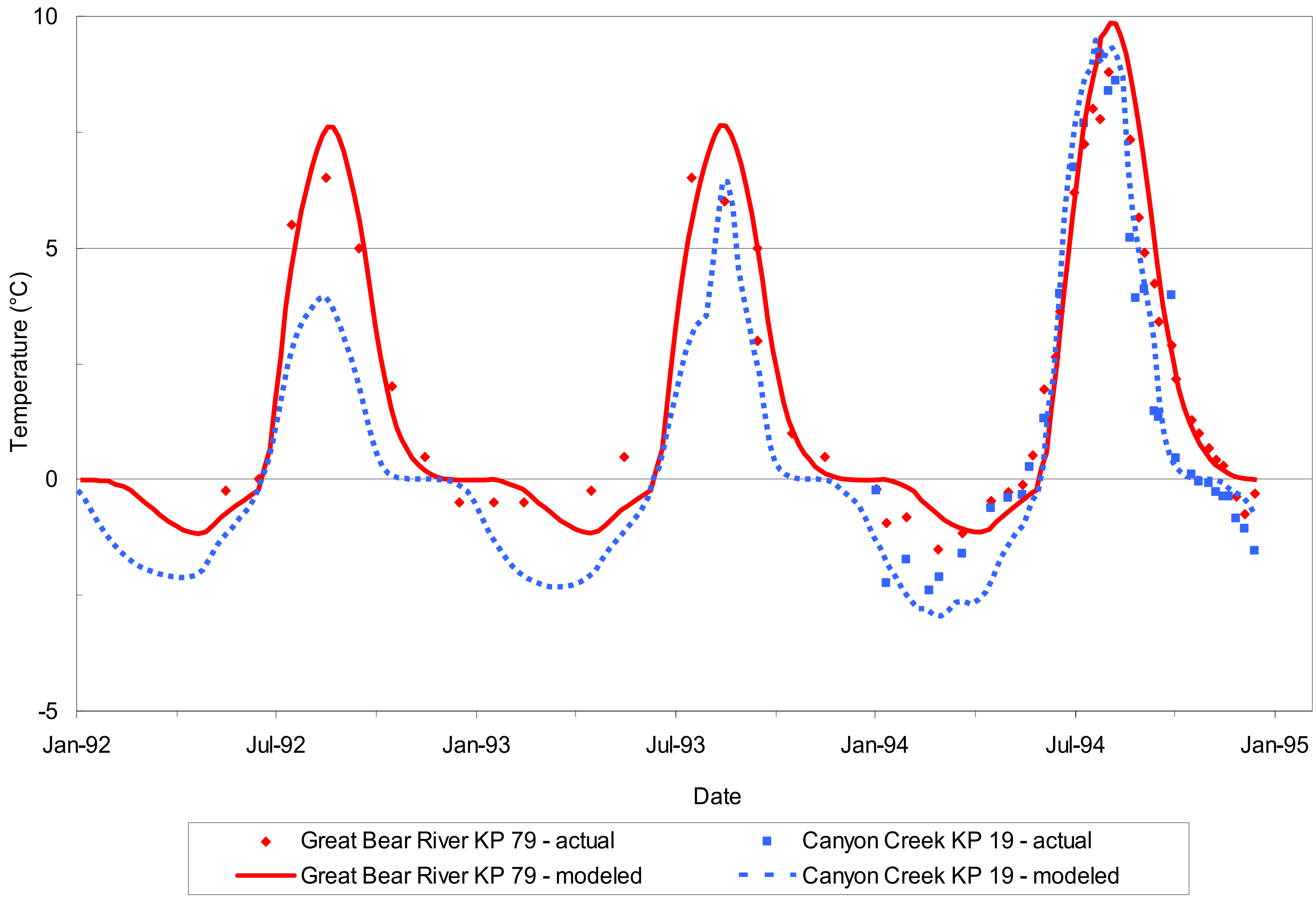

Figure 5-7: Comparison of actual and predicted (modeled) pipe temperatures at two locations: KP 19 and KP 79.

1950s and stable temperatures during the 1960s, the mean annual air temperatures began warming in the 1970s. The early 1980s were certainly warmer than average by nearly 2°C compared to the 1960s. This was followed by a cooling period to the late 1980s and another warming trend in the 1990s. In the larger view, this suggests a warming trend of nearly 2.0°C in the past 30 years might be interpreted. The five year running average between 1970 and 2006 increased by 1.74 °C and 1.85 °C for Norman Wells and Fort Simpson, respectively. 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 to 25 year period, the typical design lifetime of a pipeline.

5.1.3.2 Pipe and Ground Temperature

Figure 5-9a through Figure 5-9d show the running mean pipe temperatures with time for four selected sites (KP 79.4, 272.3, 557.8, and 819.5) along the pipeline for the first approximately seven years of operation. The sites show a general warming trend from 1985 to about 1990, and levelling 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 in light of a lack of warming off the right-of-way in the years prior to construction. Also shown on the same plots are ground temperatures, measured at pipeline depth, on and off the cleared right-of-way. In all cases, except the most southern site, near-surface (1 m depth) ground temperatures on the right-of-way increased in a general trend similar to the measured pipeline temperatures. Off right-of-way ground temperatures were more variable between sites. In two cases, the ground temperatures remained quite stable during the first seven years, while two sites experienced a warming trend of 1 to 2 °C.

The following conclusions are 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 1990s.
- The more rigorous pipe temperature simulator provides results that are in close agreement with those observed at downstream locations.
- The effects of a short term (seasonal) temperature excursion (such as that applied in 1993 and thereafter) 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.

Smith, Burgess, Riseborough and Nixon (2005) examined near surface ground temperatures at several undisturbed sites along the Mackenzie Valley from 1984 through 2003. Using ground temperatures from about 10 m depth, the authors found that the generalized warming trend at depth at Canyon Creek (KP 19) is 0.03 °C/year over the study period. Further south much lower warming rates were noted.

Further implications of right-of-way temperatures are discussed in Section 5.3, “Active Layer Changes and Thaw Settlement”.

5.1.4 Ditchwall Logging for Ice, Permafrost and Soils

During the construction of the pipeline, a continuous ditchwall log was created during ditching for pipeline burial. The ditch was typically 1.2 m deep. The side wall was logged by experienced geotechnical field personnel every 50-100 m, depending on changing conditions. The ditch was generally logged from 1 to 12 hours after the passage of the ditcher. 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, Saunders and Smith (1991) examined 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 Geological Survey of Canada open files (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, knowledge of the percentage of frozen ground is important when deciding whether to operate a gas or oil pipeline above or below freezing.

The number of frozen-unfrozen interfaces has 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 percent in the north to a low of around 16 percent at the south end of the study area, as might be expected. The distribution is shown on Figure 5-10.

The amount of permafrost as evidenced by the ditchwall records appears to be between 80 and 90 percent 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 percent from KP 250-300, whereas the boreholes and geophysics indicate 60 to 70 percent 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 to 18 percent around KP 650. This corresponds to an area of low-lying and wet terrain. The route then rises over the Alberta Plateau, with an associated increase in permafrost distribution up to 40 to 50 percent, before falling to around 22 percent 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 were identified as being 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.

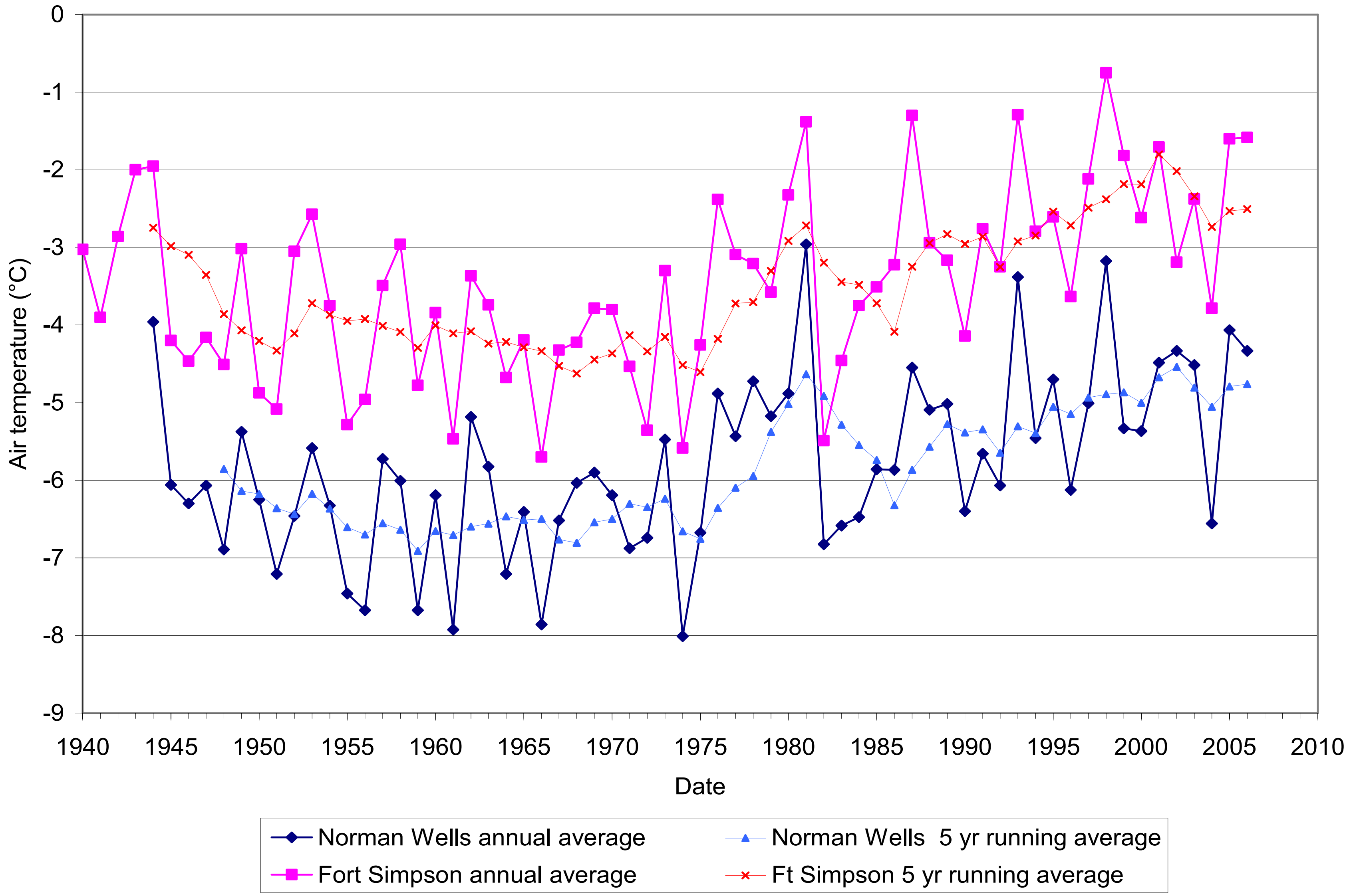

Figure 5-8: Mean annual air temperature and 5 year running average air temperature for Norman Wells and Fort Simpson.

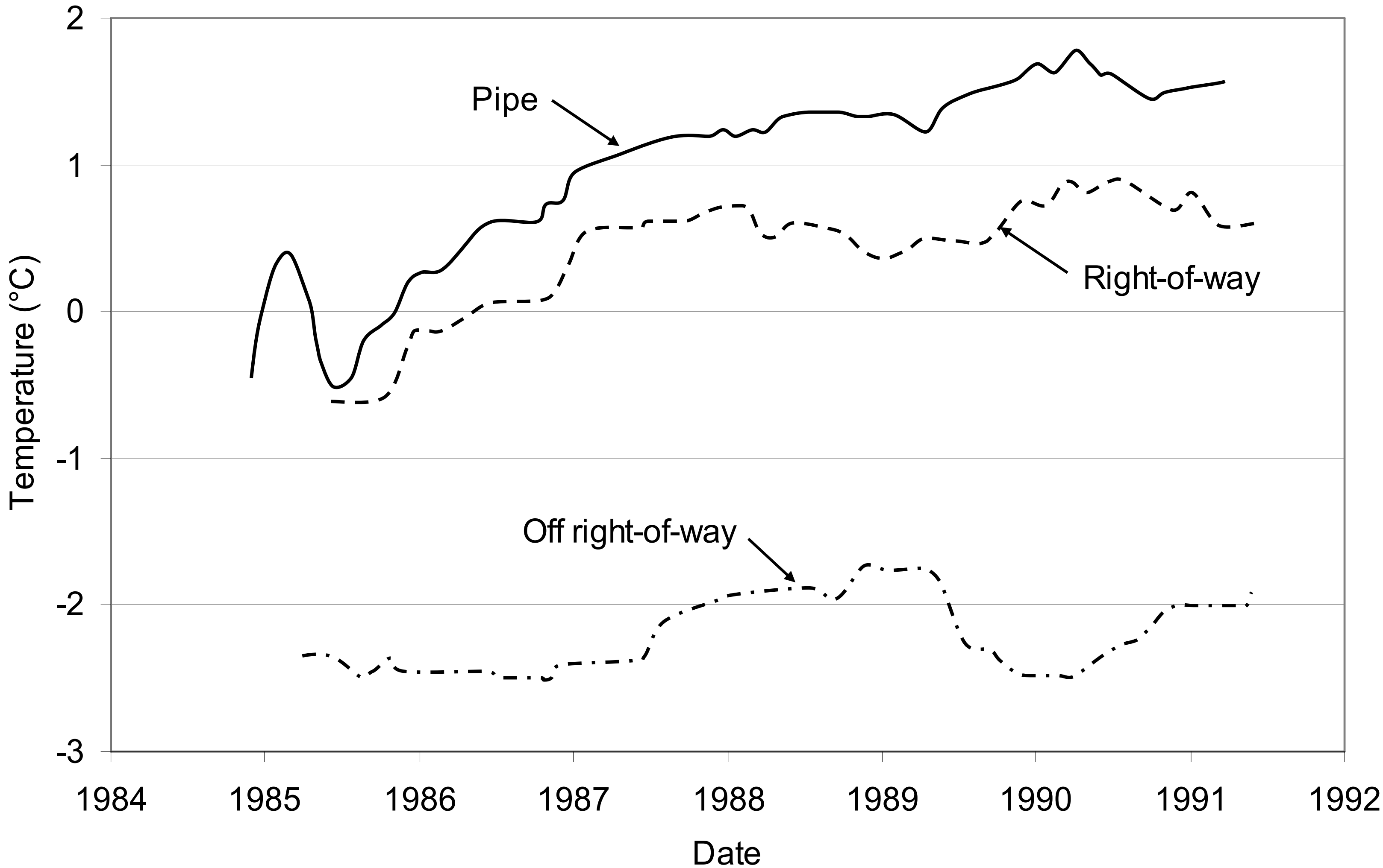

Figure 5-9a: Running mean pipe and right-of-way temperatures – NRCan Site 84-3B, KP 79.4 – Great Bear River south.
Note: Ground temperatures are measured at pipeline depth (1 m depth). Figure re-plotted from Burgess (1992).

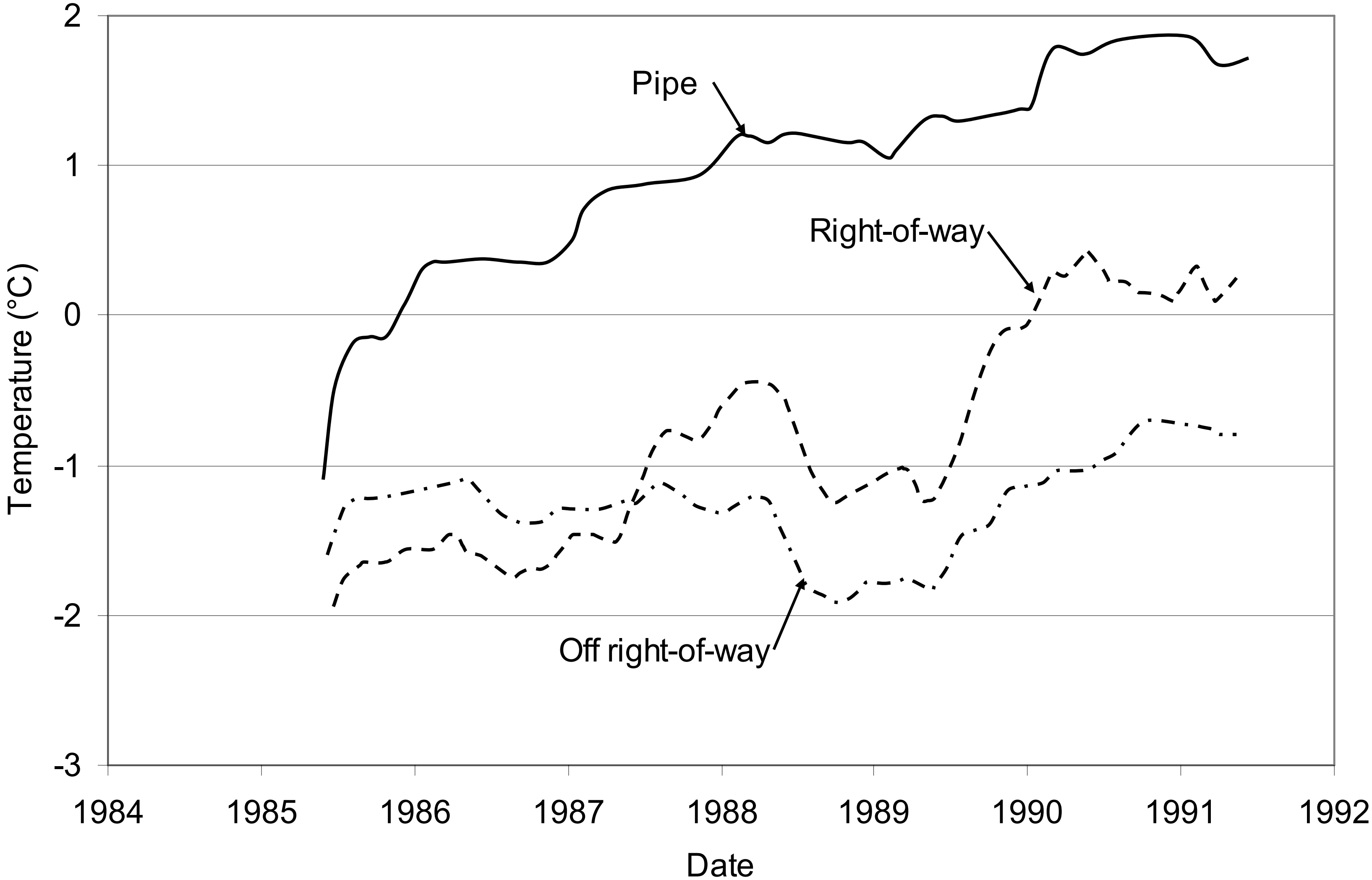

Figure 5-9b: Running mean pipe and right-of-way temperatures – NRCan Site 85-7C, KP 272.3 – Table Mountain.
Note: Ground temperatures are measured at pipeline depth (1 m depth). Figure re-plotted from Burgess (1992).

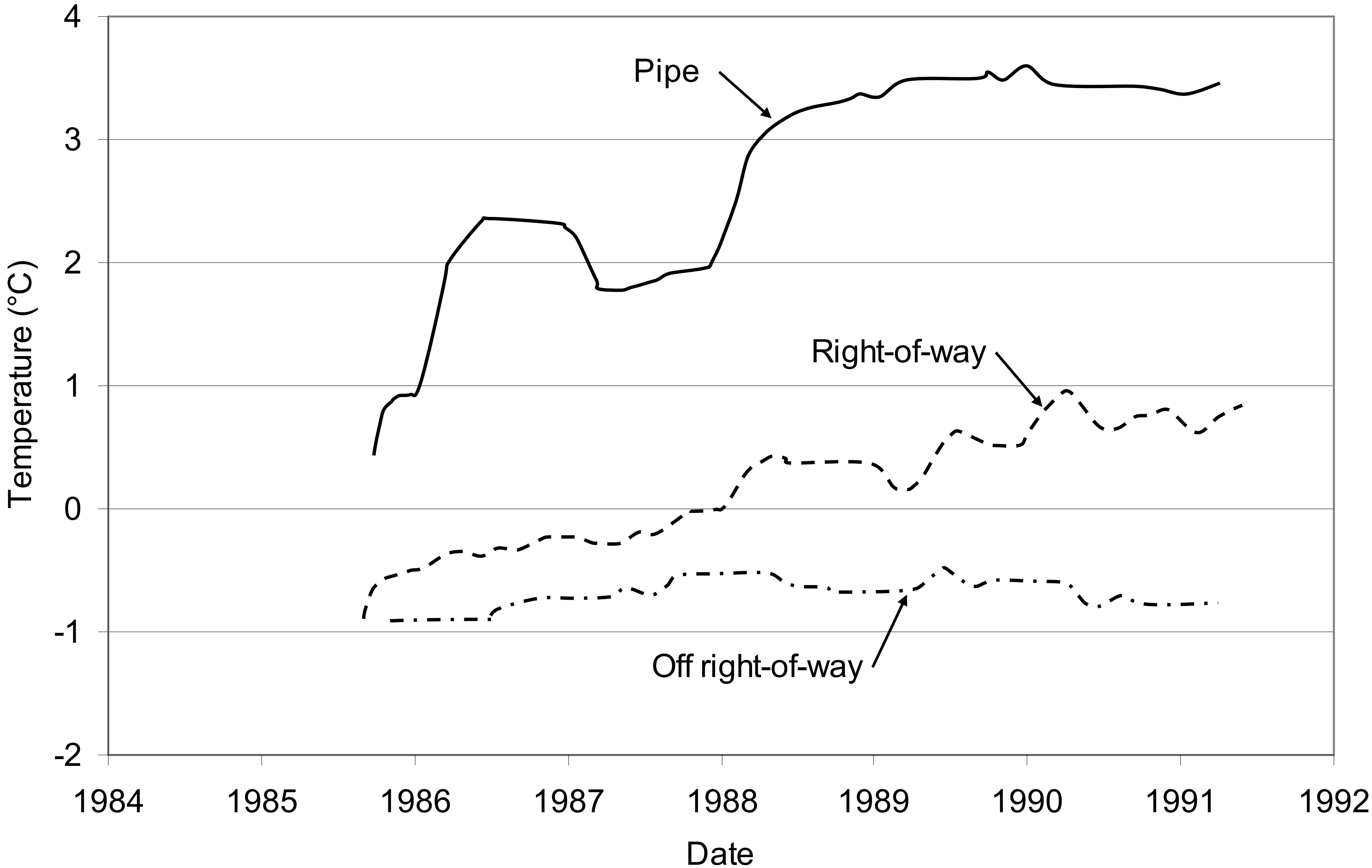

Figure 5-9c: Running mean pipe and right-of-way temperatures – NRCan Site 85-8A, KP 558.3 – Manner's Creek.
Note: Ground temperatures are measured at pipeline depth (1 m depth). Figure re-plotted from Burgess (1992).

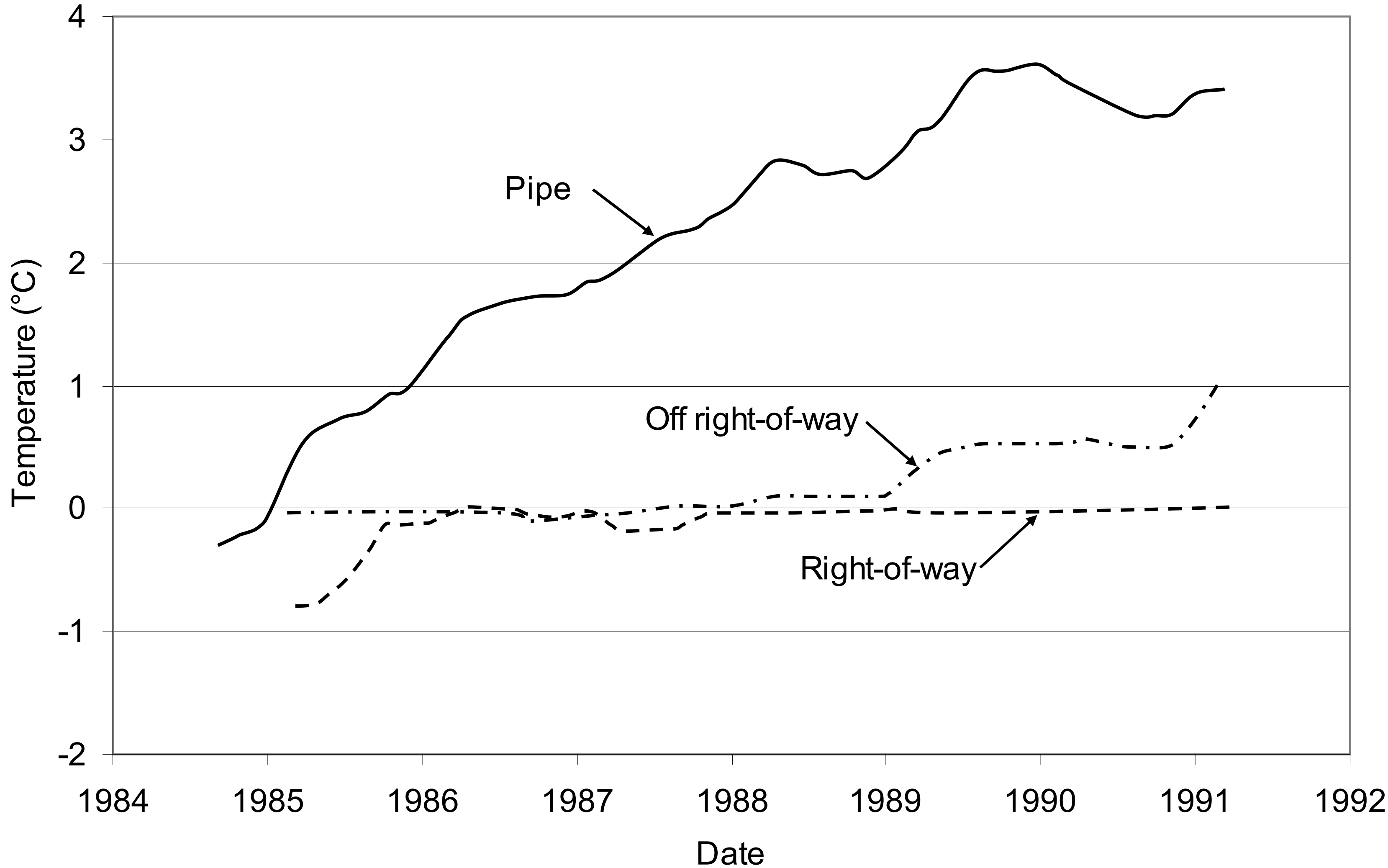

Figure 5-9d: Running mean pipe and right-of-way temperatures – NRCan Site 84-6, KP 819.5 – Petitot River south.
Note: Ground temperatures are measured at pipeline depth (1 m depth). Figure re-plotted from Burgess (1992)

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.

Figure 5-11 presents the amount of observed permafrost under newly cleared or previously cleared sections of the route. To the extent that was practical, the pipeline route followed previously cleared cut lines. One the primary cut lines used was the Canadian National Telegraph (CNT) right-of-way. It was initially cleared in the late 1950s or early 1960s, and therefore had been subjected to approximately 25 years of terrain disturbance prior to pipeline construction. Sections of the route that used other clearings have been plotted separately in Figure 5-11 from the CNT sections. As evidenced in the figure, pre-clearing has resulted in a lower presence of permafrost on the pipeline right-of-way.

This data also helps explain the differences in interpretation of permafrost terrain by the different methods shown in Figure 5-10. The ditchwall log examined only the very near surface thermal conditions. Thus, in sections of the route subject to previous disturbance, such as along the CNT cut line, the terrain would be classified as unfrozen. However, the geophysical methods, which observe thermal conditions at depths of 3 to 9 m, could detect permafrost as being present in those sections of the route that were previously cleared.

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 5-12.

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. Burgess and Lawrence (2000) have completed such a comparison of the ditchwall log and surficial geological terrain units to assess the accuracy of the terrain mapping.

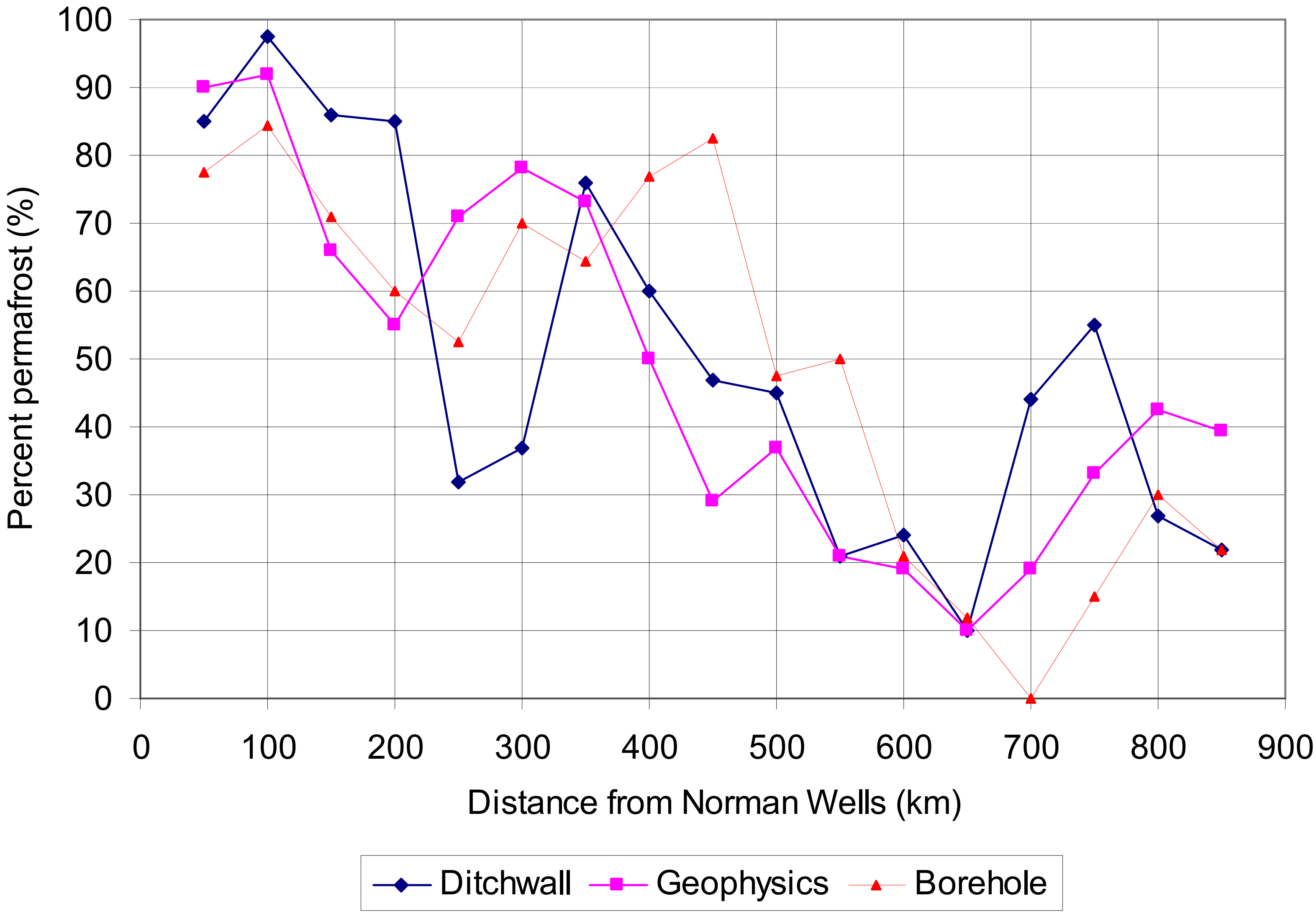

Figure 5-10: Permafrost distribution along pipeline route determined by different methods.
Note: Figure re-plotted from Nixon, Saunders and Smith (1991).

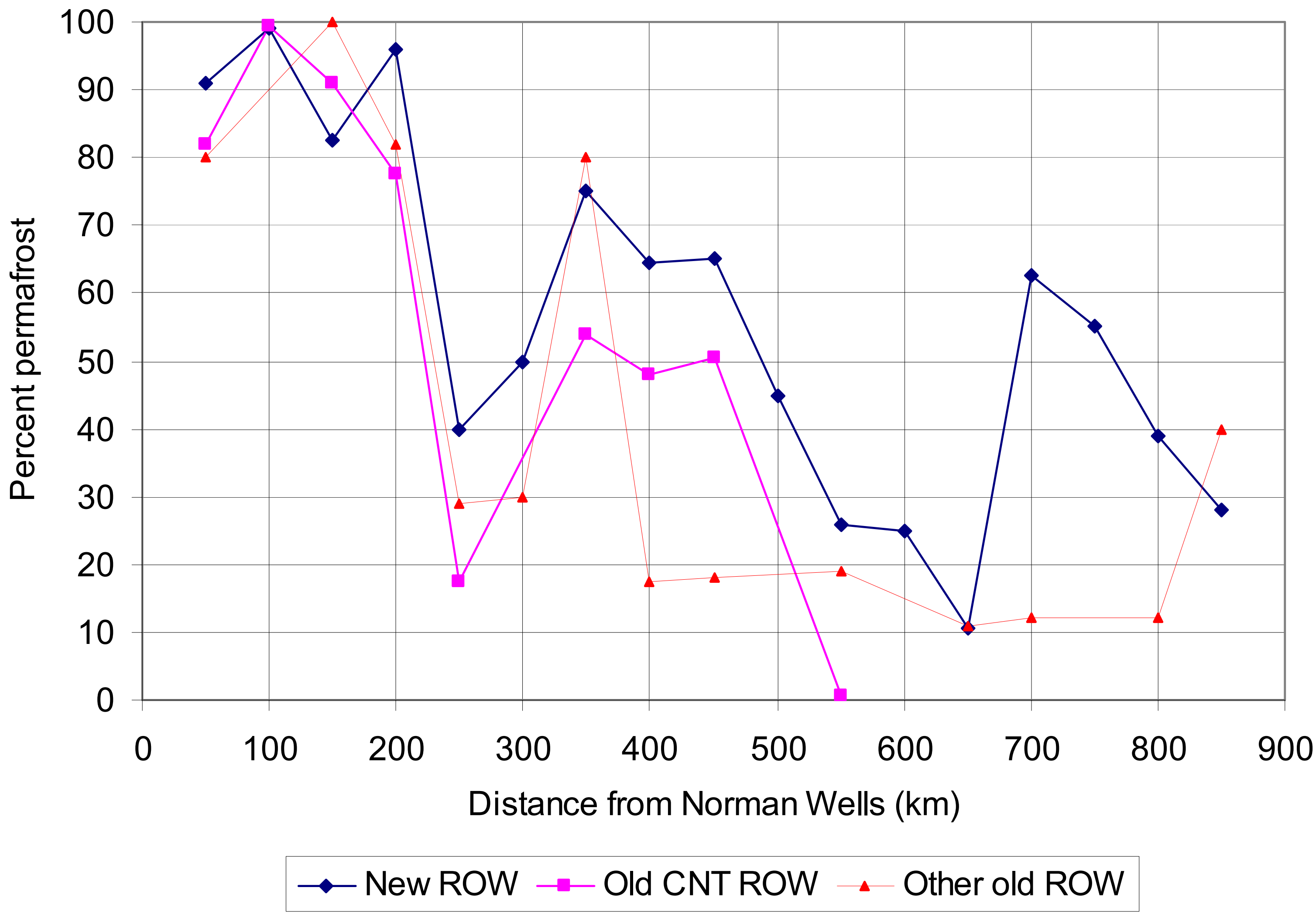

Figure 5-11: Permafrost distribution along pipeline route as a function of previous clearing.
Note: Figure re-plotted from Nixon, Saunders and Smith (1991).

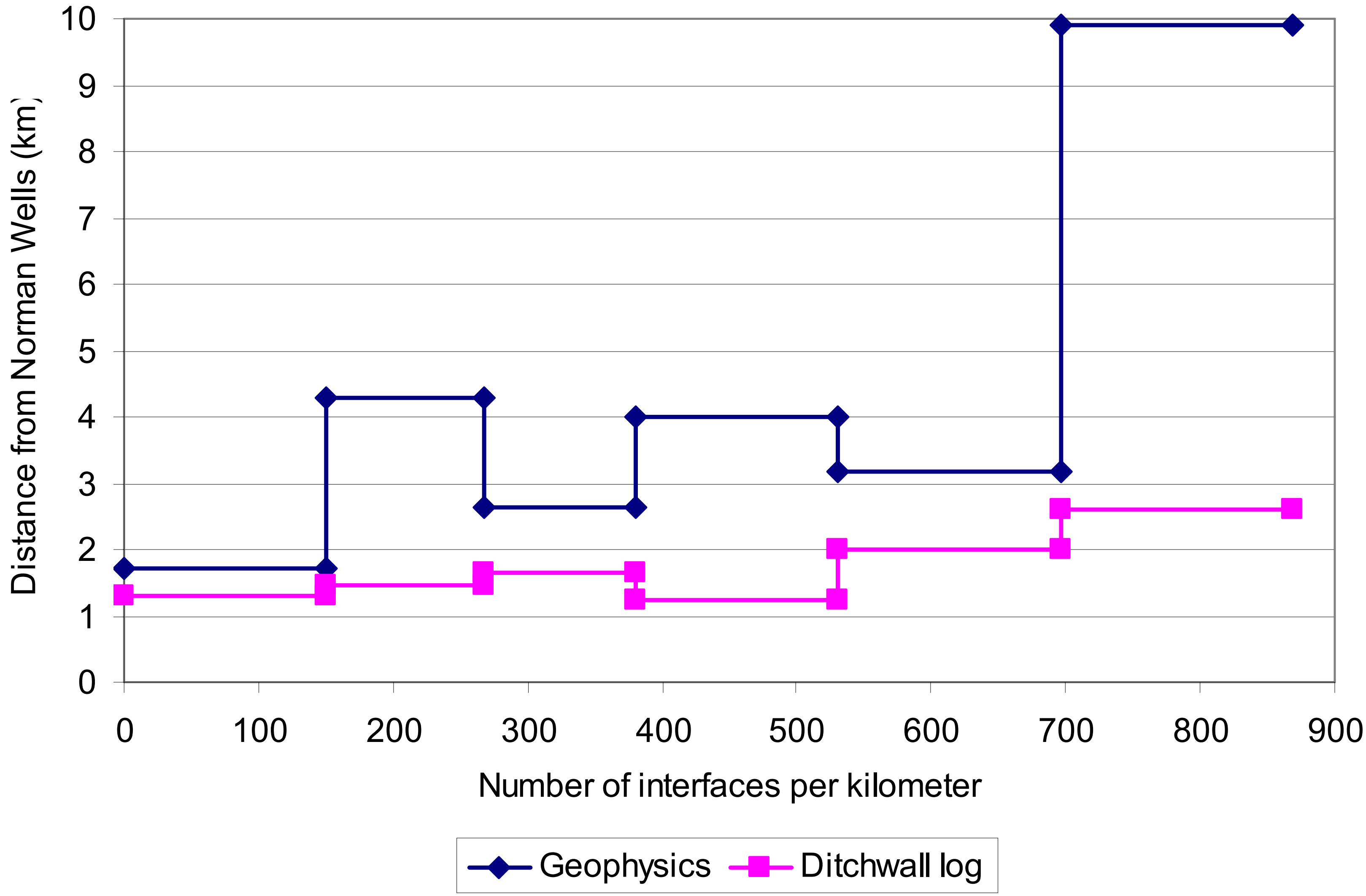

Figure 5-12: Number of thermal interfaces along pipeline route as determined by geophysical techniques and physical observations of the ditchwall.
Note: Figure re-plotted from Nixon, Saunders and Smith (1991).

5.2 Pipeline Performance and Strain Monitoring

Overall, the pipeline has performed well for the first 22 years of operation. One pin-hole leak was detected and fixed in May, 1992, with only minor loss of oil. The problem has not reoccurred, and it is considered to be an isolated case. A second leak occurred at a valve site near the Mackenzie River crossing in 2003. The case of the leakage was determined to be a bear that pulled some geotextile material from the valve pit, which caught and broke a small nipple valve.

