ENVIRONMENTAL SERVICES

PLANS
FOR DEALING WITH
FROST-HEAVE AND THAW SETTLEMENT

SUBMISSION 4-2

NOVEMBER, 1981

Foothills Pipe Lines (South Yukon) Ltd.

This document is one of a series of addenda prepared to meet information requirements placed on Foothills Pipe Lines (South Yukon) Ltd. by the Federal Environmental Assessment and Review Office. Addenda within the series are divided into seven sets of submissions dealing with separate subject areas:

1. Introduction to Addenda Submissions.
2. Project Description and Update for Addenda Submissions.
3. Alternative Routes.
4. Geotechnical, Hydrological, Design Mode and Revegetation Issues.
5. Fisheries, Wildlife and Scheduling Issues.
6. Issues Related to Pipeline Facilities.
7. Other Issues.

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1.0 INTRODUCTION

The effect of permafrost conditions on pipeline construction and operation has been the most important and contentious issue addressed during the planning phase of the Alaska Highway Gas Pipeline. The exchange of information between Foothills Pipe Lines (South Yukon) Ltd. (the Project) and regulatory authorities, including the Environmental Assessment and Review (EAR) Panel, has been extensive.

In its application to the National Energy Board in 1976, Foothills described the physiographic regions crossed by the pipeline as well as the surficial geology along the route and proposed a buried pipeline system operating in both "chilled" and "warm" modes; the former was designed to overcome the problem of thaw subsidence in ice-rich permafrost soils.

After review of material submitted in 1976, the EAR Panel concluded that the pipeline could be constructed only if extensive and detailed soils information (particularly with reference to permafrost conditions) was obtained, if adequate mitigative measures were developed to overcome difficulties associated with permafrost terrain and, if an elevated mode was considered as a mitigative approach. The Panel suggested mitigative measures would of necessity avoid significant changes in drainage, erosion and aesthetic conditions.

In responding to the Panel's requirements during the preparation of the Project's Environmental Impact Statement (EIS), Foothills supplied a further description of physiographic regions and surficial geology along the pipeline route together with a kilometre by kilometre description of the route in Yukon in tabular form. The table used presented terrain types crossed by the pipeline in sequence of occurrence, noting the lineal distance crossed in each case, the composition and stratigraphy of each deposit, permafrost conditions and ice content, relief and slope, drainage, depth to bedrock, under lying bedrock type, potential to supply borrow material and pertinent engineering considerations associated with each terrain unit. In addition, results of drilling programs completed to that date were supplied as an annex to the EIS in the form of laboratory reports.

Information on the design approach to permafrost conditions was supplied in the form of general discussions of design requirements for pipe placement in both the warm and chilled modes, the kinds of soils information being collected and how it was to be incorporated in the designs. Also supplied were preliminary designs being considered to overcome potential frost-heave and thaw-settlement problems. These latter designs were described in text and as drawings. In response to questions raised immediately prior to and during hearings in 1979 the project responded by supplying:

1. detailed information on terrain types and specifics of design in example sections of line (KP 80 to 90 and KP 225 to 240) including sheets from the Project's Geotechnical Atlas;

2. details of the number of frozen/thawed transitions anticipated;

3. details of allowable differential heave and pipe curvature;

4. the results of studies completed by various authors indicating the degree of accuracy that can be expected from geothermal analyses;

5. a discussion of the likely kind and magnitude of geothermal effects arising from climatic changes along the pipeline route;

6. a discussion of the differences and similarities between designs developed by Northwest Alaskan Pipeline Company and those of Foothills;

7. a discussion of the experience with permaforst conditions during operation along the Pointed Mountain and Alyeska pipeline systems;

8. details of a field survey initiated to locate springs, seepage areas, and naturally occurring aufeis along the pipeline route in Yukon Territory;

9. a discussion of information available respecting naturally occurring icings and those that might be expected to occur during construction;

10. a discussion of available approaches for predicting and dealing with construction-induced icings and subsurface freezing due to pipe operation;

11. a discussion of the likely geothermal events arising from abandonment of the pipeline with special reference to subsidence; and

12. a discussion of the procedures used in determining the presence of permafrost.

After review of the information submitted in the EIS, that supplied by way of deficiency responses and that presented during technical hearings, the Panel requested additional information of several kinds. Briefly the Panel requested:

1. a description of, and results from, a survey to delineate permafrost along the entire route;

2. information about other pipelines operating in permafrost conditions;

3. a demonstration of the feasibility of large diameter pipelines in permafrost based on precedent;

4. a description of a geothechnical program aimed at solving problems associated with frost heave and thaw settlement to be submitted well in advance of hearings;

5. "knowledge of the extent of terrain information" and detailed examples of solutions including, specifically, sections at Snag and Mirror creeks and other "problem areas" such transitions between frozen and unfrozen soils;

6. details of insulation or other techniques planned to control frost heave and thaw settlement;

7. details of design in both cross and longitudinal section;

8. information on tolerable differential heave and pipe curvature in relation to metallurgical properties of pipe;

9. "convincing documentation" of pipe integrity when exposed to frost heave;

10. results of an error analysis with respect to geothermal calculations; and

11. information on impacts associated with subsidence following abandonment of the pipeline.

Subsequently, during meetings between Panel members, Foothills personnel and representatives of the Northern Pipeline Agency (NPA), Panel requests were clarified to include only a report prepared by Foothills dealing with the frost-heave/thaw-settlement study program. This decision was based on discussions of the role of the Northern

Pipeline Agency in relation to that of the Panel. Agreement was reached that the Northern Pipeline Agency was responsible for review and approval of final designs, including much of the detailed engineering information pre-requisite to approval. The role of the Panel was defined as being that of public review of environmental impact statements and the adequacy of preliminary environmental planning for the Project. It was decided that the frost-heave/thaw-settlement study program being developed and executed by Foothills would allow review of procedures and preliminary plans by the Panel. Review of detailed design information will be completed by the Northern Pipeline Agency as this information becomes available.

In responding to the most recent Panel request, two reports dealing separately with the topics of frost-heave and thaw-settlement are being submitted. These reports make up Parts 2 and 3 of this submission. The reports submitted are in fact revisions of the frost-heave/thaw-settlement study program discussed during joint meetings between Panel, Foothills and NPA representatives. The original document dealing with both topics has been separated into reports.

Readers should note that the two reports included in this submission were prepared for Foothills by consultants for eventual submission to the NPA. The reports have been duplicated in their entirety without changes.

2.0 GEOTECHNICAL REPORT ON FROST HEAVE

DESIGN APPROACH

HARDY ASSOCIATES (1978) LTD.
CONSULTING ENGINEERING & PROFESSIONAL SERVICES

FROST HEAVE DESIGN APPROACH

PREPARED FOR

FOOTHILLS PIPE LINES (SOUTH YUKON) LTD.

CALGARY, ALBERTA

BY

HARDY ASSOCIATES (1978) LTD.

CALGARY, ALBERTA

AUGUST, 1981

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SECTION 4(b) - FROST HEAVE DESIGN APPROACH

1.0 INTRODUCTION AND SUMMARY

The pipeline will be operated continuously at temperatures below $0^{o}C$ for the first 60 km or so of the pipeline route between the Alaska/ Yukon border and Station 311. A conventional buried pipe would tend to freeze a "bulb" of soil around the pipe over its operating life. If the soil has a large percentage of the fine particles and free access to water, segregated ice can form within the frozen soil zone, and vertical displacement or heave of the pipe can result. Excessive heave is not desirable, as it may occur differentially, and cause overstressing of the pipe.

This document reviews the various components of the frost heave design process, and in particular assembles the extensive data base available for evaluation of frost heave effects in different soils. This information, when coupled with the known extent of unfrozen and frost susceptible soils, provides a good indication of the extent of the frost heave problems, and the associated remedial measures that might be required. In the light of the conservative approach to dealing with any soils having appreciable fines content, the need for future testing or verification of design modes is reviewed.

1.1 Components of Frost Heave Design

The primary elements involved with assessing the magnitude of the frost heave problem are as follows:

(a) Initial Thermal Condition, i.e. extent of unfrozen ground.

(b) Field and Laboratory Frost Heave data on different soil types, including frozen soils.

(c) Thermal predictive model to predict:

   i) depth to frost advance for a given pipe (and possibly insulation) configuration;

   ii) effectiveness and design of measures to inhibit or prevent frost heave.

(d) Theory to predict frost heave. In the absence of rigorous theory, a semi-emperical model is currently being used.

These components are reviewed, in order to place the importance of each in perspective.

1.2 Mitigation and Verification

The measures for frost heave mitigation are then discussed, and the likely extent of these are indicated. Based on the limited total extent of the anticipated frost heave mitigation, and the level of confidence in the conservative approaches adopted in handling frost susceptible soils, no further field verification is considered necessary.

In summary, it is considered that:

(a) Where there is no heave or little potential for heave, no mitigation is required.

(b) Otherwise, unless a proven heave predictive model is developed, an above grade mode will be used.

2.0 EXTENT OF UNFROZEN GROUND AND FROST SUSCEPTIBLE SOIL

Foothills Pipe Lines (South Yukon) Ltd. have carried out extensive programs of terrain analysis, field reconnaissance, borehole drilling and geophysical surveys in the northwest part of the pipeline route in the Yukon. As this section of pipeline is within the zone of chilling, particular attention was paid to delineating unfrozen ground and its properties. This information is presented in the Geotechnical Atlas, together with information on the available boreholes and geophysical traverses.

Based on a tabulation of the lengths of pipeline route within each terrain unit or length of frozen or unfrozen ground, a file has been created and stored on a minicomputer. This file can be accessed to list lengths of unfrozen ground, the corresponding landform or terrain unit and some information on available data on soil type. More definitive data on soil type and groundwater conditions can be obtained by consulting the logs of boreholes drilled at a specific location. If a borehole has not been drilled at a specific location, data from other boreholes in the same terrain unit can be consulted, together with the geological interpretation of subsurface stratigraphy shown on the Geotechnical Atlas.

Table 1 presents the route information on all areas of unfrozen ground traversed by the chilled pipe in the Yukon. It can be assumed conservatively that a sufficient supply of groundwater may be available to supply a frost susceptible soil, and so this factor need not receive further attention.

It is seen that a total of 24% of the route, or 15.65 km of the pipeline between KP 0 and Station 311 is underlain by unfrozen ground. This is a conservative (high) estimate, as areas where significant doubts exist about the thermal condition of the ground have been assumed unfrozen. A total of 4.3 km of this length is underlain by the clean gravel floodplains of Beaver Creek and other small creeks. As will be seen later, these materials are considered to be essentially non-frost susceptible at depth, and may not require any special mitigation for

TABLE 1

OCCURRENCE OF UNFROZEN GROUND AND BOREHOLE DATA BASE BETWEEN KP 0 AND STATION 311

OCCURRENCE from Kp	to Kp	LENGTH (m)	TERRAIN TYPE	DESIGN MODE (Preliminary)	BOREHOLE INFORMATION	KP
3.50	3.70	200	$r.fC_B/R$	Deep Bury	80-11-001	3.56
					80-11-002	3.59
					80-11-003	3.62
6.92	7.02	100	$fA_B/g.tM_M$	Deep Bury	80-11-007	6.96
					78-1-3	6.96
					78-A-3	6.97
					80-11-008	7.01
7.37	7.46	90	fAT	Buried Insulation	79-A-1	7.38
7.46	8.39	930	fAP(K)	Above Grade	79-B-1	7.69
					80-11-009	8.36
8.39	13.05	4660	fAP	Above Grade	80-11-010	8.63
					77-3-1	8.71
					77-3-5	9.13
					77-3-4	9.39
					77-3-3	9.48
					77-3-2	9.56
					79-B-2	10.28
					80-11-011	11.99
					80-11-012	12.34
13.05	13.76	710	fAP(K)	Above Grade	79-A-2	13.72
13.76	14.33	570	fAT	Above Grade	79-A-3	13.93
14.33	14.69	360	fAT	Above Grade		
14.69	15.42	730	fAP	Above Grade	79-B-4	15.01
					80-11-013	15.17
17.94	21.33	3390	gGP	Buried Insulation (Special design)	80-11-015	18.09
					80-11-016	18.40
					80-11-017	20.08
					78-B-5	21.00
					80-01-07	21.06
21.33	23.09	1760	gGP	Deep Bury	80-11-019	21.52
					79S-A-1	22.28
					80-11-020	22.38

TABLE 1 (Continued)

OCCURRENCE OF UNFROZEN GROUND AND BOREHOLE DATA BASE BETWEEN KP 0 AND STATION 311

OCCURRENCE from Kp	to Kp	LENGTH (m)	TERRAIN TYPE	DESIGN MODE (Preliminary)	BOREHOLE INFORMATION	KP
23.09	23.90	810	f/gAT	Deep Bury	80-11-08 78-BC-1 79S-A-2 78-BC-2	23.30 23.60 23.64 23.66
23.90	24.14	240	f/gAP(A)	Deep Bury	79-B-6	24.13
24.14	24.19	50	gAC(A)	Deep Bury		
24.19	24.66	470	f/gAP(A)	Deep Bury	79-A-4 80-01-09	24.23 24.63
30.74	30.77	30	fAB/gGP	Deep Bury		
54.55	54.68	130	f/aAP(A)	Deep Bury	78-A-17	54.60
59.01	59.19	175	f/gAF	Standard		
59.19	59.36	170	f/aAP(A)	Deep Bury	79-A-7 79-A-19 79-A-8	59.21 59.27 59.29
61.02	61.09	75	fAB/GP	Deep Bury		

SUMMARY

From KP 0.00 to 64.70 there is 24% unfrozen terrain.

Total Number of Occurrences = 20.

frost effects other than deep burial to cross a stream or creek bed. The remaining total of 11.35 km is assumed at our present state of understanding to require some frost heave mitigation. This estimate may be revised later as the frost heave test data base improves.

A total of 42 boreholes have been drilled in the 20 potentially unfrozen areas to evaluate the subsurface conditions.

3.0 FROST HEAVE TEST DATA

Foothills Pipe Lines (Yukon) Ltd. and Northwest Alaskan Pipeline Company have been carrying out frost heave testing of various kinds for several years. In addition, parallel efforts by government agencies have provided a great deal of additional data on other soil types, and different testing procedures. These data will be reviewed, in addition to the evidence for frost heave in frozen ground.

Many variables might be used to describe a soil texture and particle size distribution. In the following, Casagrande's grain size parameter, the percentage finer than 0.02 mm, is used as a simple measure of the particle size of a soil.

Frost heave prediction can involve the measurement of heave rate, and the use of this heave rate as an index to frost heave behaviour over many years. Alternatively, the cumulative ice-segregation ratio (total heave divided by the depth of soil frozen) can be used as a measure of total frost heave. According to its definition, frost heave can be estimated in an approximate fashion by multiplying this parameter by the predicted depth of frozen soil. Both of these parameters are known to depend on the rate of freezing and pressure, and neither should be used directly for frost heave predictions without accounting for the highly variable rate of freezing and pressure that might be experienced beneath a chilled pipe.

In plotting data from various sources, the heaving characteristics at a pressure of 50 kPa has been selected. This is the effective stress on the base of the frost bulb after a short period of operating and is lower than the average pressure that the frost bulb will experience over its operating life.

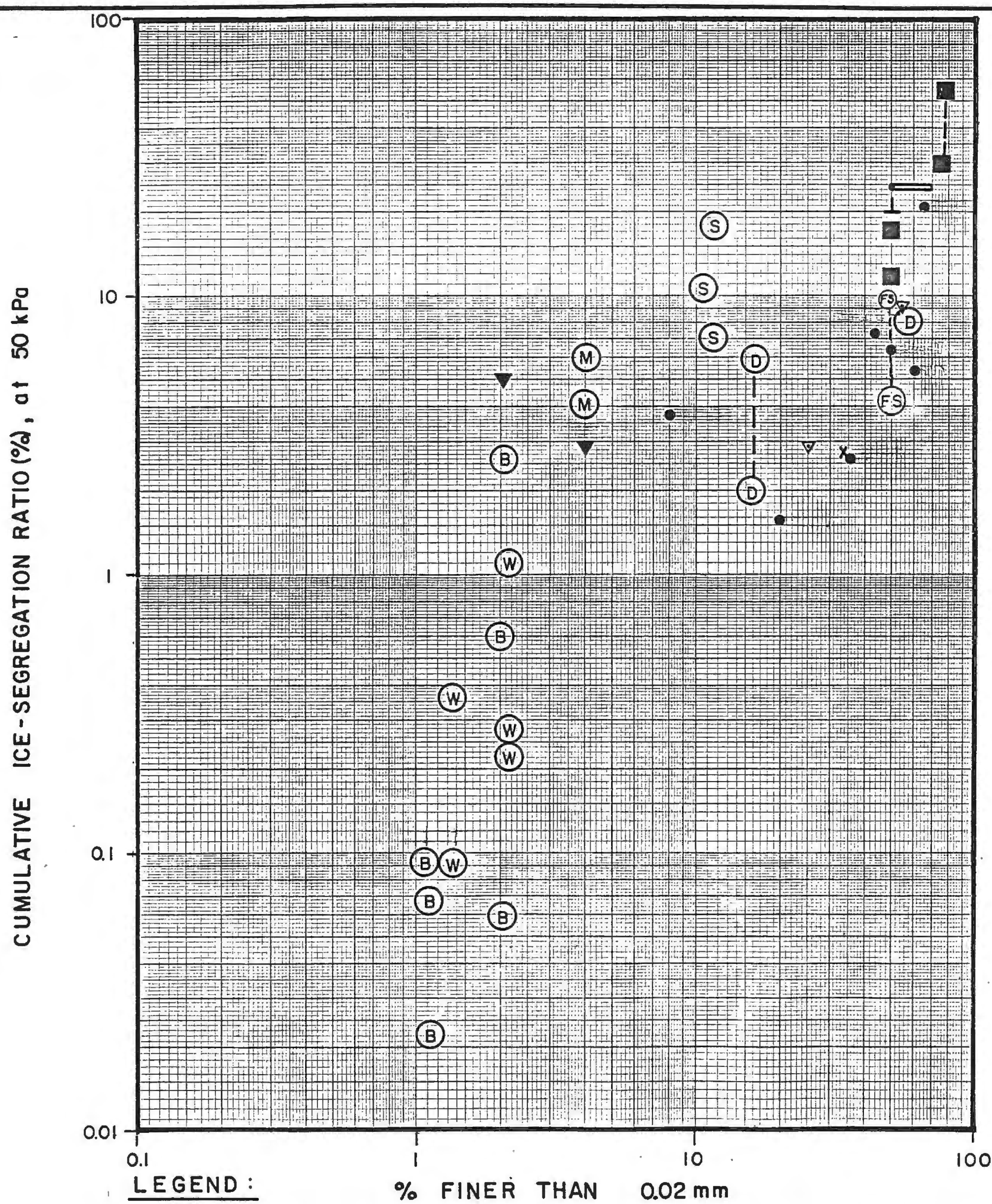
In evaluating frost heave rates, another important variable is the rate of frost advance, over the time interval when the heave rate has been defined. The initial, higher heave rate measured in a frost heave test is not considered appropriate here for comparing soil behaviour as the rate of frost advance during this period could be 20-50 mm/day or greater. This rate of frost advance would not be maintained under field pipeline conditions for any significant length of time. Many of the laboratory tests reviewed below lasted for 10,000 minutes or longer, at which time the rate of frost advance was typically in the range of 2-10 mm/day. Therefore, in order to compare the results for different soils and laboratory conditions, the heave rate at a time of 10,000 minutes was evaluated. in cases where the test duration was less than this, the heave rate at the end of hte test was determined. This is the case for the HAL (1979a), and NRC test series.

In almost all test series, the more realistic heave rates of Table 2 are different than the somewhat higher initial heave rates tabulated in the initial reports.

3.1 Laboratory Frost Heave Data

One of the earliest extensive frost heave testing programs at higher overburden stresses is reported by Linell and Kaplar (1959). A variety of soils were tested using the Standard U.S. Army CRREL procedure, at an approximate frost advance rate of 1.2 cm/day. These data can be interpreted to yield a heave rate and a total ice-segregation ratio at an overburden pressure of 50 kPa, and are shown plotted on Figures 1 and 2.

Mr. E. Penner of the National Research Council of Canada has tested many soil samples in co-operation with Foothills Pipe Lines.



LEGEND:

Ⓕ Fairbanks Silt, Penner & Goodrich, 1980	Ⓓ Foothills Dry Creek Bulk Samples
Ⓒ Calgary Silt, Penner & Goodrich, 1980	⊥ Foothills Yukon Test Plates
• CRREL Soils (1959)	Ⓦ Foothills White River Gravel
+ Parè et.al. - James Bay Till	Ⓑ Foothills Beaver Creek Gravel
Ⓜ Foothills Series I - Mirror Creek	═ Foothills Full Scale Test Facilities
▽ NRC Fairbanks Silt	⊥ Foothills Calgary Test Plates
▼ NRC White River and Beaver Creek	■ Foothills Calgary Lab Tests
Ⓢ Foothills Snag Creek	× Fairbanks Silt, Aitken, 1963
	(FS) Fairbanks Silt, NWA Full Size Sections

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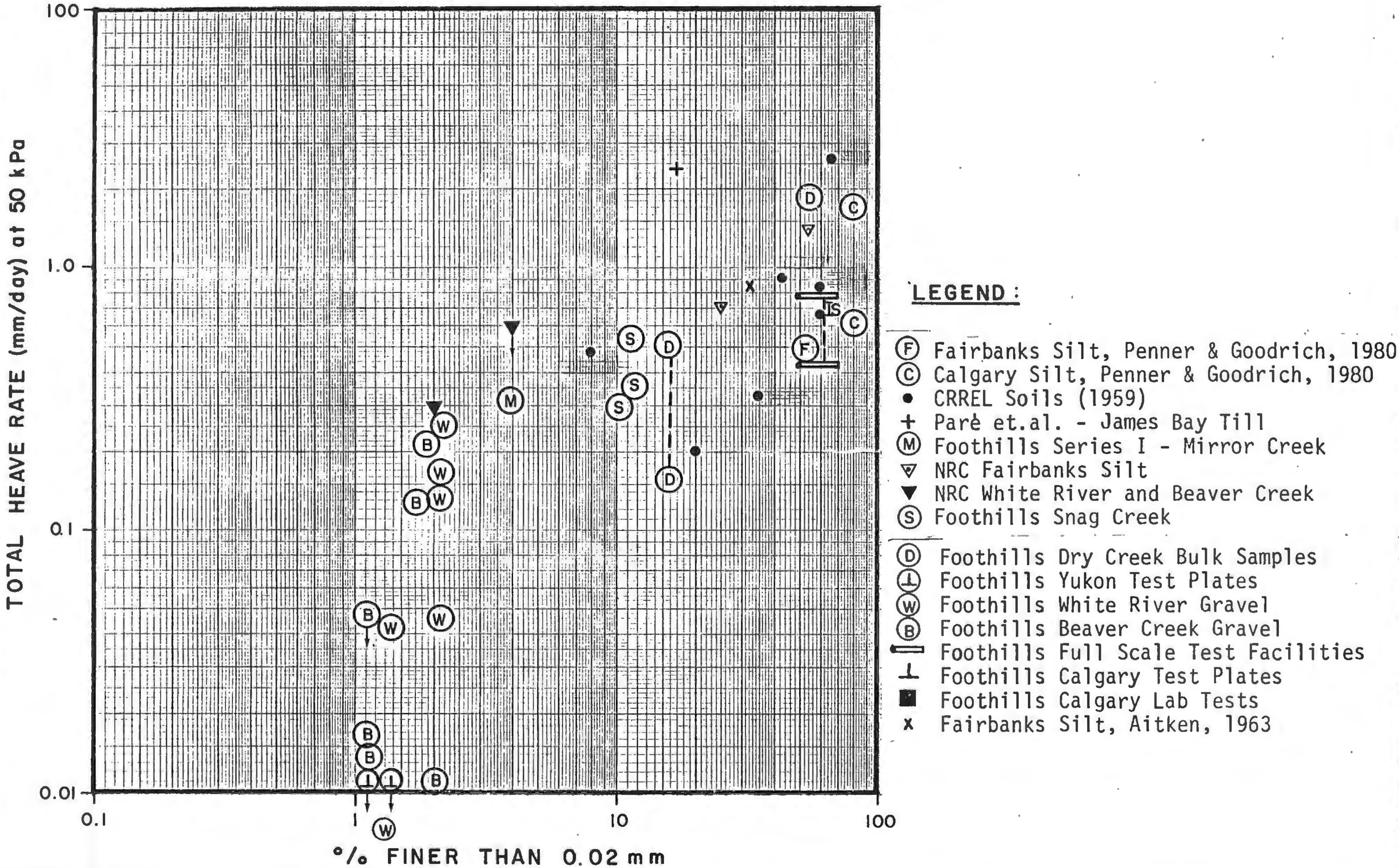
CUMULATIVE ICE SEGREGATION RATIO AND GRAIN SIZE

K 5500K

FIG. 1



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TOTAL HEAVE RATE AND GRAIN SIZE

K 5500 K

FIG. 2

These soils include Fairbanks Silt and Calgary Silt, and data for several soil types may be re-plotted from Penner and Goodrich (1980), and other data reported to Foothills Pipe Lines (South Yukon) Ltd. Heave rates and ice-segregation ratios are shown plotted with the 0.02 mm grain size characteristics on Figures 1 and 2.

Field data for Fairbanks silt at the U.S. Army CRREL test facility are given by Aitken (1963), and these are also plotted. This provides in-situ frost heave behaviour for Fairbanks Silt.

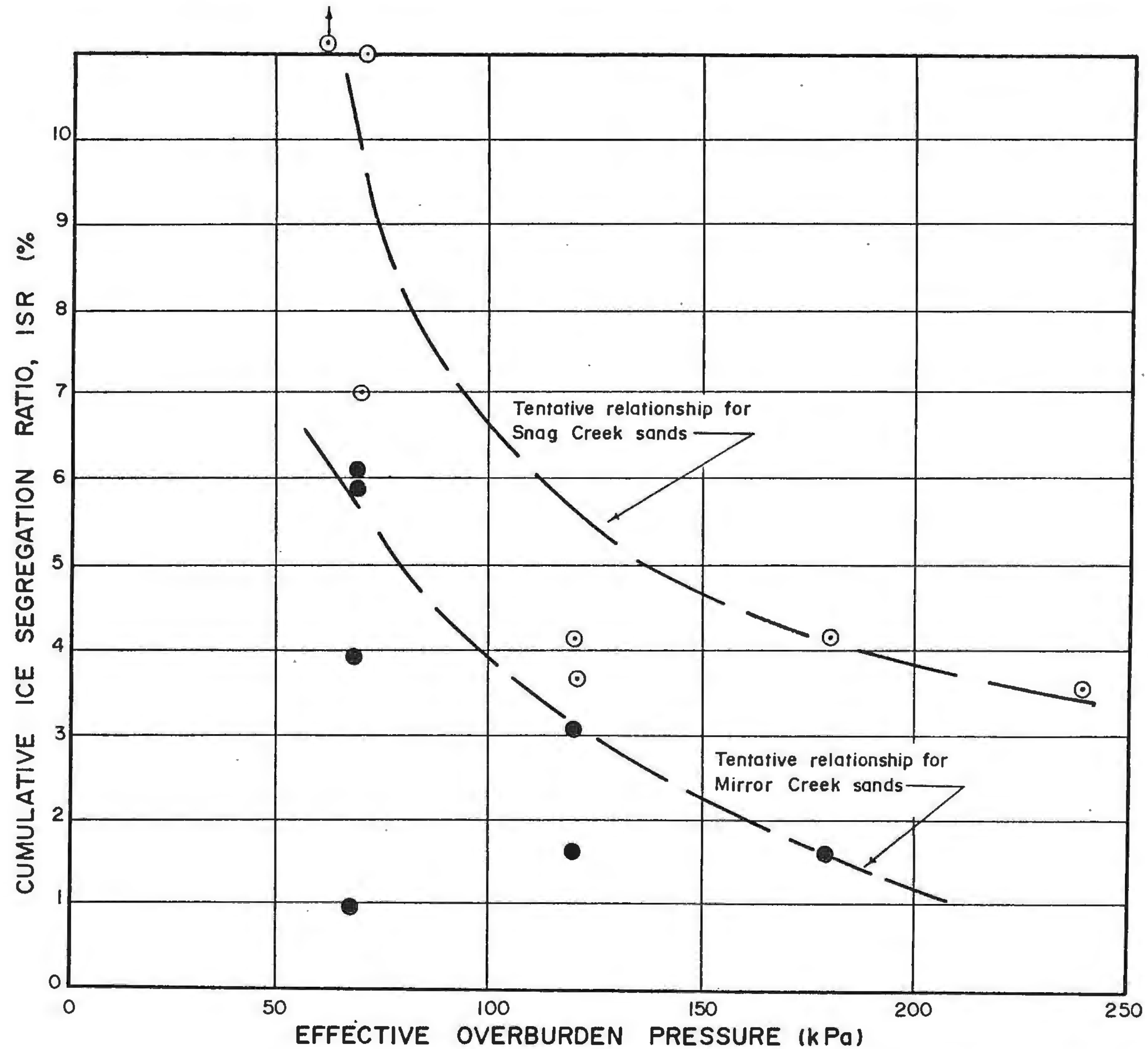
Turning to data gathered specifically for the Alaska Highway Gas Pipeline Project, the frost heave test data for the Yukon floodplain gravels are considered first. Three different studies were conducted in connection with the proposed pipeline crossings of Beaver Creek and the White River. At the time of testing, the zone of pipe chilling continued to Station 313, and included the crossing of the White River. However, the data for the White River gravels remain of general interest. The first study involved small diameter laboratory tests carried out on bulk samples from the two floodplains, with particles larger than 1.8 cm screened from the samples. These data are reported by Hardy Associates (1979a). The second study involved large diameter testing of screened and non-screened samples, and details are contained in Hardy Associates (1979b). During 1978 and 1979, a total of six test plates, 0.76 m in diameter, were operated at the two sites. The installation and results are reported in Hardy Associates (1978), Hardy Associates (1980a) and by

Nixon, Ellwood and Slusarchuk (1981). No vertical movements of significance were observed beneath the plates over the operating period, which was in excess of one year for some of the plates. The accuracy of the heave measuring system was of the order of 1 - 2 mm.

All of these data are presented in summarized form on Figures 1 and 2. Some of the smaller diameter laboratory tests may not have adequately represented the large grain sizes present in the sample, and this fact in addition to the short-term nature of the tests may have resulted in frost heave rates that appear rather high for the soil tested.

Another major testing effort in the laboratory involved soils from the Calgary frost heave test facility at University Park in Calgary. These soils are highly frost susceptible clayey silts, and underlie the site of several full scale pipe test sections that have been operated for several years. Several laboratory frost heave tests have been carried out on this material to correlate laboratory results with the full scale pipe tests. These were carried out by EBA Engineering Consultants Limited (1979), and summary results for ice-segregation ratios are given on Figure 1. It is difficult to assign any single heave rate to these tests, due to the thermal boundary conditions adopted. Constant warm and cold side temperature were applied to the specimens to obtain a cumulative ice segregation efficiency for the sample over a wide range of freezing rates. The results provide an overall indicator of the frost susceptibility of the soil, and a measure of the average ice-segregation ratio that might be expected.