Monitoring of the pipeline using smart internal inertial tools (GEOPIG) has been conducted on a regular basis since 1989 to identify any areas where pipe strains have developed. The tool measures distance, internal diameter, vertical and horizontal curvature, and other physical parameters of the pipeline. 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 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, Nixon and Lawrence, 1998). The GEOPIG profiles for the same area and roughly the same time of year have been obtained from Enbridge. Figure 5-13 presents a comparison of the GEOPIG data to a manual survey. 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’s 300 and 318, small wrinkles in the pipe were confirmed by physical inspection. Details of these events are provided in following subsections.

5.3 Active Layer Changes and Thaw Settlement

5.3.1 <u>Ground Impacts</u>

In the initial years following construction, one of the more visible issues with the overland sections of the pipeline right-of-way was subsidence of the ditch line. This in itself did not pose a threat to pipe integrity, but could result in ditch line erosion and eventual pipe exposure in some areas. Wishart (1988) reported that approximately 200 km of the route had experienced some form of subsided pipeline ditch in the first few years following construction. Around 80 km had experienced subsidence in excess of 200 mm in depth, and this was considered to require remediation. This took the form of re-establishment of the ditch backfill mound by placement of new fill, and also placement of diversion berms to re-direct surface water flow.

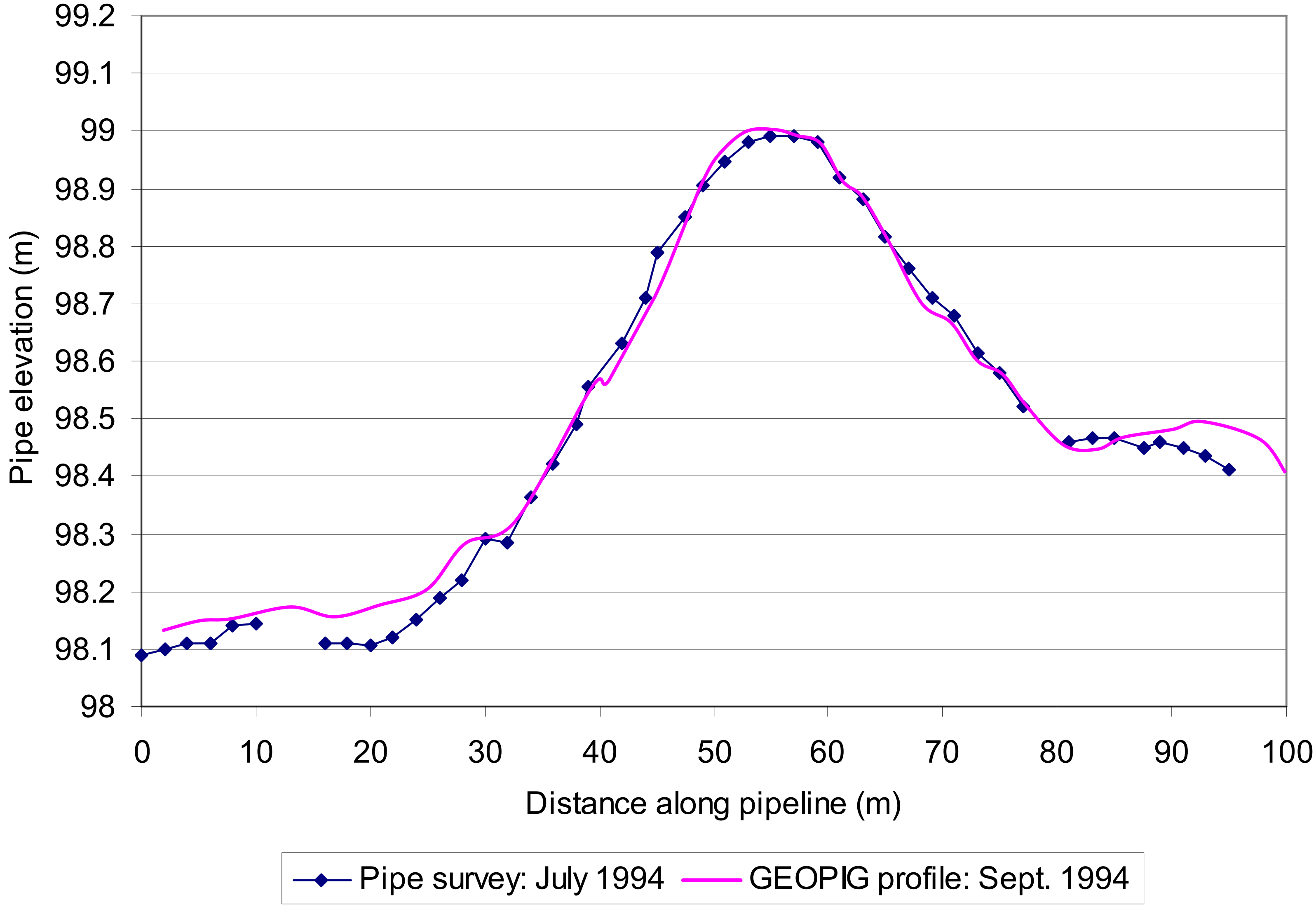

Figure 5-13: Comparison of manual elevation survey to GEOPIG profile of pipe, KP 2.0.

Figure 5-14 shows the schedule of ditch line backfilling in the years following construction. Overall, approximately ten percent of the ditch line was rehabilitated to address settlement. Seeding was used to resist erosion where the subsidence was less pronounced. In later years (1990s), subsidence of the ditch line, in organic terrain has become more prominent. As discussed earlier (see Table 4-1), the design thaw settlement of the pipeline 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.

Surveys of the right-of-way by the NRCan/GSC have been conducted on a regular basis from construction to the early 2000s. Measurements of the active layer thickness and surface settlement as a result of deepening of the active layer were measured. These data are presented in Figure 5-15a through Figure 5-15d for four sites, covering the route from KP 0.1 to KP 783 (Burgess and Smith, 2003; Smith, Burgess, Riseborough, Coultish and Chartland, 2004, with up dates provide by the GSC). The active layer on the cleared right-of-way continues to deepen at many locations, even after 25 years. The active layer off the cleared right-of-way has displayed deepening at several locations. The deepening can be attributed to several causes. First, some deepening may be caused by “collateral” warming from the cleared right-of-way; one example would be the loss of trees on the right-of-way that would reduce the shading of the sun that would otherwise fall on the off right-of-way instrumentation location. A second cause, as most prominently displayed in the off right-of-way active layer depth data from the mid 2000s at Site 84-5B (Figure 5-15d) is the result of surface disturbance caused by forest fires that occurred in 2004.

Transects across the right-of-way at several locations have also been surveyed by the GSC and Enbridge. Figure 5-16 shows a cross section of the right-of-way at a site south of Fort Simpson. The time-progression of thaw settlement is clear in the data. Given the shape of the cross right-of-way profile it is obvious that the first survey was undertaken after some disturbance (thermal, mechanical or both) of the ground had occurred.

The surface or right-of-way settlement observed after one to two decades of operation has been generally less than that estimated for 25 to 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 Table 5-1.

5.3.2 Pipe Settlement Impacts

Some direct measurements of pipe settlement have been made, notably at KP 2.0 by NRCan/GSC. Figure 5-17 shows the survey elevations at KP 2.0. Although the data do not show the total settlement that occurred since pipeline start up in 1985, cumulative settlement in the order of 0.25 m has occurred between 1994 and 2004. Seasonal heave and settlement associated with the freeze-back and thaw is in the range of 0.12 m. It is noted that after 2001 only survey measurements in the Fall were made; hence the seasonal heaving of the pipe was not measured. Other data sources of inferred pipe settlement are provided by the NRCan/GSC, and have been included in Appendix B.

Table 5-1 provides a summary of observed thaw depths and settlement of the right-of-way surface to 1997, for the northern part of the route. The locations of the sites referenced are given in Table B1, Appendix B. The average thaw strain of the soils at monitored sites can be calculated by dividing the observed settlement by the increased 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 (assumed pipe cover plus pipe diameter). The thaw strains are calculated, 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 5-18. The thaw strains in the Alberta Plateau south of Fort Simpson are certainly numerically greater than more northern sites. It is not known how extensive or widespread the situation as shown on Figure 5-18 is along the southern part of the route.

The interaction of the pipeline with thick peat deposits further south along the route has been dramatic. In the peat plateau and fenland areas (Site 12B at KP 608.7 for example), the trench has settled considerably over the pipe. Figure 5-19 presents pipe settlement with time data. These data are taken from Figure 5-16, which presents right-of-way cross section and pipe elevations at a site on the Alberta Plateau south of Fort Simpson. The pipe settled about 0.7 m at this location. 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.

GEOPIG or precise elevation surveys were not run since start-up. This is unfortunate, as much valuable information on initial 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 that 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.

5.3.3 Ground Penetrating Radar

Enbridge, working with the NRCan/GSC carried out trials on the use of ground penetrating radar to determine thaw depth on the pipeline right-of-way. This was carried out in 1993 and1994 under a NRCan/GSC-Enbridge industrial partnership program. Burgess, Robinson, Moorman, Judge and Fridel (1995) describe the use of the method in determining thaw beneath insulated wood chip slopes.

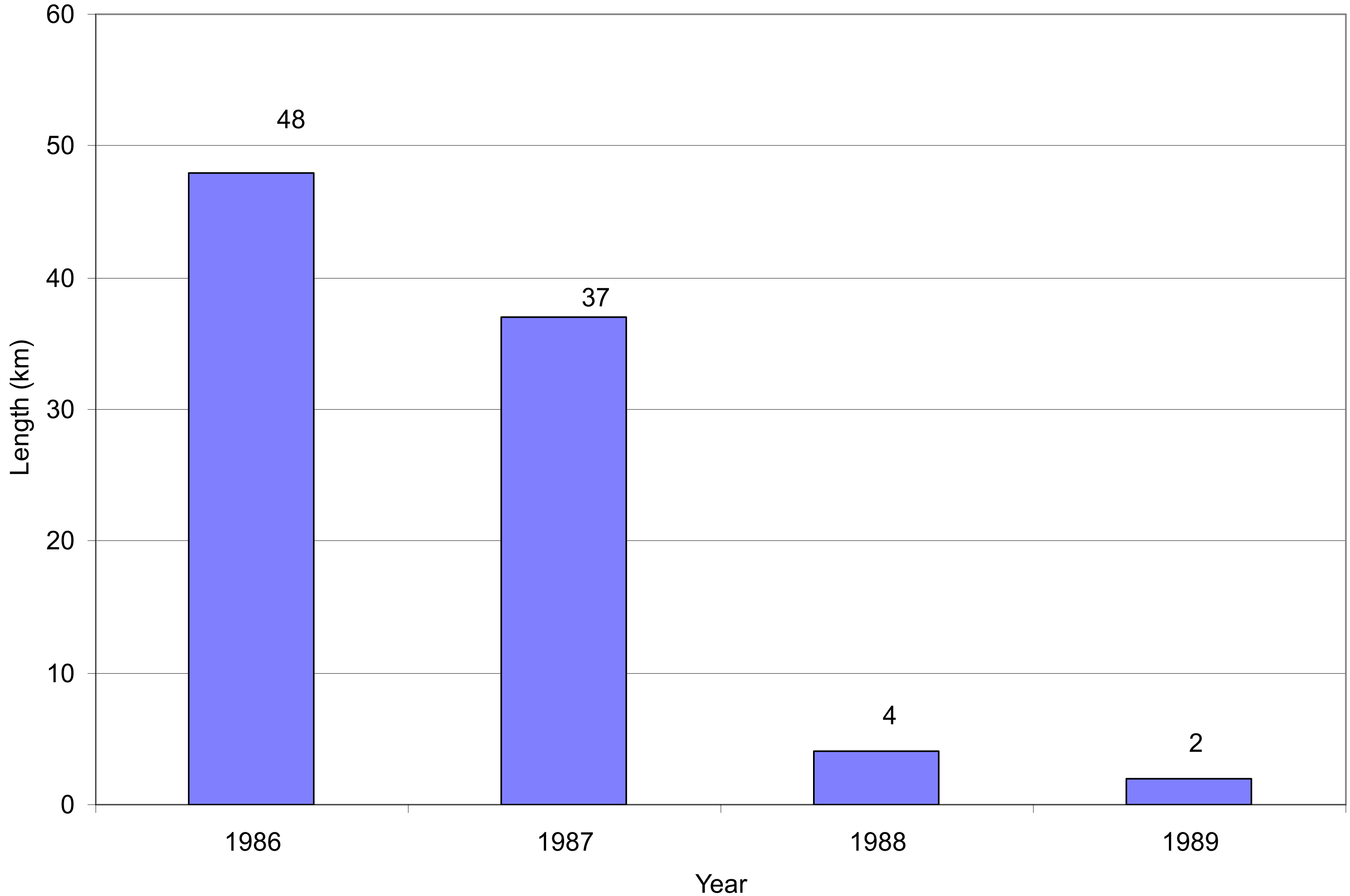

Figure 5-14: Length of pipeline ditch backfilled between 1986 and 1989.

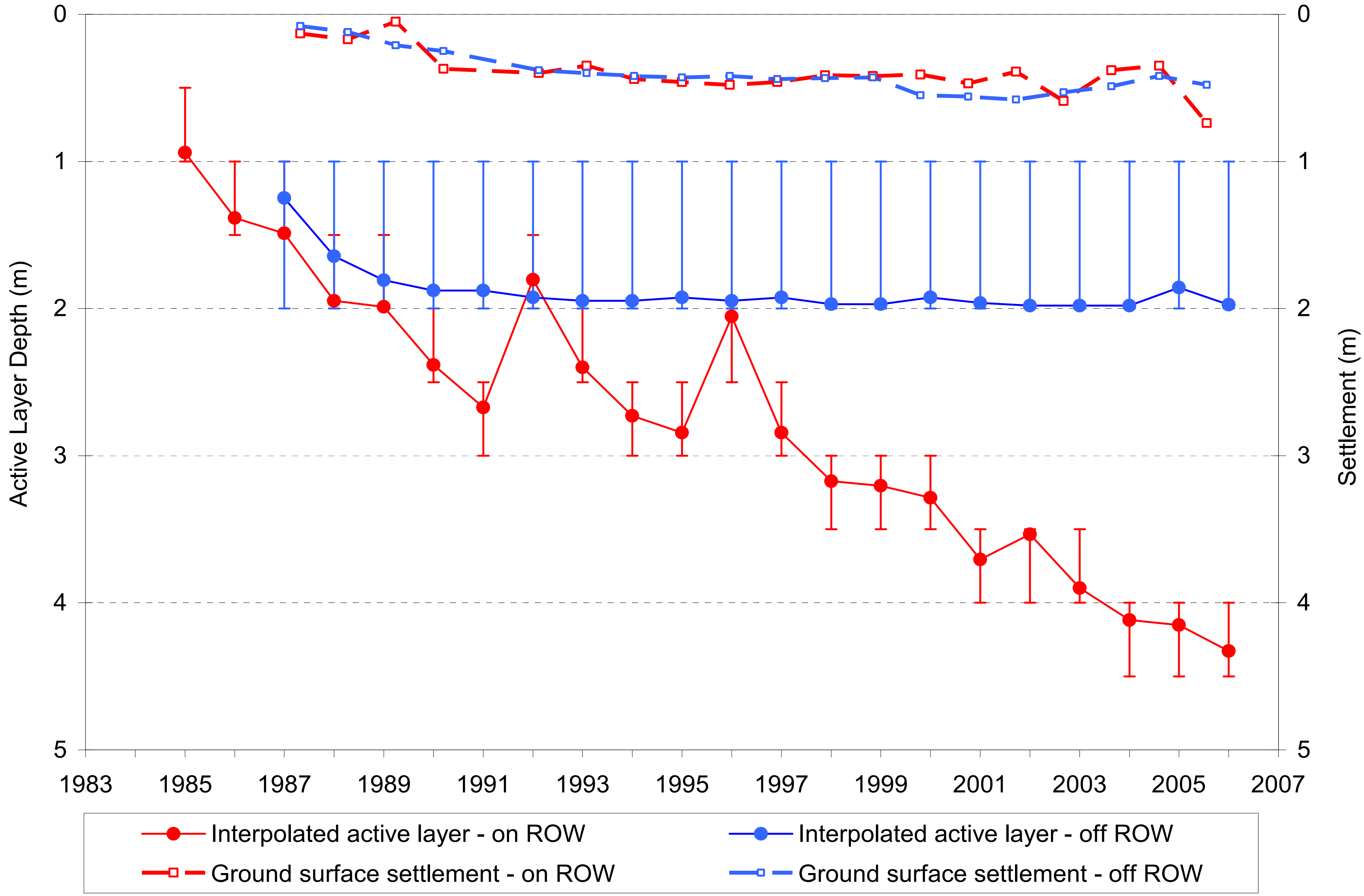

Figure 5-15a: Active layer thickness and ground surface settlement at NRCan site 84-1, KP 0.1.

Note: On right-of-way data in years since 2003 may be unreliable because of potential frost jacking of the surface casing. Data from Smith et al. (2004); Burgess and Smith (2003), with updates provided by the GSC.

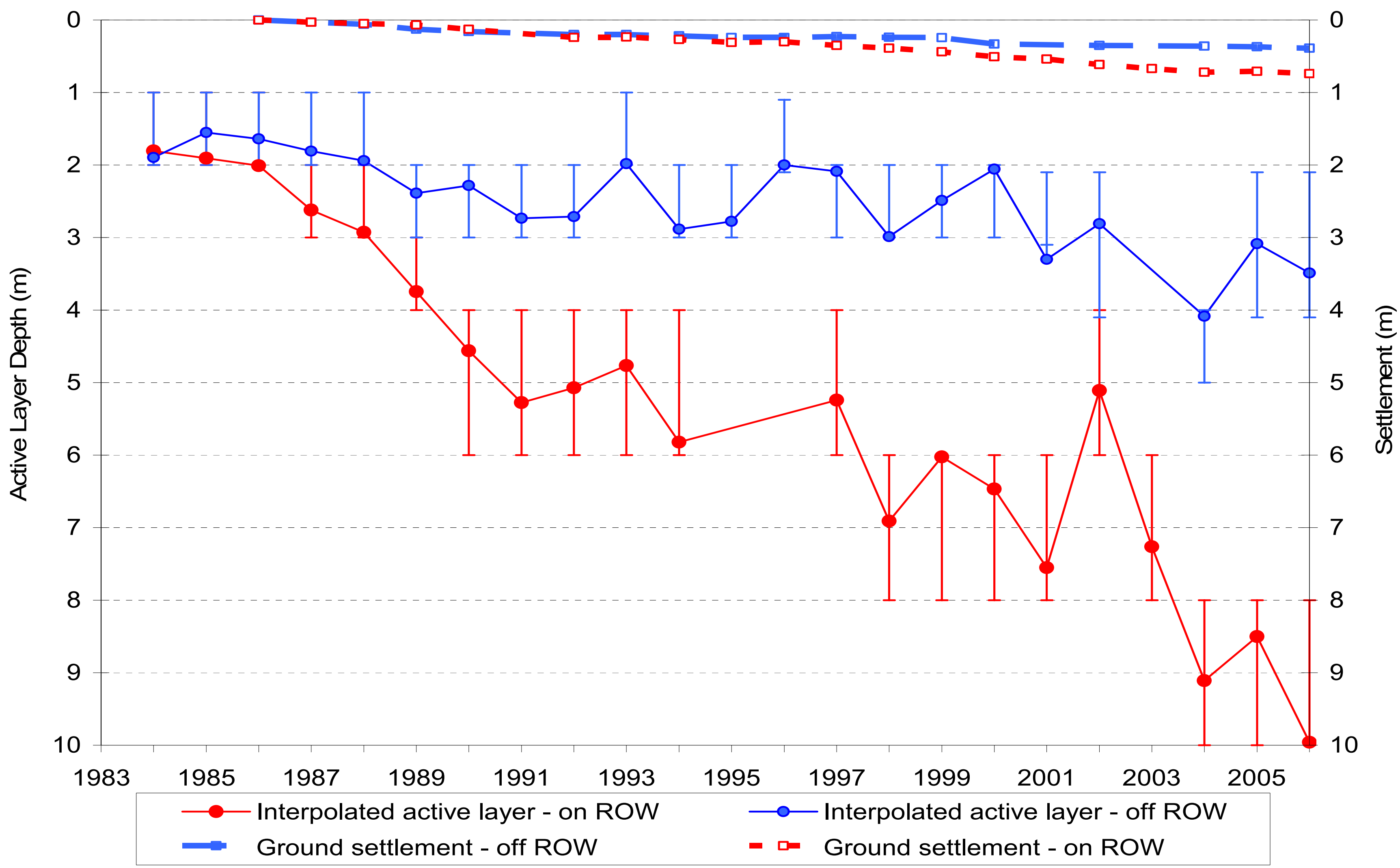

Figure 5-15b: Active layer thickness and ground surface settlement at NRCan site 84-2A, KP 19.
Note: On right-of-way data in years since 2003 may be unreliable because of potential frost jacking of the surface casing. Data from Smith et al. (2004); Burgess and Smith (2003), with updates provided by the GSC.

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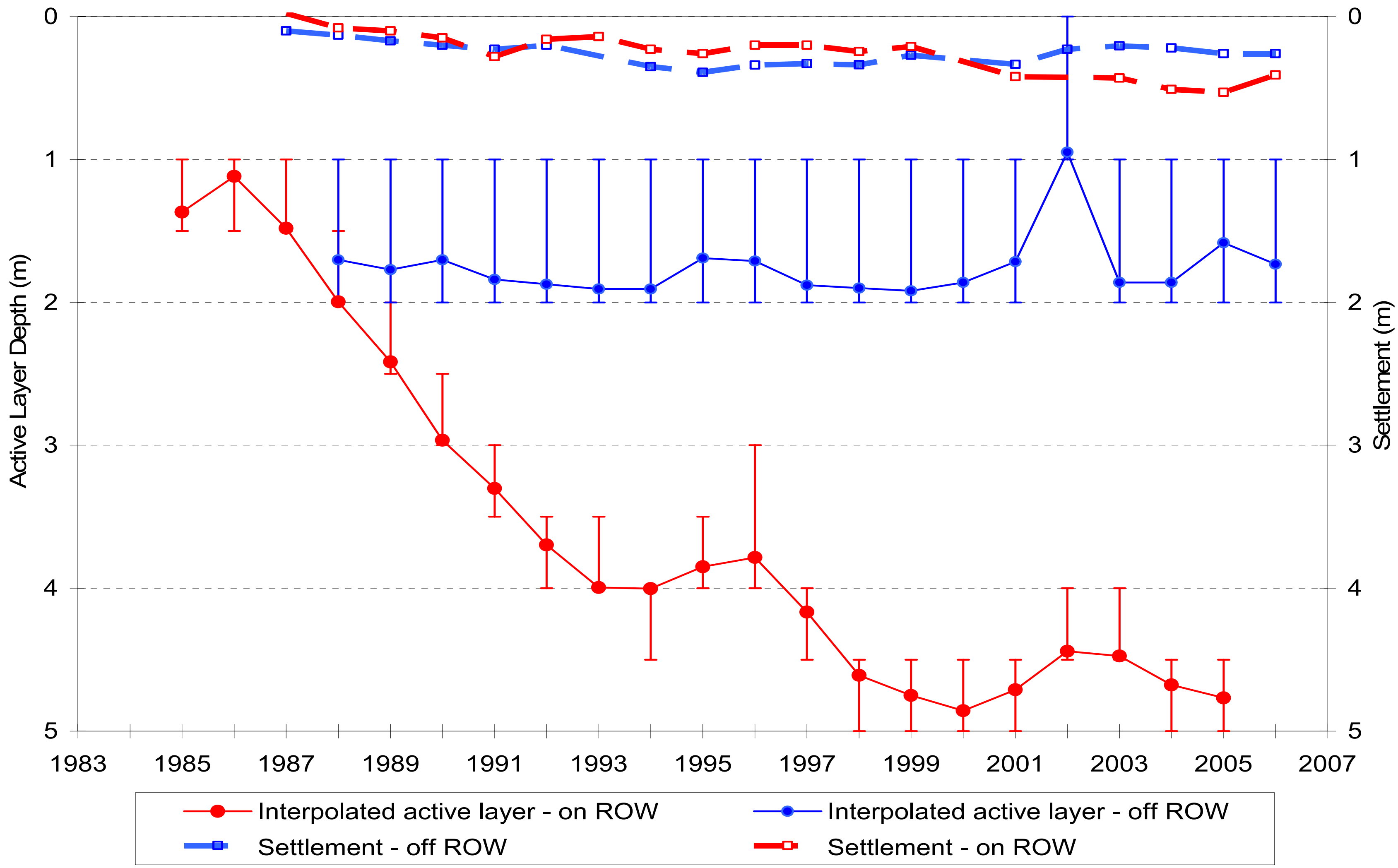

Figure 5-15c: Active layer thickness and ground surface settlement at NRCan site 85-7A, KP 272.
Note: On right-of-way data in years since 2003 may be unreliable because of potential frost jacking of the surface casing. Data from Smith et al. (2004); Burgess and Smith (2003), with updates provided by the GSC. In 2006 thawing had progressed deeper than the lowest thermistor bead at 5 m.

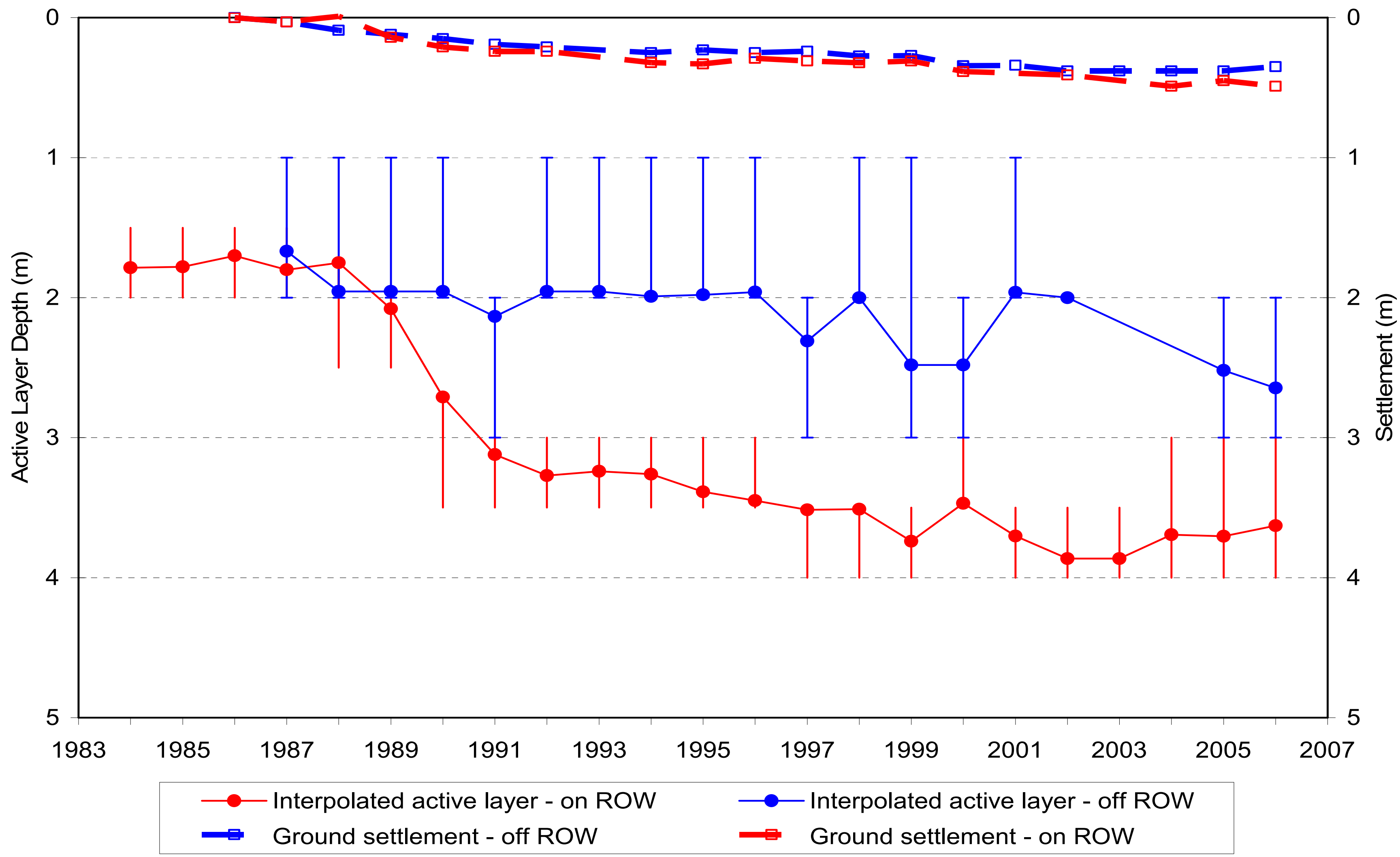

Figure 5-15d: Active layer thickness and ground surface settlement at NRCan site 84-5B, KP 783.
Note: A forest fire occurred in 2004 that damaged some of the thermistor installations. Data from Smith et al. (2004); Burgess and Smith (2003), with updates provided by the GSC.

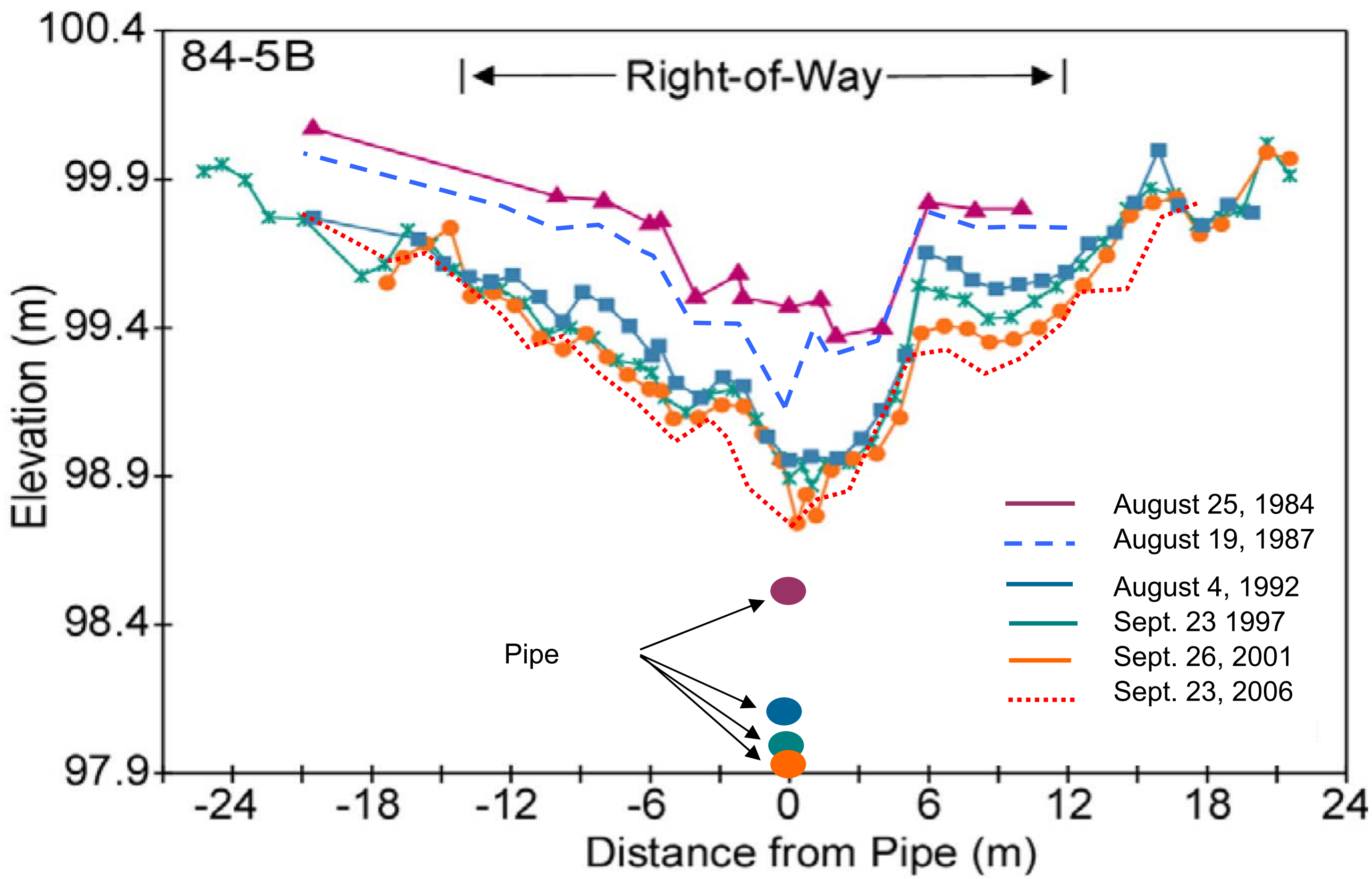

Figure 5-16: Ground surface profile with time at a peatland site south of Fort Simpson (GSC Site 84-5B, KP 783).
Note: A forest fire occurred in 2004 that damaged some of the thermistor installations. Data provided from Burgess and Smith (2003), with updates by the GSC.

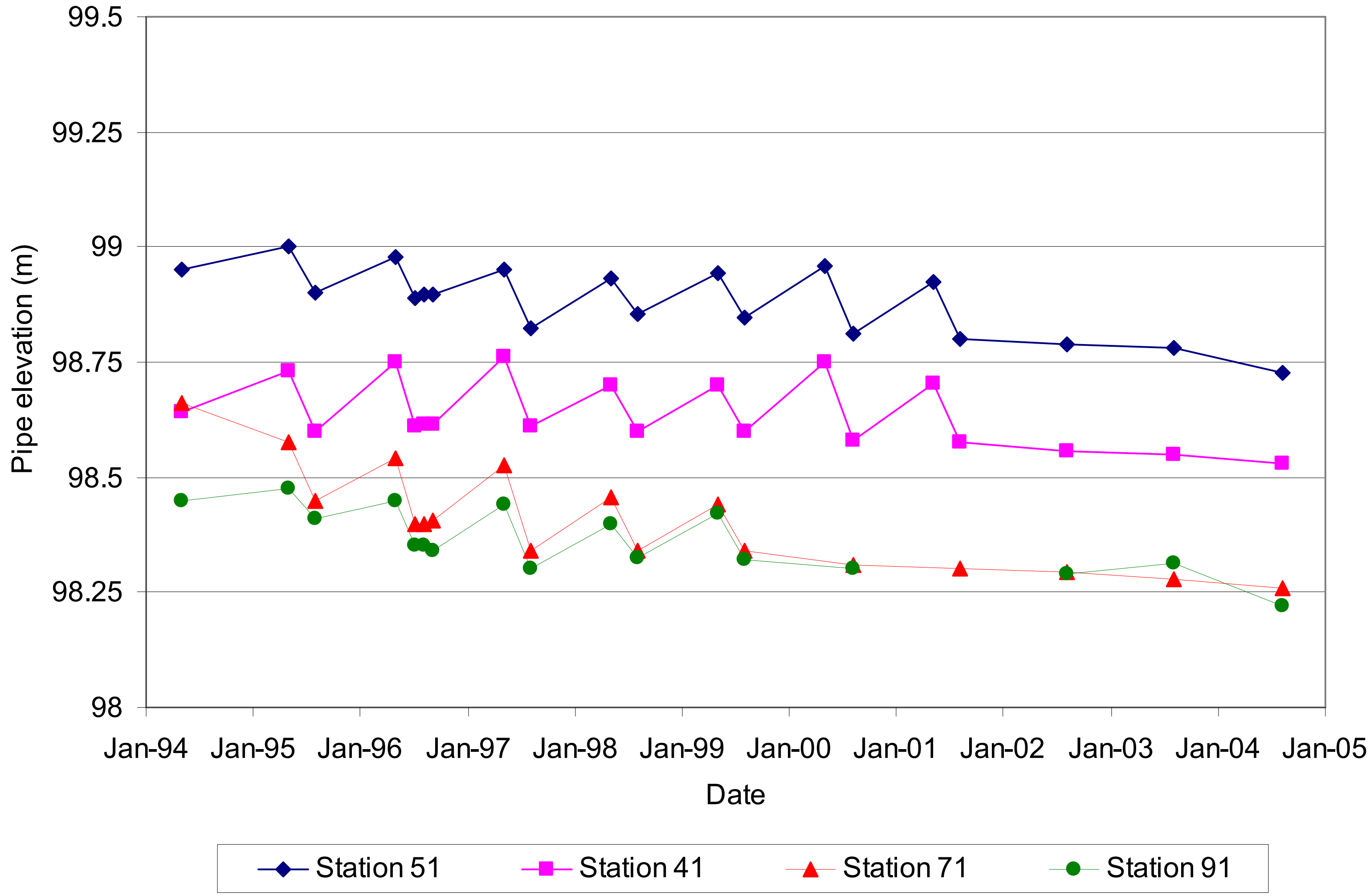

Figure 5-17: Pipe elevation (manual survey) at KP 2.
Data provided from Burgess and Smith (2006).

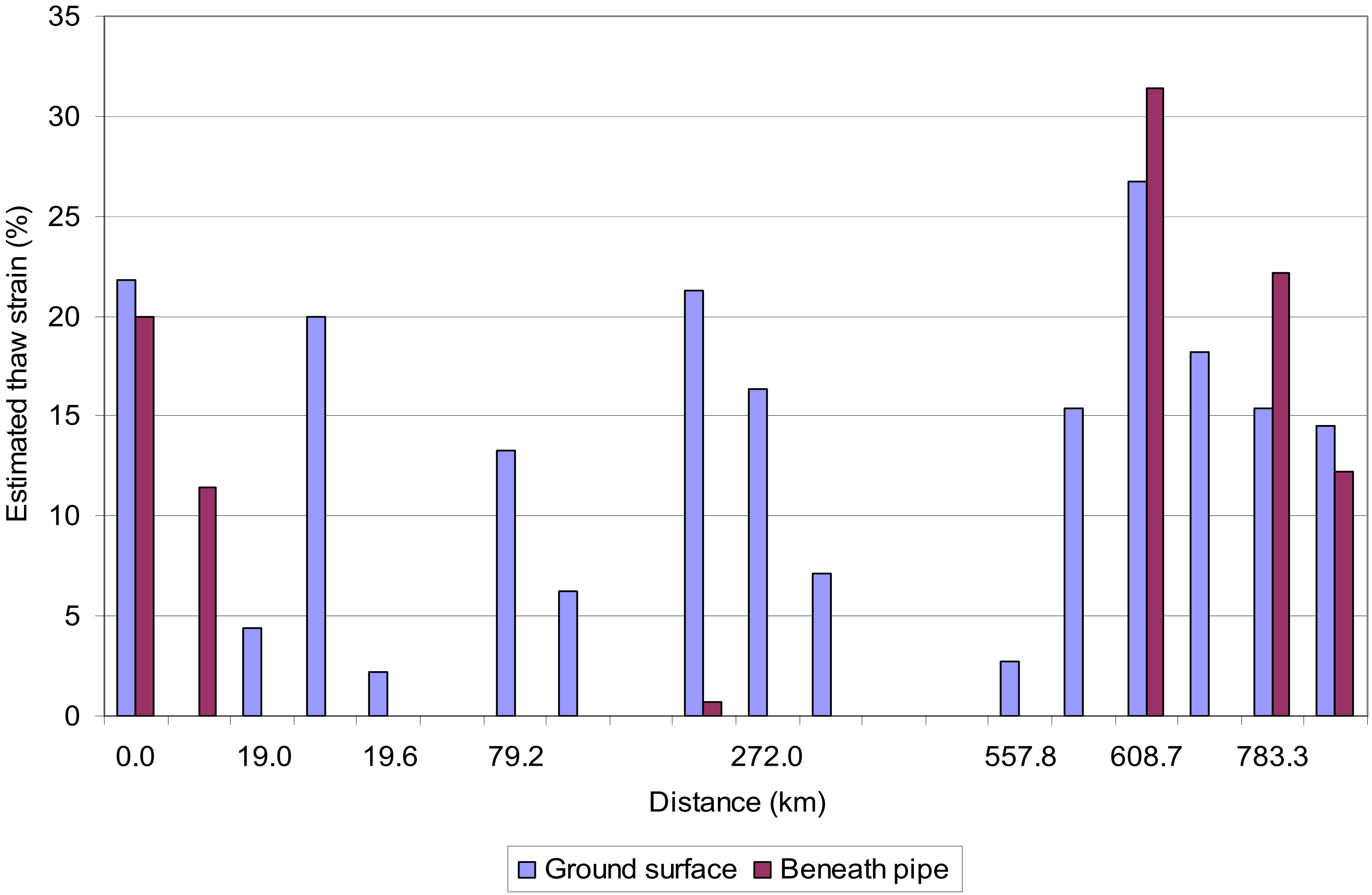

Figure 5-18: Observed right-of-way and pipe thaw strains. Data from Table 5-1.