One of the more extensive areas of unfrozen ground that will be traversed by the chilled pipe is located in the floodplains of the Mirror Creek/Snag Creek complex at approximately KP 8-13. Two test series were completed on the gravelly silty sands from this area. One is denoted by "Mirror Creek - Series I", and was completed by Hardy Associates (see Hardy Associates Ltd., 1980b). A second series was carried out by EBA Engineering and denoted by "Snag Creek" on Figures 1 and 2 (see EBA Engineering Consultants, 1981). The total number of tests in this area amounted to seven for the Mirror Creek soil, and six for the Snag Creek material, giving a total of thirteen long term tests. The pressure dependence of ice-segregation ratio has been established in some detail, as shown in Figure 3. A similar relationship has been established for heave rate and pressure for the two soils as shown on Figure 4. It is important to note that the original test results for the Mirror Creek and Snag Creek soils reported initial heave rates in most cases. The tests all ran for more than 10,000 minutes. In order to present a typical heave rate consistent with a slow rate of frost penetration, the heave rate at 10,000 minutes was re-interpreted from the original frost heave laboratory data for presentation here on Table 2. This was done, where possible,to allow comparison of heave rates from different tests and different sources. These upperbound relationships for the Mirror Creek and Snag Creek soils are only preliminary in nature, designed to illustrate the general trends in the data.



LEGEND

● .... MIRROR CREEK

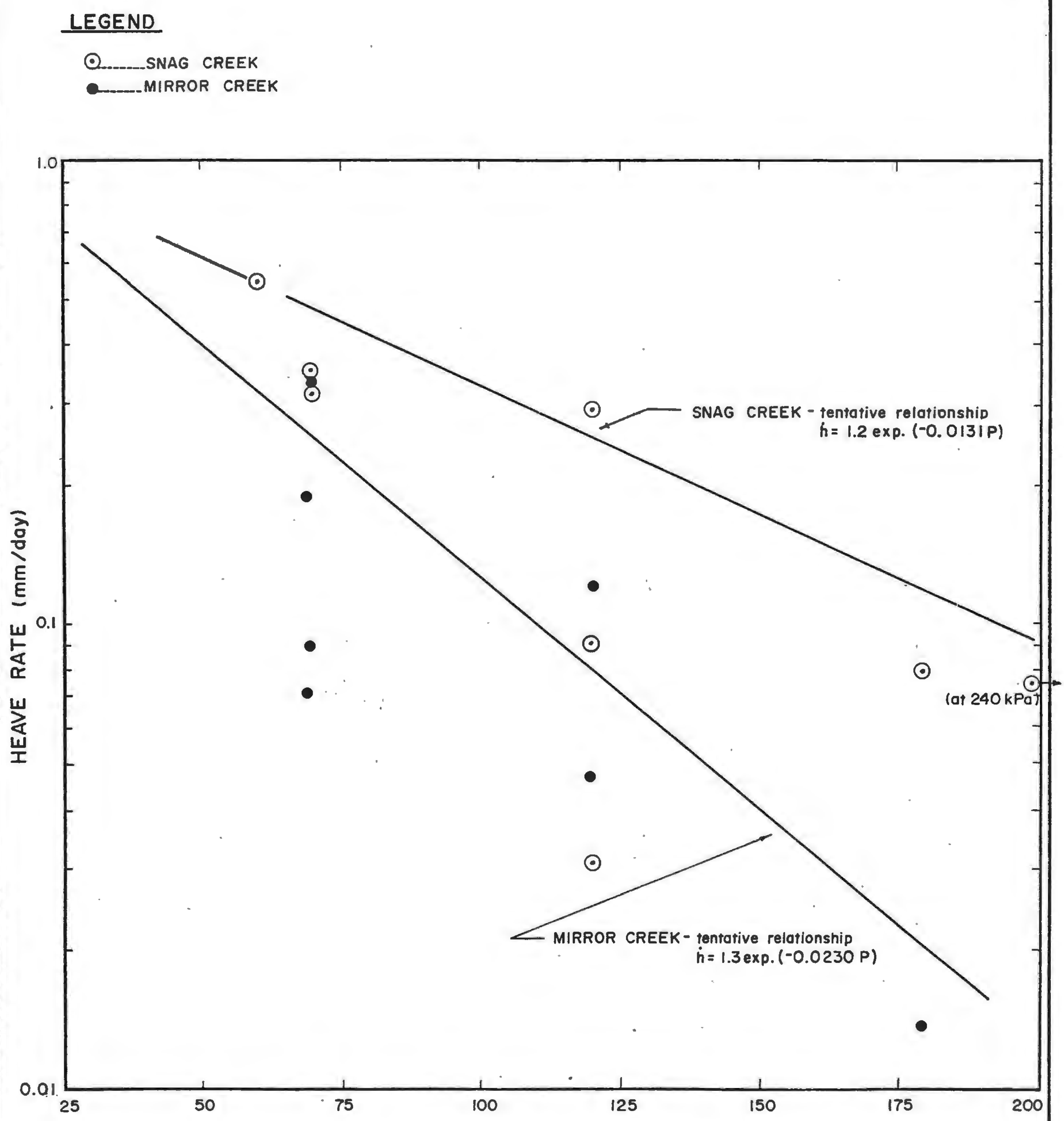
⊙ .... SNAG CREEK

NOTE: Number on data point represents cold side test temperature in °C.

RELATIONSHIP BETWEEN ISR AND OVERBURDEN PRESSURE FOR MIRROR & SNAG CREEK SANDS

K5500K

Figure 3



NOTE : Heave Rate evaluated after 10,000 minutes. Number on data points.

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TOTAL HEAVE RATE Vs. PRESSURE

K 5500 K

FIGURE 4

A site at the Alaska Highway crossing of Dry Creek, Y.T. was selected as a site for another series of frost heave test plates, and Foothills returned several bulk samples from this site for frost heave testing. These tests (under conditions of controlled frost penetration rate), are currently underway at Hardy Associates laboratories, and some preliminary values are shown plotted on Figures 1 and 2.

Mr. E. Penner of the National Research Council of Canada has completed two series of laboratory frost heave tests in co-operation with Foothills Pipe Lines (Yukon) Ltd. The first program was carried out on the gravel samples obtained from the Beaver Creek and White River floodplains. These results essentially confirmed the data obtained in Hardy Associates laboratories, and are shown summarized on Figures 1 and 2. The second program of six tests was completed on two samples of Fairbanks Silt, known as TS(8) and TS(9). These materials had different grain size distributions, and expanded further the data base available.

In all, a total of 69 laboratory tests have been carried out to date on soils from pipe test sections, or soil samples collected from the South Yukon pipeline route. Total testing time has amounted to over 1,500 hours to the present time, involving eight different soil types (see Table 2). This does not include the extensive data base available for other soils in previous published studies.

TABLE 2

SUMMARY OF LABORATORY FROST HEAVE TESTS CARRIED OUT ON PIPELINE ROUTE SAMPLES

SOURCE/LAB	TEST	LOCATION/ SOIL	% FINER THAN 0.02 mm	DURATION (DAYS)	PRESSURE (kPa)	COLD SIDE TEMP °C	WARM SIDE TEMP °C	HEAVE* RATE (mm/DAY)	FINAL FROST DEPTH (mm)	TOTAL HEAVE (mm)	FINAL ICE-SEG. RATIO (%)
HAL,	WR-2	White River	2.1	1.1	48.9	-1.8	-	0.249	102.2	0.262	0.26
(1979a)	WR-3	White River	2.1	-	14.4	-0.7	-	-	0	-	-
	WR-4	White River	2.1	2.8	97.3	-1.8	-	0.119	98.8	0.312	2.4
	WR-5	White River	2.1	2.1	48.9	-1.8	-	0.131	105.0	0.271	1.1
	WR-6	White River	2.1	2.1	14.4	-1.8	-	0.296	119.0	0.610	2.2
	WR-7	White River	2.1	1.9	14.4	-1.8	-	0.394	116.0	0.709	2.7
	WR-8	White River	2.1	1.7	97.3	-1.8	-	0	83.9	-0.025	0
HAL,	BC-1	Beaver Creek	2.0	1.8	48.9	-1.8	-	0.137	119.7	0.196	0.6
	BC-2	Beaver Creek	2.0	1.9	97.3	-1.8	-	0.014	85.9	-0.014	0
	BC-3	Beaver Creek	2.0	1.8	48.9	-1.8	-	0.236	111.9	0.310	2.6
	BC-4	Beaver Creek	2.0	1.9	14.4	-1.8	-	0.620	111.1	0.959	3.5
	BC-5	Beaver Creek	2.0	1.8	14.4	-1.7	-	0.639	115.1	0.795	4.9
	BC-6	Beaver Creek	2.0	1.8	97.3	-1.7	-	0.004	73.8	0.003	0.1
HAL,	BC-1	Beaver Creek	1.1	8.1	73.0	-1.7	-	0	350.5	-0.096	0
(1979b)	BC-2	(Large Cells)	1.1	12.0	73.0	-1.8	-	0.010	363.2	0.212	0.05
	BC-1N	Beaver Creek	1.1	9.0	73.0	-1.8	-	0.016	368.3	0.078	0.02
	BC-2N	Beaver Creek	1.1	10.8	73.0	-1.7	-	0.015	292.1	0.190	0.06
	BC-3N	Beaver Creek	1.1	20.0	130.0	-1.8 & -3	-	0-0.048	180.0	0.062	0.09
	WR-1	White River	1.3	7.9	83.0	-1.7	-	0	198.1	0.371	N/A
	WR-1A	White River	1.3	3.1	48.0	-1.8	-	0.046 - 0.175	86.4	0.234	0.27
	WR-1A	White River	1.3	2.8	83.0	-1.8	-	0	109.2	-0.079	0
	WR-1N	White River	1.3	10.1	83.0	-1.7	-	0.041	155.6	0.554	0.36
	WR-2N	White River	1.3	21.0	130.0	-1.7 & -3	-	0-0.0061	196.8	0.183	0.09

* Heave rate evaluated after a test period of 2 - 10 days, while frost line is advancing.

TABLE 2 (Cont'd)

SUMMARY OF LABORATORY FROST HEAVE TESTS CARRIED OUT ON PIPELINE ROUTE SAMPLES

SOURCE/LAB	TEST	LOCATION/ SOIL	% FINER THAN 0.02 mm	DURATION (DAYS)	PRESSURE (kPa)	COLD SIDE TEMP °C	WARM SIDE TEMP °C	HEAVE* RATE (mm/DAY)	FINAL FROST DEPTH (mm)	TOTAL HEAVE (mm)	FINAL ICE-SEG. RATIO (%)
EBA,	CS-9	Calgary Test Site	75.0	98.0	80.0	-1.7	+0.8	0.7 - 1.7	65.5	41.2	62.9
(1980)	CS-10	Calgary Test Site	75.0	98.0	80.0	-6.7	+1.7	0.4 - 1.0	110.5	33.6	30.4
	CS-11A	Calgary Test Site	75.0	28.9	80.0	-1.0	+1.0	0.78	38.1	12.6	33.1
	CS-11B	Calgary Test Site	75.0	26.9	80.0	-1.0	+1.0	0.21 - 0.77	30.7	13.1	42.7
	CS-12A	Calgary Test Site	50.0	30.0	70.0	-1.2	+1.7	0.32	39.6	4.86	12.3
	CS-12B	Calgary Test Site	50.0	26.1	70.0	-1.1	+1.7	0.09 - 0.18	36.3	3.59	9.9
	CS-13	Calgary Test Site	75.0	41.0	46.0	-1.0	+1.5	0.65	30.0	15.0	50.0
	FS-15	Fairbanks Silt		75.0	51.0	-1.0	+1.0	0.07 - 0.1	61.5	5.11	8.3
	FS-16	Fairbanks Silt		75.2	56.0	-1.0	+1.0	0.1	51.6	4.9	9.5
	FS-18	Fairbanks Silt		63.9	57.0	-1.0	+1.0	0.12 - 0.18	38.4	3.78	9.9
	FS-19	Fairbanks Silt		209.0	51.0	-1.0	+1.0	0.04 - 0.05	51.05	6.31	12.4
EBA,	#22	Snag Creek Sand	11.5	87.8	60.0	-1.0	+1.1	0.55	46.0	8.18	18.0
	#27	Snag Creek Sand	10.2		69.0	-0.5	+0.5	0.32	52.0	5.8	11.0
	#25	Snag Creek Sand	11.5	63.9	69.0	-4.1	+2.05	0.35	81.0	5.45	7.0
	#21	Snag Creek Sand	5.3		120.0	-1.0	+0.9	0.09	47.0	1.91	4.1
	#24	Snag Creek Sand	5.3	84.2	180.0	-1.05	+1.1	0.078	57.0	2.38	4.2
	#23	Snag Creek Sand	10.3		120.0	-1.95	+2.05	0.29	94.0	3.5	3.7
	#26	Snag Creek Sand	10.2		240.0	-1.0	+1.1	0.079	53.0	1.95	3.6
NRC	WR-4	White River	2.0	2.0	39.2	-1.1	-	0.73	55.4	2.02	3.6 (+)
	WR-5	White River	2.0	4.0	39.2	-1.1	-	0.38 - 0.75	61.6	3.13	5.1
	WR-2	White River	2.0	2.0	73.5	-1.1	-	0.39	57.3	0.72	1.3 (+)
	WR-3	White River	2.0	2.0	122.6	-1.1	-	0.15	59.8	0.24	0.40(+)
	BC-1	Beaver Creek	4.0	2.0	39.2	-1.1	-	0.80	62.2	1.72	2.8 (+)
	BC-4	Beaver Creek	4.0	1.9	39.2	-1.1	-	0.67	60.2	1.79	2.9 (+)
	BC-2	Beaver Creek	4.0	2.0	73.5	-1.1	-	0.45	59.7	0.88	1.5 (+)
	BC-3	Beaver Creek	4.0	2.0	122.6	-1.1	-	0.10	64.1	0.18	0.3 (+)

* Heave rate evaluated after a test period of 2 - 10 days, while frost line is advancing.

(+) Results only available for two days. ISR would be higher after longer test duration.

TABLE 2 (Cont'd)

SUMMARY OF LABORATORY FROST HEAVE TESTS CARRIED OUT ON PIPELINE ROUTE SAMPLES

SOURCE/LAB TEST	LOCATION/ SOIL	% FINER THAN 0.02 mm	DURATION (DAYS)	PRESSURE (kPa)	COLD SIDE TEMP °C	WARM SIDE TEMP °C	HEAVE* RATE (mm/DAY)	FINAL FROST DEPTH (mm)	TOTAL HEAVE (mm)	FINAL ICE-SEG. RATIO (%)
NRC, TS(8)-3	Fairbanks Silt	54.0	2.0	34.3	-1.08	-	2.07	49.8	6.23	12.5
-1	Fairbanks Silt	54.0	2.0	73.6	-1.05	-	0.79	50.6	2.19	4.3
-2	Fairbanks Silt	54.0	2.0	122.6	-1.12	-	0.16	54.5	0.48	0.9
TS(9)-2	Fairbanks Silt	25.0	2.0	34.3	-1.07	-	1.27	55.1	2.79	5.1
-1	Fairbanks Silt	25.0	2.0	73.6	-1.13	-	0.22	58.0	0.51	0.9
-3	Fairbanks Silt	25.0		122.6	-1.16	-	0.09	58.7	0.18	0.3
HAL, 1	Mirror Creek	4.0	42.0	69.0	-1.0	+0.9	0.09	50.8	2.02	4.0
2a	Mirror Creek	4.0	35.0	69.0	-0.55	+0.9	0.07	48.1	0.46	0.9
2b	Mirror Creek	4.0	49.0	69.0	-0.4	+1.3	0.19	34.0	2.03	6.0
4	Mirror Creek	4.0	42.0	120.0	-1.0	+0.7	0.12	65.2	1.97	3.0
5	Mirror Creek	4.0	60.0	180.0	-1.0	+1.0	0.013	50.9	0.83	1.6
6	Mirror Creek	4.0	60.0	69.0	-1.05	+1.0	0.33	46.1	2.79	6.1
7	Mirror Creek	4.0	60.0	120.0	-1.0	+1.0	0.046	47.0	0.81	1.7
HAL, DC-1	Dry Creek	16.0	6.0++	48.0	-0.3 to -1.2	0.9 to 0.1	0.38	91.4	2.04	2.2
DC-2	Pit #5	16.0	3.0++	48.0	-0.3 to -1.4	0.5 to 0.2	0.51	90.2	1.45	1.6
DC-3	Pit #5	16.0	8.0++	48.0	-0.3 to -1.0	0.9 to 0.1	0.25	94.0	2.08	2.2
DC-4	Pit #5	16.0	6.0++	48.0	-0.5 to -0.8	0.3 to 0.0	0.26	91.4	1.55	1.7
DC-5a	Pit #5	16.0	10.0++	48.0	-0.3 to -0.7	0.6 to 0.2	0.16	74.9	1.59	2.1
DC-5b	Pit #5	16.0	4.0++	48.0	-0.4 to -1.0	0.4 to 0.0	0.30	94.0	1.18	1.3
DC(4)-1	Pit #4	56.0	4.0++	48.0	-0.4 to -1.5	0.9 to 0.1	1.9	85.1	6.85	8.0

* Heave rate evaluated after a test period of 2 - 10 days, while frost line is advancing.

++ These tests were carried out by varying the boundary temperatures of the test at a constant rate to obtain a constant frost penetration rate.

In addition to the above, Northwest Alaskan Pipeline Co. Ltd. is completing an exhaustive laboratory study of the frost heave characteristics of Fairbanks Silt. An extensive testing program involving many different soil types along the route will then be undertaken. These studies are designed to correlate laboratory results with those from several full scale field test sites recently constructed or under construction along the route between Delta Junction and the Alaska/Yukon border. Detailed results from these programs are not available yet, but will undoubtedly add greatly to the data base currently available.

3.2 In-Situ Plate Tests

The volume of soil frozen by a small laboratory test and a full scale test or operating pipeline are vastly different. For these reasons, it was considered desirable to employ a freezing test method that would freeze a much larger volume of soil than a conventional laboratory test, would allow a better representation of the undisturbed soil and groundwater conditions, and would freeze the soil over a longer time frame. All of these items would allow a closer representation of the field freezing conditions adjacent to a large chilled structure. In relation to the Alaska Highway Gas Pipeline Project, this testing method received particular interest when it became necessary to test silty gravels, loose sands, shattered bedrock and areas having a depressed groundwater table. Any of these conditions would prove very difficult to adequately test in the laboratory.

Circular frost heave test plates were adopted to meet these requirements. The plates were constructed 0.76 m in diameter, and could be installed at any reasonable depth. Frost heave was measured using two vertical risers, connected to the cold plate. Excavation for the plates have been completed mostly by backhoe. In one case, however, a truck mounted auger rig excavated a hole 0.9 m in diameter to a depth of 3 m. A variable number of thermistors may be installed beneath the centre and edge of the plate, to monitor the frost advance. More details of the installation are given Nixon, Ellwood and Slusarchuk (1981).

A total of six plates were installed at the two river floodplain sites at Beaver Creek and White River, in the Yukon. These sites were eventually discontinued after about 400 days of operation. At this time, it was considered that the installations had yielded all possible useful data on the behaviour of these floodplain gravels. The Yukon sites are both situated in floodplain gravel deposits, which have 2 – 3% silt and clay sizes, and 1 – 2% finer than 0.02 mm.

The water table at both sites was close to, or above the plate elevation. The plates were located at depths of 3.0 and 3.5 m at Beaver Creek and White River respectively. No significant frost heave was observed during the operating period at any of the six plates at the Yukon locations.

A further two plates were installed at the Calgary frost heave test facility to assist in correlating laboratory and field frost heave behaviour. These plates are still in operation at the time of writing.

The Calgary subsoils are clayey silts, having 85 – 90% silt and clay sizes, and 50 -70% finer than 0.02 mm. For this site, Figure 5 shows the data for Plate #7, installed at a depth of 3 m below ground surface. This corresponds to the elevation of the frost bulb beneath the fullsize test pipeline sections after a few months of operation. Heave rates and ice-segregation efficiency values can be obtained from these data.

Average frost heave rates for the first 200 days were in the range of 0.4 mm/day with initial heave rates during the first 20 days in the range of 1 mm/day. These are very similar to the heave rates observed for the full size test sections, where heave rates about 1 mm/day were observed for the first year of operation, and lower rates in the several years following.

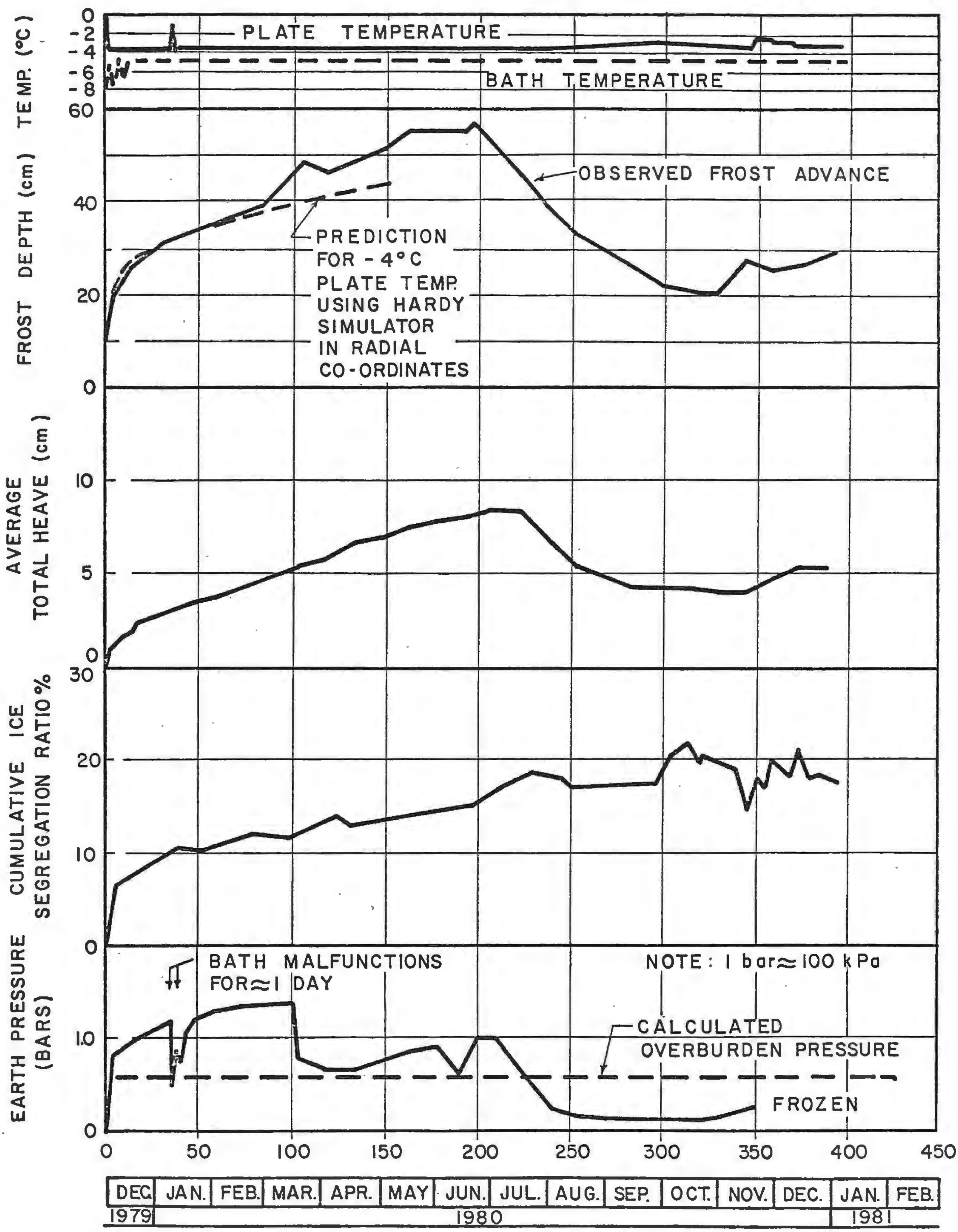
In October/November of 1980, six new plate installations were completed at the proposed pipeline crossing of Dry Creek, in the South Yukon. These tests are designed to investigate the freezing response of silty sands and silty gravels, in addition to one plate installed in very warm, fine-grained permafrost. Report on the analysis of the results from these installations is currently underway.

3.3 Field Test Facilities

Two full scale field test sites have been operated by Foothills Pipe Lines (South Yukon) Limited and Northwest Alaskan Pipe Lines Limited. The first of these involves six test pipe sections buried in a highly frost susceptible clayey silt at Calgary, Alberta. The second site involving ten pipe sections has been constructed near Fairbanks, Alaska, in a silt of moderate frost susceptibility. These test sections have provided full scale correlation with laboratory test data, together with valuable data on the long term frost heave performance of a full size pipe section. In addition, the effectiveness of various mitigative measures involving pipe insulation has been examined.

(a) Calgary Frost Heave Test Facility

The Calgary Frost Heave Test Site facility was constructed by Canadian Artic Gas Study Ltd. in 1974. The construction and operation of the site has been reported by Slusarchuk et al (1978) and subsequently



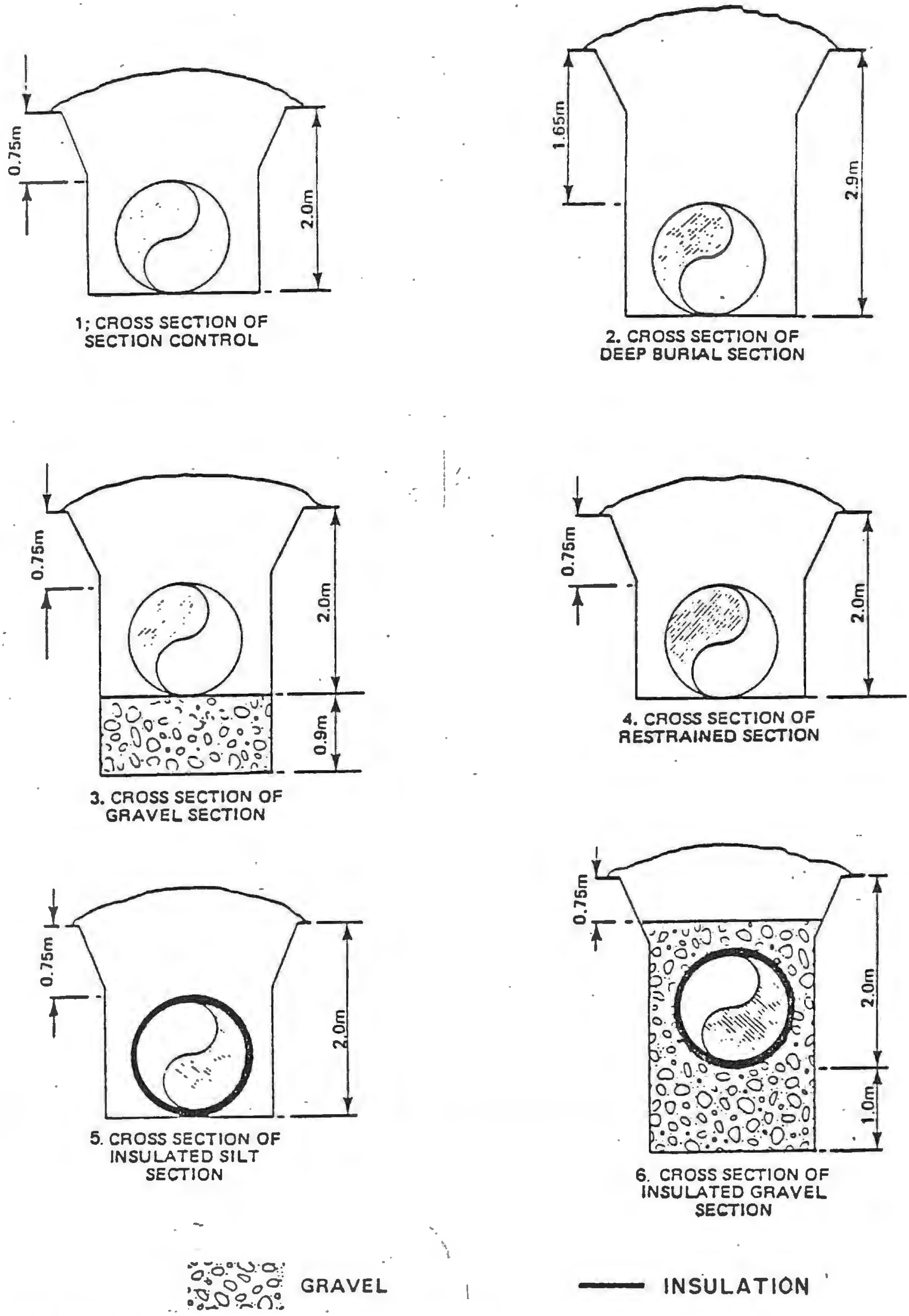
FROST HEAVE TEST
DATA - CALGARY
PLATE 7

Fig 5

by Carlson et al (1981). The operation of the test site was taken over by Foothills Pipe Lines at the end of 1977.

The facility consists of six full size pipe sections. The first four are non-insulated 1.2 m diameter pipe section approximately 12 m long. These pipeline sections were installed with the original construction of the site in 1974. In 1978 two insulated pipe sections of similar size were installed in the facility. These latter two sections were insulated with a 15 mm thickness of polyurethane insulation. The ground around the pipes is frozen by circulating air at about -10°C through the system.

Figure 6 shows the cross sections for the four non-insulated pipe section. The control section represents the standard burial configurations for a gas pipeline of this dimension. The base of the pipe is buried at approximately 2 m depth from the original ground surface and there is a silt cover over the top of the pipe approximately 0.75 m thick. The deep burial section is similar to the control section except that the base of the pipe is buried to a depth of 2.9 m under the original ground surface. The gravel section is similar to the control section with the addition of 0.9 m of gravel placed underneath the pipe. The fourth section in this sequence is the restrained section. The restraints provided a constant hold down pressure for this section. Figure 6 also presents the cross sections for the two insulated pipe sections. The insulated silt section has a geometry which is identical



HARDY ASSOCIATES (1978) LTD.
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CROSS SECTIONS FOR ALL
TEST SECTIONS AT THE
CALGARY FROST HEAVE TEST FACILITY

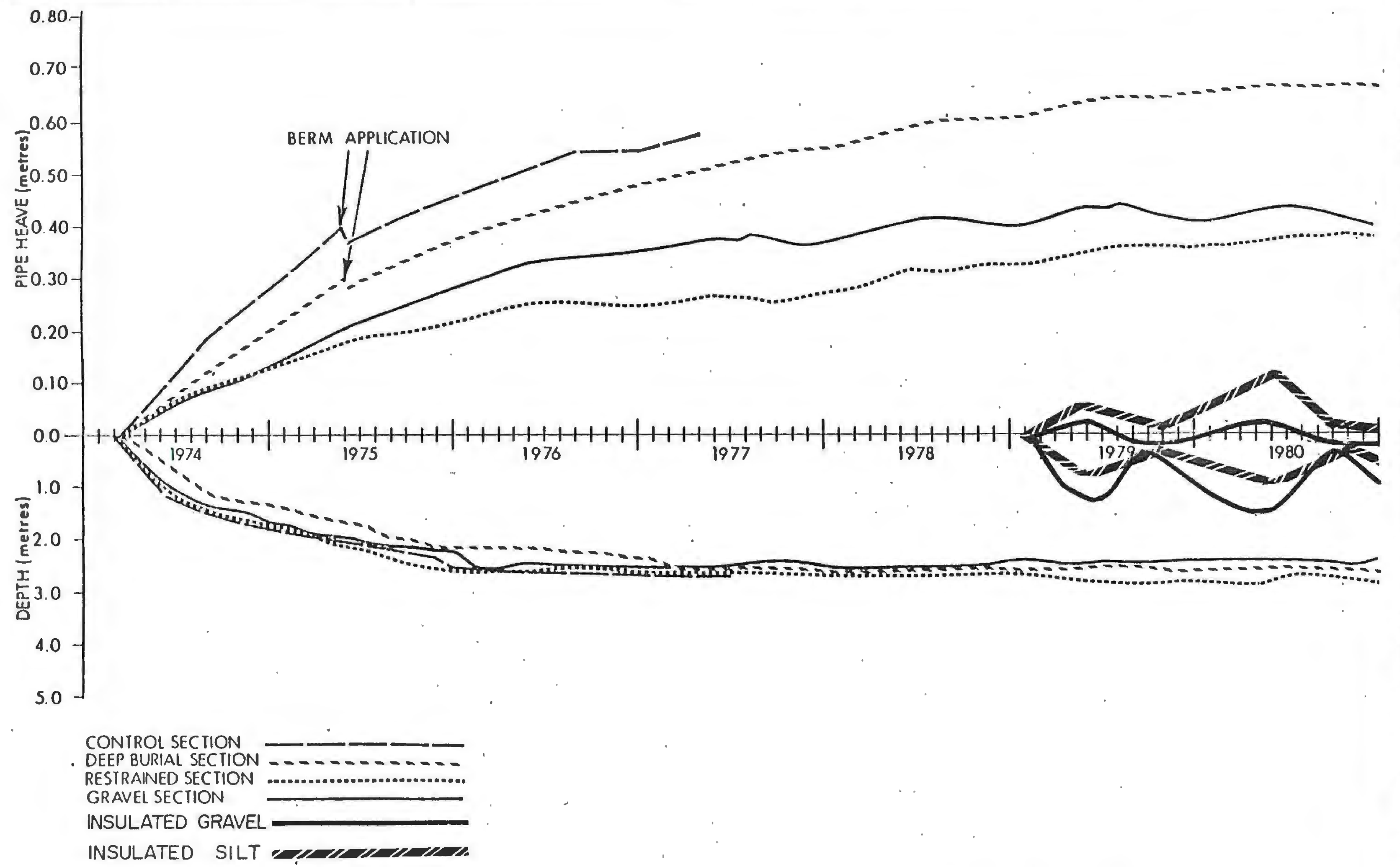
K 5500 K

FIGURE 6

to the original control section. The insulated gravel section as is seen in Figure 6 is similar to the gravel section with the addition of insulation wrapped around the pipe. The control section was removed in the Fall of 1976.

The observed pipe heave and depth of frost penetration below the bottom of the pipe is presented in Figure 7. It is seen that the frost heave of the control section was the largest of the four sections. The effects of the design variations represented in these sections is also seen on Figure 7. The bottom portion of Figure 7 shows that the frost penetration depths for all four sections was approximately the same being roughly 3 m in depth.

For the insulated pipe sections, the frost heave and frost front penetration below the bottom of the pipe show a trend which is quite different from that for the non-insulated sections. Very significant annual fluctuations in both the heave and frost penetration are obvious in Figure 7. This data shows that the insulation around the pipe has been effective in isolating the pipe temperature to a large extent from the external ground temperatures. The effectiveness of the gravel soil replacement around the insulated pipeline is shown through the much lower heave values for the insulated gravel section. The frost heave of the gravel section was in the order of 20 and 35 mm in the first and second seasons respectively.



CALGARY TEST FACILITY

HARDY ASSOCIATES (1978) LTD.
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OBSERVED PIPE HEAVE & DEPTH OF FROST BELOW BOTTOM OF PIPE AFTER CARLSON et al (1981)

K5500 K

Figure 7

For both insulation sections, the warmer summer ground temperatures have thawed back the frost bulbs, negating the heave accumulated over the winter period.

These results are in general, quite similar to the results obtained at the Calgary Frost Heave Site using 760 mm diameter in-situ freeze plates. Summary data points plotted for the field test facility on Figures 1 and 2 also show similar results to the laboratory frost heave test results.

Carlson et al (1981) have shown that better comparison between laboratory and field data can be obtained by evaluating the ice-segregation ratio as a function of frost penetration rate. For fine-grained soils, this will form the basis for semi-empirical predictive methods reviewed later in this document.

(b) <u>Fairbanks Test Facility</u>

The Fairbanks Frost Heave Test Facility is located some six miles from Fairbanks, Alaska, on the Chena Hot Springs Road.

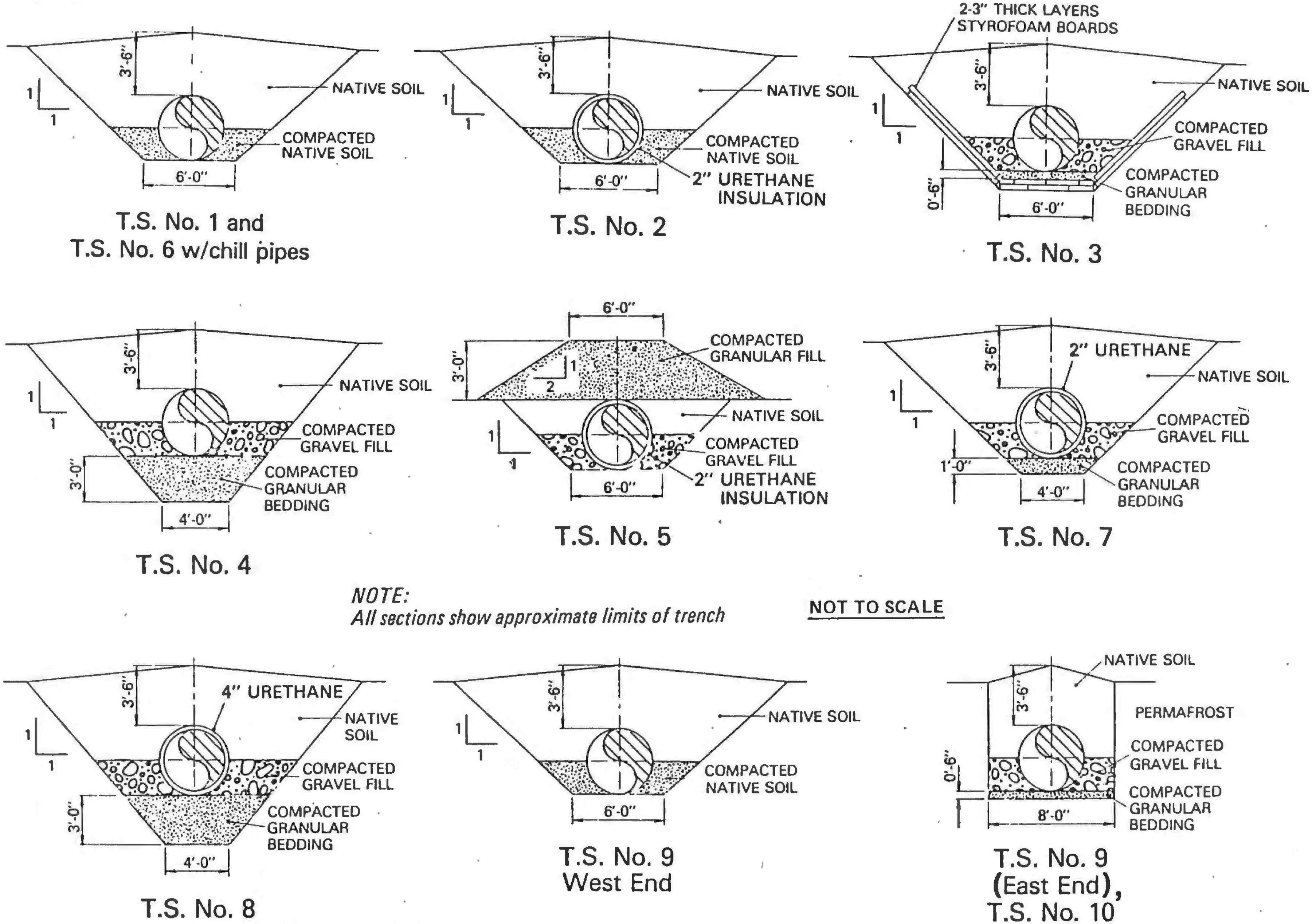
The facility has 10 separate sections of 1.2 m diameter pipe. Combinations of soil replacement, pipe insulation, shallow pipe burial, chill pipes, and select bedding material are employed to characterize the most effective frost heave control measures. These configurations are presented in Figure 8. One important feature of the site was the presence of a transition from frozen to thawed ground, which was necessary so that the structural effects of differential frost heave on a pipe structure (i.e., Test Section 9) could be observed.

Excepting Test Section 5, all test sections are buried 0.76 m below natural grade with a 0.3 m thick crown directly above the pipe. The top of Test Section 5, the shallow-burial section, is flush with the natural grade and is covered with a 0.9 m thick layer of compacted granular fill.

Ditch walls for Test Sections 1 through 8 plus the west end (thawed soil) of Test Section 9 were constructed at a 1-to-1 slope. Ditch walls for the east end of Test Section 9 and Test Section 10, both of which are buried in permafrost, are vertical.

Ditch Configuration, Test Section 1

Test Section 1 is a bare, 36 m long, uninsulated pipe installed in a trench backfilled with native soil (Fairbanks silt). The the pipe lies upon in-situ soil. Constructed with no frost heave mitigation measures, this section is intended to be used as a reference for the mitigative sections.



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FAIRBANKS FROST HEAVE TEST FACILITY

DITCH CONFIGURATIONS

K 5500 K

Figure 8

Ditch Configuration, Test Section 2

Test Section 2, insulated with 5 cm of urethane insulation, is 36 m long, and is insulated in a trench backfilled with native soil. The pipe is founded on natural soil.

Ditch Configuration, Test Section 3

Test Section 3 is 36 m long and is insulated from the in-situ soil by 6 inches of polystyrene board insulation which is placed on the bottom and sloped walls of the ditch. A 0.15 m compacted granular bedding separates the bare pipe from the board insulation underneath the pipe.

Between the springline of the pipe and its bottom, compacted gravel fill was used to separate the pipe from the insulated ditch walls. From the springline of the pipe to the ground surface, the ditch is backfilled with native soil.

Ditch Configuration, Test Section 4

Test Section 4 is a 36 m long uninsulated pipe installed in an overexcavated ditch. The pipe lies upon a 0.9 m thick layer of compacted granular bedding. The ditch was backfilled with compacted gravel fill up to the pipe springline and with native soil from the pipe springline to the ground surface.

Ditch Configuration, Test Section 5

Test Section 5, insulated with 5 cm of urethane insulation, is 36 m long, and is buried such that the top of the pipe insulation is at the same elevation as the natural ground surface. A 0.9 m thick berm of compacted granular fill was used to cover the pipe. The pipe is founded on native soil. The ditch was backfilled with compacted gravel fill up to the pipe springline and with native soil from the pipe springline to the natural ground surface.

Ditch Configuration, Test Section 6

Test Section 6 is a bare, 36 m long, uninsulated pipe installed in a trench backfilled with native soil. Chill pipes, using water and methanol as the convective heat transfer medium were installed along both sides of the pipe. They extend (at an angle) approximately 4.5 m beneath the pipe bottom.

Ditch Configuration, Test Section 7

Test Section 7, insulated with 5 cm of urethane insulation, is 36 m long, and is installed in an overexcavated ditch. The pipe, lies upon 0.3 m of compacted granular bedding.

The ditch was backfilled with compacted gravel fill up to the pipe springline and with native soil from the pipe springline to the ground surface.

Ditch Configuration, Test Section 8

Test Section 8, insulated with 10 cm of urethane insulation, is 36 m long, and is installed in an overexcavated ditch. The bottom of the pipe, lies upon a 0.9 m thick layer of compacted granular bedding. The ditch was backfilled with compacted gravel fill up to the pipe springline and with native soil from the pipe springline to the ground surface.

Ditch Configuration, Test Section 9

Test Section 9 is a 120 m long uninsulated pipe with approximately 81 m (at the west end) buried in thawed soils and the remaining 39 m buried in permafrost. This pipe demonstrates the heave of thawed soils, the heave of initially frozen and also the effect of differential frost heaving of the pipe at the transition between permafrost and thawed soils.

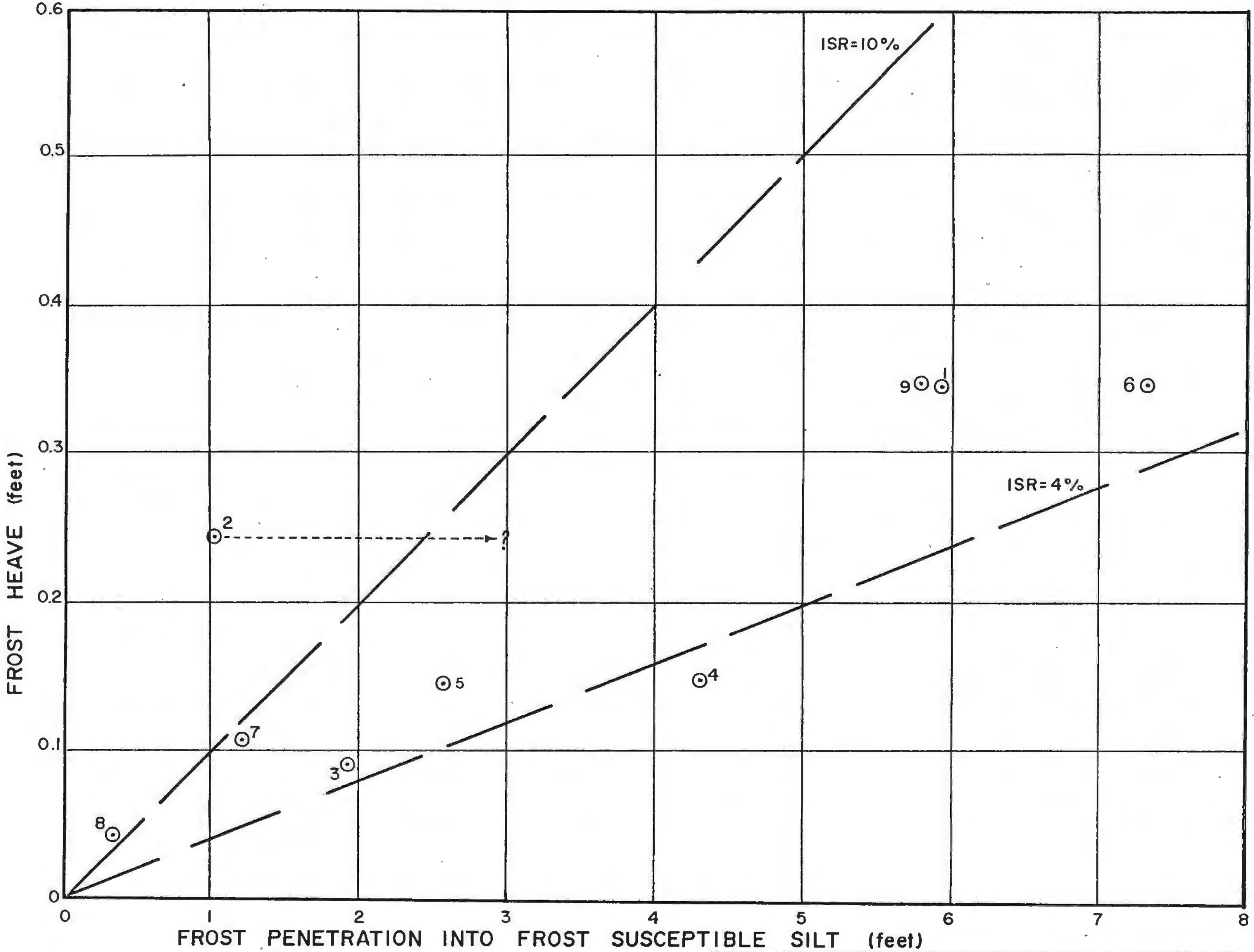
The west end of Test Section 9 is installed in a 1-to-1 side-slope ditch which is backfilled with native soils. The bottom of the pipe lies upon in-situ soil.

At the east end of Test Section 9, the bottom of the pipe lies upon 6 inches of compacted granular bedding within a ditch with vertical walls. The ditch is backfilled with compacted gravel fill up to the springline of the pipe and its bottom and with native soil between the springline of the pipe and the ground surface.

Ditch Configuration, Test Section 10

Test Section 10 is a bare, 12 m long, uninsulated pipe wholly installed in permafrost. The bottom of the pipe lies upon 0.15 m of compacted granular bedding within a ditch with vertical walls. The ditch is backfilled with compacted gravel fill up to the pipe springline and with native soil from the pipe springline to the ground surface.

The site has provided extensive data in the first 1.5 years of operation. A very cursory presentation of results is shown on Figure 9. In this graph, the frost heave is plotted with frost penetration into frost susceptible silt for the eight test sections installed in unfrozen ground. To illustrate the range of results, two straight lines representing ice-segregation ratios of 4 and 10% have also been plotted. The data points lie close to or within this range. Again, it should be noted that no account has been taken of the increasing pressure with time, and the variable frost advance rates from section to section in this summary interpretation of results.



NOTE: The number of each test section is shown beside the corresponding data point.

FAIRBANKS TEST SITE
FROST HEAVE AND PENETRATION
IN JULY, 1980

K5500 K

Figure 9

This range of data has also been plotted for comparison purposes on Figure 1, and shows broad agreement with the laboratory test data, and previous studies in the same soil type.

3.4 Heave in Permafrost

It has been considered for some time that water movement may occur behind the $0^{o}C$ isotherm in response to a thermal gradient within the frozen soil. This has been observed in the laboratory, and undoubtedly occurs in the field (see Hoekstra, 1969; Horiguchi and Miller, 1978; Loch, 1979). Two factors tend to greatly reduce the practical significance of this, however. Firstly, the permeability values obtained to date appear very low, and values of 6 - 8 x 10^{-10} cm/sec have been inferred by Horiguchi and Miller (1980), and Konrad (1980), at temperatures of $-0.1^{o}C$ or so. At temperatures colder than this, the permeability continues to fall off rapidly. The second factor that tends to de-emphasize the practical impact of fluid migration in frozen soils is the fact that fluid flow generally involves motion of water or ice from one part of the frozen zone to another with little increase in the total amount of ice in the frozen zone beneath the pipe. Consequently, it is felt that although a few centimetres of movement may occur in frozen ground over a long time period, the amount of differential motion by this mechanism will not have a significant impact on the pipeline. In addition, the pipe is encapsulated in a zone of chilled frozen soil, which would tend to provide additional resistance to differential movements.

Field evidence (or the lack of it) for heave in frozen ground beneath a chilled pipeline can be obtained from observing the motion of heave gauge pairs at the Calgary test facility. Slusarchuk et al (1978) and EBA Engineering Consultants Ltd. (1979) have plotted the relative movement of several gauge pairs, and observed that once the frost line had passed the lower gauge in any vertical pair, the height of frozen soil between the two gauges remained essentially constant within precise survey accuracy. This led to the conclusion that heave due to water migration in frozen soil behind the active ice lens was likely of minor significance when compared with the active ice-segregation taking place at or close to the freezing front.

Field evidence for little or no heave in frozen ground can also be obtained from the Fairbanks facility. Gauge pairs located 0.6 m (2 feet) and 1.05 m (3.5 feet) below pipe base at Test Section 1 indicated zero frost heave between the gauge pairs after the lower gauge was frozen. These data are summarized on Table 3.

Based on the above, it is concluded that burial of a chilled pipe in warm permafrost will not cause significant vertical differential movements in the pipeline, and need not require further consideration in the design effort.

TABLE 3

RELATIVE HEAVE OF GAUGE PAIR AT SECTION 1 IN FAIRBANKS

Time After Start-Up Days	Heave in Feet		Heave Diff.
	Plate 2' Below Pipe	Plate 3.5' Below Pipe	(2-3 feet)
0	0	0	0
9	0.006	0.010	-0.004
20	0.001	0.007	-0.006
52	0.006	0.013	-0.007
83	0.007	0.014	-0.007
111	0.031	0.009	0.022
143*	0.057	0.022	0.035
171	0.077	0.042	0.035
202	0.099	0.063	0.036
234	0.110	0.075	0.035
265	0.129	0.094	0.035
297	0.139	0.104	0.035

* Note: Frost line passed lower gauge on approximately day 120.

Summary Notes:

(a) Ice-segregation ratio for this soil layer = 0.035/1.5 = 0.023

(b) Frost heave in 150 days following "freeze-up" of gauge pair = 0

4.0 THERMAL MODEL

4.1 Use of Thermal Model in Design

Verification of the thermal model used for frost depth predictions is described in the EBA Engineering Consultants Ltd. (1981) report on "A Brief Description of EBA Geothermal Model".

The thermal model is intimately involved in the design process for frost heaving in two prime ways. Initially, thermal simulations are carried out for a non-insulated pipe at standard burial depths for different pipe temperatures and soil conditions. This provides (a) the ultimate depth of frost (i.e. the zone of soil that may eventually be frozen) and; (b) the time rate of frost advance. The latter data are required in assessing the appropriate heave rates or ice-segregation efficiency for the corresponding frost penetration rate. These values are required in the semi-empirical prediction of frost heave.

If the soils have some fine-grained particle sizes, and may require mitigation or prevention of frost heave, the thermal model is employed to evaluate the frost advance with time for two design modes discussed later. Each of these involves insulation and possibly some depth of granular fill beneath the pipe, and these configurations are subjected to thermal analysis with appropriate soil properties. The depth of frost penetration with time beneath the pipe or trench insulation is plotted for these special design geothermal analyses. Charts relating pipe temperature, thickness of insulation and granular fill beneath the pipe and depth of frost penetration into native soil are then prepared. These charts are used in conjunction with an assessment of the frost heave potential for the soil to obtain a design thickness of insulation and granular fill beneath the pipe. This forms the basis for the possible design to mitigate frost heave.

The thermal properties of insulating materials and earth materials are known with sufficient accuracy that, following a conservative selection of input parameters for geothermal analysis, the maximum depth of frost can be predicted with confidence. As stated in the previous sub-section, the formulation and accuracy of the numerical solution have been compared many times in the past against known exact analytical solutions and field performance, giving close agreement between prediction and observation.

EBA Engineering Consultants Ltd. (1981) provides a description of the thermal model as well as the procedures used to verify its accuracy for use in design.

5.0 HEAVE PREDICTION

5.1 Introduction

There is no complete and proven theory to predict frost heaving at the present time. Consequently, where an estimate of the magnitude of frost heave is required, Foothills has employed two semi-empirical approaches to carry this out. Both approaches rely heavily on laboratory test results for the soil, or previous data on similar soil types. As shown earlier, Casagrande's grain size criterion, the percentage of soil particles finer than 0.02 mm, can be used as a convenience measure of the likely frost heave behaviour.

5.2 Ice-Segregation Ratio

The first method involves a consideration of the ice-segregation ratio. At any given point during the freezing process, the instantaneous ice-segregation ratio can be written as:

$$\Delta ISR = \Delta h / \Delta X$$

where: Δh is an increment of total heave

and ΔX is an increment of the depth of soil frozen.

At the end of the freezing period, when frost advance within the soil is essentially complete, the total or cumulative ice-segregation ratio can be expressed as:

$$ISR = h_{total}/X_{total}$$

The instantaneous ice-segregation ratio, ΔISR, was shown by Carlson et al. (1981) be dependent on the rate of freezing. It is generally observed that the instantaneous ISR will decrease with faster freezing rates. In addition the ΔISR is dependent on overburden pressure. In general, increasing effective overburden pressure (or increasing depth to the freezing plane in the ground) causes a decrease in the ice-segregation ratio. The dependence appears stronger in coarser-grained soils, and weaker for very fine-grained soils. In general, then, the instantaneous ice-segregation ratio is a function of two variables, the rate of freezing dX/dt, and effective overburden pressure, σ'; i.e.

$$\Delta ISR = f(dX/dt, \sigma')$$

A similar relationship has been proposed by Konrad (1980), where the pore water velocity to the freezing frost, v, is dependent on the thermal gradient near the freezing front, dT/dX, and the effective overburden pressure.

These relationships are difficult and time-consuming to generate, and involve a series of tests on a soil type at different pressures and freezing rates. The dependence of the cumulative or total ISR on the effective overburden pressure has already been shown on Figure 3 for Mirror and Snag Creek gravelly sands.

Once the function relating ΔISR to freezing rate and pressure is developed, the prediction of heave can be carried out in a simple fashion by coupling the results with a thermal prediction. Over any time period, Δt, the frost line moves a distance ΔX, and the associated increment of frost heave is given by:

$$\Delta h = \Delta ISR . \Delta X$$

where: ΔISR is understood to be a function of frost penetration rate and pressure.

The total heave is then simply:

$$h = \sum_{i=1}^{n} \Delta h$$

where: n is the number of time increments considered up to the time, t.

Foothills Pipe Lines and Northwest Alaskan pipeline are continuing to develop this approach for soils of low and moderate frost susceptibility. Generally, more frost susceptible soils having a high fines content will not be tested to any extent, as frost heaving beneath a bare pipeline in such sites will likely prove excessive, and mitigation to reduce or prevent frost advance will be required. In addition, the more prevelant soils in the zone of pipe chilling are gravels and sands with relatively low silt content. Consequently, testing efforts are being directed towards increasing the data base for coarser-grained soils along the route.

5.3 Heave Rates

A second semi-empirical method involves the evaluation of the heave rate of a soil, which is also considered to be a function of frost penetration rate and effective stress, i.e.

$$dh/dt = f(dX/dt, \sigma')$$

For the coarser-grained soils, the dependence on the freezing rate, dX/dt, appears less than for fine-grained soils, and this approach may prove easier to employ for coarser grained soils. Figure 2 has shown a composite plot of heave rates for several different soils. The heave rate selected for this plot can only be considered as representative or typical, and does not represent the fairly wide range of heave rates that can be experienced in a single test, particularly for the fine-grained tests. The purpose of the plot is to indicate the broad dependence of heave rate on the grain size of the soil.

In the coarser-grained soils, a strong dependence exists between heave rates and effective overburden pressure. These relationships have been investigated in particular for the Beaver Creek and

White River soils. This causes a general reduction in heave rate with time, as the frost line progresses deeper and the stress on the frost line increases. In general, then, heave is predicted by this method using the following:

$$h = \int_0^t (dh/dt)\, dt$$

where: dh/dt is the heave rate, and is understood to be a function of freezing rate and more importantly of pressure.

Ignoring the dependence on freezing rate will normally provide a conservative answer, if the laboratory test data were collected at relatively fast freezing rates. Data from Penner (1972, Figure 4)* show that at rates of frost advance less than 15 cm/day, the total heave rate steadily decreases as the frost advance rate decreases. Normal laboratory frost heave tests carried out for this project freeze the soil at a rate of 0.5 - 2.5 cm/day. Therefore, ignoring the dependence of heave rate on rate of frost advance will cause an over-estimate of the amount of heave. It then remains to establish the dependence of heave rate on pressure, and this can be obtained from a single series of frost heave tests where precise control on frost advance rate is not mandatory.

Foothills Pipe Lines are also pursuing this approach, particularly for coarser gained soils, and are currently carrying out laboratory tests on silty gravels from the Dry Creek crossing in the South Yukon. The dependence of both heave rate and ice-segregation ratio on freezing rate and pressure will be examined.

Both of these methods will be considered for use where it becomes necessary to carry out a frost heave prediction at a specific site. Both types of predictions will likely be carried out at a given site, and the results compared. Some judgement is clearly involved in selecting the more appropriate method. At the present time, the ice-segregation ratio method is considered more suitable for finer soils, and evaluation based on heave rates will be used for coarser grained soils.

5.4 Application of Heave Predictions in Design

Heave predictions will enter into the design process in the following ways:

i) Based on the existing data, and correlations with Casagrande's grain size parameter, a range of soil types will be defined as non-heaving or low heave potential soils. These materials will likely include the Beaver Creek floodplain gravels, and other relatively clean granular materials. The precise assessment of what range of soil types constitutes "non-heaving or low heave potential soils" has not yet been completed, as it will depend on results of soil-pipeline interaction stress analysis studies. To date, only a limited number of these pipe stress analyses have been carried out, to determine the tolerable heaving conditions beneath the pipe. When a total allowable heave has been selected, heave predictions can be carried out for different soil types for which data are available. These heave predictions will account for the known dependence on effective stress and freezing rate. By comparing the long-term heave prediction with the "allowable" or tolerable heave, a range of soil types can be defined as non-heaving or low heave potential soils. This range will be specified by the percent finer than 0.02 mm, in keeping with previous work by Casagrande and the U.S. Army Cold Region Research and Engineering Laboratory (CRREL).

This range of soil types will therefore not require mitigation for frost heave, as the total predicted heave will be less than the tolerable heave as defined by stress analysis studies.

ii) In more frost susceptible soils, not covered by the range of soils described above, predictions using the ice-segregation ratio method suggest that limiting frost heave can be achieved by limiting frost advance. If the ISR is selected to represent the ice segregation ratio at a low rate of frost advance, the allowable or tolerable frost advance can be related to the tolerable heave by:

$$X_{max} = h_{max}/ISR$$

For example, if the tolerable heave, h_{max}, were found to be 0.3 m based on stress analysis studies, and laboratory or field frost heave evidence suggested a maximum ice-segregation ratio of 30%, the maximum allowable frost advance into frost susceptible soil would be 1.0 m. This allowable frost penetration would then be combined with the results of a series of thermal analyses of the kind described earlier to obtained a design thickness of insulation, and also the possible thickness of granular fill to limit frost penetration in the frost susceptible soil to the desired level. This configuration might be achieved practically by at least two possible design configurations, discussed in the next sub-section.

In highly frost susceptible soils, it may be necessary to limit frost advance into the frost susceptible soil almost completely. This would involve large thicknesses of insulation and granular fill; or the use of heat tracing; or taking advantage of the summer heat introduced through the surface of gravel embankment or the pad beneath a concrete restrained mode.