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Table 5-1: Summary of selected pipe and right-of way thaw and settlement measurements to 1997.

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 Creek	19	4.5	0.2	0.3	4.4			
2b	Canyon Creek	19.3	0.75	0.15	0.05	20.0			
2c	Canyon Creek	19.6	4.5	0.1	0	2.2			
3a	Great Bear River	79.2	2.25	0.3	0.7	13.3			
3b	Great Bear River	79.4	3.25		0.2	6.2			Row Settlement To 1992 Only
7a	Table Mountain	271.2	4	0.2	0.85	21.3	0.2	0.67	
7b	Table Mountain	272	5.5	0.5	0.9	16.4			
7c	Table Mountain	272.3	4.25	0.3	0.3	7.1	0	0	Row Settlement To 1992 Only
8a	Manners Creek	557.8	5.5		0.15	2.7			Pipe Survey In 1996
8b	Manners Creek	558.2	3.25		0.5	15.4			
12b	Jean Marie River	608.7	4.5	1.2	1.2	26.7	1.1	31.4	Settlement To 1992 Was 0.65 M
5a	Petitot River North	783	2.75		0.5	18.2			Pipe Survey In 1992
5b	Petitot River North	783.3	3.25	0.5	0.5	15.4	0.5	22.2	Pipe Survey In 1992
6	Petitot River South	819.5	5.5	0.8	0.8	14.5	0.6	12.2	Row Settlement To 1992 Only; Pipe Survey In 1993

NOTE: Reported thaw depth of 5.5 m represents a minimum. Actual thaw depth exceeds this value.
Average Beneath Surface Strain = 15.0%
Average Beneath Pipe Strain = 14.9%
Source: Burgess (1995, 1997) and Burgess and Lawrence (1997)

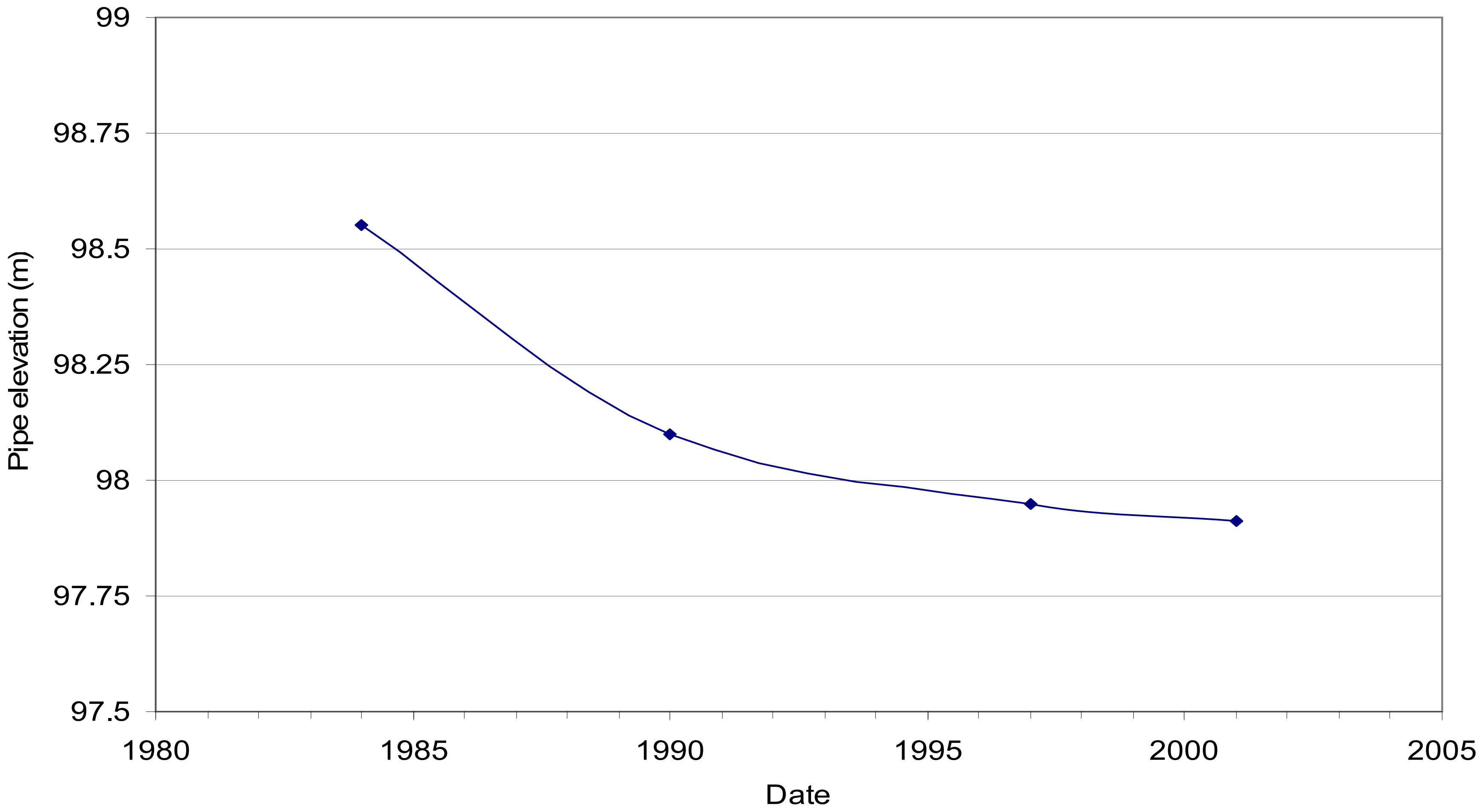


Figure 5-19: Pipe elevation with time at site south of Fort Simpson (GSC Site 84-5B, KP 783). Data from Figure 5-16.

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 primary 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 into a three dimensional framework, in a non-intrusive and rapid manner.

5.4 Frost Heave and Other Loadings

5.4.1 Frost Heave

Within the first 5 to 10 km from the Norman Wells pump station there is evidence of frost heave due to seasonal re-freezing of the thaw bulb formed around the pipe each year. Figure 5-17 shows the seasonal pipe elevation changes at KP 2 between 1994 and 2001. Seasonal movements in the order of 120 mm at KP 2 were observed, and this is considered to be due to seasonal frost heave. No frost heave is evident in the years 2001 to 2004 because of the lack of bi-annual survey readings. These data should be compared with estimates of frost heave of 10 to 13 cm made during the design.

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.

Frost heave was observed during early route reconnaissance along the first few kilometres of the route, as evidenced by cracking and apparent uplift of the backfill mound over the pipe. (Subsequent 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. GEOPIG monitoring has not identified any frost heave development at watercourses.

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 uplift at KP 5.2 has received considerable attention. Ground surface and pipe elevation surveys were carried out by the NRCan/GSC and Enbridge at this location since 1993. The pipe profile and depth of soil cover in 1995 and 1996 are shown on Figure 5-20. 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.

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 the ground surface from beneath the seasonal frost cap, indicating a plentiful water supply. These factors could combine to provide weak, low-density soils that would provide reduced resistance to upward pipe movement. In light of the high axial loads in the pipe resulting from the thermal expansion of the pipe (resulting form the warming of the pipe from its backfilled temperature of near – 30°C to its post-1993 summer temperature of about + 8°C), the pipe may have buckled towards the ground surface.

Figure 5-21 presents the seasonal movement of several survey monitoring points between 1996 and 2006. The sinusoidal displacement is clearly evident. This pipe movement had likely initiated prior to June 1993, before the revised pipe temperature regime had been instituted. Therefore, it is likely that this pipe section began displacing 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 groundwater springs on, or off the right-of-way would also provide a natural cause for greater localized thawing.

It was concluded that the uplift was likely initiated by frost heave with further movements caused by 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.

The issue of upheaval buckling in permafrost regions was recently addressed by Palmer and Williams (2003). The authors suggest a combination of small amounts of frost heave in combination with topographic highs may be sufficient to initiate an upheaval mechanism and "threaten the security of arctic pipelines". Discussions by Oswell, Cavanagh, and Skibinsky (2005) and Nixon and Vebo (2005) refuted many of the contentions raised by Palmer and Williams.

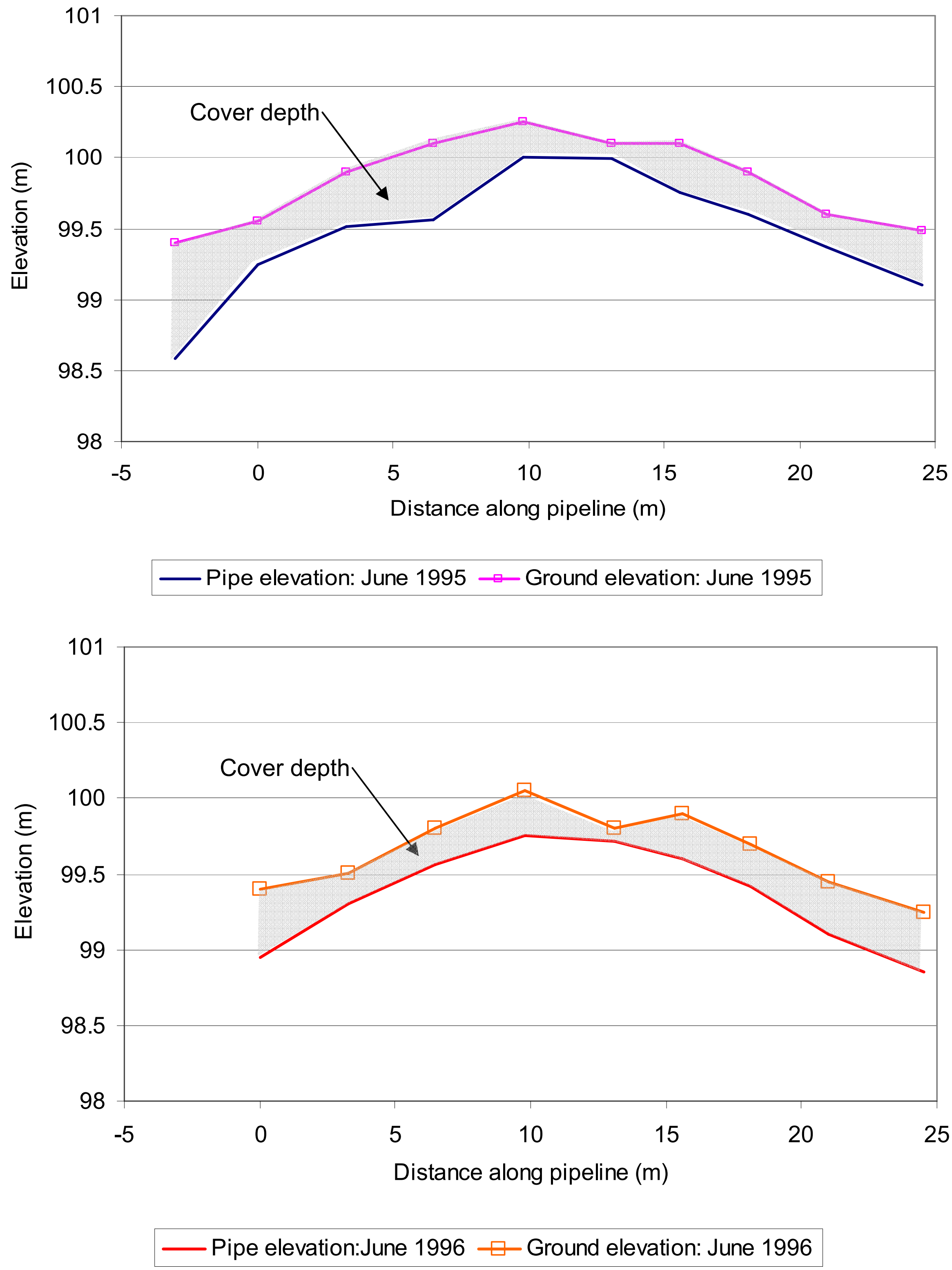


Figure 5-20: Ground surface and pipe survey at KP 5.2 in June 1995 and June 1996.

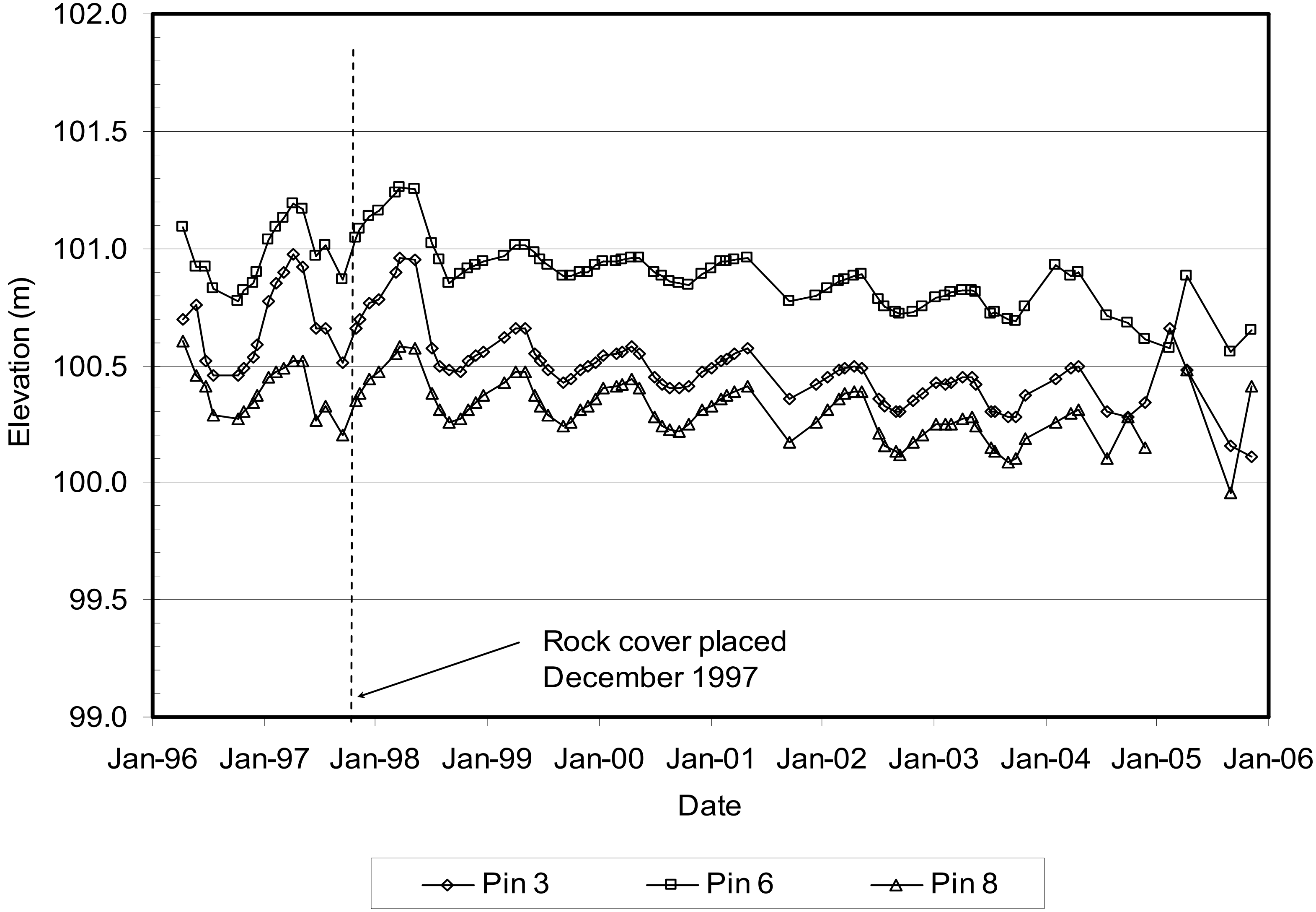


Figure 5-21: Survey data at KP 5.2.

Enbridge, in the fall of 1997, undertook a remediation program. Although several options were available (physically cutting and replacement, reburial below existing grade, covering) it was considered that covering the pipe first with sand bags and a geotextile, followed by rock fill and then mineral soil, would provide sufficient cover to protect the pipeline integrity.

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 2005. Post remediation monitoring has shown continued seasonal movement. The placement of the rock fill in December 1997 was sufficient to control the delta-T thermal expansion, but small amounts of seasonal frost heave and thaw settlement continue to occur. The rock cover may also be heavier than necessary, as slow cumulative settlement of the pipe since the rock placement has developed. The settlement between the summer of 1998 and the end of 2005 averages about 3 mm/month. Although the amount of settlement is small, GEOPIG monitoring of the buried sag bend at the upstream end of the up-lift section is showing progressively increasing bending strains.

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.

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 that were essentially similar to those of the Design Probable Event. 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 were noted relating to pipe integrity or equipment operation. Savigny, Sego, and MacInnes, (1992) documented a possible earthquake induced landslide in the Nahanni region of the Northwest Territories as a result of the December 1985 event.

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, 12 and 15 thaw seasons, the depth of thaw would be in the order of 4.25 m, 5.6 m, and 6.2 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 5-22. 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 15 year thaws are consistent with the predicted behaviour. [1]

The sites selected for this analysis consist of slopes that were not insulated with wood chips, or where thermistor cables were installed beyond the wood chips, often on the crest of the slope. In general, the sites with colder initial ground temperatures (Slopes 2, 18, 22, 29 and 74) are all "north" facing slopes. The two warmest slopes (Slopes 13 and 52) are "south" facing. Slope 22L, although having a warm initial ground temperature has experienced less than expected thawing, likely due to the slope's orientation.

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 4.6.5, 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.

In the spring of 1986, a slope reconnaissance 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 fungal activity were thought to have been triggered by wood rot in the original wood chips.

[1] Figure 5-22 is revised from the version presented in the 1999 version of the Monograph (Figure 5.21). In the course of preparing the 2007 update, the original ground temperature data was re-examined. This resulted in the re-positioning of some slopes in terms of their initial ground temperatures.

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

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 5-23 shows a photograph of the installed ventilation pipes.

By the early 1990s, several wood chip slopes (23 slopes or 41 percent) continued to experience localized hot spots (Burgess, Lawrence and MacInnes, 1993; Burgess, Lawrence, MacDonald and Desrochers, 1995). Most of these were not of significant magnitude and no remedial action was required, as the general performance of the insulating layer was considered to be satisfactory. By the mid to late 1990s, there were no continuing issues related to hot spots.

The degradation of the wood chips has been variable in the past 20 or more years. Figure 5-24 presents two photographs of wood chips taken in the Fall 2005. Both wood chip samples are from the same slope. Figure 5-24a is of wood chips in a former hot spot while Figure 5-24b is of wood chips in an adjacent non hot spot area. In the areas where the wood chips were not subject to heating, the physical degradation has been relatively slow.

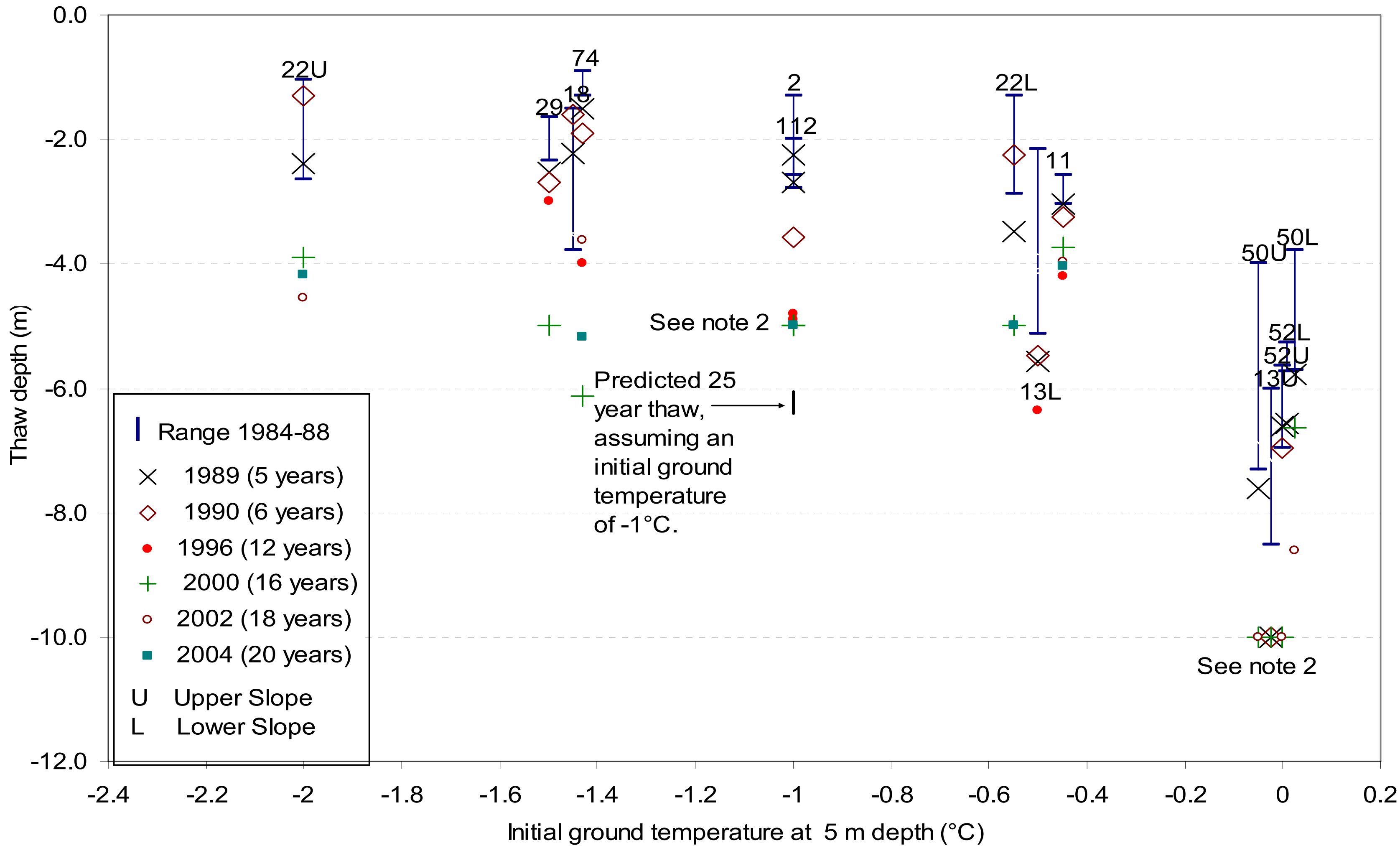


Figure 5-22: Thaw depth development on non insulated ice-poor slopes.

Notes: 1. Years shown in legend (ex. 5 years) denote years since ROW clearing not start of operation.

2. Thaw depth at Slopes 2, 112, 50U, 13U, 52L and 52 U exceeded the depth of the last thermistor bead.

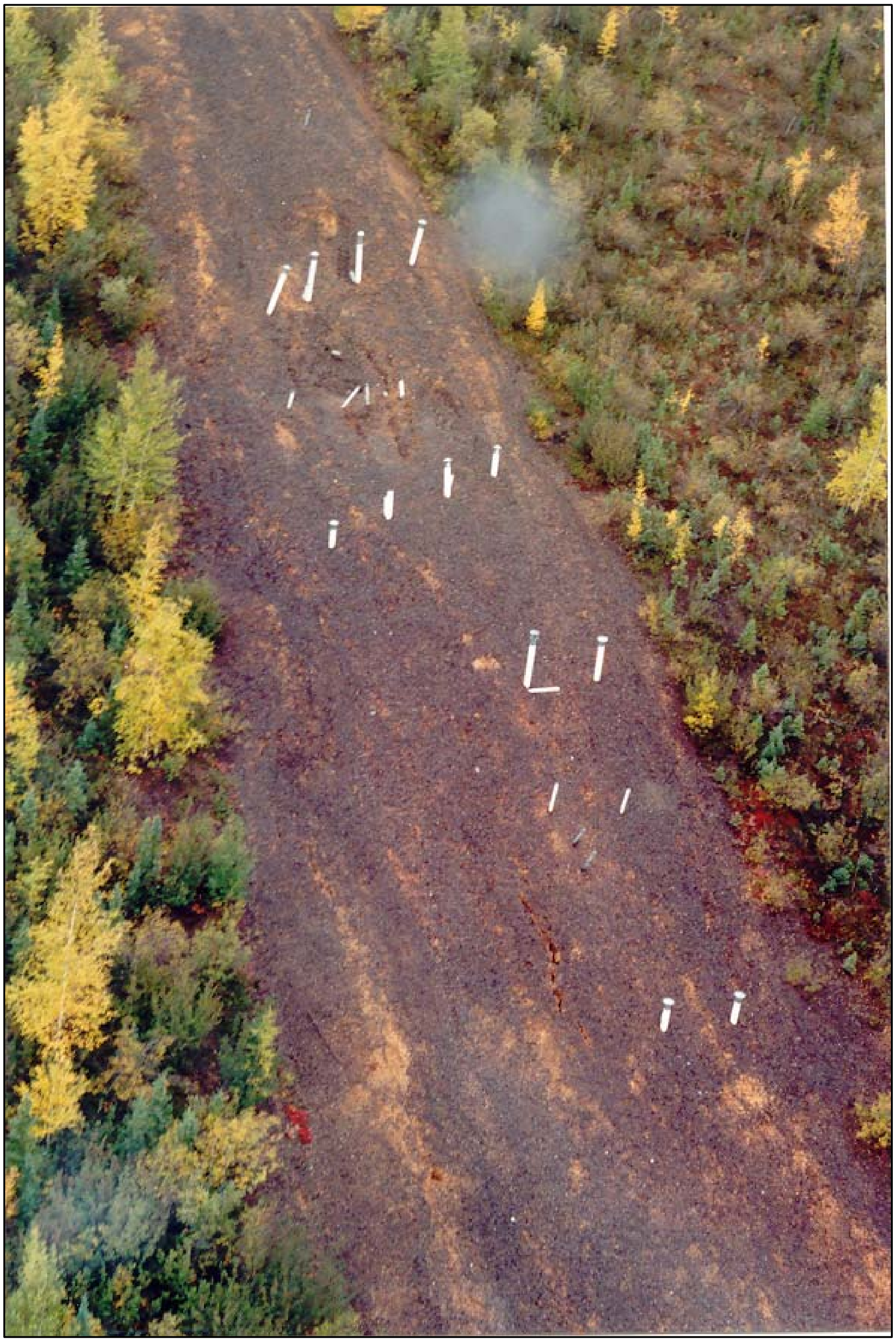


Figure 5-23: Aerial view of ventilation pipes on slope of Ochre River south (KP 286.7).
(Photograph courtesy on Enbridge Pipelines (NW) Inc.)

(a) Decayed wood chips

(b) Undecayed wood chips

Figure 5-24: Decayed and undecayed wood chips at Slope 79, KP 279 – September 2005.
(Photographs courtesy on Enbridge Pipelines (NW) Inc.**)**

5.6.2 Thaw Performance

The thaw performance of the slopes has been monitored on a regular basis since construction. Thermistors and pneumatic and standpipe piezometers were installed to permit the evaluation of thaw depth progression and porewater pressure generation. As the thaw depth progressed past the depth of the lowest thermistor bead new and deeper thermistors were often installed to provide on-going data. The depth reference is taken as the ground (mineral soil) surface. Where wood chips are present, this is the base of the wood chips.

As part of the on-going stability review of the insulated slopes, physical probing has been carried out on selected slopes. Probing was first performed in 1992, with follow-up surveys being conducted in 1996, and every second year since. Probing is conducted in the late September to early October period when season thaw would be the greatest. 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. Figure 5-25 presents two examples of the physical probing. Examination of the data shows that on any particular slope there is often very good agreement in thaw profiles between one year and the next. Between slopes, there are often significant differences between the shapes of the thaw bulbs. In some cases the thaw is very well confined to a narrow area over the ditch line (for example Slope 29B, Great Bear south – Figure 5-25a), and in other cases, a very wide, deep and broad thaw bulb has developed (for example Slope 44 – Figure 5-25b). There may be differences between the thaw depth measured by thermistor cables and measured by probing. The former case measures the depth to the 0°C isotherm, whereas the latter measures the depth to ice-bonded permafrost. These two depths may not always be the same.

The purpose of the wood chips was to retard the rate of thaw, particularly in ice-rich soils that could generate high pore water pressures. It was recognized that thaw would occur over time. Figure 5-26 shows the thaw depth as a function of wood chip thickness. The solid line shows the predicted 25-year thaw. Measured thaw depths after four periods are shown for comparison. The general observation to be made from the data is that the depth of thaw had, even by the mid 1990s greatly exceeded the 25-year design prediction. Examining the data on the basis of slope aspect (north facing versus south facing), as was done for the non insulated slopes, shown on Figure 5-22 provides little insight. In general, the slopes with less thawing (above the 25 year design line) are predominantly north facing slopes, but of the slopes that have deep thawing, there are equal numbers of north facing and south facing slopes. On numerous slopes the actual depth of thawing is not possible to determine because the thawing has exceeded the depth of the available thermistor beads.

There are a number of reasons for the thaw exceeding design 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 many slopes. On other slopes, the ice contents may have been lower than assumed in the geothermal design, which would result in more rapid thaw progression. Secondary effects such as warmer air temperatures, or subsurface groundwater flow near the ditch line may be contributing to the higher ground 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 well below 3 m.

The thaw performance of the slopes can be compared to the predictions and historical data presented in Figure 4-3 (Oswell and Skibinsky, 2006). Figure 5-27 presents this data for all slopes (insulated and non insulated) for which data are available. The dataset presented in Figure 5-27 provides little, if any, way to meaningfully consider the data and to compare them to that presented in Figure 4-3. Hence it is necessary to consider a number of key parameters. The parameters that can be examined for the dataset include the following:

- Pre-clearing versus no pre-clearing. This comparison considers the fact that much of the Enbridge right-of-way followed pre-existing cut lines, most notably the Canadian National Telegraph (CNT) right-of-way. The pipeline route followed these cut lines as a means of reducing the likelihood of encountering ice-rich soils. Where feasible, the pipeline also followed cut lines on slopes.
- North versus south insulated slopes. This comparison considered the fact that southward facing slopes may be more susceptible to warming than northward facing slopes because of the greater solar exposure.
- Wood chip thickness. Thicker layers of wood chips could be expected to reduce the long-term thaw rate on slopes. Several figures present data separated by wood chip thickness ranges, from 0 m to greater than 1.2 m.
- Ground temperature. Slopes with warmer ground temperatures may be reasonably expected to thaw faster than slopes with colder ground temperatures. The criteria used to differentiate warm slopes from cold slopes was a ground temperature warmer or colder than -1ºC at 5 m depth, as measured by thermistor cables in September of the first year of construction (either 1984 or 1985).

<u>Impact of right-of-way clearing</u>: Figure 5-28 and Figure 5-29 present the pre-cleared and non-cleared data, respectively. In Figure 5-28, there is no clear trend in the data, except to observe that a number of slopes have initial thaw depths well in excess of 1 m. Conversely, Figure 5-29 for non cleared sites has fewer sites with initial thaw, as would be expected; but, most of the non cleared sites that have initial thawing were not insulated with wood chips, perhaps because these sites were assessed prior to construction to be thaw stable prior to construction. For the remaining insulated slopes, the majority of the thaw depth data fall in the upper “cleared and undisturbed” zone or the “cleared and disturbed” zone.

Given the time period in which pre-clearing took place, it may be inappropriate to begin the square root time scale at 0 for Figure 5-28. As the clearing and thermal degradation began in about 1959 corresponding to the construction of the CNT line, then by 1984, the time scale at the start of pipeline construction would be at 5 $years^{1/2}$ (25 years). Figure 5-30 presents the data with the adjusted time scale. For this arrangement, the thaw depth progression for most of the pre-cleared slopes falls within the band of “cleared and not disturbed”. Because of the impact of the pre-clearing has on thaw depth, all subsequent data for pre-cleared sites will be presented using the adjusted time scale.

Impact of Slope Orientation: Figure 5-31 and Figure 5-32 presents thaw depth data by slope orientation. Figure 5-31 presents the data for north facing slopes and Figure 5-32 presents the data for south facing slopes. Comparison of Figure 5-31 and Figure 5-32 suggests, at least tentatively, that the north facing slopes have more slopes with less long-term thawing than south slopes.

<u>Impact of wood chip thickness</u>: Figure 5-33, Figure 5-34, Figure 5-35 and Figure 5-36 present thaw depth separated by wood chip thickness. Figure 5-33 presents data for non insulated slopes. It is clear that thaw depth progression was rapid in these slopes and all sites had some initial thaw, ranging from 1 m to 6 m. The thaw was either consistent with the historical thaw limits of "cleared and disturbed" or exceeded these limits.

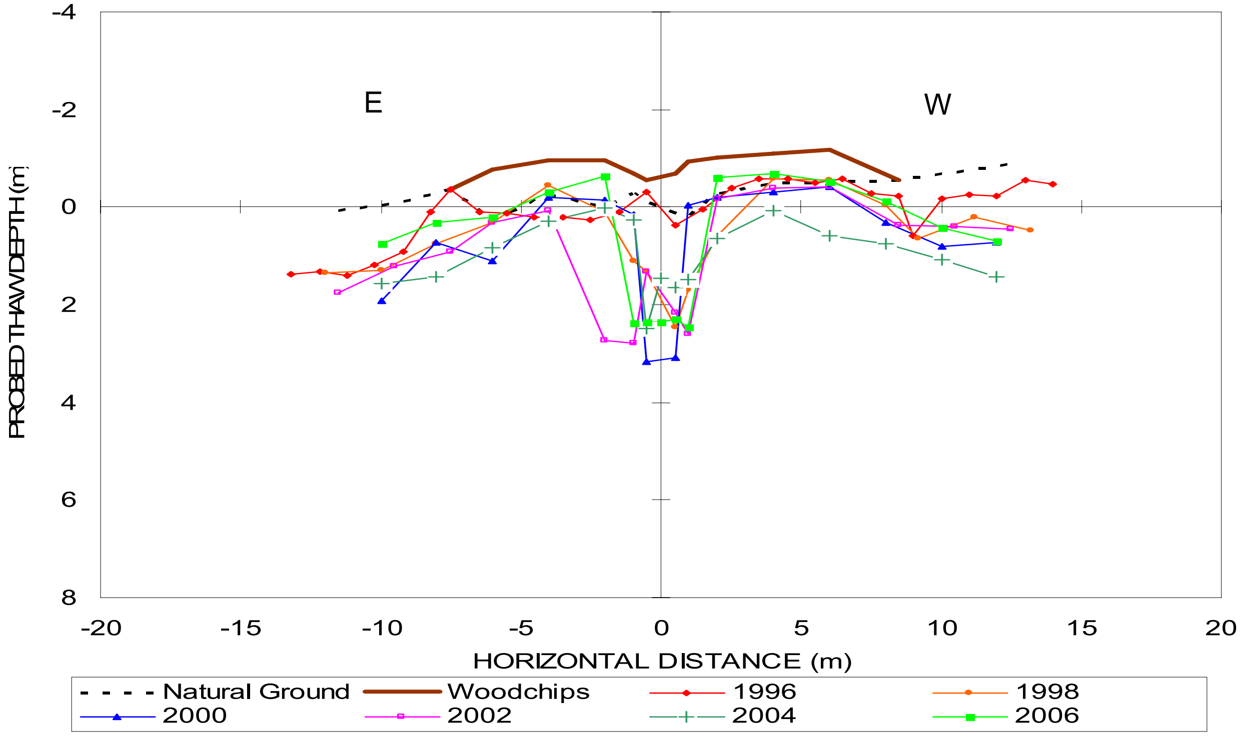
Figures 5-34 to 5-36 present data for slopes with progressively thicker wood chip layers. In most cases, the data suggests that the thaw progression was generally consistent with the upper portion of the historical thaw limits of "cleared and undisturbed" or the lower portion of the historical thaw limits of "cleared and disturbed". Given that the design intent for the slopes design was, in most cases, to permit long-term thawing, albeit at a slow rate, these data are entirely consistent with the design intent and the historical data.

Although self-heating of wood chips occurred on a number of slopes in the early years, this heating did not impact thaw depth measurements at the thermistor locations.

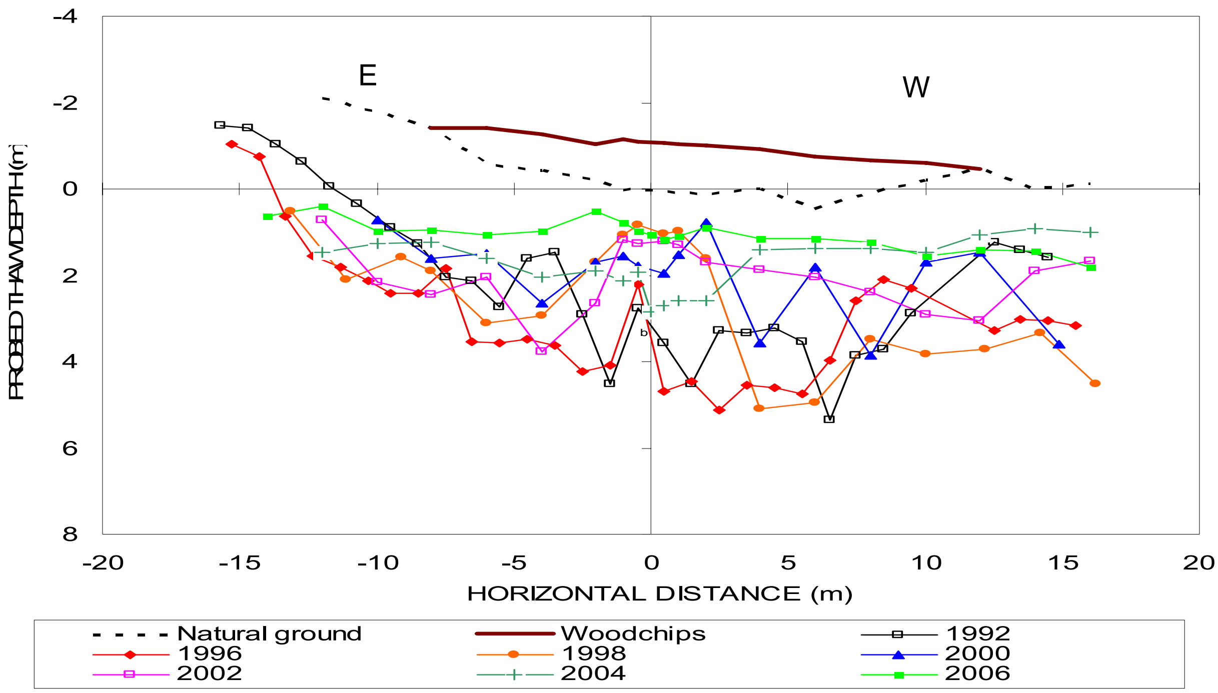
<u>Impact of ground temperature</u>: Figure 5-37 and Figure 5-38 present data for warm and cold slopes, respectively. As noted previously, the criteria for designating slopes either warm or cold was the initial ground temperature at 5 m depth being warmer or colder than - 1°C. In general the slopes designated as warm experienced more thaw than the cold slopes. In many cases this was partly due to the deeper initial thaw depth that was measured. There were no warm slopes that experienced thawing consistent with the historical trends of "cleared and not disturbed".

Review of the historical thaw data on slopes has shown the thaw behaviour to be generally consistent with the historical data that was gathered in the 1970s and early 1980s in support of the initial design. Key points made from this analysis are:

- Time is perhaps the most important factor in determining thaw depth. The monitored sites that were cleared as part of the CNT construction had already experienced considerable thaw when the pipeline was constructed.
- The success of the surface wood chip insulation at retarding thaw is apparent by comparison of the various thicknesses.
- Slope orientation appears to have a relatively small impact on thaw rates.
- The initial ground temperature appears to have a strong influence on the thaw rates over time. Sites with "warmer" ground temperatures will thaw faster than sites with initially colder temperatures.



(a) Probe data from Slope 29B, Great Bear River south, KP 79.4



(b) Probe data from Slope 44, Unnamed creek north, KP 133

Figure 5-25: Physical thaw depth probing data from two slopes. Measurements were taken in late September, early October.