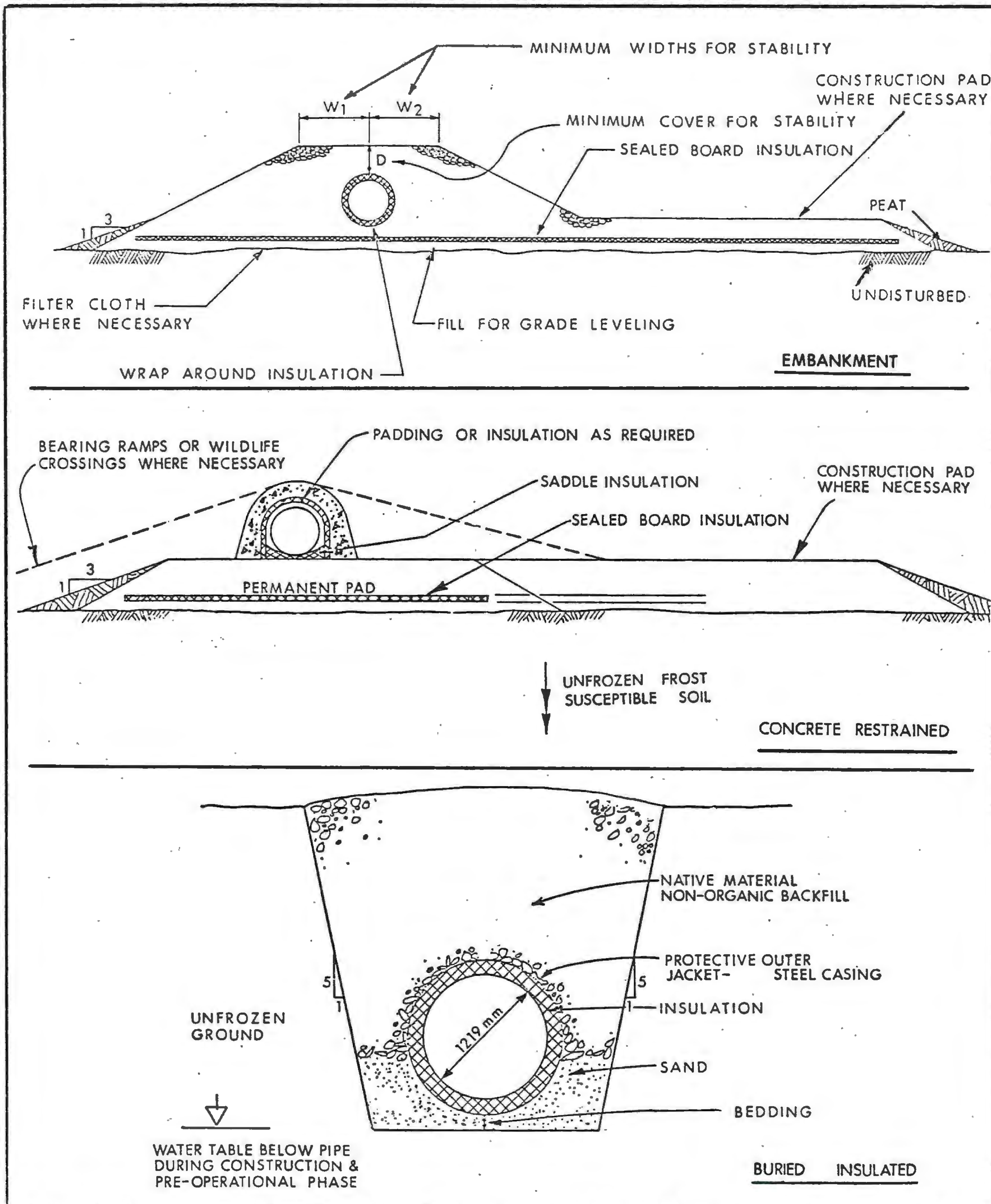
6.0 MITIGATIVE MEASURES

As described in the previous section, design for frost heave in soils other than relatively clean granular materials will involve the use of an above grade mode. Foothills Pipelines proposes to employ two design modes to minimize or prevent frost heave. These are: (a) above grade embankment or concrete restrained mode, or (b) buried insulated pipe with or without heat tracing. Typical layouts for these design modes are given on Figure 10. As shown in the Figure, each mode may have variable thicknesses of insulation and gravel between the pipe and the underlying unfrozen soil.

6.1 Above-Grade

The embankment mode has been studied from a preliminary stress analysis and thermal point of view by Petrotech-Lavelin (1979), and is currently under detailed study by Foothills Pipe Lines (South Yukon) Ltd. The dimensions of gravel around the pipe will be selected to achieve some minimum acceptable lateral and vertical soil restraint. A work pad may be added to one side of the pipe restraining berm to facilitate construction.

A pioneer granular fill is first constructed, and a design thickness of insulation is placed on this levelled pad. A second lift of fill is then placed, and shaped or ditched to receive the pipe. The pipe is then laid on the pad, and fill placed adjacent to, and over the



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DESIGN MODES FOR FROST HEAVING SOILS

K 5500 K

FIGURE 10

pipe to the desired dimensions. The constructability of this mode was rigorously tested and proven at the Foothills Pipe Lines test site at Quill Creek, Y.T. A more complete description of instrumentation and subsoil conditions beneath the embankment test sections may be found in Hardy Associates (1981).

The above grade concrete restraining mode is also presently being studied by Foothills Pipe Lines (South Yukon) Ltd., and is also shown conceptually on Figure 10.

In this case, pipe restraint is obtained by placing a concrete weight over the pipe, which rests on an insulated pad.

The effects of the cold pipe are greatly attenuated by the heavy thickness of synthetic insulation beneath the pipe. Some small frost advance is permissible into the fill beneath the pipe insulation. During the summer, the elevated gravel pad surface warms to a higher temperature than the ambient temperature, and introduces more heat into the ground than would occur in undisturbed ground. This effect has been documented and has sometimes resulted in long-term thawing in warm discontinuous permafrost areas (see for example; Jahns et.al, 1973; or Berg and Aitken, 1973). Geothermal simulations for the embankment will account for the warmer gravel surface temperatures during summer, and the beneficial effects of this will be incorporated in design. The design depth of frost advance in the long term will be limited to maintain heave within tolerable limits as described earlier.

6.2 Buried Insulated Pipe

This mode will generally be employed where the pipe must remain buried below ground surface, such as:

(a) highway crossings;

(b) river or creek crossings;

(c) over short lengths of unfrozen ground where it would clearly be uneconomical to change mode to the embankment.

A ditch will be excavated to some depth greater than pipe burial depth, and partially backfilled with select granular fill. The precise depth and width of the trench are not yet established, and will depend on the results of thermal simulations. The trench dimensions will be selected to limit the depth of frost penetration around and beneath the pipe to some design amount. As detailed previously, this will limit the amount of frost heave to a tolerable quantity, to be established by stress analysis studies.

In cases where thermal analysis indicates that an excessive amount of insulation and/or granular fill is required beneath the pipe to prevent excessive frost penetration, heat tracing will be incorporated into the design. This design may involve two metal pipes buried below and on either side of the insulated pipe. Steam, warm water or warm air can be circulated through these special design sections intermittently, or as indicated by pipeline performance. Electrical heat trace lines may also be used to provide the required heat input to the exterior of the insulated pipe. The requirement for such special design (heat traced) sections is very limited, and will only be employed where other 'no-maintenance' designs cannot be easily applied.

7.0 VERIFICATION OF MITIGATIVE MEASURES

The feasibility of constructing both the embankment and buried insulated modes has been tested at the Foothills Pipe Lines Quill Creek Test Site (see Foothills Pipe Lines, 1981). No full-scale testing of and embankment with a chilled pipe is contemplated. As described in Section 4 of this document, the thermal model has been rigorously verified under field conditions and has been employed successfully on several pipeline thermal simulations in the past. Extensive frost heave test data exists for both laboratory tests and for insulated field pipe test sections. These, in conjunction with other testing of pipeline route soils currently underway will allow conservative or bounding predictions of frost heave to be made. Thus, the appropriate design configuration can be selected.

Respectfully submitted,

HARDY ASSOCIATES (1978) LTD.

Per:

J.F. Nixon, Ph.D., P.Eng.
Project Manager

JFN:lbh

REFERENCES

1. Aitken, G. 1963. Reduction of frost heave by surcharge loading. Proc. 1st Intl. Permafrost Conference, Lafayette, Indiana.

2. Carlson, L., Ellwood, J., Nixon, J. and Slusarchuk, W., 1981. Field Test Results of Operating a Chilled Buried Pipeline in Unfrozen Ground. Proc. 4th Can. Permafrost Conf., Calgary, Alberta.

3. Berg, R. and Aitken, G. 1973. Some passive methods of controlling Geocryological conditions in roadway construction. Proc. 2nd Intl. Permafrost Conf. Yakutsk, U.S.S.R.

4. EBA Engineering Consultants Ltd., 1979. Performance analysis of Calgary Frost Heave Test Facility. Report to Foothills Pipe Lines (Yukon) Ltd.; September, 1979.

5. EBA Engineering Consultants Ltd., 1981. A Brief Description of EBA Geothermal Model. Report to Foothills Pipe Lines (Yukon) Ltd., Calgary, May 1981.

6. EBA Engineering Consultants Ltd., 1981. One-dimensional laboratory frost heave testing - Series 2. Report to Foothills Pipe Line (Yukon) Ltd.; January, 1981.

7. Foothills Pipelines (South Yukon) Ltd., 1981. Quill Creek Test Facility, Design Modes, Construction Procedures and Anticipated Results.

8. Hardy Associates (1978) Ltd., 1978. Installation of frost heave test plates. Report submitted to Foothills Pipe Lines (South Yukon) Ltd.; December 1978.

9. Hardy Associates (1978) Ltd., 1979a. Frost heave testing of granular soils: Beaver Creek and White River, Y.T. Report to Foothills Pipe Lines (South Yukon) Ltd.; January, 1979.

10. Hardy Associates (1978) Ltd., 1979b. Large diameter frost heave testing of granular soils. Report to Foothills Pipe Lines (South Yukon) Ltd.; May, 1979.

REFERENCES (cont'd)

11. Hardy Associates (1978) Ltd., 1980a. Performance of frost heave test plates: Beaver Creek and White River. Report submitted to Foothills Pipe Lines (Yukon) Ltd.; October, 1980.

12. Hardy Associates (1978) Ltd., 1980b. Series I-Log term laboratory frost heave tests. Report to Foothills Pipe Lines (Yukon) Ltd.; November, 1980.

13. Hardy Associates (1978) Ltd., 1981. Quill Creek Test Site: Geotechnical Data Base. Report to Foothills Pipe Lines (Yukon) Ltd., May, 1981.

14. Hoekstra, P., 1969. Water movement and freezing pressures. Proc. Soil Science Society of America. Vol. 33, 4, p. 512.

15. Horiguchi, K. and Miller, R., 1980. Experimental studies with frozen soil in an "ice sandwich" permeameter. Cold Regions Science and Technology, 3. pp. 177 - 183.

16. Jahns, H. et al., 1973. Permafrost Protection for Pipelines. Proc. 2nd International Permafrost Conf. Yakutsk, U.S.S.R.

17. Konrad, J.M., 1980. Frost heave mechanics. Ph.D. thesis submitted to Department Civil Engineering, University of Alberta, Edmonton.

18. Linell, K. and Kaplar, C., 1959. The factor of soil and material type in frost action. Proc. H.R.B. Bull. #225, p. 81.

19. Loch, J., 1979. Influence of the heat extraction rate on the ice-segregation rate of soils. In Frost I Jord, No. 20; May, 1979, p. 19.

20. Nixon, J., Ellwood, J., and Slusarchuk, W., 1981. In-situ frost heave testing using cold plates. Proc. 4th Canadian Permafrost Conference, Calgary.

REFERENCES (cont'd)

21. Penner, E., (1977). Influence of freezing rate on frost heaving. Proc. Highway Res. Bull., #393.

22. Penner, E., 1979. Frost Heave Studies of Granular Soils from Beaver Creek and White River, Y.T. Report to Foothills Pipe Lines (Yukon) Ltd.; October, 1979.

23. Penner, E., 1979. Frost Heave Study of Soil from Frost Heave Test Facility at Fairbanks, Alaska. Report to Foothills Pipe Lines (Yukon) Ltd.; October, 1979.

24. Penner, E. and Goodrich, L., 1980. Location of segregated ice in freezing soil. Proc. 2nd Intl. Symp. on Ground Freezing, Trandheim, Norway.

25. PetrotechLavelin Inc. 1979. Preliminary Embankment Design to Prevent Frost Heave and Thaw Settlement at KP 40. Report to Foothills Pipe Lines (Yukon) Ltd., August, 1979.

26. Slusarchuk, W., Clark, J., Morgenstern, N. and Gaskin P., 1978. Field test results of a chilled pipeline buried in unfrozen ground. Proc. 3rd. Intl. Permafrost Conf., p. 877.

3.0 GEOTECHNICAL REPORT ON THAW SETTLEMENT DESIGN APPROACH

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GEOTECHNICAL REPORT
ON
THAW SETTLEMENT
DESIGN APPROACH
REVISED JANUARY 1982

Prepared For

FOOTHILLS PIPE LINES (SOUTH YUKON) LTD.

Calgary, Alberta

By

HARDY ASSOCIATES (1978) LTD.

Calgary, Alberta

JANUARY, 1982

CG05500K

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APPENDIX "A" Thaw Settlement Correlations

APPENDIX "B" Verification of Thaw Settlement Calculations

1.0 INTRODUCTION

The pipeline will be operated continuously at temperatures below 0°C for the first 214 km of the pipeline route between the Alaska/Yukon border and Station 313.

The remaining portion of the line is proposed to be operated in a warm mode between Station 313 and the Yukon - B.C. border. Between Kluane Lake and the Takhini River (KP 379), a high percentage of the route is underlain by permafrost. If significant amounts of excess ice are present in the ground, the thawing created by the warm pipe would cause thaw settlement beneath the pipe over a period of time and lead to possible pipeline stability problems.

This report reviews the various components of the thaw settlement design process, and in particular assembles the extensive data base available for evaluation of thaw settlement effects in different soils. This information, when coupled with the known extent of frozen, and icy soils, provides a good indication of the extent of the thaw settlement problems, and the associated remedial measures that might be required. In the light of the extensive study that has taken place in the past, the need for future testing or verification of design modes is reviewed.

1.1 Components of Thaw Settlement Design

The primary elements involved with assessing the magnitude of the thaw settlement problem are as follows:

(a) Initial Thermal Condition, i.e. extent of frozen ground.

(b) Thaw settlement data on different soil types and correlations with easily identifiable soil properties such as density and water content.

(c) Thermal predictive model to predict:

  i) depth of thaw under a given pipe (and possibly insulation) configuration;

  ii) effectiveness and design of measures to inhibit or prevent thawing.

(d) A calculation procedure is required to integrate the effects of settlement in different soil layers, and to assess the overall thaw settlement behavior of different terrain units.

These components are reviewed, in order to place the importance of each in perspective.

1.2 Mitigation and Verification

This report then reviews the measures for thaw settlement mitigation, and the likely extent of these are indicated. Based on the level of confidence in the conservative approaches adopted in handling thaw settlement, the extensive field verification that has been carried out in the past, and current studies at Quill Creek, Y.T., no additional field verification is considered necessary.

2.0 EXTENT OF FROZEN GROUND AND THAW SETTLING SOIL

Foothills Pipe Lines (Yukon) Ltd. has carried out extensive programs of terrain analysis, field reconnaissance, geophysical surveys and borehole drilling in the Yukon. As the section of pipeline between Station 313 and the B.C. border is within the zone of warm pipe operation, particular attention was paid to delineating permafrost, and its properties where present. This information is presented in the Geotechnical Atlas, together with information on the available boreholes and geophysical traverses.

2.1 Geotechnical Atlas

The Geotechnical Atlas is submitted separately, and covers the entire Yukon portion of the pipeline route. It is a detailed visual presentation and interpretation of available subsurface information along the pipeline route.

The Atlas illustrates stratigraphy, ice content, soil type, terrain type and whether the ground is frozen or unfrozen. The location of all boreholes, test pits and geophysical traverses completed close to the pipe centre-line are shown. Simplified abbreviated boring logs are included at the top of each sheet. By consulting the Atlas, a quick impression of the presence of permafrost and possible icy subsurface conditions can be obtained. Thaw settlement calculations are also included on the Sheets, and these represent estimates of thaw settlement if the pipe were located at depths of 2 m or 4 m. In addition, estimates of the ground surface settlement are included.

2.2 Borehole Data Bank

Over 1100 boreholes have been drilled in the South Yukon since 1976, and it is necessary to evaluate the settlement that might occur at these borehole locations. As will be seen, the thaw settlement of frozen soil is highly dependent on soil type, water content or bulk density. This volume of data cannot be handled with ease manually, and so a computerized borehole data bank was established. Each borehole log was examined by experienced technicians, and subdivided into a number of depth increments, each having near-uniform water content, density and soil type. The properties of each layer (i.e. unified soil classification, water content, bulk density, visible ice content and any thaw settlement data) were recorded, together with the depth over which the properties are relevent. Each borehole record could contain up to a maximum of eight different layers. The boreholes were identified by borehole number, terrain type, kilometre-post, and offset from the pipe centre-line. Additional information on depth of hole, depth of thaw and depth of permafrost were also included. A blank coding form is given on Figure 1.

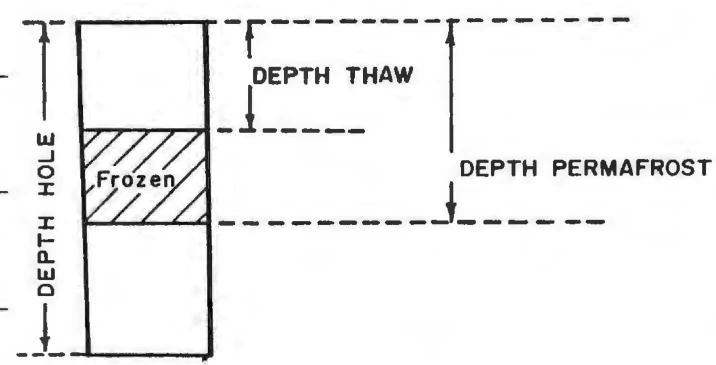
Borehole records stored on the computer can be accessed by construction section, terrain type, borehole number or other identifiers. The computer used was a Hewlett-Packard 9845 mini-computer operated by Foothills Pipe Lines.

Peripheral devices include a "floppy disk" unit and a plotter. For example, with a certain zone or section of the pipeline, rapid access to all boreholes drilled in a certain terrain unit can be obtained. This aids greatly in terrain assessment, thaw settlement analysis, assessment of soil densities for buoyancy control, and the planning of further field programs.

In the present context, the borehole data bank is a major component of the thaw settlement design process. As will be seen, relatively straightforward software computer programs can be written to access the data bank, evaluate the thaw settlement of each borehole assuming the pipe might be founded at different depths, and report on the results.

BOREHOLE NO.	INST.	TERRAIN TYPE	ALIGNMENT SHEET	KP
DEPTH HOLE DEPTH THAW		DEPTH PERMAFROST	COBBLES, BOULDERS	OFFSET (m)

DEPTH (m)		USC	W%	BULK DENSITY	VISIBLE ICE	THAW SETTLEMENT	
1 LAYER							
2							
3							
4							
5							
6							
7							
8							



INSTRUMENTATION

N = NOTHING (DO NOT CODE - IT IS ASSUMED)
T = THERMISTOR
O = EMR PIPE
P = PIEZOMETER
S = STANDPIPE
I = INCLINOMETER
G = SETTLEMENT GAUGE

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BOREHOLE CODING SHEET

K5500K

FIGURE 1

2.3 Assessment of Terrain Units

Prior to carrying out borehole-specific thaw settlement calculations, it was necessary to broadly assess the subsurface characteristics of the different major terrain units in each construction section. To date, the terrain units along the entire route have been reviewed and categorized by Dr. V. Rampton of Terrain Analysis and Mapping Services Ltd., consultant to Foothills Pipe Lines (Yukon) Ltd. Anticipated geological or stratigraphic sequences and geotechnical properties based on this work have been incorporated in the Geotechnical Atlas. In order to obtain more precise subsurface properties for general terrain assessment purposes, the borehole data bank was employed to review the extensive borehole data base, and generate some specific geological and engineering parameters for the subsurface soils in the major terrain units.

This effort has concentrated to date on Construction Sections 213, 214, 215 and 222 for the most part, as these are the sections currently involved in the detailed design process. Detailed profiles of water content and soil type have been prepared for the major terrains along the entire route. In order to represent the subsurface stratigraphic data for many different terrain units along 830 km of the pipeline route, it was necessary to perform a preliminary grouping of terrain units having a similar geological origin, and having similar subsurface properties. Therefore, profiles of the subsurface stratigraphy and properties have been assembled for terrain groups, rather than for individual terrain types. An example of these profiles is given on Figure 2.

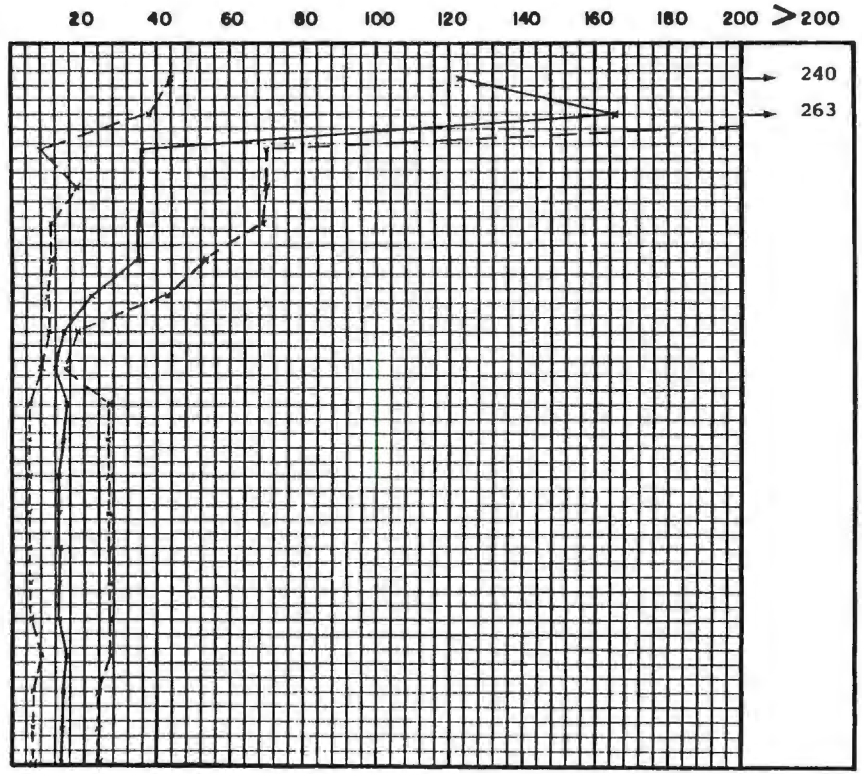
As shown on Figure 2, a quick impression of the predominant soil type may be gained from the plots, and a broad judgement regarding engineering properties as evidenced by water content can be made (i.e. whether wet or dry, icy or ice-free, high or low thaw settlement potential, high or low variability) within a given terrain grouping.

2.4 Extent of Permafrost

Having obtained a method of broadly assessing the geological and engineering properties of each prevalent terrain grouping, it becomes necessary to determine the extent of each terrain type or grouping of terrain types.

This was achieved by reviewing the Geotechnical Atlas, and other project data on lengths of pipeline sections, and creating a file on the computer known as the "Route File". This file recorded each occurrence of all terrain types along the entire route in the South Yukon. As the pipeline route crossed each terrain unit occurrence, the following information was recorded: length of the occurrence, the terrain type, frozen or unfrozen condition, the construction section and the alignment sheet number. An example listing of a portion of this file is included

MOISTURE CONTENT, (%)



DEPTH, (m)

Depth (m)	Soil	No. of Values	DERIVED VALUES w %	DERIVED VALUES ρ_f kg / m³	(USC)
0	PT	13	123		PT
	PT	8	166		
1	M,C	5	36		ML
	M,C	6	36		
2	M,C	7	35		
	M,C	7	25		
3	M,C	9	22		
	S	5	14		
4	S	6	12		S-
	S	9	15		
5	S	8	14		
	S	7	13		
6	S	7	13		
	S	7	13		
7	S	7	13		
	S	7	13		
8	S	6	15		
	M,C	3	14		ML
9	M,C	3	14		
	M,C	3	14		
10					

SECTION: Kilometer Post 131.84 to 220.89

No. OF BOREHOLES = 17

TERRAIN TYPE (S): (Group 5) Alluvial or lacustrine deposits overlying morainal or drift deposits

REVISIONS NO.		DATE	APPR.	BY	CHKD

Foothills Pipe Lines (South Yukon) Ltd.

	DRAWN	CHECKED	APPROVED	APPROVED
BY	M.N			
DATE	MAR/18/81			

TITLE

THE ALASKA HIGHWAY GAS PIPELINE PROJECT

TERRAIN ASSESSMENT
TYPICAL SOIL PROFILES

SCALE

PREPARED BY HARDY ASSOCIATES (1978) LTD

Figure 2

in Table 1. As can be seen from the Table, the number of occurrences of each terrain type, and the extent of each can be quickly ascertained.

Small computer programs have been written to list this file, and generate statistics on the percentage occurrence of each unit, amount of frozen ground etc. within a prescribed section of the pipeline.

The extent of permafrost, or permafrost areas that may require mitigation for thaw settlement are results that can easily be generated from this Route File. The enclosed Table 2 lists some basic statistics for the route south of Kluane Lake. For each Physiographic Region the table indicates the extent of the permafrost, and the extent of areas that may require special design efforts to accomodate thaw settlement. It is observed that the percentage of the route that may require mitigative design for thaw settlement decreases significantly towards the south.

2.5 Summary

Use of the Geotechnical Atlas, and assessments of subsurface terrain properties with the borehole data bank provide a broad impression of the likely surface and subsurface behaviour of a terrain unit following burial and operation of a warm pipeline. Where a terrain unit is considered to have a potential for ground ice and associated thaw settlement, the extent of these areas and the possible associated

TABLE 1

EXAMPLE LISTING OF ROUTE FILE

LENGTH	KP	TERRAIN TYPE	FROZEN OR UNFROZEN	CONSTR SECTION	ALIGN SHEET
720	.720	rfCB/R	1	1	1
190	.910	fAA	1	1	1
680	1.590	fAP	1	1	1
110	1.700	fAA	1	1	1
70	1.770	fAP	1	1	1
120	1.890	fAA	1	1	1
380	2.270	rfCB/R	1	1	1
300	2.570	fAA	1	1	1
925	3.495	rfCB/R	1	1	1
200	3.695	Rx	2	1	1
240	3.935	fAA	1	1	1
230	4.165	o/fAP(Z)	1	1	1
200	4.365	fAP	1	1	1
360	4.725	fAA	1	1	1
1060	5.785	rfCB	1	1	1
360	6.145	fAA	1	1	1
525	6.670	aDB/R	1	1	1
250	6.920	fAA	1	1	1
100	7.020	Rx	2	1	1
250	7.270	fAP	1	1	1
100	7.370	fAP	1	1	2
90	7.460	fAT	2	1	2
930	8.390	fAP(K)	2	1	2
4660	13.050	fAP	2	1	2
710	13.760	fAP(K)	2	1	2
50	13.810	fAT	2	1	2
100	13.910	Wx	2	1	2
420	14.330	fAT	2	1	2
360	14.690	fAT	2	1	3
730	15.420	fAP	2	1	3
2520	17.940	fpOB/gGP	1	1	3
3390	21.330	gGP	2	1	3
950	22.280	gGP	2	1	4
100	22.380	Rx	2	1	4
710	23.090	gGP	2	1	4
810	23.900	f/gAT	2	1	4
240	24.140	f/gAP(A)	2	1	4
50	24.190	gAC(A)	2	1	4
470	24.660	f/gAP(A)	2	1	4
920	25.580	fAB/aGP	1	1	4
2790	28.370	fAA	1	1	4
490	28.860	fAA	1	1	5

NOTE: (a) 1 = Frozen; 2 = Unfrozen

TABLE 2

EXTENT OF PERMAFROST SOUTH OF KLUANE LAKE

PHYSIOGRAPHIC REGIONS (KP RANGE)	LENGTH OF REGION (km)	FROZEN LENGTH (km)	PERCENTAGE OF PERMAFROST %	PERCENTAGE OF ICE-RICH PERMAFROST* %
226.3 - 314.5	88.2	41.4	46.9	24.3
314.5 - 439.0	124.5	24.4	19.6	13.3
439.0 - 549.6	110.6	18.7	16.9	11.1
549.6 - 662.0	112.4	20.8	18.5	13.4
662.0 - 769.1	107.1	3.9	3.6	0.5
769.1 - 829.5	60.4	1.9	3.1	3.1
226.3 829.5	603.2	110.4	18.3	11.2

* Potential extent of mitigative design

43.134

mitigation for a warm pipeline can be determined by reviewing the Route File on the computer.

3.0 THAW SETTLEMENT TEST DATA

3.1 Foothills Test Data

Thaw settlement tests have been performed on frozen core smaples obtained during the various drilling programs along the route since 1976. The earlier test data, including a significant testing program in the Summer of 1978, formed the basis for a preliminary evaluation of thaw settlement and a preliminary grouping of terrains according to thaw settlement potential. These preliminary calculations for thaw settlement along the permafrost areas of the route were included on early editions of the Geotechnical Atlas.

Subsequent geotechnical investigation programs by Foothills Pipe Lines have supplemented the earlier data. Particular emphasis has been made to more clearly define the thaw settlement characteristics for soils with relatively low ice contents. Also, the testing programs have concentrated on increasing the data base for the more dominant soil types encountered along the route.

A complete listing of data sources for the thaw settlement testing carried out for Foothills Pipe Lines is contained Table 3. In

TABLE 3

LIST OF THAW SETTLEMENT DATA REPORTS

PREPARED FOR FOOTHILLS PIPE LINES (SOUTH YUKON) LTD.

TITLE	COMPANY	DATE OF REPORT	NO. OF TESTS
Thaw Consolidation Test results and estimated Settlements	Klohn Leonoff Consultants	December 31, 1976	24
Geotechnical Laboratory Data Report - Summer 1978 Drilling Program	Hardy Associates (1978) Ltd. Calgary	December, 1978	57
Geotechnical Field Data Report - Summer 1979 Drilling Program	Hardy Associates (1978) Ltd. Calgary	November 9, 1979	19
Geotechnical Data Report 1980 Winter Drilling Program	EBA Engineering Consultants Ltd. Calgary	July 1980	23
Geotechnical Report Compressor Stations Summer 1980 Drilling Program	Hardy Associates (1978) Ltd. Calgary	April 1981	3
Geotechnical Laboratory Data Report - Summer 1980 Drilling Program	Hardy Associates (1978) Ltd. Calgary	April 1981	47
Geotechnical Data Base for Quill Creek Test Facility	Hardy Associates (1978) Ltd. Calgary	May 1981	12

all, 185 tests have been performed prior to the Summer of 1981 of which 81 were available at the completion of the 1978 testing program.

Generally, tests have been performed on cores obtained from below the nominal pipe burial depth of 2 to 3 m. Some cores from above this depth have been tested, however, to provide representative data for settlement predictions for the ground surface within the pipeline right-of-way. The shallower tests were generally performed on samples of peat, organic silts and volcanic ash. The deeper cores tested usually consisted of sands, silts and clays. Only a limited number of gravel cores have been obtained and tested in Foothills programs. Other data is available, however, for design purposes.

The actual test procedures used in all of the tests is outlined in the respective reports (Table 3). Basically, a frozen core is thawed out under some nominal (5 kPa) pressure and then two or more subsequent load increments (e.g. 40 kPa and 80 kPa) are applied and detailed time-settlement data are obtained under each load. This test procedure permits a distinction between the settlement attributable to thaw of excess ice alone, and additional settlement attributable to an increase in pressure. The actual interpretation of the results will be discussed in a later section.

Moisture content and frozen bulk density tests are carried out on the test specimens to enable a correlation between thaw settlement and these more readily available soil properties. Soil classification tests (grain size distribution or Atterberg limits) are often carried out to confirm the soil type. Correlations between thaw settlement and both mositure content and frozen bulk density for various soil type are discussed in Section 3.3. The frozen density and initial frozen water content are evaluated on a routine basis for permafrost boreholes, and consequently these correlations between thaw strain and these easily obtainable soil properties allow a more detailed layer-by-layer evaluation of thaw settlement, without resorting to a great number of the more expensive thaw settlement tests.