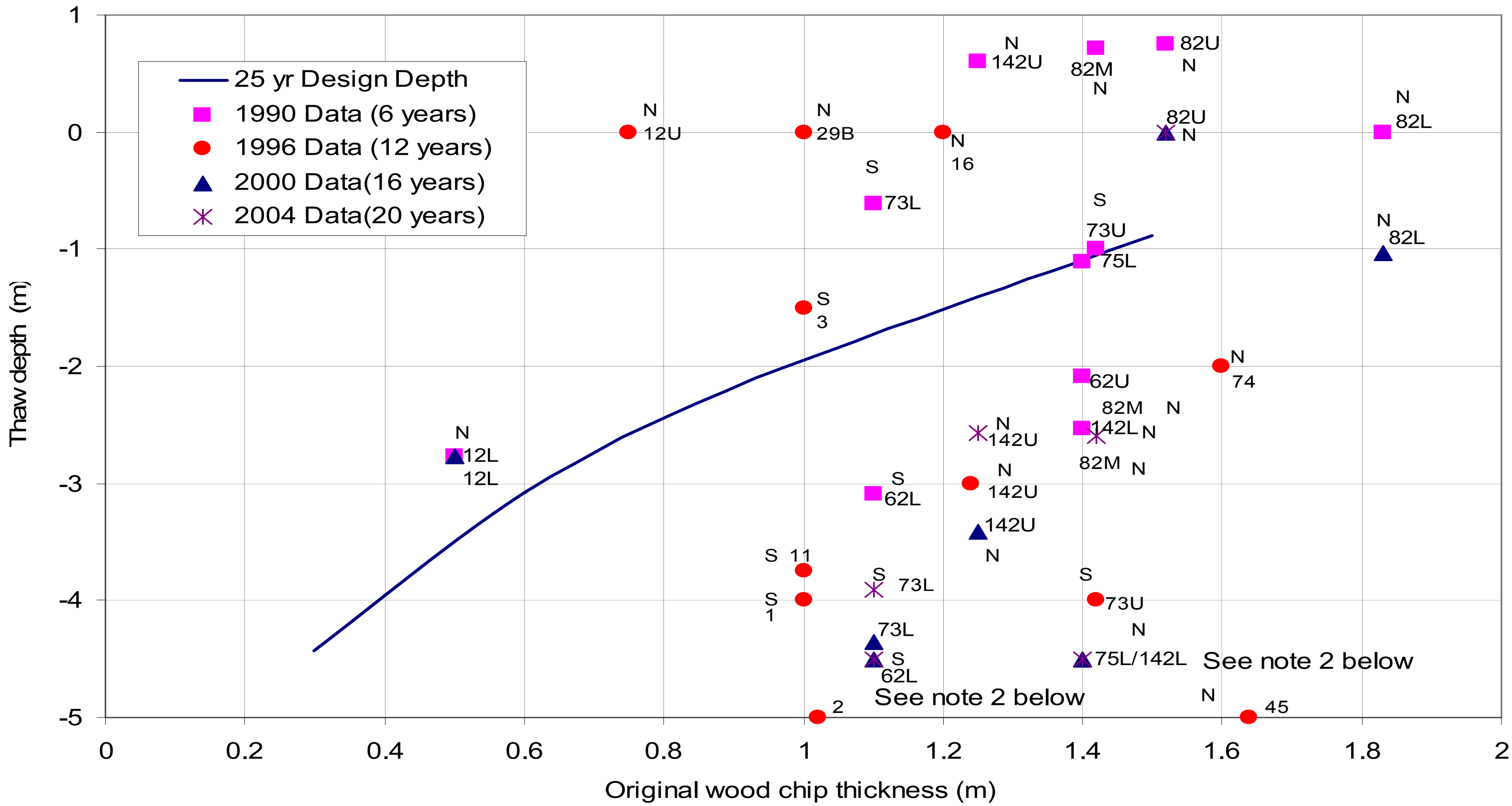


Figure 5-26: Thaw depth versus wood chip thickness for insulated slopes.

Notes 1. N and S refer to north facing and south facing slopes, respectively. L and U refer to upper and lower portions of the slope, respectively.

2. Thaw depth at 75L, 142L, and 62L exceeded the depth of the last thermistor bead.
3. Years shown in legend (ex. 6 years) denote years since ROW clearing and woodchip application, not start of operation.
4. Readings for 2001 and 2003 were used for thermistor 85T10 at slope 142. Readings were not taken for 2002 and 2004 at this location.

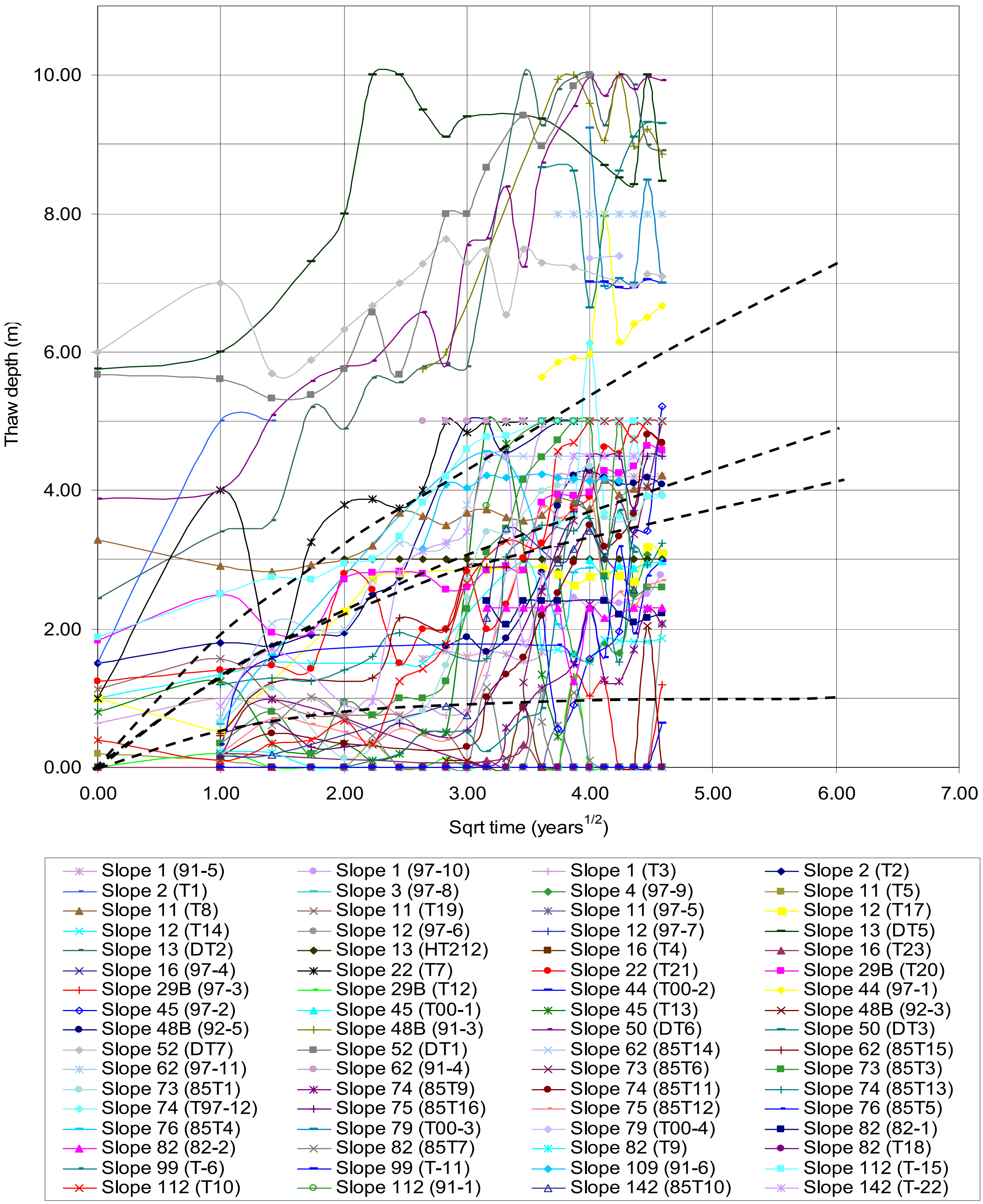


Figure 5-27: Measured thaw depths on all slopes (insulated and non-insulated). The dashed lines represent the thaw depth limits shown on Figure 4-3. The time scale commences in 1984.

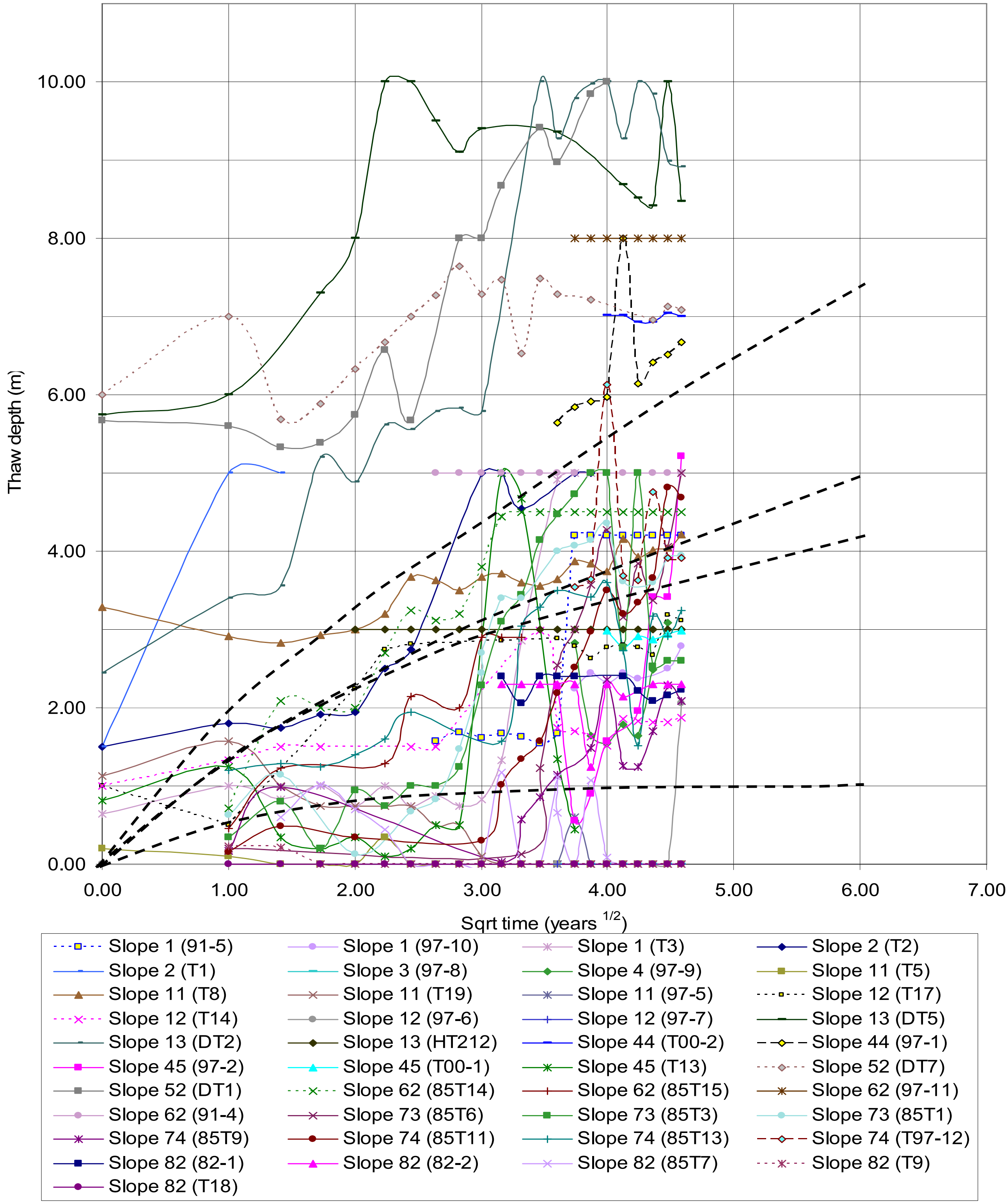


Figure 5-28: Measured thaw depths on slopes subject to pre-clearing of the right-of-way.

The dashed lines represent the thaw depth limits shown on Figure 4-3. The time scale commences in 1984.

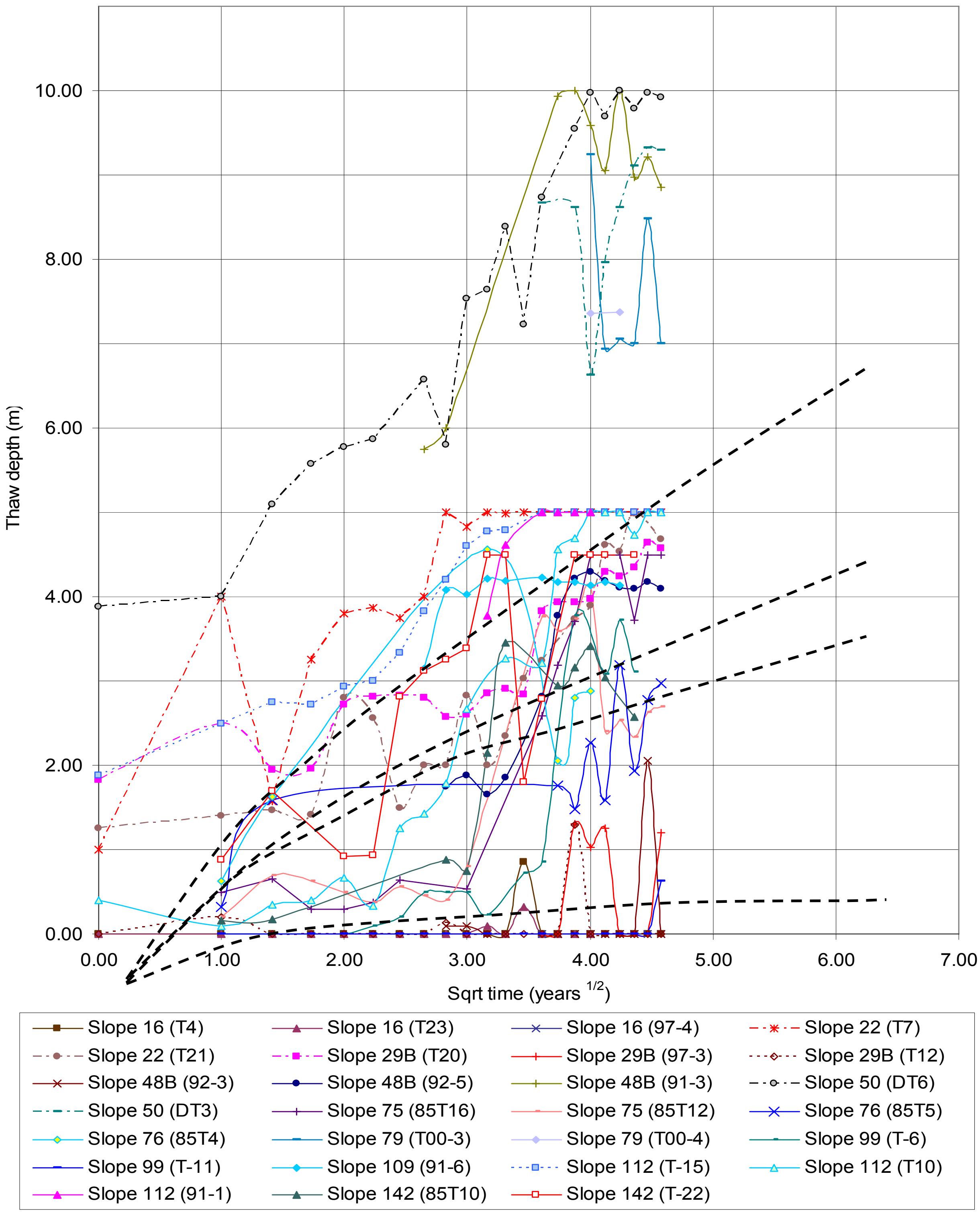


Figure 5-29: Measured thaw depths on slopes not subject to pre-clearing of the right-of-way.
The dashed lines represent the thaw depth limits shown on Figure 4-3. The time scale commences in 1984.

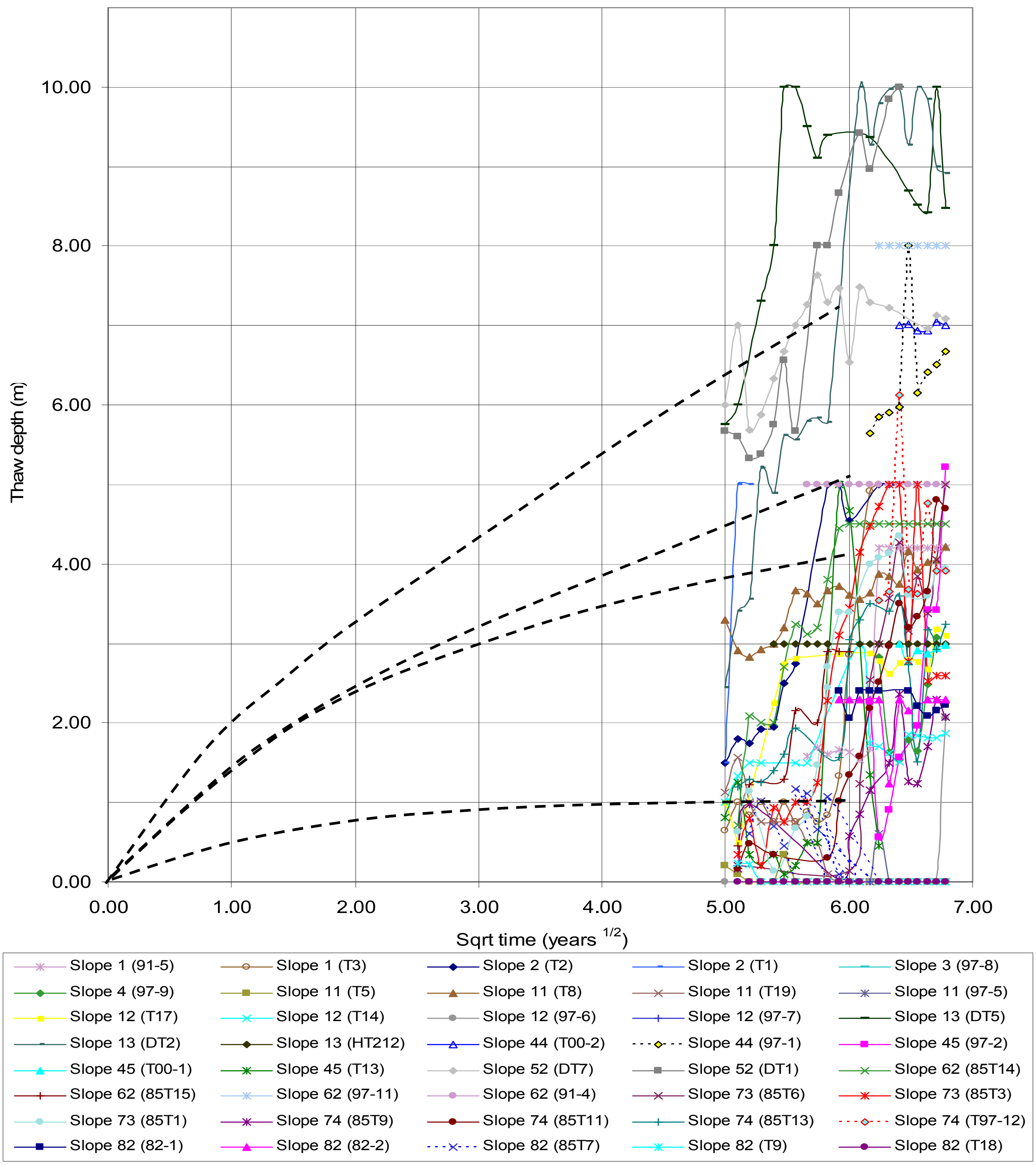


Figure 5-30: Measured thaw depths on slopes subject to pre-clearing of the right-of-way. The time scale has been adjusted to account for the 25 years that the sites were cleared prior to pipeline construction; the time scale begins in 1960. The dashed lines represent the thaw depth limits shown on Figure 4-3.

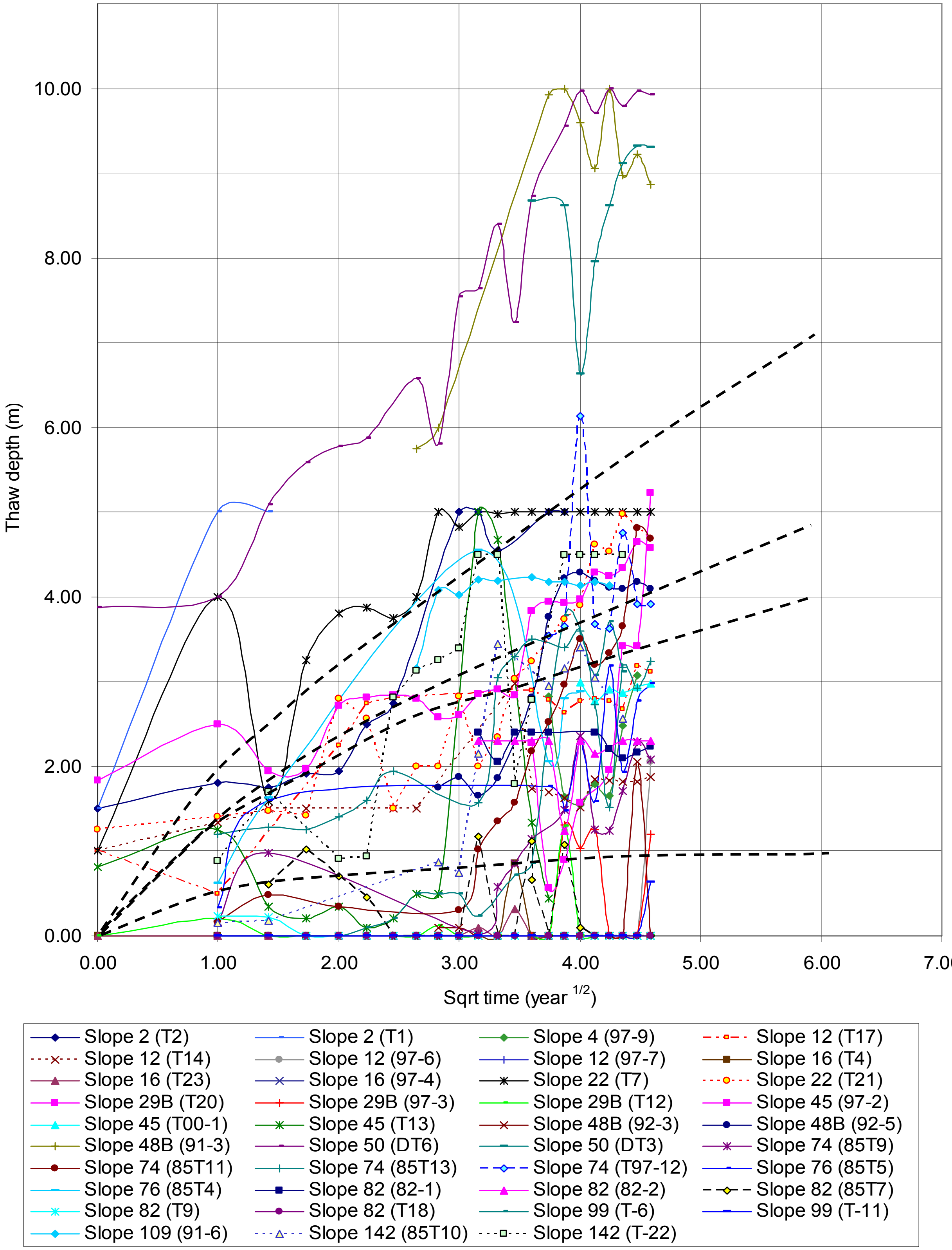


Figure 5-31: Measured thaw depths on north facing slopes.
The dashed lines represent the thaw depth limits shown on Figure 4-3.

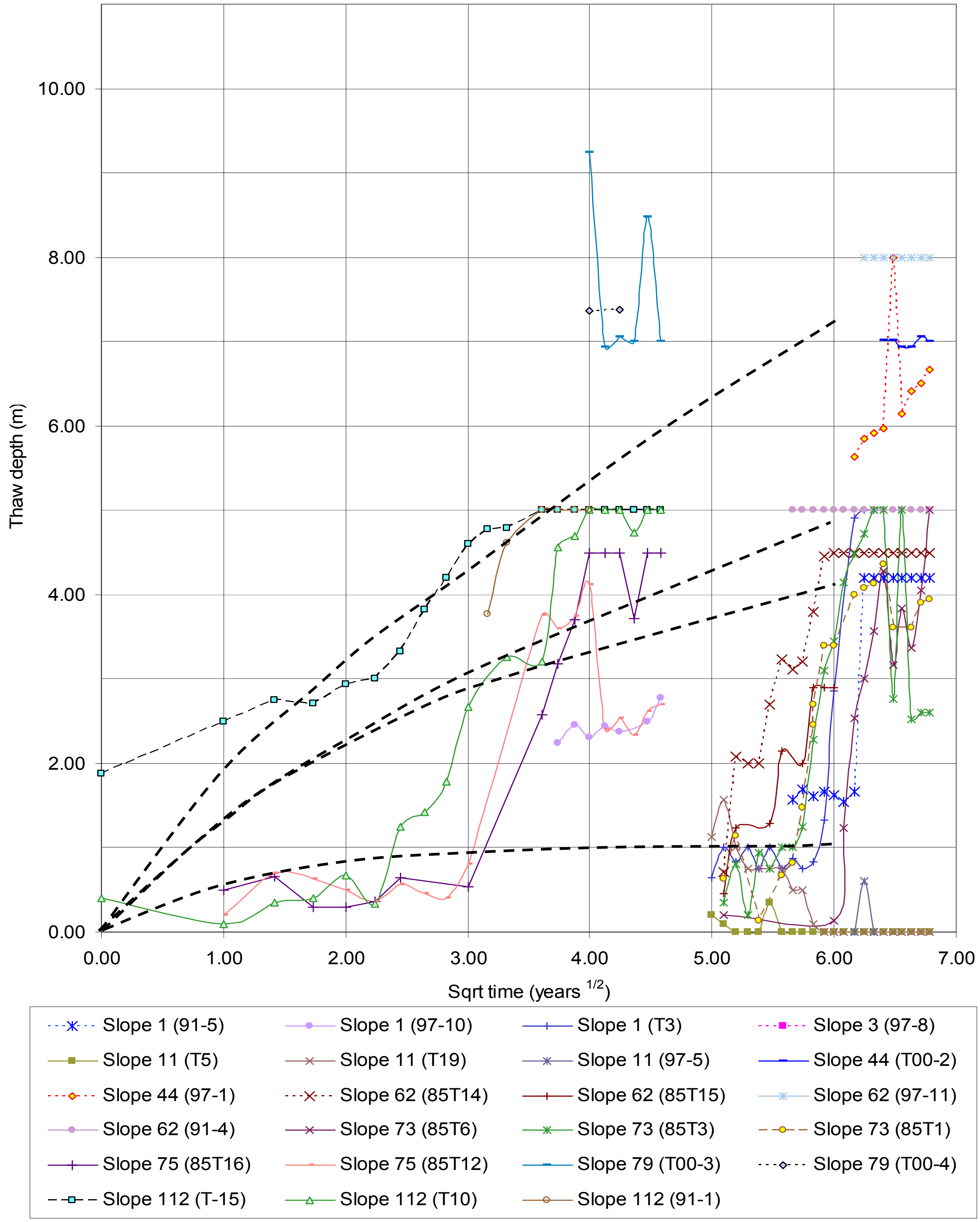


Figure 5-32: Measured thaw depths on south facing slopes.
The dashed lines represent the thaw depth limits shown on Figure 4-3.

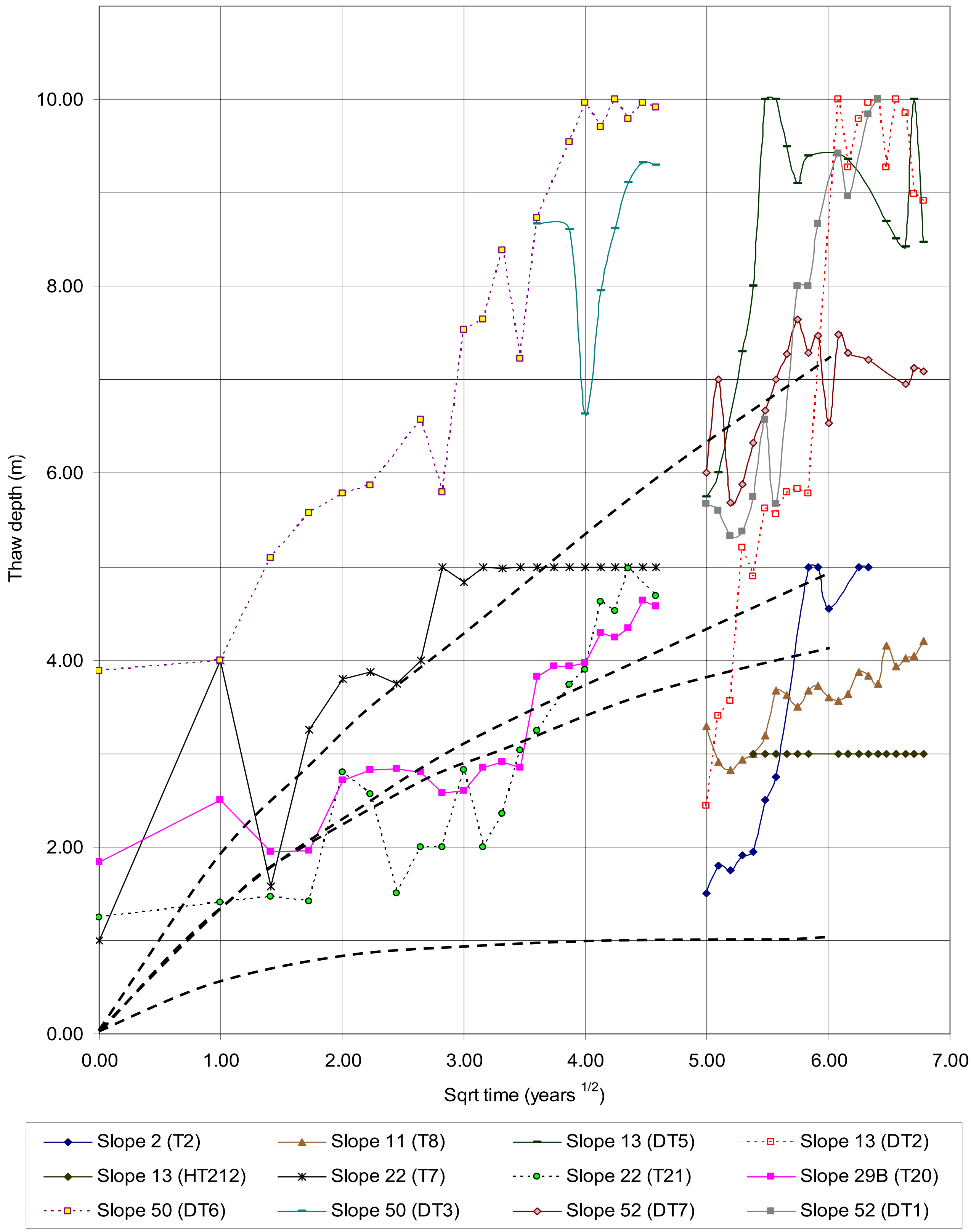


Figure 5-33: Measured thaw depths on all sites with no wood chip insulation.
The dashed lines represent the thaw depth limits shown on Figure 4-3.

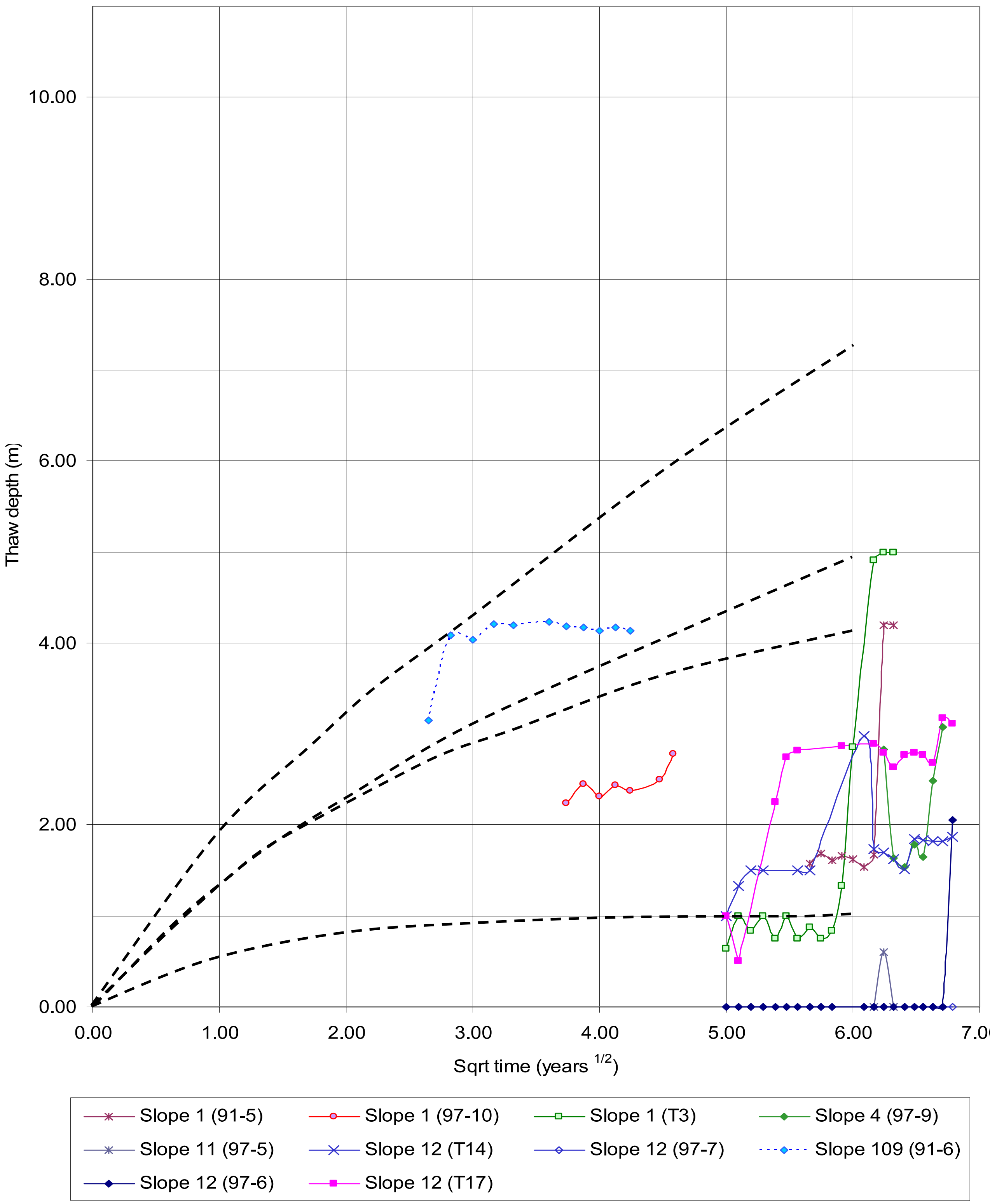


Figure 5-34: Measured thaw depths on all sites with 0.25 m to 0.99 m wood chip insulation.
The dashed lines represent the thaw depth limits shown on Figure 4-3.

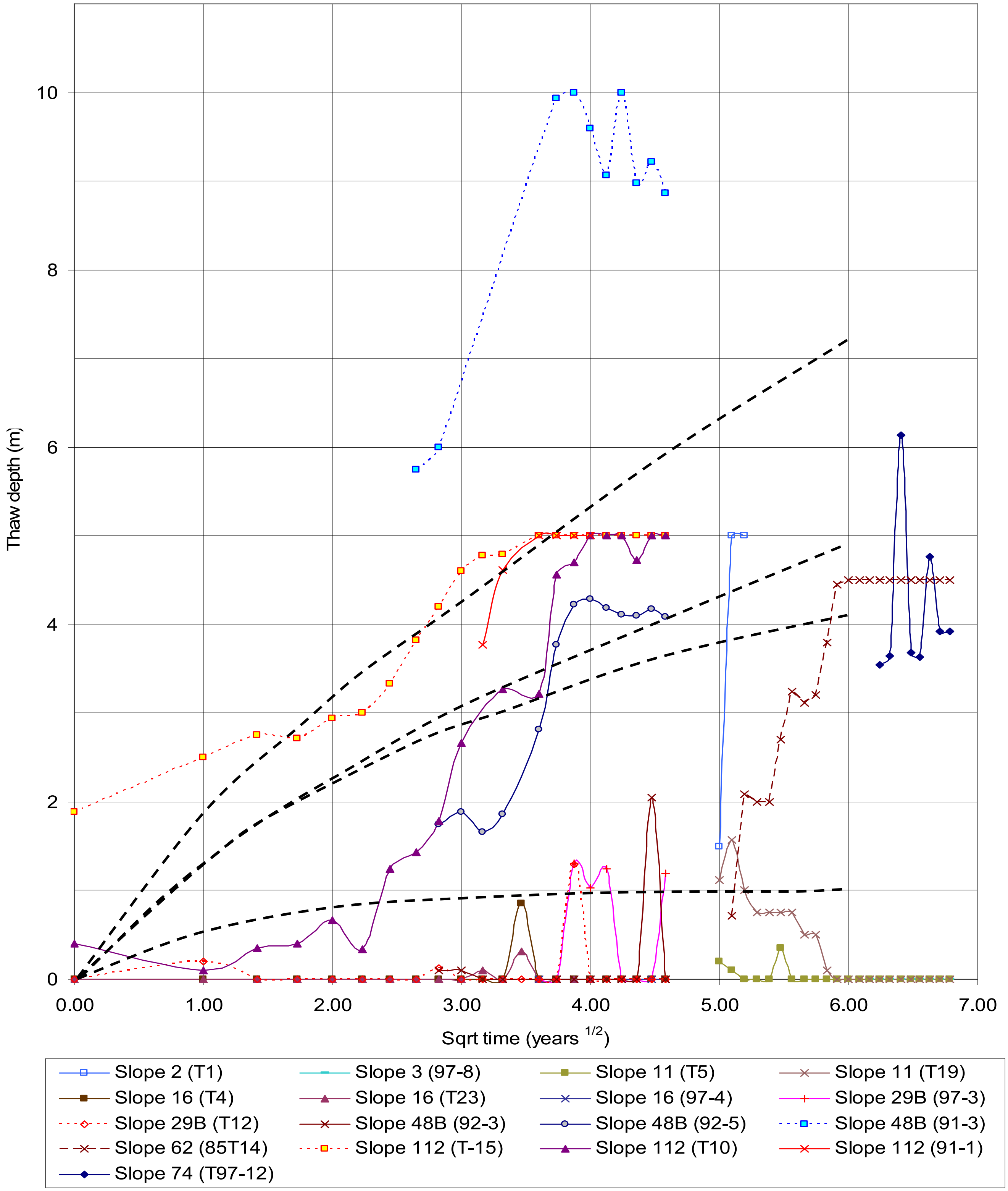


Figure 5-35: Measured thaw depths on all sites with 1.0 m to 1.19 m wood chip insulation.

The dashed lines represent the thaw depth limits shown on Figure 4-3.

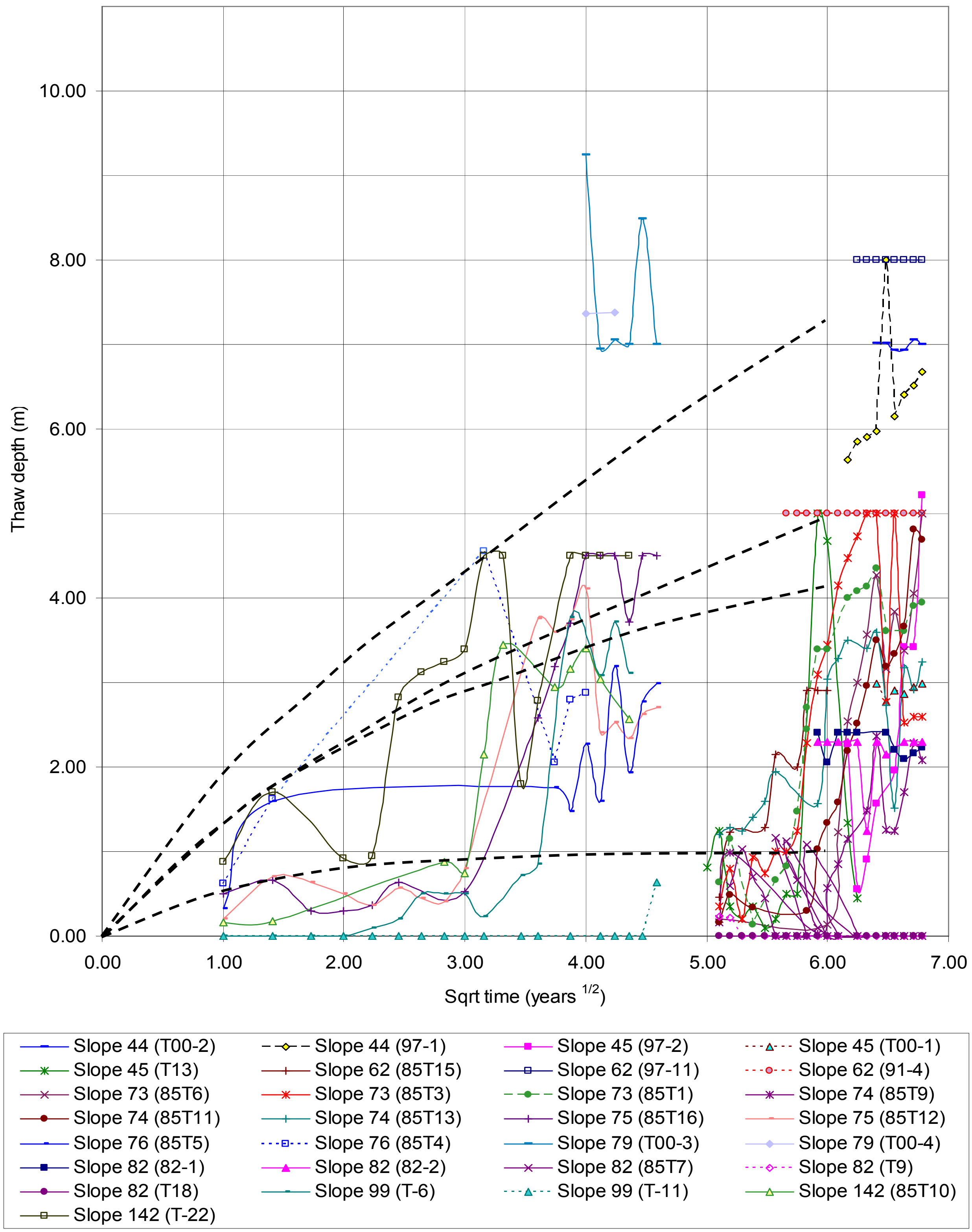


Figure 5-36: Measured thaw depths on all sites with more than 1.2 m wood chip insulation.

The dashed lines represent the thaw depth limits shown on Figure 4-3.

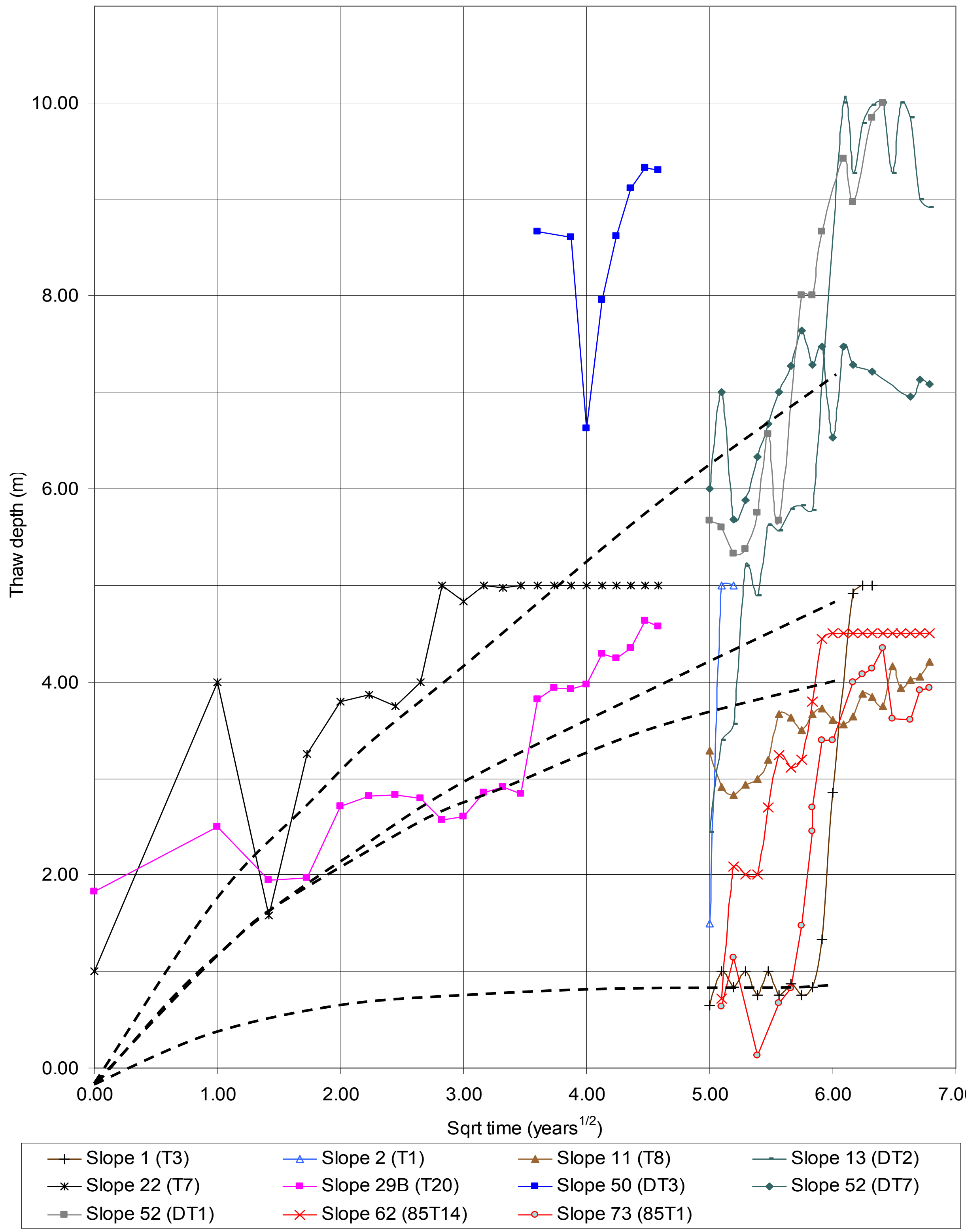


Figure 5-37: Measured thaw depths on all sites with initial ground temperatures warmer than -1 °C at 5 m depth.
The dashed lines represent the thaw depth limits shown on Figure 4-3.

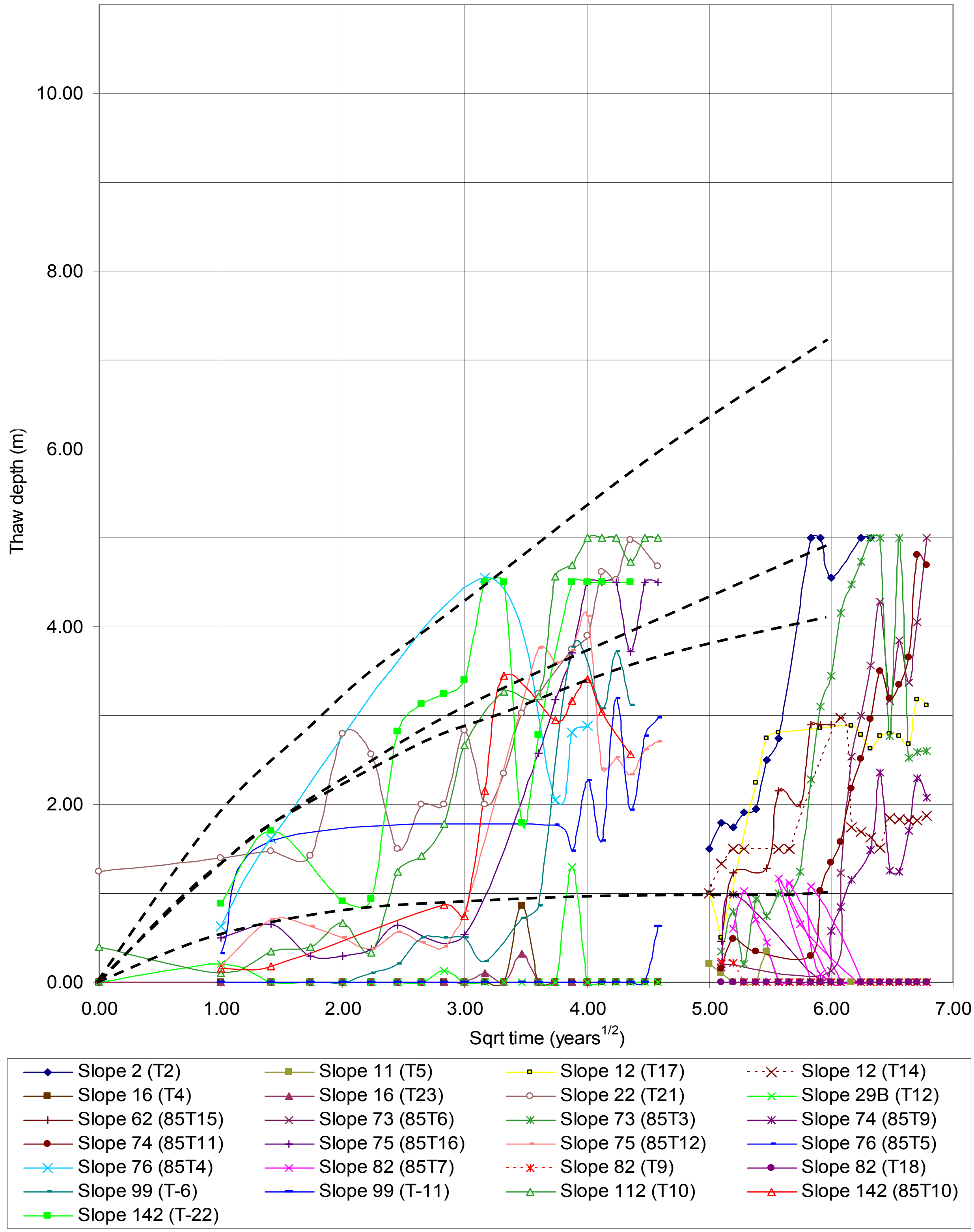


Figure 5-38: Measured thaw depths on all sites with initial ground temperatures colder than -1 °C at 5 m depth.
The dashed lines represent the thaw depth limits shown on Figure 4-3.

5.6.3 Pore Water Pressure Behaviour

The importance of the thaw on any particular slope is not so much the depth of thaw but rather if the thawing is such that ice is melted and converted to water. The release of water has a significant influence on the stability of the slope. For slopes where the released water can not rapidly drain away, soil pore water pressures will increase. For thawing that occurs rapidly in an ice-rich slope, the release of water could be sufficient to destabilize the slope. Equation 4-1 through Equation 4-4 details the influence of thaw rate and thaw depth on the development of pore water pressures. The important stability parameter relative to pore water pressures is the pore pressure ratio, "m," defined in Equation 4-4 (Hanna and McRoberts, 1988). When the groundwater table is coincident with the ground surface, the pore pressure ratio, m, is 1.

For analysis of soil pore water data from the instrumented slopes it is considered that the pore pressure ratio (m) is more illustrative than the excess pore water pressure ratio (h_e/d). In many cases the data suggests that the slopes did not generate excess pore water pressures (where $m > 1$) during thawing. A graph using h_e/d rather than m would have fewer data points, missing all those readings representing hydrostatic conditions. Figure 5-39, Figure 5-40 and Figure 5-41 present the pore water pressure response with time for the available data (Oswell, Skibinsky and Radmard, 2007).

The reduction of the data was difficult to complete for several reasons. First, it is necessary to have a piezometer and thermistor cable in close proximity to each other so that the pore water pressure and thaw depth are comparable. Second, in many cases the thaw depth was less than the piezometer tip, and the porewater pressures were not representative of the actual conditions. Third, in other cases, the depth of thawing had exceeded the depth of the lowest thermistor bead. Hence any calculation of "m" could be artificially high if only the lowest thermistor bead was used, rather than the true thaw depth. For these reasons, the available useful data is limited. The range of the pore water pressure ratio shown on the figures has been limited to 2.0; there are a number of higher values for several slopes, but these data are relatively few.

Figure 5-39 presents all data, irrespective of soil type and other factors. The majority of the data lie below unity, which indicates a condition of no excess porewater pressure. For those slopes with excess pore water pressures early in the operations period, there is an apparent general decrease in the excess pore water pressure parameter in the late 1990s through the 2000s. Conversely, for those sites with low pore water pressure conditions, there appears to be a general increase in groundwater levels. That is, there appears to be a general long-term trend towards hydrostatic pore water conditions (groundwater level coincident with the ground surface). In terms of conditions of excess porewater pressure ($m > 1$), the majority of these data are between 1.0 and 1.4, which is in the same order of magnitude predicted during design for ice-rich clay soils. Also shown is the design prediction for long-term pore water pressure dissipation, which assumes the slopes reach a hydrostatic condition after 25 years.

The data is differentiated by soil ice content in Figure 5-40 and Figure 5-41. Figure 5-40 present data for ice-rich soils (clays and tills) and Figure 5-41 present data for ice-poor soils (tills).

Figure 5-40 shows that in the late 1990s the maximum recorded pore water pressure parameter decreased toward unity (groundwater table at the ground surface). This trend could mean that the slopes were now draining the excess ice that was present shortly after construction. With this drainage, the stability of the slopes should increase proportionally. Also shown on the figure is the design curve for long-term pore water pressure dissipation for ice-rich clay soils. Not all the data on this figure is for this soil type. A number of sites represent ice-rich till soils, which have a design dissipation curve slightly lower than the design curve shown.

Figure 5-41 has a limited data set. Firm conclusions from the data would be tenuous. High excess pore water pressures are observed at one slope through the 1990s and 2000s. For an ice-poor slope, the values on Slope 12 may be anomalous. For the other two slopes, pore water pressure parameters less than unity are observed, which would be expected; these two slopes also experience a generally declining trend with time. The design curve for pore water pressure dissipation provided previously is not shown on this slope because it does not apply to ice-poor slopes.

Linear regression lines have been determined for each of the data sets. A linear regression fit is likely a poor choice for modeling the pore water pressure behaviour with time; the general trend of excess pore water pressure dissipation from thaw consolidation theory is closer to exponential decay. Nevertheless, a linear regression fit shows the overall general trend of the data with time, which is sufficient for the current purposes. Figure 5-42 presents the curves for ice-rich sites. The regression curves show a converging trend towards hydrostatic conditions (or slightly below hydrostatic) in the long-term. The general trend for the data is similar to the design curve for dissipation of pore water pressures.

The trend towards hydrostatic conditions at these sites may be result of the following. When the slopes and the upland area beyond the crest of the slope were frozen, there was no groundwater table, except perhaps within the seasonal shallow active layer zone. As a result of construction disturbance the ground has experienced thawing and the active layer is now much deeper or permafrost has completed degraded. As a result groundwater flow can develop within the thawed zone and is not longer controlled by the shallow permafrost table. With time, a groundwater regime within these disturbed areas may develop similar to normal unfrozen terrain.

Figure 5-43 presents the three linear regression curves for ice-poor slopes. Because of the limited data it is difficult to assess long-term trends, except that lower pore water pressures with time may be developing.

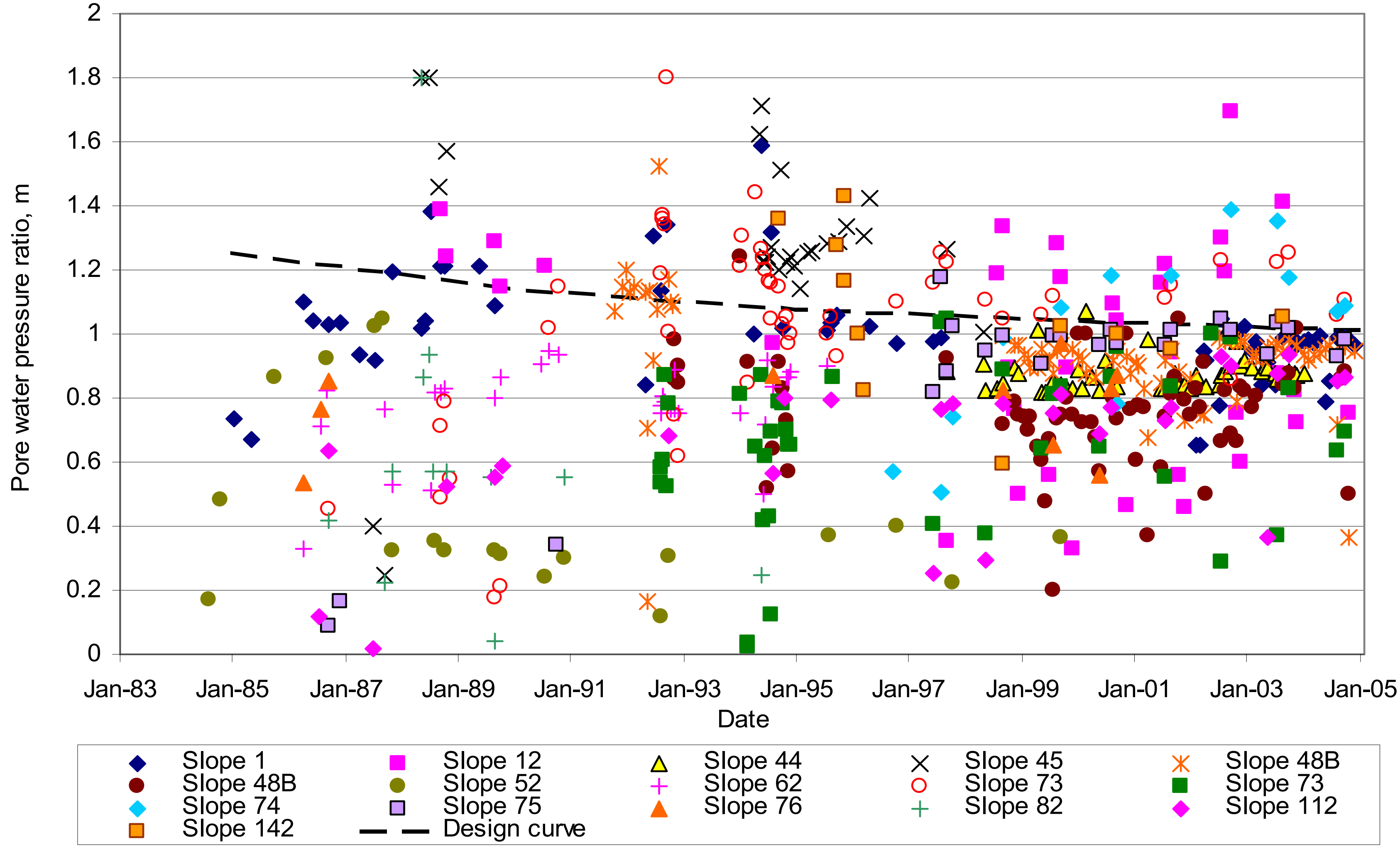


Figure 5-39: Pore water pressure parameter (m) for all monitored slopes.
Note: Design curve is data from Table 4-4.

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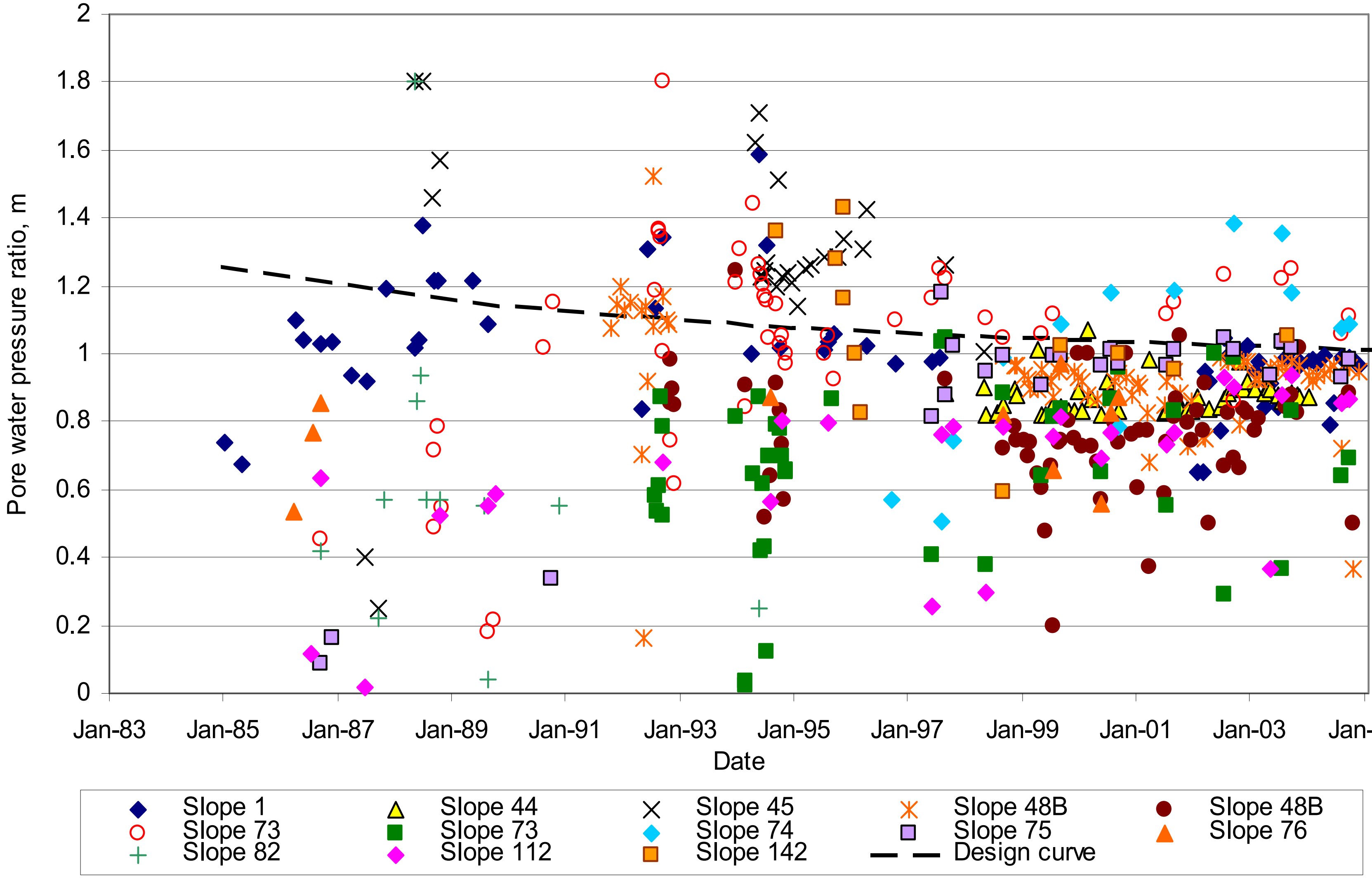


Figure 5-40: Pore water pressure parameter (m) for ice-rich slopes (ice-rich clay and ice-rich till).
Note: Design curve is data from Table 4-4.

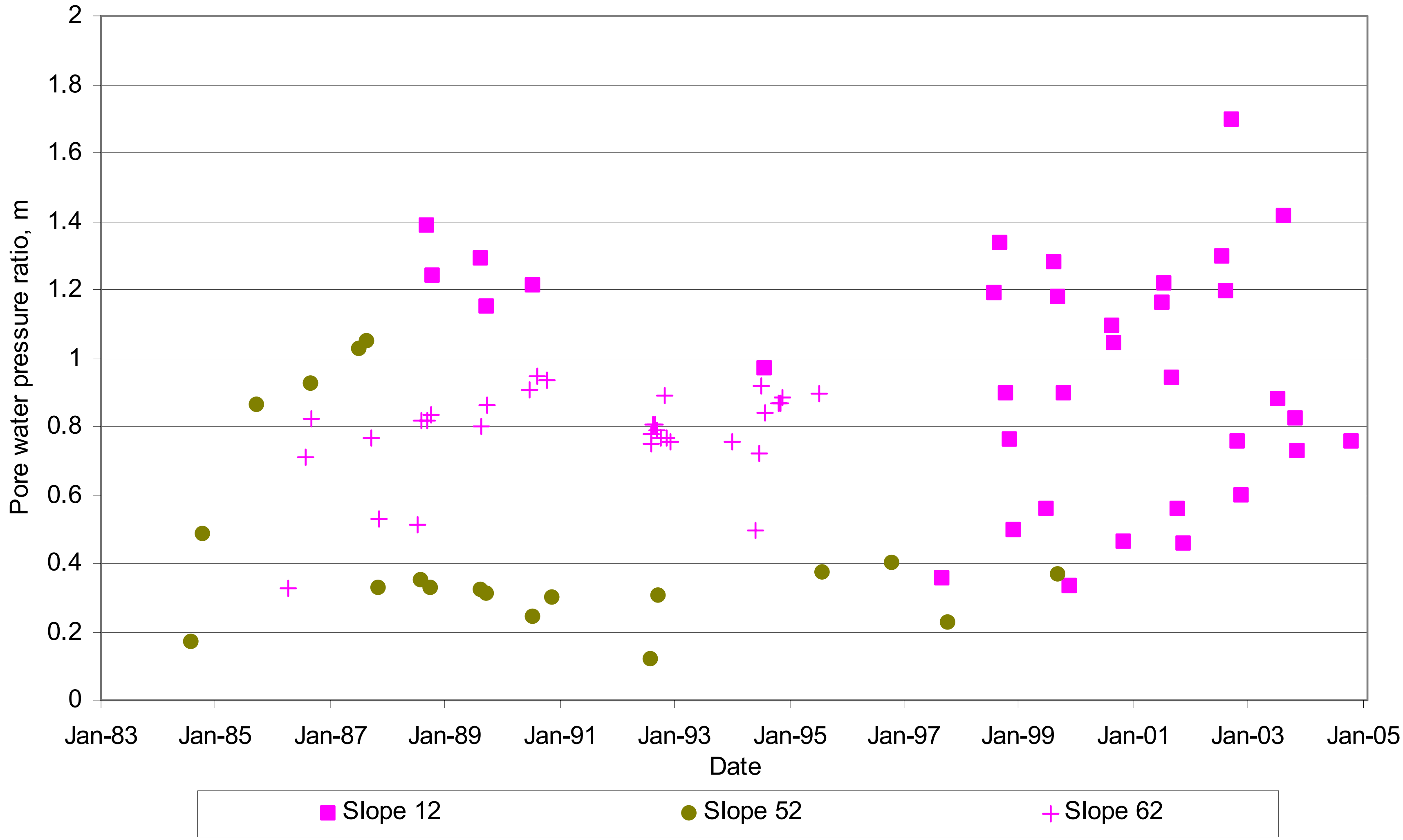


Figure 5-41: Pore water pressure parameter (m) for ice-poor slopes.

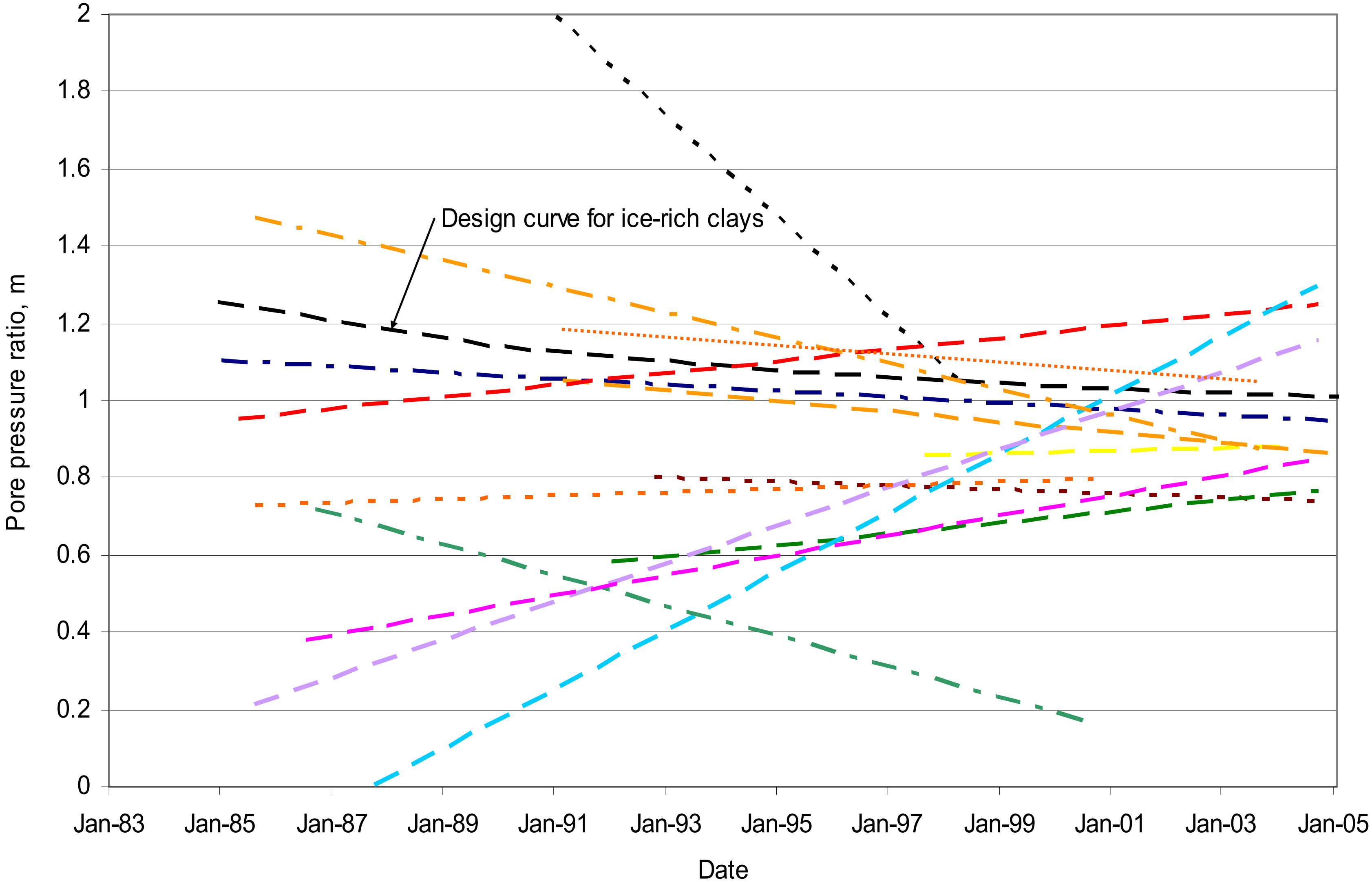


Figure 5-42: Linear regression relations of pore water pressure ratio (m) for ice-rich slopes.
Note: the regression relations are colour coded in the same colours as the data presented in Figure 5-40.

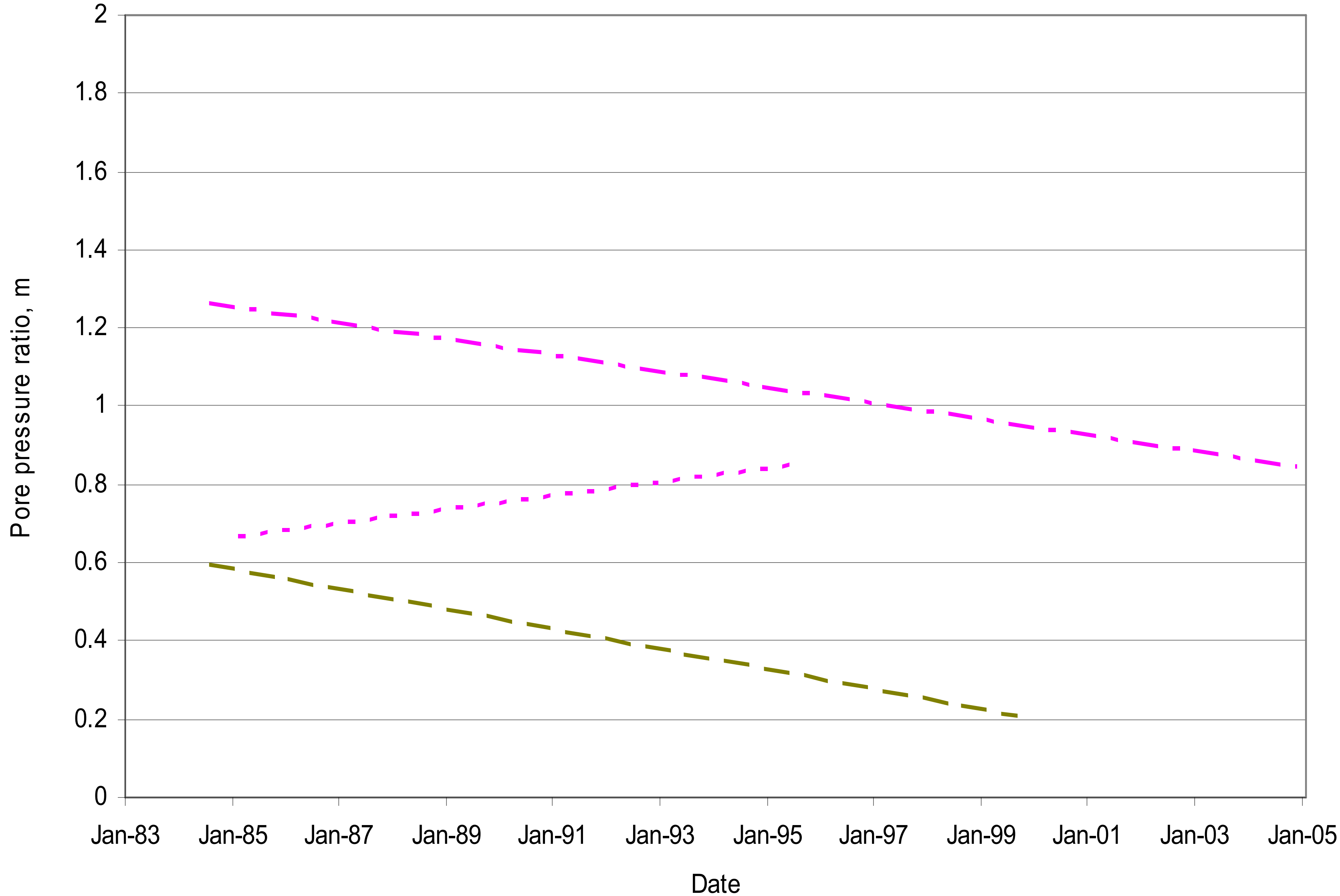


Figure 5-43: Linear regression relations of pore water pressure ratio (m) for ice-poor slopes.
Note: the regression relations are colour coded in the same colours as the data presented in Figure 5-41.

5.6.4 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 pore water pressures measured throughout the year, but particularly in the fall, at the point of maximum thaw were important. To assess the stability of the slopes, the thaw and pore water pressure data was used in the original slopes design formulation, subsequently referred to as the Design Base Method (DBM) (See Equation 4-5, Section 4.6.3). 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 apparent excess pore water pressures and the shape of the thaw bulb, such that the restraining side shear effect was reduced.

In 1996, 17 slopes out of 55 insulated slopes were assessed. Following that review, ten 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 pore water pressure data. New instrumentation was installed at some of these slopes in February 1997 to address some of these concerns. Many of these slopes remain on the “watch list” but to a great extent, these slopes now serve as bellwethers for the entire set of slopes along the route. Should unexpected or undesirable behaviour develop at any of these slopes, the entire slope inventory can be reviewed.

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 probabilistic methods 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.

By the mid 2000s, approximately 25 slopes were routinely assessed for stability. Of these slopes only approximately four slopes were known to be actually experiencing some movement, while the others were subject to greater analysis due to observed pore water pressures and other factors. The factor of safety remains greater than 1.5 for the great majority of the slopes, and only several slopes have factors of safety that are of concern, when assessed in light of the measured pore water pressures. In some cases there is conflicted data, with some instruments recording excess pore water pressures while other instruments recording low pore water pressures.

Several specific slopes are subject to more detailed discussion below.

5.6.5 Slopes Creep

5.6.5.1 General

In the period of pipeline operations from 1985 to 1997 the stability of slopes focussed on limiting equilibrium stability issues. Since 1997 a number of creep-like movements have been identified along the right-of-way. The movements have been identified by the GEOPIG, which monitors (amongst other parameters) the internal diameter of the pipeline and is capable of detecting very small changes in the internal diameter and ovality of the pipe. From this information further investigations and the installation of geotechnical instrumentation such as slope indicators has been undertaken. To 2006 four slopes have been identified as experiencing creep-like movements. These slopes are: Slope 44 (KP 133), Slope 45 (KP 133), Slope 84 (KP 311) and Slope 92 (KP 318). Each of these sites is discussed below.

In light of the proximity of two slopes to each other (Slopes 84 and 92), the NRCan/GSC undertook an extensive instrumentation program at an intervening site (Slope 88, KP 313.6) in hopes of documenting initiation of creep and pipe strain. This site is also discussed below.

5.6.5.2 Slopes 44 and 45 at KP 133

Pipe strain changes were first identified from GEOPIG monitoring in the late 1990s. A plot of the GEOPIG data comparing 2006 with 1989 is shown on Figure 5-44. Several points of interest in vertical and horizontal curvature (representative of bending strain) and curvature radius are shown. The comparison clearly shows that the pipe is experiencing some deformation in these areas. The pipeline vertical profile on the north side has also changed between 1989 and 2006. The deformation on Slope 44 at KP 133.720 appears to be within a section of heavy wall pipe. Hence, the initiation of a wrinkle at this location may be delayed. For the deformation on Slope 45 (KP 133.765) there does not appear (as of Fall 2006) any change in the internal diameter that could be reasonably identified as being an incipient wrinkle.

To assess the slope movement mechanisms associated with the strain development, several slope indicators were installed in February 2000; two on Slope 44 (north) and one on Slope 45 (south). Plots of accumulative displacement of the two slopes indicators on Slope 44 are presented on Figure 5-45 and Figure 5-46. Surficial downslope movement of more than 175 mm and 100 mm has been recorded between 2000 and the fall of 2006 for the upper and lower indicators, respectively.

The slope indicator installed on Slope 45 in February 2000 operated until the fall of 2002, before becoming inoperative. In February 2004 two new indicators were installed, one on the cleared right-of-way and one off the "west" side of the right-of-way. Displacement plots for these two instruments are presented on Figure 5-47 and Figure 5-48, respectively. Plots of cumulative displacement at about 5 m depth are shown on Figure 5-45.

All slope indicators clearly indicate the presence of at least one movement zone. For the Slope 44 instruments (Figures 5-45 and 5-46) the movement zones are located at about 7 to 9 m below the top of wood chips. For the Slope 45 instruments (Figures 5-47 and 5-48) a movement zone is tentatively identified at about 9 m in the on-slope instrument and at about 4 m in the off-slope instrument.

Figure 5-49 present the cumulative displacement with time of each of the slope indicator at a depth of 5 m below the top of wood chips. The data shows the on right-of-way movements on both slopes are uniform and the rate has been reasonably constant at about 1.7 mm to 2 mm per month since early 2000. Movement off the cleared right-of-way is also occurring, but at a much lower rate. Monthly movement in the order of 0.7 mm per month has been recorded for several years.

During field survey work at this site in June 2002, a series of nails were placed along the length of the cribs on each side of the creek. The baseline separation of the nails was recorded in June 2002 with a subsequent set of measurements taken in September of each year, during the Fall reconnaissance. The table below (Table 5-3) provides the change in separation of the cribs.

Table 5-3: Separation of Crib Monitoring Points from June 2002.