3.2 Other Available Thaw Settlement Data

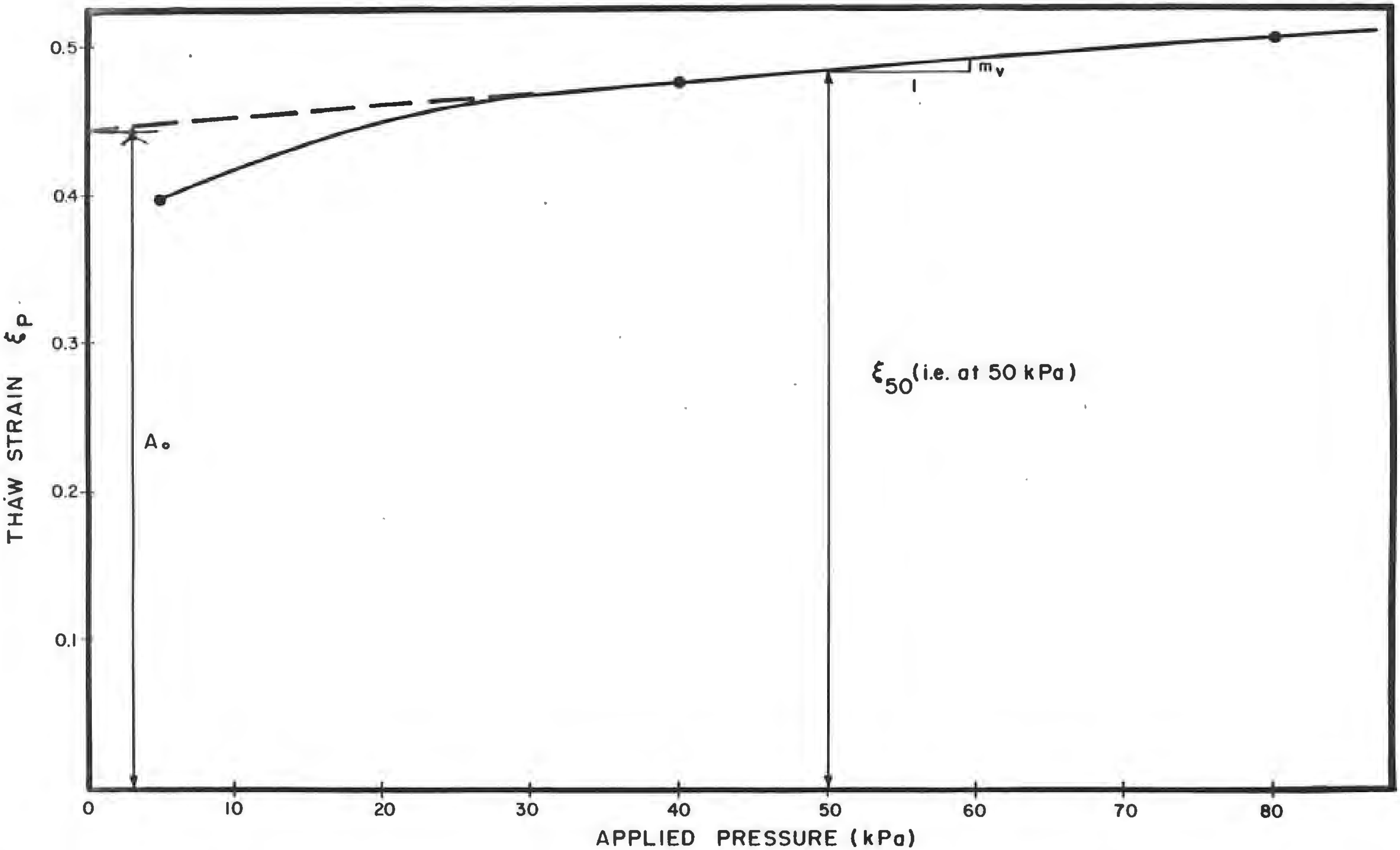
Two main sources of thaw settlement data have been used to supplement the Foothills data: Canadian Artic Gas Study Ltd. data (see McRoberts et al, 1978) and secondly; data published by Luscher et al (1978). The former data comprises mostly of tests on fine-grained soils and peats, while the latter provides data on some relatively ice-poor gravels.

In order to determine whether the Canadian Arctic Gas Study Ltd data was valid or representative for application to the South Yukon route, all of the data from the two sources was plotted separately. A visual comparison was made for each of the soil types represented, i.e. peats, low plastic fine-grained soils and high plastic soils. The test data for peats was found to differ very considerably and so the Canadian Arctic Gas Study Ltd. data for peat has not been used. The other soil types were found to compare favourably and this is discussed later in the actual data presentation.

The test data for gravels presented by Luscher et al (1973) represents the only major source of data on gravels and therefore no real comparison with Yukon gravels is possible. However, no real discrepencies between Alaskan and Yukon gravels need be expected, due to the general similarities of granular materials from either location.

3.3 Thaw Settlement Correlations

As discussed earlier, the thaw settlement test data provides the strain due to thaw alone and also the strain due to subsequent load applications to the thawed soil. A typical test result is illustated in Figure 3, where the thaw strain parameter A_o, represents the strain under essentially zero pressure. The strain under greater pressure is shown by the line for which the slope is m_v such that the thaw strain under any given pressure is expressed by:



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STRAIN VERSUS APPLIED PRESSURE FOR A THAW SETTLEMENT TEST

K5500K

FIG. 3

$$_p = A_o + m_v . p \qquad (3.1)$$

where: $_p$ = thaw strain at pressure, p

p = the effective pressure (kPa)

m_v = coefficient of compressibility (1/kPa)

For the purposes of thaw settlement evaluation over the pipeline route the thaw settlement test data have been simplified by selecting the thaw strain at a representative effective pressure (50 kPa). This pressure conservatively represents the in-situ effective pressure in the upper layers of the thaw bulb where most of the thaw settlement occurs. The actual pressure selected is not very critical for general route evaluation as the major component of the thaw strain value is the initial thaw value A_o.

The thaw strain value $_{50}$, at 50 kPa, has been obtained from all the available test data by reading the value from a data plot similar to Figure 3. In addition to the comparison between the various sources of data (Foothills and CAGSL), the test data was plotted separately by soil type and by test program. This enabled a systematic determination of the reliability of all data and the confirmation of the soil groupings to be used. Based on the visual comparisons of all data the soil groups presented in Table 4 were selected.

TABLE 4

CG05500K

THAW SETTLEMENT CORRELATIONS

DEFINITION OF SOIL GROUPS FOR

(as established for computer program)

SOIL GROUP NO.	SOIL SYMBOL	SOIL DESCRIPTION
1	PT	Peat
2	VA	Volcanic Ash
3	GW	Gravel, well graded
	GP	Gravel, poorly graded
	GM	Gravel, silty
	GC	Gravel, clayey
	SW	Sand, well graded
4	ML	Silt, low plastic
	CL	Clay, low plastic
	SM	Sand, silty
	SP	Sand, poorly graded
	SC	Sand, clayey
5	MH	Silt, high plastic
	CI	Clay, intermediate plasticity
	CH	Clay, high plastic
6	BR	Bedrock
7	I	Ice-plus, very high ice content
8	Error	Computer check for invalid soil types
9	OL	Organic silt, low plastic
	OH	Organic silt, high plastic

The listing has been established for the thaw settlement computer program and hence the "ERROR" group which simply identifies incorrect input. Other points to note in the listing are the placing of "SW", well graded sand, with the gravels. This is because the data from Luscher et al (1973) for gravels includes some data for SW, and is the only data for this soil type. Also, the organic silts, OL and OH, which were initially grouped with peat, PT, were found to be sufficiently different, and were separated into different soil groups.

Correlations for the actual thaw settlement at 50 kPa with both frozen bulk density and moisture content are presented for each of the soil groups 1 to 5 and 9, on the Plates A-1 to A-12 in Appendix A.

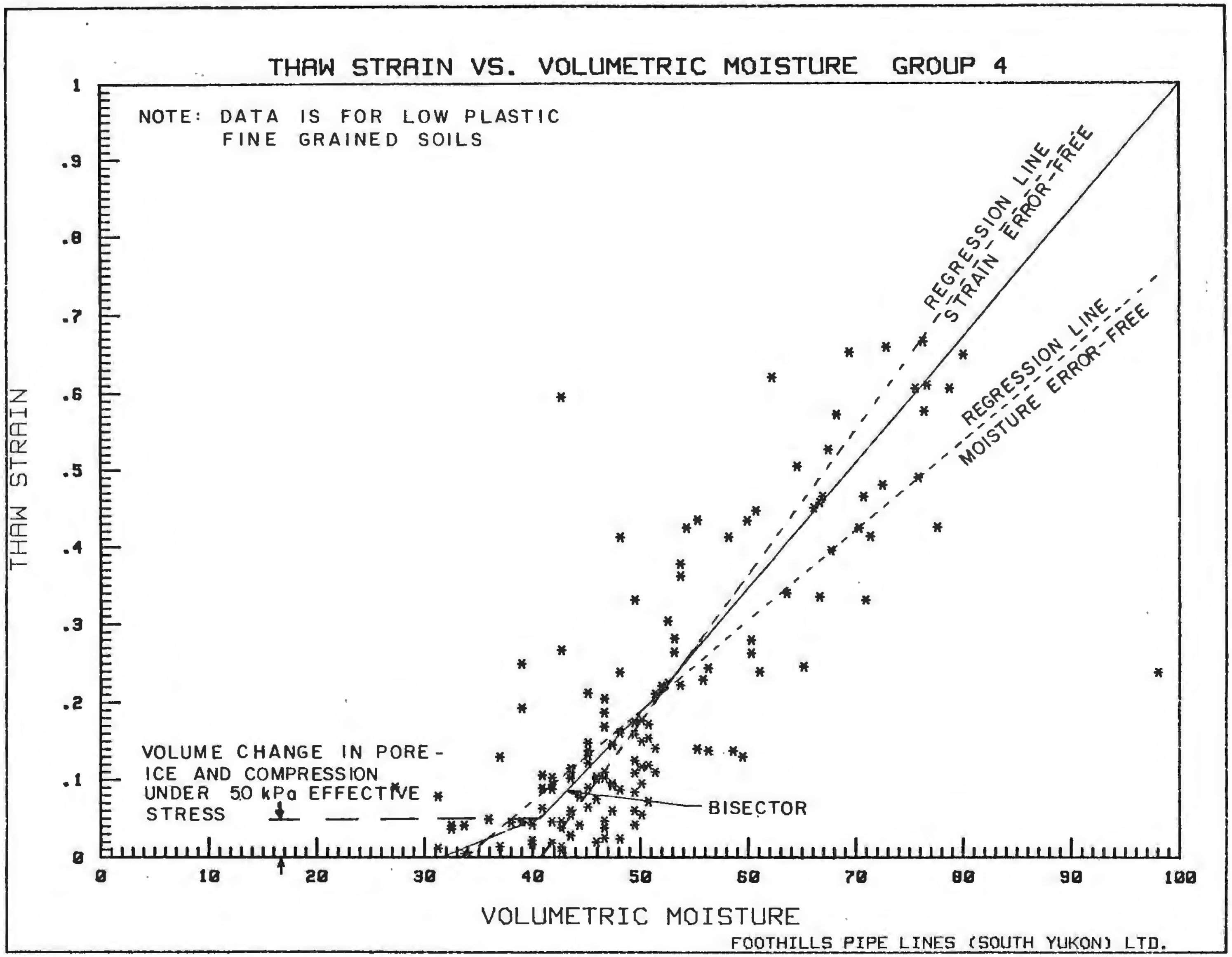
For the peat, volcanic ash and organic silts, the only data available is from Foothills Pipe Lines data sources. For the gravel the majority of the data is in the very low ice/moisture content range as shown in Plates A-5 and A-6. Two Foothills and one Arctic Gas test provide some means for extrapolating to higher ice content gravels. Such high ice content gravels are infrequent so the lack of data is not a major concern.

Sands and low plastic silts and clays form the most dominant soil group encountered along the pipeline route. The Canadian Arctic Gas Study Ltd. test data (35% of total) is generally contained within the scatter of the Foothills data (Plates A-7 and A-8) and is therefore considered acceptable for present design purposes.

The Arctic Gas data for high plastic soils forms about 75% of the total data, due primarily to a lesser occurrence of this soil type in the South Yukon. The data falls to one side of the Foothills range of data at the low ice/moisture range but provides almost the only data at higher ice content ranges. All of the data has been included, thereby producing a relationship somewhat on the conservative side.

The thaw settlement correlations that have been established for the thaw settlement computer analysis are shown on each of the Plates A-1 to A-12 in Appendix "A". The correlation between thaw strain and the volumetric moisture content and frozen bulk density for all of the test data has been statistically determined for each of the soil groups. Volumetric water content was chosen as a dependent variable rather than the more usual gravimetric water content, as the full range of volumetric water content is confined between 0 and 100%. The relationship between volumetric and gravimetric mositure contents is shown for the various soil groups in Plates A-13 to A-17, Appendix "A".

Because there is the possibility of error or random variation in both the thaw strain value measured and the moisture content or bulk density determined for each test, there exists a joint probability distribution. Two "best fit" lines have been obtained by linear regression considering each variable error-free respectively, as illustrated in Figure 4. The best estimate of the true correlation lies between these two lines. (Lyon, 1970)



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TYPICAL THAW STRAIN CORRELATION

K5500A

FIGURE 4

HT10 - 79/05

The actual correlations have been obtained as follows:

- for the upper thaw settlement range the correlation has been defined to provide a thaw strain of 1.0 when the sample becomes pure ice (vol. moisture content 100%; frozen bulk density 917 kg/m^3). This upper limit thaw settlement is joined to the intersection of the two regression lines (see Figure 4).

- the lower portion of the correlation is taken as the bisector of the two regression lines, below the point of intersection.

In the case of Soil Group 4, the moisture content (37% by vol.) for which zero thaw settlement was indicated by the bisector appeared too high. This corresponds to a dry weight moisture content of about 23% and a frozen bulk density of about 1960 kg/m^3. It is considered that some thaw settlement could occur in these low-plastic, fine-grained soils at these moisture and density values. Therefore the lower portion of the correlation for Group 4 has been constructed as follows:

- the moistures and density corresponding to zero thaw strain are judged to be:

moisture content (dry wt.)	= 17.0%
moisture content (vol.)	= 31.3%
frozen bulk density	= 2165 kg/m^3

- from this lower limit, the correlation is projected, as shown on Fig. 4, to a point on the bisector where the thaw strain is a minimal value due to volume change in the pore-ice and compression on thawing under the 50 kPa effective stress value adopted for all the thaw strain data.

All of the actual thaw strain correlations are presented in Appendix "A". Most of the correlations are bilinear in form and the "standard error" of the thaw strain data about the mean has been determined for each linear portion of the correlations. The values obtained are shown on the respective plates in Appendix "A." This represents the statistical error involved in applying the mean thaw strain value to calculate the thaw settlement for each individual soil layer.

This section has reviewed the very extensive data base for thaw settlement behaviour in permafrost soils, relevant to this project. The following sections will demonstrate how these data, and the derived thaw settlement correlations are introduced into the overall design procedure.

4.0 GEOTHERMAL PREDICTIONS

4.1 Use of Thermal Predictions in Design

The geothermal model (EBA, 1981) is intimately involved in the design process for thaw settlement in the following ways. Initially, thermal simulations are carried out for a non-insulated pipe at standard burial depths for different pipe temperatures and soil conditions. This provides (a) the ultimate depth of thaw (i.e. the zone of soil that may eventually be thawed and; (b) the time rate of thawing. The latter data are required in assessing any excess pore pressures that may develop within the thaw bulb, and that might have an effect on stability of sloping terrain.

While the geothermal model has been used for the more rigorous analysis of specific situations and soil profiles, several generalized, uniform soil profiles have been analysed using a quasistatic solution. (EBA, 1981). Thaw depths predicted for different pipe temperatures for varying durations have been used as the basis for selecting the base of the thaw bulb for thaw settlement calculations. This is primarily dependent on soil type and water content, as these influence the thermal conductivity of the thawed soil, and the soil latent heat.

If the soils contain ice in excess of some tolerable amount as specified by ongoing pipeline stress analysis studies, the thermal model is employed to evaluate the thaw depth with time for two special design modes discussed later. The first involves deeper burial to access a stratum exhibiting a lower potential for thaw settlement. Thermal simulations have therefore been carried out for pipes buried at various depths. It is generally found that the depth of thaw, when measured below the base of pipe, is only weakly dependent on the pipe burial depth. If this design mode (i.e. deeper burial) reduces thaw settlement of the pipe to less than the specified tolerable level, then this is considered to be the appropriate design mode, subject to ensuring that ditchwall and uplift resistance are adequate.

If the subsurface permafrost remains icy below readily accessible depths, then the pipe will generally be consturcted in an above-grade mode. This mode is discussed later, and basically involves surrounding the pipe with granular fill or concrete to maintain pipe stability, and introducing synthetic insulation between the warm pipe and the underlying permafrost.

Thermal analyses are being performed relating long term thaw depth to (a) the thicknesses of fill and insulation beneath the pipe, (b) location along the pipeline route and (c) pipe temperature. At any given location, the year-round average pipe operating temperature and climatic conditions are known, and the depth of fill and insulation required within the embankment beneath the pipe can be determined. The depth of thaw will be limited to some tolerable level depending on the thaw strain of the near-surface soils, and the allowable total settlement determined from ongoing stress analysis studies. The thermal calculations therefore form an integral part of the design procedures to mitigate thaw settlement.

The thermal properties of insulating materials and earth materials are known with sufficient accuracy that, following a conservative selection of input parameters for geothermal analysis, the maximum depth of thaw can be predicted with confidence. As stated in the previous sub-section, the formulation and accuracy of the numerical solution have been compared many times in the past against known exact analytical solutions and field performance, giving agreement between prediction and observation.

5.0 THAW SETTLEMENT PREDICTIONS AND TERRAIN STATISTICS

5.1 Thaw Settlement Computer Program

A computer program has been developed to calculate thaw settlement for both the pipe and the right-of-way (ROW). The settlement can be calculated for each individual borehole and for any given depth range by defining the depths to the top and bottom of the thaw zone. A flow chart for this program is presented in Figure 5.

5.1.1 Thaw Zone

A major portion of the program relates to the definition of the thaw zone between depths X_1 and X_2. Unfrozen and shallow (3 m) holes are not selected for thaw settlement analysis. Using the soil information as stored in the borehole data bank (refer to Section 2.2) the program selects the soil layers or partial soil layers that occur between the specified depth range X_1 to X_2. Another feature is that soil conditions for the bottom layer of any borehole are extrapolated to the full depth of thaw X_2.

Thaw depths for buried, uninsulated pipe are expected to range from about 14 m to 28 m below the surface depending on pipe temperatures, the soil type, the moisture content and the pipe burial depth. The moisture content, or more precisely the ice content, is the most significant variable in relating thaw depths to thaw settlement. For the purposes of confirming which terrain types are suitable for a design mode involving a buried, uninsulated pipe, the range of thaw depths that are of interest are for soil profiles with a sufficiently low enough moisture content that the thaw settlement would be below a certain critical value, and therfore suitable for standard pipe burial. Stated in another way, soil profiles having high water contents were not used for determining the thaw depth range for settlement analysis, as these soil types would not permit standard pipe burial in any case due to excessive thaw settlement.

As an illustration, it can be shown for the established thaw settlement correlations and an average pipe temperature of 20°C, the average moisture content within the thaw bulb is limited for a given amount of settlement as follows:

Average Moisture Content (% by dry wt) Within the Thaw Zone for Predicted Settlement (20°C Pipe).

Soil Type	Approx. Moisture Content for 0.3 m Settlement	Approx. Moisture Content for 1.0 m Settlement
Clay	21	27
Gravel	9	18

The thaw settlement analysis that is incorporated into the pipeline design is directed primarily towards soil profiles with relatively low ice/moisture contents. For moisture contents higher than the above values, the thaw depth would be correspondingly less. As mentioned above, the thaw settlement would be greater for such moisture content profiles, and buried uninsulated pipe would not be considered. The thaw settlement analysis therefore generally overestimates actual settlement values for situations where the settlement would be greater than about 1.0 m, but no refinements are considered necessary as this settlement will almost certainly exceed the tolerable levels.

The generalized soil profiles described in Section 2.3 indicate that for most profiles, the moisture contents remain low below the surficial ice-rich layer, (See Figure 2). The fine-grained soils indicate more variable moisture contents at depth and hence an above-grade mode is considered for many fine-grained terrains. Terrain units considered suitable for standard pipe burial are more often underlain by coarse-grained soils. While the majority of the boreholes are a maximum 10 m deep, all the data available indicate that moisture contents should remain low if not actually decrease below the 10 m depth. Therefore, provided the soil moisture contents between about 3 and 10 m in any given borehole are low enough to be considered for a buried pipe design mode, it is valid and even conservative to extrapolate the conditions for the bottom layer in the borehole data bank to the full depth of the thaw bulb. Some deeper boreholes were drilled during the Summer 1981 Drilling Program to verify this. In all cases the moisture contents below 10 m depth were consistently lower than the moisture above 10 m.

5.1.2 Settlement Calculation

The actual thaw settlement calculation is carried out for each individual borehole, layer by layer and then summed to provide the total settlement prediction for the thaw zone defined. The thaw strain values of the individual layers are obtained, as shown on Figure 5, on a preference basis as follows:

- from the correlation between thaw strain and moisture content where moisture content available.

- from the correlation between thaw strain and frozen bulk density where frozen bulk density available.

- from the visible excess ice content where none of above available. Thaw strain is taken as equal to visible excess ice content with a correction factor applied as required.

- from the correlation between thaw strain and moisture content where the layer moisture content under consideration is interpolated from the layers above and below.

The actual thaw settlement is calculated for each borehole as:

$$S_B = \Sigma (X_i)(\varepsilon_i)$$

where S_B = the borehole settlement

X_i = thickness of soil layer

ε_i = mean thaw strain for layer obtained from the correlation for soil type

The standard deviation in the above settlement value is calculated based on the standard error for each soil layer and the layer thickness as follows:

$$\sigma_B = \sqrt{(X_1 \cdot s_1)^2 + (X_2 \cdot s_2)^2 + \ldots (X_n \cdot s_n)^2}$$

where σ_B = the individual borehole standard deviation

X_1 = thickness of layer one

s_1 = standard error in the mean thaw strain for layer one

These above relationships for S_B and σ_B provide the mean settlement and the standard deviation for each borehole.

For most design purposes it is desireable to evaluate the thaw settlement potential for a given terrain unit by examining all the boreholes in that terrain unit. Therefore, the mean value of settlement for a terrain unit with M boreholes

$$\bar{S} = \frac{(S_B)_1 + (S_B)_2 + \ldots\ldots (S_B)_M}{M}$$

and the overall standard deviation for the terrain unit is

$$\bar{\sigma} = \sqrt{\frac{(\sigma_B)_1^2 + (\sigma_B)_2^2 + \ldots\ldots (\sigma_B)_M^2}{M}}$$

For any required confidence level, the thaw settlement can then be stated as

$$S = \overline{S} + Z_c\,(\overline{\sigma})$$

where Z_c = the number of standard deviations for a given confidence level

= 0.0 for 50% confidence level

= 1.04 for 85% confidence level

= 2.33 for 99% confidence level

5.2 Verification of Thaw Settlement Program

Several actual borehole data records and artificial data records were run on the program to ensure that all sections of the program were following the intended logic of the flow chart. All necessary modifications were made and rechecked. The final verfication is documented in Appendix "B", and shows that the algorithm for thaw settlement is being implemented by the computer program as intended.

5.3 Presentation of Thaw Settlement Data

The presentation of the thaw settlement data will take two forms. General thaw settlement values will be shown on the revised Geotechnical Atlas, and more specific thaw settlement predictions will be made for detailed design purposes and will be presented in the form of computer print-outs.

The general thaw settlement values will represent the thaw of the surface of the right-of-way as well as the thaw below nominal pipe burial depths of 2, 3 and 4 m. The base of the thaw is taken as 10 m in this situation as, with the exception of some fine-grained terrains, the ice-rich soils are confined well within the upper 10 m. The settlements on the revised Atlas will be the 85% and 99% confidence values for each particular terrain type within a physiographic region. These data will provide a general comparison of the thaw settlement potential of the various terrains. This comparison is essentially independent of the actual pipe burial depth, the actual pipe operating temperature and the actual thaw depth, all of which may be subject to change until the design is actually finalized. The data is sufficient, however, to complement all the other soil and permafrost profile information on the atlas and to indicate the relative potential for thaw settlement along the route.

The more specific thaw settlement predictions to be presented for design purposes are based on the actual pipe operating temperatures for a given portion of the route, the actual thaw depths that have been predicted by thermal analyses and for a wider range of potential burial depths between 2.0 and 5.0 m in 0.5 m intervals.

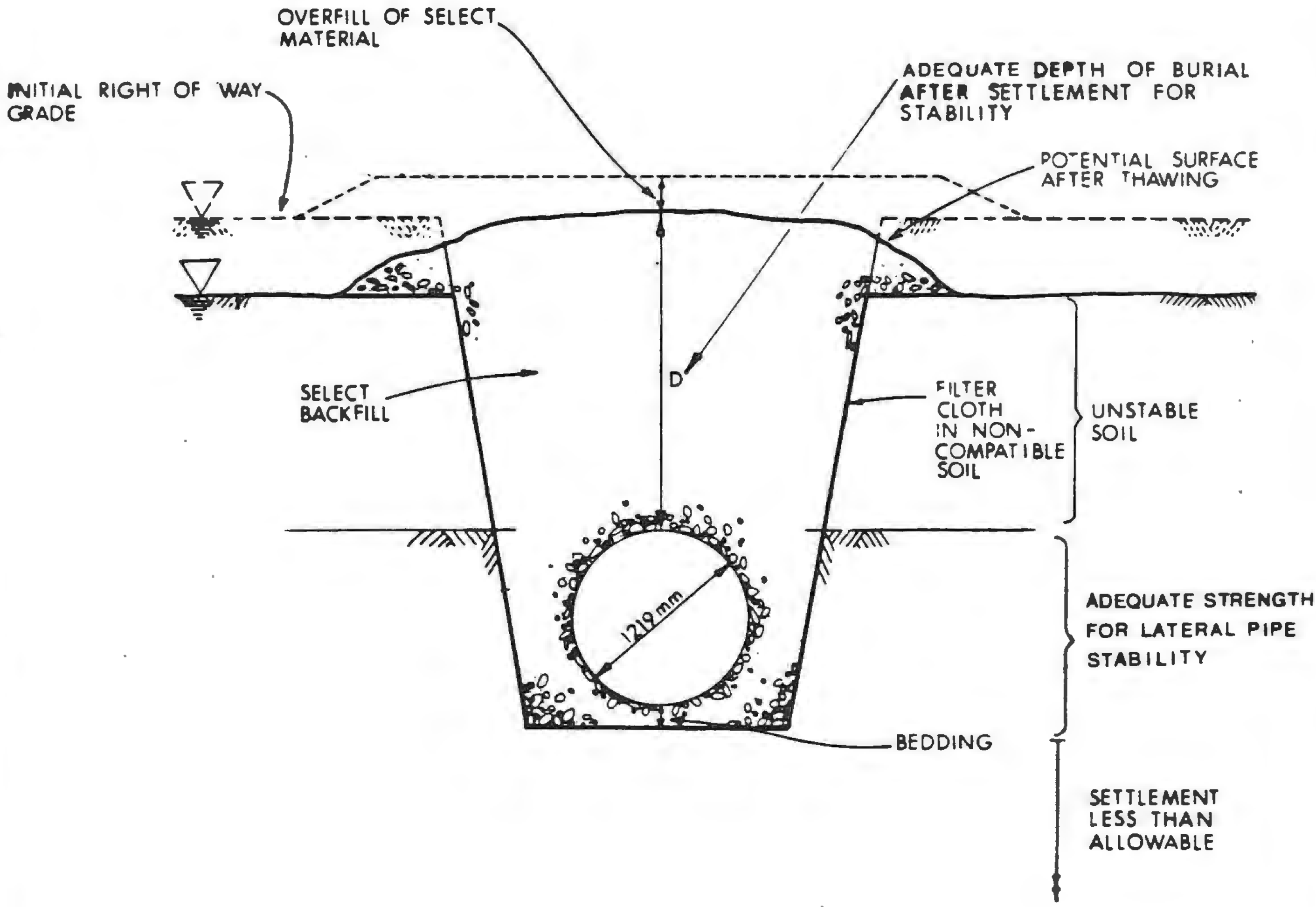
The settlement values are again presented for the different terrain types within a physiographic region and three values are provided: the mean settlement value based on all the boreholes within the terrain type; the settlement value for which there is a statistically determined confidence limit of 85% that settlement would not be exceeded and; a corresponding value for which there is a 99% confidence limit that it would not be exceeded. While these statistically determined data will provide the major input into final thaw settlement design, the actual thaw settlement predictions for each individual borehole will also be available in the form of computer print-out and will be included in the final thaw settlement design reports for each construction section.

6.0 MITIGATIVE MEASURES

Assuming that the predicted settlement for a pipe buried at a standard depth (pipe invert approx. 2 m below original grade) exceeds some tolerable level, the design procedure will involve selecting one of seveal design modes to mitigate the thaw settlement problem.

6.1 Deep Burial

The first option is to bury the pipe deeper, thereby accessing a more stable stratum, if available, (example Figure 6). This mode will be particularly appropriate where icy surface soils extend to depths greater than standard pipe burial depth, but are underlain at reasonable depth by dense, thaw stable soil types. The practical limits to the depth of ditch are normally 3.5 to 4 m though in special situations up to 5 or even 6 m may be considered. Should icy soils extend to depths greater than this for significant lengths of pipeline, the next design option is considered.



NOTE:
DIMENSIONS AND SPECIFICATIONS WILL VARY WITH LOCAL OPERATING PIPE GEOMETRY AND SOIL CONDITIONS

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DEEP BURIAL DESIGN MODE

K 5500 K

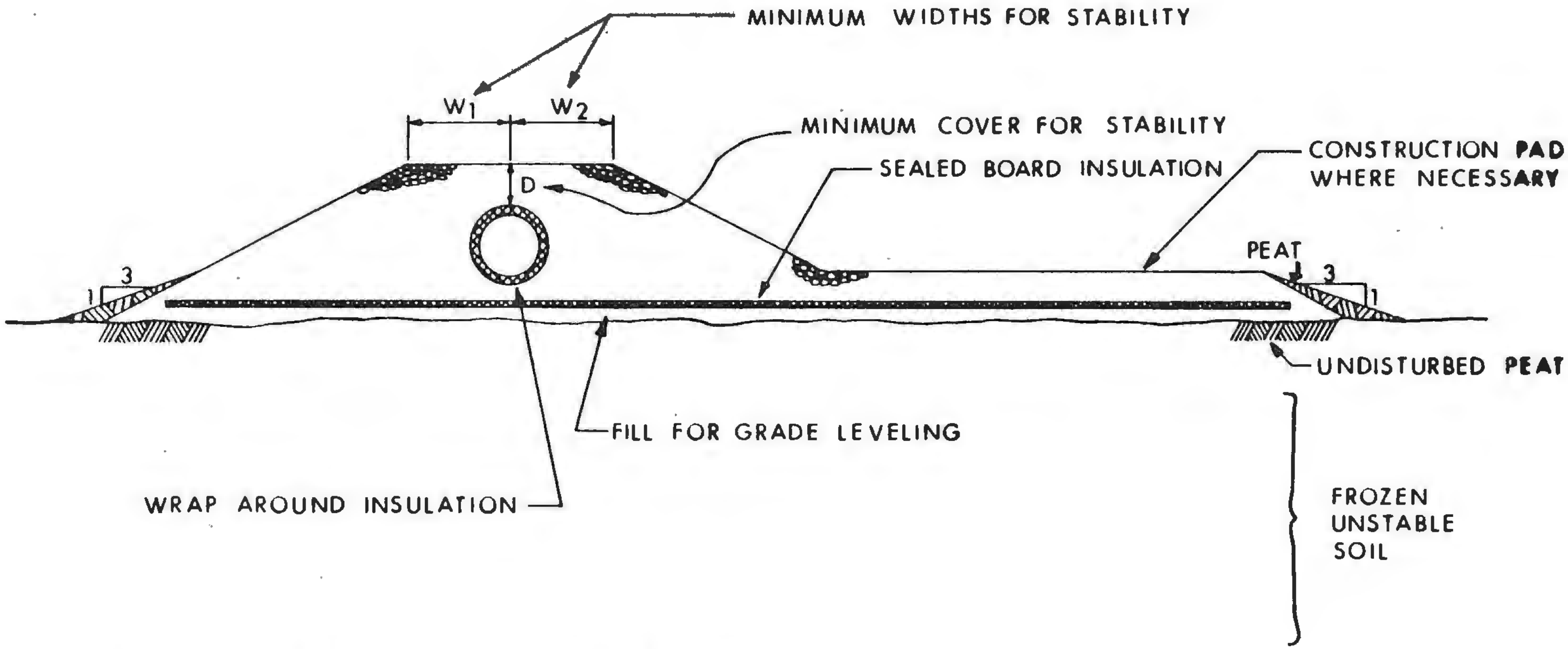
FIG. 6

6.2 Above Grade Modes

6.2.1 Insulated Gravel Embankment

This will involve construction in an above-ground granular embankment, with an associated construction pad where necessary (example Figure 7). The crest width and thickness of cover over the pipe will be dimensioned so as to provide adequate lateral soil restraint and uplift soil resistance for pipe stability. This will normally involve about 1.0 m of cover over the pipe for nominally straight pipe sections, with considerable increases in granular cover thickness required at overbends. The dimensions W_1 and W_2 on Figure 7 may be varied to restrain the pipe at sidebends as required.