Date	Set 1 (East side)	Closure Rate per month	Set 2	Closure Rate per month	Set 3	Closure Rate per month	Set 4 (West side)	Closure Rate Per month
September 2002	-0.070 m	23 mm	-0.030 m	10 mm	-0.040 m	13 mm	-0.040 m	13 mm
September 2003	-0.085 m	6 mm	-0.040 m	3 mm	-0.065 m	4 mm	-0.080 m	5 mm
September 2004	-0.140 m	5 mm	-0.060 m	2 mm	-0.095 m	4 mm	-0.120 m	4 mm
September 2005	-0.150 m	3 mm	-0.068 m	2 mm	-0.105 m	1 mm	-0.142 m	2 mm
September 2006	-0.180 m	4 mm	-0.095 m	2 mm	-0.125 m	2 mm	-0.160 m	3 mm

The four years of data show continued closure movement. This is consistent with the slope indicators, and further confirmation that both slopes are creeping towards the common creek. The average closure rate in the past year is about 25 mm/year. This is similar to the movement rate at depth of the slope indicators (recognizing that the crib movement represents movement from two sides and hence should be greater than the individual slope movements).

The implications of the slope movement to the integrity of the pipeline are addressed by the annual GEOPIG surveys. As shown in Figure 5-44, although changes in the pipeline geometry are evident, the changes have not given rise to wrinkles in the pipe.

The causes of the movements are speculative. The lower movement zones are approximately coincident with the depth of the thaw front. But the fact that the two slopes are converging, and that one slope is not buttressing the other implies that the soils below the toe of the slopes must be very soft. Indeed, one could speculate that the small creek at the toe of the slope could be rising, pushed upwards by the converging soil masses on each side. No physical surveys have been conducted to verify this.

5.6.5.3 Pipe Wrinkle Study at KP 318

In the fall of 1997, Enbridge ran the 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). Figure 5-50 presents the GEOPIG data. 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. A winter dig would allow access to the site that would otherwise be impossible during the summer thaw season. The excavation took place in late February 1998. Figure 5-51 shows several photographs of the exposed pipe and wrinkle.

5.6.5.3.1 KP 318 Instrumentation: 1998 - 1999

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 5-52 shows the layout of the monitoring instrumentation).

Full details on the instrumentation and the data are also discussed by Oswell, Hanna, Doblanko and Wilkie (2000).

<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 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 the pipe section adjacent to wrinkle</u>

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

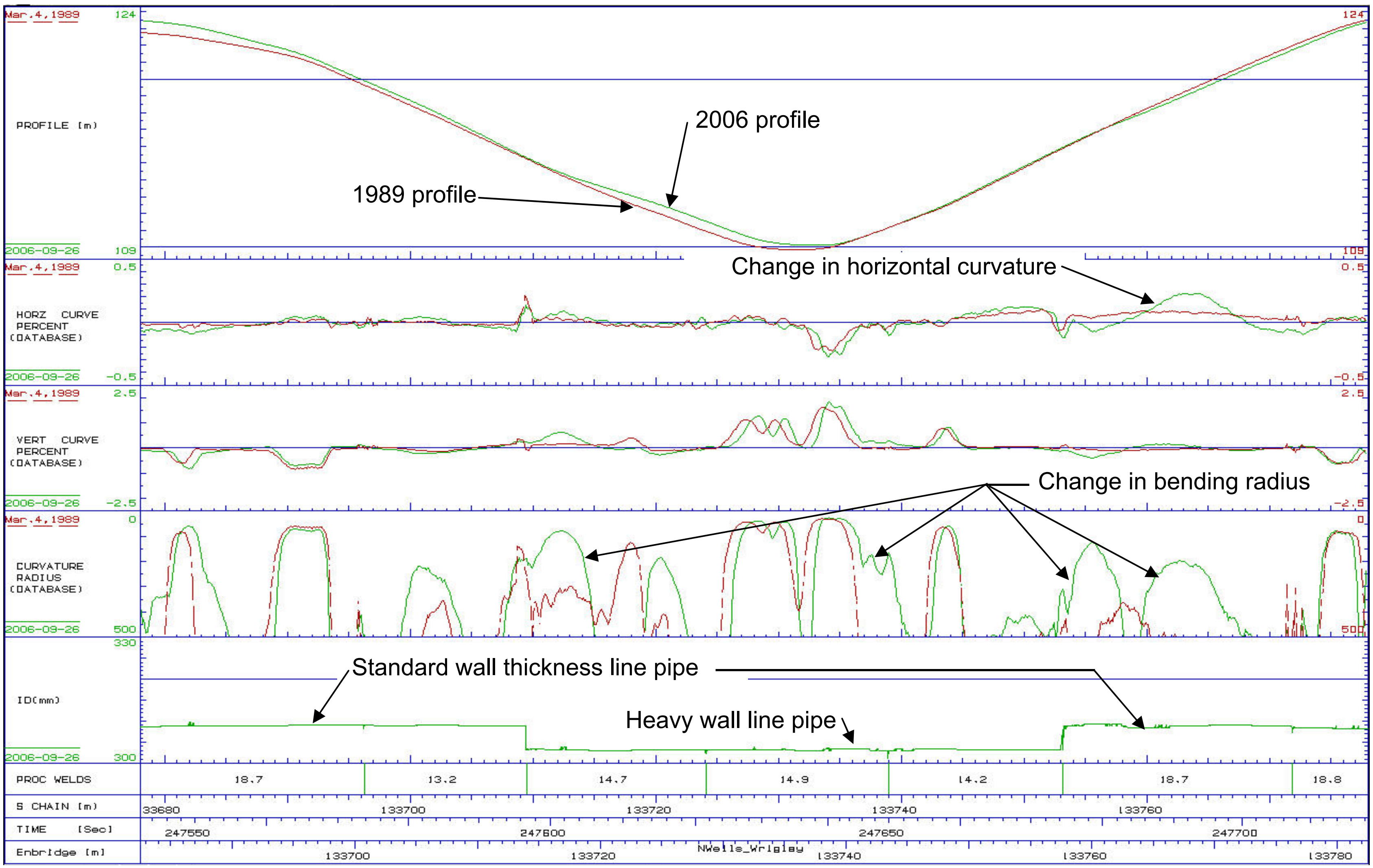
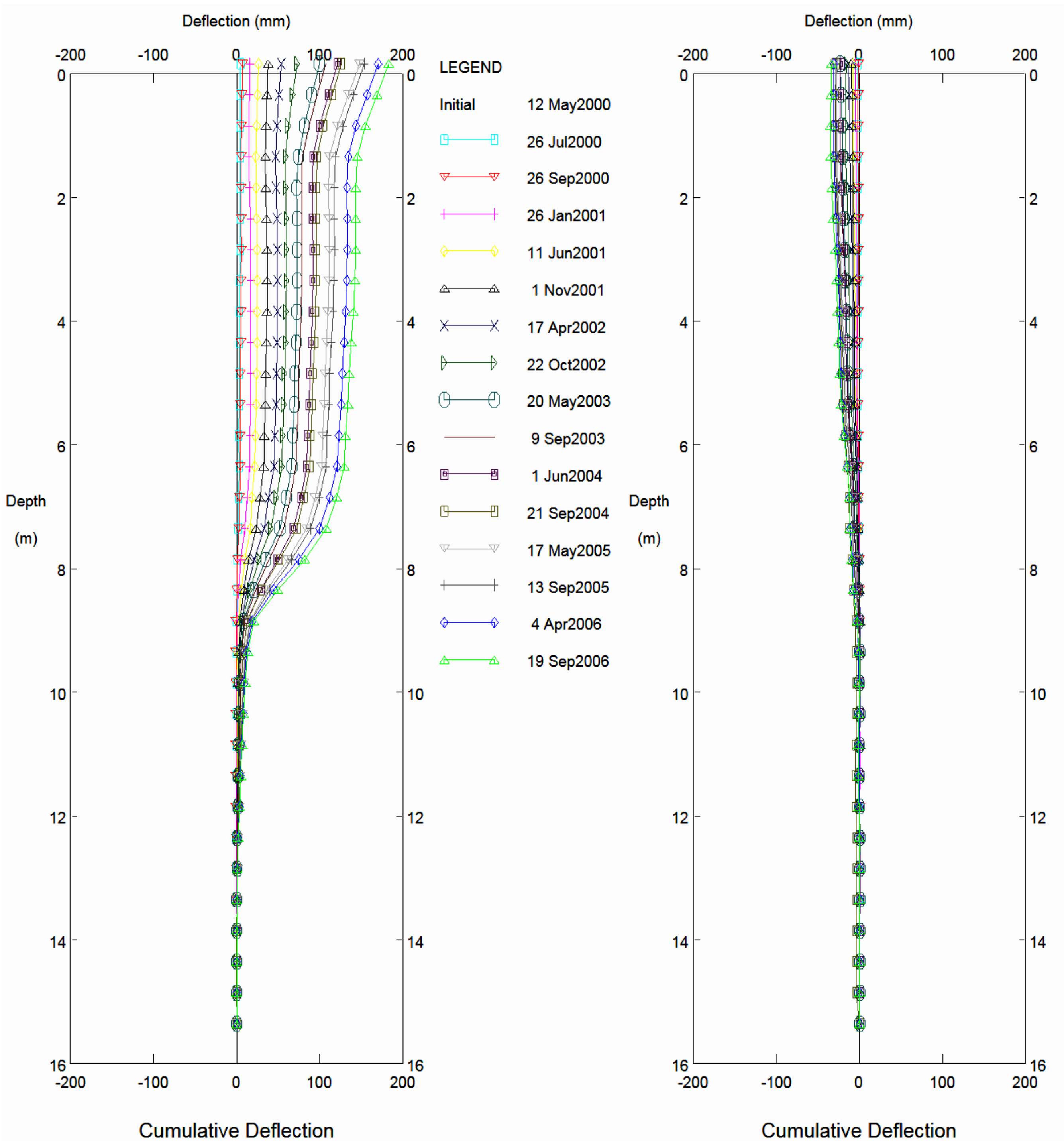


Figure 5-44: GEOPIG data for Slopes 44 and 45 (KP 133), comparing results from 2006 (green) to 1989 (red).
Figure courtesy of Enbridge Pipelines (NW) Inc.



Direction parallel to slope **Direction perpendicular to slope**

Figure 5-45: Slope indicator at Slope 44 (upper).

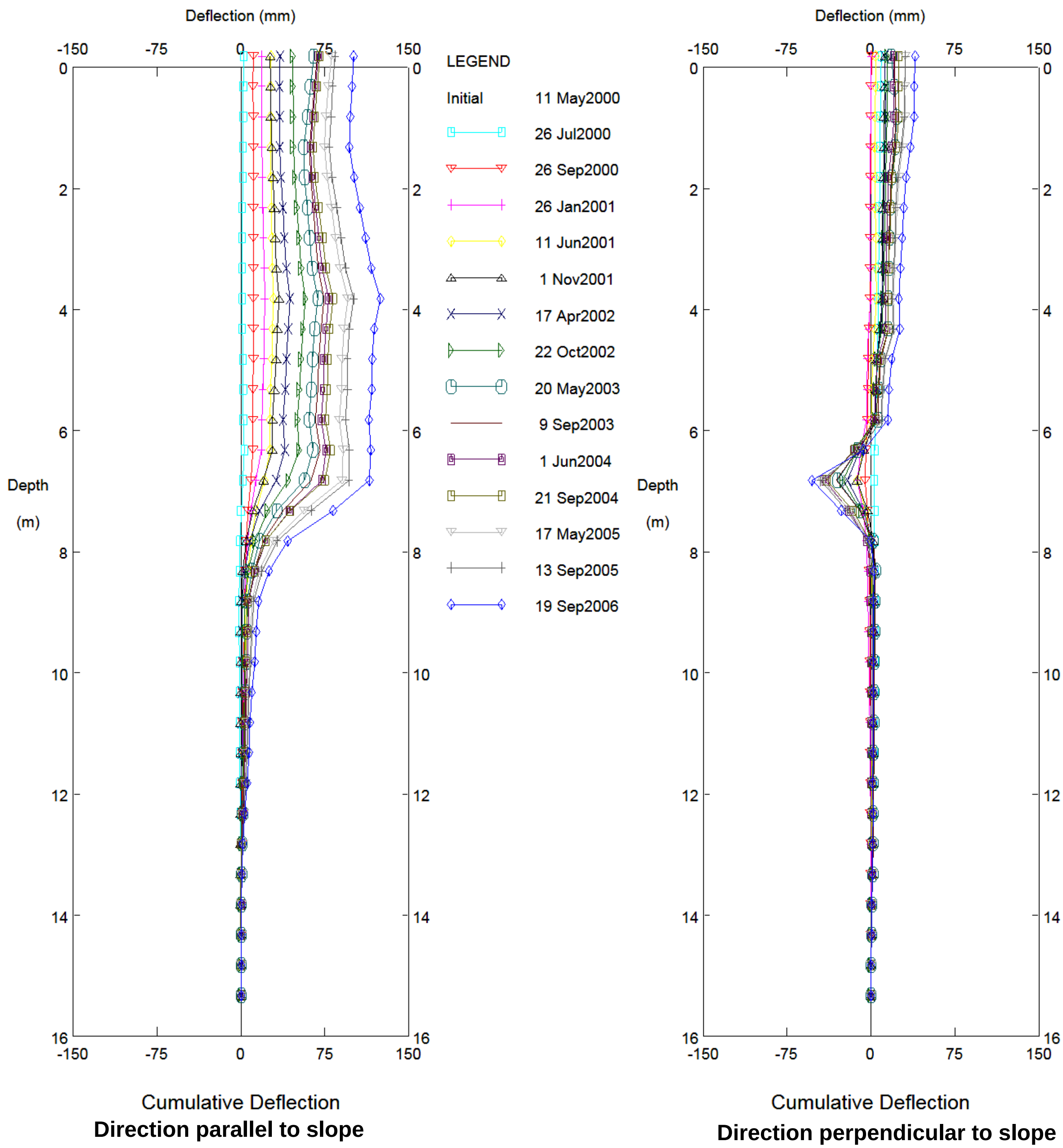


Figure 5-46: Slope indicator at Slope 44 (lower).

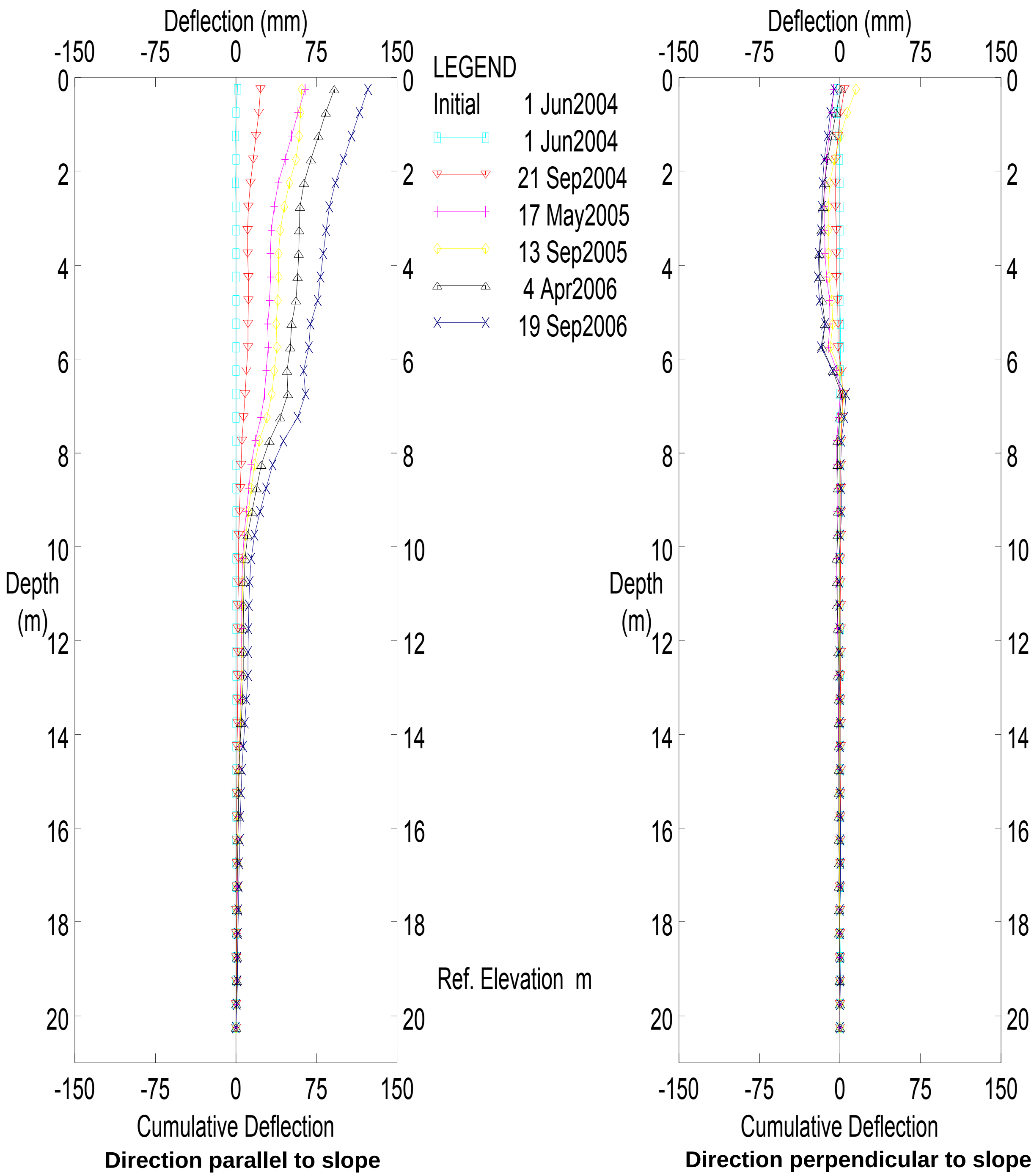


Figure 5-47. Slope indicator at Slope 45 (on-slope).

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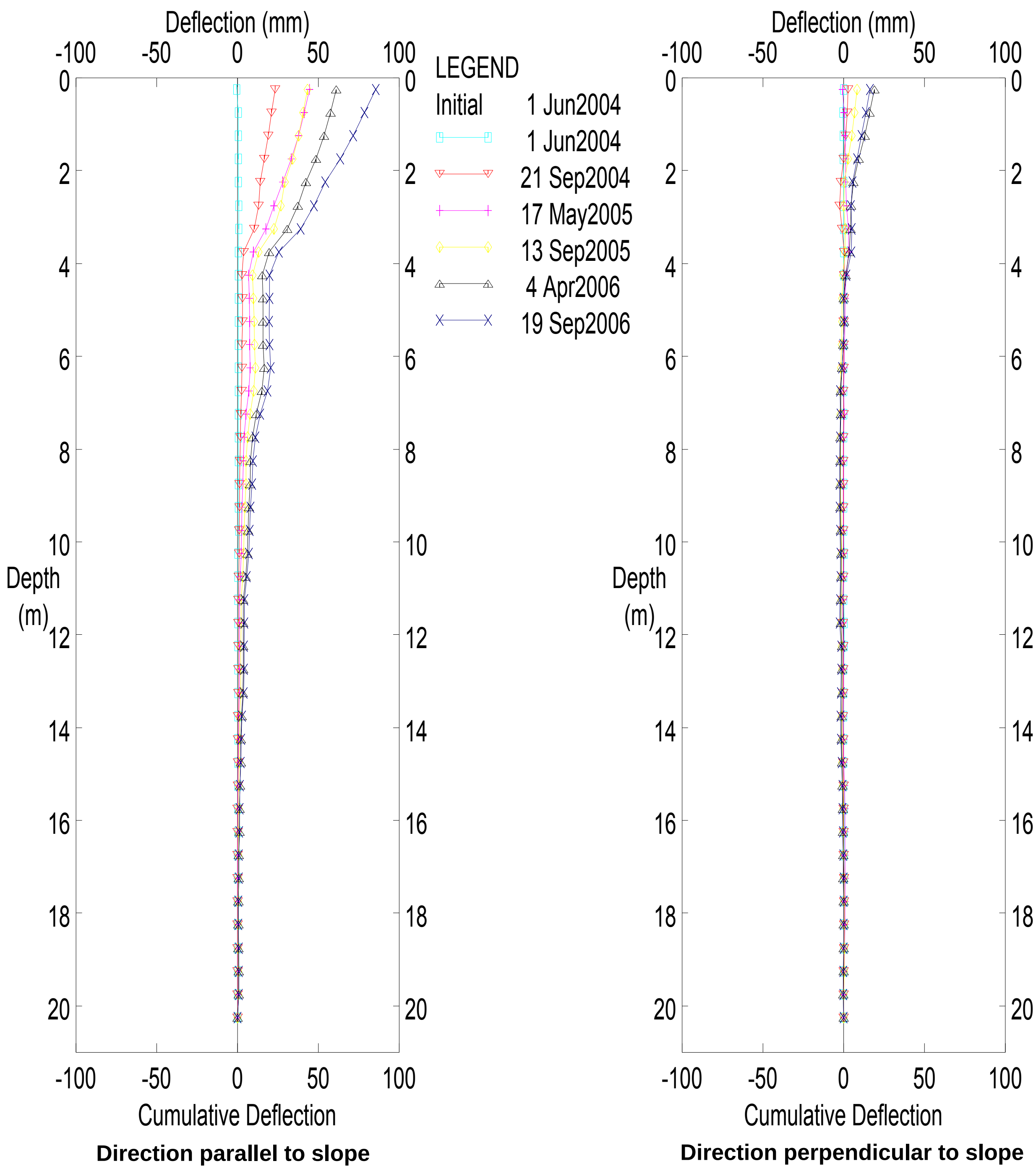


Figure 5-48: Slope indicator at Slope 45 (off-slope).

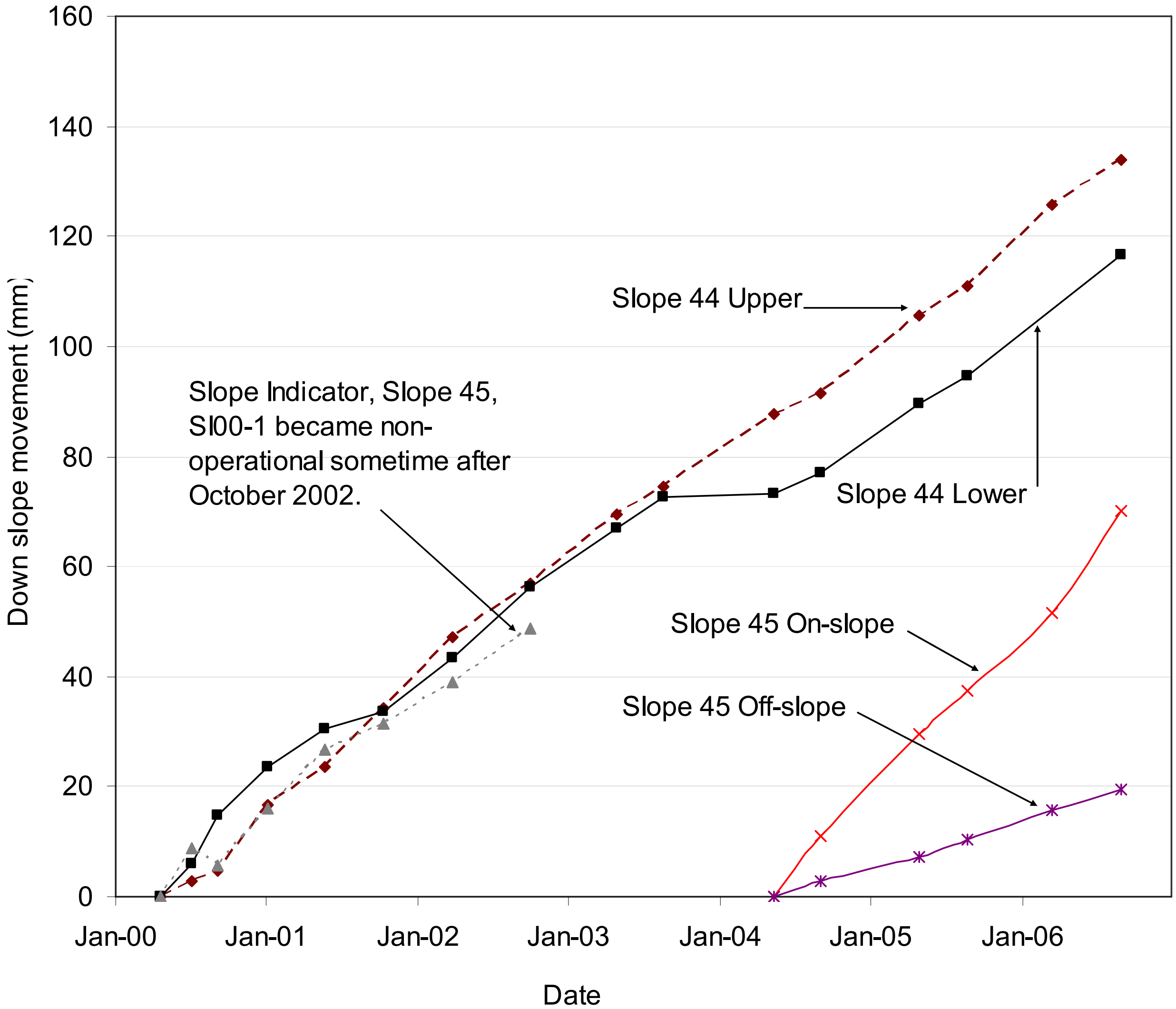


Figure 5-49: Slope movements at Slope 44 and Slope 45 with time at 5 m depth.

In the Surrounding Ground

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 are shown in Figure 5-52. 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.

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. The temperatures for the entire depth to 8 m showed the ground to be marginally frozen. The upper 4 m of soil displayed the seasonal variations in ground temperatures. Marginally frozen conditions were observed from about 4 m depth to about 7 m depth. Below 7 m the ground temperatures appear to be warming. This suggested that within the right-of-way the permafrost may be degrading and was present over a relatively thin thickness.

5.6.5.3.2 Pipeline Cut-out, Replacement

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

Plans were then prepared by Enbridge to replace the section of pipe at KP 318 in February, 1999. The replacement program required 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 replaced after the excavation and cut-out was completed.

In February 1999 the excavation and pipeline replacement work was completed by Enbridge. This work was scheduled to be completed with a 72 hour window, which was a condition dictated by oil storage capacity at Norman Wells. The GEOPIG profile of the pipe was used to calculate the sag bend and over bend angles for the new heavy wall section pipe. The pipe was then pre-bent and hydro-tested prior to being shipped to the site. Details of the replacement work are discussed in detail by Wilkie, Doblanko and Fladager (2000).

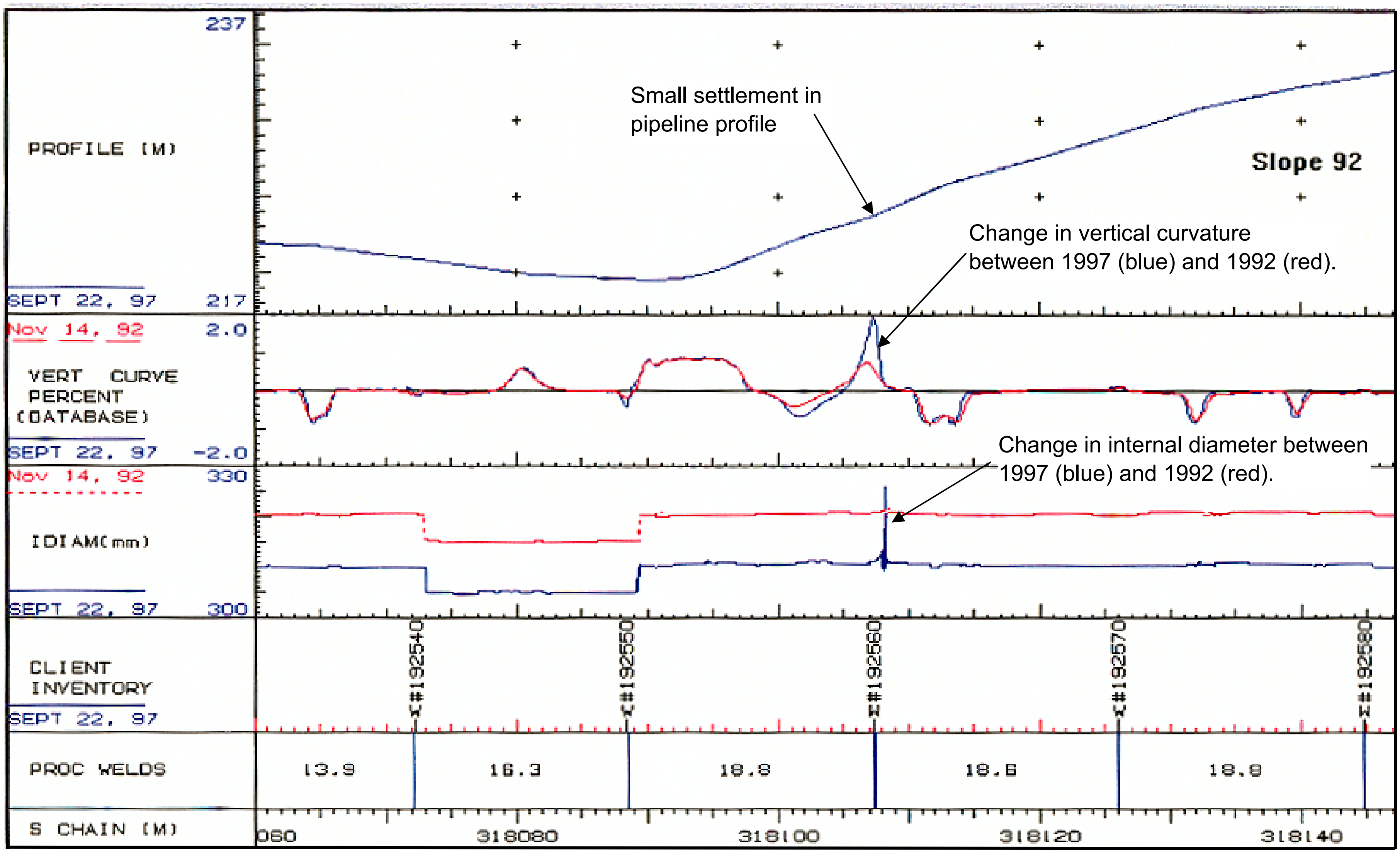
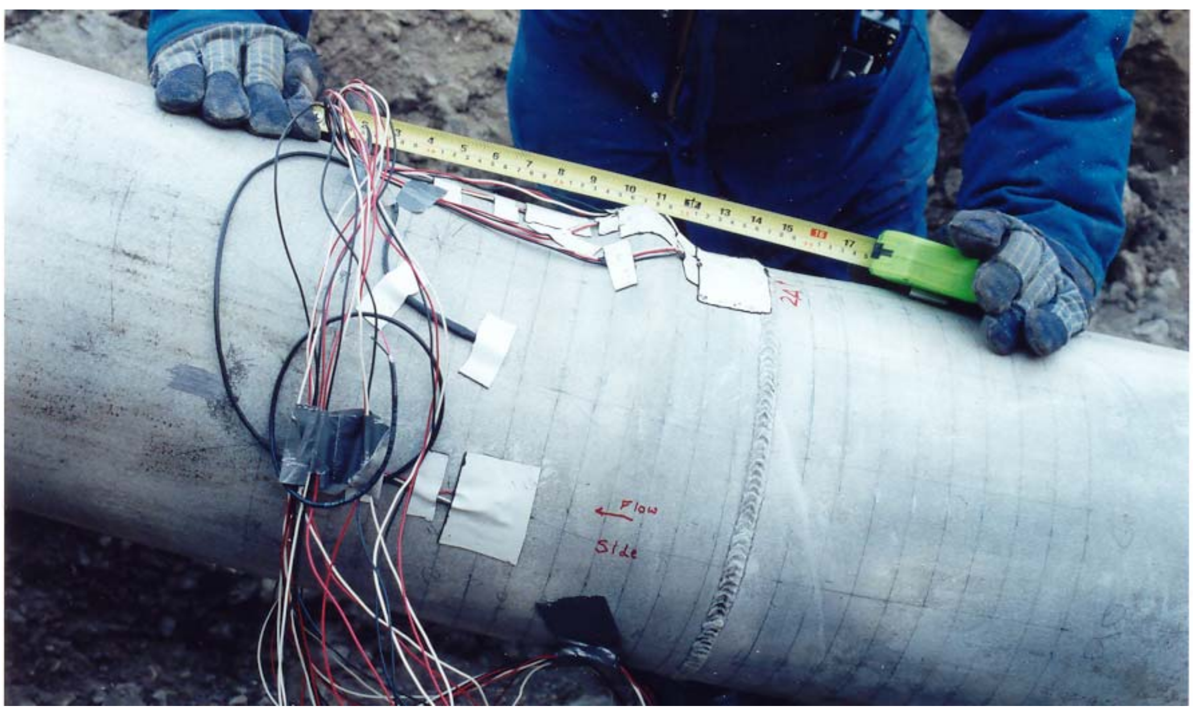


Figure 5-50: GEOPIG data from KP 318, comparing 1997 to 1992.

(a) Exposed pipe at KP318, showing how backfill soil had displaced downslope resulting in tearing and gathering of the protective yellow jacket. (On this particular pipe joint a double layer of protective yellow jacket was applied. This was not normal practice.)



(b) Exposed wrinkle at KP318. The wrinkle is located approximately 100 mm for a joint weld. The wrinkle is approximately 25 mm high.

Figure 5-51: Photographs of exposed pipeline at KP 318 (February 1998).
(Photographs courtesy of Enbridge Pipelines (NW) Inc.)

The replacement of the pipe section required the purging of the line of hydrocarbons. As purging pigs passed the mainline block valves immediately up stream of the site, the valve was closed and nitrogen was injected into the pipeline to advance the pigs. As the pigs passed the down stream valves, these were then closed and the nitrogen was removed by vacuum trucks.

Coincident with the purging, the wood chips on the slope were stripped and stockpiled. The ditch was then excavated and the pre-bent pipe was laid in the trench to verify its alignment relative to the existing pipe.

When the pipeline was purged of nitrogen, the line was cut, the wrinkled section removed and transported to the University of Alberta for testing, and the new section was welded in. The welds were X-rayed, and the weld joints covered with anti-corrosion coating.

The valves were opened and the Norman Wells pump station resumed operations approximately 48 hours after shut-down.

The pipeline was bedded and padded with compacted sand, and covered with native soils and wood chips. Subsequent to the remediation work, additional slope indicators were installed on the slope.

5.6.5.3.3 Monitoring

Figure 5-53 and Figure 5-54 show the results for two slope indicators. Slope indicator (SI) #2, (Figure 5-53) was located within 2 m of the pipe. This instrument was installed in 1999 immediately after the pipe section was replaced and was intended to replace the slope indicator that was in-place prior to the pipe replacement. Slope Indicator SI-4 (Figure 5-54) was located off the “west” side of the right-of-way.

Review of the slope indicator data showed that 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 nine 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 was considered that this creep/straining zone was concentrated on the cleared right-of-way and on the steepest section of the slope (at least prior to the pipeline excavation and replacement in February 1999).

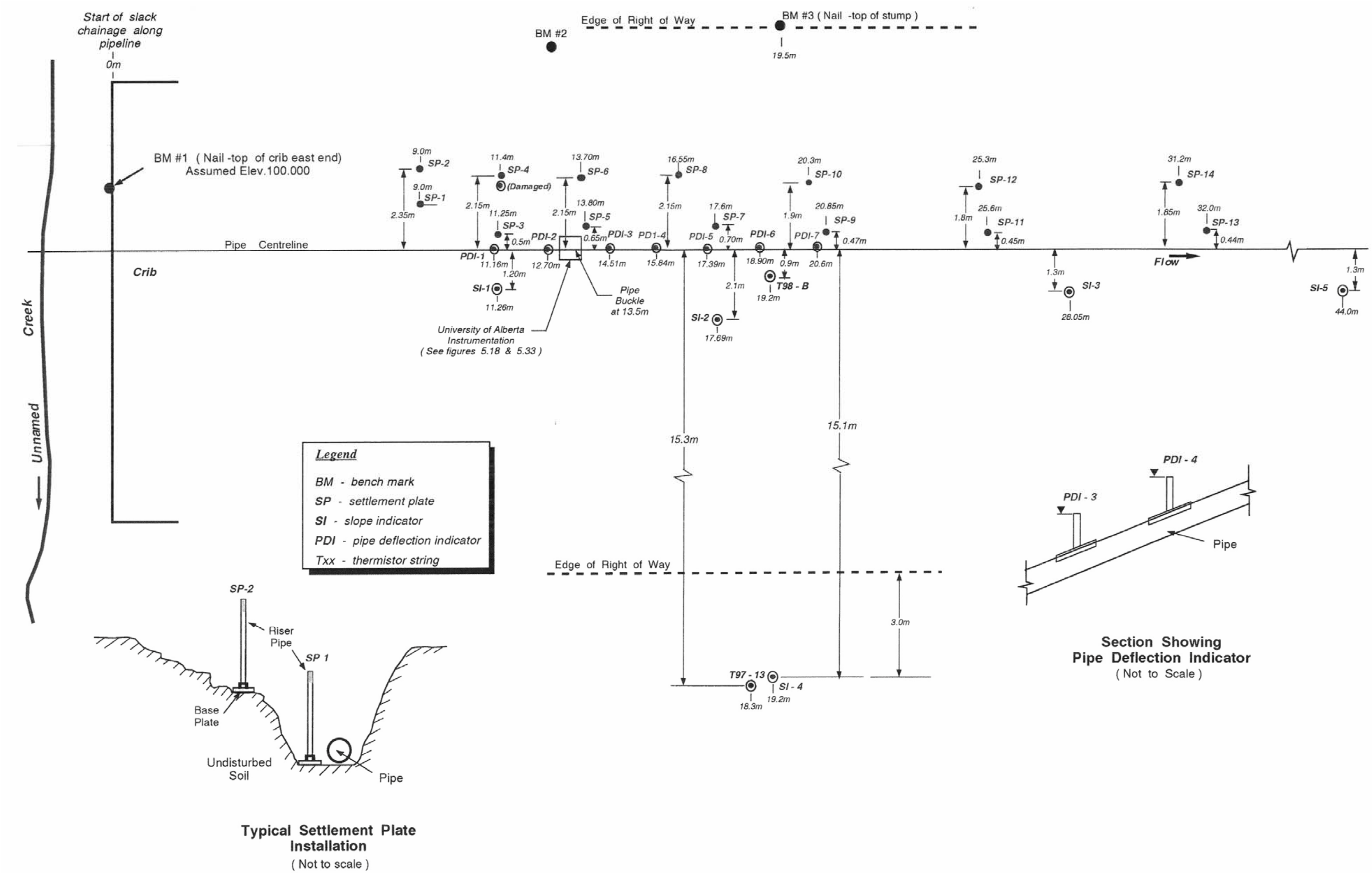


Figure 5-52: Layout of instrumentation installed at KP 318 in February 1998 to monitor slope and pipeline deformations after verification of wrinkle in the pipe near the base of slope.

In SI 4, located off the right-of-way, very little movement was observed through the summer and fall of 1998. Within the active layer, the down slope movement of about 30 mm observed was considered "normal" active layer movement.

Two new slope indicators were installed to replace the two on-slope indicators that were destroyed during the cut-out program. The replacement on-slope indicator (SI99-2, shown on Figure 5-53) produced data consistent with the pre-remediation instrument. Two movement zones are identified, one at about 3 m depth and a second deeper zone at about 6.5 m to 8.5 m depth. The off-right-of-way slope indicator (SI 4) operated throughout the period and data is presented for it from March 1998 to June 2001 (Figure 5-54). It is interesting to note that prior to the cut out and replacement, the off right-of-way slope indicator displayed essentially no movement, and subsequent to the replacement in February 1999 movement was initiated in this instrument.

The displacement of the on-slope and off-slope instruments at a depth of about 5 m have been plotted with time, and are shown on Figure 5-55. The pre-replacement period was from February 1998 to February 1999. In the approximately six months following the pipe replacement in February 1999, the movement of the on-slope instrument and the off-slope instrument increased. In 2000 the rate of movement subsided to approximately pre-replacement rates. The continued movement of the slope caused both instruments to become in-operative in 2001. In 2004, one new slope indicator was installed off the right-of-way to provide continued slope movement data. The instruments continue to provide movement data that shows on-going slope movements.

Figure 5-56 presents a comparison of the 2006 and 2005 GEOPIG data. The comparison shows that there have been no changes in the pipeline geometry or strains that could be related to the slope movements. No doubt the installation of "heavy" wall pipe in February 1999 has provided additional strain capacity of the pipeline.

5.6.5.4 Pipe Wrinkle Study at KP 311

Similar to the identification of a wrinkle in the fall 1997 at KP 318, two smaller, incipient wrinkles were identified within a short distance of each other at KP 311 in 1998. In February 1999 the pipeline was excavated and thick wall sleeves were welded over the sections of concern. Slope indicators were also installed on this slope to monitor slope movements. In a similar fashion to the KP 318 event, two movement zones were identified, one near surface at about 2 m and a second deeper movement zone at 13.5 m below the top of wood chips. These instruments remained active until 2002, when it is believed the casing cracked and filled with water and ice.

The 2003 GEOPIG inspection identified a new incipient wrinkle developing on the pipe at a location between the two sleeved original wrinkles. This wrinkle was also sleeved in February 2004.

In February 2004 a new slope indicators was installed on the slope, off the right-of-way, so as not to interfere with any future excavation and remediation work.

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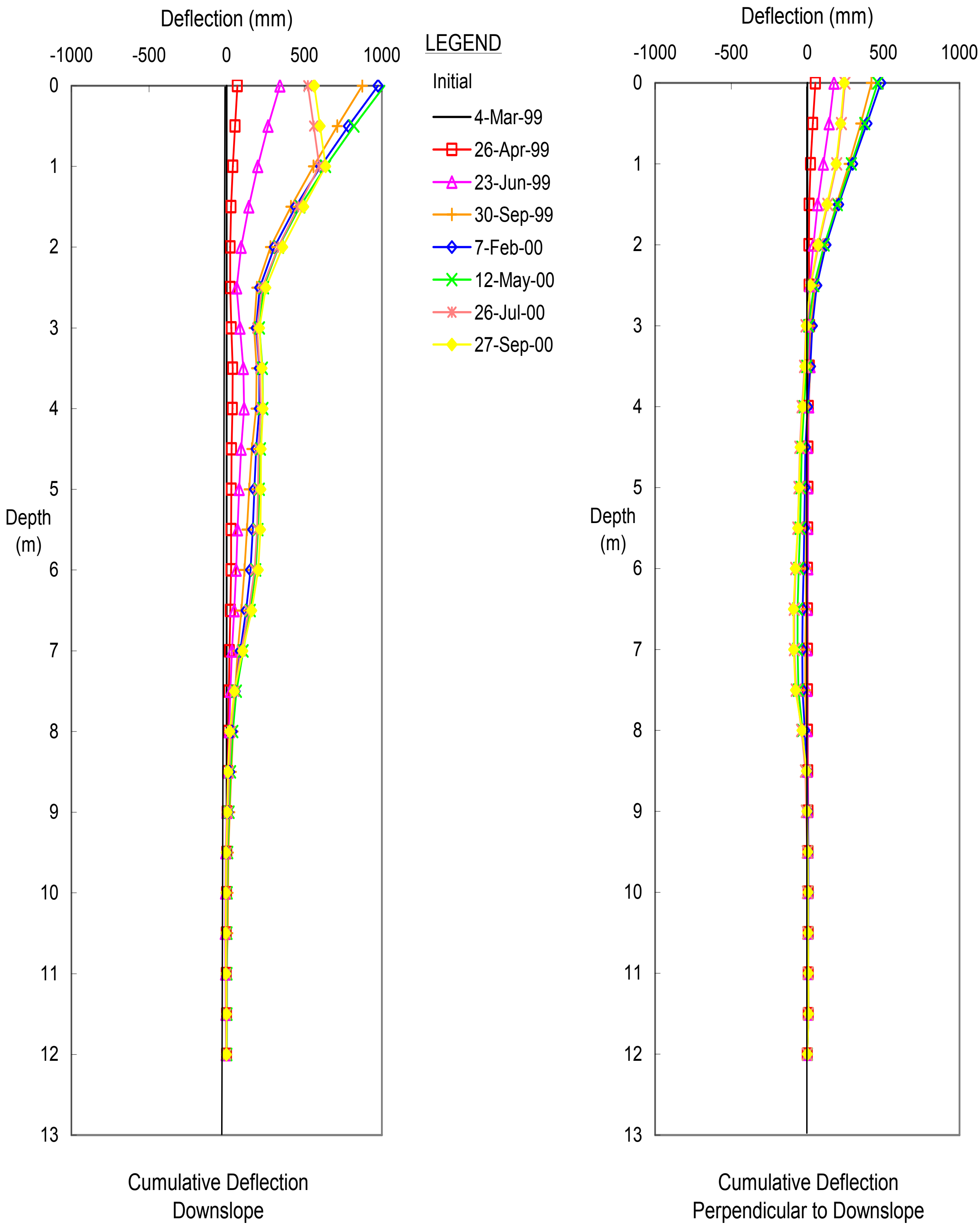

Figure 5-53: KP 318 on right-of-way slope indicator, SI #2.
Note: this slope indicator was installed after the pipe replacement work in February 1999.

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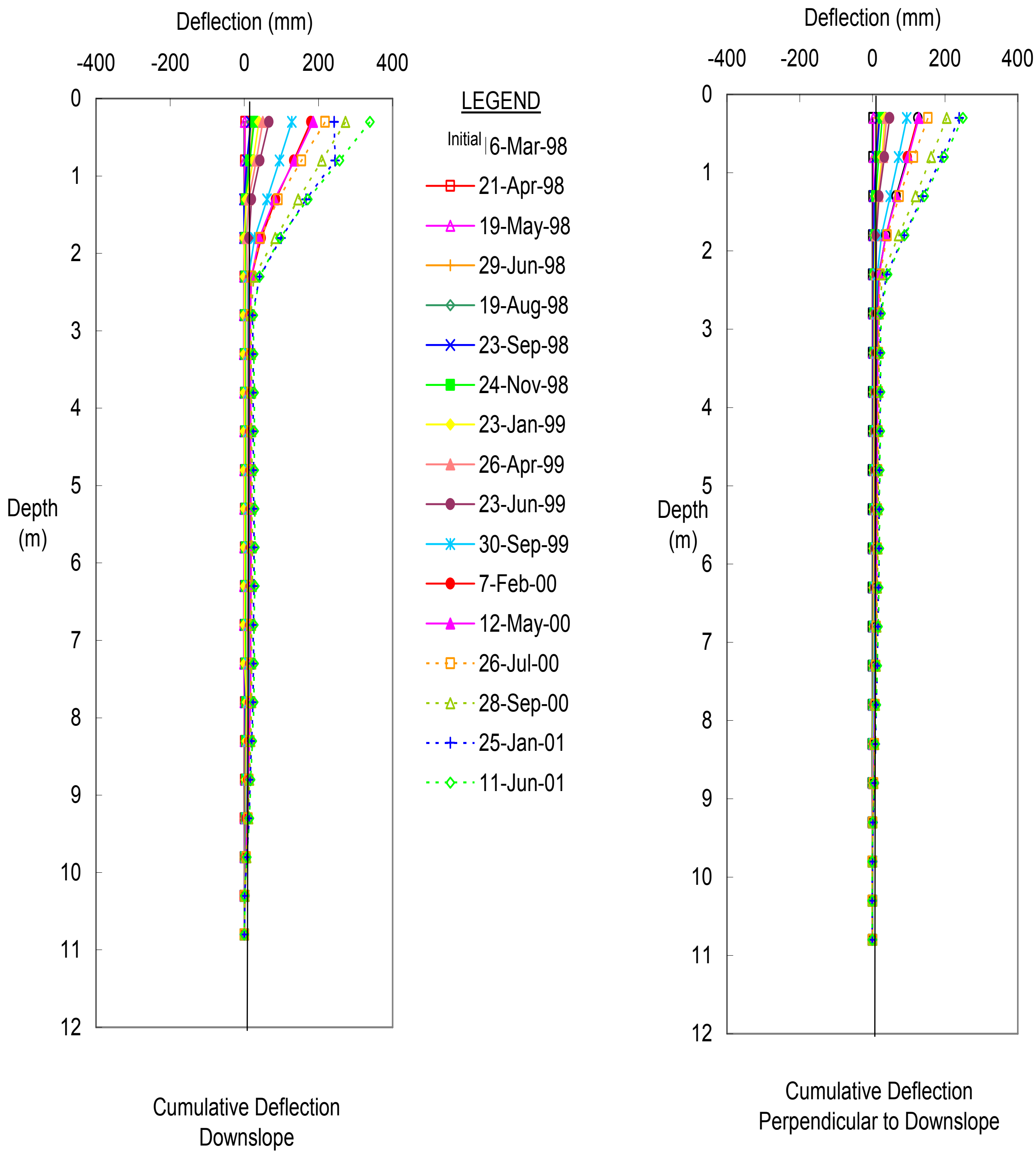

Figure 5-54: KP 318 off right-of-way slope indicator #4.

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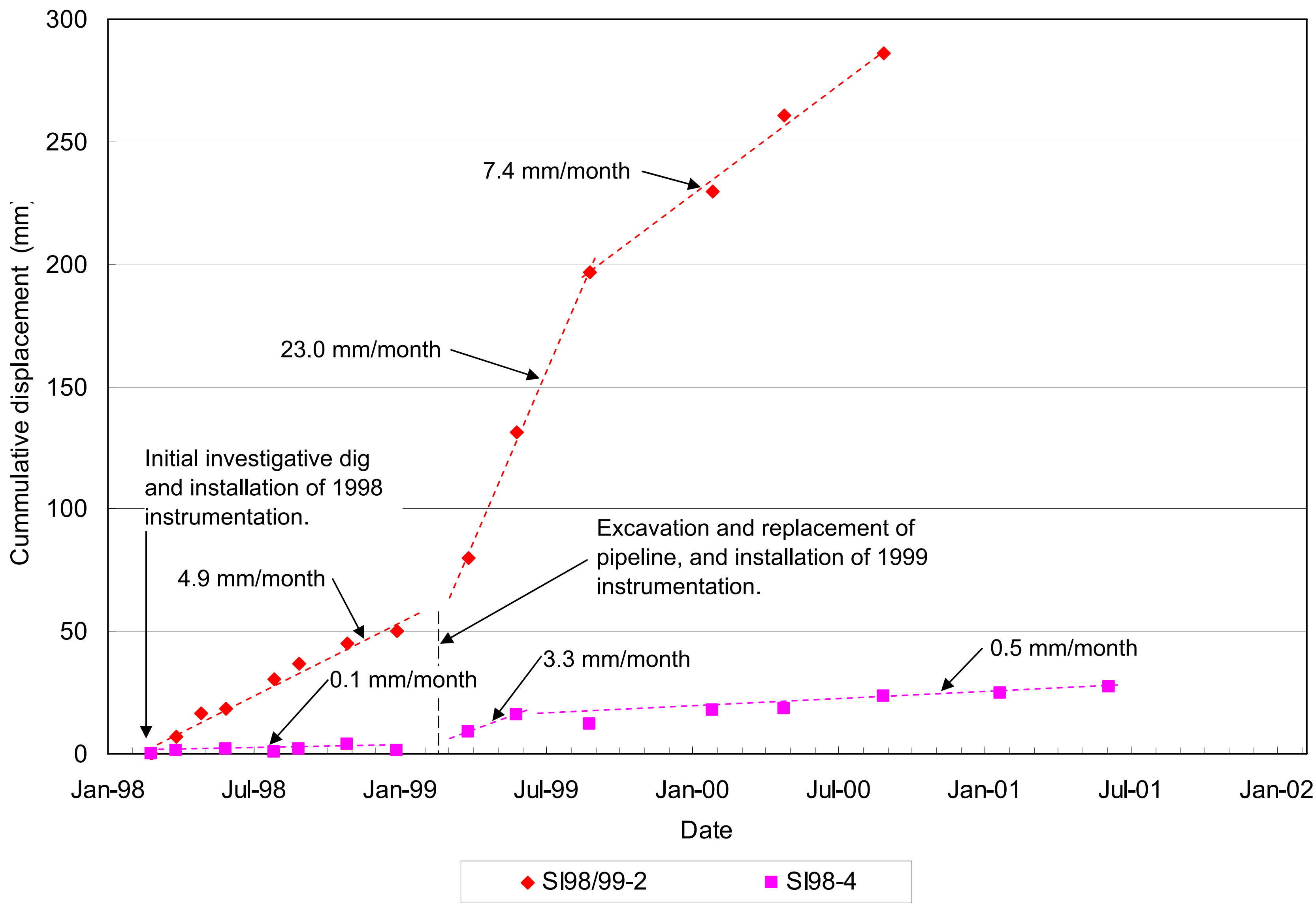


Figure 5-55: Slope movements at KP 318 with time at 5 m depth.

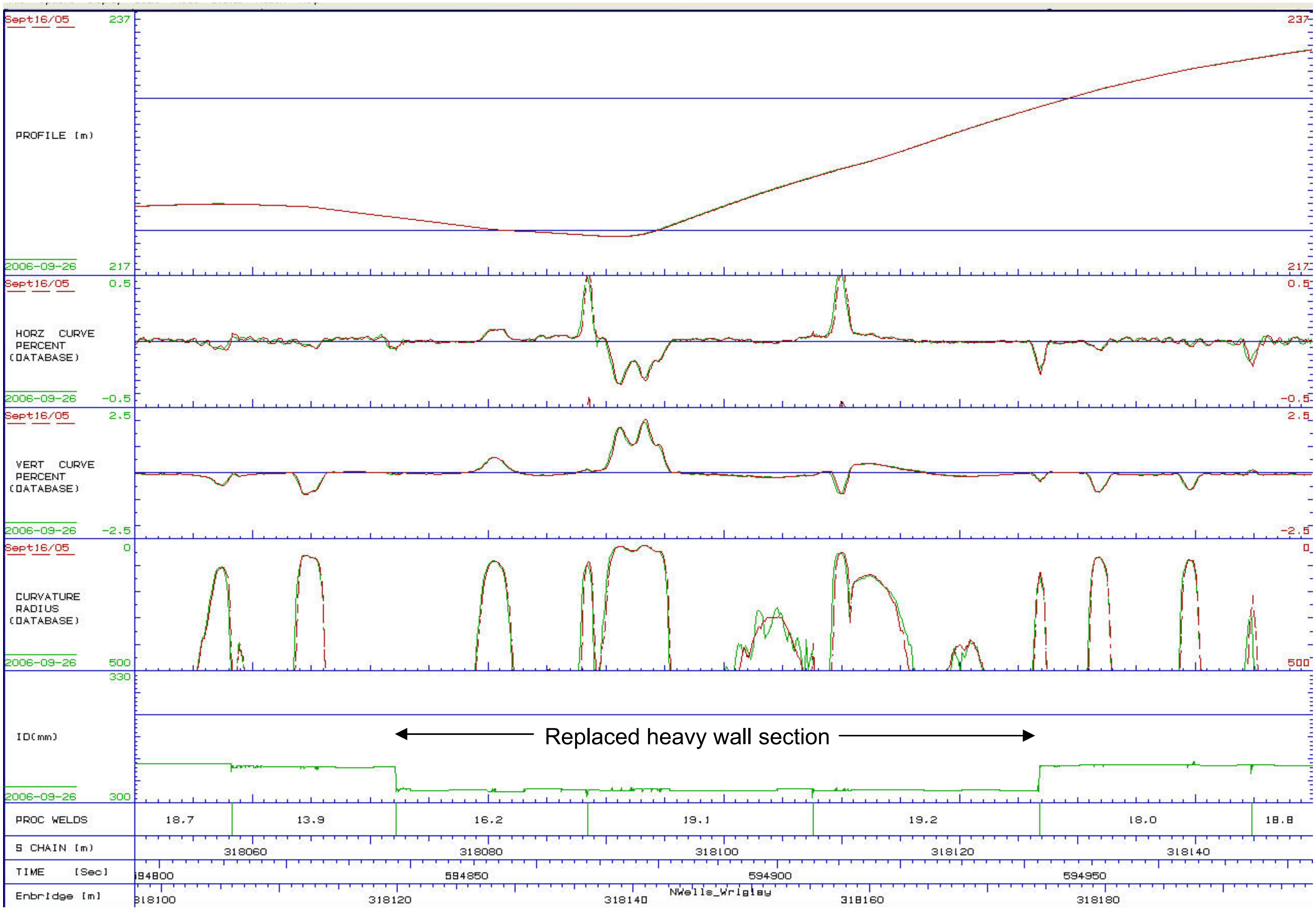


Figure 5-56: Comparison of 2006 and 2005 GEOPIG data for Slope 92, KP 318.

In 2005, the GEOPIG once again identified the initiation of another (fourth) wrinkle. In response to this event, the Enbridge decided to replace the existing section of pipe with a heavy wall pipe from a point on the north side of the small creek to near the crest of the south slope. The work program was very similar to that conducted at KP 318 in 1999. The cut-out and replacement work was successfully completed in February 2007.

One new slope indicator and a thermistor cable were installed on the slope in 2007, immediately after the pipe replacement work was completed.

5.6.5.5 Creeping Slope Study at KP 313.6

In light of the proximity of sites where wrinkles that developed on the pipeline at KP 311 and 318, the Geological Survey of Canada undertook a slope monitoring program at a site that had not developed any wrinkles on the pipe, but which was, by most measures, of similar morphology to the sites at KP 311 and KP 318: moderate slopes in the order of 10^o to 14^o, relatively short (typically about 100 m long), and north facing. The purpose of the work, in cooperation with Enbridge was to instrument a site so that should slope movement take place, the data set would be useful in examining possible initiation mechanisms. The work was commenced in the winter of 2001

The site instrumentation consisted of numerous slope indicators, thermistor cables, active layer indicators and a portable climatic station. During the initial deployment of instrumentation the project site was also surveyed and mapped with geophysical, primarily conductivity methods.

To 2007 minimal movement has been observed in any of the six slope indicators installed on the slope. Small amounts of movement are observed within the wood chips and immediately within the underlying mineral soil, but not to the degree that has been observed at KP 311 and KP 318.

5.6.5.6 Movement Mechanisms of Creeping Slopes

The identification of creeping slopes on the pipeline route has been primarily through the observation of strain events with the GEOPIG. The study site at KP 313.6 instrumented by the GSC has not indicated any slope movement, and GEOPIG results for this slope do not indicate any strain development. When strains are seen to be accumulating in the pipeline, then geotechnical instrumentation is installed to characterize the movements.

Oswell et al (2000) identified a potential mechanism that may be responsible for the strain development. But, the identification of slopes that are susceptible to these creep mechanisms remains under consideration.

5.6.6 Other Slope Movements

There are several landslides or slumps that may interfere with the right-of-way. Two of these sites are located at KP 158 and KP 182.

5.6.6.1 KP 158 Little Smith Meander

The potential interaction of river erosion and bank instability on the right-of-way at this location has been of concern since the initial route selection during the design phase for the pipeline. In 1992, toe erosion of the right bank commenced following a major spring runoff. The resulting slope instability on the right bank of Little Smith Creek may threaten the right-of-way if additional mass soil movements occur. A detailed hydrotechnical and geotechnical report for this site was completed for Enbridge in 2002, which detailed a number of remedial approaches.

An indicator of slope movement is a split tree near the crest of the slope, shown in Figure 5-57. Comparison of photographs taken in past years confirms that spreading of slump graben is ongoing. Measurement of the widening of the split in the trunk, either by comparison to other photographs or by direct measurement provides information on the rate of movement of the head-scarp (graben). This analysis is presented on Figure 5-58. The data suggests that a relative uniform rate of graben spreading of about 30 mm/year occurred between 1992 and about 1999, and then accelerated between 2000 and 2001 at a rate of about 360 mm/year. From 2001, the rate of graben spreading slowed to nearly the pre-2000 rate. The rate of spreading movement since 2001 is estimated to be approximately 45 mm/year. It is considered that continued graben spreading is inevitable given the ongoing erosion at the toe, the evidence of groundwater discharge and other factors. The time scale for a large scale movement is likely measured in years rather than decades.

In 2004 a slope indicator was installed on the right-of-way adjacent to the instability area. To 2007, no slope movement has been detected by the instrument.

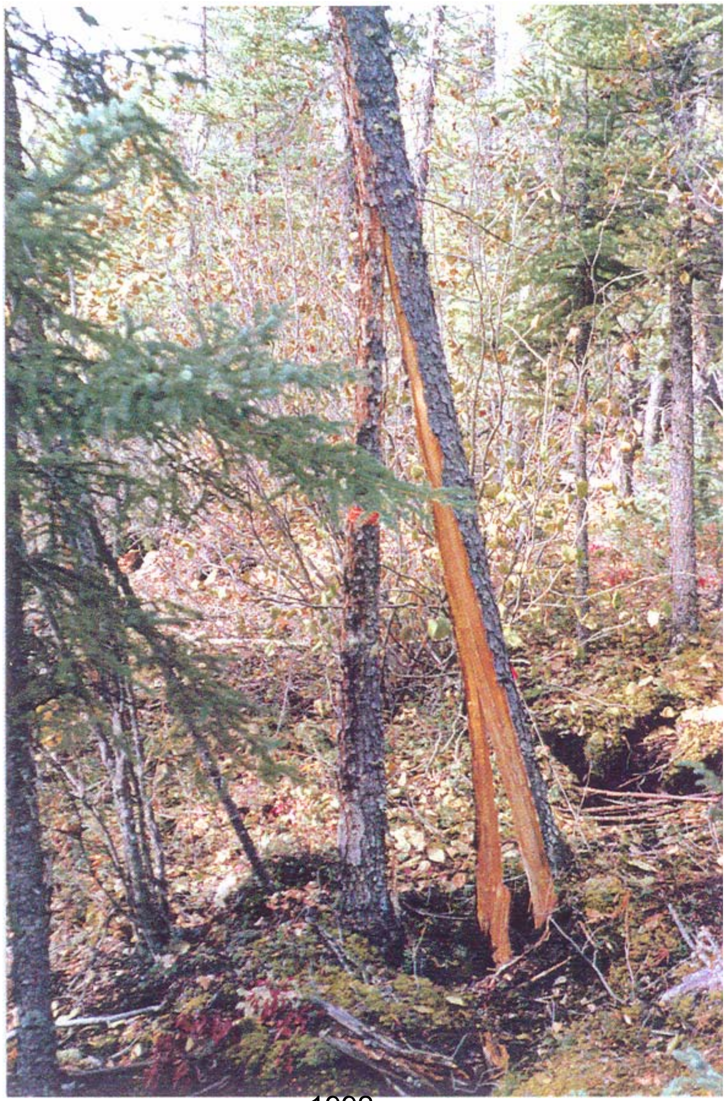
5.6.6.2 KP 182 Fire Burn Area

A forest fire damaged the areas adjacent to the right-of-way at KP 182 in 1994. In fighting the fire some additional damage occurred. The area remains quite dry and reasonably covered with vegetation. The area damaged by aerial water dumping and the wood chips appeared satisfactory although considerable decay of the wood chips is apparent.

In the fall of 1994, thermistors and survey pins were installed at two locations near the right-of-way. The locations were adjacent to areas where mass slumping was occurring.

Figure 5-59 presents plots of the relative distances between the survey points at the two slump sections. In each case, Pin 1 is closest to the slumps. For the north (upstream) section (Figure 5-59a), there is measurable lateral spreading between Pins 1 and 2. Between 1997 and 2006 there was measurable lateral spreading between Pins 1, 2 and 3. In the first nine years of monitoring, Pins 1 and 3 separated by approximately 0.8 m. These pins are closest to the headscarp of the slide and are all located off the cleared right-of-way.

NCI amec


1992


2004

Figure 5-57: Photographs of a tree spanning the graben of a slump developing adjacent to Little Smith Creek, KP 158.

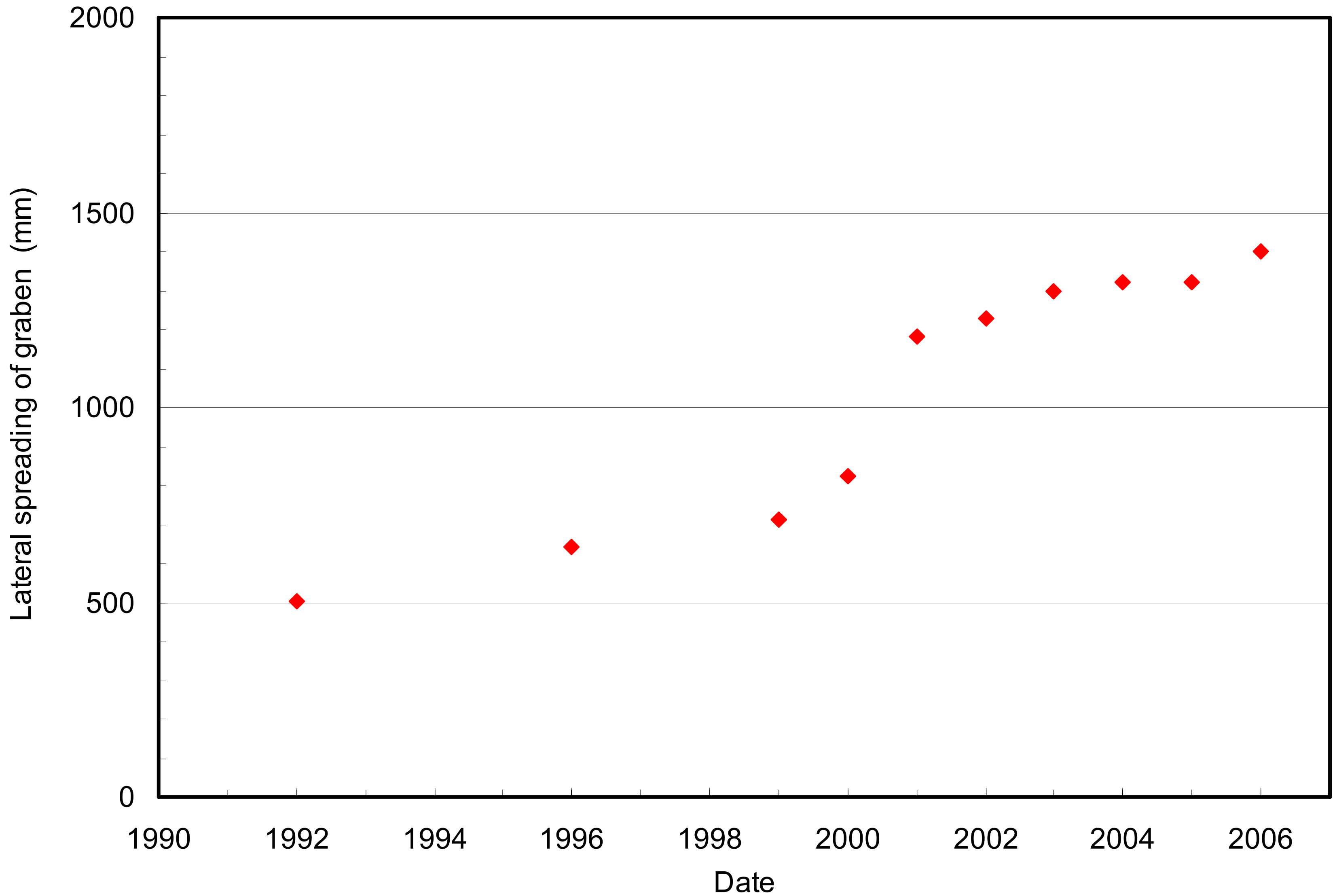


Figure 5-58: Lateral spreading of graben of slump, adjacent to Little Smith Creek, KP158.

Monitoring shows no appreciable lateral movement of the right-of-way. (At this site the pipeline is located on the west side of the right-of-way, furthest from the valley crest.) Lateral movement is also measured at the southern transect. In nine years of monitoring, Pins 1 and 2 have separated about 0.4 m. No movement is detected across the cleared right-of-way.

Figure 5-60 shows the thaw depths at the thermistors, which are located on the “east” side of the right-of-way, near the two larger slump areas. At 182-T1 thawing has progressed to approximately 4 m (bottom of thermistor cable is at 6.0 m). At 182-T2 thawing has exceeded the base of the thermistor cable at 3.7 m. Between 1997 and 2000, the rate of thawing of 182-T1 was approximately 0.36 m/year. Between 2001 and 2002 a slight freeze-back was measured, by additional thaw was recorded in 2003 and 2004. The rate of thawing is certainly slowing, and some equilibrium may be being achieved. Further monitoring of ground temperatures is hampered by damage to the thermistor cables in 2006, presumably by bears.

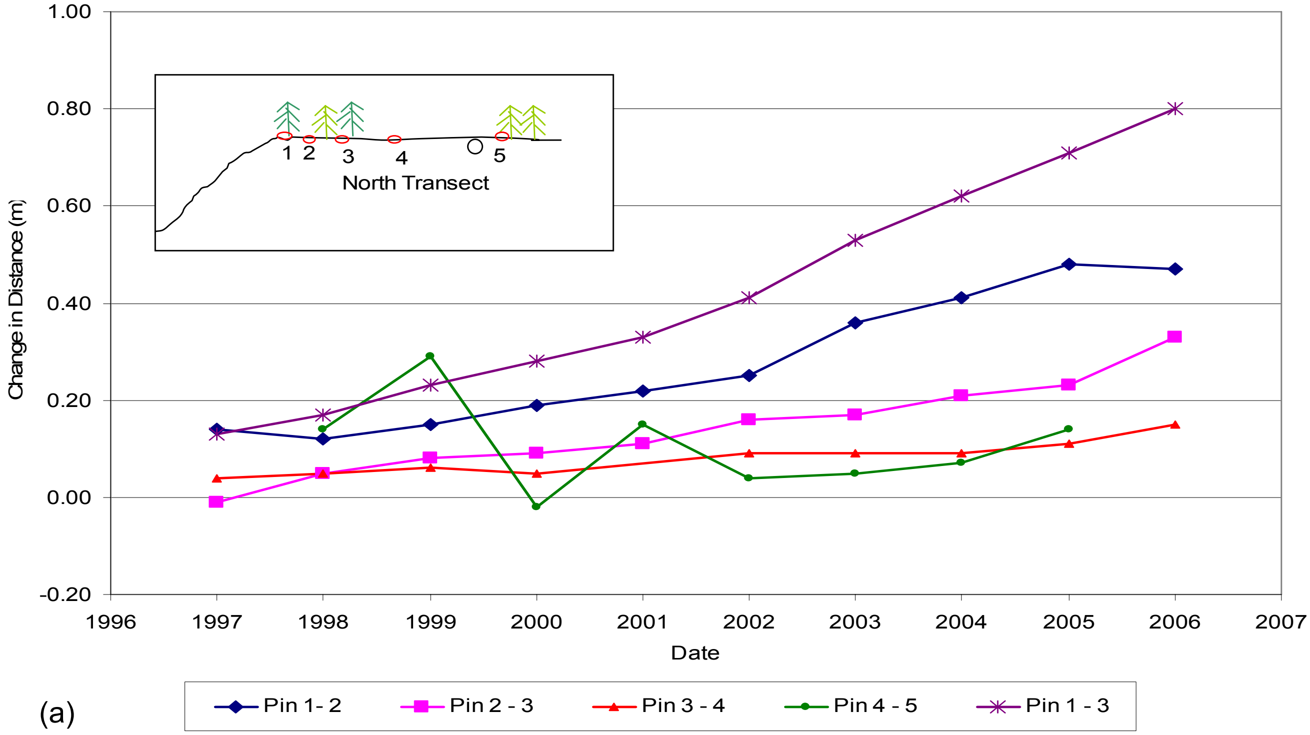
The initiation of slope instability as a result of the forest fires in this specific area has been studied by Lewkowicz and Harris (2005). They studied a series of flows that initiated on the valley walls between the Enbridge right-of-way and the confluence with the Mackenzie River. Lewkowicz and Harris identified 45 flows in this stretch of valley. The median length of the flows was 35 m (the maximum length was 120 m), and the slope angles that failed had a median of 20° and a minimum of 13°. These authors concluded that the flows were initiated by high pore water pressures associated with rapid melting of ice-rich permafrost.

Related to the same series of forest fires in the mid-1990s, Savigny, Logue, and MacInnes, (1995) analyzed geotechnical conditions to initiate flows resulting from forest fire induced disturbance. Their conservative assessment found that the threshold angle for flow movements ranged from 14° to 24° depending on the time, depth of thaw and other factors.

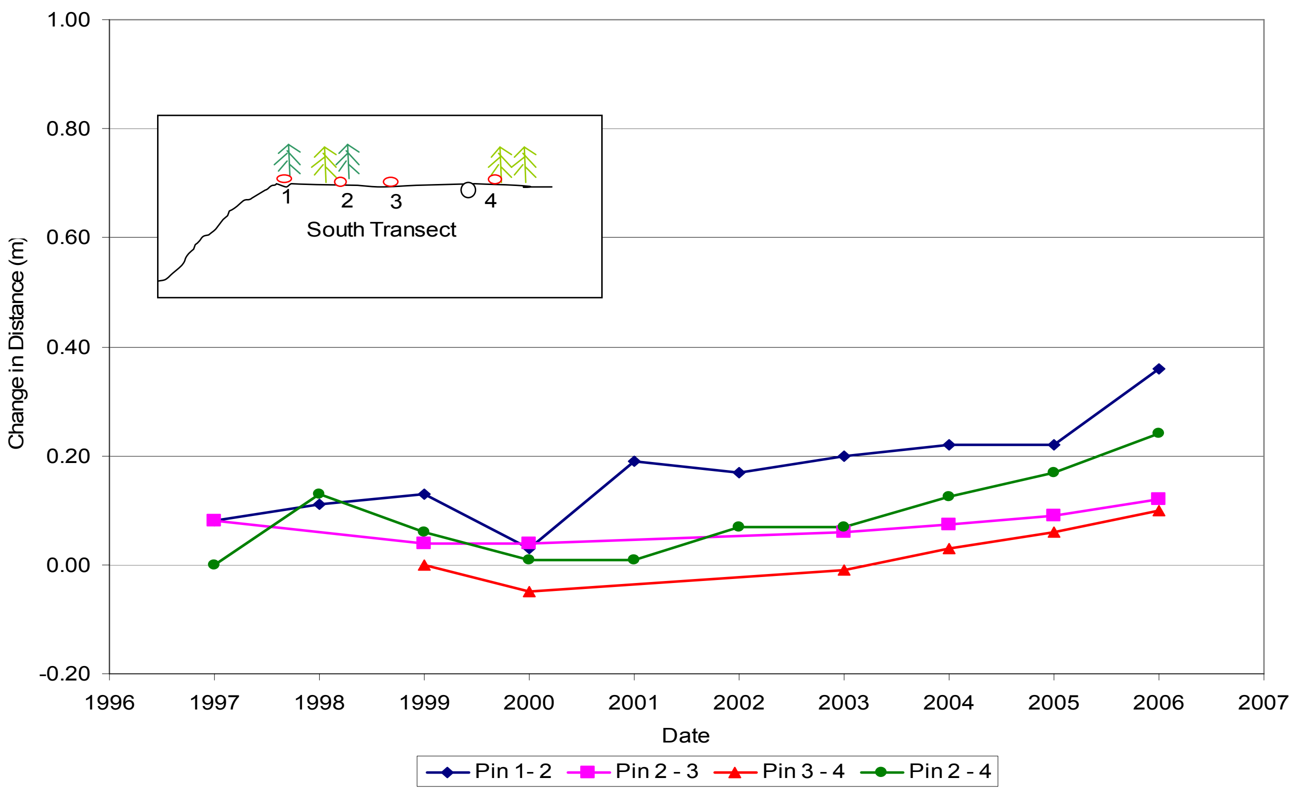
5.6.7 Piping and Ground Loss

There have been several occurrences that could be described as ground loss within the ditchline. These events are distinct from erosion, which is typically used to describe loss of soil from surface events (run-off, wind, surface disturbance et cetera). Ground loss and piping, in this context, is intended to address internal or subsurface erosion.

The most prominent example of ground loss occurred at Slope 29B, Great Bear River south. This site is a northwest facing wood chip insulated slope. In the early 1990s, a section of wood chips over the ditch line in the upper mid-slope area collapsed into a void that was present over the pipe. Investigations of the collapse and the void revealed that a void over the pipeline extended for some distance down the slope. Figure 5-61 presents a photograph of the void and a schematic prepared by Dr. K. W. Savigny, P.Eng., consulting to the Department of Indian Affairs and Northern Development. Enbridge filled in the void and re-established the wood chip cover of the pipe.



(a) North transect of survey pins.



(b) South transect of survey pins.

Figure 5-59: Lateral spreading of survey pins across right-of-way at KP 182.

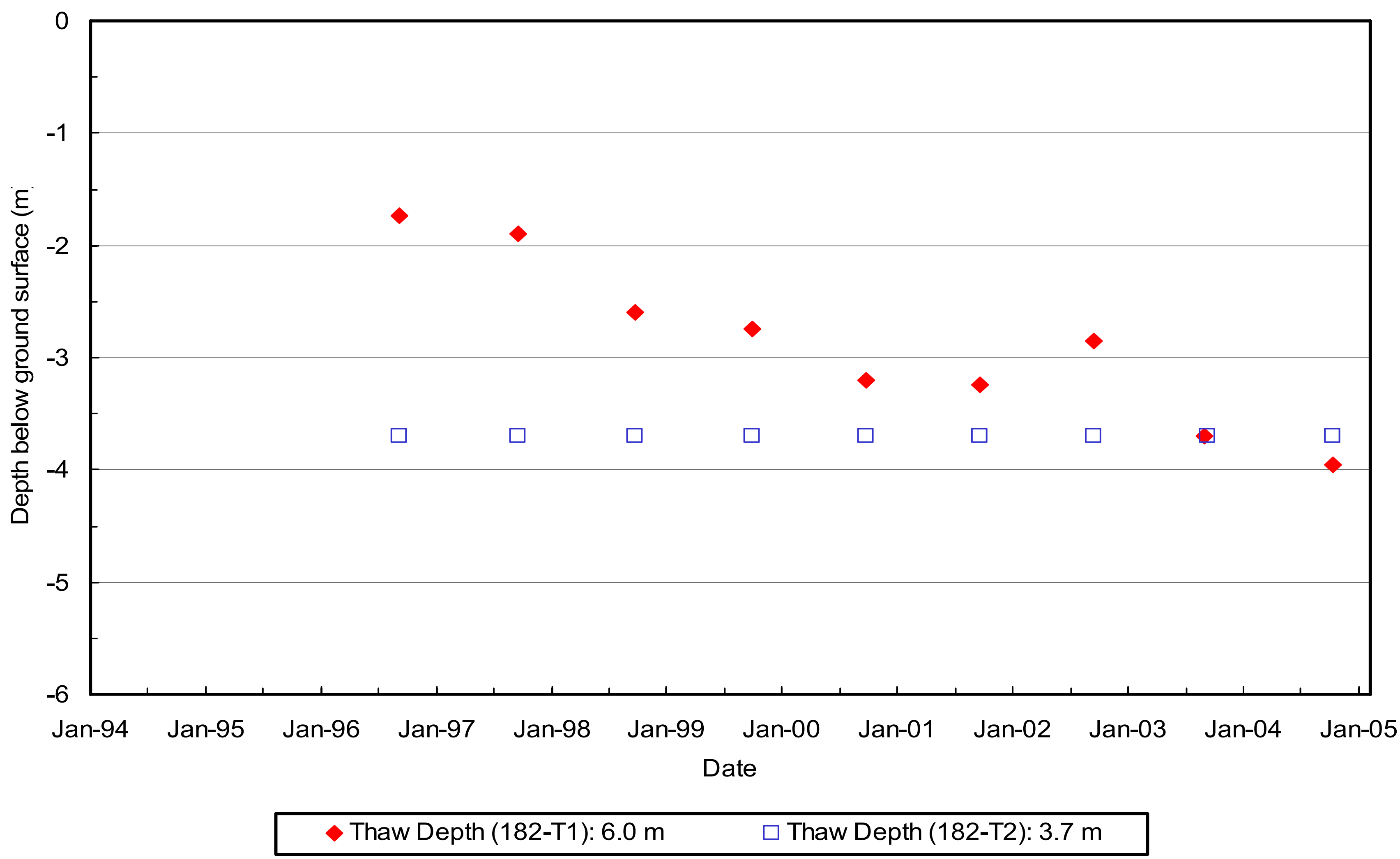


Figure 5-60: Thaw depth with time at forest fire burn area, KP 182.