A pioneer or levelling fill is placed on the ground surface. A thickness of synthetic insulation is then laid across the entire embankment and workpad area (if present). This layer will normally be from 5 to 10 cm in thickness. Subsequent lifts of granular material are placed to an elevation equal to the pipe springline. This fill is then shaped, and the pipe with up to 200 mm of preformed insulation is placed on the pad. Gravel placement will then continue until the desired fill dimensions are achieved. This procedure will provide a reasonable level of compaction in the fill up to the springline of the pipe, and relatively loose fill above this elevation.



NOTE: DIMENSIONS AND SPECIFICATIONS WILL VARY WITH LOCAL OPERATING PIPE GEOMETRY AND SOIL CONDITIONS.

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INSULATED GRAVEL EMBANKMENT

K 5500 K

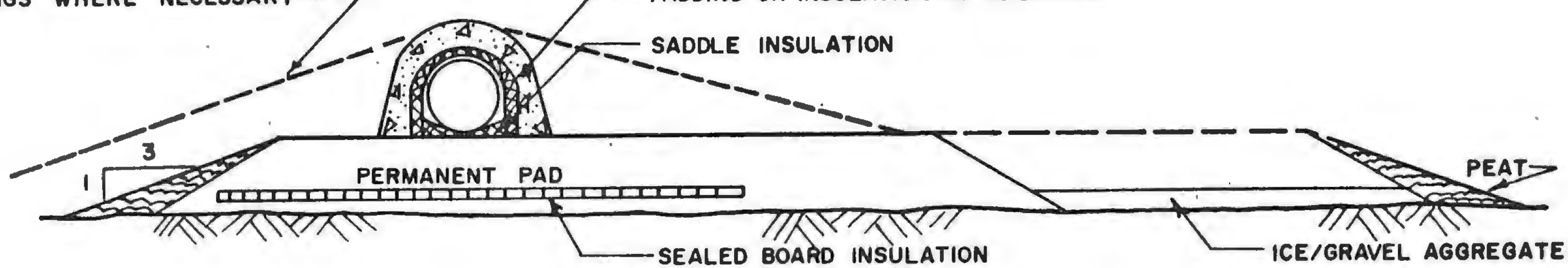
FIG. 7

6.2.2 Concrete Restrained

In this above-grade mode the permanent pad and construction pad are designed and constructed in the same manner as for the gravel embankment (example Figure 8). The pipe rests on an insulation saddle placed on the pad. Then a concrete weight complete with insulation, is set over the pipe. Berms will be placed at sidebends and for wildlife crossings, as required.

6.3 Buried Insulated Pipe

This mode will only be considered where an above-grade mode would otherwise intersect a highway, and a buried crossing of the highway is required. The mode might also be employed on isolated patches of icy permafrost near river crossings, where the pipeline would otherwise be constructed in an above-grade mode. Details of this mode have not been examined in detail, but it will involve a ditch over-excavated and replaced with select granular fill, and sufficient thickness of pipe insulation to limit thawing beyond the granular fill layer to an acceptable amount. This mode will not be used extensively, and will be confined to locations where it is highly desireable to maintain the pipe in a buried mode, even where icy permafrost conditions are present.



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CONCRETE RESTRAINED MODE

K 5500 K

FIG 8

6.4 Special Designs

Certain combinations of difficult or unstable terrain conditions may arise in isolated locations, so as to prevent the use of the design modes reviewed so far. For example, thick deposits of icy, fine-grained permafrost have been delineated at a few locations in the more southerly part of the pipeline route. Construction disturbance, placement of gravel and other activities may result in a slow warming trend in the permafrost, and it may prove difficult to prevent ultimate thawing beneath the pipeline in the long term. Should thermal simulations involving the embankment mode in these areas indicate this possibility, then it may become necessary to install heat removal devices such as heat pipes (known commerically as thermo-piles or Cryo-Anchors) into the subsoils at regular intervals along the embankment. Pipeline stress analysis studies would be carried out to indicate the necessary spacing of these devices to maintain sufficient frozen ground for the vertical support of the pipe. Normally, these devices can maintain a year round bulb of frozen ground around a pile up to 3 m in radius. Thermal studies would be carried out to determine the best type of heat removal device, its optimum location in relation to the pipe, and the size of the bulb of frozen ground that can be expected at each heat removal device location.

The requirement for these special designs will be examined and applied on a site-specific basis, and are not anticipated to be necesary at more than a few isolated locations in the more southerly part of the route in the South Yukon.

7.0 VERIFICATION

Full-scale experimental verification of some of the different design modes has been carried out at various locations.

7.1 MVPL at Inuvik, N.W.T.

Mackenzie Valley Pipeline Research Ltd. constructed a 0.61 m diameter warm test pipe near Inuvik, N.W.T., and these results are reported by Watson et al (1973). A layer of icy silt approximately 2 m thick was thawed out beneath the pipe, before the thaw line entered a thaw stable gravel till at greater depth. Approximately 1 m of thaw settlement occurred during, and immediately after thawing of the icy silt layer. This corresponded well with the laboratory thaw settlement values of 40 - 56% measured for these icy layers beneath the pipe. Once the thaw line entered the dense granular till, essentially all pipe settlements ceased, except some small time-dependent settlements resulting from the final stages of consolidation of the surface silty layers. Ground surface settlements over the pipe were slightly greater than 1.0 m, indicating that most of the surface settlement resulted from thaw beneath the pipe.

Morgenstern and Nixon (1975) subsequently analysed these data, and showed that the results for excess pore pressures and degree of settlement during the thawing period were predictable using a relatively straight forward analysis for thaw consolidation. Consequently, both the total settlement, and the transient effects involving excess pore pressures are predictable at this site using currently available techniques.

7.2 Arctic Gas Norman Wells Facility

Hwang (1976) has presented a detailed thermal analysis and description of the thermal model being used for geothermal simulations on this project. The results for thawing and settlement beneath a 1.22 m diameter warm pipe test section maintained at 18° C are also reviewed. Approximately 2.3 m of thaw took place in the brown and grey silt layers beneath the pipe. The frozen density of these layers varied between 1747 and 1891 kg/m^3. Visual excess ice values of 5 to 15% were logged in the field, with an average value close to 8% within the zone of interest.

Referring to the correlations between thaw strain and frozen density reviewed in Section 3, much of the data for thaw strain at this range of densities would predict between 5 and 10% thaw strain, with an upper bound of 12% at a density of 1850 kg/m^3. These correlations, together with the visual excess ice contents provide realistic estimates of thaw settlement in the range of 5-10% beneath the pipe. Measured settlements of the pipe were 0.14 to 0.23 m during the year-long operation period, and this corresponds to a thaw strain range of 6-10%. The laboratory data and field data are therefore in reasonable agreement, both in terms of the magnitude and variability of thaw settlement.

7.3 Foothills Quill Creek

The above records have confirmed the methods of predicting thaw settlement for essentially un-insulated pipe sections. The procedure for calculating thaw settlement is essentially the same whether the pipe is insulated, deep-buried or placed in an embankment. The key components of the design involve a prediction of thaw depth, and a knowledge of the thaw strain of the soils. The preceeding sections have described the extensive data base for thaw strain of permafrost soils, and the confirmatory studies that have proven the accuracy of the thermal model.

The primary remaining component of the design and construction procedure that requires verification is the feasibility of the design modes from a construction standpoint. Foothills Pipe Lines (South Yukon) have completed an extensive study of construction feasibility for the different design modes at thèir Quill Creek test site. Concentrating on the deep burial and embankment modes for mitigation of thaw settlement, several test sections were constructed in January - March, 1981, that have demonstrated the construction feasibility of these design modes. In August 1981, seveal of these test sections will commence operation under warm pipeline condition. The quill Creek test site will be operated for several years. Observations will be carried out on both the dormant (not heated) and the warm pipe sections as well as on the right-of-way. Predictions of long-term settlement will be reliably based on the long-term geothermal predictions, coupled with terain and borehole data, and the known thaw settlement response of the range of soil types reviewed earlier.

Respectfully submitted,

HARDY ASSOCIATES (1978) LTD.

Per:

A. Hanna, M.A.Sc., P.Eng.

Per:

J.F. Nixon, Ph.D., P.Eng.
Project Manager

JFN:lt
43:136

REFERENCES

EBA Engineering Consultants Ltd., 1981. A Brief Description of EBA Geothermal Model. Report prepared for Foothills Pipe Lines (Yukon) Ltd.

FOOTHILLS PIPELINES (SOUTH YUKON) LTD., 1981. Quill Creek Test Test Facility - Design Modes, Construction Procedures and Anticipated Results.

HWANG, C.T., 1976. Predictions and observations on the behaviour of a warm gas pipeline on permafrost. Can. Geotech J.13, p.452.

LUSCHER, U. and AFIFI, S., 1973. Thaw consolidation tests on Alaskan silts and granular soils. Proc. 2nd Intl Permafrost Conf., p. 325, Yakutsk, U.S.S.R.

LYON, A.J., 1970. Dealing with Data. Pergamon Press Ltd.

MCROBERTS, E.C., LAW, T.C. and MONIZ, E., 1978. Thaw settlement studies in the discontinuous permafrost zone. Proc. 3rd Intl. Permafrost Conf., p. 700, Edmonton, Canada.

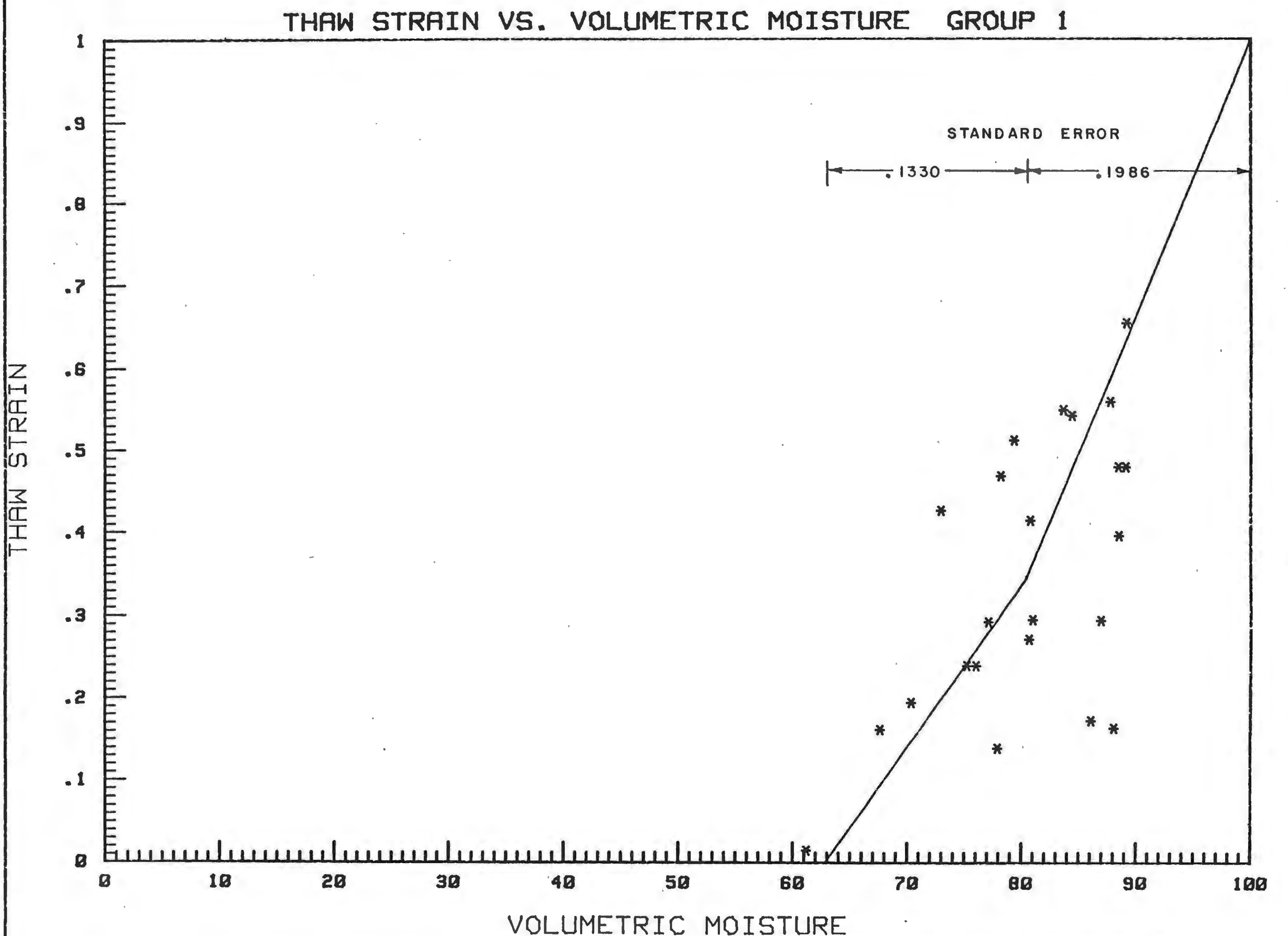
MORGENSTERN, N.R. and NIXON, J.F., 1971. One-dimensional consolidation of thawing soils. Can. Geot. J. 8, No. 4.

MORGENSTERN, N.R. and NIXON, J.F., 1975. An analysis of the performance of a warm-oil pipeline in permafrost, Inuvik, N.W.T. Can. Geot. J. 12, No. 2, p. 199.

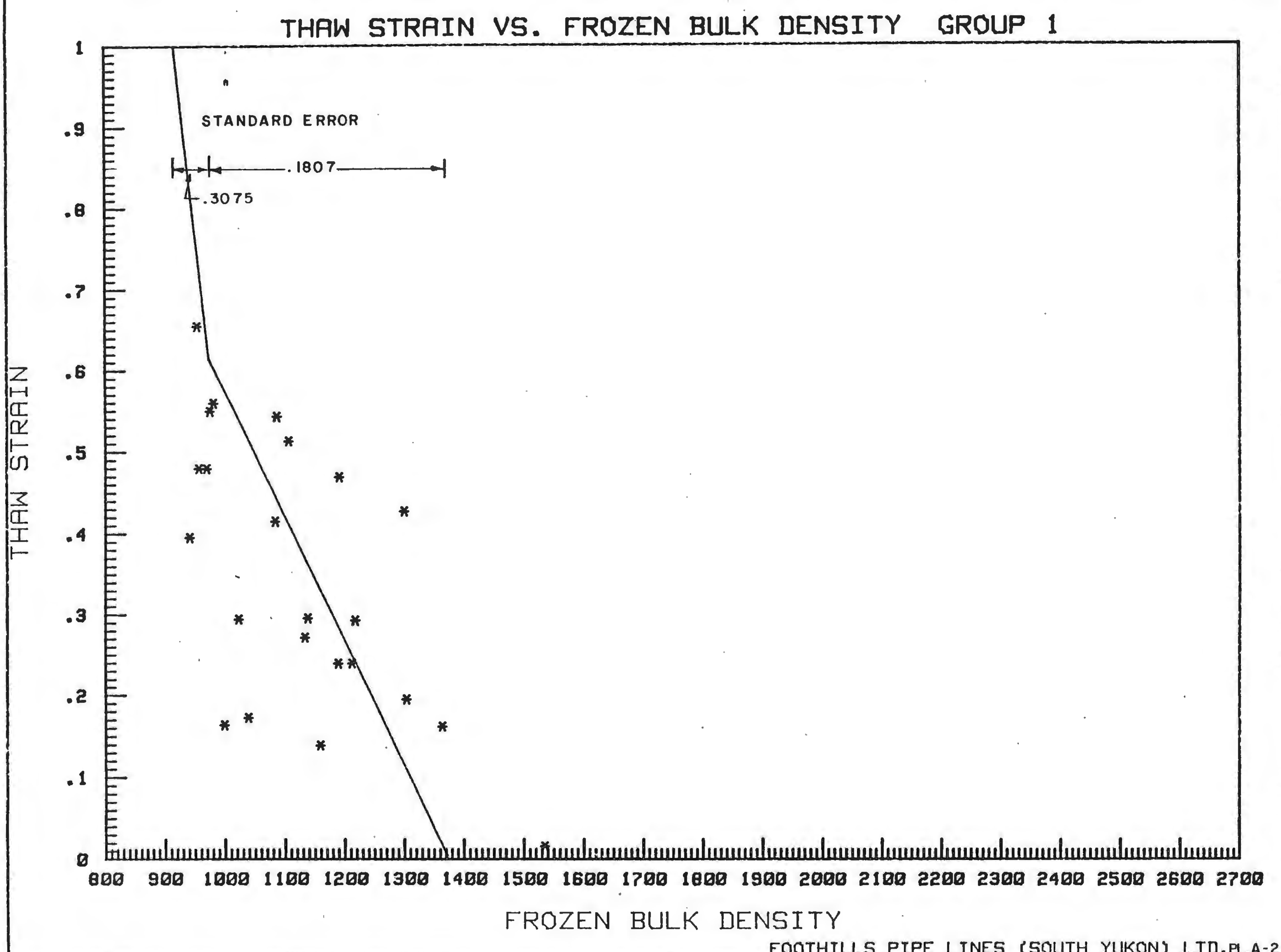
WATSON, G.H., ROWLEY, R.K. and SLUSARCHUK, W.A., 1973. Performance of a warm-oil pipeline buried in permafrost. Proc. 2nd Intl. Permafrost Conf., Yukutsk, U.S.S.R. p. 759.

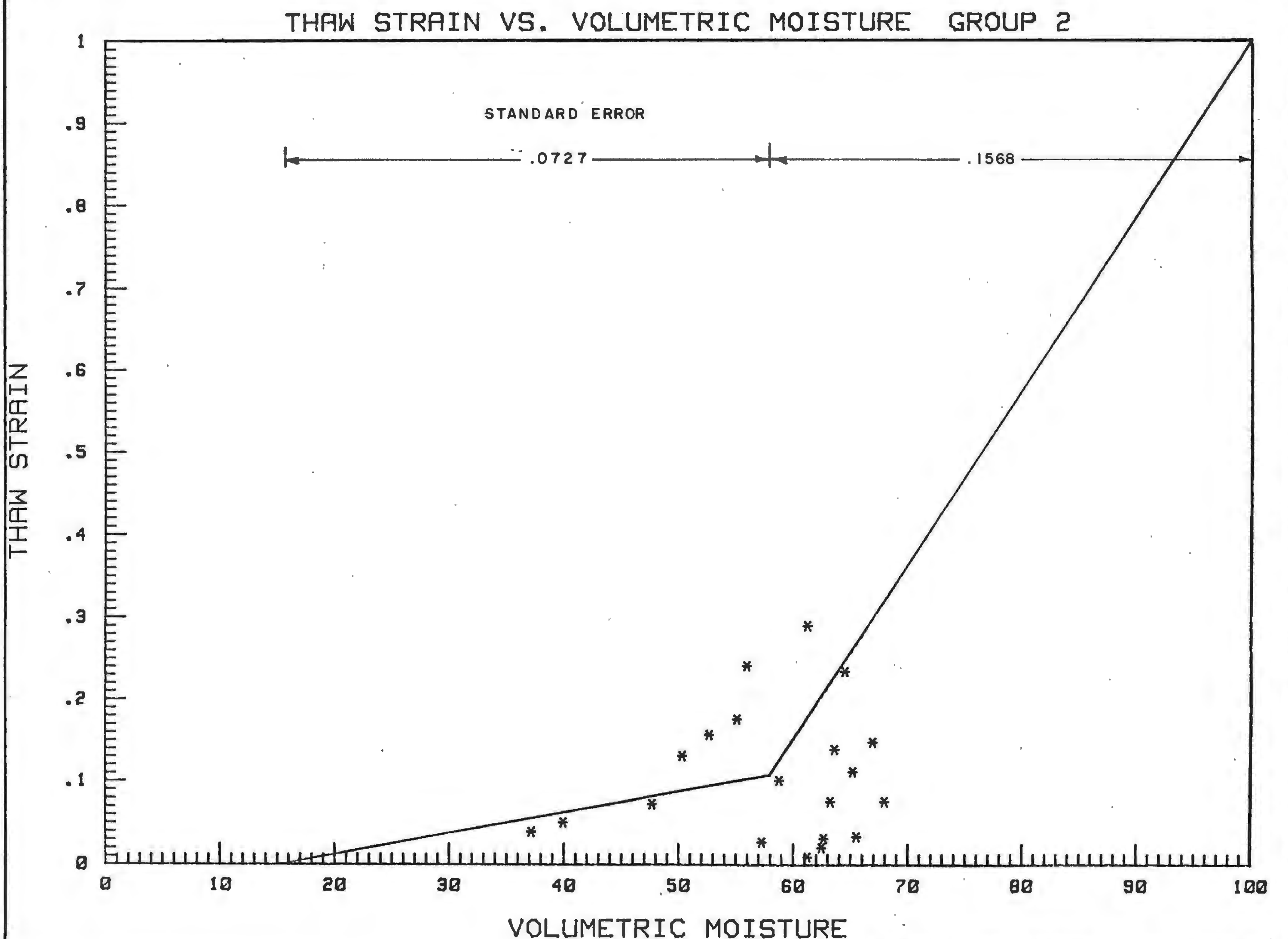
APPENDIX "A"

THAW SETTLEMENT CORRELATIONS



FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL.A-1


THAW STRAIN VS. FROZEN BULK DENSITY GROUP 1
STANDARD ERROR
.1807
.3075
THAW STRAIN
FROZEN BULK DENSITY
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
2200
2300
2400
2500
2600
2700
FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL.A-2



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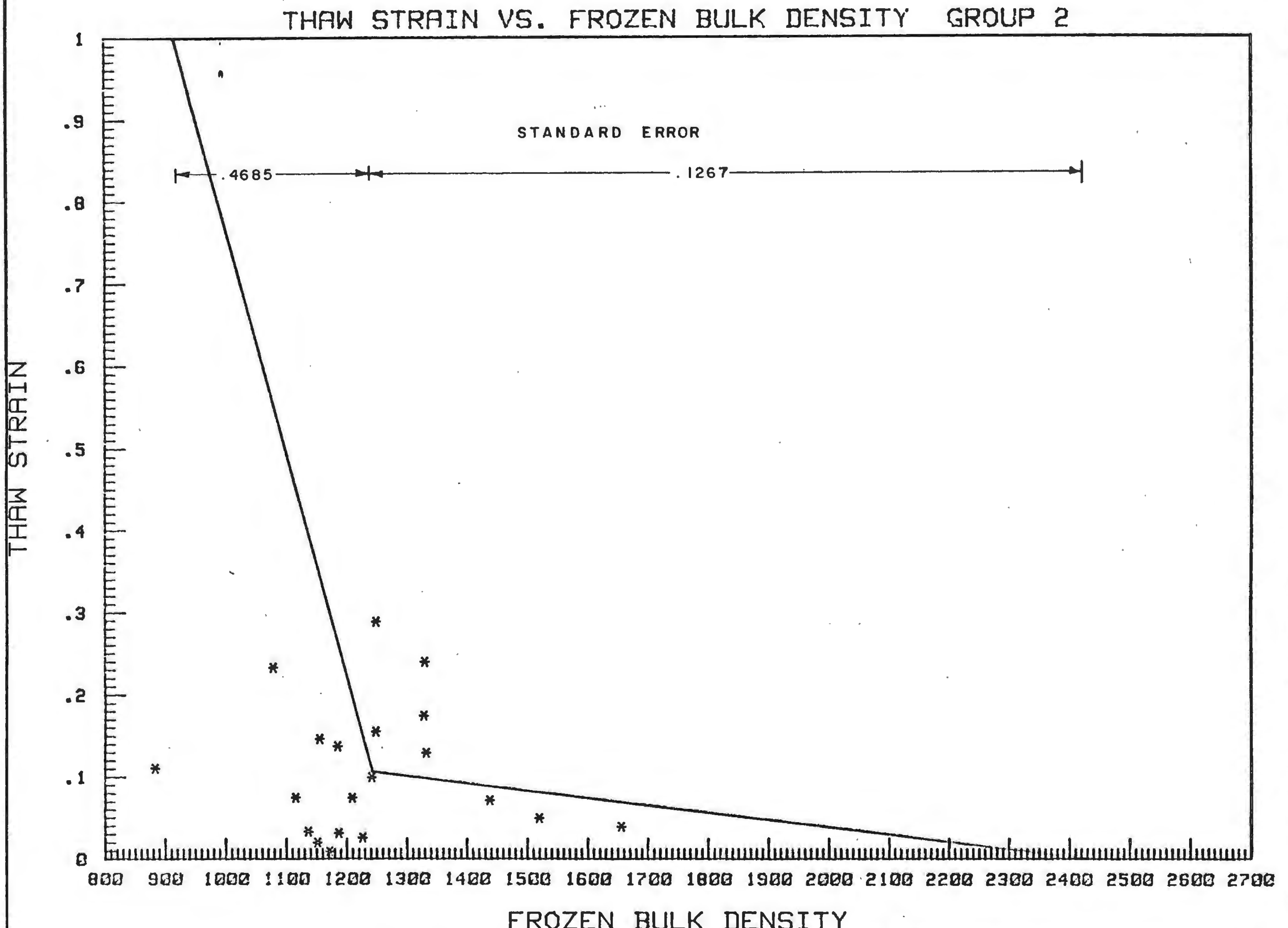