The potential for ground loss arising from the melting of ice-rich permafrost along the ditchline and elsewhere on the right-of-way has been raised as a potential pipeline integrity concern. Savigny (2004) presented a scenario to explain the ground loss at Slope 29B (KP79, Great Bear River south) described above and shown in Figure 5-61. The scenario proposed by Savigny was that a warm pipeline operating in permafrost could experience thaw settlement that would leave an open void over the pipeline. However, monitoring by the GEOPIG has shown that minimal settlement of the pipeline took place along this slope (Skibinsky, 2006). Thus, the void formed at Slope 29B must have another cause.

An alternative theory of the void formation below or within the wood chip layer could be snow/ice contaminated ditch backfill soils. If the backfill soils contained large volumes of ice and snow, then during pipe operations, the snow and ice could melt, forming a void, bridged by the wood chips. The melt water flowing down slope could then induce internal erosion.

No other voids or collapses of the wood chip cover have occurred at any other site,

Ground loss resulted in a discharge of sandy soil onto the right-of-way and slope at Slope 45, KP 133. In the early 2000s, a volume of fine was discharged from an off right-of-way area west of Slope 45 (KP 133). The sand appeared to originate immediately below the organic mat approximately halfway up the slope. The direct cause of the discharge is not known, but it was postulated that snow melt had entered the subgrade through cracks or tears in the organic mat up slope of the discharge point and induced internal erosion of the fine sand that discharged onto the right-of-way. The discharge was considered to be unrelated to the presence of the pipeline or its operation.

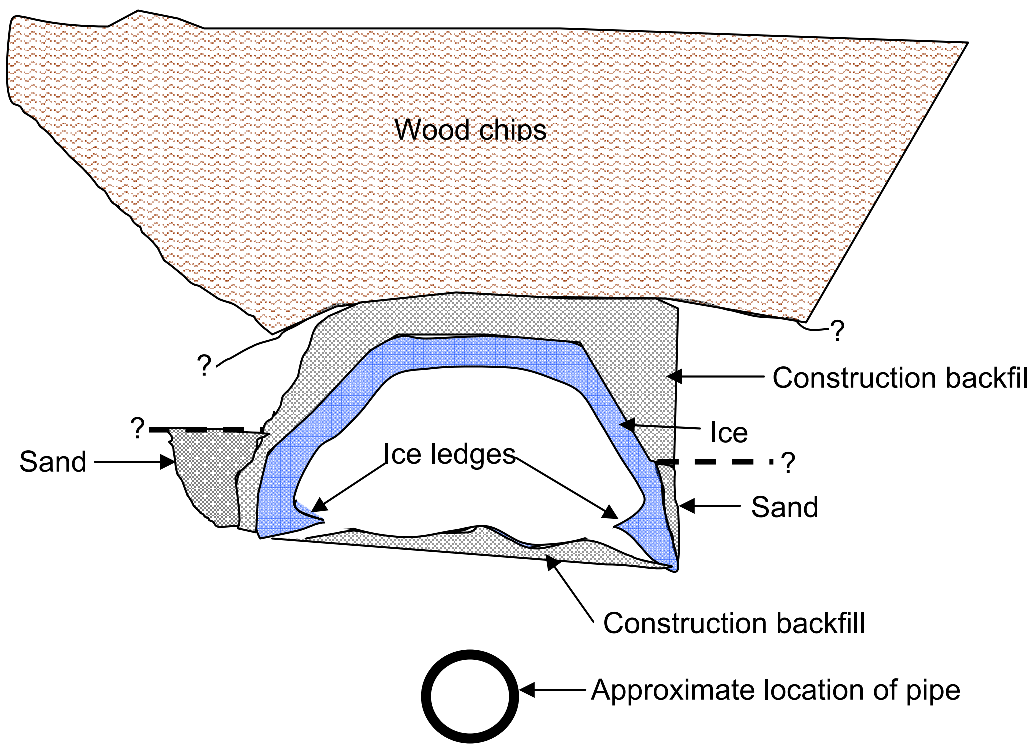
5.7 River Crossings

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 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.

(a) Photograph of void under wood chips at Slope 29B, KP 79, Great Bear River south.



(b) Schematic of void at Slope 29B, KP 79, Great Bear River south.

Figure 5-61: Photograph of void under wood chips and schematic of void at Slope 29B, KP 79, Great Bear River south. (Savigny, 2004)
Note: Schematic redrawn from original presented by Savigny (2004).

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.

In the spring of 2003, a rock berm on the left bank of Hodgson Creek was breached. This breach occurred approximately 500 m upstream from the pipeline crossing at the outside curve of an “S” bend. As a result of the breach upwards of one-half of the creek flow diverted onto the right-of-way creating numerous scour holes, deposition of fines and granular soils, and several erosion channels. A remediation plan was develop to restore the site conditions. The first phase of the program was to remove any fish that were present in the overland flow sections. Cut-off nets were installed at the site of the rock berm breach and at the points where the overland flow re-enter the main channel. This work was completed in October 2003. The second phase consisted of rebuilding the rock berm, which was completed in March 2004. Since that reconstruction, there have been no new breaches of the creek banks in the vicinity of the right-of-way.

5.8 Right-Of-Way Disturbance

Following the end of the first and third years of operation, 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. 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.

Figure 5-62 presents two views of the overland right-of-way. The photographs were taken in the same vicinity, and 20 years apart. Surface grading was implemented on the left side of the right-of-way to facilitate construction. After 20 years, considerable vegetation has re-established and a state of long-term stability achieved.

Several significant forest fires have impacted the right-of-way in the past 12 years. In 1994, a forest fire initiated by a lightening storm, burned an area paralleling 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; McNeill, Hanna, Fridel and Babkirk, 1996). The main areas burnt in the fires were between Norman Wells and the Ochre River (KP 286).

In 2004 forest fires along the Northwest Territories – Alberta border crossed over the pipeline right-of-way. Although no damage to the pipeline was noted in the predominantly flat, organic terrain, two ground temperature monitoring sites operated by the Geological Survey of Canada were damaged. These damaged sites were 84-5A and 84-5B, near the Petitot River.

During the 1990s forest fires, pumps and sprinkler systems were set up on several 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 (see Section 5.6.5.2). 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 Right-of-Way 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 5-63 shows the physical changes in the right-of-way in 1986 and 1988. (No additional specific studies have been conducted since the 1988 program.)

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 5-64.

(a) View of right-of-way during clearing, 1984.

(b) View of right-of-way in 2004.

Figure 5-62: Two views of the overland right-of-way, 1984 and 2004.
(Photographs courtesy of Enbridge Pipelines (NW) Inc.)

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).

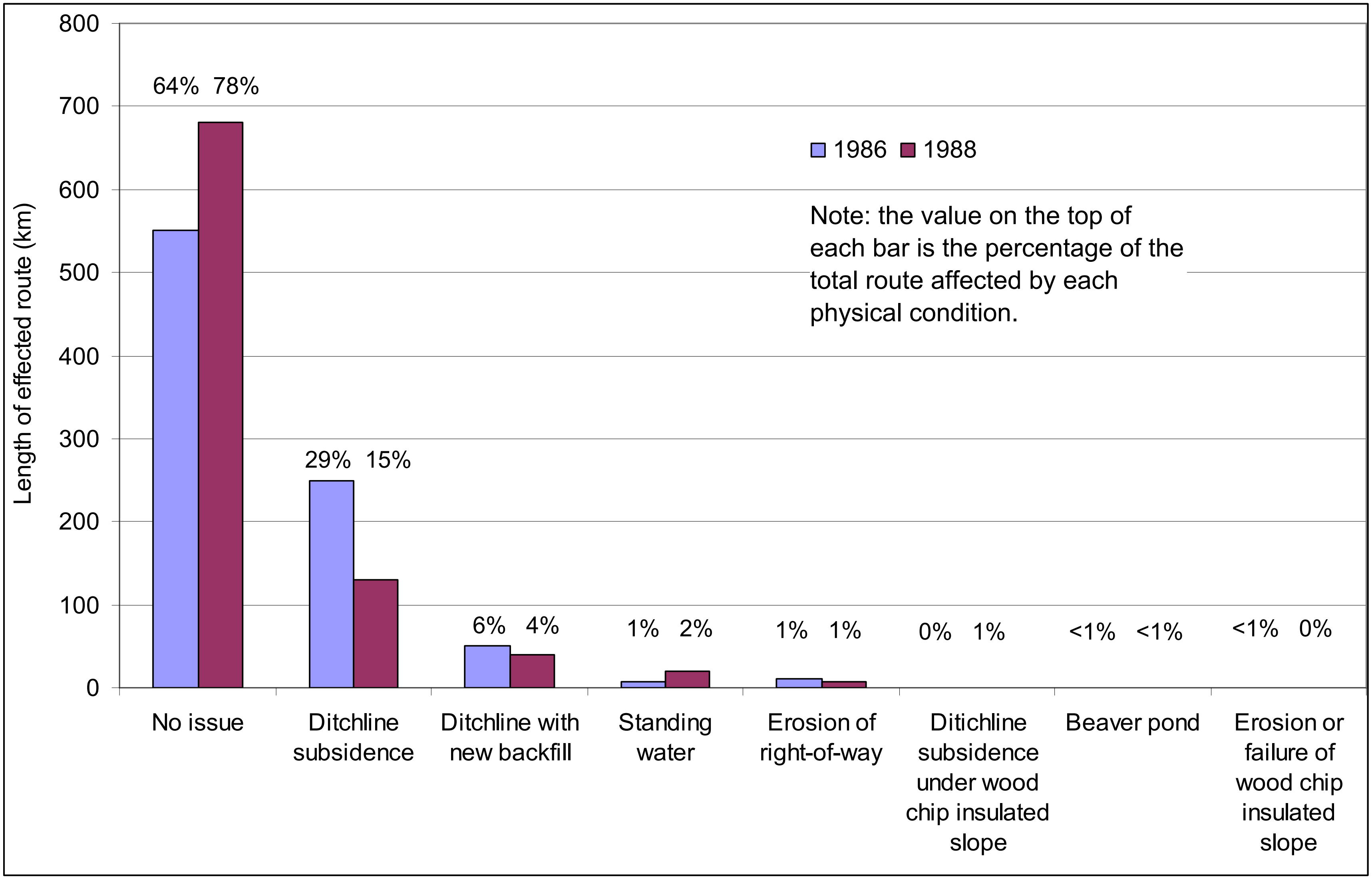


Figure 5-63: Extent of changes in physical conditions along pipeline route from 1986 to 1988.

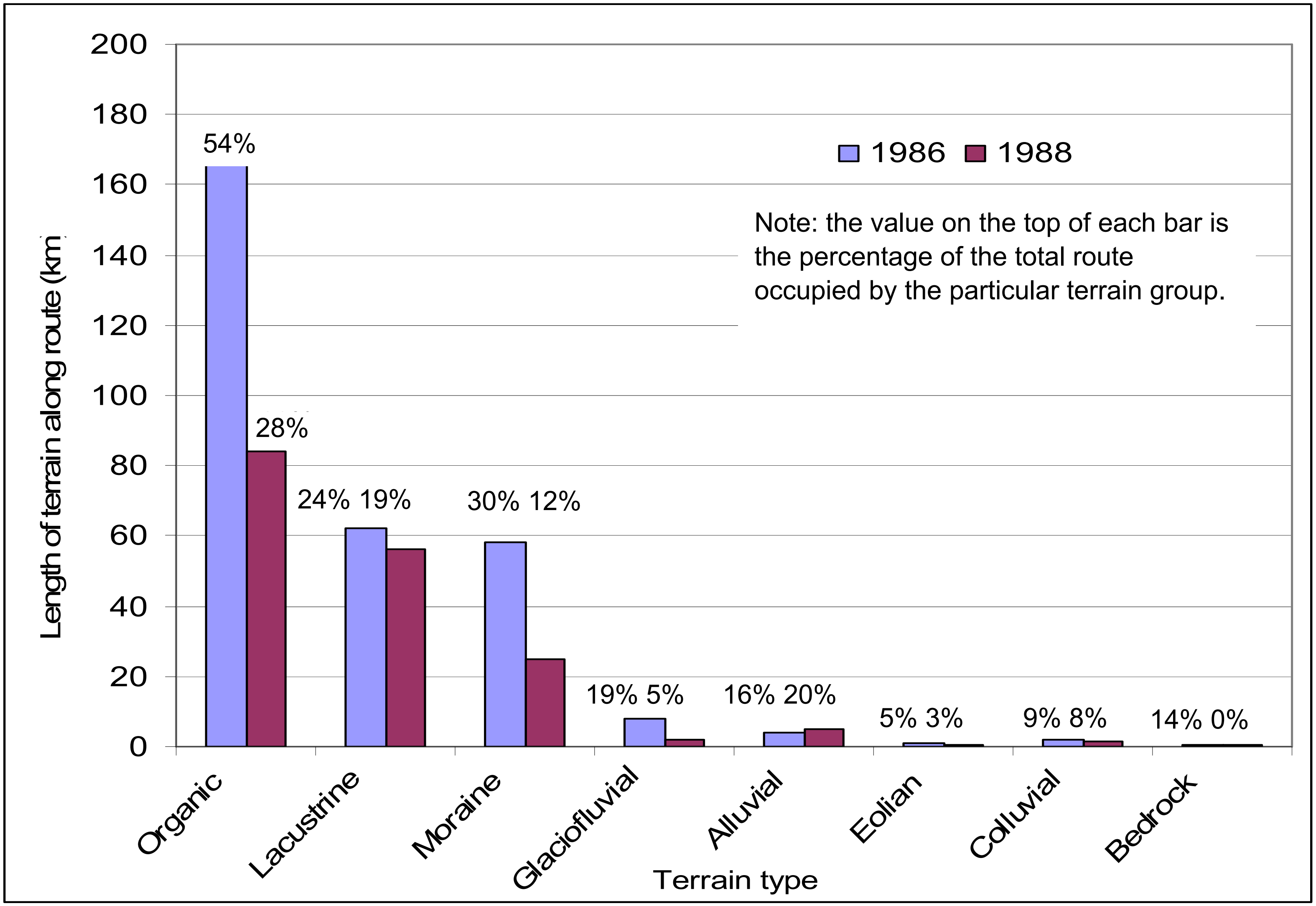


Figure 5-64: Extent of ditchline subsidence on each of major terrain types along the pipeline route.

6.0 ECONOMIC ASPECTS

Construction of the 868 km pipeline and the three 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 labour 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 (1985 dollars).

Maintenance/monitoring costs for 1994 and 1995 were $2.92 and $0.32 million, 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 labour 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 was 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 kilometres, 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. Pipe rupture or loss of service will occur at a strain much higher 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 (up-heaval) 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 the 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 22 years of pipeline operation and nearly 25 years of right-of-way disturbance have been generally similar to, or greater than the predicted 25 to 30 year design values. Although amount of thaw settlement may be greater than predicted, there is 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 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 lead to some problems interpreting the conditions close to the pipe. In the future, it would be recommended that the pipeline be accurately 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 meaningful data at the thaw front. Enbridge has, as needed, installed new deeper instrumentation to address this problem.

In future projects, some instrumentation 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 right-of-way 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 this pipeline. The formation of wrinkles at a few locations has necessitated the installation of additional instrumentation to assess the geotechnical processes at work. Creeping slopes, often associated with pipe wrinkles or pipe strain is an issue requiring additional study.

Some temperature monitoring cables have not been maintained, and are not currently being read. As necessary, they should be repaired, with a resumption of regular readings, based on recommendations from the consultants.

Several small pipeline leaks have occurred and were repaired with only minor fluid loss from the pipeline. The potential risk of these types of leak re-occurring should be considered. It should be noted that no leaks have been because of 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 20 plus 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.

Thaw settlement along the right-of-way as an environmental impact could be studied by a LiDAR study (**Li**ght **D**istance **a**nd **R**anging). This technique provides a very accurate representation of the ground surface profile. Comparison of “off” right-of-way profiles to “on” right-of-way profiles will highlight the amount of settlement that has developed since initial clearing.

Certain slopes 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 on-going 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 two-volume set by MacInnes et al, 1990 provides an excellent background to the terrain monitoring that took place in the first five years of the project. A large number of excellent photographs are included, and links to other reference material are provided.

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

Smith, S.L., Burgess, M.M., Riseborough, D., and Chartland, J. In press. Permafrost and terrain research and monitoring of the Norman Wells – Zama pipeline. April 1985 to September 2001. GSC Open File 5331. Natural Resources Canada/The Geological Survey of Canada.

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 NRCan, Enbridge, 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, Bryant, Look and Higgenbottom, 1981.) provides an environmental perspective of the project prior to final design and construction. Some of the research deficiencies perceived at that time are reviewed.

10.0 ACKNOWLEDGEMENTS

The author would like to thank Ms. M. Burgess and Dr. S. Smith of the Geological Survey of Canada for provision of data and extensive assistance with temperature, thaw depth, thaw settlement databases and records for the Norman Wells pipeline, and for advice on reference material. Enbridge Pipelines (NW) Inc. in Edmonton and Norman Wells provided extensive data on the operating temperature conditions at Norman Wells, and data from GEOPIG pipe monitoring. In addition, the pipeline maintenance staff in Norman Wells and Fort Simpson have collected most of the geotechnical data during operations. They deserve grateful acknowledgement for their diligence and abilities.

Finally, the authors are grateful to Dr. K. MacInnes and the Department of Indian Affairs and Northern Development in Yellowknife for suggesting the original monograph.

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12.0 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 upward 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 characteristics 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 ^{o}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.

APPENDIX A

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

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NORMAN WELLS PIPELINE PROJECT
ENBRIDGE-EMR/NRCan INSTRUMENTATION PROGRAM
(Updated to December 2006)

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

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 3 Canyon Creek North	19+200		T97-8(8) P21336(3.0)		
Slope 4 Canyon Creek South	19+400		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

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

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
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)
Slope 44	133+594	BH00-7	T00-2(15)		0.70 wood chips
	133+596	BH00-6	SI00-2(14.08)		0.70 wood chips
	133+600 (approx)	P	17614(1.0)		Depth of tip assumed to be 1.0 m
	133+604		P21332(4.0)		
	133+605 (approx)	P	[17613 (4.57)]		Depth of tip assumed to be 4.57 m
	133+607		T97-1(8.0)		
	133+611	BH00-5	SI00-1(14.17)		0.60 wood chips
Slope 45	133+747	-	17611 (4.57)	IRC	Depth of tip not confirmed. Assumed to be 4.57 m
Unnamed S	133+744	-	17612 (4.57)		Depth of tip not confirmed. Assumed to be 4.57 m
	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		Depth assumed to be 0.3 m
	133+765	BH00-9	T00-1(13.5)		1.68 m wood chips
		BH00-8	[SI00-3(16.28)]		1.68 m wood chips
		BH04-1	SI04-01(19.2 m below wood chips)		0.6 m wood chips
		BH04-2	SI04-02 (20.4)		Off Right-of-way (“west” side)
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
Little Smith Meander	158		SI-01	SI-01 (16.8)	Slope indicator on “west” side of right-of-way
			SI-02	SI-02 (16.8)	Slope indicator on “west” side of right-of-way

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Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information	
Slope 48B	160+174.5	P	17610 (3.55)	IRC/T		
Little Smith River	160+181.5	P	17609 (3.66)			
S	160+206	T	T91-3 (10.0)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)			
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

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 63, Steep S	195+000 (approx) 195+010 (approx) 195+011 195+100 195+120	SI T P P T SI T P SI T	GSC-SI06-01 (8) GSC-T-01 (8) P58743 (8) P21367 (6) T98-63 (11) SI-06-02(20.7) GSC-T-03 (20) P58740 (10) SI06-01 (25.3) GSC-T-02 (20)		Toe of slope, west side. GSC Instrumentation Toe of slope, west side. GSC Instrumentation Near crest of slope, west side Near crest of slope, west side. GSC Instrumentation. Crest of slope, east side. Crest of slope, east side. GSC Instrumentation.
Slope 64, Unnamed N	197+022 197+023 197+049	P-2A P-2 P-1	6122(5.80)S(4.0) 6134(16.15), C(16.15) 6143(14.02), [6109(6.86)], S(5.0)	UFT	
Slope 65, Unnamed S	197+132 197+159 197+161	P-1 P-2 P-2A	[6112(8.08)], S(5.5) 6138(16.00), S(8.3) 6117(10.67), S(5.2)	UFC/UFT	
Slope 68, Slope S	230+949 231+019 231+089	- - -	S(1.2) S(1.2) S(1.2)	IPT/UFT	2 standpipes in ditch 2 standpipes in ditch 2 standpipes in ditch
EMR-85-7A	271+231	T-1 T-2 T-3 T-4 - G-1 PT	85-7A-T1(5.0)Y1[3] 85-7A-T2(5.0)Y1 85-7A-T3(20.0)Y1 85-7A-T4(20.0)Y1 HA108(20)Y1, HA111(100)Y1 76 mm PVC(20.1) 85EPT1Y1	IRC/IRT	 0.9 m depth of cover

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Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 73, Unnamed N	271+442 271+458 271+459 271+459 271+490 271+491 271+491 271+519 271+524 271+540	T-3 P-1 T-2 - - T-1 - - - -	85T6(5.9)A, TA16(0.9)A 6105(2.90), S(2.3), SP(1.40) 85T3(6.4)A, TA3(1.2)A CT1(1.42)A, 85CT2(1.42)A P14011 (2.2) 85T1(6.15)A, TA18(1.2)A EMR-91-3536(1.8) EMR-91-3539(1.8) EMR-91-3540(1.8) T91-2 (4.5) P14012 (3.8) 119(0.90)A	IRC	Previous 0.91 m wood chips 1.42 m wood chips 1.42 m wood chips Horizontal strings 2.2 m below top of wood chips 1.10 m wood chips 1.45 m wood chips 0.5 m west of pipe; 1.8 m below ground surface 0.25 m west of pipe; to base of wood chips 0.25 m west of pipe; 1.8 m below ground surface 4.5 m below top of wood chips 3.8 m below top of wood chips 1.00 m wood chips
Slope 74, Unnamed S	271+779 271+780 271+802 271+803 271+819 271+819 271+909	T-1 P-1 P T T-2 - - T-3	85T9(6.45)A, 6103(2.75)S(2.5),SP(1.75) P22849 (3.8) T97-12(8) 85T11(6.3)A, TA5+85TA16(1.55)A 85T2(0.6)A T97-12(8.0), P22849(3.8) 85T13(5.0)A	IRC/IRT	Previous 1.4 m wood chips 1.75 m wood chips 1.4 m wood chips 1.75 m wood chips 1.6 m wood chips 1.60 m wood chips Horizontal string No wood chips
EMR-85-7B/ IPL-PSS	271+986	- T-1 T-2 T-3 T-4 - - G-1 PT	HA132(10)Y1 [5-7B-T1(5.0)Y1] [85-7B-T2(5.0)Y1] 85-7B-T3(20.0)Y1 85-7B-T4(20.0) HA110(20)Y1 HA129(10)Y1 76 mm PVC (20.3) 85EPT3Y1	IRC/IRT UFT (in cleared area)	UF hole at interface Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 F hole at interface Abandoned by GSC/INAC - Sept 1996 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+144 279+145 279+169 279+170 279+197	- T-1 - BH00-3 BH00-4 T-2 - T-3 BH00-1 BH00-2	85T8A 85TA14(1.29)A [85TA3(1.8)] [85T17(1.0)] T00-4(15) P25359(5.79) HT147(1.15)A [85TA10+TA1(1.64)] [85PT1-4(0.6)] 85TA13(1.34)A T00-3(15) P25358(6.55)	IRC	Side cut string 1.10 m wood chips 1.8 m wood chips Horizontal string 0.76 m wood chips 0.76 m wood chips 1.00 m wood chips 1.65 m wood chips Horizontal string 1.00 m wood chips 0.76 m wood chips 0.76 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+731 286+738 286+739 286+740 286+740 286+746 286+756 286+757 286+763 286+764 286+764.5 286+765 286+772 286+773 286+788 286+804 286+804 286+819 286+820 286+821 286+822 286+822 286+835 286+836 286+858	- T-1 P-1 - - - - - - - - - - - T-2 - - - - - - - - - T-3	82-21(0.8)A 85T7(4.7)A, 85TA2+85TA8(1.8)A 6120(2.83), S(2.4), SP(1.83) 81-16(1.0)A 82-4(2.8)A 82-8(1.2)A, 82-23(0.8)A 82-24(0.8)A 82-20(0.8)A 82-15(0.9)A 82-1(4.0)A 82-2(4.0)A 82-3(2.4)A 82-18(0.8)A, 82-10(1.1) 82-22(0.8)A, 82-9(1.1) T9(4.75)A, 85TA6(1.3)A 82-26(0.2)A, 82-12(1.3)A[3] 82-17(0.6)A 82-14(1.2)A 82-131(1.2)A 82-5(3.0)A 82-6(1.2)A 82-27(1.0)A 82-25(0.6)A 82-19(0.2)A, 82-11(1.2)A T18(4.7)A, 85TA15(1.3)A	IRC	Small hot area 1.83 m wood chips 1.83 m wood chips Small hot area Small hot area Small hot area Main hot area Main hot area Main hot area Main hot area Main hot area Main hot area Main hot area Main hot area 1.42 m wood chips Experimental area Experimental area Experimental area Experimental area Experimental area Experimental area Experimental area Experimental area Experimental area 1.52 m wood chips
Slope 84 Unnamed Creek South	311	T-9 T SI SI SI	T99-1(15)Y3 T99-2(14.4)Y3 [99-S1(13.5)] [99-S2(14.0)] SI04-01 (16.8)		All depths are from below base of wood chips. Instrumentation installed in March 1999 following exposure of the pipe and the placement of sleeves over two wrinkles. Installed off right-of-way, “west side”
FH 8	311+739	PT	PT2-4A	-	0.95 m depth of cover

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 88, Unnamed Creek South	313.6	SI01-1 to SI01-7 T01-1 to T01-7	SI01-1(11.1), SI01-2(11.2), SI01-3(20.2), SI01-4(20.6), SI01-5(20.7), SI01-6(26.5), SI01-7(26.5) T01-1(15), T01-2 to T01-5(20), T01-6(16.5), T01-7(15) Note: All instrumentation installed on this slope was provided by the Geological Survey of Canada.	IRC	SI01-1, SI01-3 and SI01-7 (and the corresponding thermistors) were installed on the right-of-way at the bottom, midslope and crest of the slope, respectively. SI01-2, SI01-4 and SI01-6 (and the corresponding thermistors) were installed off the right-of-way on the west side at the bottom, midslope and crest of the slope, respectively. SI01-5 (and the corresponding thermistor) were installed off the right-of-way on the east side at midslope.
Slope 92, Unnamed Creek South	318+	T SI T T S1 S2 SI	T97-13(8.0)Y3 [SI-4 (11.2)] T99-3(10.6)Y3 T99-4(12.0)Y3 [T99-S1(11.4)] [99-S2(12.8)] SI04-01 (9.2) SI04-02 (14.3)	-	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 were removed in February 1999 as part of a scheduled pipe replacement program. T99-3, 4 and SI99-S1, S2 were installed in March 1999 following the scheduled pipe replacement. Installed on right-of-way, terminated in gravel layer Installed off "west" side of right-of-way, near SI-4
Slope 99, Smith S	325+338 325+388 325+389	T-1C T-2 P-1	T11(4.2)A, [85TA9(1.29)A] T6(5.0)A, 85TA11(1.27)A 6113(2.80), S(2.2), SP(1.27)	IRC	1.63 m wood chips 1.27 m wood chips 1.27 m wood chips
IPL PT7	325+583	PT	85PT1-2A	-	0.95 m depth of cover
Slope 109	352+010 351+014 352+014	- - -	P14049 T91-6 Y1 EMR-91-3628(1.8) EMR-91-3629(1.8) EMR-91-3653(1.8)	-	2.7 m below top of wood chips 6.0 m below top of wood chips 0.5 m west of pipe; 0.3 m in wood chips 1.5 m below ground surface 0.25 m west of pipe;/ installed from 1.9 m to 3.7 m below ground surface 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

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 112, RBTM N	352+560 352+560 352+613 352+613 352+615 352+621	T-2 P-2 T-1 P-1 - -	T15(5.0)A 6130(2.0) T10(5.8)A 6127(2.8), SP(0.8) T91-1Y1 P14010	IRC	 0.8 m wood chips 0.8 m wood chips 6.0 m below top; of wood chips 4.5 m below top of wood chips
FH 9	359+538 359+398	PT PT	PT1-2A PT2-8A	- -	2.0 m depth of cover 0.76 m depth of cover
FH 10	403+823 403+988	PT PT	PT1-2A PT2-8A	- -	0.9 m depth of cover 0.9 m depth of cover
IPL-PSS	469+961 469+988	5 6	HA131(10.0)Y1 HA130(10.0)Y1	UFS/UFC IPC	UF hole at interface F hole at interface
EMR-84-4A	477+988	T-1 T-2 T-3 T-4 G-1 PT -	[84-4A-T1(20.0)Y1[3]] 84-4A-T2(20.0)Y1 [84-4A-T3(5.0)Y1] [84-4A-T4(5.0)Y1] [76 mm PVC (5.6)] EMR-1A 118A	UFS/UFC -	 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Abandoned by GSC/INAC - Sept 1996 0.9 m depth of cover Ditch thermistor
EMR-84-4B	478+116 478+838	T-1 T-2 T-3 T-4 G-1 PT	[84-4B-T1(20.0)Y1] [84-4B-T2(5.5)Y1] [84-4B-T3(5.5)Y1] 84-4B-T4(20.0)Y1 [76 mm PVC(13.5)] PT1-9A	UFS	Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Abandoned by GSC/INAC - Sept 1996 0.9 m depth of cover

Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
Slope 142, Mackenzie S	529+727 529+743 529+753 529+760 529+764 529+776 529+777 529+778	- T-1 P-2 - - - T-2 P-1 -	P14048 T22(4.5)A, [85TA13(1.3)] HT144(0.95)A 6135(2.83), S(2.4) 85CT1A P14052 (2.2) T91-7Y1 85T10(4.75)A, [85TA14(1.3)] HT145(0.95)A 6141(2.45), 23070(2.45) S(2.1)SP(1.45) 85CT4A	IPC	5.2 m below top of wood chips 1.40 m wood chips Side cut string 2.89 m below top of wood chips 6.0 m below top of wood chips 1.25 m wood chips 1.25 m wood chips 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 T-2 T-3 T-4 G-1 PT DT	[85ET2(5.0)Y1] [85ET1(5.0)Y1] 85EDT8(20.0)Y1 85EDT5(20.0)Y1 [76 mm PVC(20.3)] 85EPT8Y1 [115A]	IPS/IRC/UFC	Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Abandoned by GSC/INAC - Sept 1996 0.9 m depth of cover Ditch thermistor. Abandoned by GSC/INAC Sept 1996
EMR-85-8B	588+158	T-1 T-2 T-3 T-4 G-1 PT	[85ET4(5.0)Y1] [85ET5(5.0)Y1] 85EDT1(20.0)Y1 85EDT6(20.0)Y1 [76 mm PVC (20.4)] 85EPT7Y1	PT/UFT	Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Abandoned by GSC/INAC - Sept 1996 0.9 m depth of cover
EMR85-8C	558+333	T-1 T-2 T-3 T-4 G-1 PT	85ET6(5.0)Y1[3] 85-8C-T2(5.0)Y1 85EDT3(20.0)Y1 85EDT2(20.0)Y1 76 mm PVC(20.3) 85EPT12Y1	IRC/UFC	Site abandoned by GSC/INAC - Sept 1998. Site too wet to access and remove instrumentation. 0.9 m depth of cover

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Site	Location As-Built Chainage	Borehole[1]	Instrumentation[2] Installed	Soil/Ice[4] Conditions	Other Information
EMR-85-9	583+339	T-1 T-2 T-3 T-4 PT DT	[85-9-T1(5.0)Y1] [85-9-T2(5.0)Y1] 85EDT9(20.0)Y1 85EDT4(20.0)Y1 85IPT9Y1 [116A]	BR -	 Removed by GSC/INAC - Sept 1996 0.9 m depth of cover Ditch thermistor. Abandoned by GSC/INAC Sept 1996
EMR-85-10A	588+276	T-1 T-2 T-3 T-4 G-1 PT	[85-10A-T1(5.0)Y1] [85-10A-T2(5.0)Y1] [85EDT10(20.0)Y1] [85EDT11(20.0)Y1] [76 mm PVC (5.6)] 85EPT4Y1	UFT/BR	Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 0.95 m depth of cover
EMR-85-10B	588+680 588+686 588+686	G-1A T-1 T-2 T-3 T-4 PT	[76 mm PVC(8.8)] [85-10B-T1(5.0)Y1] [85-10B-T2(5.0)Y1] 85-10B-T3(10.5)Y1 85-10B-T4(10.5)Y1 85EPT5Y1	PT/UFT/BR	Abandoned by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 0.95 m depth of cover
EMR-85-11	597+396	T-1 T-2 T-3 T-4 G-1 PT	[85-11-T1(5.0)Y1] (85-11-T2(5.0)Y1) 85-11-T3(12.0)Y1 85-11-T4(12.0)Y1 (76 mm PVC(14.2)) 85EPT11Y1	IPS/UFT	Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Abandoned by GSC/INAC - 1996 0.95 m depth of cover
EMR-85-12A	608+534 608+562 608+562	T-3A T-1 T-2 T-3 T-4 G-1 PT	(85-12A-T3A(16.4)Y1) [85-12A-T1(5.0)Y1] [85-12A-T2(5.0)Y1] HA135(7.5)Y1 85-12A-T4(12.0)Y1 [76 mm PVC(10.9)] 85EPT6Y1	UFC/UFT	Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Removed by GSC/INAC - Sept 1996 Abandoned by GSC/INAC - 1996 0.95 m depth of cover
IPL-PSS	608+672 608+694	1 3	HA133(6.7)Y1[3] HA134(7.35)Y1	UFC IRC	UF hole at interface F hole at interface

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

NCI

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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 kΩ
- Y1 = thermistor string fabricated with YSI 44033 thermistor beads, 0°C at 7.355 kΩ
- Y2 = thermistor string fabricated with YSI 44032 thermistor beads, 0°C at 94.98 kΩ
- Y3 = thermistor string fabricated with YS144007 thermistor beads, 0°C at 16.330 kΩ

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	

APPENDIX B

SITE MONITORING LOCATIONS AND PIPE SETTLEMENT SOURCES

Table B1. Site Descriptions

Site Number	Name	KP	Description at time of original installation
84-1	Pump Station 1	0.02	Widespread permafrost - Ice-rich silty clay
84-2	Canyon Creek		Previously cleared alignment, thaw sensitive slopes
	A	19.0	Widespread permafrost - Level terrain. Frozen till with low ice content.
	B	19.3	Widespread permafrost – East facing slop with 1 m wood chip layer at surface.
	C	19.6	Widespread permafrost – West facing slope, uninsulated site.
84-3	Great Bear River		Joint Enbridge site at thaw sensitive slope
	A	79.2	Stratigraphically complex ice-rich alluvial terrace deposits in widespread permafrost; slope base.
	B	79.4	Slope crest, lacustrine deposits with Aeolian veneer.
85-7	Table Mountain		Joint Enbridge site at thaw sensitive slope. Previously cleared alignment.
	A	271.2	Ice-rich lacustrine plain
	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	583.3	Pipeline previously traversed frozen ground. Unfrozen granular soils.
85-10	Mackenzie Highway South		Frozen – unfrozen interface.
	A	588.3	Helipad clearing in unfrozen terrain.
	B	588.7	Thin peat (2 m) with thin (3 m) permafrost.
85-11	Moraine South	597.4	Thin (<4 m) permafrost in helipad clearing.
85-12	Jean Marie River		Frozen – unfrozen interface.
	A	608.6	Thin unfrozen peat.
	B	608.7	Thick ice-rich peat plateau with 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

Site Number	Name	KP	Description at time of original installation
	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	819.5	Peat plateau preceded by unfrozen fen. Thick (5 m) ice-rich peat; 7 m permafrost.

Notes: The above are the principal study sites established in 1984 and 1985 during pipeline construction. In the 1990s some sites were de-activated and are no longer monitored. Additional key sites instrumented since initial construction are:

- Freeze-thaw/pipe soil study site at KP 2, established in July 1994.
- KP 182 forest fire burn area slope thermal investigations, established in August 1995.
- Short-term studies of hot spot were conducted in 1993 to 1995 at selected slopes.
- KP 314 (Slope 88) slope creep monitoring site, established in late 1990s.

Source: Burgess, 1995; PRTM contribution to 1995 annual report.

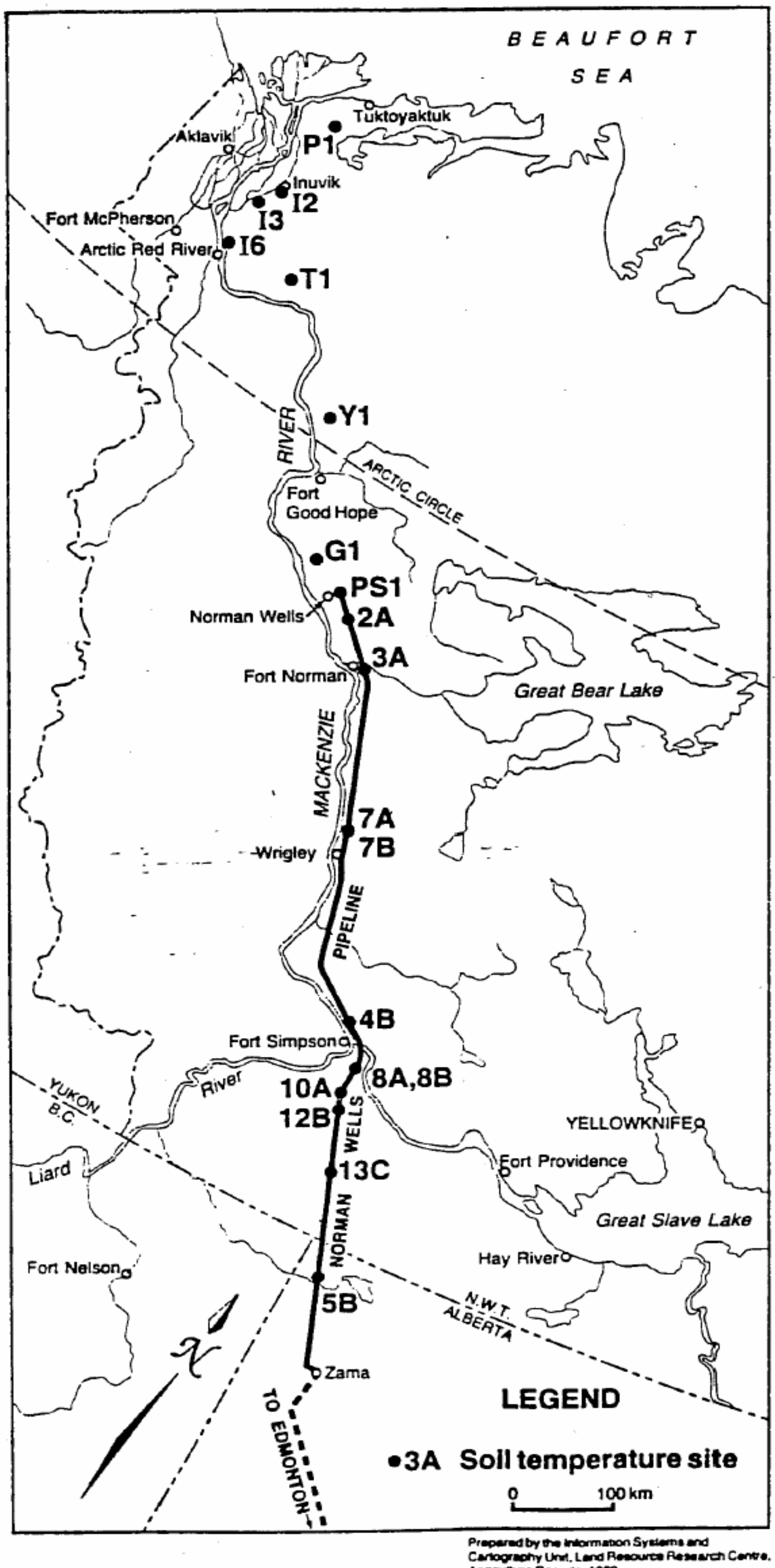

Figure B1.
Location of Soil temperature and climatic monitoring sites along pipeline route.