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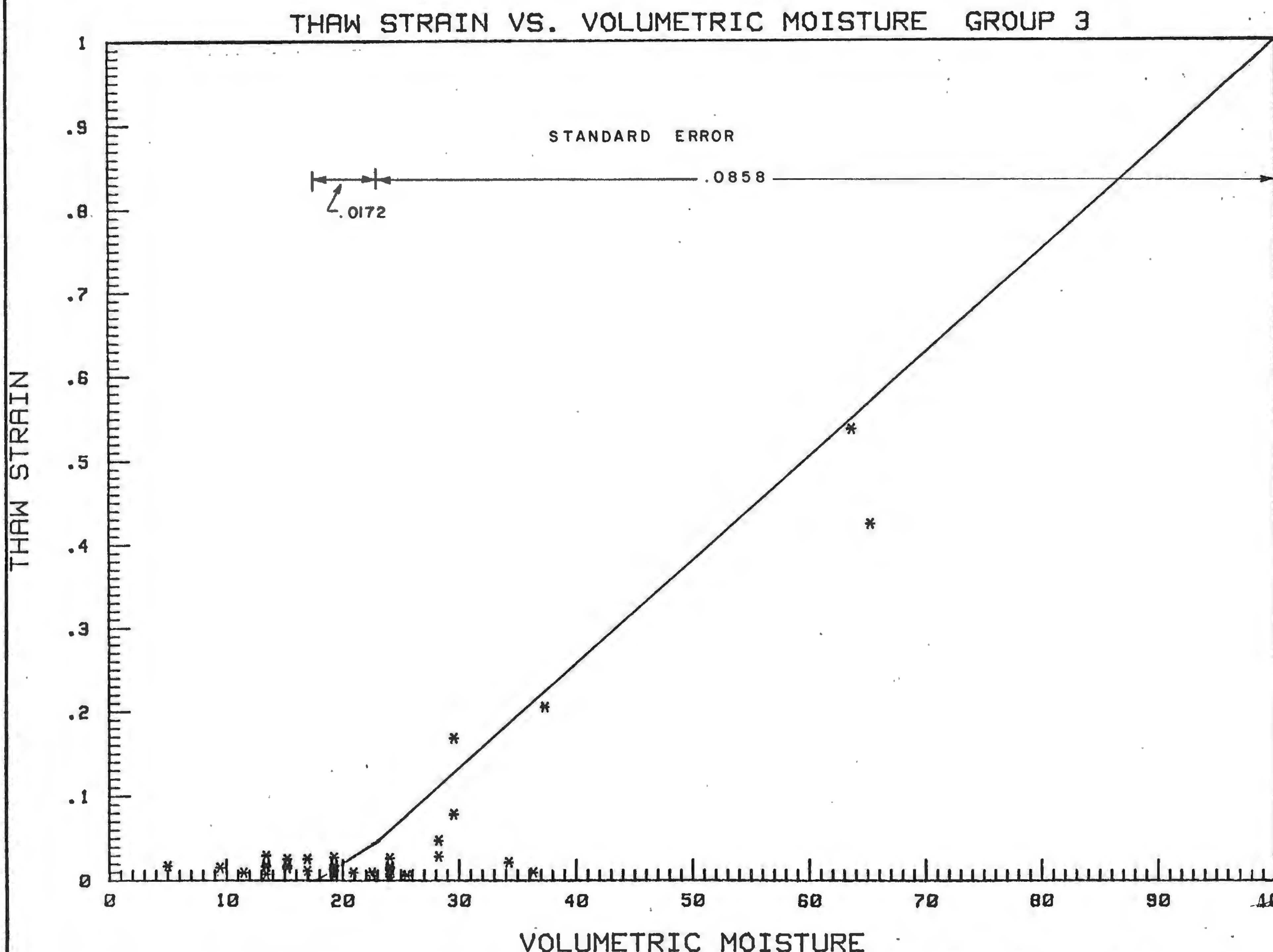
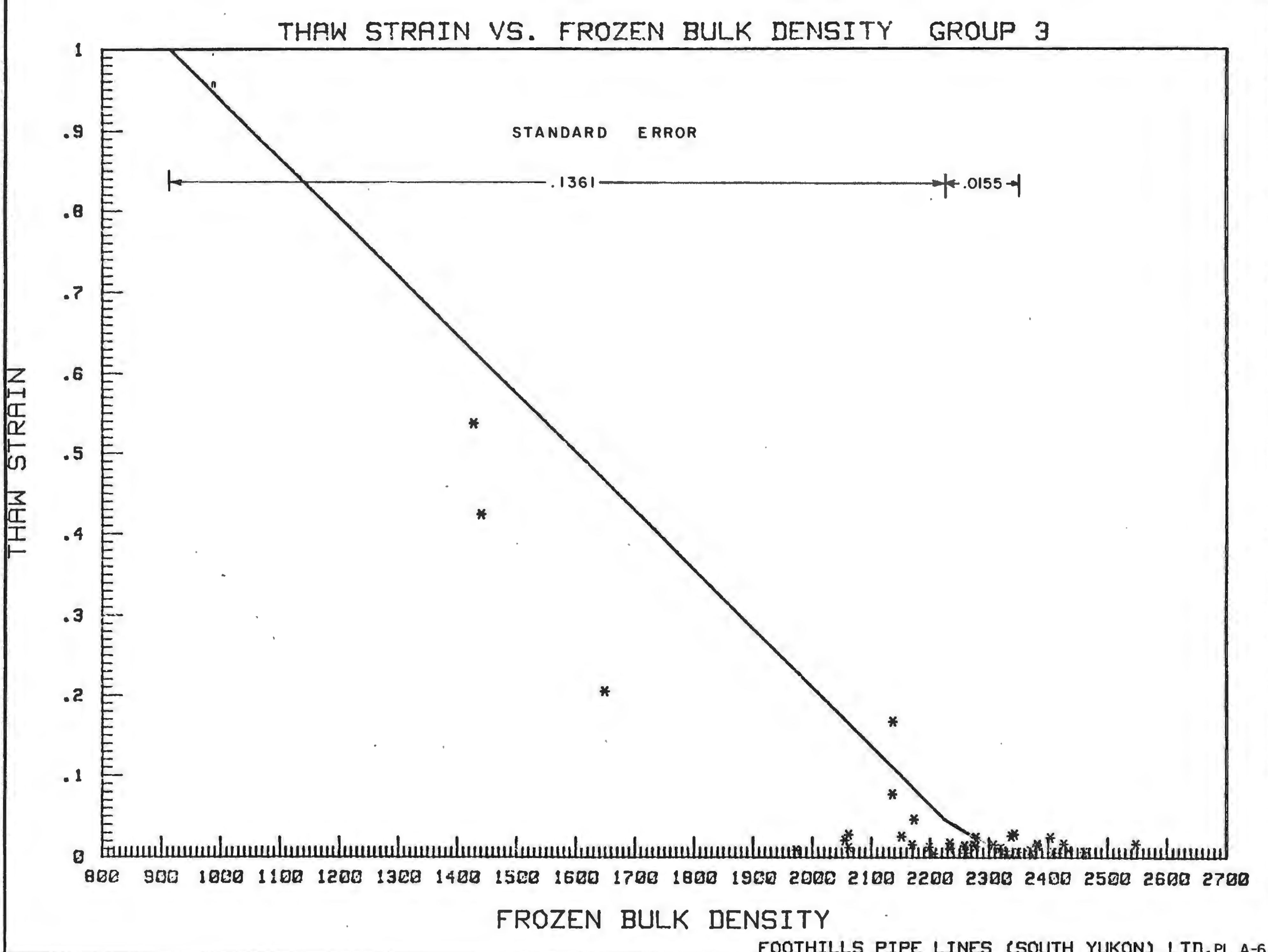
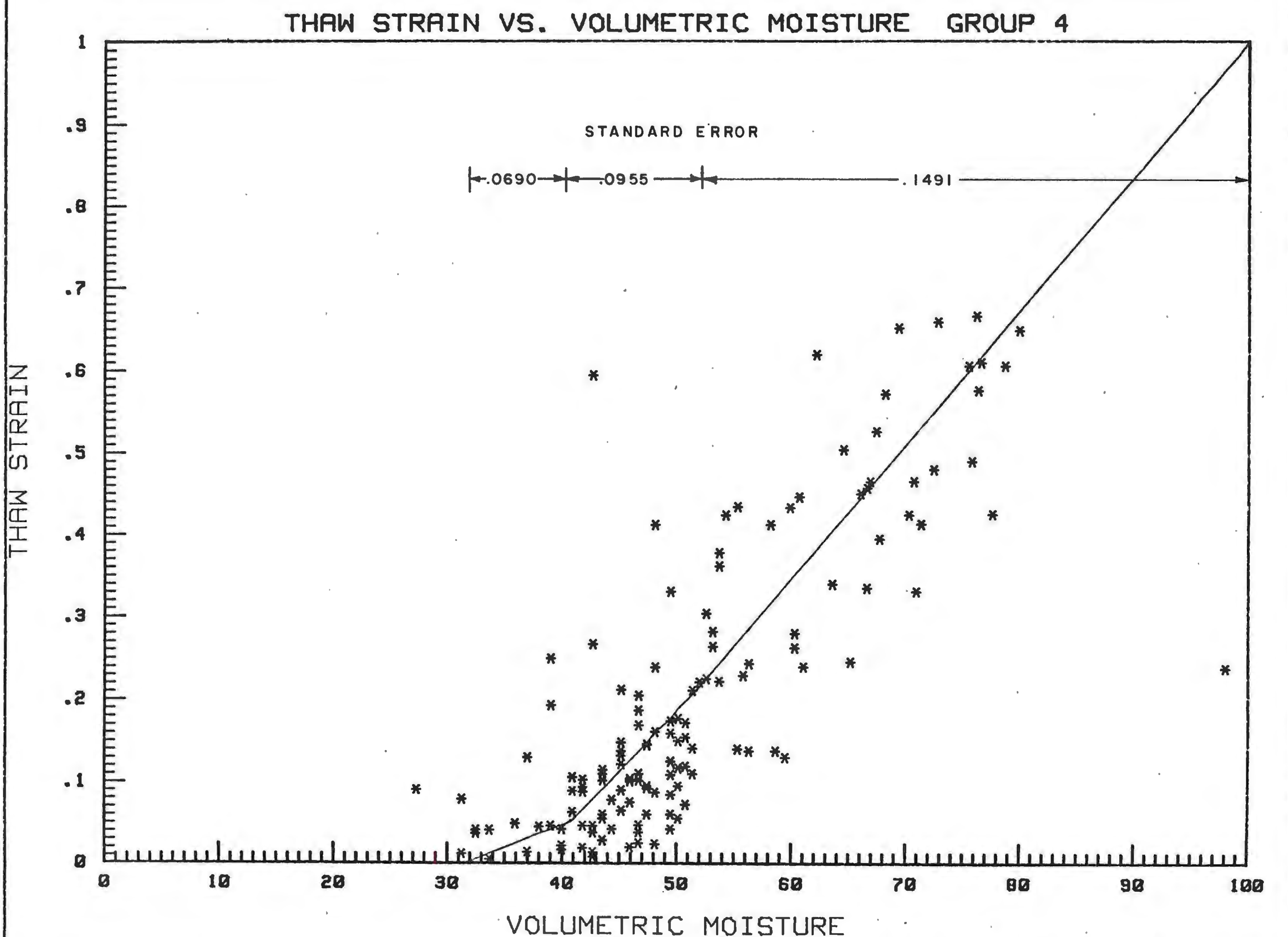


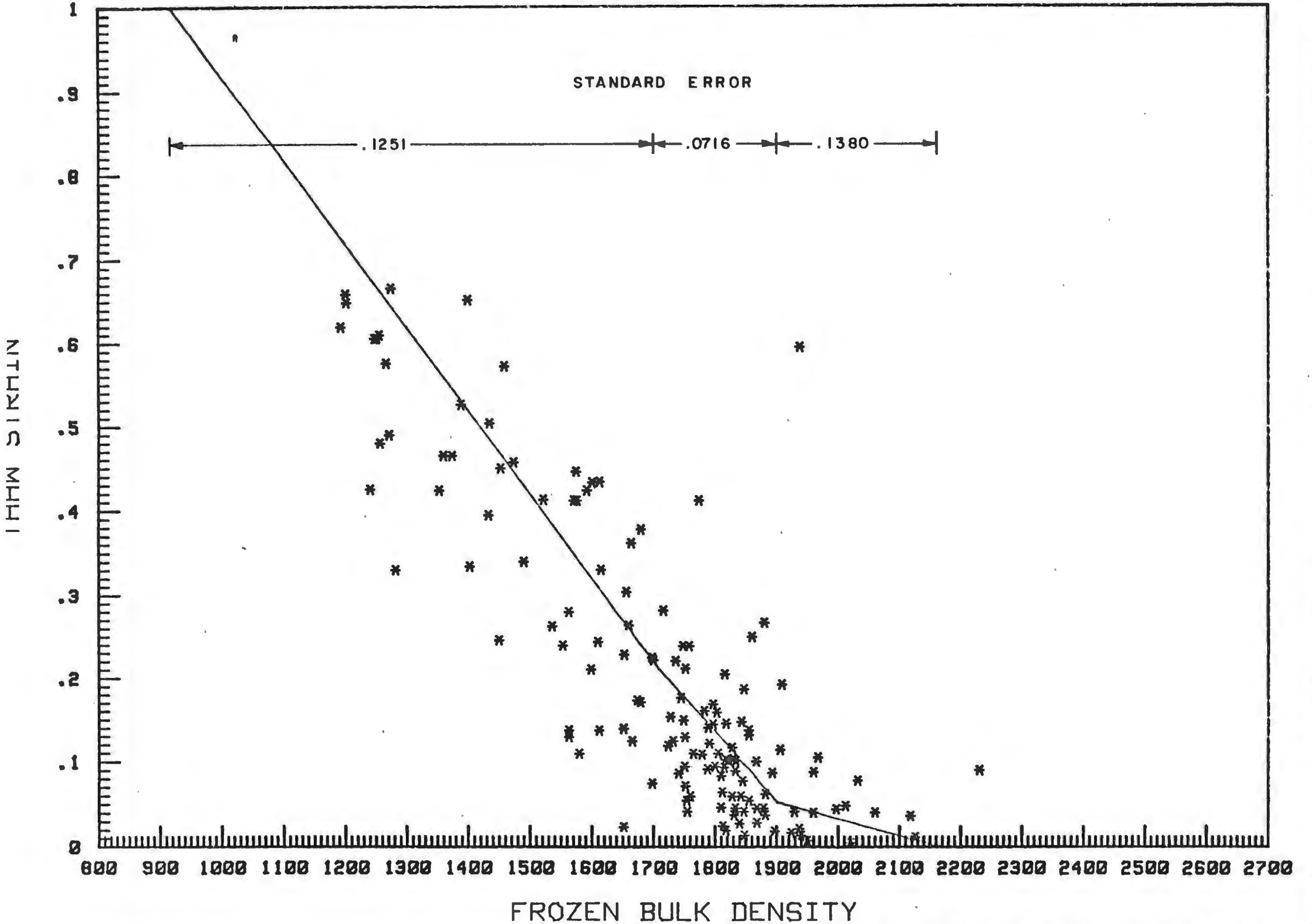
FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL. A-5


THAW STRAIN VS. FROZEN BULK DENSITY GROUP 3
STANDARD ERROR
.1361
.0155
THAW STRAIN
1
.9
.8
.7
.6
.5
.4
.3
.2
.1
0
800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 2700
FROZEN BULK DENSITY
FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL. A-6

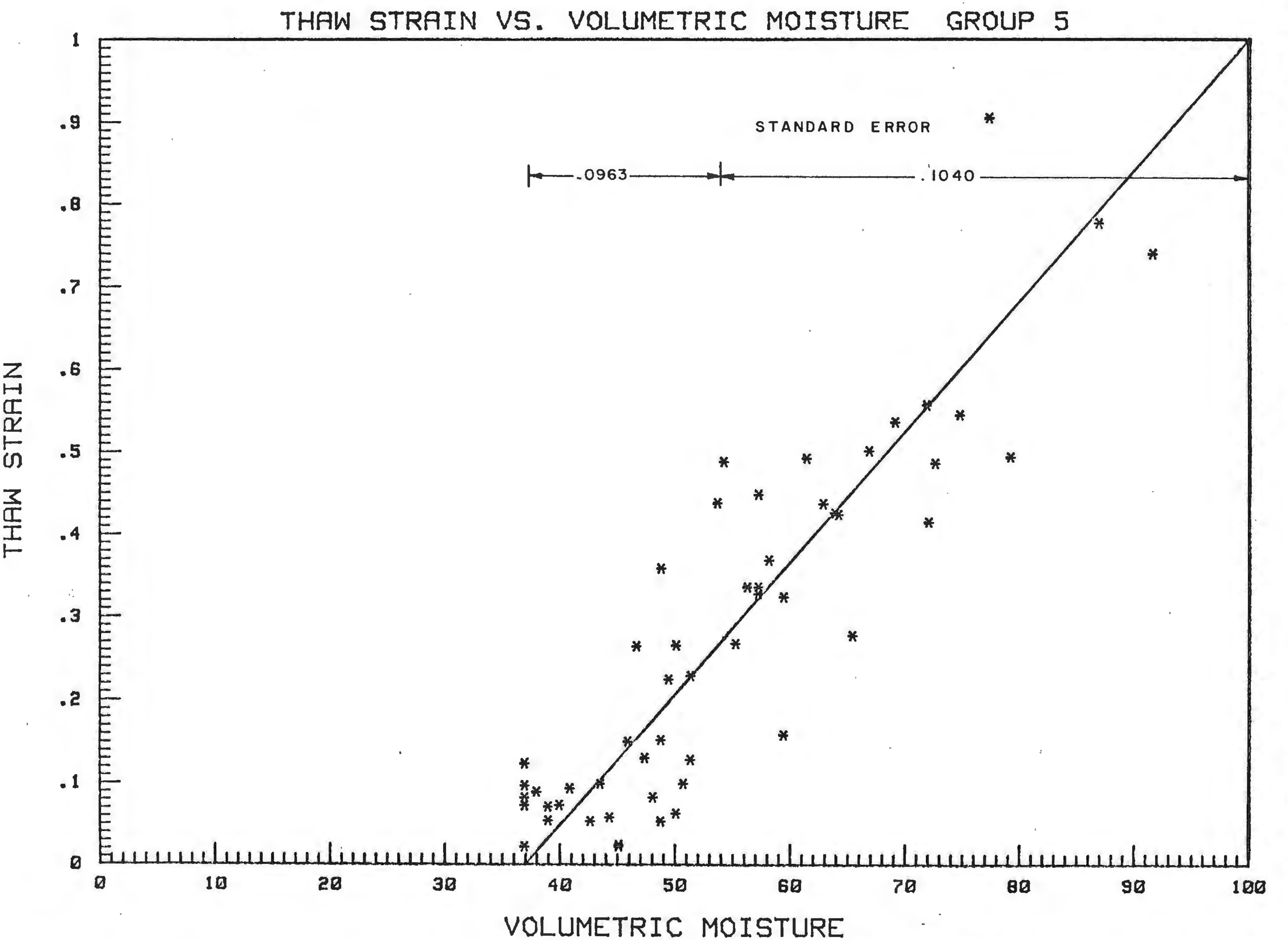


FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL. A-7

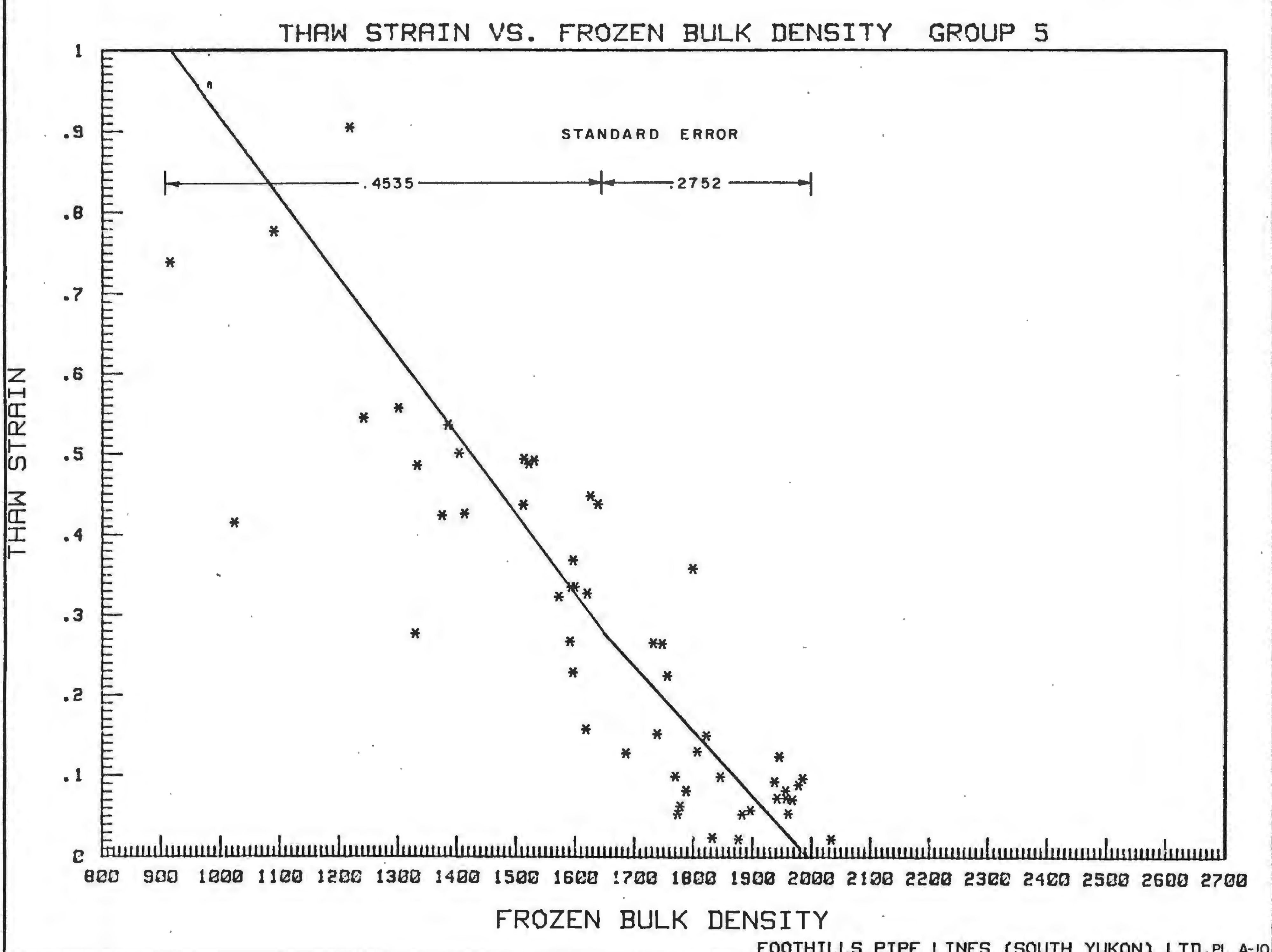
THAW STRAIN VS. FROZEN BULK DENSITY GROUP 4

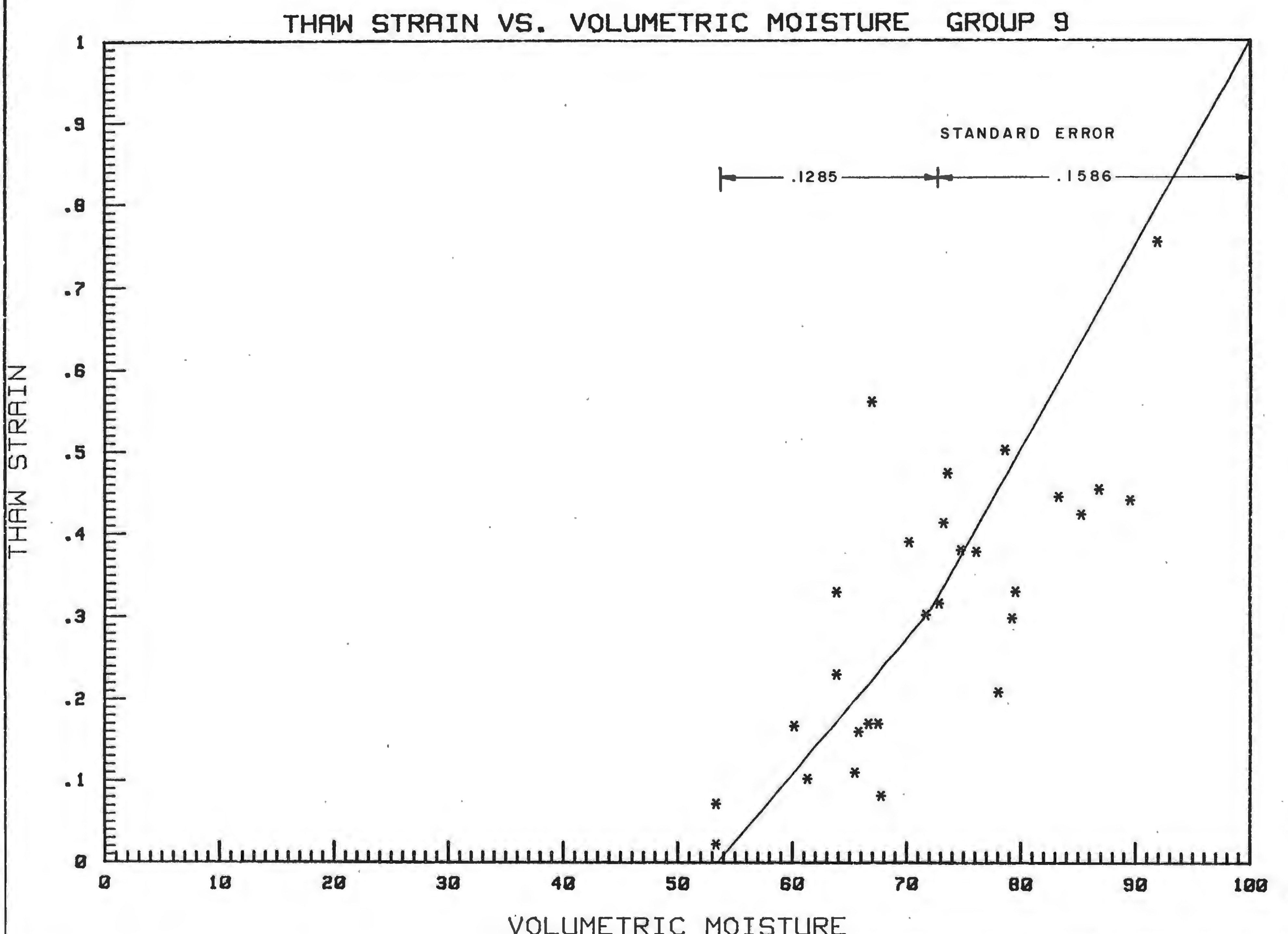


FOOTHILLS PIPE LINES (SOUTH YUKON) LTD.PL.A-8

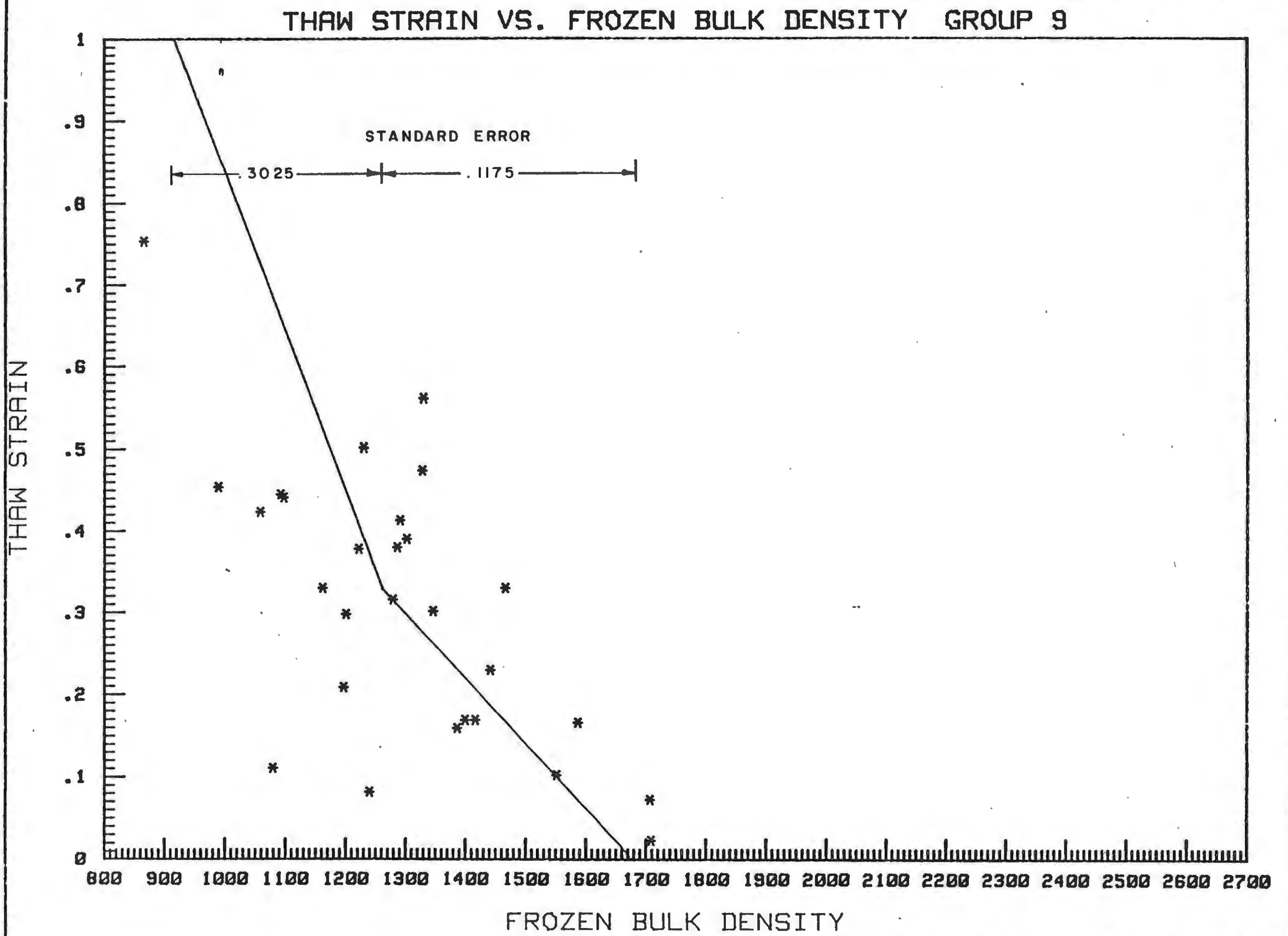


FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL. A-9


THAW STRAIN VS. FROZEN BULK DENSITY GROUP 5
STANDARD ERROR
.4535
.2752
THAW STRAIN
1
.9
.8
.7
.6
.5
.4
.3
.2
.1
0
800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 2700
FROZEN BULK DENSITY
FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL. A-10



FOOTHILLS PIPE LINES (SOUTH YUKON) LTD. PL. A-11



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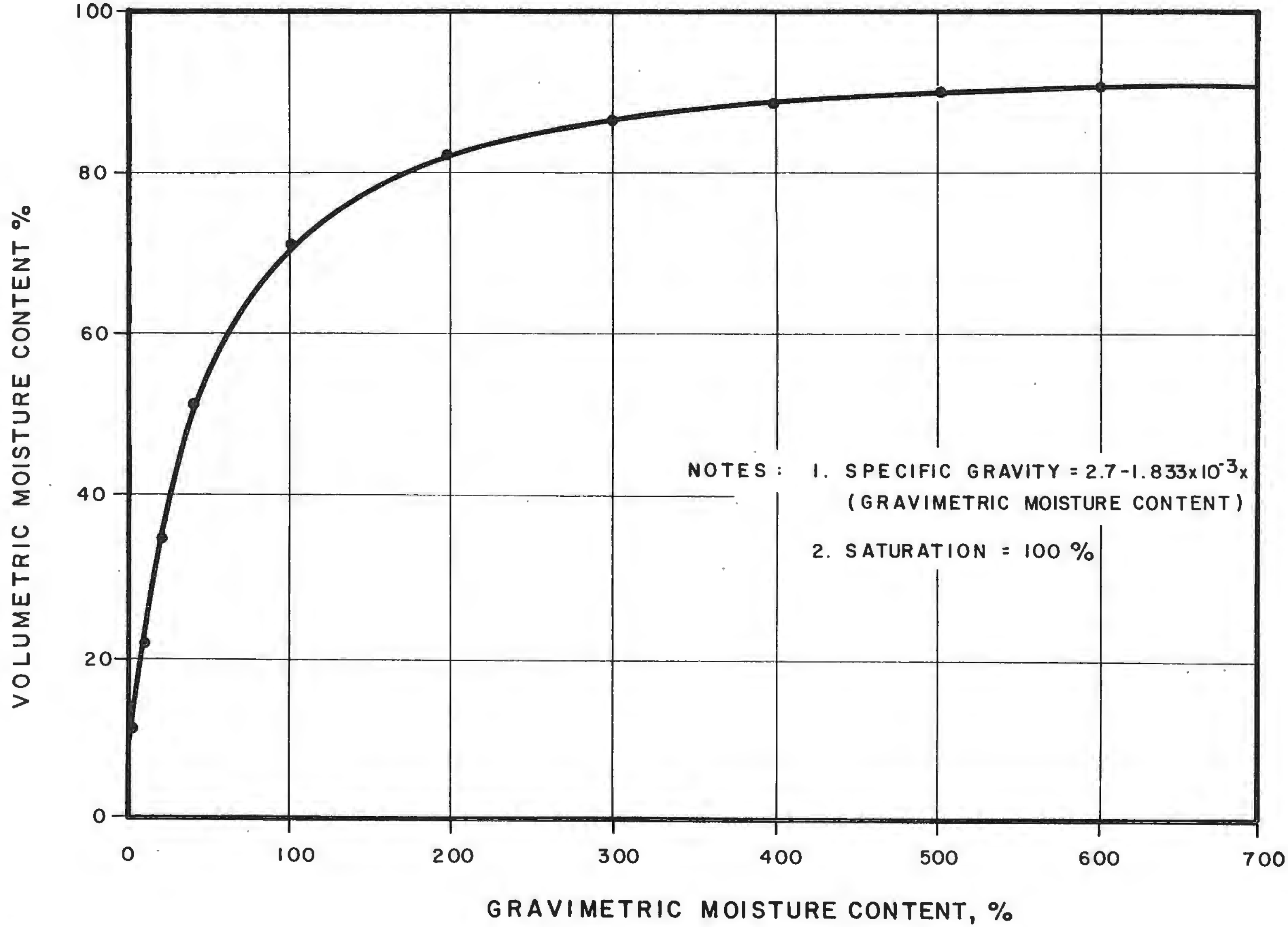

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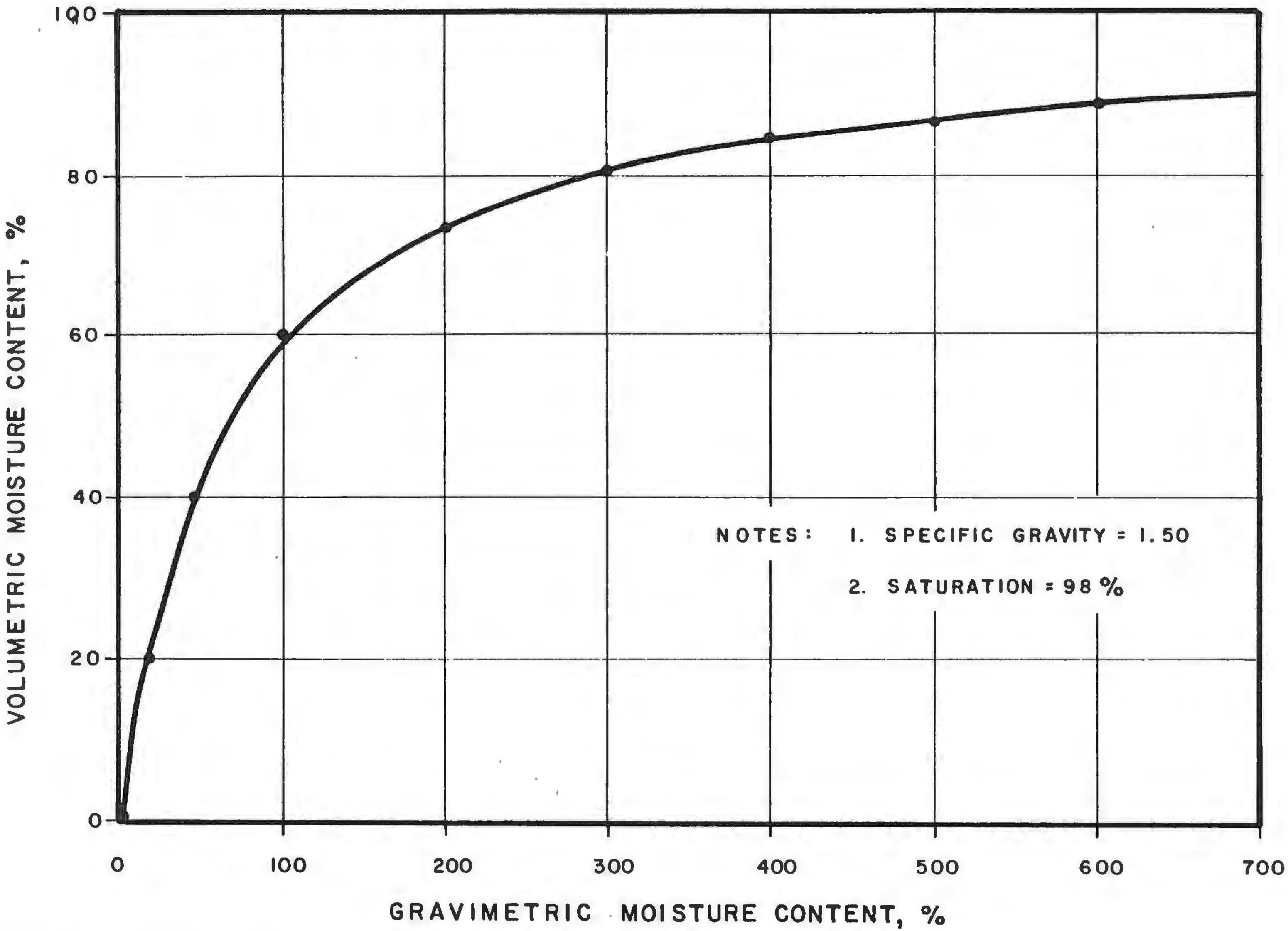
VOLUMETRIC Vs. GRAVIMETRIC
MOISTURE CONTENT
GROUP I - Pt

CG05500K

PLATE A-13

HT10 - 79/05



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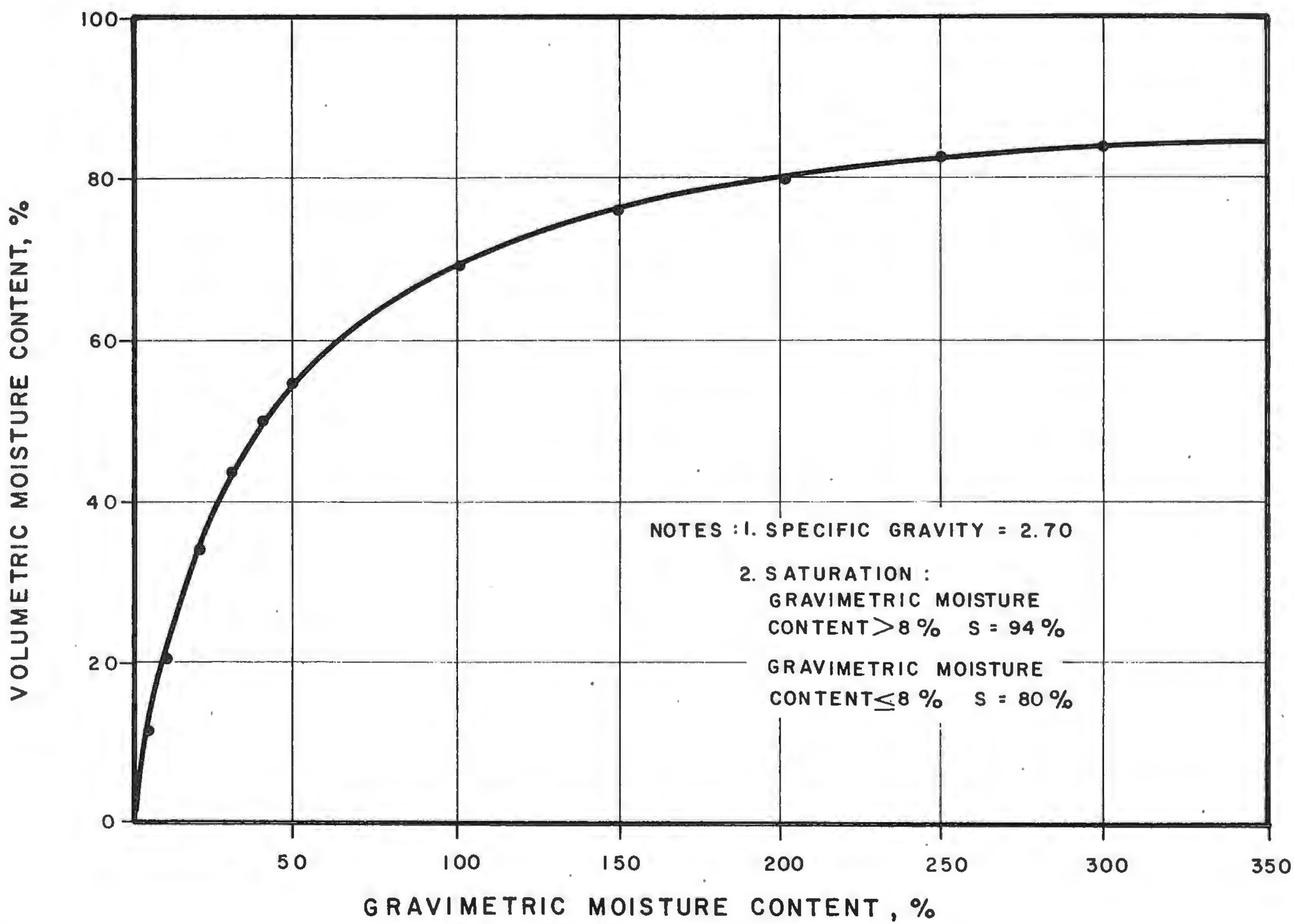
VOLUMETRIC Vs. GRAVIMETRIC
MOISTURE CONTENT
GROUP 2 - VA

CGO 5500 K

PLATE A-14

HT10 - 79/05



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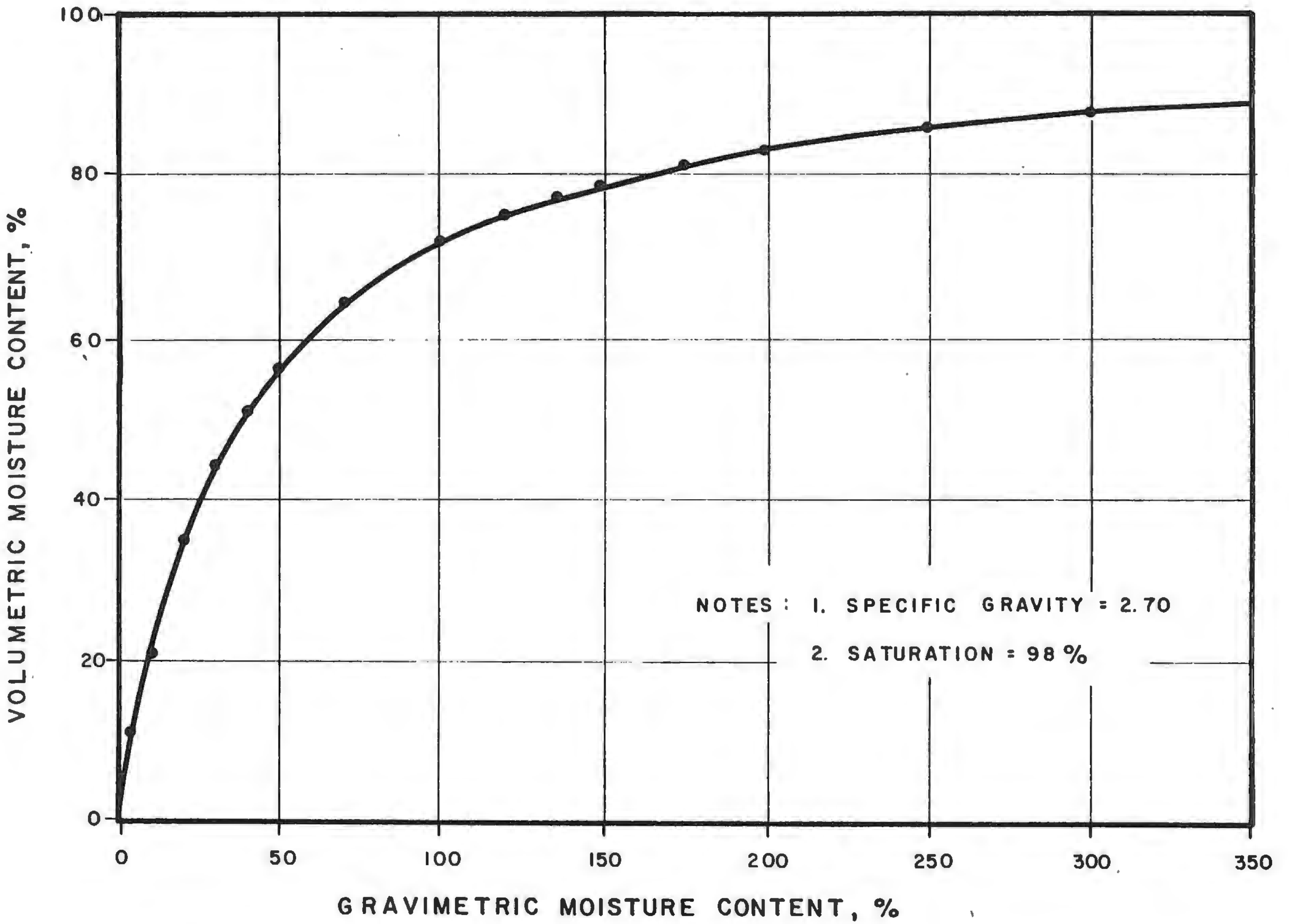
VOLUMETRIC Vs. GRAVIMETRIC MOISTURE CONTENT
GROUP 3 - GP, GW, SW, GM, GW - GM.

C G0 5500 K

PLATE A-15

HT10 - 79/05



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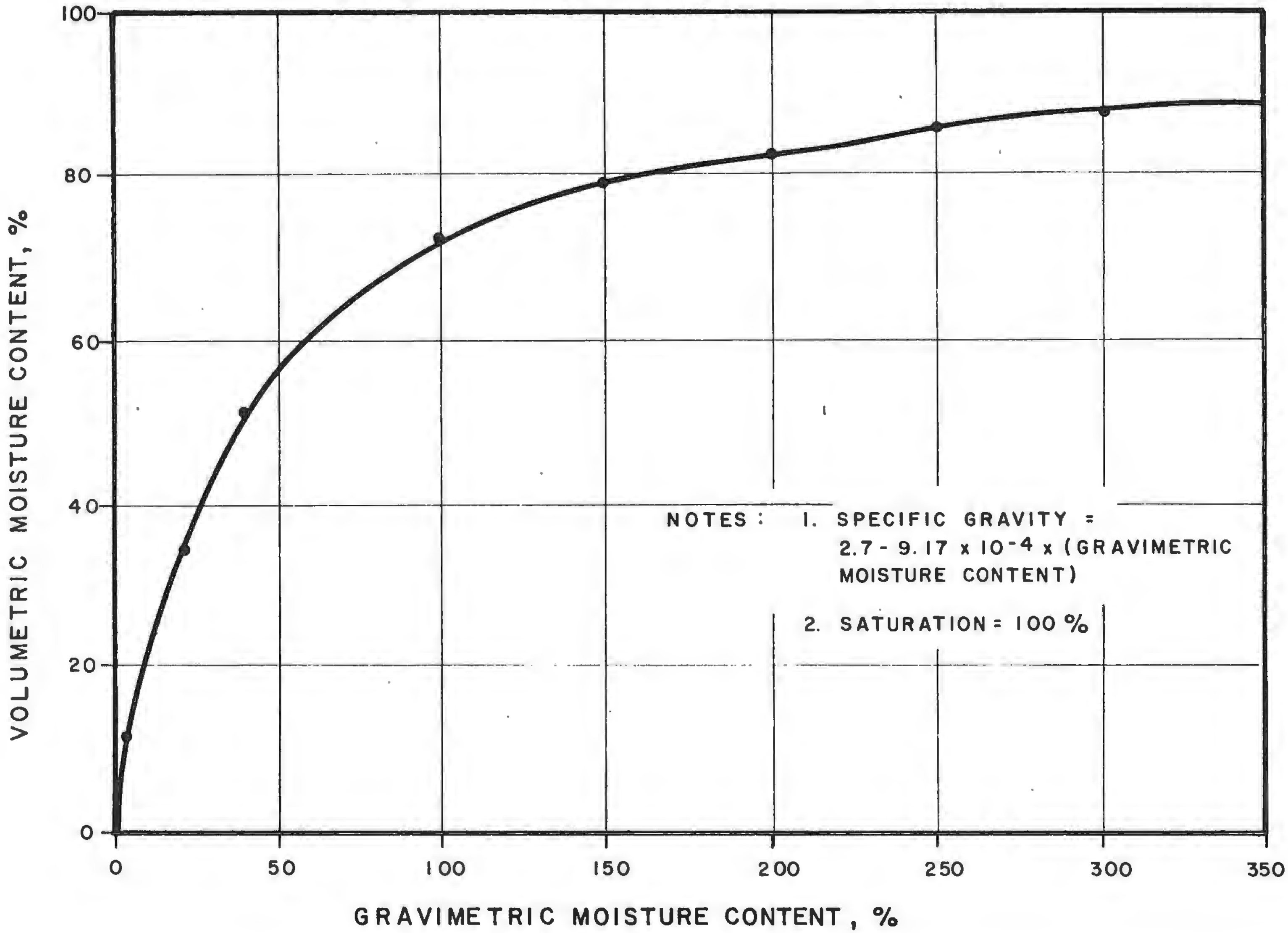
VOLUMETRIC Vs. GRAVIMETRIC MOISTURE CONTENT

GROUP 4 — SP, SC, SM, CL, ML
GROUP 5 — CI, CH, MH

CG05500K

PLATE A-16

HT10 - 79/05



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VOLUMETRIC Vs. GRAVIMETRIC
MOISTURE CONTENT
GROUP 9 - OH, OL

CG05500K

PLATE A-17

HT10 - 79/05

APPENDIX "B"

VERIFICATION OF THAW SETTLEMENT CALCULATIONS

TABLE B-1

BOREHOLE	THAW STRAIN CORRELATION USED*	SOIL GROUP**	REMARKS ***
80-11-001(4)	2, 6, 7	1, 3, 6, 9	1, 3
78-A-30	2, 3, 6	2, 4, 7, 9	3, 6
80-11-068	2	3, 4	3, 7
80-11-084	2, 6	1, 2, 4	3, 6
80-01-45	2, 6	2, 3,	4
80-11-207	2, 6	4, 5	5
79-B-17	6	2, 3, 4	2
80-11-075	2, 4, 7	2, 4, 5, 6, 9	-
81-05-14	1, 2, 4, 5	1, 3	6

*	**	***
	1 Pt	1 shallow borehole
1 Correlation with Density	2 VA	2 unfrozen borehole
2 Correlation with Moisture Content	3 GM, GP, GW, GC, SW	3 U/F borehole
3 Correlation with V. Ice Content	4 ML, CL, SM, SP, SC	4 F/U borehole
4 Zero Thaw settlement assumed in top 2 m if no data is available	5 MH, CI, CH	5 U/F/U borehole
5 Moisture Content Data interpolated between layer	6 BR	6 extrapolation to X2 required
6 Unfrozen Soil	7 IP	7 cut-off at X2
7 Bedrock	9 OL, OH	

TABLE B2

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
80-11-001(4)	S	rfCB/R	1	3.48	0

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
2.0	0.6	100	1	5	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	0.1	PT	-	-1	0	0	UF, ε = 0.0	0.0	0.0
2	0.6	OL	-	-1	0	0	UF, ε = 0.0	0.0	0.0
3	1.18	OL	43.7	-1	0	-1	ε = 0.0	0.0	0.0
4	1.90	GC	7.4	-1	0	-1	ε = 0.0	0.0	0.0
5	2.0	BR	-	-1	-1	-1	BR, ε = 0.0	0.0	0.0

Σ = 0.0

COMPUTER CALCULATION

Borehole = 80-11-001(4)	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
layer is unfrozen	0.00 - .10m	PT	-1	0.0	0.000	0.000
layer is unfrozen	.10 - .60m	OL	-1	0.0	0.000	0.000
water content	.60 - 1.18m	OL	29	43.7	0.000	0.000
water content	1.18 - 1.90m	GC	3	7.4	0.000	0.000
bedrock	1.90 - 10.00m	BR	-1	0.0	0.000	0.000

80-11-001(4) Thaw settlement = 0.000 Sigma = 0.000

TABLE B3

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
78-A-30	N	fvAP	14	94.47	35S

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
7.75	0.2	100	0	8	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	0.2	OL	-	-1	0	0	UF, $\varepsilon = 0.0$	0.0	0.0
2	0.5	IP	-	-1	85	-1	ε = ice/100	0.850	0.255
3	1.9	VA	67.0	1157	20	14.7	$\varepsilon = -1.1252 + 0.02125 \times$ vol	0.299	0.419
4	3.6	VA	65.7	1164	30	7.45	$\varepsilon = -1.1252 + 0.02125 \times$ vol	0.271	0.461
5	4.6	OL	64.9	1455	12	-1	$\varepsilon = -0.8803 + 0.0165 \times$ vol	0.191	0.191
6	5.1	OL	53.4	1709	10	2.25	$\varepsilon = 0.0$	0.0	0.0
7	6.65	SM	58.6	1563	5	13.75	$\varepsilon = -0.6312 + 0.0163 \times$ vol	0.324	0.502
8	7.75	SP	30	1950	3	-1	$\varepsilon = 0.0$	0.0	0.0

Σ = 1.828

COMPUTER CALCULATION

Borehole = 78-A-30	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
layer is unfrozen	0.00 - .20m	OL	-1	0.0	0.000	0.000
visible ice	.20 - .50m	IP	323	0.0	.850	0.000
water content	.50 - 1.90m	VA	141	67.0	.298	.157
water content	1.90 - 3.60m	VA	133	65.7	.271	.157
water content	3.60 - 4.60m	OL	70	64.9	.190	.129
water content	4.60 - 5.10m	OL	43	53.4	0.000	0.000
water content	5.10 - 6.65m	SM	54	58.6	.325	.149
water content	6.65 - 10.00m	SP	16	30.0	0.000	0.000

78-A-30 Thaw settlement = 1.826 Sigma = .435

TABLE B4

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
80-11-068	N	fvAT	14	94.1	1S

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
10.5	0.4	100	0	8	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	4.7	ML	78.7	1192	40	46.8	ε =-0.6312 + 0.0163 x vol	0.652	2.804
2	5.0	CL	61.1	-1	35	-1	ε =-0.6312 + 0.0163 x vol	0.365	0.110
3	6.5	ML	50.1	1745	10	18.4	ε =-0.5426 + 0.0146 x vol	0.189	0.284
4	6.9	ML	64.5	1485	5	-1	ε =-0.6312 + 0.0163 x vol	0.420	0.168
5	7.3	CL	42.7	-1	0	-1	ε =-0.5426 + 0.0146 x vol	0.081	0.032
6	8.9	SM	46.7	1831	0	2.9	ε =-0.5426 + 0.0146 x vol	0.139	0.222
7	9.5	GW	32.0	-1	10	-1	ε =-0.2380 + 0.0124 x vol	0.159	0.095
8	10.5	SM	39.0	-1	0	-1	ε =-0.1729 + 0.00552 x vol	0.042	0.021

Σ = 3.736

COMPUTER CALCULATION

Borehole = 80-11-068	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
water content	0.00 - 4.70m	ML	148	78.7	.653	.149
water content	4.70 - 5.00m	CL	60	61.1	.365	.149
water content	5.00 - 6.50m	ML	38	50.1	.189	.096
water content	6.50 - 6.90m	ML	70	64.5	.422	.149
water content	6.90 - 7.30m	CL	28	42.7	.080	.096
water content	7.30 - 8.90m	SM	33	46.7	.138	.076
water content	8.90 - 9.50m	GW	18	32.0	.159	.086
water content	9.50 - 10.00m	SM	24	39.0	.043	.056

80-11-068 Thaw settlement = 3.737 Sigma = .682

TABLE B5

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
80-11-084	N	fvAAP	17	116.23	OS

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
9.9	1.0	100	0	8	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	1.0	PT	-	-1	0	0	UF, ε = 0.0	0.0	0.0
2	1.9	VA	39.4	-1	10	-1	ε =-0.0383 + 0.0025 x vol	0.060	0.054
3	3.0	PT	76.2	1250	10	-1	ε =-1.225 + 0.0195 x vol	0.261	0.287
4	4.0	PT	73.5	1295	18	-1	ε =-1.225 + 0.0195 x vol	0.208	0.208
5	6.8	PT	75.4	1130	25	-1	ε =-1.225 + 0.0195 x vol	0.245	0.686
6	7.8	PT	77.8	1210	23	-1	ε =-1.225 + 0.0195 x vol	0.292	0.292
7	8.0	ML	50.8	-1	5	-1	ε =-0.5426 + 0.0146 x vol	0.199	0.040
8	9.9	CL	66	-1	5	-1	ε =-0.6312 + 0.0163 x vol	0.445	0.89

Σ = 2.457

COMPUTER CALCULATION

Borehole = 80-11-084	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
layer is unfrozen	0.00 - 1.00m	PT	97	0.0	0.000	0.000
water content	1.00 - 1.90m	VA	44	39.4	.061	.073
water content	1.90 - 3.00m	PT	130	76.2	.263	.133
water content	3.00 - 4.00m	PT	111	73.5	.210	.133
water content	4.00 - 6.80m	PT	124	75.4	.247	.133
water content	6.80 - 7.80m	PT	144	77.8	.295	.133
water content	7.80 - 8.00m	ML	39	50.8	.198	.096
water content	8.00 - 10.00m	CL	75	66.0	.446	.149

80-11-084 Thaw settlement = 2.473 Sigma = .538

TABLE B6

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
80-01-45	N	fvEBV/tMM	20	135.54	26S

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
8.3	100	3.6	1	7	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	0.75	VA	39.4	1500	1	-1	ε =-0.03831 + 0.0025 x vol	0.060	0.045
2	1.8	GM	17	-1	1	-1	ε = 0.0	0.0	0.0
3	3.0	GM	15.3	-1	0	-1	ε = 0.0	0.0	0.0
4	3.6	GM	28.3	-1	3	-1	ε =-0.238 + 0.0124 x vol	0.113	0.068
5	6.35	GM	-	-1	0	0	UF, ε = 0.0	0.0	0.0
6	7.65	GM	-	-1	0	0	UF, ε = 0.0	0.0	0.0
7	8.30	GM	-	-1	0	0	UF, ε = 0.0	0.0	0.0

Σ = 0.113

COMPUTER CALCULATION

Borehole = 80-01-45	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
water content	0.00 - .75m	VA	44	39.4	.061	.073
water content	.75 - 1.80m	GM	8	17.0	0.000	0.000
water content	1.80 - 3.00m	GM	7	15.3	0.000	0.000
water content	3.00 - 3.60m	GM	15	28.3	.112	.086
layer is unfrozen	3.60 - 6.35m	GM	6	0.0	0.000	0.000
layer is unfrozen	6.35 - 7.65m	GM	11	0.0	0.000	0.000
layer is unfrozen	7.65 - 8.30m	GM	1	0.0	0.000	0.000

80-01-45 Thaw settlement = .113 Sigma = .075

TABLE B7

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
80-11-207	S	fLBV/tMM	37	257.63	3N

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
10.35	2.15	5.4	1	8	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	2.15	CL	24.3	-1	0	0	UF, $\varepsilon = 0.0$	0.0	0.0
2	2.9	CL	24.3	-1	3	-1	$\varepsilon = 0.0$	0.0	0.0
3	3.3	CL	33.7	2028	50	-1	$\varepsilon = -0.1729 + .0055 \times vol$	0.012	0.005
4	4.0	CL	33.7	2060	12	-1	$\varepsilon = -0.1729 + .0055 \times vol$	0.012	0.008
5	5.4	CL	27.3	2152	2	-1	$\varepsilon = 0.0$	0.0	0.0
6	8.4	CL	-	-1	0	0	UF, $\varepsilon = 0.0$	0.0	0.0
7	9.9	SM	-	-1	0	0	UF, $\varepsilon = 0.0$	0.0	0.0
8	10.35	CI	-	-1	0	0	UF $\varepsilon = 0.0$	0.0	0.0

Σ = 0.013

COMPUTER CALCULATION

Borehole = 80-11-207	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
layer is unfrozen	0.00 - 2.15m	CL	12	0.0	0.000	0.000
water content	2.15 - 2.90m	CL	12	24.3	0.000	0.000
water content	2.90 - 3.30m	CL	19	33.7	.013	.017
water content	3.30 - 4.00m	CL	19	33.7	.013	.017
water content	4.00 - 5.40m	CL	14	27.3	0.000	0.000
unfrozen U/F/U	5.40 - 8.40m	CL	13	0.0	0.000	0.000
unfrozen U/F/U	8.40 - 9.90m	SM	16	0.0	0.000	0.000
unfrozen U/F/U	9.90 - 10.35m	CI	14	0.0	0.000	0.000

80-11-207 Thaw settlement = .014 Sigma = .014

TABLE B8

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
79-B-17	N	fvAAP	17	119.03	15N

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
8.1	100	0.0	0	6	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	1.1	VA	-	1380	0	0	UF, ε = 0.0	0.0	0.0
2	2.6	SW	-	-1	0	0	UF, ε = 0.0	0.0	0.0
3	5.1	VA	-	-1	0	0	UF, ε = 0.0	0.0	0.0
4	6.1	GM	-	-1	0	0	UF, ε = 0.0	0.0	0.0
5	7.1	ML	-	-1	0	0	UF, ε = 0.0	0.0	0.0
6	8.1	ML	-	-1	0	0	UF, ε = 0.0	0.0	0.0

Σ = 0.0

COMPUTER CALCULATION

Borehole = 79-B-17 DEPTH RANGE USC GRAV VOL STRAIN STD ERROR

79-B-17 is unfrozen fvAAP

TABLE B9

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE	INST.	TERRAIN	SHEET	KP	O/S
80-11-075	N	fvAT	14	98.93	3S

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
9.0	0.4	100	0	8	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	0.6	OL	-	-1	-1	-1	ε = 0 (negligible layer thickness)	0.0	0.0
2	1.5	ML	70.3	1370	1	-1	ε =-0.6312 + 0.0163 x vol	0.515	0.464
3	3.0	ML	58.2	1580	1	-1	ε =-0.6312 + 0.0163 x vol	0.317	0.476
4	3.5	ML	77.7	1625	1	-1	ε =-0.6312 + 0.0163 x vol	0.635	0.318
5	3.9	VA	44.3	-1	1	-1	ε =-0.0385 + 0.00251 x vol	0.072	0.029
6	5.4	SP	52.5	1680	-1	-1	ε =-0.6312 + 0.0163 x vol	0.226	0.339
7	7.5	CI	45.9	-1	3	-1	ε =-0.5852 + 0.0158 x vol	0.140	0.294
8	9.0	BR	-	-1	-1	-1	BR, ε = 0.0	0.0	0.0

Σ = 1.920

COMPUTER CALCULATION

Borehole = 80-11-075	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
negligible layer thickness	0.00 - .60m	OL	-1	0.0	0.000	0.000
water content	.60 - 1.50m	ML	92	70.3	.515	.149
water content	1.50 - 3.00m	ML	53	58.2	.318	.149
water content	3.00 - 3.50m	ML	139	77.7	.636	.149
water content	3.50 - 3.90m	VA	54	44.3	.073	.073
water content	3.90 - 5.40m	SP	42	52.6	.226	.149
water content	5.40 - 7.50m	CI	32	45.9	.142	.096
bedrock	7.50 - 10.00m	BR	-1	0.0	0.000	0.000

80-11-075 Thaw settlement = 1.924 Sigma = .407

TABLE B10

ALASKA HIGHWAY GAS PIPELINE PROJECT

THAW SETTLEMENT CALCULATION VERIFICATION

BOREHOLE (1)	INST.	TERRAIN	SHEET	KP	O/S
81-05-14	N	tMM	150	950.00	0

DEPTH	THAW	P'FROST	COBBLES	LAYERS	
10.0	0.0	100	0	8	

HAND CALCULATION

LAYER	DEPTH	USC	VOL. MC	BD	ICE	$\epsilon_{(50)}$	STRAIN CORRELATION	STRAIN	LAYER SETTLEMENT
1	1.0	GM	-1	2000	-1	-1	ε= 1.668 - 0.00073 x BD	0.210	0.210
2	2.0	GM	-1	-1	-1	-1	ε= 0.0 (negligible layer thickness)	0.0	0.0
3	3.0	GM	21.0	-1	-1	-1	ε=-0.160 + .0089 x vol	0.028	0.028
4	4.0	GM	34.3	-1	-1	-1	ε=-0.238 + 0.0124 x vol	0.187	0.187
5	5.0	PT	82.4	-1	-1	-1	ε=-2.346 + 0.0335 x vol	0.411	0.411
6	6.0	GM	-	-1	-1	-1	ε=-0.238 + 0.0124 x vol (2)	0.187	0.187
7	8.0	GM	34.3	-1	-1	-1	ε=-0.238 + 0.0124 x vol	0.187	0.374
8	10.0	GM	-	-1	-1	-1	ε=-0.238 + 0.0124 x vol (3)	0.187	0.374

(1) Dummy Borehole
(2) MC interpolation (from layer below)
(3) MC interpolation (from layer above)

Σ = 1.771

COMPUTER CALCULATION

Borehole = 81-05-14	DEPTH RANGE	USC	GRAV	VOL	STRAIN	STD ERROR
bulk density	0.00 - 1.00m	GM	-1	0.0	.211	.136
negligible layer thickness	1.00 - 2.00m	GM	-1	0.0	0.000	0.000
water content	2.00 - 3.00m	GM	10	21.0	.029	.017
water content	3.00 - 4.00m	GM	20	34.3	.187	.086
water content	4.00 - 5.00m	PT	200	82.4	.410	.199
interpolated water content	5.00 - 6.00m	GM	-1	34.3	.187	.086
water content	6.00 - 8.00m	GM	20	34.3	.187	.086
interpolated water content	8.00 - 10.00m	GM	-1	34.3	.187	.086

81-05-14 Thaw settlement = 1.769 Sigma = .363